ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Study of Index Properties and shear Strength Parameters of Laterite soils in Southern Part of Ethiopia the case of Wolayita - Sodo

By

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Study of Index Properties and shear Strength Parameters of Laterite soils in Southern Part of Ethiopia the case of Wolayita - Sodo

A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the requirements for the Degree of Masters of Science in Civil Engineering

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I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr. Mesele Haile and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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Symbols and abbreviations

Designation		Units
UU	Unconsolidated Undrained	---
UCT	Unconfined Compression test	---
C_U	Cohesion for Undrained Shear Strength	KN/m^2
q_U	Unconfined Compressive Strength	KN/m^2
Φ	Angle of internal friction	---
S	Degree of saturation	---
e	Void ratio	---
γ_d	Dry unit weight	KN/m^3
γ_w	Wet unit weight	KN/m^3
LL	Liquid limit	%
PL	Plastic limit	%
PI	Plasticity Index	%
LS	Linear shrinkage	%
FS	Free Swell	%
S_g	Specific gravity	---
W	Moisture content	%
N	No. of blows for Liquid limit	---
NMC	Natural moisture content	%
AD	Air drying	---
OD	Oven drying at a temperature of 105^oc	---
AR	As received /at the natural moisture content	---
USCS	Unified soil classification system	---
AASHO	American Association of State Highway Officials.	---
AASHTO	American Association of State Highway and Transportation Officials.	---
ASTM	American Society for testing and Materials.	---

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ABSTRACT

Most geotechnical problems such as bearing capacity, lateral earth pressure 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 of the soil. The stress state variables generally used for an unsaturated soil are the net normal stress ($\sigma - u_a$) and the matric suction. (U_a- U_w).The index properties of soils are also essential parameters for soil classification as well as for indirect estimation of its potential strength.

Because of climatic conditions, ground water tables in tropical and subtropical regions are often depressed. Evapotranspiration often exceeds infiltration. This leads to deep desiccation of the soil profile. Therefore, residual soils frequently exist in an unsaturated state with continuous air in their voids. The pore air pressure will usually equivalent to atmospheric pressure, but the pore water pressure 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, or in other words: the effective stress becomes greater than the total stress.

The extended Mohr Circle failure envelope defines the shear strength of unsaturated soil.
Identifying the soil characteristic is essential to determine the type of test and test procedure that is applied during sampling, sample preparation and testing.
In this thesis, several tests have been conducted to understand the effect of unsaturated soil mechanics on the determination of shear strength parameters and comparison of the result with tests on saturated samples. To this effect the geotechnical and geo-chemical characteristics of soils sampled from Wolayita – Sodo have been investigated.

For shear strength parameters determination, undisturbed soil samples were collected and unconsolidated undrained tests and unconfined compression tests were carried out because of the triaxial machine's limitation to suction measurements. Both saturated and unsaturated tests were conducted to make systematic comparison of the test results.

Moisture content determination, using oven temperatures of l05°C, 50°C and 35°C were carried out on the soil samples to investigate the variation of structural water.

Atterberg Limits were investigated for different testing procedures on the soil samples to evaluate the effect of test manipulation on cement bonds between clay clusters. The liquid limit tests were carried out on soil specimens mixed for 5 minutes and 30 minutes durations. From the test results, the soil under investigation has been affected by manipulation.

Specific Gravity Test, Particle Size Distribution, Atterberg Limit Tests and Free Swell Tests were conducted at three different test temperatures (as received moisture, air-dried and oven dried at a temperature of 105°c). These tests were conducted in order to understand the behavior of the soil in the area and classification.

According to the result of this research, Moisture content ranges between 26 - 41%, Plasticity Index ranges between 19 – 30%, Clay fraction ranges between 48 - 70, Shrinkage limit ranges11-22, Free swell ranges between 28-38% and Specific Gravity ranges between 2.61-2.97. The shear strength parameters(C - Φ value) as determined from UU Test ranges between 150-173 Kpa and 12 ° -20 ° for unsaturated sample respectively. For saturated soil sample, the C- value obtained is 118Kpa and unconfined compression strength (qu) ranges from 215-385 Kpa.

Laterite soils are characterized by high concentration of Iron Oxide, Aluminium Oxide (Sesqueoxide) and Kaolinite minerals. The soils samples subjected to tests fall below A-line under MH (inorganic clay with medium strength) and contain Kaolinite mineral.

The activity number which is the ratio of plasticity index to the percent of clay –size friction by weight, is below 0.75, confirming that the soil is inactive.

1. Introduction

1.1 General

Wolayita Soddo which is located in southern part of Ethiopia is covered predominantly with red brown soil. Such soils start at Bodity town, 370 Km far from Addis Ababa, covering large area.

The safety of any geotechnical works is closely related to the shear strength of the soil. The shear strength is the internal frictional resistance and Cohesion of soils to shearing forces. Shear strength is required to make estimates of the load bearing capacity of soils, the stability of geotechnical structures and in analyzing the stress strain characteristics of soils.

The shear strength of a soil can be related to the stress state in the soil. The stress state variables generally used for unsaturated soil are the net normal stress (σ –Ua) and the matric suction (Ua- Uw) (Blight, 1997).

Because of climatic conditions, ground water tables in tropical and subtropical regions are often depressed. Evapotranspiration often potentially exceeds infiltration. This leads to deep desiccation of the soil profile .Therefore, residual soils frequently exist in an unsaturated state with continuous air in their voids. The pore air pressure will usually is equivalent to the atmospheric pressure, but the pore water pressure 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, or in other words: the effective stress becomes greater than the total stress.

The shear equation of partially saturated soil can be written as

$$\zeta = c' + [(\sigma_n - u_a) + \chi (u_a - u_w)] \tan Ø' \qquad [1.1]$$

Where ζ=shear strength, c'= effective cohesion intercept, u_a-u_w = suction, u_a= pore air pressure, u_w =pore water pressure, σ_n =total normal stress, Ø'=effective angle of shear resistance and χ is dimensionless modifier to the suction.

The extended Mohr circle failure envelope defines the shear strength of unsaturated soil. The strength and permeability are likely to be greater than the temperate zone soils with comparable liquid limit (Blight, 1997).

1.2 Back ground of the problem

There is no previously done research around this area, thus the work gives us better understaning about the behavior of the soil. Identifying the soil characteristic is essential to determine the type of test and test procedure that is applied during sampling, sample preparation and testing.

Conventional soil classification systems focus primarily on the properties of soil in its remolded state. This is often misleading for residual soils, whose properties are likely to be strongly influenced by in situ structural characteristics derived from the original rock mass or developed as a consequence of weathering. Thus, Conventional soil classification systems don't reflect their true properties.

Considering the effect of negative pore pressure is essential for any geotechnical works such as:

- Construction and operation of Dam
- Natural slopes subjected to environmental changes
- Stability of vertical or near vertical excavations
- Determination of lateral earth pressures
- Determination of bearing capacity for shallow foundation

Residual soils are also affected by pre treatment condition in index properties determination such as moisture content.

1.3 Objectives of the Study

The main objectives of this research work are as the following:

i. Check whether the soil of Wolayita- Sodo area is lateritic soils or not by conducting index and chemical tests.
ii. Investigate the effect of temperature variations, pre-treatment conditions and testing procedures, on the behavior of the soil.
iii. Identity shear strength characteristic of the soil of Wolayita Sodo.

1.4 Methodology

Specimens were taken at three different locations from seven test pits. From one test pit two or three samples are collected. The laboratory investigations were carried out in accordance with the procedure given in ASTM, effect of temperature variation on moisture content determination and different pre treatment methods have been carried out.

Specimens were taken from the following sites

Around under construction Wolayita- Sodo university TP1 and TP2

Near WADU agricultural office TP3 and TP4

Gola area TP5, TP6 &TP7

1.5 Scope of the Study

The main purpose of study is to identify the shear strength characteristics of the soil.. Both saturated and unsaturated test were conducted to make systematic comparison of the differences in the test results.

Effect of temperature variations on moisture content determination have been checked in the laboratory using different drying temperatures. The difference of the results obtained for different moisture conditions helped to choose the test temperature for the rest of the tests to be carried out under this research work.

Different pre-treatment methods have been applied to a number of samples tested in the laboratory leading to different moisture contents.. Samples tested for the four moisture pre-treatment methods were prepared in the following manner (Lyon, 1971).

As Received (AR) - at natural moisture content.

Soaked (S) - immersed in water for 24 hours.

Air dried (AD) - dried to constant weight under normal temperature.

Oven dried (OD) - dried in an oven for 24 hours at 105ºc.

1.6 Structure of the Thesis

This thesis work is divided in to six Chapters, each covering a specific topic of the research work. In this introductory chapter the background of the problem, objective and scope, methodology of the thesis work and structure of the thesis are presented. Chapter two deals with a brief literature review which discusses about residual soil formation, classification, sensitivity to pre-treatment, testing procedure, and testing procedure for determining shear strength parameters. Chapter three deals with sampling areas description. The fourth Chapter deals with insitu properties with sample description and the types of laboratory tests conducted and results obtained. The test results obtained from this work by comparing with previously done test for red clay and laterities soils is indicated in chapter five. Chapter six includes the conclusions and recommendations drawn from the research. Finally,Grain size distribution curves under different testing conditions, ,Geo-chemical and X-ray diffraction test result are included in appendix .

2. Literature Review

2.1 General characteristic of Residual Soils

2.1.1 Origin and Formation of Residual Soils

Residual soils are derived from the in situ weathering and decomposition of rock which has not been transported from its original location. Particles of residual soil often consist of aggregates or crystals of weathered mineral matters that breakdown and become progressively finer if the soil is manipulated (Blight, 1997).

Residual soils are affected by

ii) Weathering process

iii) Climate

iiii) Topography

Weathering process

Residual soils are formed by the in situ weathering of rocks, through Physical, Chemical and Biological processes.

Most commonly, residual soils are formed from igneous or metamorphic parent rocks, but residual soils formed from sedimentary rocks are not uncommon. Chemical processes tend to predominate in the weathering of igneous rocks, whereas physical weathering are so closely interrelated that one process never proceeds without some contribution by the other (Blight, 1997).

Physical weathering includes the effect of such mechanical process as abrasion, expansion, and contraction. Physical weathering produces end products consisting of angular blocks, cobbles, gravel, sand, silt and even clay sized rock flour. The mineral constituents of all these products are exactly like those of the original rock. Chemical weathering, on the other hand, results in the decomposition of rock and the formation of new minerals.

The chemical changes operating in primary minerals of the rocks in temperate or semi-tropical zones tend to produce end products consisting of clay minerals predominately represented by kaolinite and occasionally by Halloysite and by hydrated or dehydrous Oxides of Iron and Aluminum.

Chemical weathering is favored by warm humid climates, by the process of vegetation and by gentle slope. Thus, tropical and subtropical regions of low relief with abundant rainfall and high temperature are the most susceptible to chemical alterations. Deep, strongly leached red, brown and yellow profiles are manifestations of the effects of sever chemical weathering.
Under conditions favorable to tropical weathering, the weathering processes may be so intense and may continue so long that even the clay minerals, which are primarily hydrous aluminum silicates, are destroyed. In the continued weathering the silica is leached and what remains consists merely of Aluminum Oxide such as Gibbsite, or of Hydrous Oxide such as Limonite or Goethite derived from the Iron. This process is known as laterization.

Climate

Climate exerts a considerable influence on the rate of weathering.Physical Weathering is more predominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and by temperature. The clay minerals of the soils of the world changed in predictable way with distance from the equator.

Climate has a further effect on the properties of tropical residual soils. In sub humid tropical and subtropical areas water tables are often deeper than 5 to 10 m and the effects of unsaturation, desiccation and seasonal or longer term rewetting have to be taken into account in geotechnical design. There are many accounts of the effect of unsaturation on the behavior of soils. The effective stress relation-ship for unsaturated soils is governed by the difference ($\sigma - u_a$) and the suction (u_{a-} u_w). Inmost practical situations u_a equals to the atmospheric pressure and can be zero. The conventional form of the effective stress equation can be used with little error for soils that are unsaturated (Fredlundand, 1993)

Topography

Topography controls the rate of weathering by partly determining the amount of available water for each zone of weathering. Precipitation will tend to run off hills and accumulate soils in valleys and hollows.

Soil profiles developed from basic Igenous Rocks on hillsides the depth of weathering increase down the slope where as Kalioite / Hallosite are the predominant clay minerals at the top of the slope and Smectite at the bottom of the slope (Blight, 1997).

2.1.2 Pedological and Lithological Classification of Residual Soils

Special classification system is required for residual soils because of the following

i) Unusual clay mineralogy of some tropical and sub tropical soils.

ii) The soil mass in situ may display a sequence of material ranging from true soil to soft rock depending on degree of weathering.

Conventional soil classification systems focus primarily on the properties of the soil in its remolded state. This is often misleading for residual soils.

A practical system for classifying all residual soils based on mineralogical composition, micro and macro structures of the soil.

The specific characteristic of residual soils which distinguish them from transported soils can generally be attributed either to the presence of specific clay or structural effects, such as the presence of unweathered or partially weathered rock, relict discontinuity and other planes of weathering and inter- particle bonds.

The first step in the grouping of residual soils is to divide them into groups on the basis of mineralogical composition alone, without referring to their undisturbed state. The following three groups are often suggested: (Blight, 1997).

1. Group A: Soils without a strong mineralogical influence
2. Group B: Soils with strong mineralogical influence derived from clay minerals also commonly found in transported soils.
3. Group C: Soils with a mineralogical influence deriving from clay minerals only found in residual soils.

Properties of the groups are discussed in greater detail in table 2.1:

Some additional description is given for sub –groups of group C

Table 2.1 Characteristic of residual soils groups (Blight, 1997).

Group		Examples	Means of identification	Comment on likely engineering properties and behavior
Major group	Sub- group			
Group A (Soils with out a strong mineralogical influence)	(a) Strong macro structure influence	Highly weathered rock from acidic or intermediate igneous rocks and sedimentary rocks	Visual inspection	This is a very large group of soils (including the Saprolites) where behavior (especially in slope) is denoted by the influence of discontinuities ,fissures etc.
	(b) Strong micro structure influence	Completely weathered rocks formed from igneous and sedimentary	Visual inspection, and evaluation of sensitivity, liquidity index, etc	Theses soils are essentially homogenous and form a tidy group much more amenable to systematic evaluation and analysis than group (a) above , identification of nature and role of bonding (from relict primary bonds to weak secondary bonds) important to understand behavior.
	(c) Little structural influence	Soils formed from very homogenous rocks	Little or no sensitivity, uniform appearance	This is relatively minor sub- group. Likely to behave similarly to moderate over consolidated soils.
Group B (Soils strongly influenced by commonly occurring minerals)	(a)Semecticte (Montmorllonite group)	Black cotton soils, many soils formed in tropical areas in poorly drained conditions.	Dark colour (grey to black and highly plasticity.	These are normally problem soils found in flat or low lying areas of low strength, high swelling compressibility, and high swelling and shrinkage characteristics.

	(b) Other minerals			This is likely to be a very minor sub group.
Group C (Soils strongly influenced by clay minerals essentially found only in residual soils)	(a) Allophane Group	Soils weathered from volcanic ash in the wet tropics and in temperate climates.	Very high natural water contents and irreversible changes on drying	These are characterized by very high natural water contents, and high liquid and plastic limits. Engineering properties are generally good through in some cases high sensitivity could make handling and compaction difficult.
	(b) Halloysite group	Soils largely derived from older volcanic rocks forming especially tropical red clay	Reddish colour ,well drained topography and volcanic parent rock are useful indicators	These are generally fine or course Soils, of low to medium plasticity, but low activity. Engineering properties generally good. (Note that there is often some overlap between Allophone and Halloysitic soils).
	(c) Sesquioxide Group	This soils group loosely referred to as Lateritic,or Laterite	Granular ,or nodular appearance	This is a very wide group, ranging from silty clay to coarse sand and gravel. Behavior may range from low plasticity to non plastic gravel.

Group C, sub-group (a)

As already stated, the Allophane- rich group is probably the most distinctive group of residual soils. Their predominant characteristics include:

- Very high natural water content and high values of L.L and P.L.
- Irreversible decrease in plasticity and increase in particle size when air or oven dried.
- They are likely to have flat compaction curves without distinctive value of optimum water content or maximum dry density
- Empirical relationships applicable to transported soils are likely to be misleading when applied to these soils.

Despite these unusual properties, their engineering behavior is good, especially when the soil is in its undisturbed state.

The soils tend to have high shear strength with moderate low compressibility and are remarkable stable on steep natural or cut slopes. However, difficulties are likely to be encountered with earthworks in these soils, first because of the wet climates in which they often occur, and secondly because conventional compaction specifications and control methods are not suitable. The soils may in some cases have high sensitivity, making handling and compaction difficult.

Group C, sub-group (b)

The clay mineral Halloysite is characterized by very small particle size and low activity. The most significant property of Halloysite soils is that despite small particle size and relatively high plasticity their engineering properties are generally good. Red clay soils of volcanic parent material are commonly composed predominantly of Halloysitic particles and form a predominant member of this group.

Group C, sub-group (c)

The soils in this group range from low plasticity silty clays through to the concretionary deposits made up predominantly of gravel-sized material.

Laterites are usually highly weathered and altered residual soils, low in Silica that contains a sufficient concentration of Sesquioxides of Iron soils, and Aluminum to have been cemented to some degree. The extent of which a residual soil has been laterized may be measured by the ratio of silica, SiO_2, remaining in the soil (except for discrete pebbles of free quartz that may remain) to the amount of Fe_2O_3 and Al_2O_3 that has accumulated. The Silica: Sequioxide ratio give as

$$\frac{SiO_2}{R_2O_3} = \frac{SiO_2}{Fe_2O_3 + Al_2O_3}$$

has served as basis for classification of residual soils. Ratios less than 1.33 have sometimes been considered indicative of true laterits, those between 1.33 and 2.00 of lateritic soils and those greater than 2.00 of nonlateritic tropically weathered soils.
Laterites are often excellent material for the building of roads and embankments. The high strength and low compressibility is suitable for placing of shallow foundations (Blight, 1997).

Pedological classification system is given by D' Hoore. The soils are broadly differentiated on a genetic basis, determined by soil forming factors. It is a means of identification of lateritic soils. Three main units are used for the description and classification of red tropical (Lyon, 1971).
Ferruginous soils show a marked separation of free iron oxide, either leached out of the profile or precipitated within the profile as concretions. There may be a high proportion of weatherable primary minerals remaining. Kaolinite is the dominant clay mineral. These soils are generally found in areas with under 1850mm rainfall a year and pronounced dry seasons.

Ferrallitic soils are generally deep, with only slightly differentiated horizons. Kaolinite is the dominant clay mineral; and they contain free iron oxides and hydrated oxides of aluminum. They generally occur in more humid areas with more than 1500mm rainfall per year.

Ferrisol soils have profiles similar to Ferralitic soils, but with very few weatherable minerals remaining. The entire clay size fraction comprises kaolinite and amorphous oxides of iron and aluminum. Ferrisol tend to develop at deeper levels, because of surface erosion, and occur in regions of between 1250 and 2750 mm rainfall per year. According to Morine W.J. and Todor P.C., Ethiopian laterites fall under this group (Lyon, 1971).

Classification of laterites is also possible depending on the degree of concretionary development. The development of concretions requires sufficient concentration of the hydrated oxides of iron and aluminum for cementation or precipitation growth to start (CIRIA, 1995).

2.1.3 Typical profile of Unsaturated Soils

The microclimatic conditions in an area are the main factor causing a soil deposit to be unsaturated. Therefore, unsaturated soils or soils with negative pore –water pressures can occur in essentially any geological deposit. An unsaturated soil could be a residual soil, a lacustrine deposit, a bedrock formation, etc...

Tropical residual soils have some unique characteristic related to their composition and environment under which they developed. Most distinctive is the microstructure which changes in gradational manner with depth. The in situ water content of residual soils 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 .The strength and permeability are likely to be greater than that of temperate zone soils with comparable liquid limits.

Degree of saturation

The degree of saturation S can be used to subdivided soils in to three groups

i) Dry soils i.e. $S = 0\%$.

ii) Saturated soils (i.e. $S=100\%$.

iii) Unsaturated soils (i.e. $0\% < S < 100\%$)

This subdivision is primarily a function of the degree of saturation. An unsaturated soil with a continuous air phase generally has a degree of saturated less than approximately 80% (i.e. S<80%). Occluded air bubbles commonly occur in unsaturated soils having a degree of saturation greater than approximately 90 %(i.e. S>90%).The transition zone between continuous air phase and occluded air bubbles occurs when the degree of saturation is between approximately 80-90% (i.e. 80% < S <90%) (Fredlundand, 1993)

2.2 The engineering characteristics of lateritic soils

2.3.1 General

In many countries of Africa and Asia, lateritic soils are the traditional materials for road and airfield construction. Though a good deal of literature is available on lateritic soils and several excellent reviews have been prepared on lateritic soils (Lyon, 1971),

The available data on lateritic soils gives the impression that the red color seems to have been accepted by most authors as the most important property by which these soils could be identified. Other obviously significant basic physical properties such as texture, structure, consistency, etc., often were ignored .It is also noted that the lack of uniformity in pretreatment and testing procedures (resulting from association with different standards in different parts of Africa) makes it difficult to compare even textural data on the same soils. It is noted that three major factors influence the engineering properties and field performance of lateritic soils. These are;

- Soil forming factors (e.g. parent rock, climate vegetation conditions, and topography and drainage conditions).
- Degree of weathering (degree of laterization) and texture of the soils, genetic soil type, the predominant clay mineral type and depth of sample.

Pretest treatments and laboratory test procedures as well as interpretation of test results (Lyon, 1971).

Climate and topography influences the rate of weathering. Physical weathering is more pronounced in dry climates, while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature. Topography on the other hand, controls the rate of weathering by partly determining the amount of available water and the rate at which it moves down through the zone of weathering. It also controls the effective edge of the profile by controlling the rate of erosion of a weathered material from the surface. Hence deeper profiles will generally be found in valleys and on gentle slopes rather than high ground or steep slopes (Blight, 1997).

Chemical, mineralogical and physico- chemical characteristics

A distinctive feature of laterite and lateritic soils is the higher proportion of Sesquioxide of Iron and/or Aluminum relative to the other chemical components. The amount of Alumina or Iron Oxides is an important factor in differentiating Aluminous and Ferruginous varieties. The base (alkalis and alkaline earths) is almost absent in lateritic horizons, except in some Ferruginous crusts developed in alluvium and some concretionary horizons in Ferruginous tropical soils. Other lateritic constituents are Manganese, Titanium, Chromium and Vanadium Oxides.

Chemical analyses do not usually reveal the origin, nature or even the composition of laterites or lateritic soils. The mineralogical composition is considered to be more important in explaining the physical properties of laterite and lateritic soil. The mineralogical constituents can be divided in to major elements, which are essential to laterization, and minor elements, which do not affect the laterization process. The major constituents are Oxides and Hydroxides of Aluminum and Iron, with clay minerals and, to a lesser extent, Manganese, Titanium and Silica. The minor constituents are residual remnants or elastic material.

The clay mineral most common in lateritic soils is Kaolinite. Halloysite is also reported. Illite and Montimorillonite are rare. The secondary minerals resulting mainly from the laterization process are Gibbsite, Goethite, Limonite and Hematite. Neither Manganese nor Titanium minerals were observed in significant amounts.

There is very little information on the pysico-chemical characteristics of lateritic soils. A few data published on this subject have been confined. The conclusions reached show that the lateritic medium is either neutral or acidic and organic content is generally low, below 2%. The Calcium Carbonate content is either very small or lacking in most lateritic soils profiles.

The distribution of such properties as the PH, base exchange capacities, hygroscopic moisture content, calcium carbonate content and chemical composition of soil can form a basis for the understanding of behaviour of lateritic soils as a material of construction (Lyon, 1971)..

2.3.2 Index Tests of Residual Soils

2.2.2.1 Moisture content

The conventional test for the determination of moisture content is based on the loss of water when a soil is dried to a constant mass at a temperature between 105 and 110 ºC. In many residual soils however, some moisture exits as water of crystallization, within the structure of minerals presented in the soils particle. Some of this structural moisture may be removed by drying at the above temperature assuring the behavior of the soil. The following procedure is therefore recommended:

Two test specimens should be prepared for moisture content determinations. One specimen should be oven dried at 105 ºC until successive weighing show that no further loss of mass. The moisture content should then be calculated in normal way. The second sample should be air dried (if feasible); or oven dried at a temperature of no more than 50ºC and a maximum relative humidity (RH) OF 30% until successive weighing show that no further loss of mass. The two moisture content results should then be compared; a significant difference (4-6% of moisture content obtained by oven drying at 105 ºC) indicates that structural water is present. This water forms part of soil solids, and should therefore be excluded from the calculation of moisture content. If a difference is detected using the two different drying process, all subsequent tests for moisture content determination (including those associated with Atterberg Limit tests, etc) should be carried out by drying at lower temperature (i.e. either air-drying, or oven–drying at 50^{o}C and 30% RH) if possible, the lower drying temperature of 50^{o}C should be used (Blight, 1997).

2.2.2.2 Atterberg limits

Because the formation of lateritic soils involves differential weathering as well as movement and deposition of dissolved materials, the variation of plasticity characteristics with depth cannot be predicted even in two similar profiles on different topographical sites.

Effect of pre-test drying

The influence of the pretreatments and testing procedures on the plasticity characteristics have been widely studied and discussed. The variations in test result due to pretreatments and testing procedures have made the interpretation of test results very difficult.
(Lyon, 1971) states that mixing the soils with water during testing procedure causes the breaking up of the fine particles and also deflocculating. Various researchers all found the limits change with drying and with manipulation. (Lyon, 1971) states ‘when liquid limit tests were carried out the aggregations of clay particles were broken down by the manipulation, this led to difficulties in consistent values for liquid limit.

Laterites formed under continuously wet regions are likely to be characterized by high natural water contents; high liquid limits are observed to result in irreversible changes up on drying. Up on drying the plasticity decreases and grain size increases such that much of clay sized particles agglomerates to the size of silt.

On the other hand, lateritic soils formed under seasons of distinct wet and dry seasons are likely to be characterized by low natural moisture content, low plasticity, and presence of concertions and cemented horizons. Laboratory tests run from natural water content or from the air-dried state lead to essentially the same result (Lyon, 1971).

According to (Blight, 1997), the effect of drying prier to testing is attributed to.

- Increased cementation due to oxidation of the iron and aluminum sesqueoxides, or
- Dehydration of Allophane, or both.

Effect of method and time of mixing on Atterberg Limits

In general, the greater the duration of mixing (i.e., the greater the energy applied to the soil prier to testing), the larger the value of the resulting liquid limit, and to a lesser extent, the larger the plasticity index. This has been attributed to longer mixing results in more extensive break down of the cemented bonds between the clay clusters and within peds (disaggregation of the particles), and thus formation of greater proportions of fine particles.

In order to address this problem:

Five test specimens should be mixed with water to give a range of moisture contents suitable for liquid limit and plastic limit determinations. The minimum amount of air-drying should be used, and preferably none at all. Thus should not be too difficult as the in-situ moisture content of majority of soils is at or below the relative plastic limit. The mixing time should be standardized at 5 minutes, and the mixed specimens should be left for moisture content equilibration overnight before testing.

On the following day the liquid limit should be determined with a minimum of further mixing. A sub-sample from each of the specimens used in the test should be used for the determination of moisture content, using the procedure. The remainder of each specimen should then be mixed continuously for a further 25 minutes before again determining the Liquid Limit. A significant difference (of >5% of the liquid limit obtained) between the liquid limit from tests using 5 and 30minutes mixing times indicates a disaggregatinon of the clay-sized particles in the soil. If this disaggregation is confirmed by repeating the above procedures, the entire program of testing should:

- Limit the mixing times to no more than 5 minutes
- Make use of fresh soil for each moisture content point in Atterberg Limit tests.

The soil should be broken-down by soaking in distilled water, and not by drying and grinding. The soil should be immersed in distilled water to form slurry, which is then washed through a

425 µm sieves until the water runs clear. The material passing the sieve is collected and used for Atterberg Limit test (Blight, 1997).

2.2.2.3 Grain –size distribution

Consistent reports of variations in particle-size distribution with methods of pretreatment and testing have been widely reported on laterite soils.
The particle size distribution of residual soils is affected by

i) **Effect of drying** The most widely reported effect of drying is reduce the percentage that is reported as the clay fraction (finer than 2µm).It is accordingly recommended that drying of the soil prior to testing be avoided.
Oven dried lateratic soils were found to give the least amount clay fraction, as compared to air dried or as received (natural moisture content) samples

ii) **Chemical pretreatment** If it is considered necessary to eliminate Carbonates or sesquioxides, then pretreatment with hydrochloric acid is recommended.

iii) **Sedimentation** is essential to achieved complete dispersion of fine particles prior to carrying out a sedimentation test. The sample should be immersed in a solution of dispersant such as dilute alkaline Sodium Hexametaphosphate and therefore washed through the standard nest of sieves (Blight, 1997).

(Lyon, 1971) found that wet sieving increase the silt and clay fraction from 7 to 20 % as compared to dry sieving .It has been found that sodium Hexametaphosphate generally gives better dispersion of the fine fractions.

2.2.2.4 Specific Gravity

The soils to be used in this test should be in its natural moisture content. Pre- test drying of the soil should be avoided as this tends to reduce the measured specific gravity. In residual soils the specific gravity may be unusually high or unusually low depending on mineralogy (Blight, 1997).

The available data indicate that specific gravities vary not only with the textural soils but also within different fractions. Lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (Lyon, 1971).

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2.3 Shear Strength Theory in unsaturated Soils

2.3.1 General

Most Geotechnical problems such as bearing capacity, lateral earth pressure 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. The stress state variables generally used for an unsaturated soil are the net normal stress ($\sigma - u_a$) and the matric suction. ($u_a - u_w$) (Fredlundand , 1993).

Because of climatic conditions, ground water tables in tropical and subtropical regions are often depressed. Evapotranspiration often potentially exceeds infiltration. This leads to deep desiccation of the soil profile .Therefore, residual soils frequently exist in an unsaturated state with continuous air in their voids. The pore air pressure will usually be approximate equal to the atmospheric pressure, but the pore water pressure will be sub – atmospheric, i.e. negative due to capillary effects in the small pores of soil. This negative pore water pressure or suction produces an additional component of effective stress, or in other words: the effective stress becomes greater than the total stress.

The extended Mohr Circle failure envelope defines the shear strength of unsaturated soil.

The two commonly performed shear strength tests are the Triaxial Shear Test and the Direct Shear Test. In direct shear test stress conditions during test are indeterminate and a stress path cannot be established. Therefore Triaxial Test is better than Direct Shear Test.

Triaxial Test for unsaturated soils are

i) Consolidated drained test
ii) Constant water content
iii) Consolidated undrained
iv) Unconsloidated undrained

Because of limitations in Triaxial machine available only the Unconsloidated undrained Triaxial Shear test and unconfined Compression Test will be run.

2.3.2 Unconsolidated Undrained (UU) test

The pore – air and pore – water are not allowed to drain in the undrained test. This applies both when the confining pressure and the deviator stress are applied to the soil specimen. The excess pore - air and pore – water pressure developed during the application of the confining pressure can be related to the isotropic confining pressure by use of B pore pressure parameters. Although the excess pore pressures built up during the application of confining pressure are not allowed to dissipate, the volume of soils specimen may change due the compression of pore – air .The soil has a net confining pressure ,((σ_3-u_a),and a matric suction.($u_a - u_w$),after the application of the confining pressure.

The soil specimen is sheared by applying an axial stress; $(\sigma_1-\sigma_3)$.Until failure is reached. Undrained loading during shear causes further development of excess pore –air and pore – water pressures. The excess pore pressure parameters for triaxial loading condition .Generally the pore pressures are not measured during shear. Therefore, the unconsolidated undrained test results are commonly used in conjunction with a total stress formulation of a problem. Here, the shear strength is related to the total stress without a knowledge of pore pressure at failure.

Test procedure for Unconsolidated Undrained (UU)

The procedure for performing an Unconsolidated Undrained test on unsaturated soil specimen is similar to the procedure used for performing a UU test on saturated soil specimen. The unsaturated soil specimen is tested at its initial water content or matric suction. There is no consolidation process allowed since the confining pressure σ_3, is applied under undrained conditions with respect to both the pore – air and pore – water phases. The specimen is axially compressed under undrained condition with respect to both the air and water phases. The test is usually run at a strain rate of 0.017-0.03% / s,and no attempt is made to measure the pore air and pore water pressures. Conventional triaxial equipment can be used. The porous disks are usually replaced by metal or plastic disks on the top and bottom of the specimen. The specimen is enclosed in rubber membrane during the test (Fredlundand, 1993).

2.3.3 Unconfined compression test (UCT)

The Unconfined Compression Test is special case of the undrained test. No confining pressure is applied to the soil specimen throughout the test. The test can be performed by applying a load in a simple loading frame. At the start of the test, the unsaturated soil specimen has negative pore – water pressure, and pore air pressure is assume to be atmospheric. The soil matric suction (u_a-u_w), is therefore numerically equal to pore –water pressure.

The soil specimen is sheared by applying an axial load and failure is reached. The Deviator stress, (σ_1-σ_3), is equal to the major principal stress and, σ_3, is equal to zero. The compressive load is applied quickly in order to maintain conditions. This should apply to both in pore –air and pore – water phases. The pore - air and pore – water pressures are not measured during compression. The excess pore pressure developed during Unconfined Compression Test can be theoretically related to the major principal stress through use of the D or B pore pressure parameter (Fredlundand, 1993).

Test procedure for Unconfined Compression Test (UCT)

The Unconfined Compression Test procedure similar to the UU test procedure, except that no confining pressure is applied to the specimen (i.e. σ_3 is equal to zero).The test is commonly performed in a simple loading frame by applying an axial load to the soil specimen (Fredlundand,1993).

3. Sampling Area Description

3.1 General

Wolayita Soddo is located in the Southern Nations, Nationalities and Peoples Regional State (SNNPRS). Two trunk roads connect the town with Addis Ababa through Shasemen, 390km from Addis Ababa and through Butajera Hossan, 307 kms. The town is junction for four roads connecting the near by towns Arbaminch, Hossana, Jimma and Sawla.

3.2 Topography and Climate

Near the entrance of the town a beautiful green mount Damota is found at 2908m above sea level. The town elevation range of 2100m at the entrance and decrease towards exit to Arbaminch to about 1900m above sea level.

Thre are two prominent rainy seasons the little rains “Belg” generally fall between March and May and the heavy rains “Kiremt” occur between June and September. The mean annual temperature and rain fall obtained from National Metrological Service Agency from year 1990-2006 shown in Table 3.1-3.3

The Mean annual rain fall ranges between 938.2 - 1619.2 mm

The Mean annual maximum temperature ranges between 24.8 - 25.8ºc

The Mean annual Minimum temperature ranges between 13.8 – 14.9ºc

Table 3.1 Annually Maximum Temperature for year 1990-2004

Year	1990	1992	1994	1995	1996	1998	1999	2001	2002	2003	2004
Max annual Tº C	24.8	24.9	25.6	25.8	25.1	24.8	25.4	24.9	25.7	25.5	25.3

Table 3.2 Annually Minimum Temperature for year 1990-2004

Year	1990	1991	1992	1994	1995	1996	1997	1998	1999	2001	2002	2003	2004
Min annual Tº C	14	14.4	14.4	14.4	14.4	14	13.8	14.4	14.6	14.4	14.6	14.9	14.5

Table 3.3 Annually rain fall for year 1990-2006

Year	1990	1991	1993	1995	1996	1998	1999	2000	2001	2002	2003	2004	2005	2006
Annual Rain Fall Tº C	1072.2	1112.7	1443.5	1080.2	1551.3	1343.9	938.2	1364.1	1506	1057	1254.2	1204	1649.2	1370.6

3.3 Soil and Geology

Visually inspected almost similar soil color dominantly red brown soil start from the near by town Bodity 370km far from Addis, dominantly covers Soddo – Houssan , Soddo – Chida and Gamo Goffa road direction .

These are alkaline and peralkaline stratoid silicics; ignimbrites,unwelded tuffs,ash flows, rhyolites, domes and trachytes. It ranges in age between 2 and 9 million years and mostly located on the escarpments. Flow thicknesses vary widely, from 1 to 30 meters on the plateau and up to 250 meters in rift (Stewart, 1998).

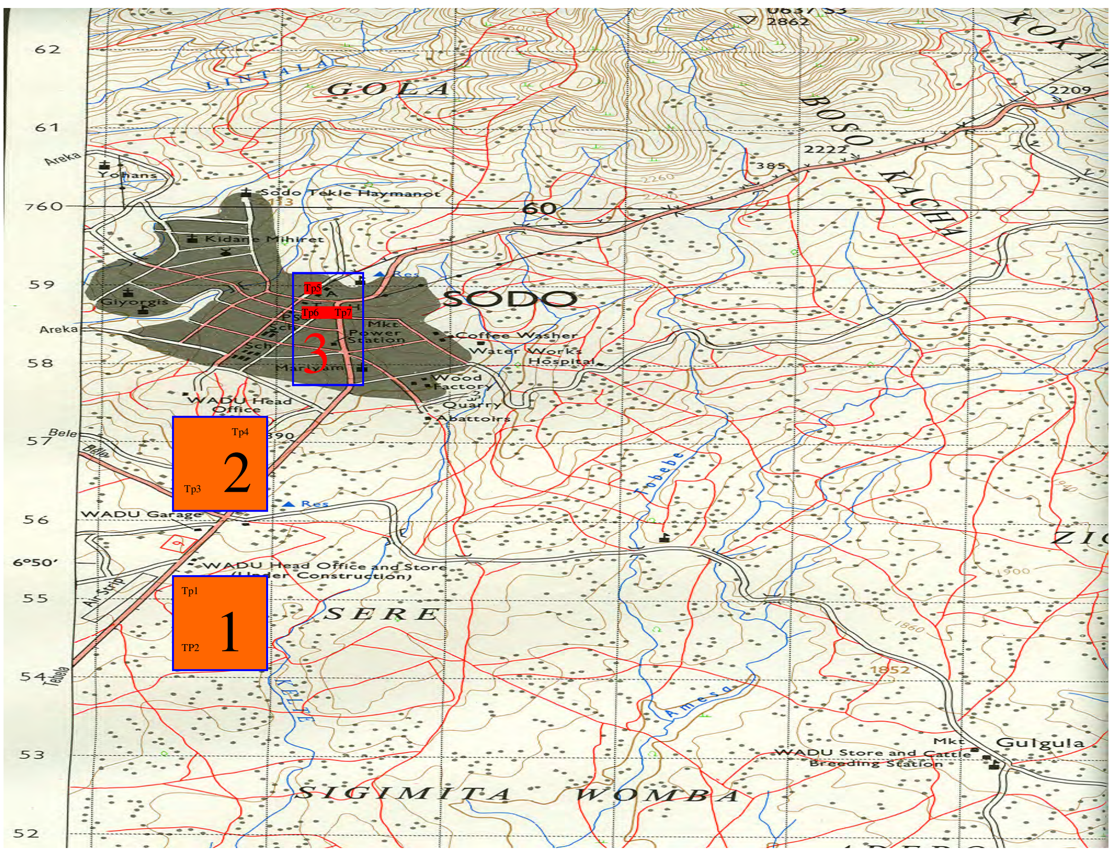

Fig 3.1 Topographic map of Wolayita - Sodo

4. In-situ Properties and Laboratory Test Analysis and Results

4.1 In-situ Properties Description

The soil specimens for this thesis work were collected from Wolayita-Sodo. Prior to sampling, visual site investigations were made to consider the different soil types and to sample evenly in the town. Accordingly seven test pits were chosen from three different areas. TP1 and TP2 are at under Construction University, 20-25m apart; TP3 and TP4 from near agricultural office (WADU) area 20-25m apart; TP5, TP6 and TP7 from Gola area 20-25m apart. The frequency of the undisturbed sample is as follows: TP1 and TP2 two undisturbed sample from 1.50 and 2.00 m depth below ground level, TP4 four samples from 2.00 m depth below ground level and TP3, TP5, TP6 &TP7 three sample from 2.00m depths below ground level. In addition to this disturbed samples were collected for this work, weighing about 150kg. The location of the test pits are shown in Fig 3-1.

Table 4-1 Sample depth and the location used for Wolayita Sodo samples.

Test pit	Sampling Depth (m)	Disturbed sample amount in Kg	Undisturbed sample number	Sample Location	Visual Color observed
TP1	-1.50	10kg	2 samples	around the new under construction university	Red brown
	-2.00	10kg	2 samples		
TP2	-1.50	10kg	2 samples	around the new under construction university	Chocolate brown
	-2	10kg	2 samples		
TP3	-1.50	10kg		Near agricultural office(WADU)	Red brown
	-2	10kg	2 samples		

TP4	-1.50	10kg		Near agricultural office(WADU)	Red brown
	-2	10kg	4 samples		
TP5	-1.50	10kg		Gola area	Red brown
	-2	10kg	2 samples		
TP6	-1.50	10kg		Gola area	Red brown
	-2	10kg	2 samples		
TP7	-1.50	10kg		Gola area	Red brown
	-2	10kg	2 samples		

Distributed samples were covered with plastic bag and undisturbed samples were sealed with wax and covered with plastic bag and moist towel to maintain surrounding moisture.

TP5, TP6 and TP7 are indicated in Fig 3.1 at. 3 elevation topography about 2100m above sea level at right side of the town entrance; TP4 and TP3 are indicated in Fig 3.1 at 2 about 3Km from the above test pit at lower elevation around 1920m above sea level. The last test pit TP1 and TP2 are indicated in Fig 3.1 at 1 elevation topography about 1900m above sea level is distance between location 1 and 2 is 1.5Km.

Fig 4.1Typical Profile of sample area

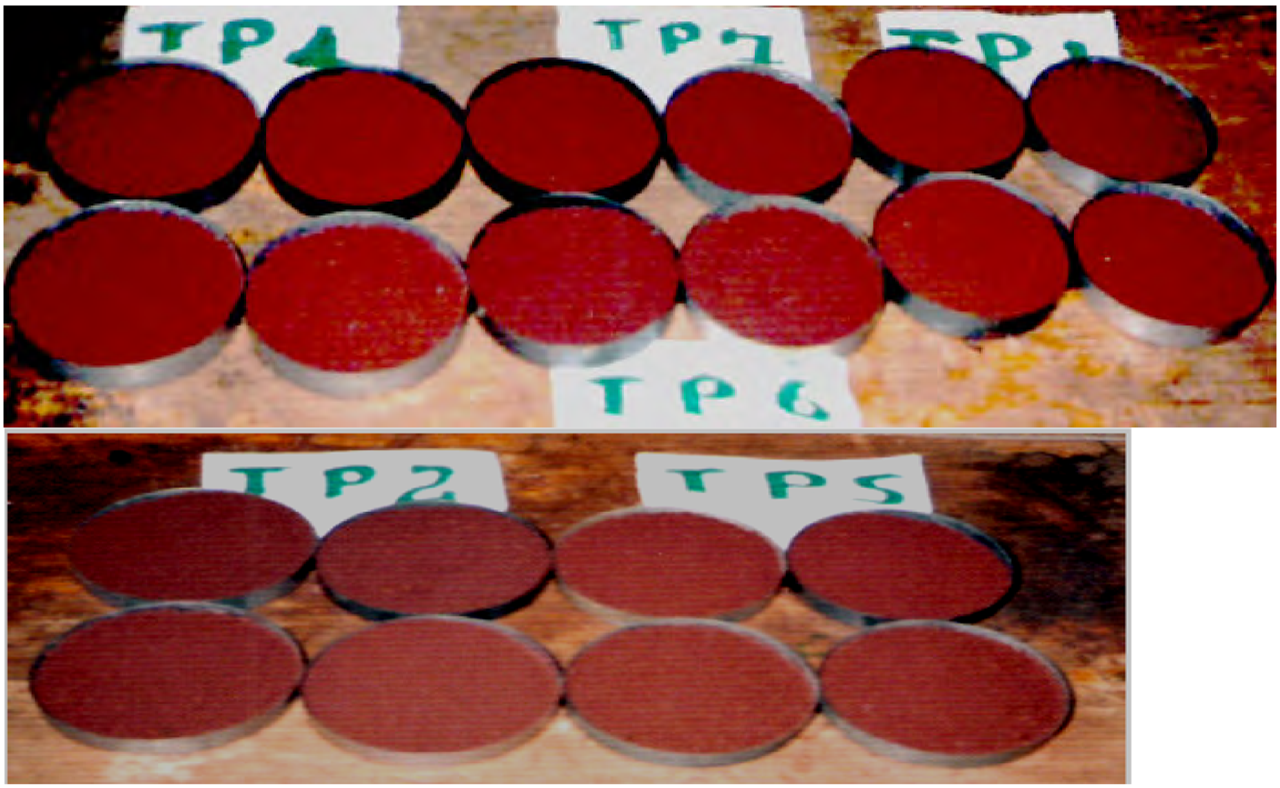

Fig 4.2 The in-situ color observation for the soil samples

4.2 Laboratory Test Results and Discussions

4.2.1 Index properties

4.2.1.1 General

Soil is a heterogeneous material. The properties and characteristics of soils vary from point to point. The tests required for determination of engineering properties are generally elaborated and time consuming. Sometimes the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting elaborate tests. This is possible if index properties are determined. The properties of soils which are not of primary interest to the geotechnical engineer but which are indicative of the engineering properties are called index properties (Arora, 2000).

The behavior of soils should thus be understood by conducting tests on physical attributes of the soil particle and soil aggregate constituents (Haile Mariam, 1992). The physical properties of soils which serve mainly for identification and classification purpose are commonly known as index properties which can be determined by simple laboratory tests. Index property tests are grain size analysis, Atterberg limits, free swell and specific gravity.

4.2.1.2 Effects of Mixing Water

Water may be chemically reacting with the oxides of lateritic soils during testing. In order to see this reaction Atterberg limits and free swell (FS) tests were carried out with distilled and tap water. The results are tabulated in Table 4.2 from the test results one can see that the respective results of Atterberg limits and free swell tests vary insignificantly up on changing of testing water type. It shows that tap water was not chemically reacting with the oxides of lateritic soils during testing. Hence tap water was used for the soil testing for this research works.

Table 4.2 Test results comparison with different mixing water.

Serial No.	Test pit	Depth	condition	Mixing water	LL (%)	PL (%)	PI (%)	FS %
1	TP3 -2.00m	-2.00	wet	Distilled	64	40	24	-
	TP3 -2.00m	-2.00	wet	Tap	64	41	23	-
	TP3 -2.00m	-2.00	Air dry	Tap	59	40	19	30
	TP3 -2.00m	-2.00	Air dry	Distilled	-	-	-	28
2	TP4 -2.00m	-2.00	oven	Distilled	56	34	22	-
	TP4 -2.00m	-2.00	oven	Tap	57	36	21	-
	TP4 -2.00m	-2.00	Air dry	Tap	-	-	-	35
	TP4 -2.00m	-2.00	Air dry	Distilled	63	42	21	35
3	TP7 -2.00m	-2.00	oven	Distilled	52	30	22	-
	TP7 -2.00m	-2.00	oven	Tap	54	31	23	-
	TP7 -2.00m	-2.00	Air dry	Tap	59	36	23	38
	TP7 -2.00m	-2.00	Air dry	Distilled	-	-	-	35

4.2.1.3 Moisture Content

4.2.1.3.1 Effect of Temperature on Moisture Content Determination

The oven temperature 110ºc for water content determination is too high for certain clays and tropical soils. These soils contain loosely bound water of hydration or molecular water which can be lost at this high temperature, resulting in a change of the soil characteristics (Bowles, 1978). This effect was checked using different oven temperatures.

Moisture contents of the soil samples were determined in the laboratory according to ASTM 2216-92. Six samples from each site were taken for moisture content determination. Two set of samples were dried to constant weight using drying oven at temperature of 105ºc, 50 ºc and a maximum relative humidity (RH) of 30% due to limitation of humidity (RH) of 30% of oven all the thesis work is done by 50 ºc without considering the humidity and 35 ºc taking a minimum of ten days to get a constant mass in successive measurements. The values of the moisture content variations are compared and summarized in Table 4.3. As mentioned in section 2.2.2.1 moisture variations of 4 - 6 % or more indicates that loosely bound molecular water is present. From the test results, one can see that the differences in moisture contents for all samples at 105ºc, 50 ºc under consideration are below 4%. At 35 ºc for Tp1, Tp2 and Tp3 the difference is below 4 % .But for the remaining test pits the difference is above 4%, which means that the soils under investigation contain loosely bound water of hydration. Hence, for subsequent tests execution for the thesis work can be done by using drying oven temperature of 105 ºc for Tp1, Tp2 and Tp3, and 35 ºc for the remaining tests.

Table 4.3 Moisture content comparison for different oven temperatures.

Test pit	Depth	Condition	Moisture content	Difference
TP1	-1.50	Oven dry 105º	30.46	
		Oven dry 50º	28.29	2.17
		Oven dry 35º	27.02	3.44
	-2.00	Oven dry 105º	29.23	
		Oven dry 50º	27.20	2.03
		Oven dry 35º	25.88	3.35
TP2	-1.50	Oven dry 105º	31.62	
		Oven dry 50º	29.48	2.14
		Oven dry 35º	27.66	3.95
TP2	-2.00	Oven dry 105º	31.26	
		Oven dry 50º	28.71	2.55
		Oven dry 35º	27.58	3.68
TP3	-1.50	Oven dry 105º	31.31	

TP3	-1.50	Oven dry 50º	28.89	2.42
		Oven dry 35º	27.66	3.65
	-2.00	Oven dry 105º	31.72	
		Oven dry 50º	29.53	2.19
		Oven dry 35º	27.88	3.84
TP4	-1.50	Oven dry 105º	35.57	
		Oven dry 50º	32.66	2.91
		Oven dry 35º	31.43	4.14
	-2.00	Oven dry 105º	38.07	
		Oven dry 50º	34.96	3.11
		Oven dry 35º	33.67	4.40
TP5	-1.50	Oven dry 105º	39.31	
		Oven dry 50º	36.17	3.14
		Oven dry 35º	34.93	4.38
	-2.00	Oven dry 105º	41.12	
		Oven dry 50º	37.71	3.41
		Oven dry 35º	36.67	4.45
TP6	-1.50	Oven dry 105º	35.43	
		Oven dry 50º	32.58	2.85
		Oven dry 35º	30.78	4.65
	-2.00	Oven dry 105º	36.31	
		Oven dry 50º	33.96	2.35
		Oven dry 35º	31.64	4.67
TP7	-1.50	Oven dry 105º	34.63	
		Oven dry 50º	32.32	2.31
		Oven dry 35º	30.04	4.59
	-2.00	Oven dry 105º	34.38	

TP7	-2.00	Oven dry 50º	31.96	2.42
		Oven dry 35º	29.80	4.58

4.2.1.4 Atterberg Limits

Atterberg Limits are arbitrary boundaries between each of the two states such as liquid limit, plastic limit and shrinkage limit. These boundaries are defined by moisture contents. As stated in section 2.2.2.2 lateritic soils are affected by pretreatment and mixing time.

4.2.1.4.1 Test procedures

Atterberg Limits were determined for air-dryed (AD), oven dryed(OD), soaking(S) and as received (AR) or at the natural moisture content. (Air dry and oven dry) as per the procedure of ASTM D4318-00. The air- drying samples were prepared by spreading the specimen in the laboratory for about 10 days. The room temperature was about 20-22ºc. The oven drying samples were prepared by putting the sample in an oven for 24 hours at a temperature of 110 ºc + 5º. The portions of the samples passing the No. 40(0.425mm) sieve were used for the preparation of the sample for this purpose.

As received samples are difficult to be sieved at natural moisture content Hence, wet preparation was used. In this procedure, to reduce disaggregation, the soil should be broken-down by soaking in distilled water. The soil should be immersed in distilled water to form slurry, which is then washed through a 425 µm sieves until the water runs clear. The material passing the sieve is collected and air dried until it is wet with out any free water used for Atterberg Limit test.

4.2.1.4.2 Test results and discussions

In order to investigate the effect of temperature on the Atterberg limits, the samples were tested oven dryed, air- dryed, soaked and as received. The test results are shown in Table 4.4. From the test results one can see that the different treatments affect the Atterberg Limits of these particular soils. The test results show great difference for almost all soils. Hence pretreatment has only slight effect on the values of Atterberg limits for the soil samples under investigation. Hence, when these soils are dried, the fine particles do not come together and reduce the available surface for interaction with water to reduce the plasticity characteristics. (Zelalem, 2005).

The unsoaked soil samples are drying at oven temperature of 105 ºc and conducting the Atterberg Limit with out keeping the sample for moisture equilibration for 24-hours. For the soil under investigation, that is shown on Table 4.5 below the PI values vary slightly.

Table 4.4 Atterberg limit values at different testing conditions.

Test pit	Depth	Condition	Liquid Limit	plastic Limit	plasticity Index	Type of water
TP1	-1.50	Oven dry	52	33	19	tap water
		Oven un soaked	50	31	19	tap water
		Air dry	57	35	22	tap water
		Air dry	58	38	20	distilled water
	-2.00	Oven dry	52	33	19	tap water
		Air dry 35ºc	55	30	25	tap water
		As received washed in tap water	65	40	25	tap water
TP2	-1.50	Air dry	58	32	26	tap water
		Air dry 35ºc	55	31	24	tap water
		Air dry 105ºc	57	34	23	tap water
TP3	-1.50	Oven dry	54	35	19	tap water
		Air dry	61	40	21	tap water
		As received washed in tap water	65	41	24	tap water
	-2.00	Oven dry	48	28	20	tap water
		Air dry	59	40	19	tap water

		As received washed in tap water	64	41	23	tap water
		As received washed in distilled water	64	40	24	distilled water
TP4	-1.50	Air dry	61	35	26	tap water
		Air dry 35ºc	58	38	20	tap water
		Air dry un soaked	61	39	22	tap water
	-2.00	Oven dry	57	36	21	tap water
		Oven dry un soaked	55	34	21	tap water
		Oven dry distilled water	56	34	22	distilled water
		Air dry	63	42	21	distilled water
TP5	-1.50	Oven dry	55	28	27	tap water
		Oven dry Unsoaked	53	26	27	tap water
		Air dry	62	34	28	
TP5	-2.00	Oven dry	61	35	26	tap water
		Air dry	66	39	27	tap water
		Air dry 35ºc	63	37	26	
TP6	-2.00	Oven	55	33	22	tap water
		Air dry	62	34	28	tap water
		Air dry 35ºc	59	32	27	
		As received washed in tap water	61	37	24	tap water
TP7	-1.50	Oven	55	31	24	tap water
		Air dry	63	37	26	tap water
		Air dry 35ºc	66	35	31	tap water
		As received washed in tap water	60	37	23	tap water

		As received washed in tap water 25min	65	37	28	tap water
	-2.00	Oven dry	52	30	22	tap water
		Oven dry	54	31	23	distilled water
		Air dry 5 min	59	36	23	tap water
		Air dry 35 min	74	36	38	tap water
		As received 35ºc	65	37	28	tap water
		As received 105ºc	71	41	30	

Table 4.5 Atterberg limit values at soaked and unsoaked testing conditions.

Test pit	Depth	Condition	Liquid Limit	plastic Limit	plasticity Index	Type of water
TP1	-1.50	Oven dry	52	33	19	tap water
		Oven un soaked	50	31	19	tap water
TP4	-2.00	Oven dry	57	36	21	tap water
		Oven dry un soaked	55	34	21	tap water
TP5	-1.50	Oven dry	55	28	27	tap water
		Oven dry Unsoaked	53	26	27	tap water

4.2.1.4.3 Effect of Test Procedures on Atterberg Limits

Lateritic soils are susceptible to breakdown with manipulation; hence test procedures should be more rigidly controlled. Excessive manipulation during testing leads to crumbling of the soil structure and disaggregating; both produce fines which result in higher liquid limit values. To reduce these effects the mixing time was kept to a minimum, generally about 5 minutes for each limit point (Lyon, 1971).

Five air dried test portions were mixed with water to give the range of water contents suitable for liquid and plastic limit determinations. The mixing time was about 5 minute, and the mixed samples were left for moisture equilibrium for 24 hour before testing. After determining the moisture content for each test point on each test portion, the remaining was then mixed for a further 25 minutes before again determining the liquid limit. The liquid limit values of the specimens 5 minutes (LL 5min) and 30 minutes (LL 30min) mixing times were determined. The difference between liquid limit test values of the specimens for 5 minutes and 30 minutes mixing were calculated and summarized in Table 4.6

A significant difference (i.e. >5% of the liquid limit was obtained from the test on a specimen mixed for 5minutes) between the liquid limit from tests using 5 and 30minutes mixing times indicates a disaggregatinon of the clay-sized particles in the soil. If this disaggregation is confirmed by repeating the above procedures, the entire program of testing should be as follows:

- Limit the mixing times to not more than 5 minutes
- Make use of fresh soil for each moisture content point in Atterberg Limit tests.

Additionally the soil should be broken-down by soaking in distilled water, and not by drying and grinding. The soil should be immersed in distilled water to form slurry, which is then washed through a 425 µm sieves until the water runs clear. The material passing the sieve is collected and used for Atterberg Limit test. This method is done on as received preparation.

As seen from Table 4.6 for samples prepared in air dry condition the difference between 5 min and 30 min mix is Greater than 5 %. While for samples prepared in as received condition the difference between 5min and 30 min mix is less than 5 %,

Table 4.6 Atterberg limits at different conditions and mixing time.

Test pit	Depth	Condition	Liquid Limit	plastic Limit	plasticity Index	Difference LL30-LL5
TP1	-2.00	Air dry 5min	57	35	22	
		Air dry 30 min	65	35	30	8
		As received washed in tap water 5 min	65	40	25	
		As received washed in tap water 30min	67	40	27	2
TP2	-1.50	Air dry 5min	58	32	26	
		Air dry 30 min	64	32	32	6
		Air dry 5min 35°c	55	31	24	
		Air dry 35 min35°c	66	31	35	11
		Air dry 5min 105°c	57	34	23	
		Air dry 35 min105°c	68	34	34	11
TP4	-1.50	Air dry 5 min	61	35	26	

		Air dry 30 min	67	35	32	6
TP5	-1.50	Air dry 5 min	62	34	28	
		Air dry 30 min	74	37	37	12
TP6	- 2.00	As received washed in tap water 5min	61	37	24	
		As received washed in tap water 30min	64	37	27	3
TP7	-1.50	As received washed in tap water 5min	61	37	24	
		As received washed in tap water 30min	65	37	28	4
	-2.00	Air dry 5 min	59	36	23	
		Air dry 35 min	74	36	38	15
		As received 35ºc 5min	65	37	28	4
		As received 35ºc 30 min	69	37	32	
		As received 105ºc	71	41	30	4
		As received 105ºc 30 min	75	41	34	

4.2.1.4.4 Plasticity chart

Plasticity Index, the numerical difference between liquid limit and plastic limit, represents the range in water content through which a soil behaves like a plastic material. (Braja1997) observed that the plasticity index of the soil increase linearly with the percentage of clay- size fraction.

Experimental results from soils tested from different parts of the world indicate that clays, silts and organic soils lie in distinct regions of classification charts. A line is the boundary between clays, silts and organic clay. This line is defined by the equation [4.1].

PI = 0.73 (LL-20) [4.1]

The u line is the upper limits of the correlation between plasticity index and Liquid limit and expressed by Eq. [4.2]. Results above this line indicate error in testing. Hence conducting the test repeatedly is recommended. According to Fig. 4.3 the test results are all below the U-line. Hence the test results are considered acceptable (Budhu, 2000).

PI = 0.90 (LL-8) [4.2]

Where: Both PI and LL values are expressed in percent of equations Eq. 4.1 and Eq. 4.2.

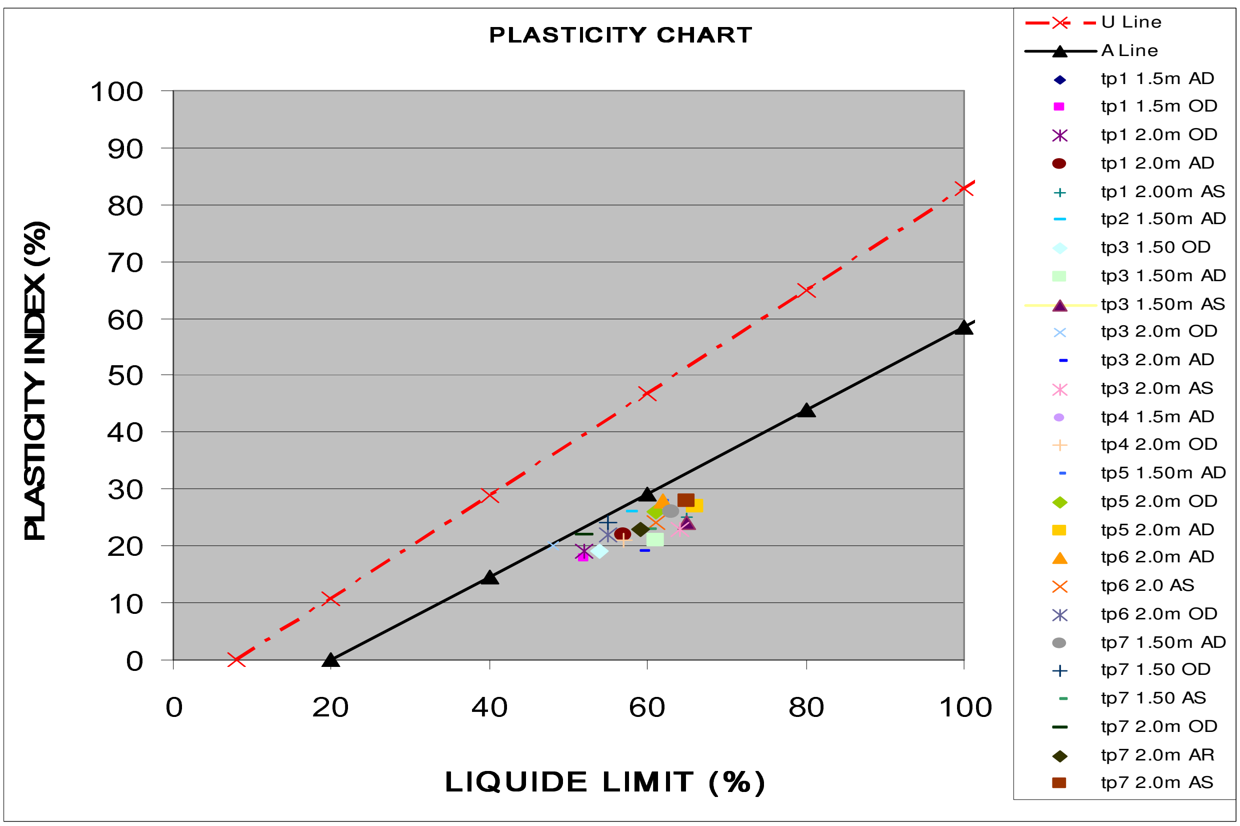

Fig 4. 3 Plasticity Chart

4.2.1.4.5 One point Liquid Limit Test Results

The one point liquid limit test is effective in determining the liquid limit of lateritic soils by using the formula

$$LL = w\,(N/25)^{\tan B} \quad [4.3]$$

Where: LL = Liquid limit

W = moisture content

N = No. of blows for Liquid limit

B = 0.12

When the number of blows is between 20 and 30, tan B is assumed to be zero. The result will be within the accuracy of the liquid limit test. Taking the value of tan B = 0.12 gives more accurate result (Lyon, 1971).According to equation Eq. 4.3 and using the value of tanB = 0.12, few results were calculated and summarized in Table 4.7

From the test results one can see that the one point liquid limit test is more or less acceptable for lateritic soils.

Table 4.7 Liquid limits comparison between conventional and one point test values.

Test pit	Depth	Condition	No. of blows	moisture content	calculated LL limit	Liquid Limit test
TP1	-1.50	Oven dry	33	51.10	52.83	52
			26	51.45	51.69	
			22	52.16	51.37	
		Air dry	34	54.33	56.38	57
			27	57.05	57.58	
			21	57.96	56.76	
	-2.00	Oven dry	33	50.29	52.00	51
			27	51.81	52.29	

TP1	-2.00		23	52.72	52.20	
		Air dry	33	51.10	52.83	57
			28	58.03	58.83	
			23	59.66	59.06	
		As received	35	60.19	62.68	65
			28	64.51	65.40	
			22	67.73	66.69	
TP2	- 1.50	Oven dry	33	51.34	53.09	54
			27	53.81	54.31	
			21	55.86	54.70	
		Air dry	34	57.93	60.12	61
			29	60.01	61.10	
			26	62.36	62.65	
		As received	34	64.65	67.09	65
TP3	-1.50	As received	27	64.96	65.57	
			22	65.20	64.20	
	-2.00	Oven dry	34	54.78	56.84	56
			28	54.85	55.60	
			20	55.91	54.43	
		Air dry	33	57.56	59.52	59
			27	58.23	58.77	
			22	60.14	59.22	
		As received	34	62.39	64.74	64
			27	63.33	63.92	
			22	64.02	63.04	
TP4	-1.50	Air dry	30	58.47	59.77	61
			28	61.85	62.70	

TP4	-1.50		22	62.22	61.27	
	-2.00	Oven dry	34	54.54	56.60	57
			27	57.10	57.63	
			20	60.13	58.53	
TP5	-1.50	Oven dry	34	52.60	54.59	55
			28	54.12	54.87	
			23	55.70	55.14	
		Air dry	36	57.72	60.31	
			27	62.41	62.99	
			22	63.64	62.66	62
	-2.00	Oven dry	36	57.70	60.30	61
			30	59.54	60.87	
			22	62.70	61.74	
Tp6	-1.50	Oven dry	35	51.90	54.05	55
			27	53.57	54.07	
			21	55.86	54.69	
		Air dry	33	57.93	59.90	62
			27	61.26	61.83	
			23	63.77	63.13	
		As received	33	58.60	60.59	61
			26	61.14	61.43	
			21	61.97	60.68	
TP7	-1.50	Oven dry	31	54.26	55.69	55
			27	54.95	55.47	
			21	56.19	55.02	
		Air dry	34	53.80	55.83	63
			28	58.03	58.83	

TP7	-1.50		23	68.66	67.97	
		As received	34	55.36	57.45	60
			28	58.21	59.01	
			21	63.19	61.87	
	-2.00	Oven dry	30	53.39	54.58	54
			26	54.26	54.52	
			23	55.15	54.59	
		Air dry	30	57.91	59.20	59
			28	59.17	59.99	
			23	60.14	59.54	
		As received	34	73.64	76.42	75
			27	73.97	74.66	
			22	75.91	74.75	

4.2.1.5 Activity

Skempton's colloidal activity is determined as the ratio of the plasticity index of the clay content to fines. He observed that, for a given soil, the plasticity index is directly proportional to the percent of clay-size fraction (i.e., percent by weight finer than 0.002 mm in size). Activity designated by "A_c" is defined as

$$A_c = \frac{PI}{C} \qquad [4.4]$$

Where C is the percent of clay - size fraction by weight. Activity has been used as an index property to determine the swelling potential of clays (Braja, 1997). Colloidal activity values for the soils under investigation are calculated and summarized in Table 4.8a.

The soil classification according to the activity number is given in Table 4.8a.

Table 4.8a Degree of Colloidal Activity.

Activity Number, AC	Soil Type
< 0.75	Inactive
0.75 ~ 1.25	Normal
> 1.25	Active

One can see from Table 4.8b, Skempton's colloidal activity values for TP1at 2.00m, TP4 at 2.00m and TP7 at 2.00m are less than 0.75 Table 4.9. Therefore, the investigated soils are in Kaolinite mineral range.

Accordingly, the soil type is inactive which is in agreement with the fact that the predominant clay minerals in lateratic soils are Kaolinite group. These soils are known to be inactive or normal. The low activity of most lateritic soils is due to the mode of weathering which involve the coating of the soil particles with Sesqueoxide, which results in the suppression of the surface activity of clay particles (Lyon, 1971).

Table 4.8 b Summary of Skempton's colloidal activity values.

Test pit	Depth	Condition	Clay Fraction %	Plasticity Index (%)	A_C (%)
Tp1	-2.00	Oven dry	49	19	0.388
		Air dry	56.3	22	0.391
		As received washed in tap water	58.3	25	0.429
TP4	-2.00	Oven dry	53.9	21	0.390
		Air dry	66.3	21	0.317
TP7	-2.00	Oven dry	48	22	0.458
TP7	-2.00	Air dry	62.9	23	0.366
		As received 35ºc	63.1	28	0.444
		As received 105ºc	61.5	30	0.488

Table 4.9 Typical value of Liquid Limit, Plastic Limit, and Activity of Some Clay minerals (Braja, 2002)

Minerals	Liquid Limit,LL	Plastic limits	Activity
Kaolinite	35-100	20-40	0.3 - 0.5
Illite	60-120	35-60	0.5 -1.2
Montmorillinite	100-900	50-100	1.5 -7.0
Halloysite (hydrated)	50-70	40-60	0.1- 0.2
Halloysite (dehydrated)	40-55	30-45	0.4 - 0.6
Attapulgite	150-250	100-125	0.4 -1.3
Allophne	200-250	120-150	0.4 -1.3

4.2.1.6 Shrinkage limit

Shrinkage limit of the soils samples under investigation was determined using ASTM test designation D427 procedures.

When moisture is gradually lost from soil, the soil mass as a whole shrinks. During drying to certain limiting value of water content, any loss of water is accompanied by a corresponding change in bulk volume (or void ratio). Below this limiting value of water content, no further change in volume occurs with loss of pore water.

Shrinkage ratio

$$SR = \frac{W_S}{\gamma_W V_f} \qquad [4.5]$$

SR=Shrinkage Ratio

W_S = Weight of dry soil

γ_W = unit weight of water in consistent units

Vf = dry volume of soil

Volumetric shrinkage test results are summarized in Table 4.10 for different pretreatment conditions. From the test results one can see that oven dried soil samples have generally higher values of volumetric shrinkage than that of air dry and as received. The drying of soil samples leads solid particles to come closer creating high cementation by Sesquioxides.

Table 4.10 volumetric Shrinkage limits at different Conditions.

Test pit	Depth	Condition	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Shrinkage limit (%)
TP1	1.50	Oven	52	33	19	20
		Oven un soaked	50	31	19	22
TP2	1.50	Air dry	64	32	32	13
TP3	1.50	As received	65	41	24	20
	2.00	As received	64	40	24	19
TP4	1.50	Air dry	61	35	26	16
TP5	2.00	Oven	61	35	26	17
		Air dry	66	39	27	13
TP6	2.00	As received	64	37	27	14
TP7	1.50	As received	65	37	28	11
	2.00	Oven	54	31	23	22
		Air dry	74	36	38	15
		As received	75	41	34	13

4.2.1.7 Free swell

The amount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals Montimorillonite influences the magnitude of swelling as compared to Illites and kaolinites (HaileMariam, 1992). The simplest test conducted is free swell test. The test is performed by slowly pouring 10cm^3 of dry soil which has passed the No. 40 (0.425mm) sieve in to 100 cm^3 graduated cylinder filled with distilled water. After 24 hours, final volume of the suspension is read. Hence, free swell is defined as

$$\text{Free swell} = \frac{\text{Final volume} - \text{Initial volume of the soil}}{\text{Initial volume}} \times 100\% \qquad [4.6]$$

Free swell test results for air dried samples are summarized in Table 4.11. From the test result one can see that the free swell of the soil under investigation ranges from 28% to 38%. Those soils having a free swell less than 50% are considered as low in degree of expansion (Teferra, 1999). Hence all soil samples under investigation are non expansive soils.

Table 4.11 Free swell test results at different Conditions.

Test pit	Depth	Test Condition	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)	
TP1	2	air dry	57	35	22	28	tap water
TP2	1.5	air dry	58	32	26	28	tap water
	2	air dry	62	35	28	30	tap water
TP3	1.5	air dry	61	40	21	38	tap water
	2	air dry	59	40	19	30	tap water
	2	air dry				28	distilled water

TP4	2	air dry				35	tap water
	2	air dry	63	42	21	35	distilled water
TP5	1.5	air dry	62	34	28	35	tap water
	2	air dry	66	39	27	33	tap water
TP6	2	air dry	62	34	28	30	tap water
TP7	2	air dry	59	36	23	38	tap water
	2	air dry				35	distilled water

4.2.1.8 Specific Gravity

Specific gravity of the soils samples under investigation was determined using ASTM test designation D854 – 92 procedures method 'A' for oven dry; method 'B' for as received and air dry samples. The dry mass of the soil for method B could be calculated by drying the soil specimen after the specific gravity test has been completed to dry the sample using 35ºC & 105ºC.
Specific gravity is used to calculate parameters such as void ratio, porosity, soil particle size distribution by means of the hydrometer and degree of saturation. The specific gravity tests were carried out and summarized for the some soil samples under investigation at different conditions, i.e air dried, oven dried and as received pretreatment conditions.

The test results summary is shown in Table 4.12 from the test results one can see that air dried and as received pretreatment conditions give nearly similar values. When the temperature decrease to 35ºc the specific gravity also decreases. This is shows that the oven drying temperature affect the value of the specific gravity. The specific gravity values of oven dry sample is less than the above two. Hence specific gravity significantly changes upon drying prior to testing and oven drying temperature. The detected specific gravity values ranges from 2.61 to 2.97 this agree with the value of specific gravity obtained for similar soils (Lyon, 1971)

The available data indicate that specific gravities vary not only with the soil textural but also within different fractions. The specific gravity has been used as a measure of the degree of maturity (laterization). Lateritic soils have been found to have very high specific gravities values between 2.6 to 3.4 (Lyon, 1971).

Table 4.12 The Values of specific gravity at different Conditions.

Test pit	Depth	Condition	Specific gravity
TP1	-1.50	Oven	2.72
		Air dry 35º	2.73
		Air dry 105º	2.8
		As received 35º	2.72
		As received 105º	2.85
	-2.00	Oven	2.72
		Air dry 35	2.77
		Air dry 105	2.81
		As received 35º	2.77
		As received 105º	2.84
TP2	-1.50	Air dry 35º	2.65
		Air dry 105º	2.74
		As received 35	2.64
		As received 105	2.72
	-2.00	Oven	2.73
		Air dry 35º	2.75
		Air dry 105º	2.81
		As received 35	2.61
		As received 105	2.8
TP3	-1.50	As received 35º	2.7
TP3	-1.50	As received 105º	2.78
	-2.00	Oven	2.79

		Air dry 35	2.67
		Air dry 105	2.82
		As received 35º	2.74
		As received 105º	2.85
TP4	-1.50	As received 35º	2.71
		As received 105º	2.79
	-2.00	Oven	2.77
		Air dry 35º	2.66
		Air dry 105º	2.78
		As received 35º	2.65
		As received 105º	2.82
TP5	-1.50	As received 35º	2.72
		As received 105º	2.82
	-2.00	Oven	2.71
		Air dry 35º	2.67
		Air dry 105º	2.83
		As received 35º	2.69
		As received 105º	2.81
TP6	-1.50	As received 35º	2.81
		As received 105º	2.92
	-2.00	Oven	2.80
		Air dry 35	2.74
		Air dry 105	2.83
		As received 35º	2.81
		As received 105º	2.97
TP7	-1.50	Oven	2.82
		As received 35º	2.83

		As received 105º	2.93
	2.00	Oven	2.79
		Air dry 35º	2.68
		Air dry 105º	2.84
		As received 35º	2.71
		As received 105º	2.83

4.2.1.9 Grain size Analysis

4.2.1.9.1 General

Grain size analysis is an attempt to determine the relative properties of different grain sizes which make up a soil mass. The soil samples under investigation are almost fine that particle size retain in 2mm sieve was insignificant; hence hydrometer analysis was used with sodium Hexametaphosphate dispersing agent.

4.2.1.9.2 Test Procedures

Dry preparation

The soil sample brought from field was first air dried and then pulverized before it was screened through the nest of sieves. Some of the soil particles passing the No. 10 sieve is oven dried at 105 ºC for 24 hours for oven dried sample (OD) an air dried (AD) sample is also taken. Both samples were subjected to hydrometer analysis and the results were expressed by a plot of percent finer (passing) by weight against size of soil particles in millimeters on a log scale (According to the procedure detailed in ASTM D422-63).

Wet preparation

Wet soil sample preparations were carried out on moist soil samples for grain size analysis tests following the procedures mentioned in (ASTM D422-63 and (Blight, 1997)).

Soil classification

A soil classification system is arrangement of different soils in to groups having similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known and to provide the engineer with accurate method of soils description. In this thesis work the following classification was used.

Average grain size classification of laterites (Lyon, 1971)

Lateritic clays < 0.002 mm
" silts = 0.002 ~ 0.06 mm
" sands = 0.06 ~ 2 mm
" gravels = 2 ~ 60 mm
and courser > 60 mm

Average grain size classification according to USCS (Budhu, 2000)

Gravel 75mm - 4.75mm
Sand 4.75mm - 0.075mm
Silt 0.075mm - 0.002mm
Clay < 0.002mm

Average grain size classification according to AASHO (Teferra, 1999)

Gravel >2mm
Sand 2mm - 0.05mm
Silt 0.05mm - 0.002mm
Clay < 0.002mm

4.2.1.9.3 Test Results and Discussions

The grain size analysis test results for all soil samples under investigation at different testing conditions and classification system are summarized in Table 4.13. The corresponding grain size distribution curves are shown in Figs. 4.4 -4.8 and A.1-A.6. The test results presented in the main body of the thesis are only the typical ones. The test result curves of all soil samples under investigation are shown in the Appendix-A. The values obtained from the gradation tests were analyzed with respect to the effect of pre-treatment, soil variations laterally and depth wise.

Effect of Pretreatment

Oven dried (OD), air dried (AD) and as received (AR) sample preparations were carried out to investigate the affect of pretreatment on grain size distribution of the soil samples under investigation. The test results are shown in Table 4.13 and Figs. 4.4 – 4.8 and FigsA.1-A.6 From the curves one can observe that the three methods of pretreatment produce a change in cumulative percentage passing between OD, AD and AR for sample TP1at 1.50m depth, TP1 at 2.00m depth, TP2 at 2.00m depth, TP4 at 2.00m depth, TP6 at 1.50m depth and TP7 at 2.00m depth the difference between OD and AR sample clay fraction greater than 7% refer to section 2.2.2.3 moreover oven drying temperature also effect on the fraction of clay when oven drying temperature decrease from 105º C to 35º C the clay fraction increase.

Effect of Soil Sampling Locations

Grain size distribution tests were carried out on soils samples from different locations to see the variation of soils laterally. The size of the particles that constitute soils has a direct influence on the density of the soil and other engineering properties. The gradation test results are shown in Fig. 4.4. From the curves one can observe that the soil samples TP1, TP4 and TP7 have the same shape of cumulative percentage passing curve. Distance of sampling is about 1.5 Kms between test pits TP1and TP4 and 3kms between TP4 and TP-7. This similarity may indicate that lateritic soils of the area under consideration have the same characteristics according to their corresponding lithological classification. The lithological classification of the soils is mentioned in section 2. 1.2.

To see the variation of soils along the depth profile, grain size distribution tests were carried out. The test results for soil samples TP1at 1.50m, TP1at 2.00m & TP2 at 2.00m shown in Fig. 4.8 have nearly identical gradation curves. Soil properties, including gradation, along the profile may change due to variation in degree of weathering. Soil samples with similar gradation curves have high possibility of having same engineering properties as far as their chemical and mineralogical compositions are similar.

Table 4.13 Percentage Amount of the Grain Sizes for different test conditions and classification

Test pit	Depth	T est Condition	Classification According to	Percentage amount of Particle Sizes			
				Gravel	Sand	Silt	Clay
TP1	-1.50	As received 35º	USCS	0	13.1	28.7	58.2
			AASHO	0	20	21.8	58.2
			lateratic	0	17	24.8	58.2
		As received 105º	USCS	0	12.9	30.4	56.7
			AASHO	0	17	26.3	56.7
			lateratic	0	20	23.3	56.7
	-2.00	Oven-dried	USCS	0	14	38	49
			AASHO	0	23	28	49
			lateratic	0	18	33	49
		Air-dried	USCS	0	11.2	32.5	56.3
			AASHO	0	20	23.7	56.3
			lateratic	0	15	28.7	56.3
		As received 35º	USCS	0	14.2	26.6	59.2
			AASHO	0	20	20.8	59.2

TP1	-2.00	As received 35º	lateratic	0	17	23.8	59.2
		As received 105º	USCS	0	14	27.6	58.3
			AASHO	0	17	24.7	58.3
			lateratic	0	20	21.7	58.3
TP2	-2.00	As received 35º	USCS	0	13.6	29.7	56.7
			AASHO	0	18	25.3	56.7
			lateratic	0	16	27.3	56.7
		As received 105º	USCS	0	8.5	37	54.4
			AASHO	0	17	28.6	54.4
			lateratic	0	12.5	33.1	54.4
TP4	-2.00	Oven-dried	USCS	0	15.4	30.7	53.9
			AASHO	0	23	23.1	53.9
			lateratic	0	19	27.1	53.9
		Air-dried	USCS	0	10.5	23.2	66.3
			AASHO	0	15	18.7	66.3
			lateratic	0	12.5	21.2	66.3
		As received 35º	USCS	0	11.2	19.1	69.7
			AASHO	0	14	16.3	69.7
			lateratic	0	12.5	17.8	69.7
		As received 105º	USCS	0	11.2	21.6	67.2
			AASHO	0	14	18.8	67.2
			lateratic	0	12.5	20.3	67.2
TP6	-1.50	As received 35º	USCS	0	8.5	36.6	55
			AASHO	0	15	30	55
			lateratic	0	11.5	33.5	55
		As received 105º	USCS	0	13.5	32.7	53.8
			AASHO	0	15	28.7	53.8

TP6	-1.50		lateratic	0	11.5	30.2	53.8
TP7	-2.00	Oven-dried	USCS	0	6.2	45.8	48
			AASHO	0	25	27	48
			lateratic	0	15	37	48
		Air-dried	USCS	0	8	29.2	62.9
			AASHO	0	16	21.1	62.9
			lateratic	0	12	25.1	62.9
		As received 35º	USCS	0	9.7	27.2	63.1
			AASHO	0	13.5	23.4	63.1
			lateratic	0	12	24.9	63.1
		As received 105º	USCS	0	9.5	29	61.5
			AASHO	0	13.5	25	61.5
			lateratic	0	13	26.5	61.5

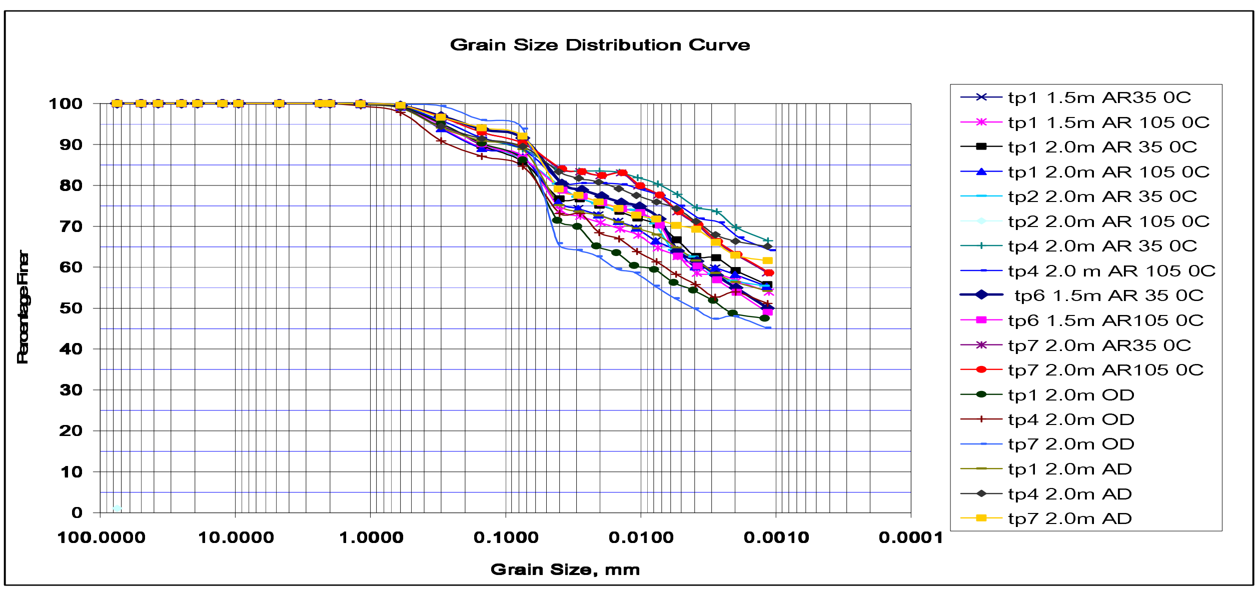

Fig. 4.4 Grain size distribution curve for at different pretreatment condition

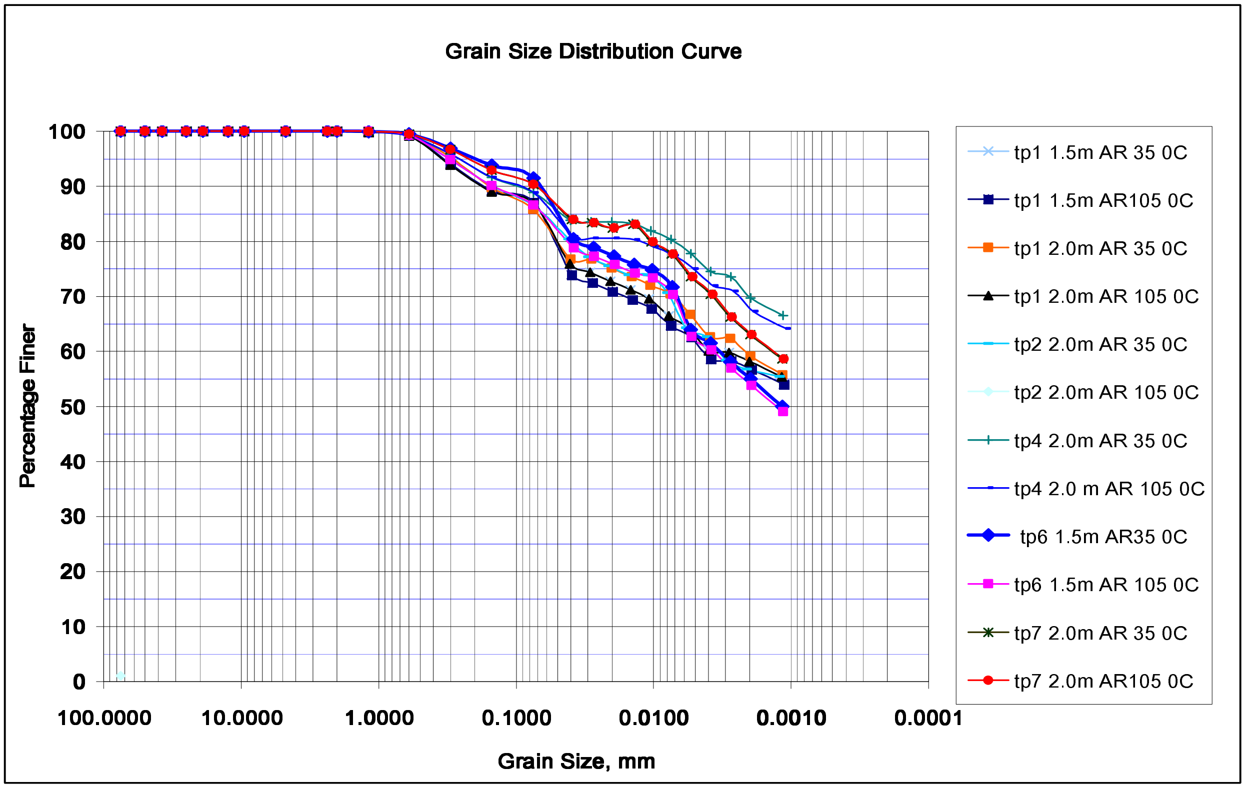

Fig. 4. 5 Grain size distribution curve for as received (AR) test condition

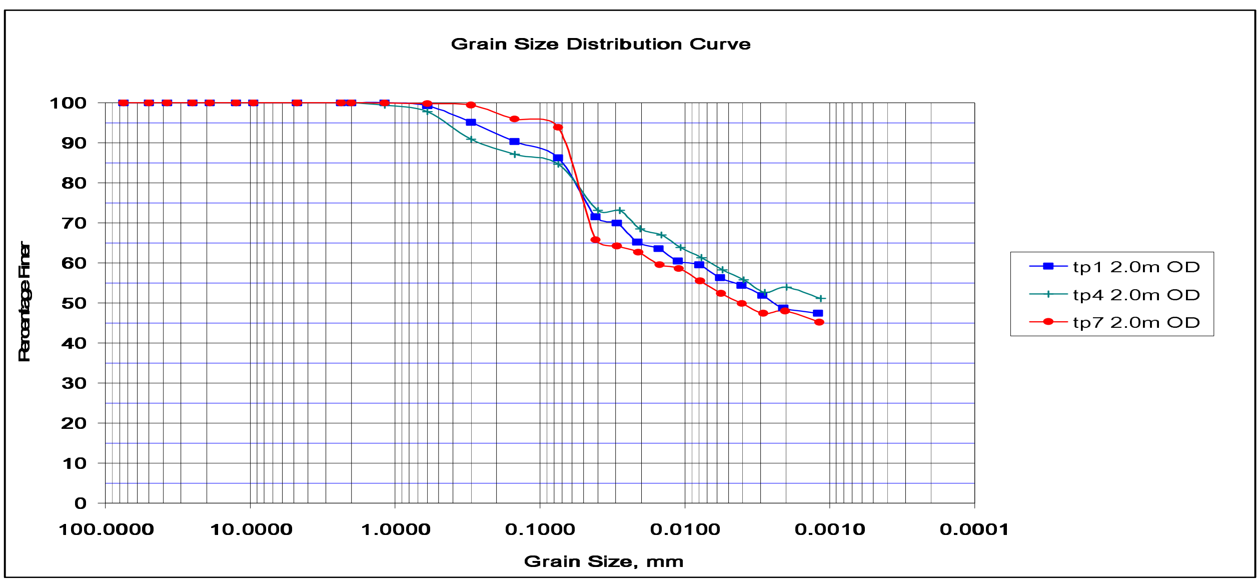

Fig. 4.6 Grain size distribution curve for Oven dry (OD) test condition

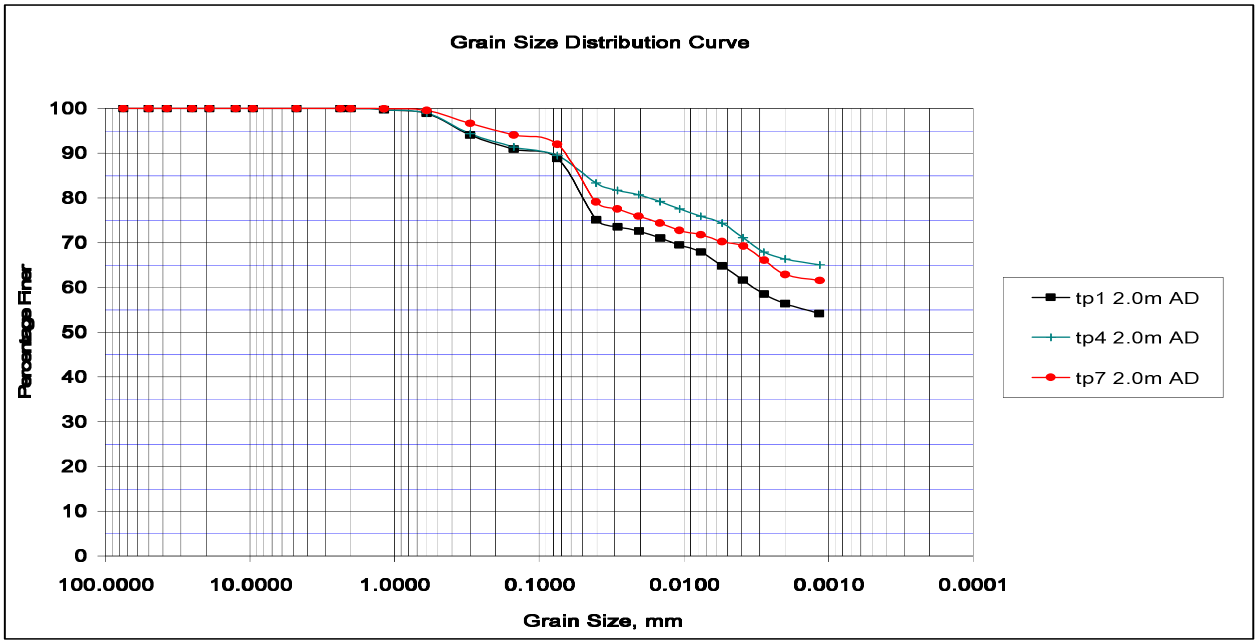

Fig. 4. 7 Grain size distribution curve for TP-1, TP-4, & TP-7 Air dry (AD) test condition

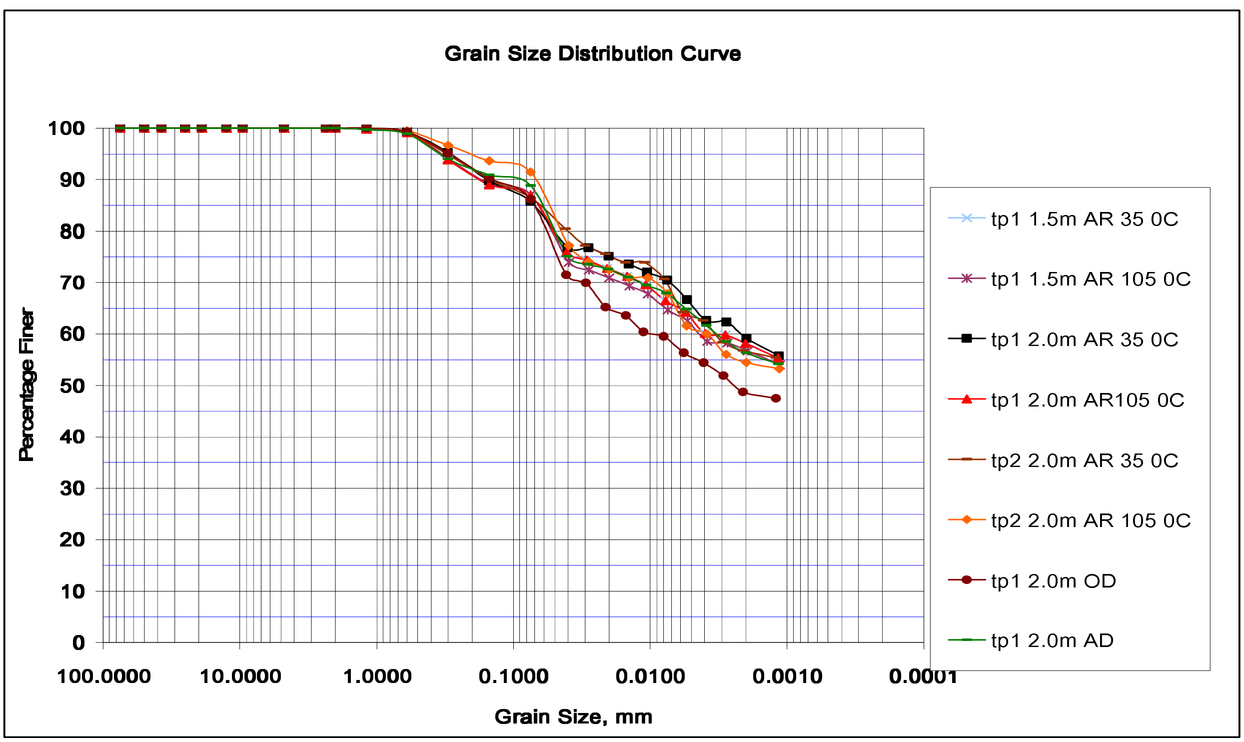

Fig. 4. 8 Grain size distribution curve for TP-1 & TP-2

4.2.1.10 Classification of the Soils

Wesley classifies residual soils on the basis of minerals and (Lyon ,1971) pedological classification system on the other hand on the basis of climate, drainage, and topography and parent material. The AASHTO classification system is convenient as a basis for classifying tropically weathered soils. Some road construction stake holders' uses conventional soil classification system using the grain size distribution and the Atterberg limit values. The soils under investigation have been classified according to AASHTO M-145 and UCSC method is also shown in Table 4.14.

All samples fall under group A-7 sub group A-7-5 (GI=20) according to AASHTO. Classification according to USCS from plasticity chart places all sample below A-line and theories of LL of oven dry / not dry > 0.75 all sample fall in this range this show that the soil is MH (Inorganic silt).

Table 4.14 Classification According to the AASHTO and USCS.

Test pit	Depth	Test cond	LL (%)	PI (%)	% amount of Passing			Classification According to AASHTO		% amount of Particle Sizes				Classification According to USCS
					2	0.425	0.075	Group	Group Index	Gravel	Sand	Silt	Clay	
TP1	2.0	OD	52	19	100	95.09	86.22	A-7-5	20max	0	14	38	49	MH
		AD	57	22	100	94.04	88.84	A-7-5	20max	0	11.2	32.5	56.3	MH
		AR	65	25	100	94.06	87.09	A-7-5	20max	0	14	27.6	58.3	MH
TP4	2.00	OD	57	21	100	90.91	84.64	A-7-5	20max	0	15.4	30.7	53.9	MH
		air	63	21	100	94.40	89.51	A-7-5	20max	0	10.5	23.2	66.3	MH
Tp7	2.00	oven	54	23	100	99.38	93.83	A-7-5	20max	0	6.2	45.8	48	MH
		AD	59	23	100	96.65	92.03	A-7-5	20max	0	8	29.2	62.9	MH
		AS	71	30	100	96.96	90.54	A-7-5	20max	0	9.5	29	61.5	MH

4.2.2 Geochemical Tests and X- Ray Diffraction (XRD) Test

4.2.2.1 Geochemical Tests

Mineralogy controls the sizes, shape & surface characteristic of the particles in the soil. These features along with interaction with the fluid phase, determine plasticity, swelling, compression strength and hydraulic conductivity behavior. Thus, mineralogical composition (together with structure) is an important factor that is fundamental to the understanding of Geotechnical properties. Geochemical (oxide) tests are carried out to know quantitatively main oxides of the soil material. Almost all soils contain some amount of colloidal oxides and hydroxides. The oxides and hydroxides of Aluminium, iron and silicon are of greatest interest since they are the ones most frequently encountered. Iron and Aluminium oxides coat mineral particles, or cement particles of soils together. It may also occur as distinct crystalline units, such as Hematite, Gibbsite and Magnetite (Mitchell, 1979).

Geochemical tests were conducted at Geological Survey of Ethiopia Geochemical Laboratory. Atomic Absorption Spectrometer and Colorometer Analysis methods were used to get the percentage oxide composition of the soils under investigation. The test results are shown in Table 4.16

The degree of laterization of the soil samples can be evaluated based on ratio of Silica/Sesquioxides as detailed in section 2.1.2 the Sesquioxide, designated as R_2O_3, is the combination of Aluminium oxide (Al_2O_3) and Iron oxide (Fe_2O_3). The chemical formula SiO_2 designates the silica. Rations less than 1.33 have been considered as true laterites, those between 1.33 and 2.00 of lateritic soils and those greater than 2.00 of non lateritic tropically weathered soils.

The test results in Table 4.16 Silica – Sesquioxide ratio below 1.33 except TP2 1.50m and TP2 2.00m. This indicates that the soils are all true laterites and TP2 1.50m and TP2 2.00m are lateritic. True laterites are simply referred as laterites. The soil of such kind are highly laterized i.e., Sesquioxides content are high. Degree of laterization increase with depth for TP4 and TP5 while for TP2 degree of laterization almost similar with depth.

Table 4.15 Oxide Composition in Percent

Test pit	Sampling depth[m]	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	k_2O	MnO	H_2O	LOI	Ti_2O	P_2O_5	$\frac{SiO_2}{R_2O_3}$
TP1	-2.00 m	42.22	21.98	17.42	0.31	<0.01	0.8	1.33	0.9	4.86	9.85	1.62	0.06	1.07
TP2	-1.50 m	49.1	19.5	13.54	0.41	0.07	1.02	1.64	0.2	4.2	9.23	1.4	0.07	1.49
Tp2	-2.00m	48.88	20.97	11.70	0.57	0.07	0.38	1.47	0.16	4.83	8.97	1.49	0.05	1.50
TP4	-1.50m	43.28	22.77	12.33	0.59	0.11	0.22	1.76	0.16	5.85	10.05	1.58	0.08	1.23
TP4	-2.00m	42.1	22.91	16.04	0.36	<0.01	0.69	1.2	0.2	4.33	10.48	1.54	0.09	1.08
TP5	-1.50 m	42.65	22.22	15.22	0.58	0.04	0.53	1.27	0.13	6.28	10.33	1.59	0.09	1.14
TP5	-2.00m	41.42	21.97	16.2	0.46	<0.01	0.52	1.18	0.27	6.25	10.29	1.62	0.1	1.09
TP6	-1.50m	44.48	22.13	11.92	0.42	<0.01	0.25	1.27	0.16	9.62	9.62	1.44	0.05	1.31

4.2.2.2 X- Ray Diffraction (XRD) Test

X-Ray Diffraction is the most widely used technique for identification and characterization of clay minerals. Clay minerals consist of tiny crystals which are themselves made up of ordered arrays of atoms, arranged in periodic or repetitive way. Using the above method the major constituent minerals were determined. These are Kaolinite,Halloysite and Montormorillonite.

XRD test was also undertaken at Geological Survey of Ethiopia Geochemical Laboratory and petrography laboratory using X-Ray Diffraction analysis by powder diffraction method after grinding the bulk soil sample without separating the clay fraction.

Table 4.16 Mineralogical Composition

Test pit	Sampling depth[m]	Mineral Identification	Chemical formula
TP1	-2.00 m	Quartz Magnetite Kaolinte Hematite	SiO_2 Fe_3O_4 $Al_2(Si_2O_5)(OH)_4$ Fe_2O_3
Tp2	-2.00m	Quartz low Magnetite Kaolinte Hematite	SiO_2 Fe_3O_4 $Al_2(Si_2O_5)(OH)_4$ Fe_2O_3
TP4	-2.00m	Phosposiderite Kaolinte Hematite Quartz low	$FePO_4(H_2O)$ $Al_2(Si_2O_5)(OH)_4$ Fe_2O_3 SiO_2

TP5	-1.50 m	Quartz low Pyrophyllite	SiO_2 $Al (Si_2O_5)(OH)$
TP5	-2.00m	Albite calcian low Kaolinte Quartz low Aluminiar Hematite	$(Na_{0.75}Ca_{0.25})(Al_{1.26}Si_{2.74}O_8)$ $Al_2(Si_2O_5)(OH)_4$ SiO_2 $(Fe_{0.86} Al_{0.14})_2O_3$

4.2.3 Discussions

The points addressed in the thesis work are reanalyzed and discussed as follows.

It is tried to check loss of water of hydration on soil samples under investigation. The difference in moisture contents between that of oven temperatures of 105^{o}c, 50^{o}c and 35^{o}c for all soils under investigation the difference between105^{o}c and 50^{o}c are below 4 % but for 35^{o}c TP1, TP2 and tp3 the difference below 4% and for other greater than 4 % variation in moisture content indicates that the soils sample higher moisture contain have structural water. Hence one may use oven temperature lower to 35 ^{o}c for water content determination for higher moisture content.

The effect of pre treatments were checked by conducting index tests on oven 105^{o}c dried, air dried and as received (moist condition). The corresponding test results show that the values under different conditions prior to testing results not significant variation. Laterites collected from distinct wet and dry seasons are not sensitive to pre treatment as mentioned by Morin W.J. and Todor P.C. (Lyon, 1971). Since soils under investigation collected from regions subjected to distinct wet and dry seasons, the pretreatment has not affect the test results which are in agreement with their findings.

The effect of disaggregation of clay – size particles upon test manipulation were obtained for some soil samples under consideration by varying liquid limit testing methods. All pretreatment conditions were considered. The test results show that the soil under investigation contains concretionary which is broken down by testing manipulation. Force induced during test manipulation detaches the bond between particles due to the presence of Sesquioxides. Hence one has conduct limit tests with fresh samples for each point.

According to USCS, all the soil samples fall under MH inorganic clay. Few research works were carried out on tropical soils both artificially combined and actual samples. The test results show that, the values on liquid limit versus plasticity index graph lie below the A – line. It indicates that the soil samples contain of the mineral kaolinite. The tropical soils properties vary with their minerals composition. Classifying laterite soils using only USCS classification system not shows any about the mineral moreover according to USCS classification the soil sample fall in MH group these soils are poor engineering and unsuitable for various engineering purpose However ,from the test result the sample have high strength and good engineering properties . Soil classification using USCS only mislead the classification system therefore it should be accompanied by mineralogical test results.

Activity test results for soil samples 0.317-0.488 are less than 0.75. Activity less than 0.75 for inactive soils. Laterite soils are inactive or normal due to the Sesquioxides suppress the activity of the clay particles.

The specific gravity test results are 2.61 to 2.97. The values are higher than the specific gravity of the temperate zoon soils, which is about 2.65 to 2.70. The contributing factor for rise of the specific gravity is due to high amount of iron oxide. Lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (Lyon, 1971).

Geochemical tests, X-Ray Diffraction and index tests indicate that the soils of Wolayita Sodo area are laterites having high concentration of Iron Oxide and Aluminum Oxide / Sesqueoxide / and clay mineral kaolinite. Based on the above result unsaturated soil mechanics test procedures were conducted for the shear strength parameter determination.

4.2.4 Shear strength test

4.2.2.1 General

The safety of any geotechnical related structure is dependent on the shear strength of the soil. Shear strength is the internal resistance of soils to shear forces. Shear strength is required to make estimates of the load bearing capacity of soils, the stability of geotechnical structures and in analyzing the stress strain characteristics of soils.

Unconsolidated Undrained (UU) or quick are useful for testing the shear strength of clayey soils for foundation analysis. During construction on cemented clays only a small amount of consolidation would be taking place in actual field, causing a little change in moisture content .Similarly, for analyzing clay slopes of cuts, the shear strength can be determined by such undrained or quick tests.

4.2.2.1 Sample preparation

4.2.3.2.1 Unsaturated sample

Sixteen Samples were made at 2m depth below the natural ground level and four Samples were made at 1.5 m depth below the natural ground level, using hydraulic jack in 100mm diameter tube. The samples were sealed by wax cover by and rapped plastic bag and moist towel to maintain natural moisture content. In laboratory the samples were stored at 20ºc room temperature. (Blight ,1997) recommended that the specimen diameter should not be less than 76mm for residual soil but due to the triaxial machine limitation 38mm diameter samples were conducted for this work. Test specimen were prepared from the 100mm diameter tube by hydraulically pushing the soil into three 38mm diameter tubes simultaneously from which a 76 mm long specimen was extruded .Three specimen from the same level were used for each of the triaxial and two specimen for unconfined compression test .

Unsaturated soils specimen have high initial matric suction. Therefore, there is no need of saturating the sample. The usual strain rate for shear testing of unsaturated soil is 0.017-0.03%/s. 1.02/60/s is used as test strain rate. The three identical specimens are confined at 100Kpa, 200kpa, 300kpa by considering over burden pressure and load ranges from the possible structure. The Limitation of the testing machine is also a factor. ASTM 2850 For Unconsolidated Undrained test and ASTM 2166 test method for Unconfined Compression were used.

4.2.3.2.2 Saturated sample

For the saturated soil the same procedure as the above was used the only difference is saturation done by using back pressuring increase of pore water pressure and confining pressure at the same time. The magnitude of a cell pressure increment is 50 KPa until the back pressure reaches 300 KPa. Increase in the cell pressure was done until the pore pressure parameter B Eq.(4.1) is equal to or greater than 0.95 for each increment maintaining the difference between cell pressure and back pressures as 10Kpa. The sample satisfied the above condition shearing stage started using ASTM 2850 procedures. Three identical samples are confined at cell pressure of 50 KPa, 100 KPa, 200 KPa this is due to the problem of the Triaxial machine above 200kpa confining pressure applied during saturation which the pressure gauge decrease back ward.

$$B=\frac{\delta U}{\sigma_3} \qquad [4.\ 7]$$

Where B=pore pressure parameters

δU = change in pore pressure (Back Pressure)

σ_3 =cell pressure

4.2.3.2.3 Test Results and Discussions

Mohr circle indicate that the curve relation between shear strength and total normal stress soil parameteers obtained form test result for unsaturated soil for TC-UU test in both Cu and Ø while for saturated one only Cu. As shown in Table 4.18 the shear parameter of unsaturated soil Cu value ranges between 150.60-172.48Kpa the Ø value 12 º - 20 º, For the saturated sample from TP4 the C value is 117.74 Kpa the value of less than the unsaturated value. Unconfined compression strength qu gives a value of 215 – 385 Kpa value. Mohr Circle and devitor stress curve are shown in Fig 4.9a - 4.19b. Degree of saturation ranges between 71-90%. From deviator stress curve for UU test one can see that at initial stage deviator stress for higher confining pressure attain lesser deviator stress this is due to structural formation of the soil. From unconfined compression test result, the entire soil sample was categorized under very stiff clay according to Table 4.17 and during sample preparation I observe that fine grain particles are dominant in TP3 and TP6 that is why one gets very lager difference between the same samples. The Devitor stress curves are shown in Fig 4.20 - 4.24.

Table 4.17 General relationship of Consistency and Unconfined Compression strength of clay (Braja, 2002)

	qu
Consistency	KN/m
Very soft	0-25
Soft	25-50
Medium	50-100
Stiff	100-200
Very Stiff	200-400
Hard	>400

a) UU Test b) UC Test

Fig 4.9 fig a & b failure plan for UU and UC test respectively

Table 4.18 Summary Table for Shear Strength Parameters

TP	Depth	Shear Strength parameters of UU test			Shear Strength parameters of UC test			wt .soil (gm)	γ wt	γ dry		moisture content		Specific gravity S_g				Degree of saturation			
		C_u unsat	C_u sat	Ø	qu1	qu2	Avg C_U			105°	35°	105°	35°	OD	AD	AR 35°	AR 105°	OD	AD	AR 35°	AR 105°
TP1	-1.5	151.59	-	18	-	-	-	155	18	13.53	13.90	30.5	27	2.72	2.8	2.72	2.85	85	83	80	81
	-2.0	150.60	-	17	-	-	-	154	17	13.53	13.89	29.2	25.9	2.72	2.81	2.77	2.84	82	79	75	78
TP2	-1.5	150.91	-	18	-	-	-	148	17	12.76	13.15	31.6	27.7	-	2.74	2.64	2.72	-	78	75	79
	-2.0	153.27	-	18	255	262	129	145	16	12.57	12.93	31.3	27.6	2.73	2.81	2.67	2.8	75	74	72	74
TP3	-1.5	-	-	-	-	-	-	145	16	12.56	12.92	31.3	27.7	-	-	2.7	2.78	-	-	71	74
	-2.0	167.14	-	12	254	334	147	149	17	12.84	13.23	31.7	27.9	2.79	2.82	2.74	2.85	78	78	74	77
TP4	-1.5	-	-	-	-	-	-	141	16	11.88	12.25	35.6	31.4	-	-	2.71	2.79	-	-	73	76

	-2.0	168.87	117.74	13	-	-	-	149	17	12.28	12.68	38.1	33.7	2.77	2.78	2.65	2.82	87	87	85	86
TP5	-1.5	-	-	-	-	-	-	151	17	12.32	12.72	39.3	34.9	-	-	2.72	2.82	-	-	87	89
	-2.0	151.43	-	12	222	215	109	148	17	11.91	12.29	41.1	36.7	2.71	2.83	2.69	2.81	90	87	86	88
TP6	-1.5	-	-	-	-	-	-	147	17	12.32	12.76	35.4	30.8	-	-	2.81	2.92	-	-	75	78
	-2.0	165.66	-	20	232	385	154	142	16	11.86	12.28	36.3	31.6	2.8	2.83	2.81	2.97	77	77	71	74
TP7	-1.5	-	-	-	-	-	-	150	17	12.70	13.15	34.6	30	2.82	-	2.83	2.92	83	-	77	81
	-2.0	172.48	-	16	262	263	131	146	17	12.35	12.78	34.4	29.8	2.79	2.84	2.81	2.83	79	78	72	78

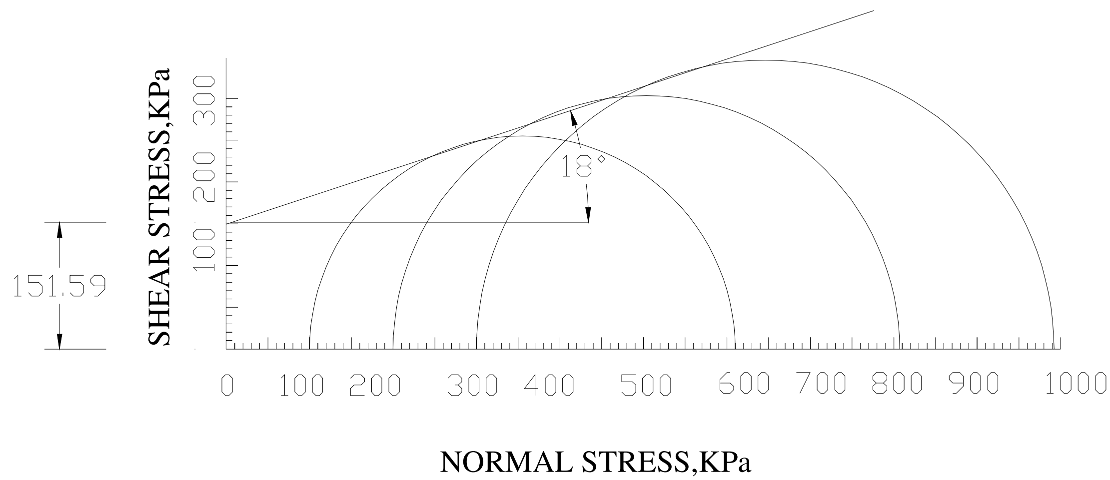

Fig 4. 10a Shear strength parameter for cell pressures of 100,200 & 300 KPa at 1.50 m depth at TP1

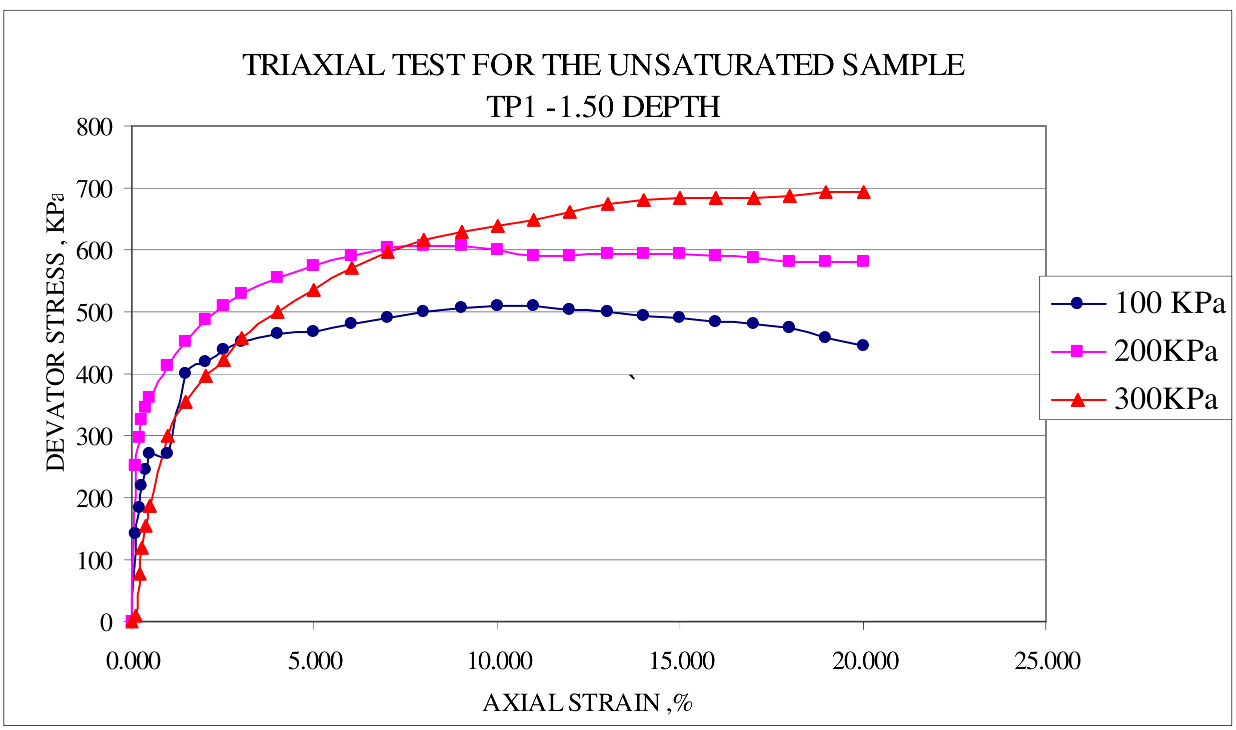

Fig 4. 10b Deviator Stress Vs Strain at 1.50 m depth at TP1

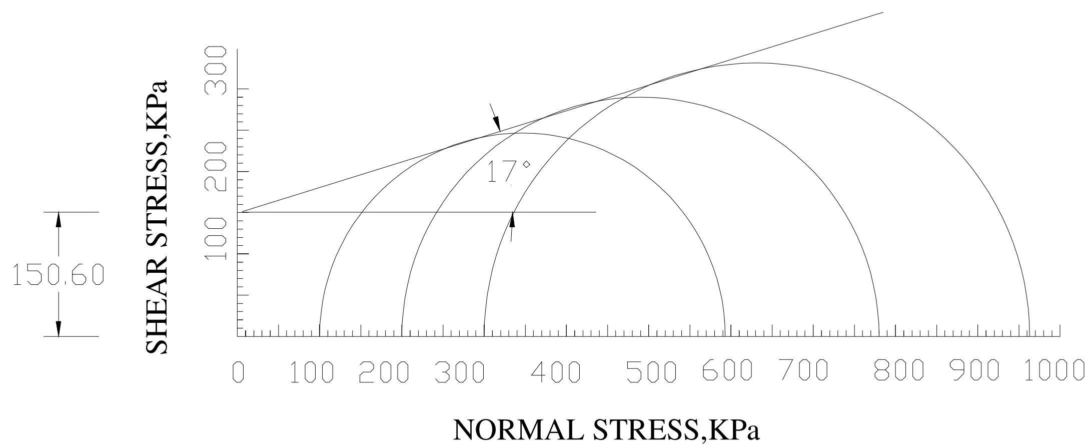

Fig 4. 11a   Shear strength parameter for cell pressures of 100,200 & 300 KPa at 2.00 m depth at TP1

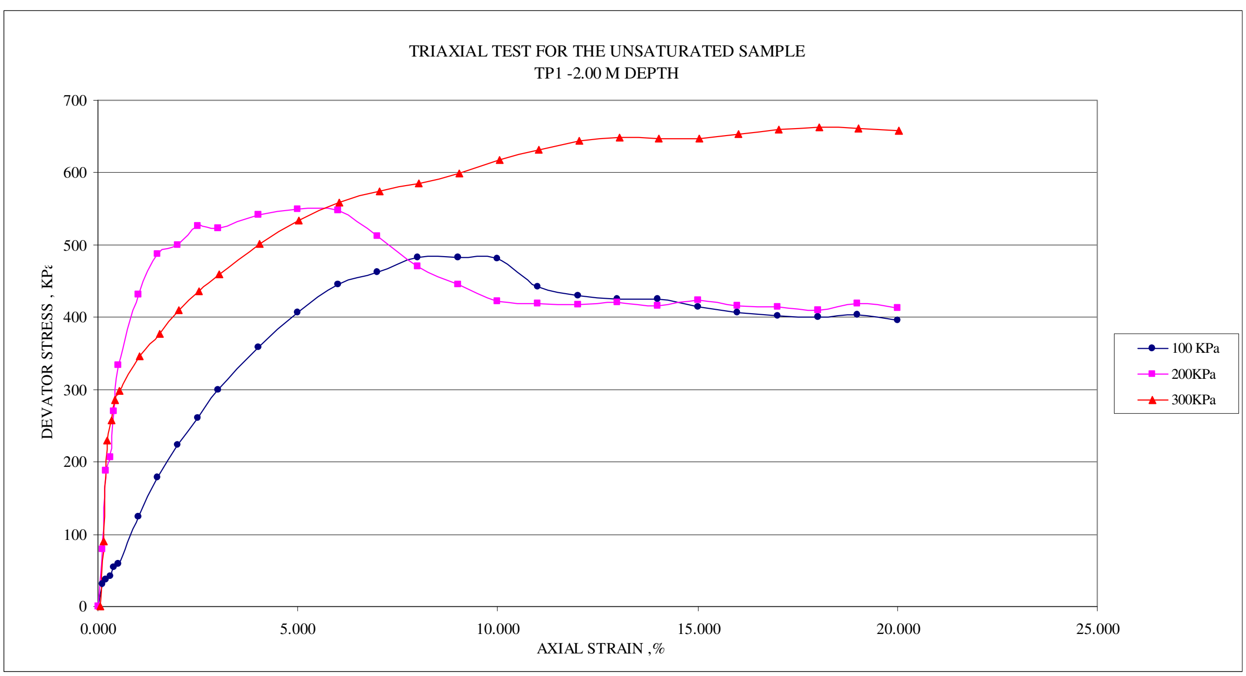

Fig 4. 11b Deviator Stress Vs Strain at 2.00 m depth at TP1

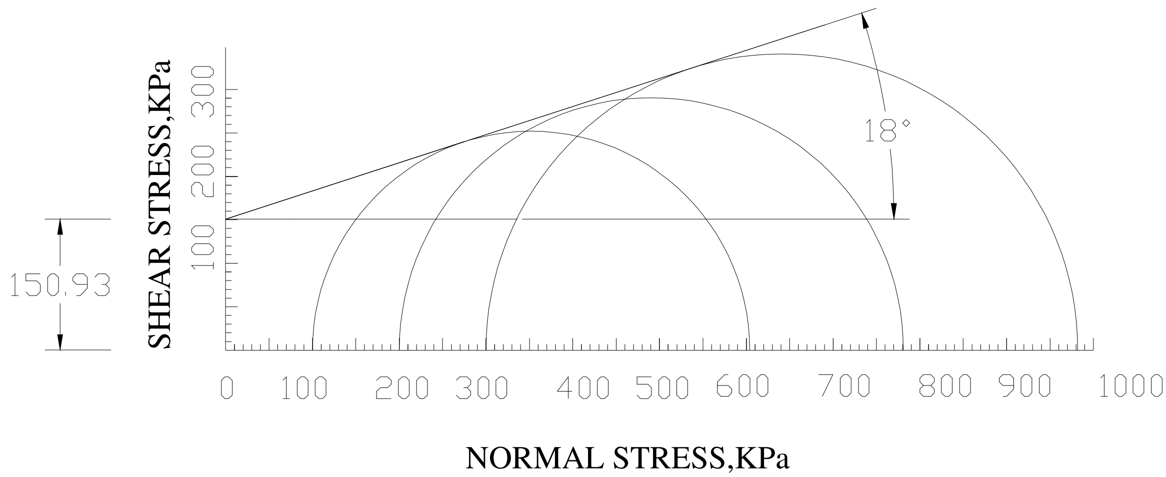

Fig 4. 12a Shear strength parameter for cell pressures of 100,200 & 300 KPa at 1.50 m depth at TP2

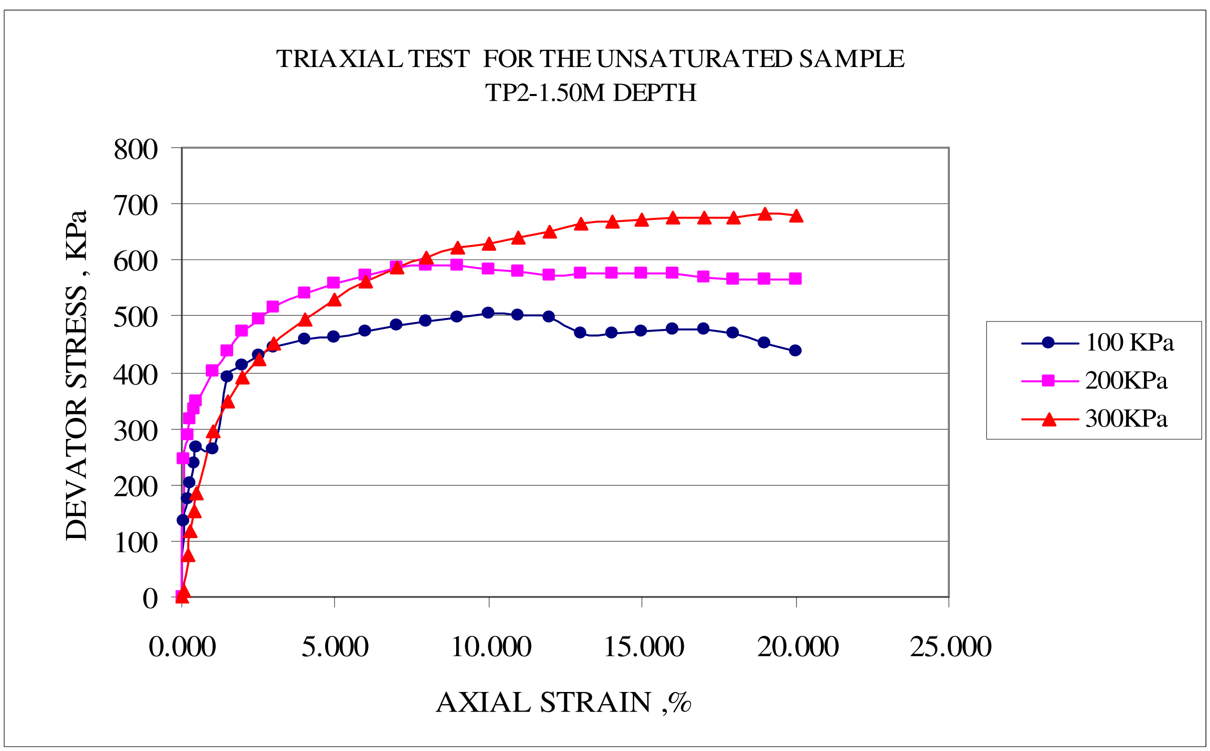

Fig 4. 12 b Deviator Stress Vs Strain at 1.50 m depth at TP2

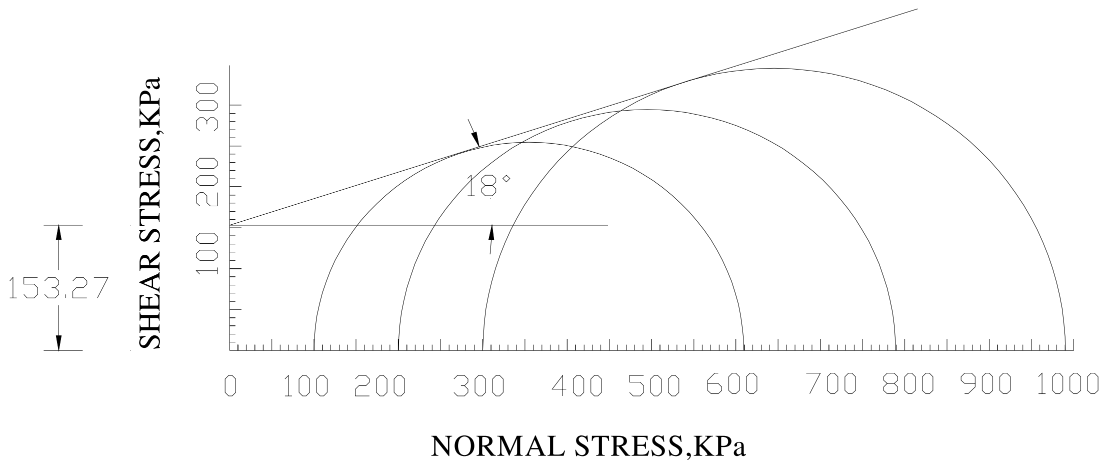

Fig 4. 13 a   Shear strength parameter for cell pressures of 100,200 & 300 KPa at 2.00 m depth at TP2

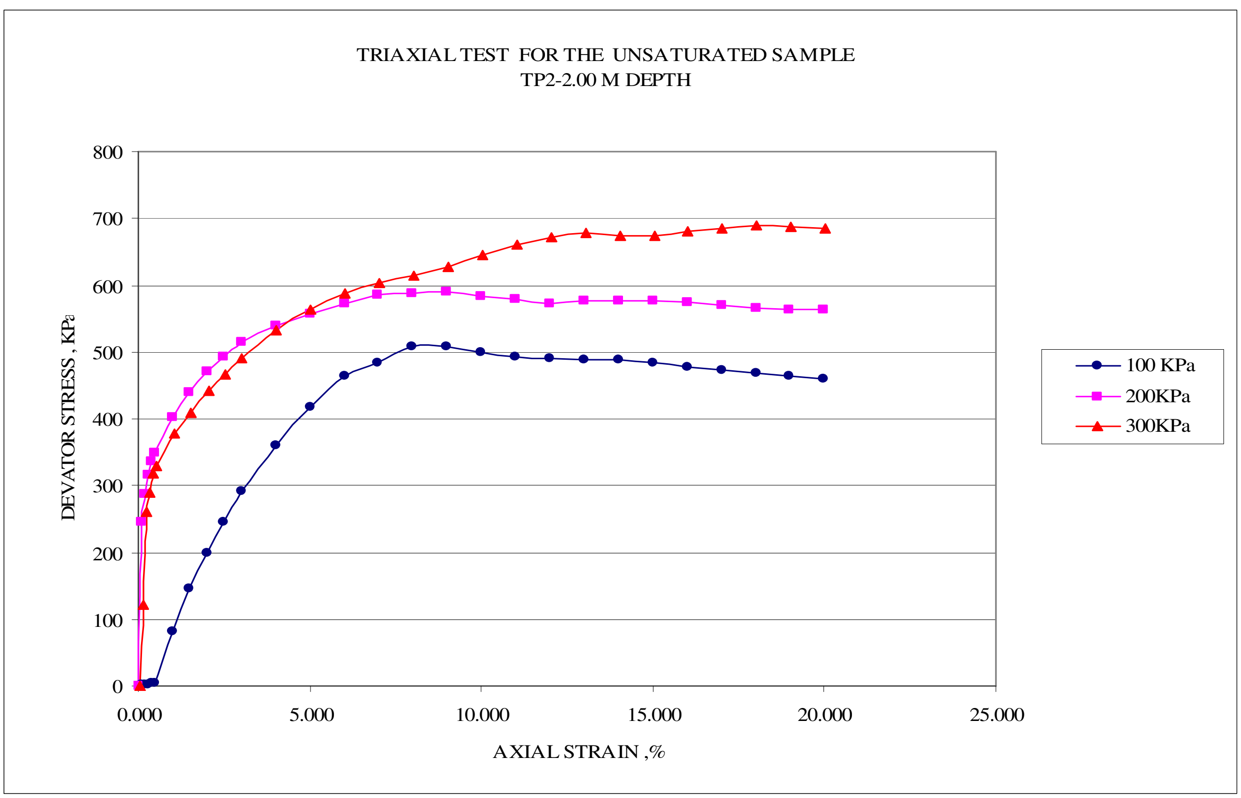

Fig 4. 13 b Deviator Stress Vs Strain at 2.00 m depth at TP2

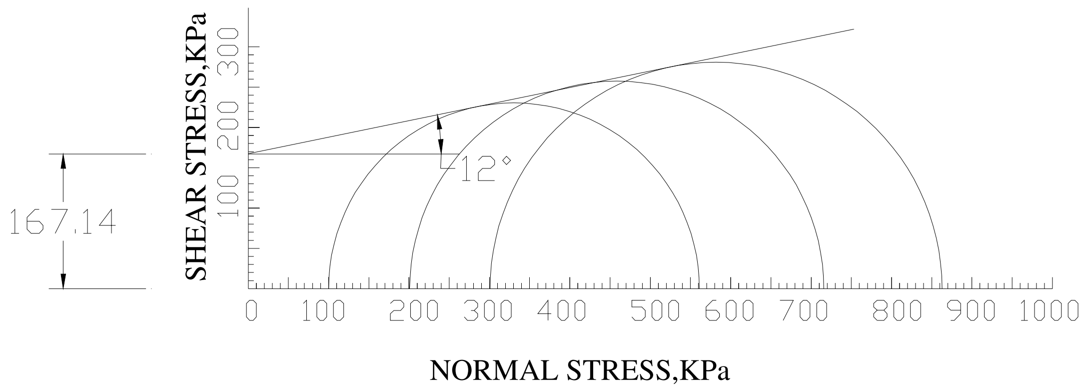

Fig 4.14 a Shear strength parameter for cell pressures of 100,200,300 KPa at 2.00 m depth at TP3

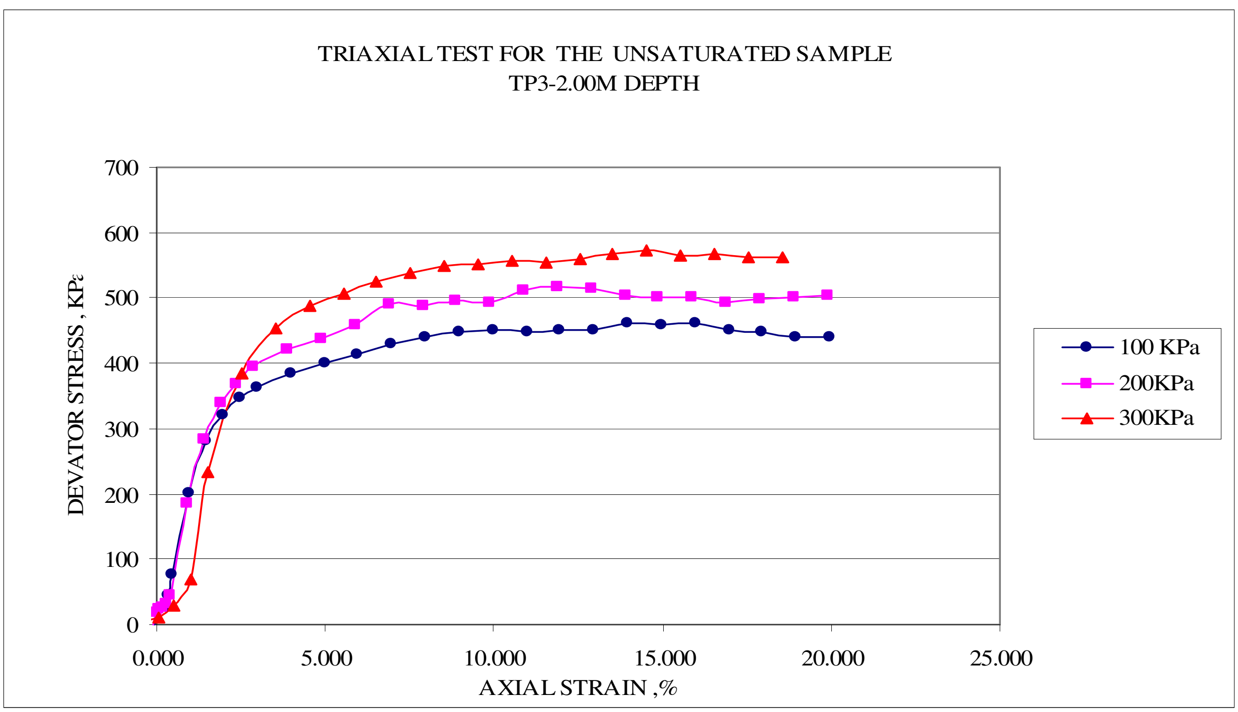

Fig 4. 14 b Deviator Stress Vs Strain at 2.00 m depth at TP3

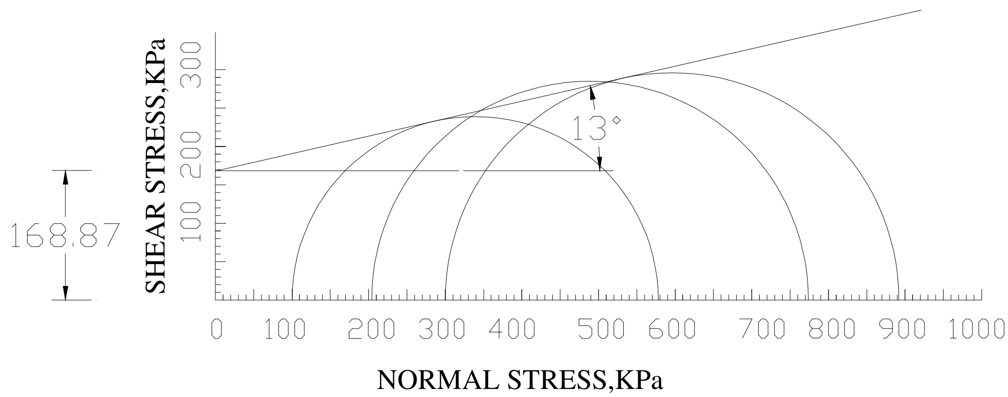

Fig 4. 15 a Shear strength parameter for cell pressures of 100,200,300 KPa at 2.00 m depth at TP4

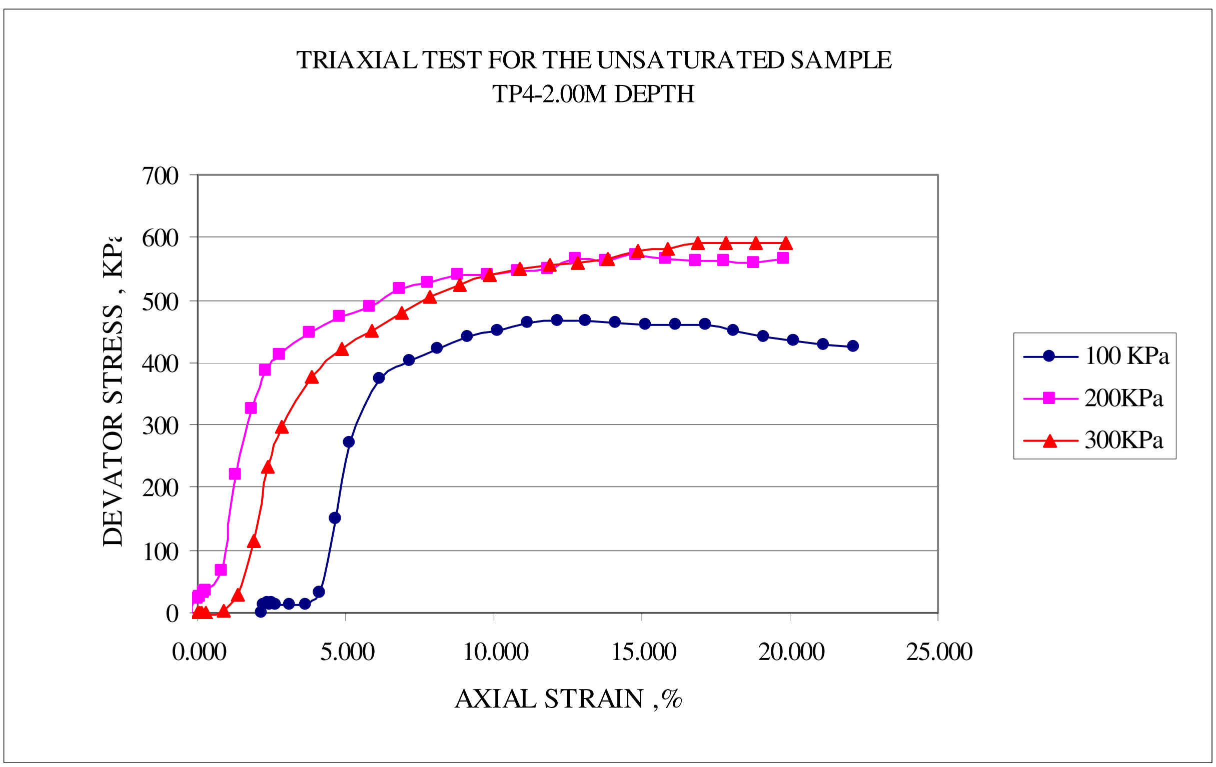

Fig 4.15 b Deviator Stress Vs Strain at 2.00 m depth at TP4

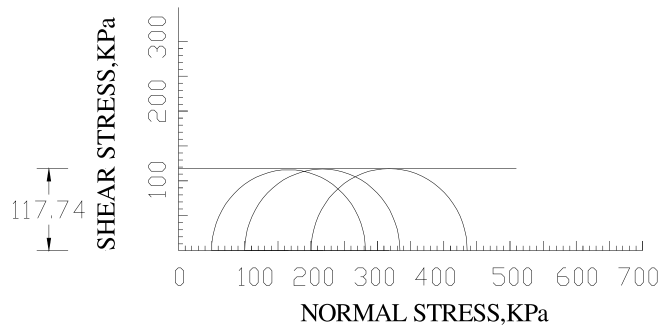

Fig 4. 16 a Shear strength parameter for cell pressures of 50,100&200 KPa at 2.00 m depth at TP4 saturated case

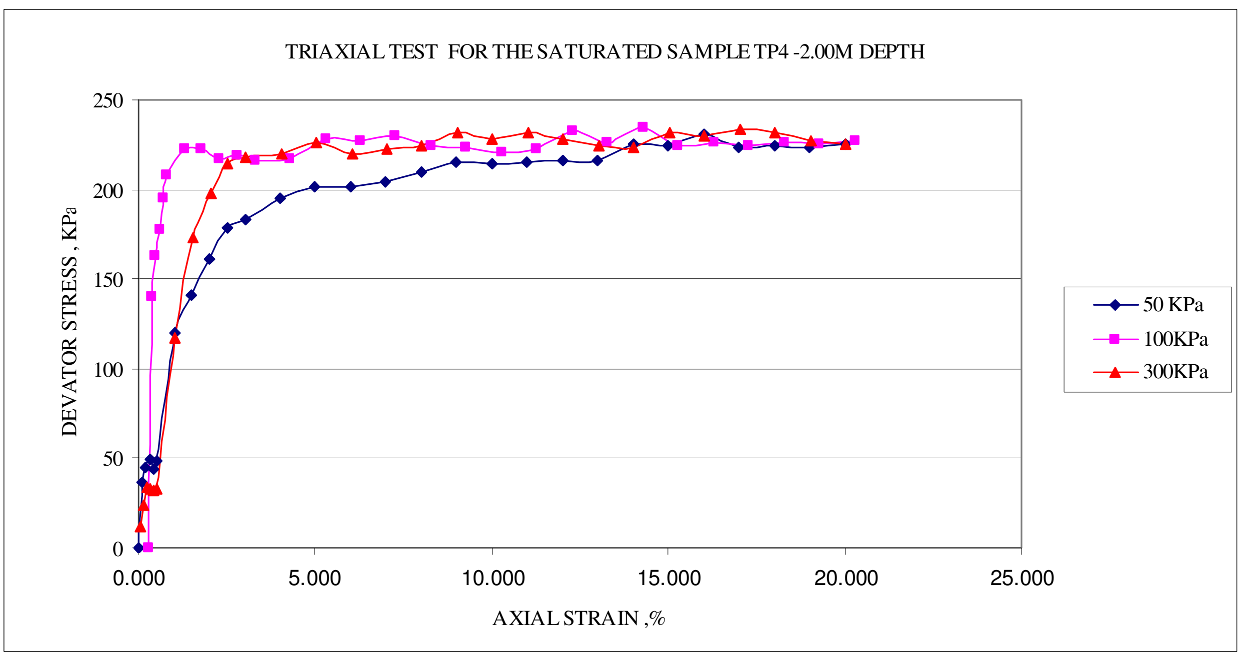

Fig 4.16 b Deviator Stress Vs Strain saturated case at 2.00 m depth at TP4

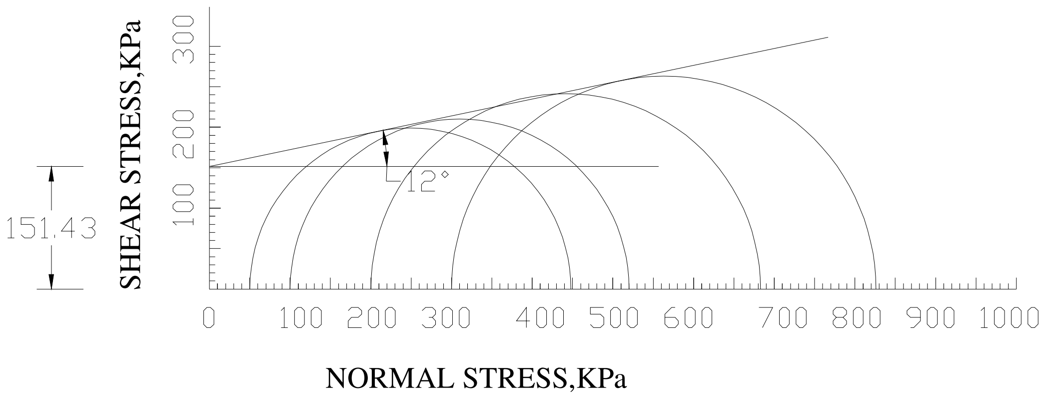

Fig 4. 17a Shear strength parameter for cell pressures of 50,100,200 & 300 KPa at 2.00 m depth at TP5

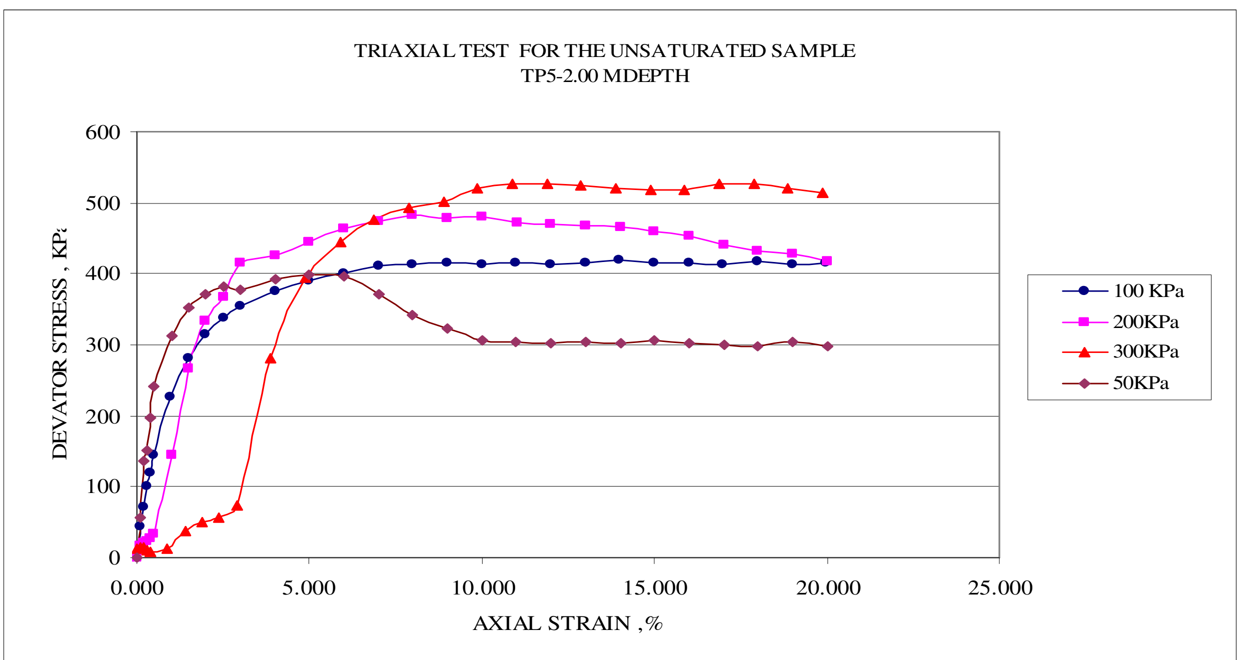

Fig 4. 17b Deviator Stress Vs Strain at 2.00 m depth at TP5

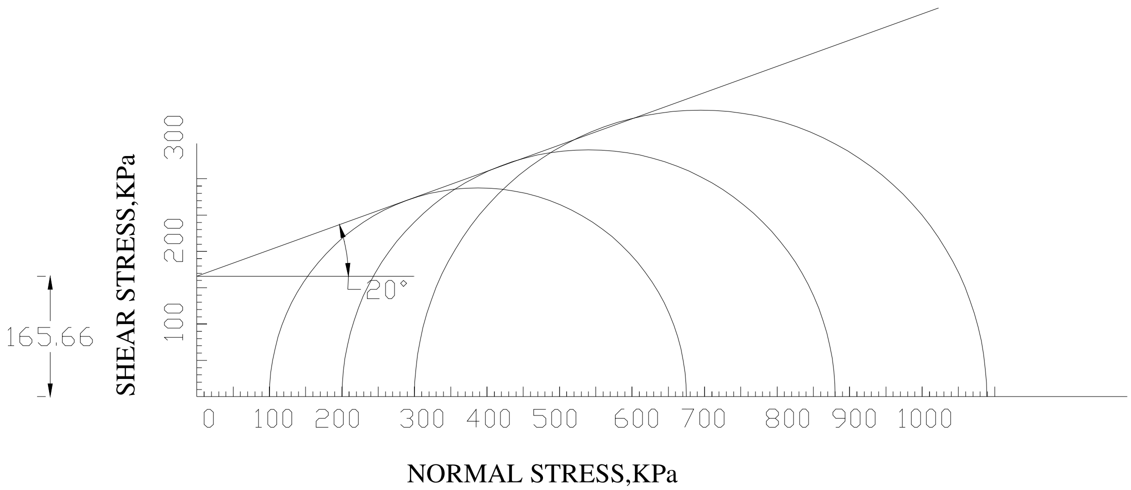

Fig 4. 18a Shear strength parameter for cell pressures of 100,200 & 300 KPa at 2.00 m depth at TP6

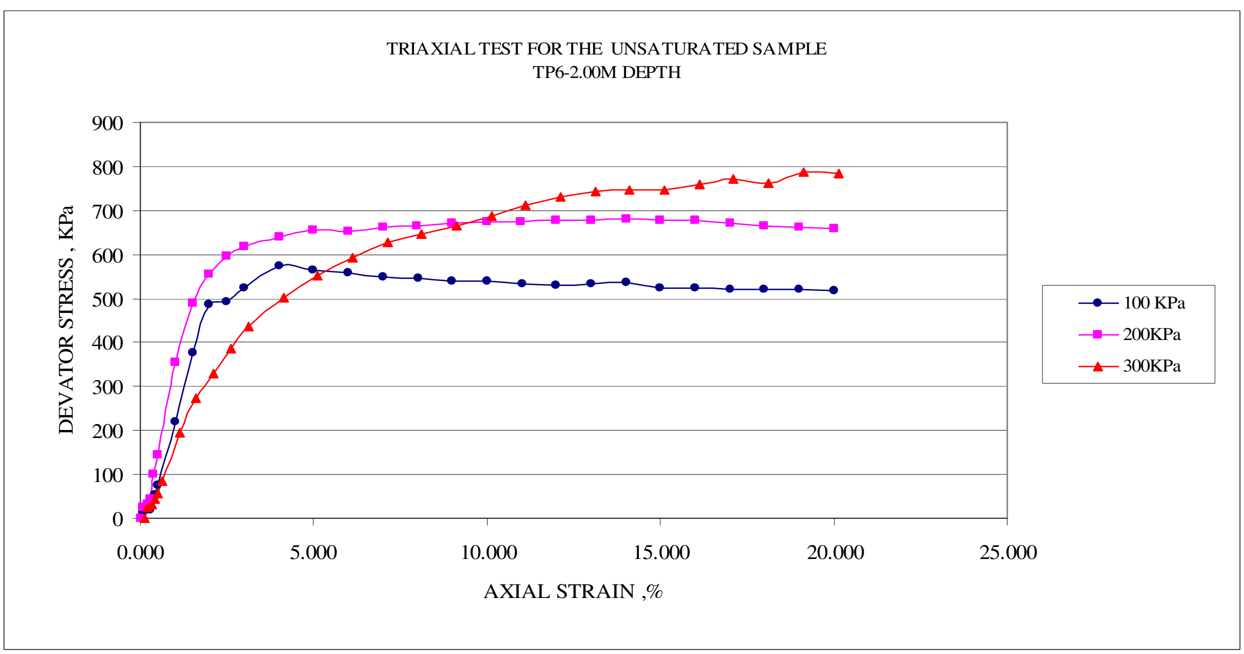

Fig 4. 18b Deviator Stress Vs Strain at 2.00 m depth at TP6

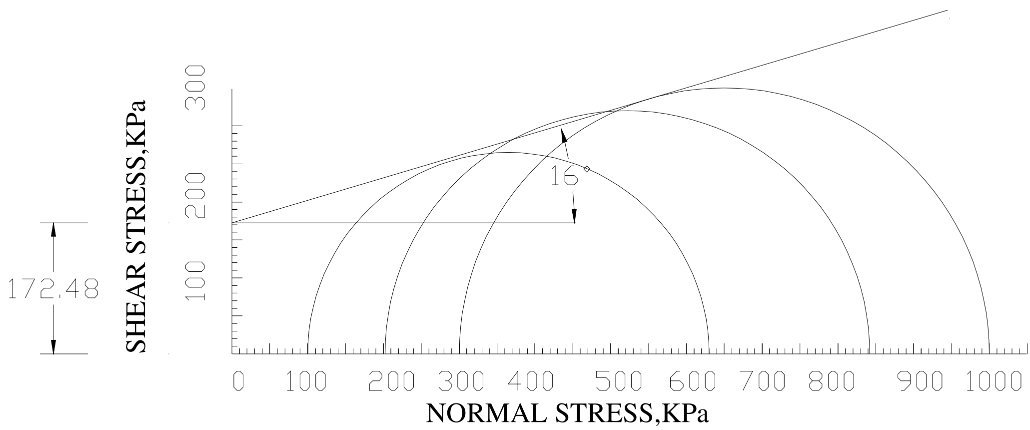

Fig 4. 19a Shear strength parameter for cell pressures of 100,200 & 300 KPa at 2.00 m depth at TP7

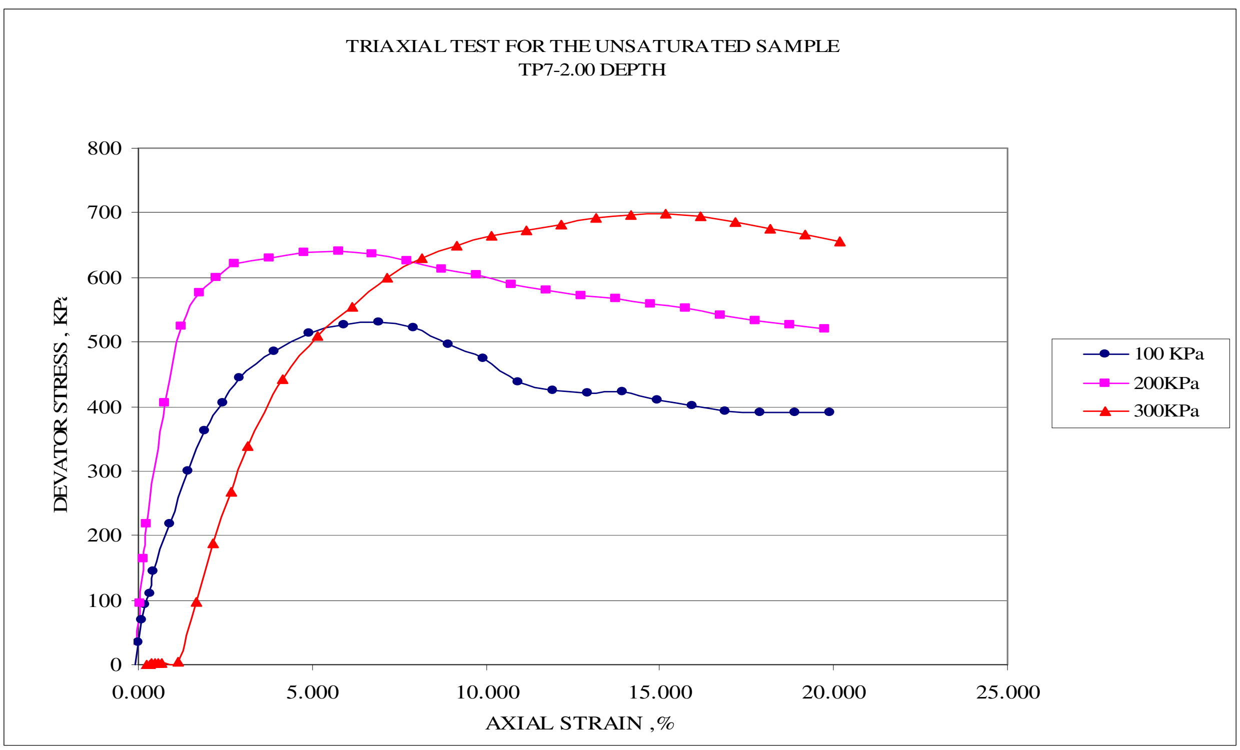

Fig 4. 19b Deviator Stress Vs Strain at 2.00 m depth atTP7

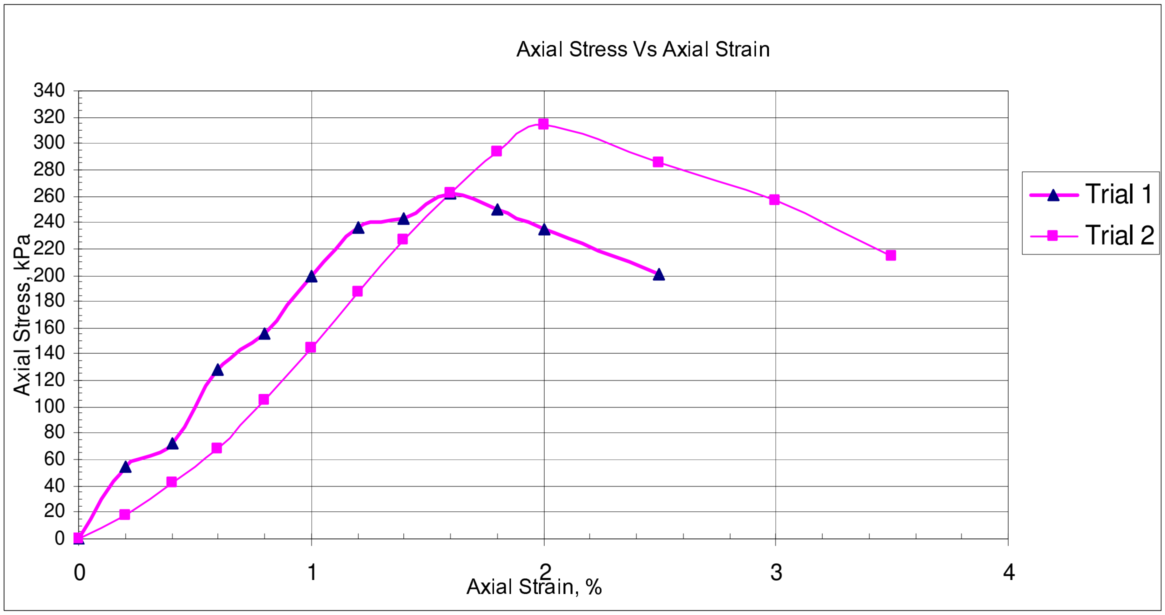

Fig 4. 20 Axial Stress Vs Strain at 2.00m depth at TP2 (UC Test)

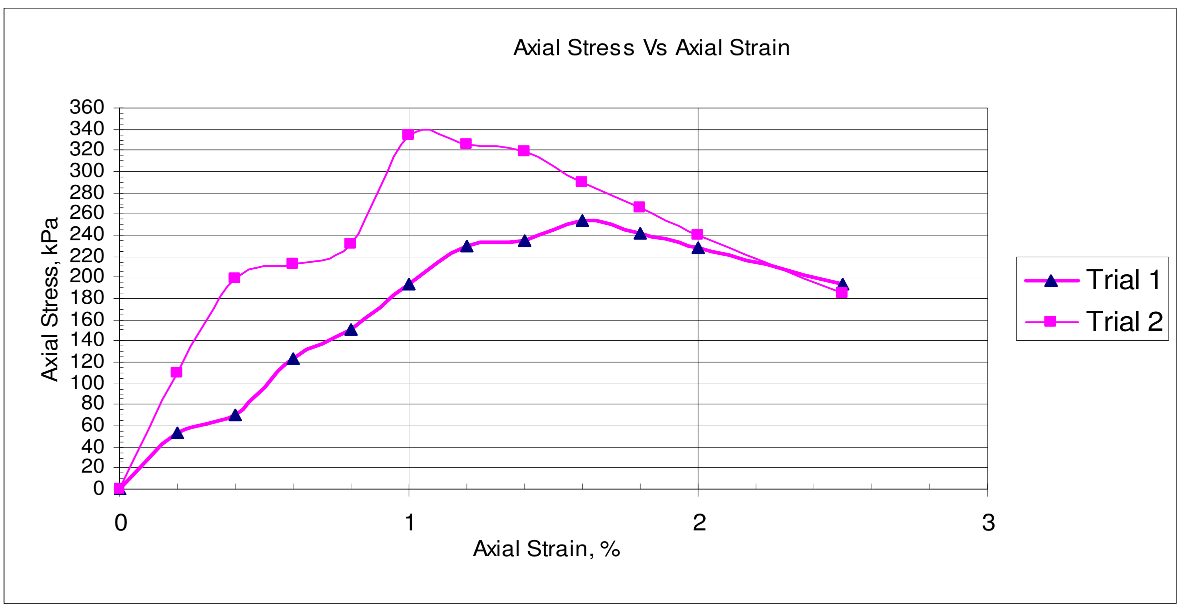

Fig 4. 21 Axial Stress Vs Strain at 2.00m depth at TP3 (UC Test)

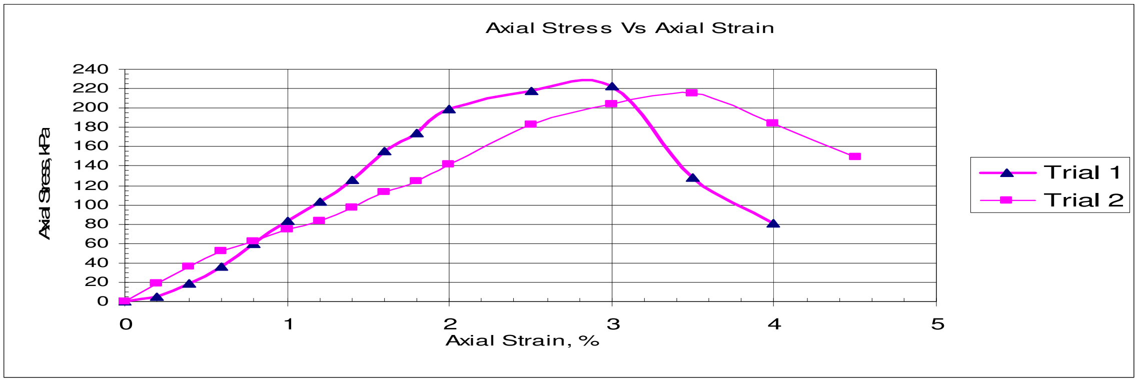

Fig 4. 22 Axial Stress Vs Strain at 2.00m depth at TP5 (UC Test)

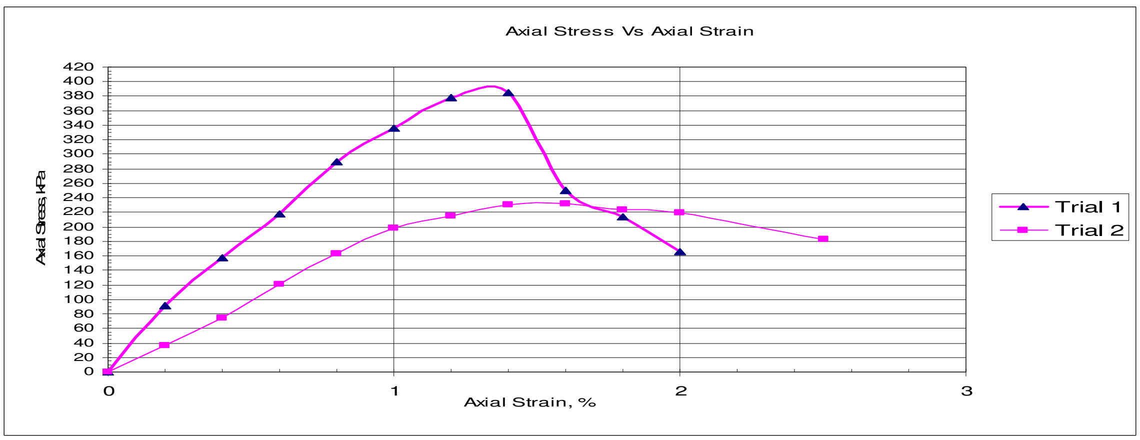

Fig 4. 23 Axial Stress Vs Strain at 2.00m depth at TP6 (UC Test)

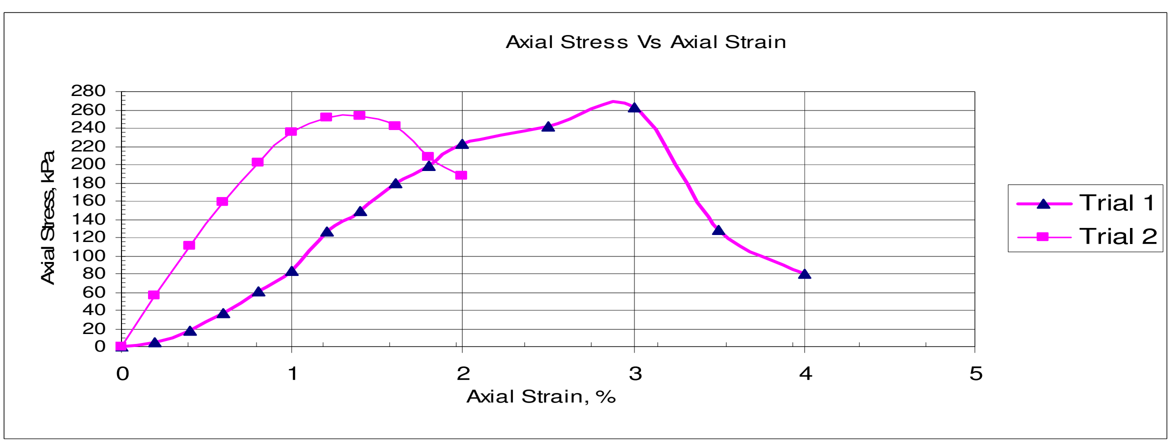

Fig 4. 24 Axial Stress Vs Strain at 2.00m depth at TP7 (UC Test)

5. Comparisons and Discussions of Test Results

5.1 Comparison of Test Results with Laterites , Lateritic Soils of Africa, Nejo-Mendi & Assosa

Laterites and Lateritic soils of Africa were studied by Morin W.J. and Todor P.C. (Lyon, 1971). During the study of the soils, samples were collected from different parts of Africa such as Ghana, Ethiopia, Kenya, Uganda etc. The characteristics and mineral content of the soil samples taken from Ethiopia was classified as Ferrisol.

For the soil under investigation; Index property were studied and a comparison was made with known laterites , lateritic soils of Africa,Nejo-Mendi and Asossa. The soil samples collected from Africa were classified according to D'Hoore classification methods in to Ferrugineous, Ferralitic and Ferrisol. The results of Index property tests, compaction tests and California Bearing Ratio (CBR) tests of Ferrugineous, Ferallitic, Ferrisol and laterite soil from Nejo- Mendi, Assosa and Wolayita-Sodo area are tabulated in Tables 5.1 to 5.6.

Tables 5.1 to 5.3 show that the average values of the various properties, i.e., liquid limit, plasticity index, gradation, compaction and CBR, indicate that the ferruginous soils, Ferrallitic soils and Ferrisol represent distinct soil groups with a characteristic range of properties.
Average soil test properties for the three sub group of African laterite and lateritic soils, Nejo- Mendi ,Asossa and Wolayita-Sodo soils are shown in Table 5.8.

Average soil test properties for the three sub group of African laterite and lateritic soils and Wolayita - Sodo soils are shown in Table 5.6. From section 2.1.2 Ferrisol soils occur in regions of between 1250 and 2750mm rainfall per year the soil under investigation are fall in this range of annual rain fall according to Table 3.3
Comparison of the gradation and Atterberg Limits values of Woalyita - Sodo soils are similar to Ferrisol soils of Ethiopia previously determined.

Table 5.1 Typical Soil Test Results for Ferruginous Soils (Lyon, 1971)

COUNTRY	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASHO	GI	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm	OMC	MDD	CBR %
Ghana	26	15	**11**	A-2-6	0						45	20	12	2.044	150
Senegal	39	20	**19**	A-2-7	0	95	91	68	46	33	27	20			
Upper V.	38	14	**24**	A-2-6	1	100	97	80	66	51	38	25	14	1.839	35
Niger	21	11	**10**	A-2-4	0	97	89	73	64	55	40	25	9	2.107	22
Tanzania	34	19	**15**	A-2-6	6	100	100	100	100	100	93	61			
Kenya	45	31	**14**	A-2-7	0	100	100	94	88	52	40	28	19		52
Uganda	38	17	**22**	A-2-6	2	100	100	96	83	61	51	34	13		19
Sudan	21	12	**9**	A-2-4	0	100	100	100	100	98	57	27			
Gambia	36	16	**20**	A-2-6	0	98	77	53	42	34	28	22			

Table 5.2 Typical Soil Test Results for Ferrallitic Soils (Lyon, 1971)

COUNTRY	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASHO	GI	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm	OMC	MDD	CBR %
Ghana	38	18	**20**	A-6	6	100	100	100	95	81	67	47	14	1.886	15
Liberia	56	29	**27**	A-2-7	2	100	95	72	57	41	36	27			
Gabon	35	18	**17**	A-2-4	0										
Sierra Leone	55	31	**24**	A-2-6	1	100	98	90	68	37	29	27			

Burundi	31	16	**16**	A-6		100	100	92	84	76	74	70			
Dahomey	45	21	**24**	A-2-7	4	100	100	99	85	72	55	39			
Ivory Cost	62	31	**31**	A-7-6	22	100	100	100	100	99	88	69	19	1.698	
Mali	35	21	**14**	A-6	3	100	100	89	67	55	51	40	16	1.824	
Uganda	39	19	**20**	A-6	2	100	100	91	82	73	53	38			

Table 5.3 Typical Soil Test Results for Ferrisol Soils (Lyon, 1971)

COUNTRY	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASHO	GI	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm	OMC	MDD	CBR %
Ghana	53	34	**19**	A-7-5	3						50	37	17	1.745	45
Niger	28	16	**12**	A-2-6	0	100	100	98	82	68	39	26			
Ivory Cost	48	24	**24**	A-7-6	18	100	99	83	65	60	50	40	17	1.729	12
Mali	55	31	**24**	A-7-5	3	100	100	89	61	51	43	38	15	1.886	9
Uganda	46	21	**25**	A-2-7	0	100	100	91	56	31	24	20	14		16
Kenya				A-7-5	27	100	100	100	100	99	98	91			
Cameron	65	37	**27**	A-7-5	19	100	100	100	98	97	88	66			
Ethiopia	68	33	**35**	A-7-5	19	100	100	98	84	63	62	58	28	1.509	12
Ghana	57	25	**32**	A-7-6	12	100	100				65	50	19	1.714	15

Table 5.4 Soil test results for Nejo-Mendi soils (Zelalem, 2005)

Designation	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	AASHTO	GI	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm	MDD	OMC	CBR %
Sp-1	56	36	20	A-2-7	0	88	77	59	43	26	19	13	24	1.68	64
Sp-2-1	56	38	18	A-2-7	0	97	84	49	32	21	17	14	26	1.538	45
Sp-2-2	59	37	22	A-2-7	0	94	90	68	52	34	28	24			
Sp-2-3	67	43	24	A-2-7	0	96	90	75	57	37	31	26			
Sp-3-1	54	34	20	A-2-7	0	83	69	46	33	22	17	15			
Sp-3-2	54	34	20	A-2-7	0	92	83	67	55	43	35	30			
Sg-1	59	39	20	A-7-5	18					100	85	76	24	1.511	37
Sg-2	54	34	20	A-7-5	17					100	90	80			

Table 5.5 Typical Soil Test Results for Asossa soils (Wakuma, 2007)

Designation	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	AASHO	GI	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm
TP1-1	57	22	35	A-7-6	35	100	100	100	100	100	92	90.42
TP1-2	54	23	31	A-2-7	0	100	100	85.34	67.8	45.5	18.4	12.52
TP2-1	57	22	35	A-7-6	35	100	100	100	100	100	92	91.638

TP2-2	58	26	32	A-7-6	32	100	100	100	100	100	90.5	88.2
TP3-1	55	24	31	A-7-6	33	100	100	100	100	100	96	94.5
TP3-2	48	20	28	A-7-5	0	100	100	96.4	73.13	46.54	18.4	13.71
TP4-1	57	27	30	A-7-6	32	100	100	100	100	100	94.4	93.04
TP4-2	52	20	32	A-7-6	31	100	100	100	100	100	93	90.458
TP5-1	68	22	46	A-7-6	44	100	100	100	100	100	90.6	87.550
TP5-2	67	21	46	A-7-6	44	100	100	100	100	100	89.5	87.4
TP6-1	50	25	25	A-7-6	26	100	100	100	100	100	94.3	92.1
TP6-2	52	21	31	A-7-6	30	100	100	100	100	100	92.3	90.7
TP7-1	58	24	34	A-7-6	35	100	100	100	100	100	94	92.9
TP7-2	56	25	31	A-7-6	32	100	100	100	100	100	93.5	92
TP7-3	55	25	30	A-7-6	31	100	100	100	100	100	94	92.5
TP8-1	60	22	38	A-7-6	40	100	100	100	100	100	95.2	94.19
TP8-2	57	23	34	A-7-6	36	100	100	100	100	100	94.6	93.81
TP9-1	69	31	38	A-2-7	0	100	100	100	100	85.42	20.95	14.10
TP9-2	72	40	32	A-2-7	0	100	100	100	100	86.06	17.43	13.06
TP9-3	57	25	32	A-2-7	0	100	100	100	100	79.6	25.99	18.44

Table 5.6 Soil Test Results for Wolayita-Sodo soils

Test pit	Depth	Test cond.	LL (%)	PI (%)	Group Index	AASHO	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm	0.002 mm
TP1	2.0	oven	52	19	20max	A-7-5	100	100	100	100	100	95.09	86.22	49
		AD	57	22	20max	A-7-5	100	100	100	100	100	94.04	88.84	56.3
		AS	65	25	20max	A-7-5	100	100	100	100	100	94.06	87.09	58.3
TP4	2.00	oven	57	21	20max	A-7-5	100	100	100	100	100	90.91	84.64	53.9
		air	63	21	20max	A-7-5	100	100	100	100	100	94.40	89.51	66.3
Tp7	2.00	oven	54	23	20max	A-7-5	100	100	100	100	100	99.38	93.83	48
		AD	59	23	20max	A-7-5	100	100	100	100	100	96.65	92.03	62.9
		AS	71	30	20max	A-7-5	100	100	100	100	100	96.96	90.54	61.5

Table 5.7 Dominant mineral contents for laterite sub group (Lyon, 1971)

Ferruginuous	Ferralitic	Ferrisol
Hematite Geothite kaolinite	Gibbsite Geothite kaolinite	kaolinite Geothite Hhematite Gibbsite

Table 5.8 comparison of average soil properties for this and previous studies

Test Executed	Ferruginuous		Ferralitic		Ferrisols		Nejo - Mendi (Zelalem,2005)	Asossa (Wakuma, 2007)	Wolayita Sodo
	Ghana	Other Countries	Ghana	Other Countries	Ghana	Other Countries			
Passinig sieve size (%)									
25 mm	99	99	99	99	95	99	94.6	100	100
19 mm	98	98	96	97	94	99	88.3	100	100
9.5 mm	93	89	86	90	86	92	73.3	99	100
4.75 mm	75	76	70	80	73	74	62.6	94.6	100
2.00 mm	51	65	54	70	52	61	51.6	89	100
0.425 mm	46	51	46	54	40	51	45.0	74.9	95
0.075 mm	30	32	34	40	37	44	38.3	72	89
0.002 mm	13	16	19	26	25	24	9.2	21	57
LL	31	33	42	47	46	55	59	59	60
PL	18	12	24	24	23	29	38	25	37
PI	14	15	19	23	23	27	22	34	23
OMC	10	13	12	13	14	21	24		
MDD	2.091	1.949	2.028	1.839	1.918	1.698	1.552		
CBR	75	33	46	24	42	14	38		

5.2 Comparison of Test Results with previously done research work

Table 5.9 Comparison of Test Results with pervious done research work

	Morin & Parry	Previous research (Haile Mariam,1992)	Previous research (Abgena,2003)	Previous research (Zelalem,2005)	Previous research (Wakuma, 2007)	Current research
Soil Type	Red clay	Red clay	Red clay	Latertic	Latertic	Latertic
Location	Ethiopia	Addis Ababa	Bahir Dar	Nejo-Mendi	Asossa	Wolyita-Sodo
Clay content %	34-76	48-73	74- 82	2.0-20.6	2.5- 60	48- 69.7
Activity			0.56	0.97- 0.98	0.62- 1.02	0.317- 0.488
Clay minerals	Kaolinite, Halloysite, Montmorinite		Kaolinite			Kaolinite
Liquid Limit,%	44-66	54-81	61-68	48-67	41-72	48-71
Plasticity Index,%	14-30	21-30	24.31	17-27	20-48	19-30
Shrinkage limit,%	10-30	14-22	9-12	7.1- 15.7		11-22
Free swell		10-40		20-40	11-45	28-38
Specific gravity	2.61-2.90	2.61-2.79	2.75-2.83	2.78-3.03	2.19-2.94	2.61-2.97

From plasticity chart				(MH)	CH,SC,MH, CL&SM	(MH)
Unconfined Compression strength (qu) KN/m^2	146.5-251	49-250	148-220	165-553		215-385

From comparison table almost all the test result ranges in the previously done thesis work.

6. Conclusions and Recommendations

6.1 Conclusions

Based on the test results investigation on the soil samples of Wolayita Sodo area the following conclusions can be drawn:

1 Geochemical tests indicate that the soils of Wolayita Sodo area are laterites having high concentration of Iron Oxide and Aluminium Oxide / Sesqueoxide /. The degree of laterization, Silica to Sesqueoxide ratio, is below 1.33 except TP2. No significant variations have been seen in different pits located at far distances with corresponding lithological classification category.

2 X- ray diffraction tests indicate that the soils of Wolayita Sodo area have clay mineral kaolinite.

3 The soils under consideration contain a significant amount of loose molecular water for some soil sample. One can check the presence of molecular water to determine amount of water content.

4 The Wolayita Sodo area soils indicate sensitive to test procedures which are affected during testing manipulation. Hence desegregation results in the test value different from the in-situ condition. Accordingly, the minimum mixing time, usually of 5minutes and fresh soil has to be used for each point on the Atterberg Limit.

5 The specific gravity test results are higher values than the temperate zone soil. This is due to high amount of iron oxide for soil under investigation.

6 From the gradation charts and the soil classifications made, Wolayita Sodo soil is a fine-grained soil with characteristics some how similar to Ferrisol soils. From the mode of

formation and comparisons of gradation charts, Wolayita Sodo soil could be grouped as Ferrisol soil.

7 Laterite soils are characterized by high concentration of Iron Oxide, Aluminium Oxide (Sesqueoxide) and Kaolinite minerals. The soil samples subjected to test fall below A-line under MH (inorganic clay with medium strength) and contain Kaolinite mineral.

8 The activity number, which is the ratio of plasticity index to the percent of the clay-size fraction by weight, is below 0.75, confirming that the soil is inactive.

9 The shear strength parameters which was obtained from UU test C_u value ranges from 150-173Kpa for unsaturated case and 118 Kpa for saturated case for TP4 at 2.00m depth Φ value ranges from 12-20º ; from UCT test q_u value ranges from 215-385 Kpa.

6.2 Recommendations

1 Further detailed investigation has to be carried out by using higher cell pressure until the matric suction measuring Triaxial machine.

2 This study should be extending the research by considering the stress history of the sample through serious of consolidation test.

3 Detailed profile investigation has to be carried out on soil samples on a localized area.

4 Cost benefit analysis should also be made to use the soils as construction materials for heavy traffic roads as compared with crushing of basaltic rocks or other alternatives.

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Appendix – A

Grain size distribution curves under different testing conditions

The graphs in this appendix show the grain size distribution curves for various soils and test conditions. Figs A.1, A.2, A.2….& A.6… was plotted for the soil samples treated at different testing temperature.

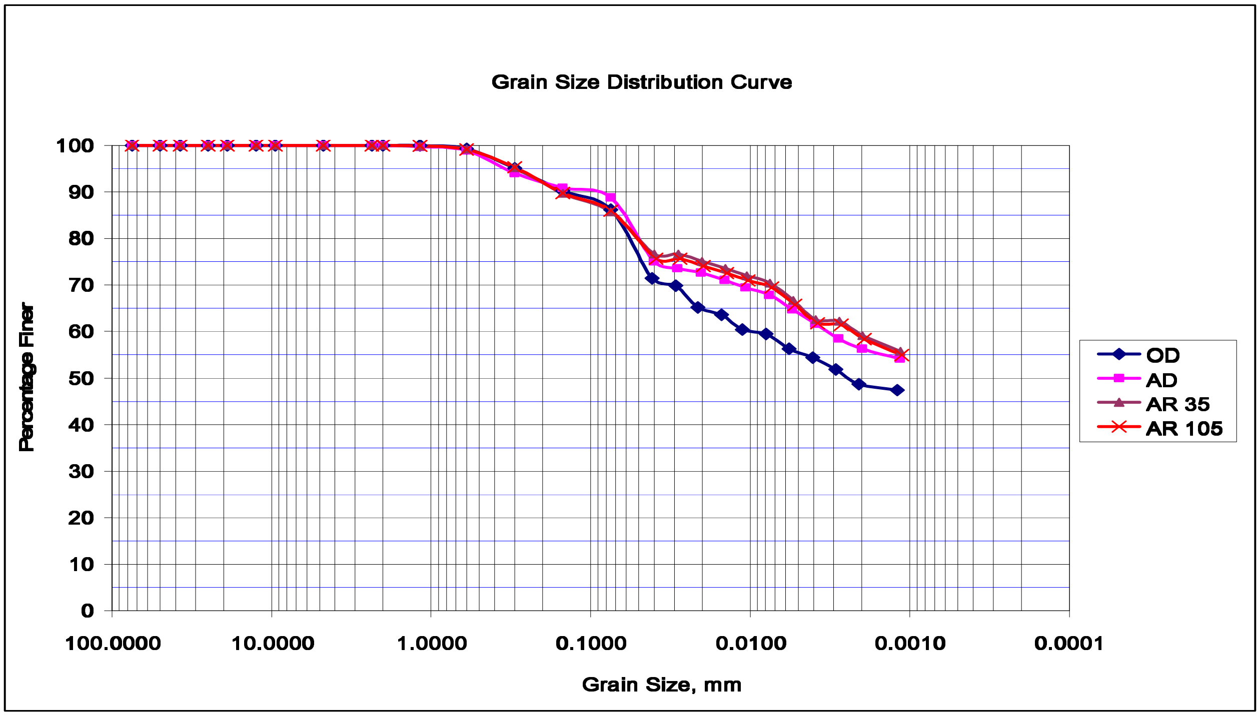

Fig A.1 TP1 2.00m depth Grain size distribution curve

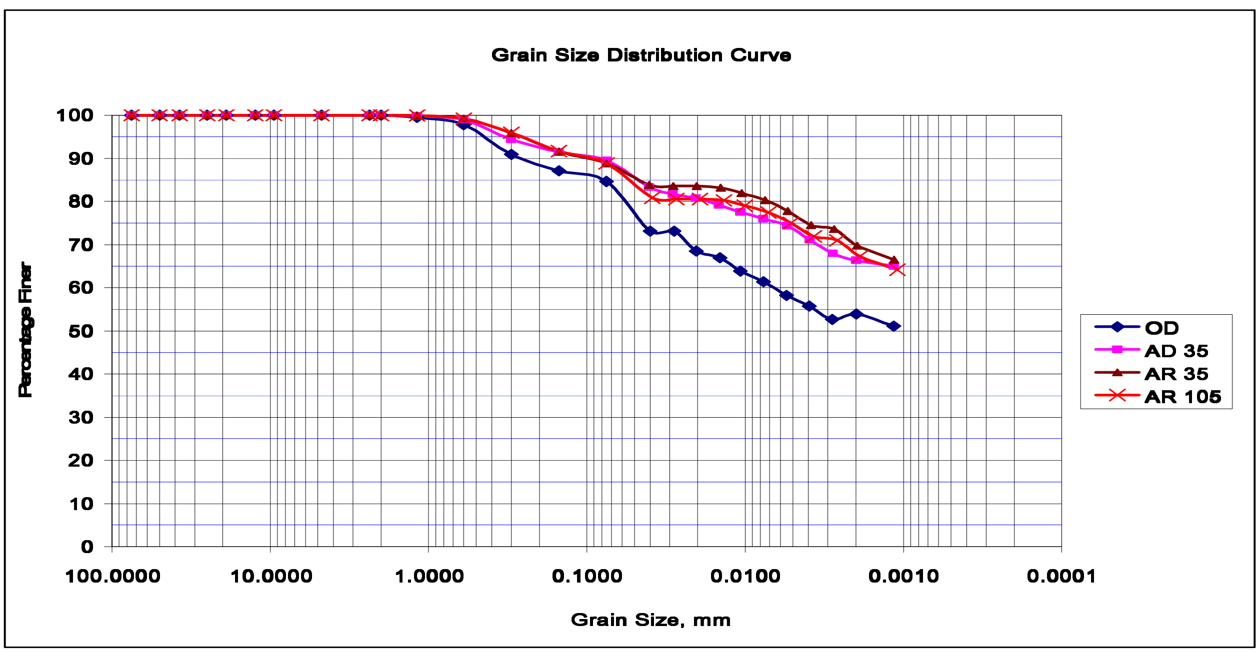

Fig A.2 TP4 2.00m depth Grain size distribution curve

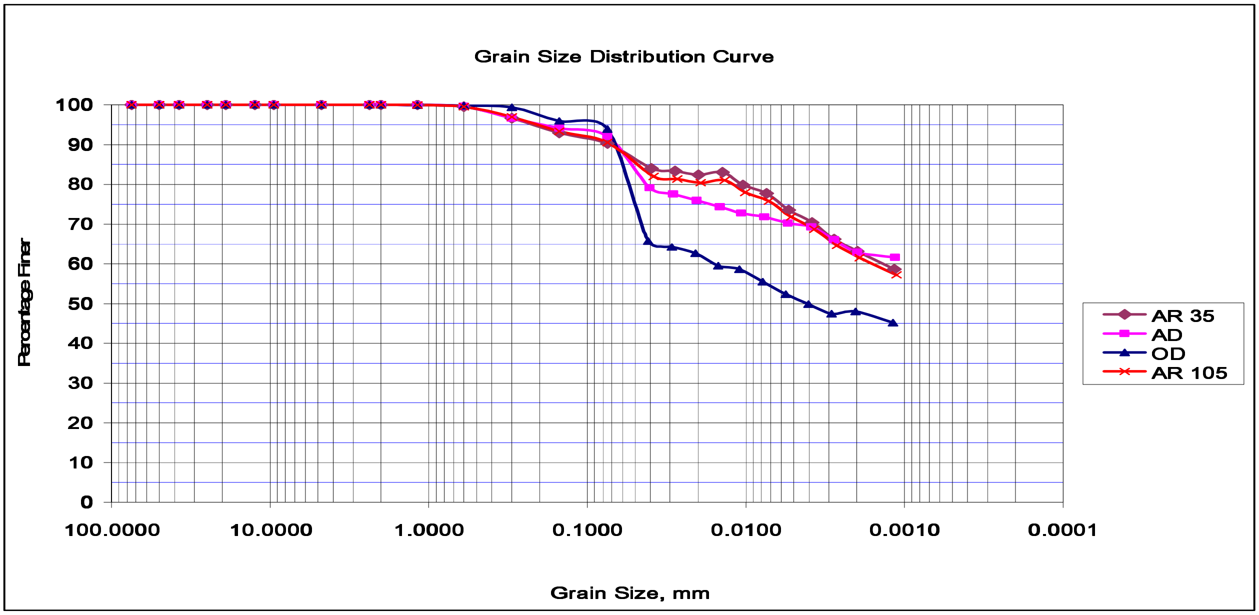

Fig A.3 TP7 2.00m depth Grain size distribution curve

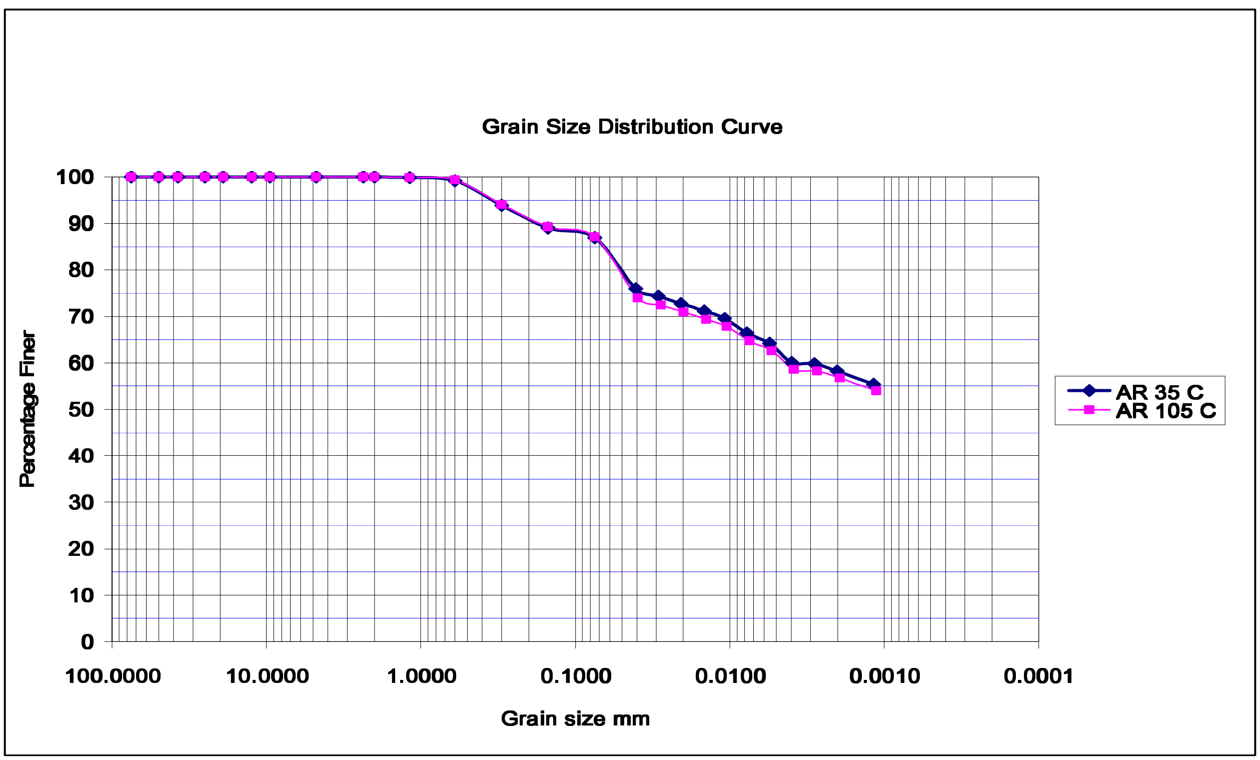

Fig A.4 TP1 1.50m depth Grain size distribution curve

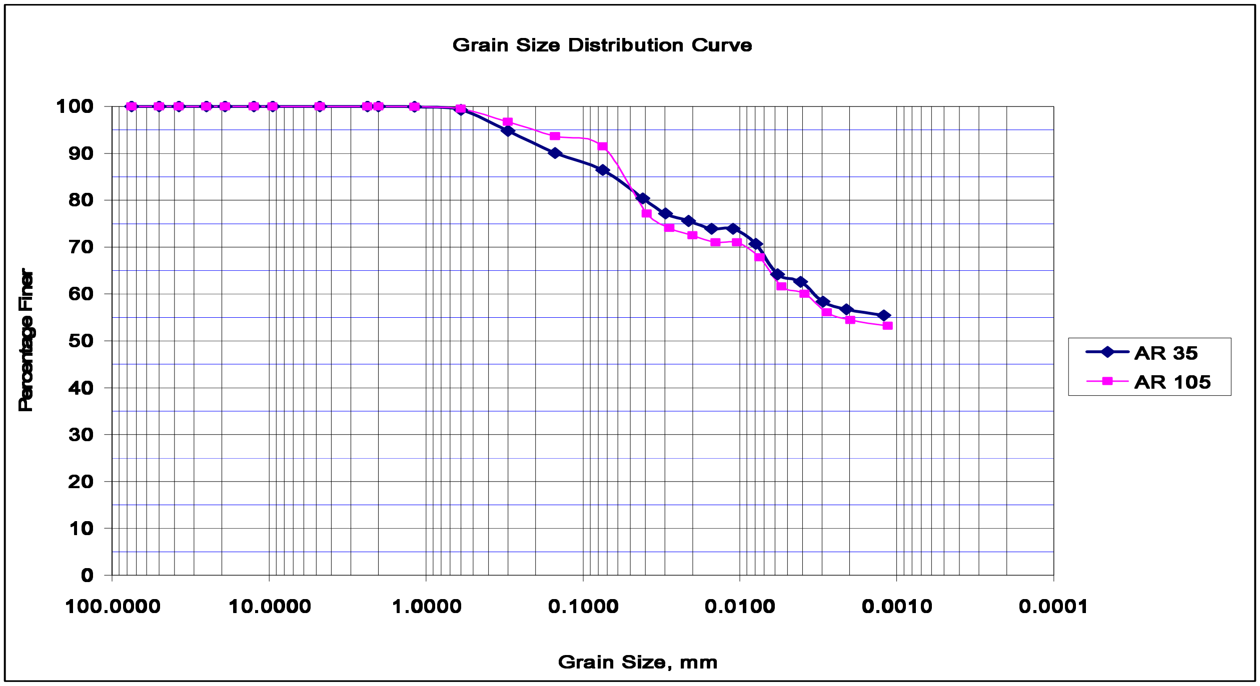

Fig A.5 TP2 2.00m depth Grain size distribution curve

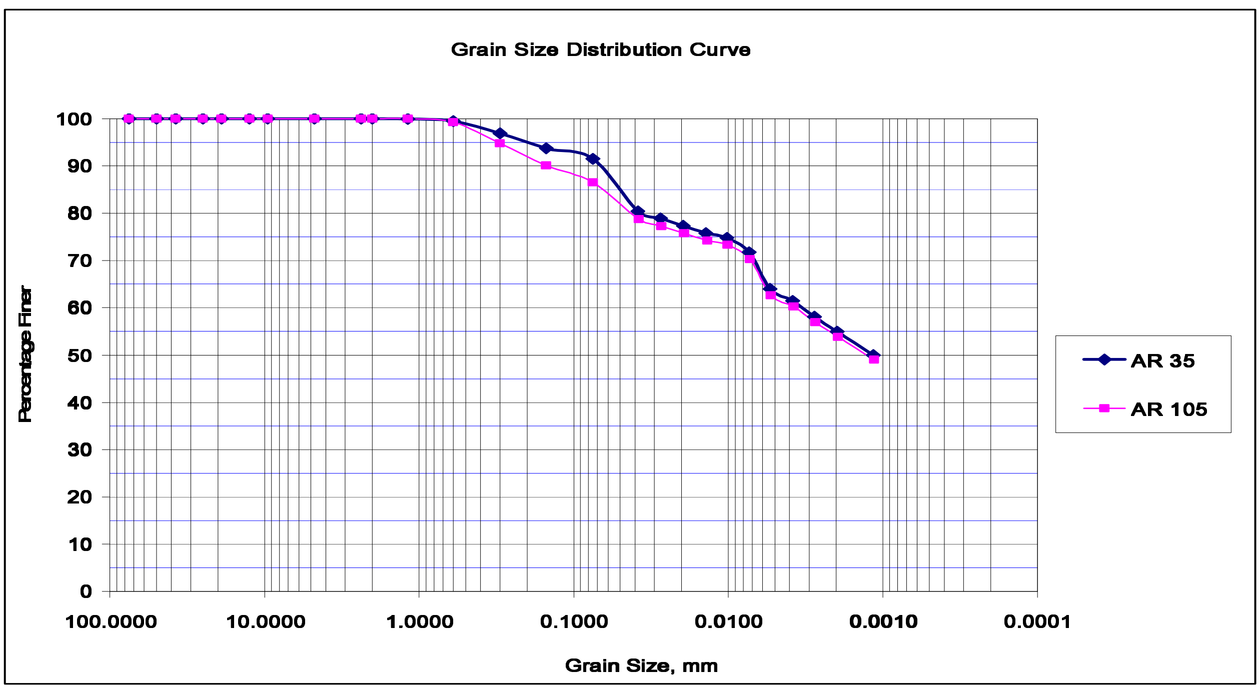

Fig A.6 TP6 2.00m depth Grain size distribution curve

Appendix – B

Test Result of Geo chemical and X-ray diffraction