

ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY



Investigation into Some of the Engineering Properties of Lateritic Soils in Southern
Part of Ethiopia Case Study of Dilla Town.

By
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Sep, 2015.

Investigation into Some of the Engineering Properties of Lateritic Soils in Southern
Part of Ethiopia Case Study of Dilla Town.

A thesis submitted to the school of graduate studies of Addis
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Masters of Science in Civil Engineering (Geotechnical Engineering).

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor **Dr. Messele 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
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	Plastic index	%
FS	Free swell	%
W	Moisture content	%
N	No. of blows for liquid limit	---
s_g	Specific gravity	---
NMC	Natural moisture content	%
AD	Air drying	---
OD	Oven drying at a temperature of 105 °c	---
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	---
RH	Relative humidity	%
GI	Group Index	---

MDD	Maximum dry density for compaction test	g/cm ³
CBR	California Bearing Ratio	%
OMC	Optimum moisture content for compaction test	%
LS	Linear shrinkage	%

ABSTRACT

In this thesis work, some of the geotechnical and geo-chemical characteristics have been investigated on soils collected from Dilla town. From the externally visible physical characteristics the soil samples under investigation was assumed to be lateritic soil. Moisture content determination using oven temperature of 105°C and oven temperature of 50°C with maximum relative humidity 30% were also carried out on the soil samples to investigate presence of loosely bound water of hydration. It was obtained that the soil samples did not contain loosely bound molecular water in a significant amount.

The soil specimens were tested at different sample preparation conditions prior to testing. The engineering properties investigated in the course of the research work include index tests temperature of 105°C dried, oven temperature of 50°C with maximum relative humidity 30% on the soil samples. In addition to the above consolidation, unconfined compression test at undisturbed states and remold samples are conducted.

Atterberg limits were investigated by different testing procedures to see effect of test manipulation on concretionary bond. The liquid limit tests were carried out on soil specimens mixed for 5 minutes and 30 minutes durations. It was observed from the test results that the mixing durations has significant effect on the values of the limits. Accordingly, the soil samples have been sensitive to test procedures.

An activity test result for soil samples is found to be 0.38-0.65 which is less than 0.75 inactive soils. Lateritic soils are inactive or normal due to the fact that Sesquioxides suppress the activity of the clay particles.

From Unconfined Compression Test, unconfined compression strength (q_u) values ranges from 110.81-383.48 kpa. The sensitivity test result showed that the soil samples are insensitive and little sensitive clay under remolding.

The results of consolidation test on the soils indicates that the soils is over consolidated .The over consolidation phenomena on geo-chemical test result show that the residual soil is oftennot due to the previous history ,but mainly due to the cementation.

CHAPTER ONE

1. Introduction

1.1 General

Dilla is the administrative center of the Gedeo zone in the Southern Nations, Nationalities, and Peoples Region (SNNPR). It is located at 360km from Addis Ababa along the main road from Addis Ababa to Nairobi. It has been covered predominantly with reddish soils which resemble lateritic or undergoing laterization. The red color seems to have been accepted as most important property by which these soils could be visually identified. The red color of a residual tropical soil in most cases comes from the presence of the iron oxides mineral in the soil.

The geotechnical properties of lateritic soils are influenced by climate, drainage, geology, the nature of the parent rock and the degree of weathering or linearization of the parent rock. These factors also differentiate laterite from other soils that are developed in the temperate or cold regions. Some lateritic soils are thought to have been transported from their place of origin by wind or other actions, but most of those soils are likely formed in-situ.

Laterites occur mostly in tropical and sub-tropical regions with hot, humid climatic conditions. It has been suggested that a mean annual temperature of around 25°C is required for their formation, and in seasonal situations there should be a coincidence of the warm and wet periods. The minimum annual rainfall required for laterite formation is generally at least 750 mm (CIRIA, 1995). The higher the rainfall above this value, the greater is the leaching effect and therefore increases the degree of laterization (Zelalem. A, 2005) This weathering process primarily involves the continuous chemical alteration of minerals, the release of iron and aluminum oxides, and the removal of bases and alkaline

Lateritic soils contribute to the general economy of the tropical and subtropical regions where they are in abundance because, they are widely utilized in civil engineering works as construction materials for roads, houses, landfill for foundations, embankment dams, etc. As a road construction material, they form the sub-grade of most tropical road, and can also be used as sub-base and base courses for roads that carry light traffic.

1.2 Back ground of the problem

There is no previously done research around Dilla area. thus the work gives us understanding about the behavior of the soil in the area. 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.

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:

- Check whether the soil of Dilla area is lateritic soils or not by conducting index and geo-chemical tests.
- Investigate the effect of temperature variations, pre-treatment conditions and testing procedures, on the behavior of the soil.
- To recommend the appropriate test procedures for Dilla soil and other tropical soils with similar geological formation.
- To determine the consolidation characteristic of soils in the city.
- To prepare tentative soil map of the city.

1.4 Methodology

Specimens were taken at 20 different locations from ten test pits. 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.

Effect of temperature variations on moisture content determination have been checked in the laboratory using different drying oven temperatures. The difference of the results helpsto choose which drying methods to follow of this research work.

This thesis work consists of different sample pre-treatment conditions for all index property determinations. These methods were air-drying (AD), oven drying (OD).

1.5 Limitation of the study

The research is limited to the index property tests, UCT, consolidation tests taken from different locations of dilla town.

Mineralogy is the primary factor controlling the size, shape, and physical and chemical properties of soil mechanics. The most widely used technique to determine mineralogical composition is x-ray defraction (XRD) (Mitchell, 1979). It is worth mentioning that the need of mineralogical tests to be conducted. The XRD machine was not functioning at Geological Survey of Ethiopia during preparation of this thesis document. Hence it was not possible carrying out the test for this thesis work.

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 the introductory Chapter the background of the problem, objective and methodology, limitations 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. Chapter three deals with sampling areas description. The fourth Chapter deals with in-situ properties with sample description and the types of laboratory tests conducted and results obtained. The test results obtained from this work was compared with previously done test for red clay and Laterites soils. This is presented in Chapter five. Chapter six contains the conclusions and recommendations drawn from the research.

CHAPTER TWO

2. Literature Review

2.1 General

At the beginning of the 19th century, Laterites obtained scientific interest when the English surgeon Francis Buchanan travelled along the western coast of southern India and published his manifold observations and results Schellmann(2014). Laterite is a residual of rock decay that is red or reddish in color. Soils under this classification are characterized by forming hard, impenetrable and often irreversible pans when dried. However, there is confusion in the use of the term, because a variety of materials with many types of compositions and various origins have been called Laterites, ranging from iron cappings to the zonal soils of the humid tropics and the whole weathered profile beneath a laterite of Buchanan's meaning to the iron-rich breccias and slope wash accumulations. Due to this confusion, most researchers now prefer to use the definitions based on hardening, such as "Ferric" for iron-rich cemented crusts, "Alcrete" or Bauxite for Aluminum-rich cemented crusts, "Calcrete" for Calcium Carbonate-rich crusts, and "Silcrete" for silica rich cemented crusts.

Blight (1997) describes Laterites as highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical condition. The three major agents of weathering being physical, chemical and biological processes. In the process the parent rock and rock minerals break down, releasing internal energy and forming soils having a lower internal energy which are more stable. Physical processes increase surface area so that chemical attack increases. Biological weathering includes both physical and chemical actions. Rocks exposed at the surface of the Earth are subject to physical and chemical weathering. At the given sufficient time and a suitable climatic condition, (hot and wet is best for chemical weathering; cold and dry or wet is best for physical weathering) even the most resistant rock will be reduced to shadow of its former self.

2.2 Formation, Occurrence and Distribution

2.2.1 Formation and Occurrence

The principal effects of the various factors on Laterites formation are well known but it is difficult to determine them in space and time in the field.

In practice of Laterites research most valuable information's are obtained by detailed studies of complete weathering sections (laterite profiles) reaching from the un weathered parent rock to the strongly altered surface layer. Sections showing physical disturbances as erosion or importation of transported material should be omitted to exclude effects other than weathering.

An adequate number of laterite profiles on different parent rocks has been analyzed which enable a clear understanding of the basic processes of lateralization (Schellmann,2014).

The transformation of rock into laterite proceeds in general gradually as indicated by the steady increase of iron and decrease of silica in laterite profiles above the parent rock. It goes without saying that the initial products of weathering cannot be called laterites. They also form in moderate climates and are essentially kaolinized rocks still showing the structure of the rock. They are called saprolites in which iron is not as strongly concentrated as in laterites.

The lateritic soil formation involves three major processes which are identified follows (Blight, 1997; Zelalem, 2005).

(1)Decomposition:

Physico-chemical breakdown of primary minerals and the release of constituent elements (SiO_2 , Al_2O_3 , Fe_2O_3 , CaO , MgO , K_2O , Na_2O , etc), which appear in simple ionic forms.

(2)Leaching:

Removing of combined silica and bases and the relative accumulation or enrichment of oxides and hydroxides of sesquioxides which is called laterization. The level to which the second stage is carried depends on the nature and the extent of the chemical weathering of the primary minerals. Under conditions of low chemical and soil-forming activity, the physico-chemical weathering does not continue beyond the clay-forming stage, and tends to produce end products consisting of clay minerals predominantly represented by kaolinite and occasionally by hydrated or hydrous oxides of iron and aluminum.

(3) Desiccation /dehydration/:

Dehydration (either partial or complete) alters the composition and distribution of the sesquioxides rich materials in a manner which is generally not reversible upon wetting.

Dehydration also influences the formation processes of clay minerals. In the case of total dehydration, strongly cemented soils with a unique granular soils structure may be formed (Blight, 1997). Dehydration maybe caused by climatic changes, upheaval of the land, or may also be induced by human activities.

Laterite occurs mostly in the tropical and sub-tropical regions with hot, humid climatic conditions. It has been suggested that a mean annual temperature of around 25°C is needed for their formation, and in seasonal situation there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely. The minimum annual rainfall required for laterite formation is generally at least 750 mm.

The higher the rainfall above this value, the greater is the leaching effect, which removes free silica, reduces the silica/sesquioxides ratio and therefore increases the degree of laterization (CIRIA, 1995; Dibisa, 2008).

2.2.2 Regional Distribution

The majority of the land area containing laterites is between Tropical and sub -tropical regions .Laterites is soils which cover extensive areas in tropical countries with intermittently moist climate.

The six main regions of the world in which laterites occur are Africa, India, South-East, Asia, and Australia, central and south America.

2.3 Pedological and Lithological Classification

Two of the soil classification systems (AASHTO &USCS) which are most widely used today were developed and based on soils from temperate zones. In both of these systems, the grading and Atterberg limits are the basis for classification and also focus primarily on the properties of the soil in its remolded state; this is often misleading with residual soils as their properties are likely to be most strongly influenced by in situ structural characteristics inherited from the original rock mass or developed as a consequence of weathering. There are specific characteristics of residual soils that are not adequately covered by those methods of soil classification. Among these features

(Blight, G.E., 1997) the unusual clay mineralogy of some tropical and subtropical soils results in characteristics that are not compatible with those normally associated with the group to which the soil belongs according to existing systems.

The soil mass in-situ may display a sequence of materials ranging from a true soil to a soft rock depending on the degree of weathering, which cannot be adequately described using existing systems based on classification of transported soils in temperate climates. The D'Hoore (1964) pedological system, which reflects the climate, drainage, topography and parent material, differentiates among the three major lateritic soil groups and used for the description and classification of red tropical soils (CIRIA, 1995) and also 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. it 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 and Todor, Ethiopian Laterites fall under this group (Lyon, 1971)Lithological classification depends on particle size of Laterites (Lyon, 1971)

Lateritic clays < 0.002 mm,

Lateritic silts = 0.002 ~ 0.06 mm

Lateritic sands = 0.06 ~ 2 mm,

Lateritic gravels = 2 ~ 60 mm,

Lateritic stones and cuirasse > 60mm.

Table 2.1 Characteristic of residual soil group (Blight, 1977)

		Examples	Means of identification	Comment on likely engineering properties and behavior
Major group	Sub- group			
Group A Soils without 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	These 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) Semectite Montmorillonite group	Black cotton soils, many soils formed in tropical areas in poorly drained conditions	Dark color 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			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 Allophane 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 silt clay to coarse sand and gravel. Behavior may range from low plasticity to non plastic gravel.

2.4 Chemical, mineralogical and physico-chemical characteristics

The mineralogical and chemical compositions of laterites are dependent on their parent rocks. The distinctive feature of laterite and lateritic soils is the higher proportion of sesquioxides of iron and/or aluminum relative to the other chemical components. The base (alkalis and alkaline earths) is almost absent in lateritic horizons and other lateritic constituents are manganese, titanium, chromium and vanadium oxides. The mineralogical composition is considered to be more important in explaining the physical properties of laterite and lateritic soils. 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 Montmorillonite are rare.

Laterites and Lateritic soils of Africa were studied by Morin W.J. and Todor P.C. (Layon, 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 studied as Ferrisol

Table 2.2 Dominant mineral contents for laterite sub group (Lyon,1971)

Ferruginous	Ferralitic	Ferrisol
Hematite	Gibbsite	Kaolinite
Goethite	Goethite	Goethite
Kaolinite	Kaolinite	Hematite, gibbsite

2.5 Laterites and Lateritic soils

Laterites and lateritic soils may vary from a loose material to a massive rock. For engineering purposes, the term “laterite” is confined to the coarse-grained vermicular material, including massive laterite. The term “lateritic soil” refers to materials with lower concentrations of oxides. A more precise definition is resulted in the application of chemical criteria to tropical weathered soils (CIRIA, 1995). Rossiter, 2004 compiled the classification of soils according to the degree of laterization. The degree of laterization is estimated by the silica-sesquioxide (S-S) ratio ($\text{SiO}_2/(\text{Fe}_2\text{O}_3 + \text{Al}_2\text{O}_3)$)

The conclusion is An S-S ratio of 1.33 or smaller = laterite.

An S-S ratio of 1.33 to 2.0 = lateritic soil.

An S-S ratio of 2.0 or higher = non-lateritic, tropical soil

2.6 Index Tests of Residual Soils

2.6.1 Moisture content

For many soils, the water content may be an extremely important index used for establishing the relationship between the water and soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil. 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 °C.

In many residual soils however, some moisture exists 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 °C 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 50°C 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 °C) 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 °C and 30% RH) if possible, the lower drying temperature of 50 °C should be used (Blight, 1997).

2.6.2 Atterberg limits

A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system. 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.

2.6.3 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 concretions 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 prior to testing is attributed to increased cementation due to oxidation of the iron and aluminum sesquioxides, or Dehydration of Allophane, or both.

2.6.4 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 prior 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. This 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 30 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 disaggregation 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).

2.6.5 Grain –size distribution

The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution, and it is required in classifying the soil. 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:

- **Effect of drying**

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

- ***Chemical pretreatment***

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

- ***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.

2.6.6 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 soil groups but also within different fractions.

In the first place lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (deGraft-Johnson and Bhatia, 1969). For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction, while alumina is concentrated in the silt and clay fractions (Nascimento et al., 1959; Novais-Ferreira and Correia, 1965). The specific gravity has been used as a measure of the degree of maturity (laterization) by some authors (e.g. Ackroyd, 1960). It is common to see specific gravities reported for the gravel and fines separately. The average of the two values can be assumed to be more representative of the specific gravity for the whole soil.

2.7 Shear strength of Claysoils

2.7.1. General

The shear strength of soils is an important aspect in many foundation engineering problems such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls.

The shear strength of cohesive soils can generally be determined in the laboratory by either unconfined compression tests or triaxial shear test equipment; however, the triaxial test is more commonly used. (Braja, 1997)

2.8. Consolidation

2.8.1 Theories of compression and consolidation

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids or extrusion of the pore air and/or water. According to Terzaghi (1943), “a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation.”

When saturated clayey soils which have a low coefficient of permeability are subjected to a compressive stress due to a foundation loading, the pore water pressure will immediately increase; however, because of the low permeability of the soil, there will be a time lag between the application of load and the extrusion of the pore water and, thus, the settlement.

Structures are built on soils. They transfer loads to the subsoil through the foundations. The effect of the loads is felt by the soil normally up to a depth of about two to three times the width of the foundation. The soil within this depth gets compressed due to the imposed stresses. The compression of the soil mass leads to the decrease in the volume of the mass which results in the settlement of the structure.

The displacements that develop at any given boundary of the soil mass can be determined on a rational basis by summing up the displacements of small elements of the mass resulting from the strains produced by a change in the stress system. The compression of the soil mass due to the imposed stresses may be almost immediate or time dependent according to the permeability characteristics of the soil. Cohesionless soils which are highly permeable are compressed in a relatively short period of time as compared to cohesive soils which are less permeable.

The compressibility characteristics of a soil mass might be due to any or a combination of the following factors:

1. Compression of the solid matter.
2. Compression of water and air within the voids.
3. Escape of water and air from the voids.

It is quite reasonable and rational to assume that the solid matter and the pore water are relatively incompressible under the loads usually encountered in soil masses. The change in volume of a mass under imposed stresses must be due to the escape of water if the soil is saturated. But if the soil is partially saturated, the change in volume of the mass is partly due to the compression and escape of air from the voids and partly due to the dissolution of air in the pore water. (Debebe.D, 2011, Kassa.M, 2005 and Kebede.H, 2008 and murthy VNS).

2.8.2 The standard one-dimensional consolidation test

The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of a saturated soil for use in determining the magnitude and rate of settlement of structures. The following test procedure is applied to any type of soil in the standard consolidation test.

Loads are applied in steps in such a way that the successive load intensity, p , is twice the preceding one. The load intensities commonly used being 1/4, 1/2, 1, 2, 4, 8, and 16 tons/ft² (25, 50, 100, 200, 400, 800 and 1600 KN/m²). Each load is allowed to stand until compression has practically ceased (no longer than 24 hours). The dial readings are taken at elapsed times of 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load

is put on the sample (or at elapsed times as per requirements). Sandy samples are compressed in a relatively short time as compared to clay samples and the use of one day duration is common for the latter.

2.8.3 Pre-consolidation pressure

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another. The practical significance of the pre-consolidation load appears in calculating settlements of structures.

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods.

The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B , on the laboratory e - $\log p$ curve of an undisturbed sample as shown in Fig 4.7. From B , a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure, P_c . (Debebe.D, 2011, Murthy1990),

Fig method of the determining P_c by Casagrande method.

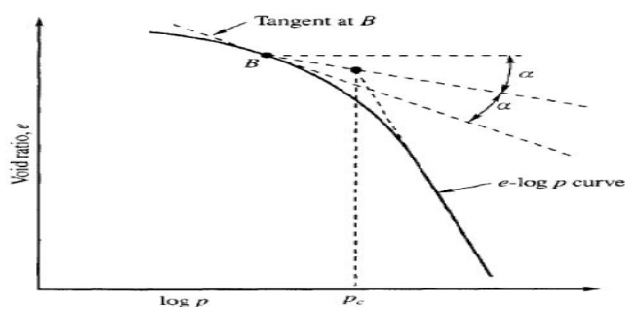


Fig 2.1 method of the determining P_c by Casagrande method

CHAPTER THREE

3. Sampling Area Description

3.1 General

Dilla town sit for different governmental institution, colleges, university, and center of trade, manufacturing company and the extend construction of high rise building.

3.2 Topography and Climate

The town has a longitude and latitude of $6^{\circ} 24' 38''$ N and $38^{\circ} 18' 37''$ E with the elevation of 1572 meters above sea level. The average annual temperature in dilla is 20.6° c and about 1129mm of precipitation annually. The variation in the precipitation between the driest and wettest months is 140 mm. The variation in annual temperature is around 2.2° C

The mean annual temperature and rain fall obtained from National Metrological Service Agency from year 1989-2014 is shown in fig 3.1-3.4

The Mean annual rain fall ranges between 1062.10 - 1669.70mm

The Mean annual maximum temperature ranges between $24.8 - 25.8^{\circ}$ c

The Mean annual Minimum temperature ranges between $13.8 - 14.9^{\circ}$ c.

Southern Ethiopia is known for its abundant rainfall where it receives rainfall for substantial period of the year. The monthly average rainfall distribution is higher in April, May, September and October, where the Dilla area receives frequent and peak rainfall prevalence. Rainy season expected from March to June and from August to November, where as the rest of months receive relatively lower rainfall.

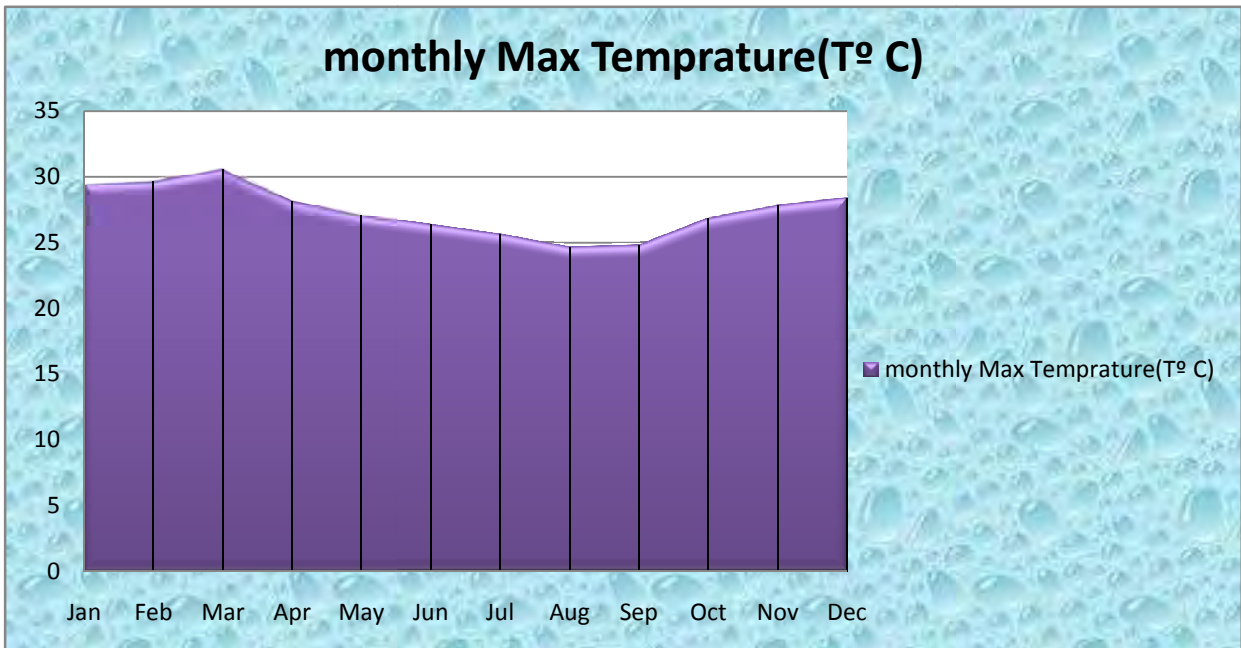


fig 3.1 Annually Maximum Temperature for year (1995-2014)

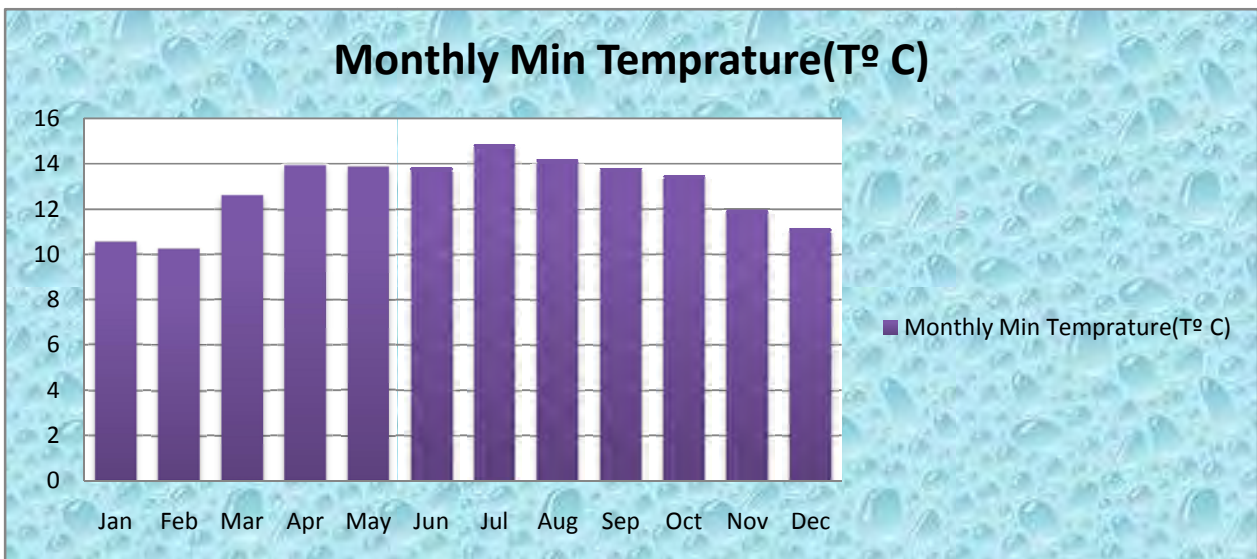


fig 3.2 Annually Minimum Temperature for year(1995-2014)

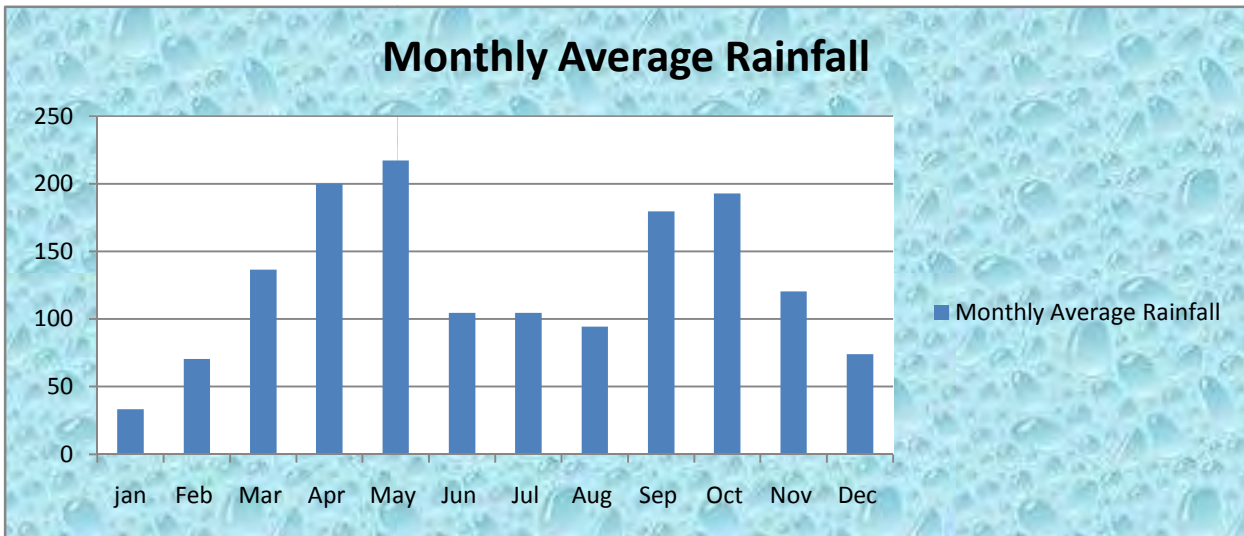


fig 3.3 Monthly average rain fall for year(1989-2014).

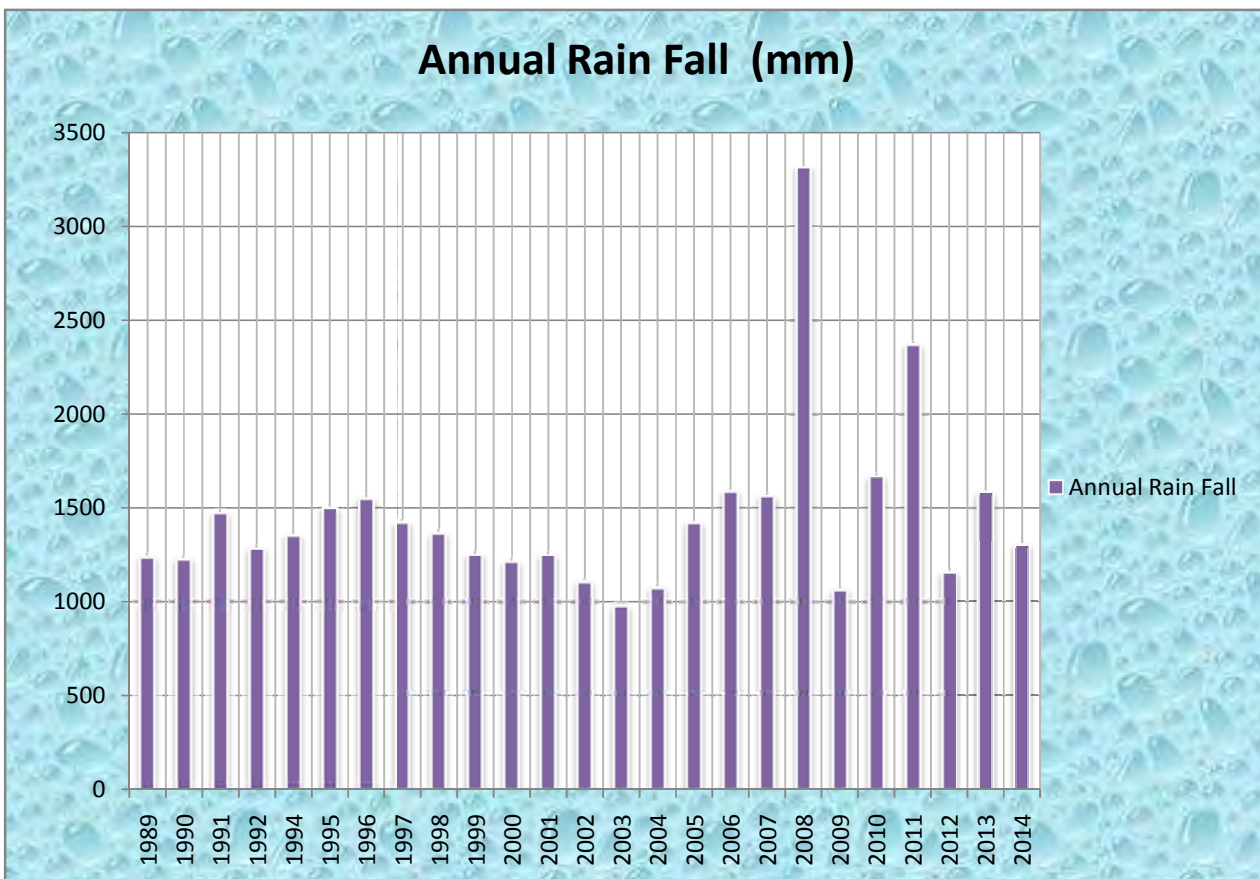


fig 3.4 Annually rain fall for year (1989-2014)

3.3 Soil and Geology

3.3.1 Geologic Condition

The main geologic group identified in the study region includes Ashenge group and Mekdela group. Ashenge group includes Paleocene, Al-gocent, and Miocent, where as the Mekdela group includes Rayalities, and Tracytictuffes.

3.3.2 Soil

Soil is probably the most important element of the natural environment that constitutes part of the land resources with the study area the common soil types identified are as follows:

Table 3.1 Soil types and coverageof the Dilla area.

No	Soil types	Coverage %
1	Dystric nitosols	90
2	Eutric nitosols	6
3	Luvic phaezems	4

As indicated on the table the largest part of the study region is dominated by Dystric Nitosols (red clay soil) and followed by Eutric Nitosols (brown clay soil).[2]fig.3.5below indicates that the location of the research area, i.e. Dilla town on the location map and fig3.6Map showing the locations of test pits.

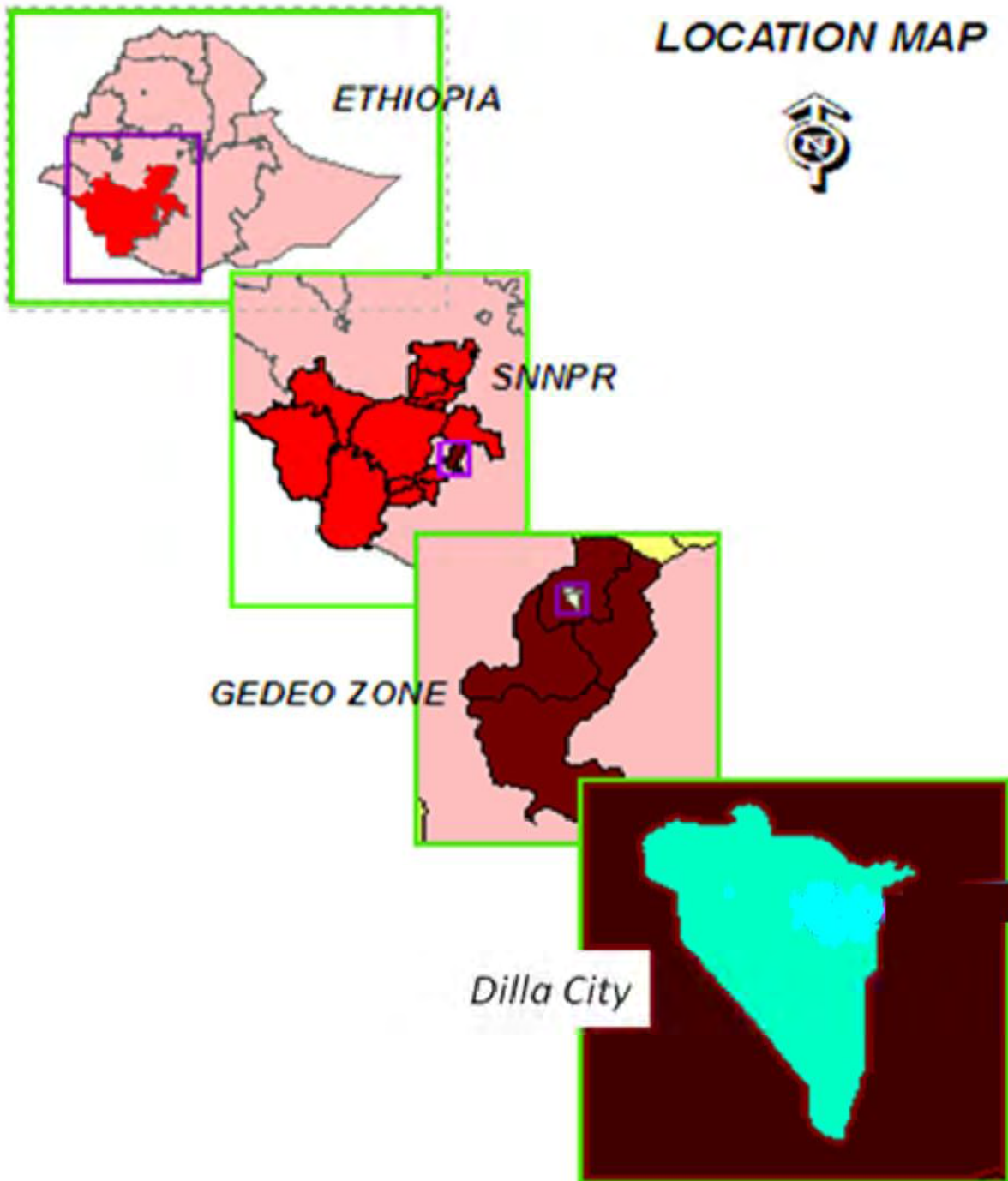


Fig 3.5 location of map

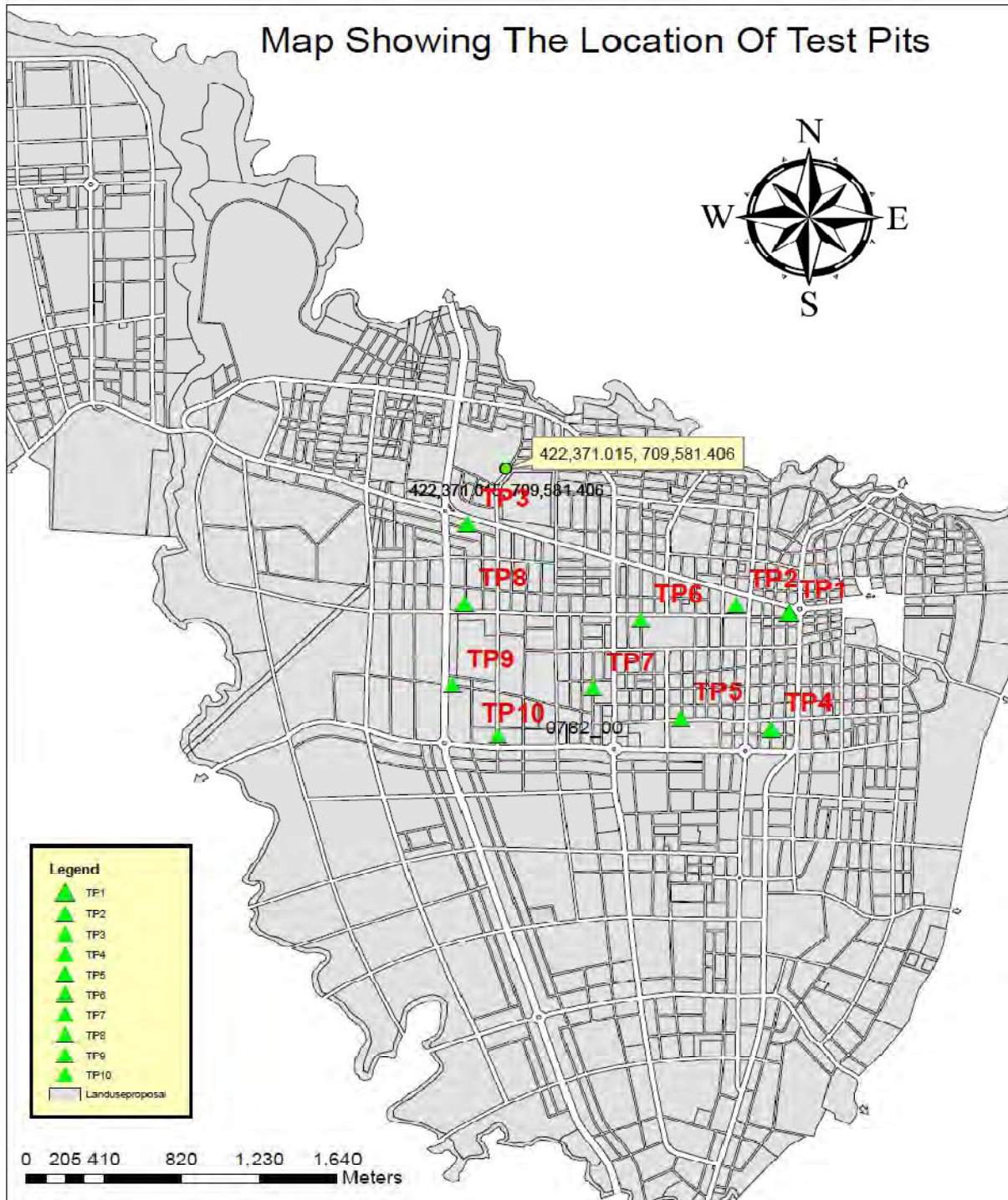


Fig 3.6 Map showing the location of test pits

CHAPTER FOUR

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

4.1 In-situ Properties Description

4.1.1 Sample description

The soil specimens for this thesis work were collected from Dilla. Prior to sampling, visual site investigations were made to consider the different soil types and to sample evenly in the town. Accordingly Ten test pits were chosen. Disturbed samples were collected for this work, weighing about 300kg. Undisturbed samples were sealed with wax and covered with plastic bag and moist towel to maintain surrounding moisture. The location of the test pits and Sample designation are shown in Table 4.1.

Table 4.1. Sample depth and the designation used for Dilla samples.

S/ no	Sampling Depth (m)	Designation	Sample location	Visual observed color
1	1.5	TP1-1	Dillasqure	Reddish
2	3.0	TP1-2	Dillasqure	Reddish
3	1.5	TP2-1	Y /chaffe coffee future bldg	Reddish
4	3.0	TP2-2	Y /chaffe coffee future bldg	Reddish
5	1.5	TP3-1	Molagolija	Reddish
6	3.0	TP3-2	Molagolija	Reddish
7	1.5	TP4-1	Around zelege hotel	Reddish
8	3.0	TP4-2	Around zelege hotel	Reddish
9	1.5	TP5-1	Dilla high school	Reddish
10	3.0	TP5-2	Dilla high school	Reddish
11	1.5	TP6-1	Dilla primary school	Reddish
12	3.0	TP6-2	Dilla primary school	Reddish
13	1.5	TP7-1	Harowolabu sub-city	Reddish
14	3.0	TP7-2	Harowolabu sub-city	Reddish
15	1.5	TP8-1	Muluwengel church	Reddish
16	3.0	TP8-2	Muluwengel church	Reddish
17	1.5	TP9-1	mechael church	Reddish
18	3.0	TP9-2	mechael church	Reddish
19	1.5	TP10-1	Addisuasifalit	Reddish
20	3.0	TP10-2	Addisuasifalit	Reddish

The fig 4.1 and fig 4.2 below give impression that typical profile of sample areas and the in-situ color observation for the soil samples respectively.

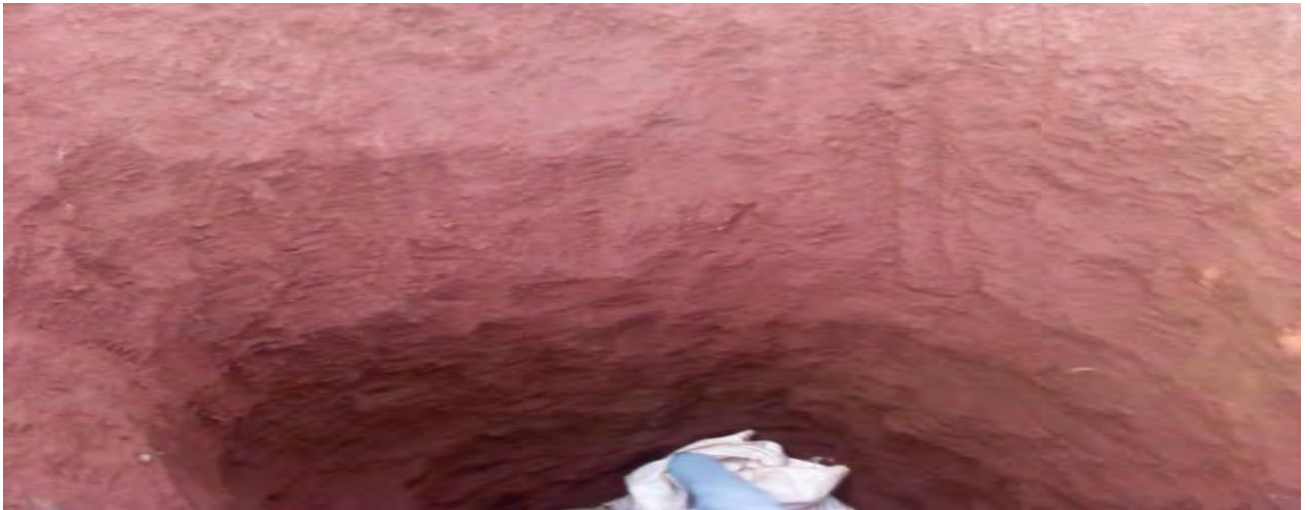


Fig 4.1 Typical Profile of sample area



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

Lateritic Soil is a complex material. The complexity is contributed by its existence in almost innumerable varieties, by its combination of solids, liquid and gases, where in many instances the solid particles also vary in size. Furthermore the relative quantities of solid, liquid and gases in a given soil are found to change due to any physical cause such as loading, seasonal variation and change of temperature which makes the situation further complicated (Samuel, 1989).

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 indexes, free swell and specific gravity.

4.2.1.2 Moisture Content

4.2.1.2.1 Effect of Temperature on Moisture Content Determination

Moisture contents of the soil samples were determined in the laboratory according to Blight, 1997; CIRIA 1995. Drying oven temperatures of 105°C and 50°C with maximum relative humidity (RH) 30% were used to dry the samples. Two samples from each site were taken for moisture content determination. One set of samples were dried to constant weight using drying oven at temperature of 105°C, and the other at a temperature of 50°C with RH 30% taking a minimum of five days to get a constant mass in successive measurements. Table 4.2 below shows the effect of temperature on moisture content determination.

Table 4.2 Effect of Temperature on Moisture Content Determination

S/ No.	Sample Designation	Sampling Depth(m)	Oven Temperature		Difference [%]
			105 ⁰ c	50 ⁰ c +30% RH	
1	TP1-1	1.5	32.75	28.97	3.78
2	TP1-2	3.0	29.39	28.78	0.61
3	TP2-1	1.5	36.07	35.77	0.3
4	TP2-2	3.0	33.68	32.87	0.81
5	TP3-1	1.5	31.07	26.90	4.17
6	TP3-2	3.0	33.82	31.56	2.26
7	TP4-1	1.5	33.34	29.99	3.35

8	TP4-2	3.0	28.67	26.7	1.97
9	TP5-1	1.5	34.26	31.73	2.53
10	TP5-2	3.0	36.28	34.74	1.54
11	TP6-1	1.5	28.08	25.89	2.19
12	TP6-2	3.0	25.32	24.3	1.02
13	TP7-1	1.5	25.65	23.54	2.11
14	TP7-2	3.0	27.17	23.98	3.19
15	TP8-1	1.5	25.19	24.2	0.99
16	TP8-2	3.0	28.73	27.3	1.43
17	TP9-1	1.5	24.23	23.6	0.63
18	TP9-2	3.0	29.14	27.12	2.02
19	TP10-1	1.5	27.18	26.44	0.74
20	TP10-2	3.0	26.73	25.12	1.61

In all the cases, except TP3-1, it can be observed that the difference in moisture content is less than 4%, which shows that the amount of structural water is insignificant. Therefore, all the subsequent tests of moisture content determinations are based on the conventional drying temperatures. But for TP3-1 the moisture difference is above 4%, which may mean that the soils under investigation contain loosely bound water of hydration. Tests for moisture content determination should be carried out by drying at lower temperature (i.e. either air drying, or oven-drying at 50°C and 30% RH) if possible, the lower drying temperature of 50 °C should be used (Blight, 1997).

4.2.1.3.1 Atterberg limits

4.2.1.3.2 General

Atterberg limits are valuable tool for evaluating and characterizing residual soils and are carried out to determine the consistency of fine-grained soils.

4.2.1.4.1 Test Procedures

For the determination of the Atterberg limit values, air and oven dried soil samples were tested following the procedure given in (Air dry and oven dry) as per the procedure of ASTM D4318-00. The air dried soil samples were prepared by spreading the material out in trays in the laboratory and leaving it open to the air for at least 10 days or equivalently put inside oven at a temperature of 50°C with maximum relative humidity 30% for at least 5 days. The room temperature was about 20°C. The oven dried samples were prepared by drying the soils overnight at 105°C. Wet sample preparations were also carried out. Portion of the soil samples passing No. 40 (0.425mm) sieve were kept wet for a period of 24 Hrs for moisture content equilibration.

4.2.1.4.2 Test results and discussions

Oven dried (OD), air dried (AD) sample preparations were carried out to investigate the effect of pretreatment on plasticity characteristics of the samples under investigation. In order to investigate

the effect of temperature on the Atterberg limits, the samples were tested oven dried, air-dried. The test results are shown in Table 4.3. From the test results, one can see that the different treatments affect the Atterberg Limits of these particular soils. The test results show little difference for almost all soils. Hence pretreatment has only slight effect on the values of Atterberg limits for the soil samples under investigation.

Table 4.3 Atterberg limit values at different testing condition

S/ no,	sample designation	sampling depth	Condition	liquid limit	plastic limit	plasticity index
1	TP1-1	1.5	50 ⁰ C	66	28	38
			105 ⁰ C	63	28	35
2	TP1-2	3.0	50 ⁰ C	68	28	40
			105 ⁰ C	63	25	38
3	TP2-1	1.5	50 ⁰ C	63	28	35
			105 ⁰ C	59	27	32
4	TP2-2	3.0	50 ⁰ C	72	32	39
			105 ⁰ C	70	31	40
5	TP3-1	1.5	50 ⁰ C	62	27	35
			105 ⁰ C	61	22	39
6	TP3-2	3.0	50 ⁰ C	66	29	37
			105 ⁰ C	61	26	34
7	TP4-1	1.5	50 ⁰ C	65	28	37
			105 ⁰ C	64	29	35
8	TP4-2	3.0	50 ⁰ C	77	38	39
			105 ⁰ C	68	29	39
9	TP5-1	1.5	50 ⁰ C	64	31	33
			105 ⁰ C	63	33	30
10	TP5-2	3.0	50 ⁰ C	64	35	29
			105 ⁰ C	62	30	32
11	TP6-1	1.5	50 ⁰ C	63	31	32
			105 ⁰ C	62	32	30
12	TP6-2	3.0	50 ⁰ C	63	31	32
			105 ⁰ C	59	31	29
13	TP7-1	1.5	50 ⁰ C	62	33	29
			105 ⁰ C	57	31	26
14	TP7-2	3.0	50 ⁰ C	61	35	26

			105 ⁰ C	59	31	28
15	TP8-1	1.5	50 ⁰ C	65	30	35
			105 ⁰ C	62	33	29
16	TP8-2	3.0	50 ⁰ C	54	29	25
			105 ⁰ C	53	25	28
17	TP9-1	1.5	50 ⁰ C	65	31	34
			105 ⁰ C	62	30	32
18	TP9-2	3.0	50 ⁰ C	64	33	31
			105 ⁰ C	61	30	31
19	TP10-1	1.5	50 ⁰ C	55	32	23
			105 ⁰ C	52	29	23
20	TP10-2	3.0	50 ⁰ C	56	31	25
			105 ⁰ C	51	30	21

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 is 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 (LL5min) 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 4.4 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 disaggregation 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

4.4 for samples prepared in air dry condition the difference between 5 min and 30 min mix is greater than 5 %.

Table 4. 4 Atterberg limits at different conditions and mixing time.

sample designation	sampling depth(m)	mixing time	Condition	liquid limit	plastic limit	plasticity index	liquid limit	
							50 ⁰ C	105 ⁰ C
TP1-1	1.5	5 min	50 ⁰ C	58	25	33	8	7
			105 ⁰ C	56	27	29		
		25 min	50 ⁰ C	66	32	34		
			105 ⁰ C	63	28	35		
TP2-1	1.5	5min	50 ⁰ C	64	33	31	5	6
			105 ⁰ C	62	30	32		
		25min	50 ⁰ C	69	34	35		
			105 ⁰ C	68	35	33		
TP3-1	1.5	5min	50 ⁰ C	63	33	28	4	5
			105 ⁰ C	63	29	34		
		25min	50 ⁰ C	67	31	36		
			105 ⁰ C	68	33	35		
TP5-1	1.5	5min	50 ⁰ C	61	33	28	8	8
			105 ⁰ C	63	31	32		
		25min	50 ⁰ C	69	31	38		
			105 ⁰ C	71	30	41		
TP9-1	1.5	5min	50 ⁰ C	58	30	28	11	8
			105 ⁰ C	63	37	26		
		25min	50 ⁰ C	69	31	38		
			105 ⁰ C	71	33	38		

4.2.1.4.4 Plasticity chart

Plasticity Index is numerical difference between liquid limit and plastic limit. The plasticity index represents the range in water content through which a soil is in plastic state. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample (Samuel, 1989). Which implies that the plasticity values increase with the corresponding increase in clay contents (Braja, 1997) it is observed that the plasticity index of the soil increase linearly with the percentage of clay- size fraction.

Experimental results of 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].

$$PI = 0.73 (LL-20) \text{-----}[4.1]$$

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 below the test results are all below the U-line. Hence the test results are considered acceptable (Budhu, 2000).

$$PI = 0.90 (LL-8) \text{-----} [4.2]$$

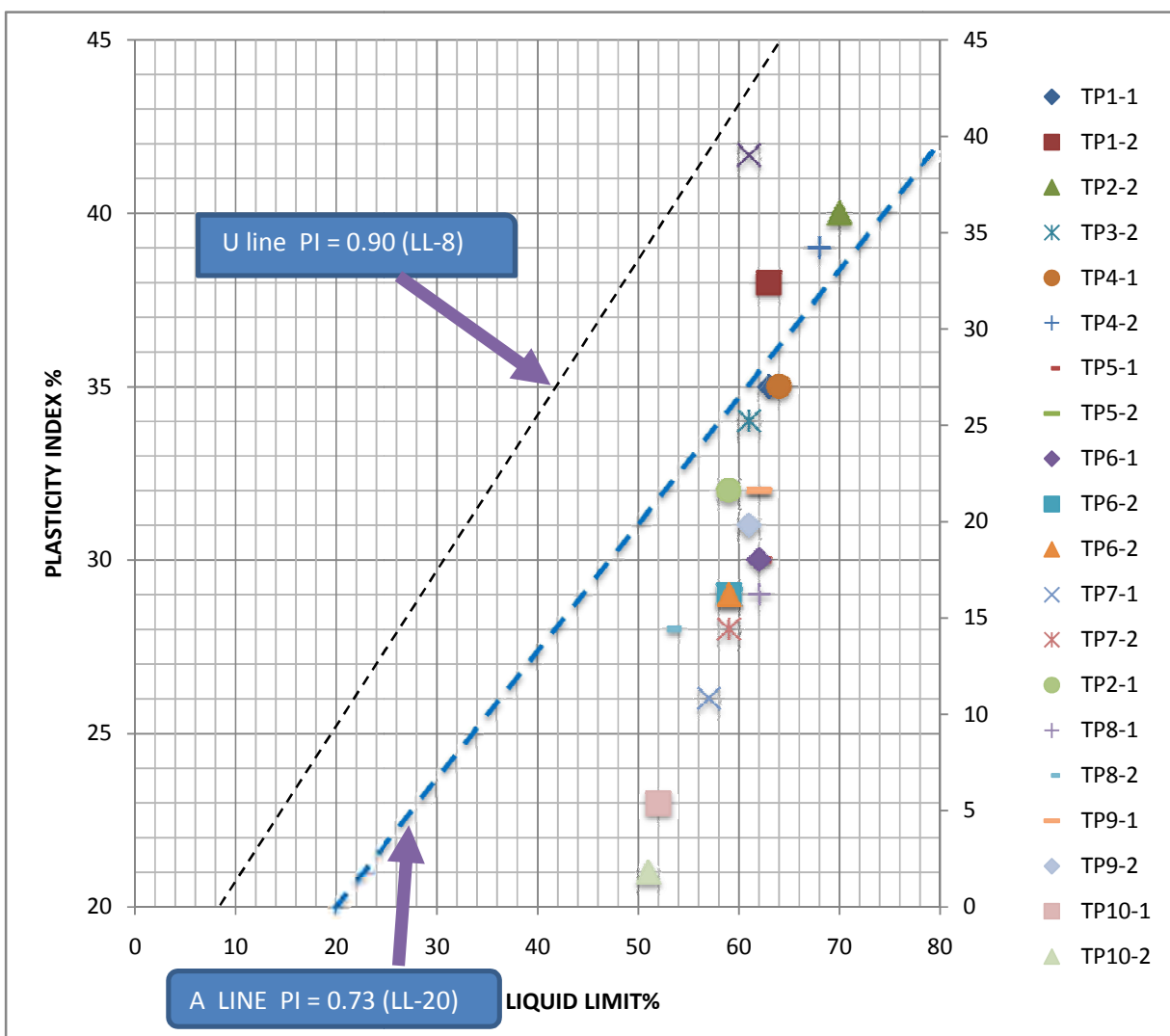


Fig 4.3 Plasticity Chart

TP6-1	1.5	Oven dry	46.15	30	0.65
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4.2.1.5 Activity

The atterberg limit of a given cohesive soil is controlled by the type and amount of clays minerals present in it. Skempton(1953) made the observation 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 “Ac” is defined as

$$A_c = PI/C \text{-----}[4.4]$$

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). Table shows below degree of colloidal activity.

Table 4.5 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 4.6, Skempton’s colloidal activity values for TP1-TP10 are less than 0.75. From Table 4.5 .One can understood, 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 Sesquioxide, which results in the suppression of the surface activity of clay particles (Lyon, 1971).

Table 4.4 Summary of Skempton’s colloidal activity values

Sample Designation	Sampling Depth(m)	Condition	Clay Fraction (%)	Plasticity Index (%)	Activity (Ac)
TP1-1	1.5	oven dry	71.42	35	0.49
TP1-2	3.0	oven dry	54.28	38	0.7
TP2-1	1.5	oven dry	49.23	32	0.65
TP2-2	3.0	oven dry	61.53	40	0.65
TP3-1	1.5	oven dry	61.90	39	0.63
TP3-2	3.0	oven dry	61.81	34	0.55
TP4-1	1.5	oven dry	62.5	35	0.56
TP4-2	3.0	oven dry	72.2	39	0.54
TP5-1	1.5	oven dry	71.42	30	0.42
TP5-2	3.0	oven dry	49.23	32	0.65

TP6-2	3.0	Oven dry	61.7	29	0.47
TP7-1	1.5	Oven dry	60.46	26	0.43
TP7-2	3.0	Oven dry	59.57	28	0.47
TP8-1	1.5	Oven dry	65.90	29	0.44
TP8-2	3.0	Oven dry	62.2	28	0.45
TP9-1	1.5	Oven dry	60.52	23	0.38
TP9-2	3.0	Oven dry	60.78	31	0.51
TP10-1	1.5	Oven dry	54.76	23	0.42
TP10-2	3.0	Oven dry	56.76	21	0.37

4.2.1.6 Free swell

Clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals Montmorillonite 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 10cm³ of dry soil which has passed the No. 40 (0.425mm) sieve in to 100 cm³ graduated cylinder filled with distilled water. After 24 hours, final volume of the suspension is read. Hence, free swell is defined as

$$\text{Free Swell} = \frac{[(\text{Final Volume}) - (\text{Initial Volume})]}{\text{Initial Volume}} \times 100\% \dots\dots\dots [4.5]$$

Free swell test results for air dried and oven dried samples are summarized in Table 4.8 below. From the test result one can see that the free swell of the soil under investigation ranges from 22.5% to 45%. 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 4.8 Free swell test results at different condition

S/n	Sample designation	Sampling depth(m)	condition	Free Swell (%)
1	TP1-1	1.5	oven dry	23
			air dry	25
2	TP1-2	3.0	oven dry	29
			air dry	33
3	TP2-1	1.5	oven dry	30
			air dry	35
4	TP2-2	3.0	oven dry	26
			air dry	28
5	TP3-1	1.5	oven dry	31
			air dry	36
6	TP3-2	3.0	oven dry	35
			air dry	41
7	TP4-1	1.5	oven dry	35
			air dry	37.5
8	TP4-2	3.0	oven dry	30.5
			air dry	32.5
9	TP5-1	1.5	oven dry	35
			air dry	39
10	TP5-2	3.0	oven dry	26
			air dry	37.5
11	TP6-1	1.5	oven dry	27.5
			air dry	32.5
12	TP6-2	3.0	oven dry	22.5
			air dry	25
13	TP7-1	1.5	oven dry	36
			air dry	41
14	TP7-2	3.0	oven dry	35
			air dry	39
15	TP8-1	1.5	oven dry	25
			air dry	32.5
16	TP8-2	3.0	oven dry	40
			air dry	45
17	TP9-1	1.5	oven dry	42.5
			air dry	42.5
18	TP9-2	3.0	oven dry	32.5
			air dry	45
19	TP10-1	1.5	oven dry	40
			air dry	45
20	TP10-2	3.0	oven dry	40
			air dry	43

4.2.1.7 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). In the first place lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (deGraft-Johnson and Bhatia, 1969).

For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction, while alumina is concentrated in the silt and clay fractions (Nascimento et al., 1959; Novais-Ferreira and Correia, 1965.)

The soil samples under investigation were determined using the ASTM procedure, designation D854-58. Moreover, the tests were conducted at different test temperatures (air-dried and oven-dried) to see the effect of pre-drying the test results is shown in Table 4-9. The specific gravity values of oven dry sample are less than the air dried one.. Hence specific gravity significantly changes upon drying prior to testing by any means. All test results are from 2.61 to 2.89. The test result is in agreement with Morin W.J. and TodorP.C.,and (Lyon, 1971). The available data indicate that specific gravities vary not only with the soil textural but also with in different fractions. The specific gravity has been used as a measure of the degree of maturity. table 4. 9 show Specific gravity comparison by different oven temperatures.

Table 4.9 Specific gravity comparison by different oven temperatures

S/ no.	Sample designation	Sampling depth(M)	Oven Temperature	
			105 ⁰ C	(50 ⁰ C + 30%RH)
1	TP1-1	1.5	2.70	2.79
2	TP1-2	3.0	2.72	2.77
3	TP2-1	1.5	2.76	2.82
4	TP2-2	3.0	2.68	2.80
5	TP3-1	1.5	2.71	2.76
6	TP3-2	3.0	2.68	2.73
7	TP4-1	1.5	2.68	2.77
8	TP4-2	3.0	2.66	2.89
9	TP5-1	1.5	2.68	2.75
10	TP5-2	3.0	2.71	2.78
11	TP6-1	1.5	2.68	2.69
13	TP7-1	1.5	2.71	2.77
14	TP7-2	3.0	2.68	2.74
15	TP8-1	1.5	2.63	2.74
16	TP8-2	3.0	2.69	2.79
17	TP9-1	1.5	2.63	2.66
18	TP9-2	3.0	2.61	2.67
19	TP10-1	1.5	2.63	2.71
20	TP10-2	3.0	2.66	2.75

4.2.1.8 Grain size Analysis

4.2.1.8.1 General

A naturally occurring soil sample may have particles of various sizes. Over the years, various agencies have tried to develop the size limits of gravel, sand, silt, and clay. For a basic understanding of the nature of soil, the distribution of the grain size present in a given soil mass must be known. The grain size distribution of coarse-grained soils (gravelly and or sandy) is determined by sieve analysis (Braja, 1997).

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 retained in 2mm sieve was insignificant; hence hydrometer analysis was used with Sodium Hexametaphosphate dispersing agent (Tebebu, 2008).

4.2.1.8.2 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.200 sieve is oven dried at 105 °C 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)). Wet preparation methods are used when the coarse-grained particles of a sample are soft and pulverize readily, as in Practice D 421, or when the fine particles are very cohesive and tend to resist removal from the coarse particles.

4.2.1.8.4 Test Results and Discussions

The grain size analysis test results for all soil samples under investigation at different testing conditions are summarized in fig.4.7-4.8. The test result curves of all soil samples under investigation are shown in the fig below 4.7-4.8. The values obtained from the gradation tests were analyzed with Figs TP1-1, TP2-1.....TP10-2 respect to the effect of pre-treatment, soil variations along laterally and depth wise.

4.2.1.8.5 Effect of Pretreatment

Oven dried (OD), air dried (AD) sample preparations were carried out to investigate the effect of pretreatment on grain size distribution of the soil samples under investigation. The test results are shown in Table 4.8 from the curves one can observe that the two methods of pretreatment produce a small change in cumulative percentage passing between AD and OD for sample as shown in Annex A-1 to A-8. One can carry out routine grain size analysis tests on air dried state. The grain size distributions of some laterite soils change as a result of hydration reaction when dried specially at high temperature. The test result for dry preparation of sample is expected to result in decreased percentage of the finer fraction (clay). On the other hand, test result on sample carried out wet preparation (as received) is expected the reverse as compared to effect of drying i.e., increased percentage of finer fraction. But the test results of the grain size analysis of the samples under investigation for this thesis work resulted in negligible difference in gradation following different sample pre-treatment procedures.

4.2.1.8.6 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. below (TP1-TP10). From the curves one can observe that the soil samples have the same shape of cumulative percentage passing curves. Distance of sampling from one test pit to another test pit is range from 1.2 Kms to 1.7 Kms. This similarity may indicate that 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 2.5

To see the variation of soils along the depth profile, grain size distribution tests were carried out. The test results for soil samples TP1-1, TP1-2, TP2-1, TP2-2, TP3-1, TP3-2, TP4-1, and TP5-1 at 1.50m and 3.0m shown in fig. 4.7-4.8 below 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 (Hana T, 2008) and (Abeba.Z, 2005).

Percentage Amount of the Grain Sizes for different conditions

Sample designation	Sampling depth(m)	Test condition	% amount of Particle Sizes			
			Gravel	Sand	Silt	Clay
TP1-1	1.5	50 ⁰ C	0	15.25	15.68	69
		105 ⁰ C	0	14.78	14.18	71.04
TP1-2	3.0	50 ⁰ C	0.12	13.98	16.4	69.5
		105 ⁰ C	0	14.9	36.06	49.04
TP2-1	1.5	50 ⁰ C	0.14	11.16	12.7	76
		105 ⁰ C	0	38.7	12.2	49.1
TP2-2	3.0	50 ⁰ C	0.04	9.36	13.1	77.5
		105 ⁰ C	0	10.6	38.9	50.5
TP3-1	1.5	50 ⁰ C	0	15.82	16.14	68
		105 ⁰ C	0	16.2	22.8	61
TP3-2	3.0	50 ⁰ C	0	10.52	15.42	74
		105 ⁰ C	0	12.02	26.44	61.54
TP4-1	1.5	50 ⁰ C	0	17.68	8.21	74
		105 ⁰ C	0	9.88	20.91	62.22
TP4-2	3.0	50 ⁰ C	0	19.38	6.28	74.02
		105 ⁰ C	0	9.88	18.36	71.77
TP5-1	1.5	50 ⁰ C	0	13.82	15.0	71
		105 ⁰ C	0.04	14.78	14.18	71
TP5-2	3.0	50 ⁰ C	0	14.58	17.22	68
		105 ⁰ C	0	15.04	36.47	48.49
TP6-1	1.5	50 ⁰ C	0	9.3	16.02	74
		105 ⁰ C	0	14.2	41.23	45.58
TP6-2	3.0	50 ⁰ C	0	10.1	16.89	74.05
		105 ⁰ C	0	14.66	24.625	60.714
TP7-1	1.5	50 ⁰ C	0	19.5	6.39	76
		105 ⁰ C	0	15.30	23.30	61.38
TP7-2	3.0	50 ⁰ C	0	11.81	14.15	74.04
		105 ⁰ C	0	9.96	29.77	60.27
TP8-1	1.5	50 ⁰ C	0	12.05	13.92	74
		105 ⁰ C	0	12.59	22.76	64.64
TP8-2	3.0	50 ⁰ C	0	8.55	17.28	74
		105 ⁰ C	0	8.93	30.179	60.89
TP9-1	1.5	50 ⁰ C	0	8.78	17.21	74
		105 ⁰ C	0	23	16.71	60.29
TP9-2	3.0	50 ⁰ C	0	10.07	15.90	74
		105 ⁰ C	0	12.83	25.2	61.97
TP10-1	1.5	50 ⁰ C	0	12.36	13.61	74.03
		105 ⁰ C	0	19.46	26.73	53.80
TP10-2	3.0	50 ⁰ C	0	10.96	15.02	74.02
		105 ⁰ C	0	15.26	28.11	56.63

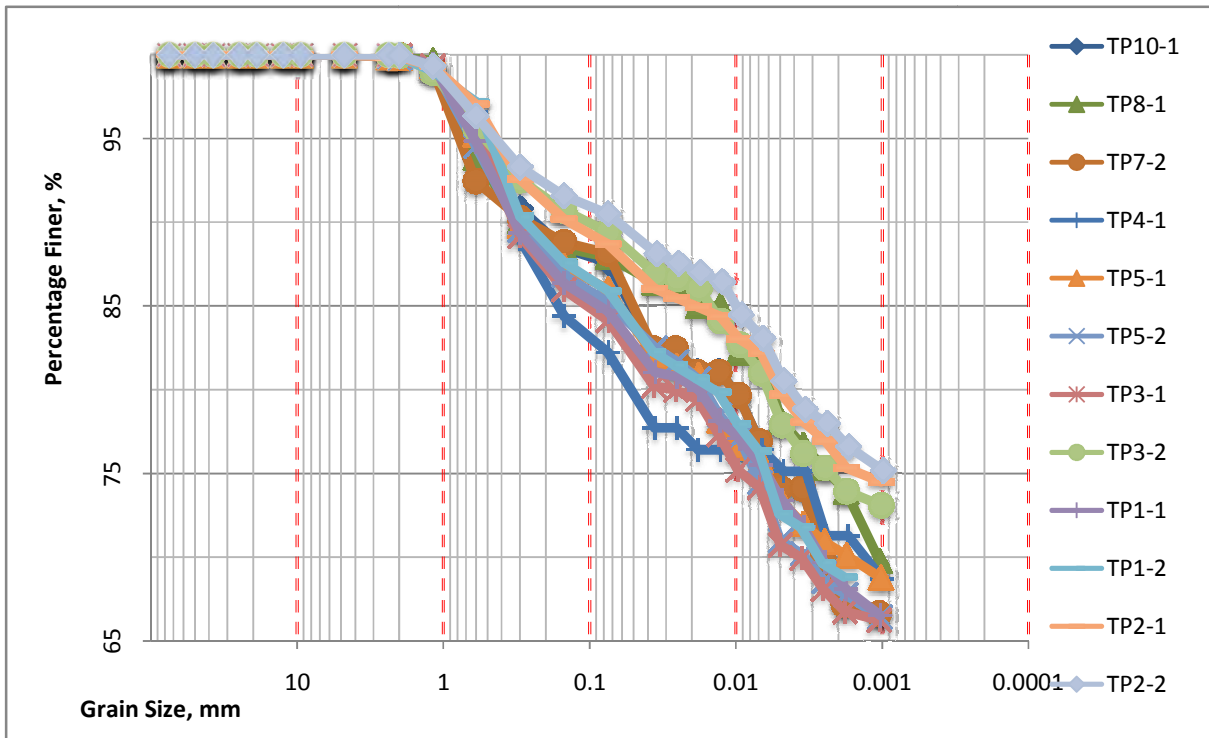


Fig4.7the Airdrying grain size distribution curve

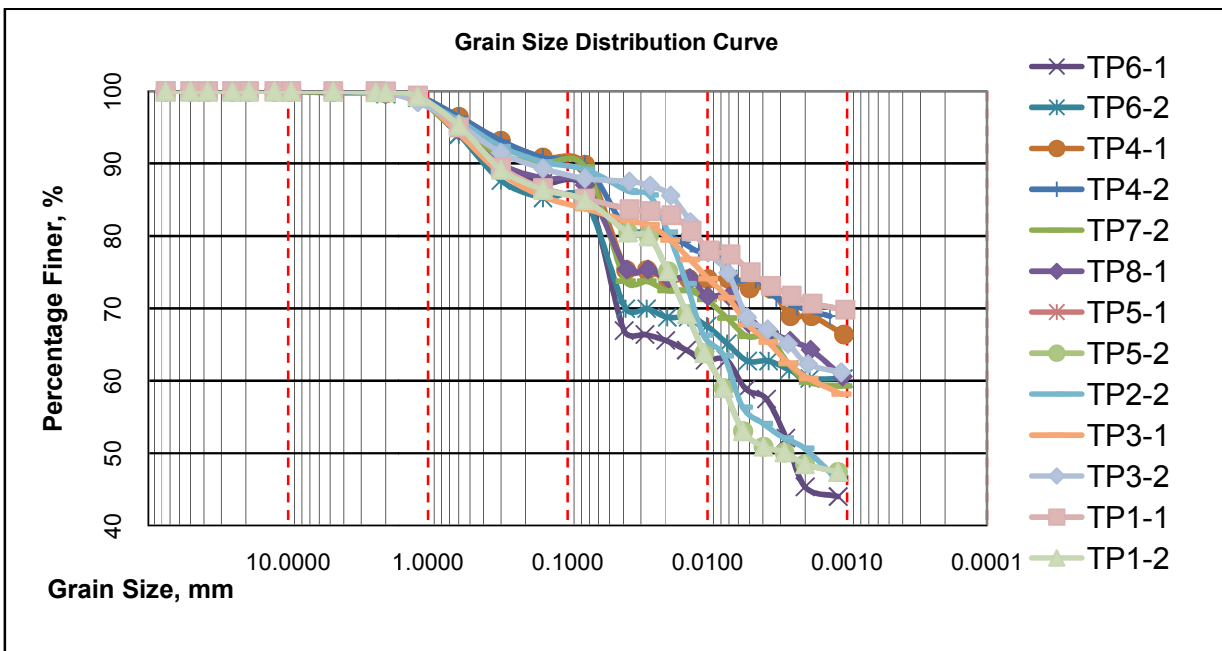


Fig 4.8 the Oven drying grain size distribution curve

Table 4.11 Classification According to the AASHTO

Sample designation	Sampling depth(m)	Test Condition	Liquid limit	Plasticity index	Group index	AASHTO
TP1-1	1.5	50 ⁰ C	66	38	20max	A-7-6
		105 ⁰ C	63	35	20max	A-7-6
TP1-2	3.0	50 ⁰ C	68	40	20max	A-7-6
		105 ⁰ C	63	38	20max	A-7-6
TP2-1	1.5	50 ⁰ C	63	35	20max	A-7-6
		105 ⁰ C	59	32	20max	A-7-6
TP2-2	3.0	50 ⁰ C	72	39	20max	A-7-5
		105 ⁰ C	70	40	20max	A-7-5
TP4-1	1.5	50 ⁰ C	65	37	20max	A-7-6
		105 ⁰ C	64	35	20max	A-7-6
TP4-2	3.0	50 ⁰ C	77	39	20max	A-7-5
		105 ⁰ C	68	39	20max	A-7-6
TP5-1	1.5	50 ⁰ C	64	33	20max	A-7-5
		105 ⁰ C	63	30	20max	A-7-5
TP5-2	3.0	50 ⁰ C	64	29	20max	A-7-5
		105 ⁰ C	62	32	20max	A-7-5
TP6-1	1.5	50 ⁰ C	63	32	20max	A-7-5
		105 ⁰ C	62	30	20max	A-7-5
TP6-2	3.0	50 ⁰ C	63	32	20max	A-7-5
		105 ⁰ C	59	29	20max	A-7-5
TP7-1	1.5	50 ⁰ C	62	29	20max	A-7-5
		105 ⁰ C	57	26	20max	A-7-5
TP7-2	3.0	50 ⁰ C	61	26	20max	A-7-5
		105 ⁰ C	59	28	20max	A-7-5
TP8-1	1.5	50 ⁰ C	65	35	20max	A-7-6
		105 ⁰ C	62	29	20max	A-7-6
TP8-2	3.0	50 ⁰ C	54	25	20max	A-7-6
		105 ⁰ C	53	28	20max	A-7-6
TP9-1	1.5	50 ⁰ C	65	34	20max	A-7-5
		105 ⁰ C	62	32	20max	A-7-5
TP9-2	3.0	50 ⁰ C	64	31	20max	A-7-5
		105 ⁰ C	61	31	20max	A-7-5
TP10-1	1.5	50 ⁰ C	55	23	20max	A-7-5
		105 ⁰ C	52	23	20max	A-7-5
TP10-2	3.0	50 ⁰ C	56	25	20max	A-7-6
		105 ⁰ C	51	21	20max	A-7-6

Table 4.12 Classification According to the USC

Sample designation	Sampling Depth (m)	Test Condition	Liquid Limit	Plastic Index	% amount of particle sizes				Classification according to USCS
					Gravel	Sand	Silt	Clay	
TP1-1	1.5	50°C	66	38	0	15.25	15.68	69	CH
		105°C	63	35	0	14.78	14.18	71.04	CH
TP1-2	3.0	50°C	68	40	0.12	13.98	16.4	69.5	CH
		105°C	63	38	0	14.9	36.06	49.04	CH
TP2-1	1.5	50°C	63	35	0.14	11.16	12.7	76	CH
		105°C	59	32	0	38.7	12.2	49.1	CH
TP2-2	3.0	50°C	72	39	0.04	9.36	13.1	77.5	CH
		105°C	70	40	0	10.6	38.9	50.5	CH
TP3-1	1.5	50°C	62	35	0	15.82	16.14	68	CH
		105°C	61	39	0	16.2	22.8	61	CH
TP3-2	3.0	50°C	66	37	0	10.52	15.42	74	CH
		105°C	61	34	0	12.02	26.44	61.54	CH
TP4-1	1.5	50°C	65	37	0	17.68	8.21	74	CH
		105°C	64	35	0	9.88	20.91	62.22	CH
TP4-2	3.0	50°C	77	39	0	19.38	6.28	74.02	CH
		105°C	68	39	0	9.88	18.36	71.77	CH
TP5-1	1.5	50°C	64	33	0	13.82	15.0	71	CH
		105°C	63	30	0.04	14.78	14.18	71	MH
TP5-2	3.0	50°C	64	29	0	14.58	17.22	68	CH
		105°C	62	32	0	15.04	36.47	48.49	MH
TP6-1	1.5	50°C	63	32	0	9.3	16.02	74	CH
		105°C	62	30	0	14.2	41.23	45.58	CH
TP6-2	3.0	50°C	63	32	0	10.1	16.89	74.05	CH
		105°C	59	29	0	14.66	24.625	60.71	MH
TP7-1	1.5	50°C	62	29	0	19.5	6.39	76	MH
		105°C	57	26	0	15.30	23.30	61.38	MH
TP7-2	3.0	50°C	61	26	0	11.81	14.15	74.04	MH
		105°C	59	28	0	9.96	29.77	60.27	MH
TP8-1	1.5	50°C	65	35	0	12.05	13.92	74	CH
		105°C	62	29	0	12.59	22.76	64.64	MH
TP8-2	3.0	50°C	54	25	0	8.55	17.28	74	MH
		105°C	53	28	0	8.93	30.179	60.89	MH
TP9-1 AAiT	1.5	50°C	65	34	0	8.78	17.21	74	CH
		105°C	62	32	0	23	16.71	60.29	CH

TP9-2	3.0	50 ⁰ C	64	31	0	10.07	15.90	74	MH
		105 ⁰ C	61	31	0	12.83	25.2	61.97	MH
TP10-1	1.5	50 ⁰ C	55	23	0	12.36	13.61	74.03	MH
		105 ⁰ C	52	23	0	19.46	26.73	53.80	MH
TP10-2	3.0	50 ⁰ C	56	25	0	10.96	15.02	74.02	MH
		105 ⁰ C	51	21	0	15.26	28.11	56.63	MH

4.2.1.8.7 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 4.11_4.12. All samples fall under group A-7 sub group A-7-5 (GI=20) and A-7-6 (GI=20) according to AASHTO. Classification according to USCS from plasticity chart places most samples below A-line and another is above A-line. The soils are grouped under CH, and MH.

4. 2.2.1 Soil Distribution Mapping

4.2.2.2 General

The general objectives of a soil distribution map is to provide comprehensive picture of geological ,morphological and hydrological condition of the area ,from which the information needed for the proposed development can be drawn.

Of which derived maps have been largely inspired by engineering will be prepared for assessing the strategic qualities of an area.

4.2.2.3 Uses and Scale related issues in Engineering Map

Generally ,soil distribution map and information systems are at least used for evaluation of building and development of the site location ,high way route alignments ,suitability study for municipal sewage valuable and slug disposal location. for the urban planning and design of industrial plants and large civil engineering construction ,geotechnical maps to a scale of 1 :5000 are commonly prepared .

4.2.2.4 Mapping Techniques

Field information, laboratory test result are collected and information from published engineering reports have been used for classifications of soils for development of engineering soil maps. In zoned maps,zones proposed and will be divided based on the chosen geotechnical parameter,average properties of soil based on classification. output at fixed depth will be used .finally documentation map which displays all test pits ,borehole ,well and other reference location used for the compilation of the map, will be plotted and will be attached with full description.

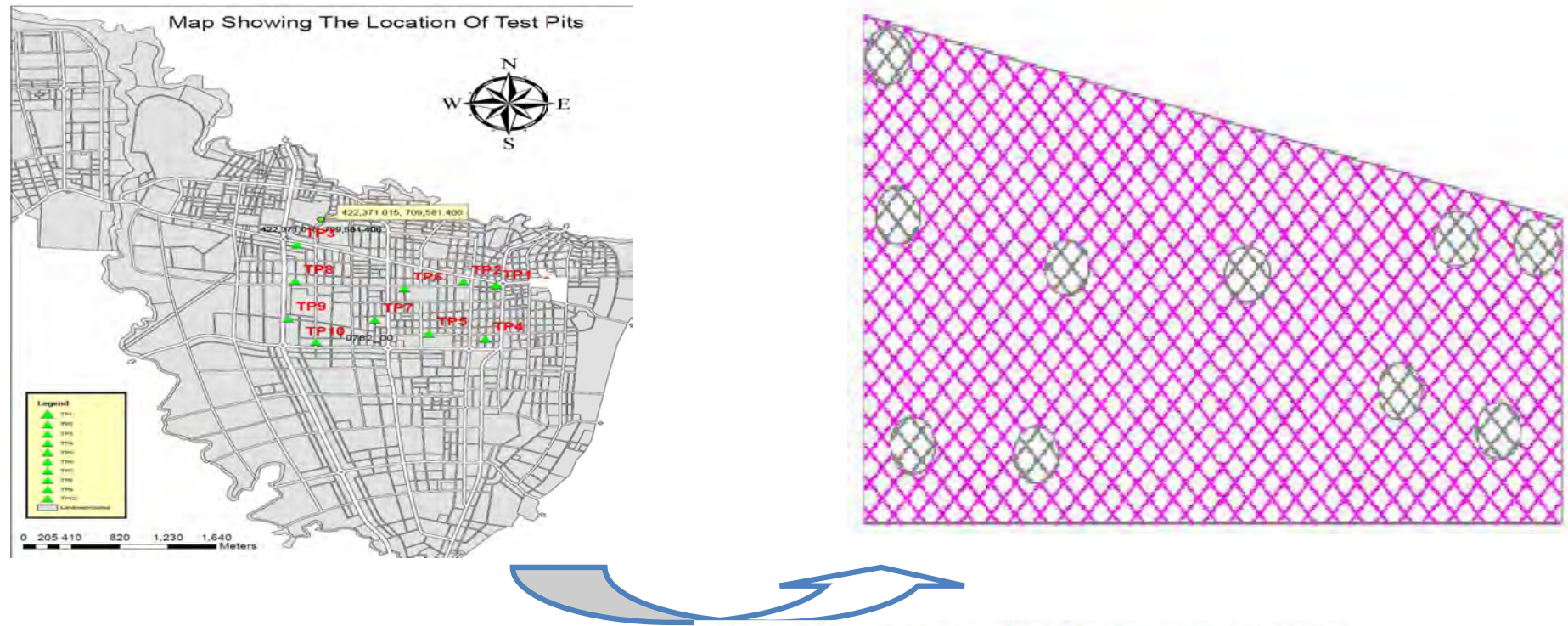


Fig 4.9 Tentative Soil Distribution Map of Dilla.

Consolidation Characteristics (50-1600Kpa)

$C_c = 0.249-0.298$

$C_v = (0.107-0.269)10^{-3} \text{ cm}^2/\text{sec}$

$a_v = (12.3-25)10^{-5} \text{ m}^2/\text{KN}$

$K = (11.4-75.8)10^{-8} \text{ cm}/\text{sec}$

As the soil in the study area is found to be similar both from visual observation and test results only one single solid layer map is produced. This is shown in fig.4.9 Tentative soil distribution map of Dilla town.

4.2.2 Geochemical Tests

Geochemical (oxide) tests are carried out to know quantitatively main oxides of the soil material. Almost all soils on earth contain some amount of colloidal oxides and hydroxides. The oxides and hydroxides of aluminum, iron and silicon are of greatest interest since they are the ones most frequently encountered. Iron and Aluminum oxides coat mineral particles, or cement particles of soils together. They may also occur as distinct crystalline units, such as hematite, gibbsite and magnetite (Dibsa, 2008 and Tibebu, 2008).

Geochemical tests were conducted at Geological Survey of Ethiopia Geosciences Laboratory center. Atomic Absorption Spectrometer, HF attack and Gravimetric Analysis methods were used to get the percentage oxide composition of the soils under investigation.

The test results are shown in Table 4.13.

The degree of laterization of the soil samples can be evaluated based on ratio of Silica/Sesquioxides as detailed in section 2.5 the Sesquioxide(S-S), 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.

S-S ratio less than 1.33 have been considered as true laterites,

S-S between 1.33 and 2.00 is for lateritic soils and

S-S greater than 2.00 is for non-lateritic tropically weathered soils.

The test resultss shown in Table 4.13 oxide compositions give Silica – Sesquioxide ratio is between 1.33 and 2.0. This indicates that the soils are lateritic.

Table 4.13 Oxide Composition in Percent

Test pit	Sample depth (m)	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Na ₂ O	k ₂ O	MnO	P ₂ O ₅	Ti ₂ O	H ₂ O	LOI	SiO ₂
														R ₂ O ₃
TP2-1	1.5	52.36	24.80	8.06	1.20	1.5	<0.01	0.16	0.08	0.02	1.09	0.02	11.72	1.59
TP5-1	1.5	57.25	20.24	8.80	1.18	1.38	<0.01	0.26	0.06	0.02	1.36	0.06	10.16	1.97

4.2.3 Shear strength of soils

4.2.3.1 General

The knowledge of shear strength of soils is important in many foundation engineering problems such as the bearing capacity of foundations, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls.

The shear strength of cohesive soils can generally be determined in the laboratory by either Unconfined Compression Tests or Triaxial Shear Test equipment; the Triaxial Test is more commonly used (Braja, 1997). However, the availability, fastness and less expensive machine the UCS more common. In this thesis the UCS is used

4.2.3.2 Unconfined Compression Tests

The unconfined compression test is a special case of a triaxial compression test in which the all-round pressure $\sigma_3 = 0$.

The tests are carried out only on saturated samples which can stand without any lateral support. The test is therefore, applicable to cohesive soils only. The test is an undrained test and is based on the assumption that there is no moisture loss during the test.

The unconfined compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils.

Table 4.14 Consistency and Unconfined Compression Strength of clays

Consistency	q_u (KN/M ²)
Very soft	0-24
Soft	24-48
Medium	48-96
Stiff	96-192
Very stiff	192-383
Hard	>383

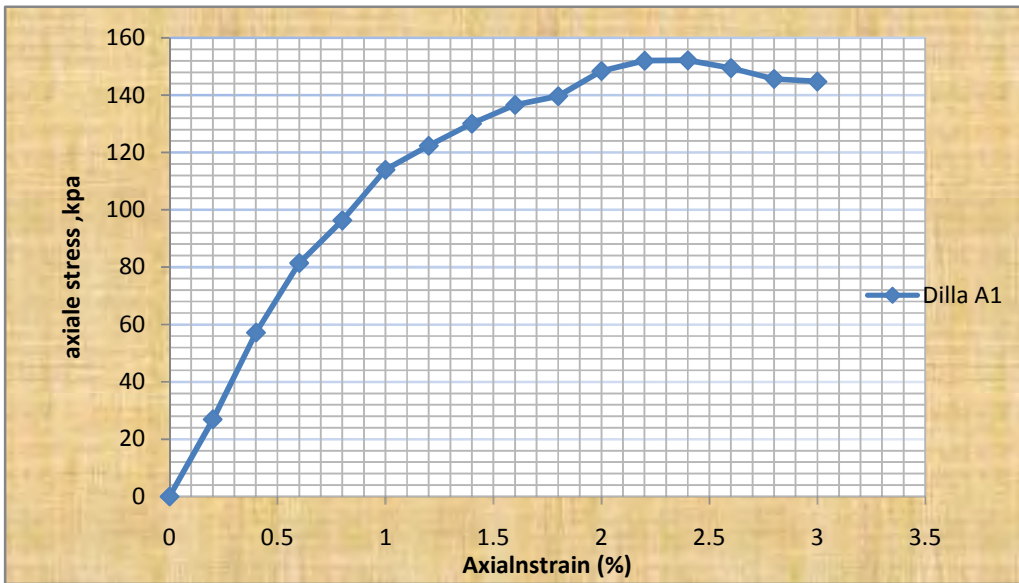
(Braja, 1997 and Tibebe, 2008)

The Consistency of clays from Unconfined Compression Test result shows that entire soil sample was ranges from stiff to very stiff clay as shown table 4.14 The conducted test results for unconfined Compression Test have shown in fig.4.10 -4.11 for test pits TP1-1, and the conducted

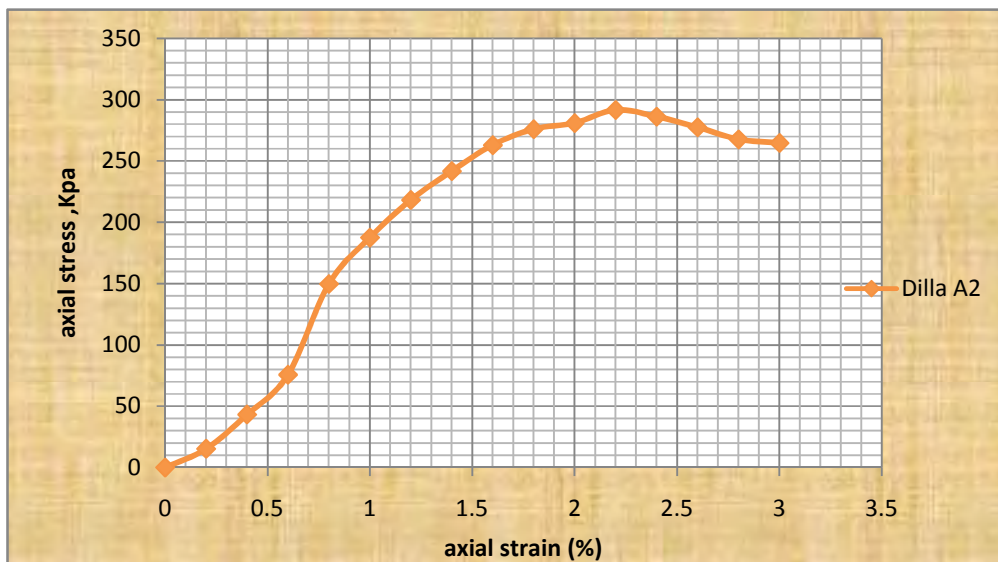
test results for Unconfined Compression Test shown in fig.4.10 -4.11 for test pits TP1-1, and TP1-2 respectively

Table 4.15 Classifications of Consistency based on Unconfined Compression Strength of clay

S/ No.	Sample Designation	Sampling Depth(M)	Unconfined Compression Strength (q_u) KN/M ²	Classification based Consistency
			Undisturbed clay (q_u) _u	
1	TP1-1	1.5	165.80	Stiff
2	TP1-2	3.0	351.37	Very stiff
3	TP2-1	1.5	141.78	Stiff
4	TP2-2	3.0	206.71	Very stiff
5	TP3-1	1.5	172.90	Stiff
6	TP3-2	3.0	253.48	Very stiff
7	TP4-1	1.5	153.60	Stiff
8	TP4-2	3.0	314.90	Very stiff
9	TP5-1	1.5	185.14	Stiff
10	TP5-2	3.0	307.50	Very stiff
11	TP6-1	1.5	153.61	Stiff
12	TP6-2	3.0	209.55	Very stiff
13	TP7-1	1.5	127.31	Stiff
14	TP7-2	3.0	176.53	Stiff
15	TP8-1	1.5	317.44	Very stiff
16	TP8-2	3.0	383.48	Hard
17	TP9-1	1.5	110.81	Stiff
18	TP9-2	3.0	173.61	Stiff
19	TP10-1	1.5	130.0	Stiff
20	TP10-2	3.0	110.81	Stiff



The fig. 4.10 Unconfined Compression Test of samples TP1-1



4.2.3.3 Sensitivity

A Cohesive soil in its natural state of occurrence has a certain structure .when the structure is disturbed, the soil becomes remolded, and its engineering properties change considerably. Sensitivity of a soil indicates its weakening due to remolding.

The ratio of the undisturbed shear strength of a cohesive soil to the remolded strength at the same water content is defined as the *sensitivity St*:

$$St = \frac{\text{Undisturbed strength } (q_u)_u}{\text{Remolded strength } (q_u)_r} \text{----- [4.6]}$$

Where

$(q_u)_u$ = unconfined cohesive strength of undisturbed clay.

$(q_u)_r$ = unconfined cohesive strength of remolded clay.

Table 4.16 Classification of Soils based on sensitivity

S.No	Sensitivity	Soil type
1	<1	Insensitive
2	1-2	Little sensitive
3	2-4	Moderate sensitive
4	4-8	Sensitive
5	8-16	Extra sensitive
6	>16	Quick

From sensitivity St : test result, the entire soil sample was categorized to little sensitive clay as shown in table 4.16.

Table 4.17 Unconfined Compression Strength (q_u) KN/m²

S/no	Sample Designation	Sampling Depth(M)	Unconfined Compression Strength (q_u) KN/M ²	
			Undisturbed clay (q_u) _u	Remolded clay(q_u) _r
1	TP1-1	1.5	165.80	152.04
2	TP1-2	3.0	351.37	286.62
3	TP2-1	1.5	141.78	98.81
4	TP2-2	3.0	206.71	176.79
5	TP3-1	1.5	172.90	135.03
6	TP3-2	3.0	253.48	203.33
7	TP4-1	1.5	153.60	148.7
8	TP4-2	3.0	314.90	308.5
9	TP5-1	1.5	185.14	141.33
10	TP5-2	3.0	307.50	222.2
11	TP6-1	1.5	153.61	152.0
12	TP6-2	3.0	209.55	198.78
13	TP7-1	1.5	127.31	123.4
14	TP7-2	3.0	176.53	168.5
15	TP8-1	1.5	317.44	310.2
16	TP8-2	3.0	383.48	379.5
17	TP9-1	1.5	110.81	108.3
18	TP9-2	3.0	173.61	167.8
19	TP10-1	1.5	130.0	125.8
20	TP10-1	3.0	110.81	107.9

4.3 Consolidation test.

4.3.1 General

The two important soil properties useful in the design of the foundation are bearing capacity and settlements. The first is determined by means of shear test. And settlement characteristics are determined by means of a consolidation test. In the consolidation test the change in volume of the soil, due to the drainage of the water from a sample induced by static load is measured. (Tadesse, 1989).

4.3.1.1 Test procedure and results The test was done according to the procedure called standard test method for one-dimensional consolidation properties of soils on the ASTM standard, Designation D2435-96.

4.3.1.2 Pre-consolidation pressure

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another.

The pre-consolidation pressure for the two samples is determined as depicted in fig 4.13.

The results are shown in Annex B, and shows almost similar pre-consolidation pressure. Further the table 4.18 below shows Summary of the consolidation test results

Soil was grouped using grain size distribution test, Atterberg limit test and classification has been done by AASH To.

Based on this merit the soil found in Dilla town is grouped under A-7-5 and A-7-6.

To minimized the energy and cost of thesis work, the Geochemical test and consolidation test was conducted only from representative sample i.e. one test from A-7-5 and A-7-6 from another test.

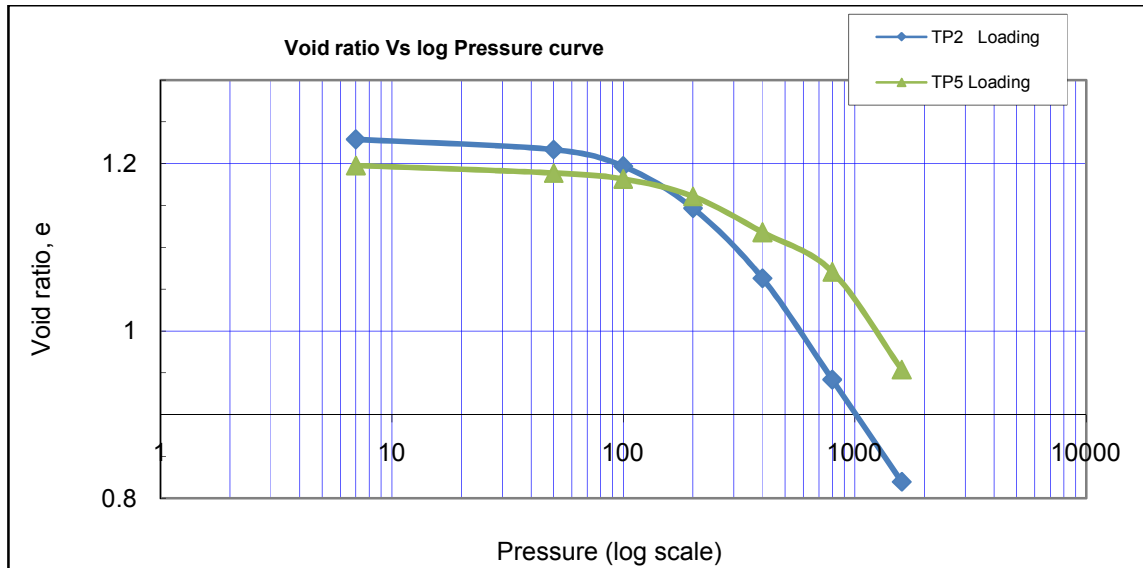
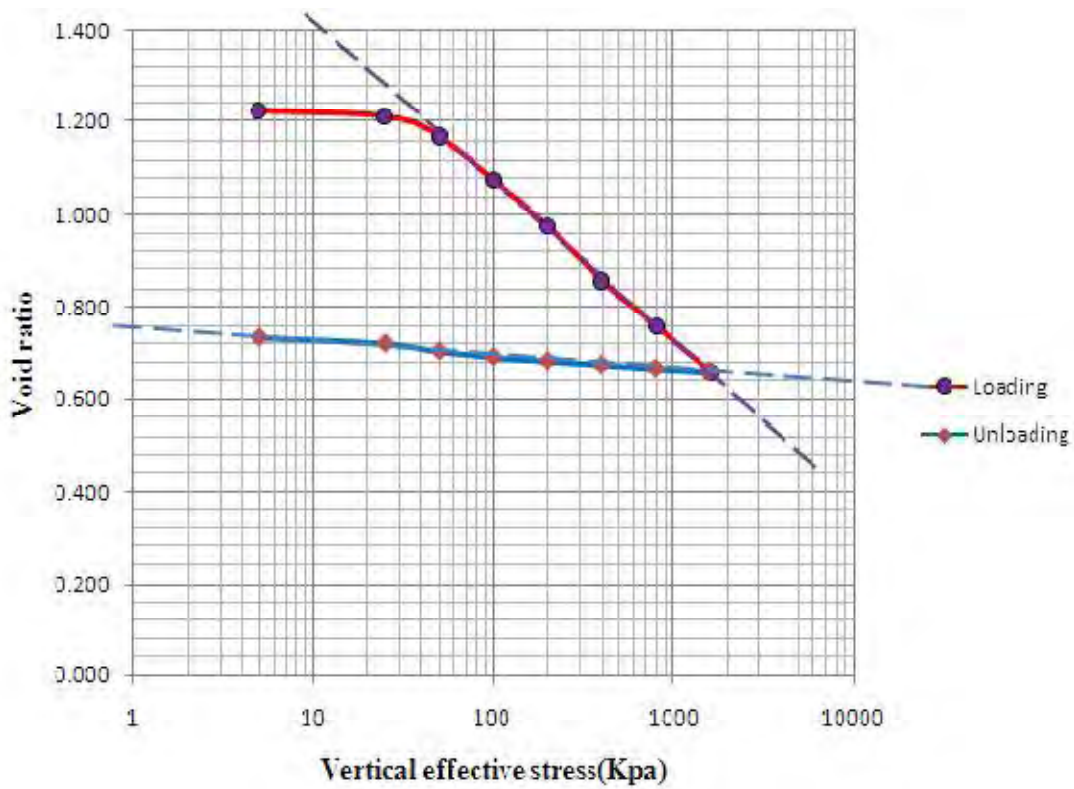


Fig 4. 13Plot of vertical effective stress Vs void ratio on semi-log scale

4.3.1.2 Compression index

The compression index, C_c , is equal to the slope of the linear portion of the void ratio versus log pressure plot. Thus:

$$C_c = \frac{e_1 - e_2}{\log \sigma'_2 - \log \sigma'_1} = \frac{\Delta e}{\log(\sigma'_2/\sigma'_1)}$$



Since the graph on the semi- log scale is approximately a straight line, it is possible to determine the slope of this line just by taking two arbitrary points on the line and this will be the compression index (cc)=0.249.

Table 4.18 Summary of the consolidation test results

Test Pit designation	Depth(M)	Natural Moisture content (%)	Total unit weight in γ (kpa)	pressure (Kpa)	Void ratio(ef)	Coefficient of Consolidation C_v 10-3cm ² /sec	Compression index CC	Overburden Pressure P_o (kpa)	Pre-consolidation pressure P_c (kpa)	Over consolidation ratio (OCR)
TP5-2	3.0	34.74	18.04	7	1.25	----	0.249	54.12	190	3.5
				50	1.25	---				
				100	1.24	3.53				
				200	1.22	0.8				
				400	1.17	0.565				
				800	1.12	0.222				
				1600	1.00	0.222				
TP2-2	3.0	32.87	18.75				0.298	56.75	200	3.52
				7	1.23	-----				
				50	1.23	-----				
				100	1.22	0.565				
				200	1.20	0.334				
				400	1.15	0.22				
				800	1.06	0.128				
1600	0.96	0.098								

4.3.1.4 Coefficient of Consolidation

For a given load increment, the coefficient of consolidation C_v can be determined from the laboratory observations of time versus dial reading.

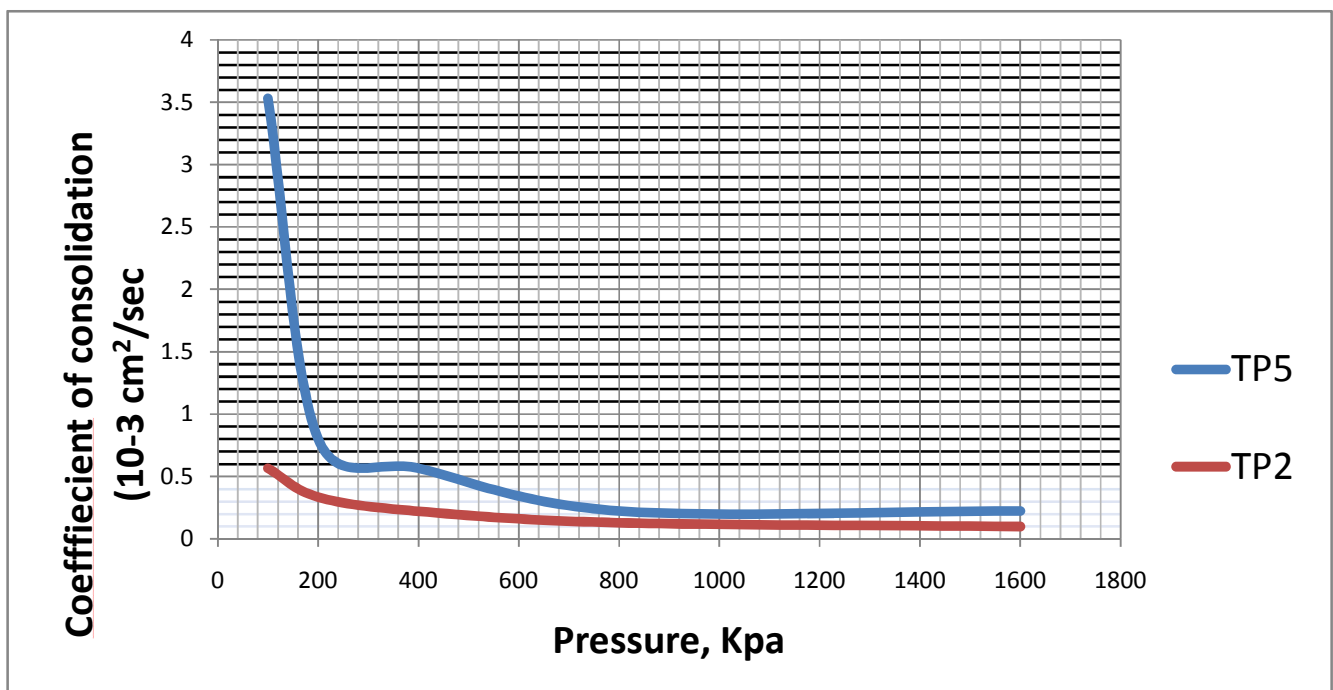
There are several procedures presently available to estimate the coefficient of consolidation, some of which are 1) Logarithm-of-time method and 2) Square-root-of-time fitting method

From the measured data and the data obtained from either of the above two methods,

The consolidation curve (pressure-void ratio relationship) can be plotted. This data is

Useful in determining the compression index, the recompression index and the preconsolidation

Pressure (or maximum past pressure) of the soil. Detail of the consolidation results is shown in table 4.18 above.



The fig. 4.14 Coefficient of Consolidation Vs Pressure

4.3.1.5 Coefficient of Permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test.

The Coefficient of permeability can be obtained from the following relationship.

$$K = \frac{C_v \cdot a_v \cdot \gamma_w}{(1 + e_0)}$$

(1+e₀)

Where: K=Coefficient of permeability

CV= Coefficient of consolidation

a_v=Coefficient of compressibility

γ_w= Unit weight of water

e₀= void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.19. It is noted that, the ratio of change in void ratio to change in pressure, was obtained from annex D and Fig4.13. As shown in Table 4.19, the range of values of coefficient of permeability lies between 10⁻⁷ and 10⁻⁹ cm/sec, which indicates that the soils are practically impervious or have low permeability.

Table 4.19 Relationship between Void ratio and coefficient of permeability

Test pit Designation	Depth (M)	Pressure in (kpa)	Void ratio(e _f)	Coefficient of Consolidation CV=10 ⁻³ cm ² /sec	Coefficient of Compressibility a _v 10 ⁻⁵ m ² /KN	Coefficient of permeability K (10 ⁻⁹ cm/sec)
TP5-2	3.0	100	1.24	3.53	15	23.8
		200	1.22	0.8	21	7.5
		400	1.17	0.565	22	5.73
		800	1.12	0.222	12.3	1.28
		1600	1.00	0.222	15	1.28
TP2-2	3.0	100	1.22	0.565	12.5	3.18
		200	1.20	0.334	19.5	2.19
		400	1.15	0.22	20.0	2.04
		800	1.06	0.128	23	1.43
		1600	0.96	0.098	20	1.0

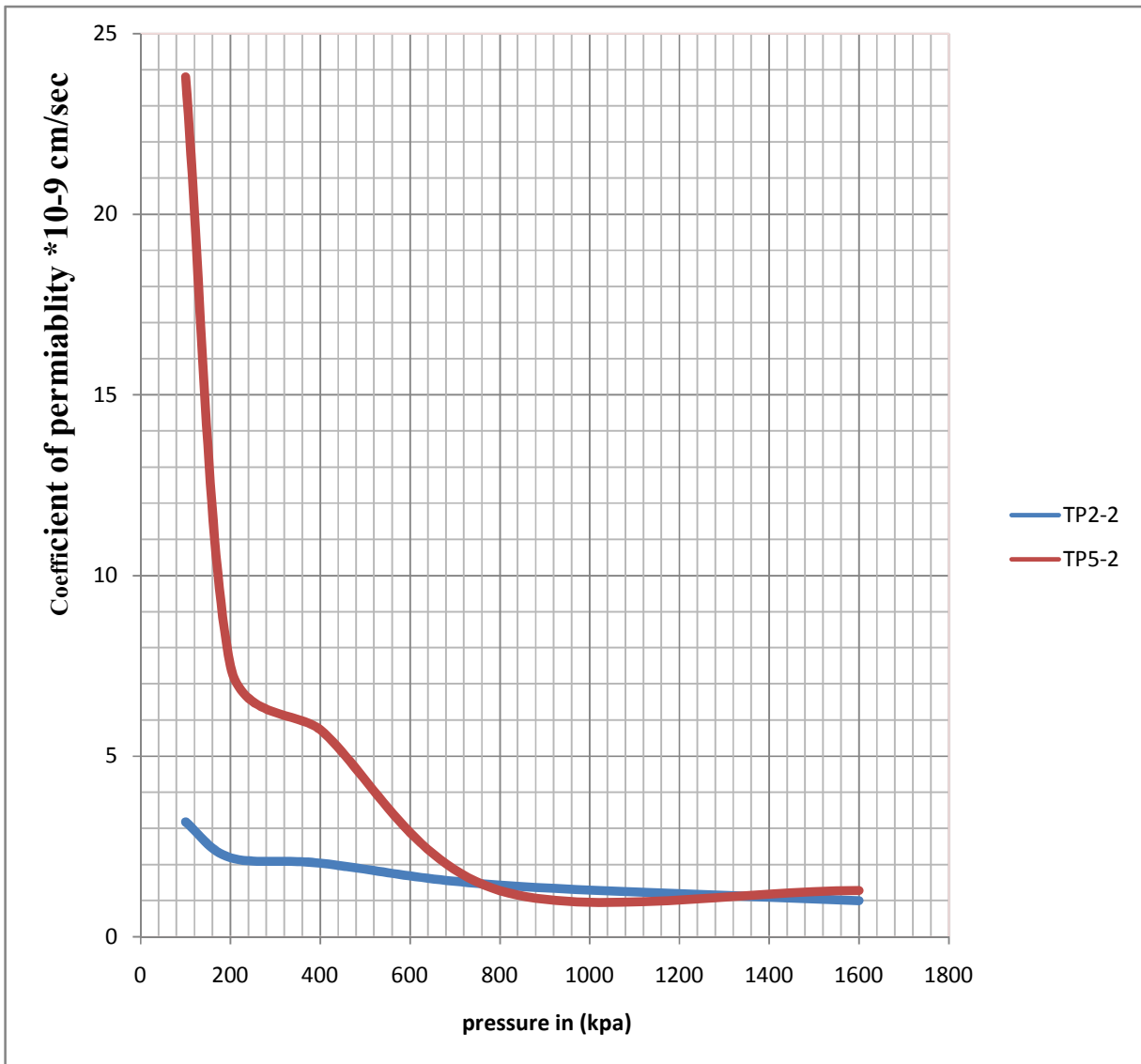


Fig 4.15 Coefficient of permeability Vs pressure for the loading range of 100-1600kPa

CHAPTER FIVE

5. Comparisons and Discussions of Test Results

5.1 Comparison of Test Results with Laterites and Lateritic Soils of Africa

For the soil under investigation; Index property tests, unconfined compression test and Consolidation test were studied and a comparison was made with known laterites and lateritic soils of Africa. The soil samples collected from Africa were classified according to D'Hoore classification methods into Ferruginous, Ferralitic and Ferrisol. The results of Index property tests, compaction tests and California Bearing Ratio (CBR) test soil from Nejo- Mendi, Assosa and Wolayita- Sodo, are shown that the average values of the various properties, i.e., liquid limit, plasticity index, gradation, compaction and CBR; indicate that 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 and Nejo-Mendi, Assosa and Wolayita- Sodo and Dilla town soils are shown in Table 5.1. From section 2.5 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 .Comparison of the gradation and Atterberg Limits values of Dilla town soils are similar to Ferrisol soils of Ethiopia previously determine

Table 5.1 Comparison of average soil properties for this and previous studies

	Ferruginous		Ferralitic		Ferrisols		Nejo-mendi	Asossa	Wolayta sodo	Dilla Town
	Ghana	Other Countries	Ghana	Other countries	Ghana	Other countries				
Passing sieves size% (mm)										
25	99	99	99	99	95	99	94.5	100	100	100
19	98	98	96	97	94	99	88.3	100	100	100
9.5	93	89	86	90	86	92	73.3	99	100	100
4.75	75	76	70	80	73	74	62.6	94.6	100	100
2.0	51	65	54	70	52	61	51.6	89	100	100
0.425	46	51	46	54	40	51	45.0	74.9	95	82.47
0.075	30	32	34	40	37	44	38.3	72	89	76.10
0.002	13	16	19	26	25	24	9.2	21	57	61.02
LL	31	33	42	47	46	55	59	59	60	54.75
PL	18	12	24	24	23	29	38	25	37	28.05
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 5.4 Comparison of Test Results with Pervious Done Research Work

	Previous research Zelalem,2005	Previous research Wakuma,2007	Previous research Hana,2008	Current Research
Soil type	Lateritic	lateritic	Lateritic	Lateritic
Location	Nejo-mendi	Asossa	Wolayyata-sodo	Dilla town
Clay contents%	2.0-20.6	2.5-60	48-69.7	46.15-71.42
Activity	0.97-0.98	0.62-1.02	0.317-0.488	0.38-0.65
Clay minerals			Kaolinite	
Liquid limits	48-67	41-72	48-71	51-70
Plasticity index	17-27	20-48	19-30	21-40
Shrinkage limit	7.1-15.7		11-22	
Free swell%	20-40	11-45	28-38	22.5-45
Specific gravity	2.78-3.03	2.19-2.94	2.61-2.97	2.61-2.89
From plasticity chart	MH	CH,SC,MH,CL&SM	MH	MH,CH
UCT(qu)kpa	165-553		215-385	110.81-383.48

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

5.3 Discussions

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

1. It is attempted to check loss of water of hydration on soil samples under investigation. The difference in moisture contents between that of oven temperatures at 105⁰c and 50⁰c with maximum relative humidity 30% for all soils under investigation are below 4 %. Below 4% variation in moisture content indicates that the soils do not contain loosely bound water of hydration in significant amount. Hence one may use oven temperature 105⁰c for water content determination.
2. The effect of pretreatments were checked by conducting index tests on oven 105⁰c dried, air 50⁰c dried .The corresponding test results show that the values under different conditions prior to testing does not results significant variation. Laterites collected from distinct wet and dry seasons are not sensitive to pretreatment 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 little effect on the test results which are in agreement with their findings.
3. It is attempted to check loss of water of hydration on soil samples under investigation. The difference in moisture contents between that of oven temperatures at 105⁰c and 50⁰c with maximum relative humidity 30% for all soils under investigation are below 4 %. Below 4%

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5. .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 a concretionary material which is broken down by test manipulation. Force induced during test manipulation detaches the bond between particles due to the presence of sesquioxides. Hence one has to conduct limit tests with fresh samples for each point.
6. According to USCS, some soil samples fall under MH and the other is CH. Few research works were carried out on tropical soils both artificially combined and actual samples. Most of 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 lateritic soils using only USCS classification system does not shows clear and accurate information and almost no information about the mineral. Hence soil classification using USCS should be accompanied by mineralogical test results.
7. An activity test result for soil samples is 0.38-0.65 which is less than 0.75, indicating inactive soils. Lateritic soils are inactive or normal as the Sesquioxides suppress the activity of the clay particles.
8. The specific gravity test results are 2.61 to 2.89. 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).
9. Geochemical tests and index tests indicate that the soils of Dilla town are lateritic having high concentration of Iron Oxide and Aluminum Oxide (Sesqueoxide) .

CHAPTER SIX

6. Conclusions and Recommendations

6.1 Conclusions

Based on the test results investigation on the soil samples of Dilla town the following conclusions can be drawn:

- 1) Geochemical tests indicate that the soils of Dilla town are lateritic having high concentration of Iron Oxide and Aluminium Oxide / Sesqueoxide /. The degree of laterization, Silica to Sesqueoxide ratio, is between 1.33 and 2.0. No significant variations have been seen in different pits located at far distances with corresponding lithological classification category
- 2) The Dilla town soils indicate sensitive to test procedures for atterberg limit during testing manipulation. Hence desegregation results in the test value different from the in-situ condition. Accordingly, the minimum mixing time, usually of 5 minutes and fresh soil has to be used for each point on the Atterberg Limit.
- 3) From the gradation charts and the soil classifications made, Dilla town soil is a fine grained soil with characteristics somehow similar to Ferrisol soils. From the mode of formation and comparisons of gradation charts, soil could be grouped as Ferrisol soil.
- 4) According to plasticity chart, most soil samples subjected to fall below A-line under MH (inorganic silts with medium strength) and the rest are above A-line CH (inorganic clays of high plasticity).
- 5) 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.
- 6) From unconfined compression test, unconfined compression strength (q_u) values ranges from 110.81-383.48 Kpa. The sensitivity test result of the soil samples are group under little sensitive clay under remolding.
- 7) The results of consolidation test on lateritic soils indicates that over consolidated. The over consolidation phenomena in the residual soil is not due to the previous history, but mainly due to the cementation.

6.2 Recommendations

- 1) In this research samples of soil were collected only from ten test pits, by increasing the number of sampling area detailed profile investigation has to be carried out on soil samples on a localized area.
- 2) As can be seen from test results, the plasticity indices are greater than 20%. To use the laterite soils as a construction material for heavy traffic roads, the Atterberg limits should be reduced so as to satisfy the specification ($< PI=15\%$). For that purpose further research may be carried out on blending the soil material with non-plastic soil, lime and / or cement. 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.
- 3) Shear strength property of Dilla soil is not studied in this research in detail. Therefore, by studying the Shear strength characteristics soils using more appropriate test using Triaxial compression Test, Correlation of the index property with shear strength parameters may also be done.

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ANNEX – A

LABORATORY TESTING PROCEDURES

1. Moisture Content Test
2. Plasticity Tests
 - 2.1. Liquid Limit (Ll) Tests
 - 2.2. Plastic Limit (Pl) and Plasticity Index (Pi) Tests
3. Grain Size Analysis Tests
4. Specific Gravity Test
5. Free Swell
6. Unconfined Compression Test

1. Natural Moisture Content

General

Change in moisture content is the most influential parameter that affects the property of soils. Moisture content is defined as the ratio expressed as a percentage of mass of water to mass of soil solids. The moisture content test is carried out in laboratory as per the procedures of ASTM D 2216.

$$\omega = \frac{W_w}{W_s} \times 100 \text{-----} 1$$

Where

ω = moisture content of soil (expressed as percentage)

W_w = weight of water in soil sample (i.e. initial weight of moist soil minus weight of oven dried soil)

W_s = weight of soil solids in sample

Apparatus Required

- Balance (with accuracy to 0.01g)
- Oven (with accurate temperature control and temperature gage)
- Metal boxes/Containers (e.g., tin or aluminum moisture cans with lids)

Test sample

A representative sample of the moist soil to be tested should be taken in an amount as Indicated in the Table A-1 below.

Table A- 1 Minimum Mass of moist sample (ASTM, 2004)

Maximum particle size (100% passing)	Standard sieve Size	Recommended minimum mass of moist test specimen for water content reported to $\pm 0.1\%$	Recommended minimum mass of moist test specimen for water content reported to $\pm 1\%$
2.0 mm or less	No. 10	20 g	20g
4.75	No. 4	100g	20g
9.5	3/8-in.	500g	50g
19.0	3/4-in.	2.5kg	250g
37.5	1½-in.	10kg	1kg
75	3-in.	50kg	5kg

Test Procedure

1. Weigh container with representative wet sample in accordance with Table A-1 to nearest 0.01 gram
And record on data sheet
2. Uncover container and place in oven to dry (see Note 1.1)
3. Remove from oven, replace cover tightly, and set aside to cool to room temperature
4. Weigh container with dried sample to nearest 0.01 gram and record on data sheet
5. If tare weight is not already available, clean, dry and weigh container with cover to nearest 0.01 gram and record on data sheet (see Note 1.2)
6. Compute moisture content as shown on data sheet and in sample calculation using the above equation (Eq. 1)

Note 1.1:

Two test portions should be prepared for moisture-content determinations.

One portion should be oven-dried at 105 °C until successive test weightings show that no further weight loss is taking place or insignificant change (i.e., less than about 0.1%), and the moisture content should then be determined.

The other portion should be air-dried or oven-dried at no more than 50 °C with a maximum relative humidity (RH) of 30% until successive test weightings show no further weight loss, and the moisture content then determined.

Note 1.2:

It is recommended that all containers used for moisture-content tests be numbered and their weights obtained in advance and recorded for reference.

Sample calculation (*Trial No TPI-1A*)

1. To calculate the weight of water in sample

$$\text{Wt water} = (\text{wt wet sample} + \text{can}) - (\text{wt dry sample} + \text{can}) = 63.5 - 55.9 = 7.6\text{gm}$$

2. To calculate the weight of dry sample

$$\text{Wt dry sample} = (\text{wt dry sample} + \text{can}) - (\text{wt can}) = 55.9 - 33 = 22.9\text{gm}$$

3. To calculate moisture content

$$\text{Moisture content} = 100 \times \text{wt water} / \text{Wt dry sample} = 100 \times 7.6 / 22.9 = 33.19\%$$

Table A- 2 Moisture content test data sheet

Sample Depth, m : 1.5
 Sample Designation : TP1-1(105 °c)

<i>Trial No</i>	<i>TP1-1A</i>	<i>TP1-1B</i>
<i>Container No</i>	<i>169</i>	<i>79</i>
<i>Mass of container, g</i>	<i>33.00</i>	<i>33.10</i>
<i>Mass of container + Wet soil, g</i>	<i>63.50</i>	<i>63.40</i>
<i>Mass of container + Dry soil, g</i>	<i>55.90</i>	<i>56.00</i>
<i>Mass of water, g</i>	<i>7.60</i>	<i>7.40</i>
<i>Mass of dry soil, g</i>	<i>22.90</i>	<i>22.90</i>
<i>Water content, %</i>	<i>33.19</i>	<i>32.31</i>
<i>Ave. moisture content, % =</i>	<i>32.75</i>	

Sample calculation (*Trial No TP1-1A*)

1. To calculate the weight of water in sample

$$\text{Wt water} = (\text{wt wet sample} + \text{can}) - (\text{wt dry sample} + \text{can}) = 62.9 - 56.50 = 6.4\text{gm}$$

2. To calculate the weight of dry sample

$$\text{Wt dry sample} = (\text{wt dry sample} + \text{can}) - (\text{wt can}) = 56.5 - 33.7 = 22.8\text{gm}$$

3. To calculate moisture content

$$\text{Moisture content} = 100 \times \text{wt water} / \text{Wt dry sample} = 100 \times 6.4 / 22.8 = 28.07\%$$

Sample Designation : TP1-1(1.5m) (50 °c)

Trial No	TP1-1A	TP1-1B
Container No	143	160
Mass of container, g	33.70	33.80
Mass of container + Wet soil, g	62.90	69.65
Mass of container + Dry soil, g	56.50	61.40
Mass of water, g	6.4	8.25
Mass of dry soil, g	22.80	27.60
Water content, %	28.07	29.89
<i>Ave. moisture content, %</i> =	28.98	

TP1-2(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>32.80</i>	<i>33.80</i>
<i>Mass of container + Wet soil, g</i>	<i>59.70</i>	<i>54.80</i>
<i>Mass of container + Dry soil, g</i>	<i>53.50</i>	<i>50.10</i>
<i>Mass of water, g</i>	<i>6.20</i>	<i>4.70</i>
<i>Mass of dry soil, g</i>	<i>20.70</i>	<i>16.30</i>
<i>Water content, %</i>	<i>29.95</i>	<i>28.83</i>
<i>Ave. moisture content, %</i> =	<i>29.39</i>	

TP1-2(50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.20</i>	<i>32.70</i>
<i>Mass of container + Wet soil, g</i>	<i>52.40</i>	<i>59.60</i>
<i>Mass of container + Dry soil, g</i>	<i>48.10</i>	<i>53.60</i>
<i>Mass of water, g</i>	<i>4.30</i>	<i>6.00</i>
<i>Mass of dry soil, g</i>	<i>14.90</i>	<i>20.90</i>
<i>Water content, %</i>	<i>28.86</i>	<i>28.71</i>
<i>Ave. moisture content, %</i> =	<i>28.78</i>	

TP2-1(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.40</i>	<i>33.40</i>
<i>Mass of container + Wet soil, g</i>	<i>74.90</i>	<i>74.90</i>
<i>Mass of container + Dry soil, g</i>	<i>63.90</i>	<i>63.90</i>
<i>Mass of water, g</i>	<i>11.00</i>	<i>11.00</i>
<i>Mass of dry soil, g</i>	<i>30.50</i>	<i>30.50</i>
<i>Water content, %</i>	<i>36.07</i>	<i>36.07</i>
<i>Ave. moisture content, % =</i>	<i>36.07</i>	

TP2-1(+50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.60</i>	<i>33.60</i>
<i>Mass of container + Wet soil, g</i>	<i>67.00</i>	<i>67.00</i>
<i>Mass of container + Dry soil, g</i>	<i>58.20</i>	<i>58.20</i>
<i>Mass of water, g</i>	<i>8.80</i>	<i>8.80</i>
<i>Mass of dry soil, g</i>	<i>24.60</i>	<i>24.60</i>
<i>Water content, %</i>	<i>35.77</i>	<i>35.77</i>
<i>Ave. moisture content, % =</i>	<i>35.77</i>	

TP2-2(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.60</i>	<i>33.20</i>
<i>Mass of container + Wet soil, g</i>	<i>65.50</i>	<i>78.80</i>
<i>Mass of container + Dry soil, g</i>	<i>57.40</i>	<i>67.40</i>
<i>Mass of water, g</i>	<i>8.10</i>	<i>11.40</i>
<i>Mass of dry soil, g</i>	<i>23.80</i>	<i>34.20</i>
<i>Water content, %</i>	<i>34.03</i>	<i>33.33</i>
<i>Ave. moisture content, % =</i>	<i>33.68</i>	

TP2-2(50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.10</i>	<i>33.10</i>
<i>Mass of container + Wet soil, g</i>	<i>66.50</i>	<i>55.30</i>
<i>Mass of container + Dry soil, g</i>	<i>58.10</i>	<i>49.90</i>
<i>Mass of water, g</i>	<i>8.40</i>	<i>5.40</i>
<i>Mass of dry soil, g</i>	<i>25.00</i>	<i>16.80</i>
<i>Water content, %</i>	<i>33.60</i>	<i>32.14</i>
<i>Ave. moisture content, % =</i>	<i>32.87</i>	

TP3-1(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.40</i>	<i>33.40</i>
<i>Mass of container + Wet soil, g</i>	<i>79.80</i>	<i>79.80</i>
<i>Mass of container + Dry soil, g</i>	<i>68.80</i>	<i>68.80</i>
<i>Mass of water, g</i>	<i>11.00</i>	<i>11.00</i>
<i>Mass of dry soil, g</i>	<i>35.40</i>	<i>35.40</i>
<i>Water content, %</i>	<i>31.07</i>	<i>31.07</i>
<i>Ave. moisture content, % =</i>	<i>31.07</i>	

TP3-1(50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>32.70</i>	<i>32.70</i>
<i>Mass of container + Wet soil, g</i>	<i>69.50</i>	<i>69.50</i>
<i>Mass of container + Dry soil, g</i>	<i>61.70</i>	<i>61.70</i>
<i>Mass of water, g</i>	<i>7.80</i>	<i>7.80</i>
<i>Mass of dry soil, g</i>	<i>29.00</i>	<i>29.00</i>
<i>Water content, %</i>	<i>26.90</i>	<i>26.90</i>
<i>Ave. moisture content, %</i> =	<i>26.90</i>	

TP3-2(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.30</i>	<i>32.80</i>
<i>Mass of container + Wet soil, g</i>	<i>59.70</i>	<i>60.20</i>
<i>Mass of container + Dry soil, g</i>	<i>53.10</i>	<i>53.20</i>
<i>Mass of water, g</i>	<i>6.60</i>	<i>7.00</i>
<i>Mass of dry soil, g</i>	<i>19.80</i>	<i>20.40</i>
<i>Water content, %</i>	<i>33.33</i>	<i>34.31</i>
<i>Ave. moisture content, %</i> =	<i>33.82</i>	

TP3-2(50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.70</i>	<i>33.40</i>
<i>Mass of container + Wet soil, g</i>	<i>58.90</i>	<i>54.80</i>
<i>Mass of container + Dry soil, g</i>	<i>52.70</i>	<i>49.80</i>
<i>Mass of water, g</i>	<i>6.20</i>	<i>5.00</i>
<i>Mass of dry soil, g</i>	<i>19.00</i>	<i>16.40</i>
<i>Water content, %</i>	<i>32.63</i>	<i>30.49</i>
<i>Ave. moisture content, %</i> =	<i>31.56</i>	

TP5-1(105)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>32.90</i>	<i>32.90</i>
<i>Mass of container + Wet soil, g</i>	<i>76.40</i>	<i>76.40</i>
<i>Mass of container + Dry soil, g</i>	<i>65.30</i>	<i>65.30</i>
<i>Mass of water, g</i>	<i>11.10</i>	<i>11.10</i>
<i>Mass of dry soil, g</i>	<i>32.40</i>	<i>32.40</i>
<i>Water content, %</i>	<i>34.26</i>	<i>34.26</i>
<i>Ave. moisture content=</i>	<i>34.26</i>	

TP5-1(50)

<i>Trial No</i>	<i>1</i>	<i>2</i>
<i>Container No</i>	<i>12</i>	<i>1</i>
<i>Mass of container, g</i>	<i>33.60</i>	<i>33.70</i>
<i>Mass of container + Wet soil, g</i>	<i>65.20</i>	<i>65.20</i>
<i>Mass of container + Dry soil, g</i>	<i>57.60</i>	<i>57.60</i>
<i>Mass of water, g</i>	<i>7.60</i>	<i>7.60</i>
<i>Mass of dry soil, g</i>	<i>24.00</i>	<i>23.90</i>
<i>Water content, %</i>	<i>31.67</i>	<i>31.80</i>
<i>Ave. moisture content,% =</i>	<i>31.73</i>	

2. Atterberg Limits

2.1 Liquid Limit:

The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic state and principally it is defined as the water content at which the soil pat cut using standard groove closes for about a distance of 13cm (1/2 in.) at 25 blows of the liquid limit machine (Casagrande Apparatus). The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit test is carried out in accordance of test procedures of ASTM D 4318. A soil containing high water content is in the liquid state and it offers no shearing resistance.

Test Procedures

Dryly prepared samples (Samples prepared using Dry method)

- 1) Transfer enough material from mixing dish into brass cup of liquid-limit device to fill the cup about one-third full.
- 2) Hold cup and use spatula to mix and spread material, forming a smooth cake about 1/2-inch deep in lower half of cup.
- 3) Hold grooving tool with rounded edges downward, index finger against flat face.
- 3) Place point of tool on upper edge of soil cake with tool perpendicular to surface of cup.
- 4) Draw tool down through the center of cake along a line extending from the center of the cup handle. At the same time, tilt the tool so it remains perpendicular to the bottom surface (see Note 2.8).
- 5) Clean grooving tool by wiping thumb over cutting edge before laying it aside. If soil dries on tool, time is lost in cleaning it later.
- 6) With striking parts of the liquid-limit device clean and dry and height of drop properly adjusted, place cup in device, turn crank at rate of about two blows per second, and count number of blows required to close bottom of groove for a distance of about 1/2-inch (see Note 2.9).
- 7) remove a slice of soil approximately the width of the spatula, extending from edge to edge of the soil cake at right angles to the groove and including the portion of the groove in which the soil flowed together. Then place soil in small weighted metal box and cover tightly.

- 8) Remove cup from device, remix to mixing dish and add a slight amount of water to increase the water content of the soil and decrease the number of blows required to close the groove. And, repeat steps (1) through
- (9) For at least two additional trials producing successively lower number of blows to close the groove. One of the trials shall be for a closure requiring 25 to 35 blows, one trial for a closure between 20 to 30 blows, and one trial for a closure requiring 15 to 25 blows.
- 10) Determine moisture content of all samples obtained during test according to the moisture content test shown above. (Cans must be properly numbered and recorded).

Wetly prepared samples (Samples prepared using wet method)

- 1) Transfer enough material from mixing dish, wetly prepared test sample, into brass cup of liquid-limit device to fill the cup about one-third full (see Note 2.7).
- 2) Hold cup and use spatula to mix and spread material, forming a smooth cake about ½-inch deep in lower half of cup.
- 3) Hold grooving tool with rounded edges downward, index finger against flat face.
- 4) Place point of tool on upper edge of soil cake with tool perpendicular to surface of cup.
- 5) Draw tool down through the center of cake along a line extending from the center of the cup handle. At the same time, tilt the tool so it remains perpendicular to the bottom surface (see Note 2.8).
- 6) Clean grooving tool by wiping thumb over cutting edge before laying it aside. If soil dries on tool, time is lost in cleaning it later.
- 7) With striking parts of the liquid-limit device clean and dry and height of drop properly adjusted, place cup in device, turn crank at rate of about two blows per second, and count number of blows required to close bottom of groove for a distance of about ½-inch (see Note 2.10).
- 8) Remove a slice of soil approximately the width of the spatula, extending from edge to edge of the soil cake at right angles to the groove and including the portion of the groove in which the soil flowed together. Then place soil in small weighted metal box and cover tightly.
- 9) Remove cup from device, remix to mixing dish and air-dry the sample slowly to decrease the water content of the soil and increase the number of blows required to close the groove. This is to perform the blow count determination for different moisture contents as the soil slowly dries out. And, repeat steps (a) through for at least two additional trials producing successively higher number of blows to close the groove. One of the trials shall be for a closure requiring 15 to 25 blows, one trial for a closure between 20 to 30 blows, and one trial for a closure requiring 25 to 35 blows.

10) Determine moisture content of all samples obtained during test according to the moisture content test shown above. (Cans must be properly numbered and recorded).

Note 2.7:

For the case of wet samples, i.e. for the test samples which were prepared according to *wet method* shown above in the “Preparation of Test Sample”. Initially the samples should be air-dried to the consistency of a thick, smooth/uniform paste having a consistency within the 15 to 25 blows range.

Note 2.8:

For clays containing little or no sand, cut groove with one smooth continuous stroke. However, for very sandy clays, silts having little plasticity, and some organic soils, the tool cannot be drawn through sample without tearing sides of groove. For these soils, cut groove with a spatula and check dimensions with tool.

Note 2.9:

If groove is not closed ½-inch in 25 to 35 blows initially, i.e. for the first point, add water and remix or dry sample to a consistency with in this range.

Note 2.10:

If groove is not closed ½-inch in 15 to 25 blows initially, i.e. for the first point, add water and remix or dry sample to a consistency with in this range.

Data collection

As indicated previously, the liquid-limit is that the water content of the soil at which the standard groove is closed a distance ½ in. by exactly 25 drops. This is determined by plotting, on semi logarithmic graph paper, water content along the ordinate (arithmetical scale) versus number of drops along the abscissa (logarithmic scale) and drawing the best straight line through the plotted points. The resulting graph is referred to as a flow curve. From the flow curve, the liquid limit can be read as the water content at 25 drops.

From the preceding, the data to be collected for each trial are *the number of drops (N) required closing the groove and the water content (ω) of the soil at that number of drops*. Recalling section A that, in order to determine water content, the weight of the container (w_c), weight of container plus wet soil (w_1), and weight of container plus oven-dried soil (w_2) must be obtained. Since more than one water contents must be determined, each container should be properly labeled or numbered and recorded. Generally the following data should be collected for each trial in this test:

- Number of drops (N)
- Container identification number
- Weight of container, w_c
- Weight of container plus moist soil, w_1
- Weight of container plus oven-dried soil, w_2

Plastic Limit:

The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begins to crumble when rolled into a thread of soil of 3.0mm diameter. The conventional plastic limit test is carried out as per the procedure of ASTM D 4318. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded.

Test Procedure

- 1) Take not more than one-half of this sample and roll with palm of hand on any clean, smooth surface such as a sheet of paper, to form a 1/8-inch thread about 3 inches long.
- 2) Fold thread and press together into a lump and then roll it out again.
- 3) Repeat step (2), gradually reducing moisture content by evaporation as the sample is handled causing threads to become stiffer.
- 4) The plastic limit (PL) is reached when the thread crumbles into a number of pieces while being rolled (see Note 2.11).
- 5) Immediately place the crumbled thread into a small metal box and determine moisture content.
- 6) Make another PL determination for a check, using material from remaining portion of original sample.

Note B.11: If any doubt exists as to whether the PL has been reached, lump the pieces together and roll out again.

Data sheet

The same data sheet is used for both the plastic-limit and liquid-limit tests as given below, i.e. Table A - 3.

Plasticity Index

The numerical difference between the liquid limit (LL) and the plastic limit (PL) is then plasticity index (PI).

$$PI = LL - PL \text{ -----(A-2)}$$

Table A- 3 Plasticity test data sheet

Sample No : TP1-1(105 °c)		Sample Depth(m)					
		Liquid Limit				Plastic Limit	
Trial No		1	2	3	4	1	2
Container No		180	108	178	160	91	197
Mass of container, g		33.60	33.70	33.40	33.40	33.30	33.60
Mass of container + Wet soil, g		47.80	46.00	43.90	42.80	40.90	41.00
Mass of container + Dry soil, g		42.40	41.30	39.80	39.00	39.30	39.35
Mass of water, g		5.40	4.70	4.10	3.80	1.60	1.65
Mass of dry soil, g		8.80	7.60	6.40	5.60	6.00	5.75
Water content, %		61.36	61.84	64.06	67.86	26.67	28.70
No of blows		32	28	21	16	-----	-----

Liquid Limit, % = 63 Plastic Limit, % = 28 PI, %= 35

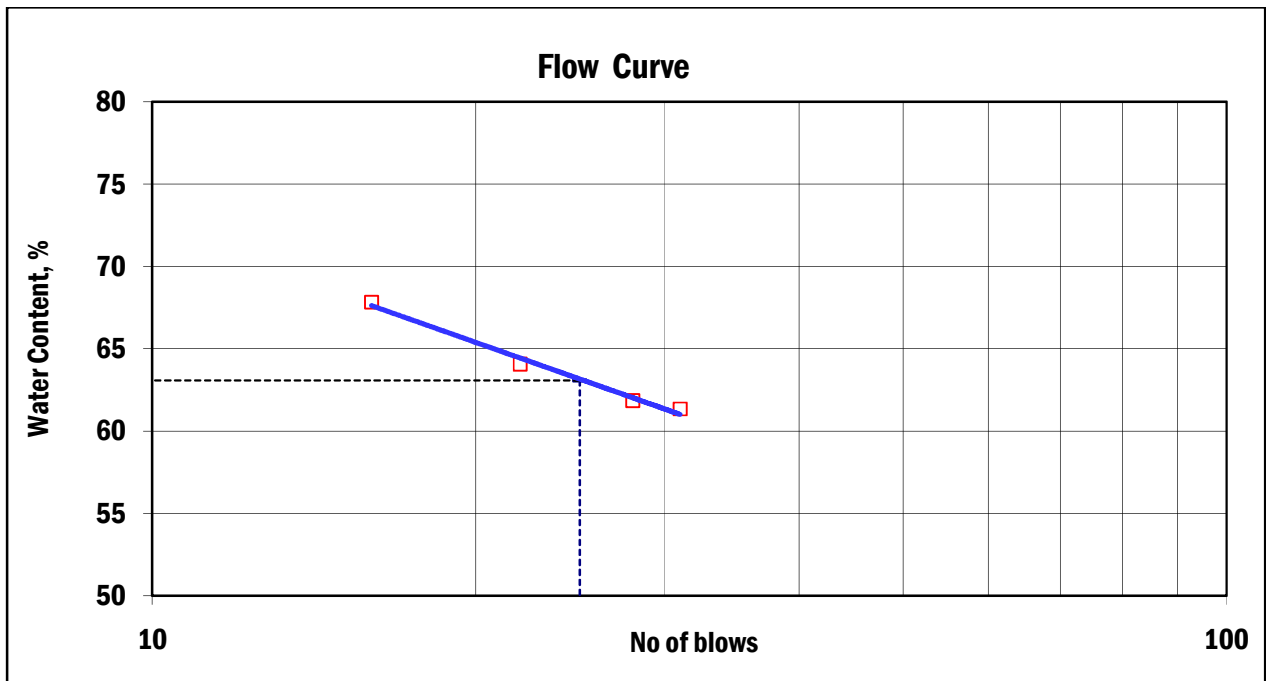
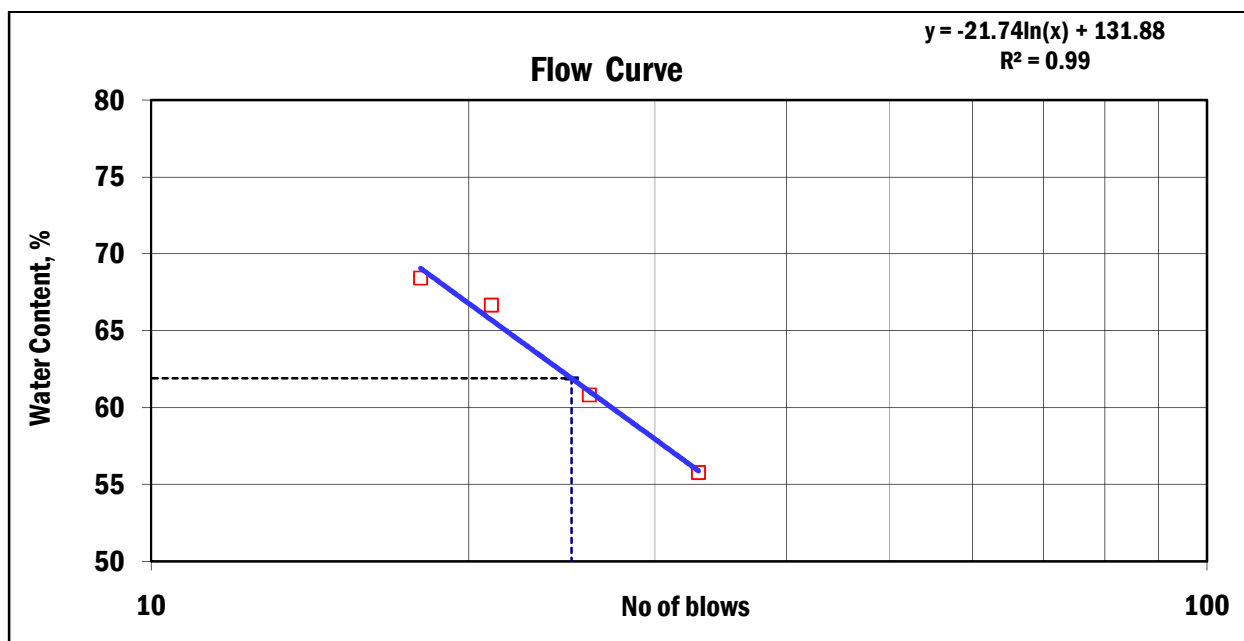


Figure A- 4 Flow curve

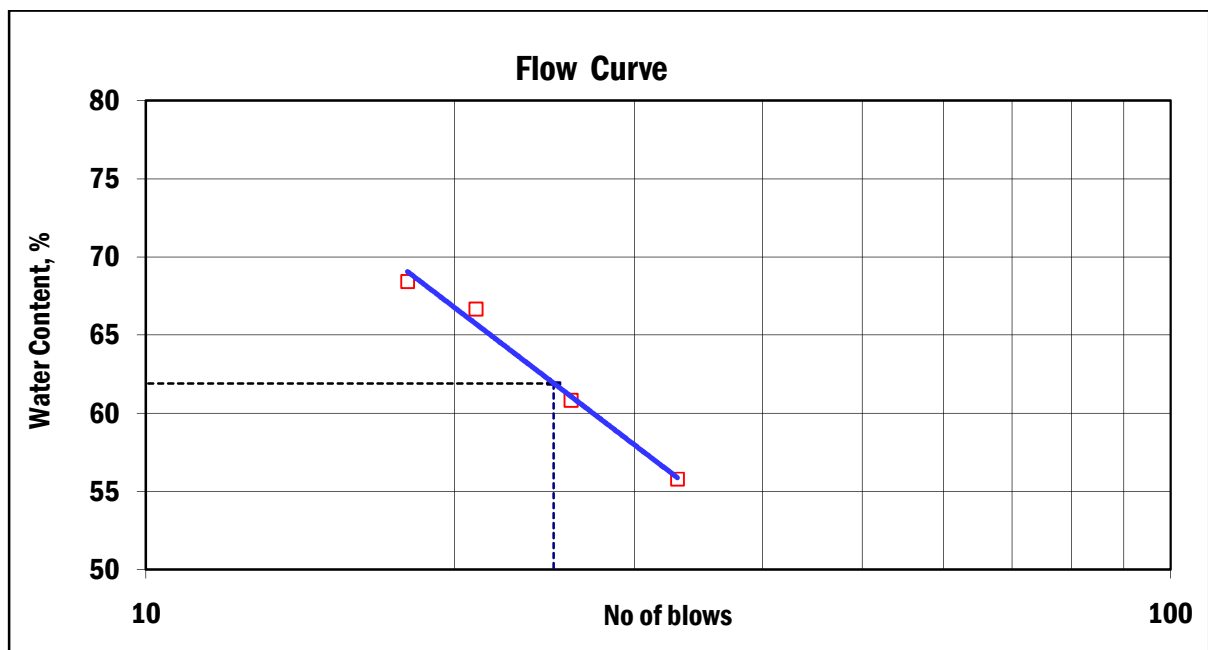
Sample No :	TP3-1(50)				Sample Depth, m :	1.5	
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	
Container No	180	108	178	160	91	197	
Mass of container, g	33.00	33.00	33.40	33.70	33.30	32.90	
Mass of container + Wet soil, g	41.10	40.80	41.40	43.30	41.10	40.30	
Mass of container + Dry soil, g	38.20	37.85	38.20	39.40	39.45	38.70	
Mass of water, g	2.90	2.95	3.20	3.90	1.65	1.60	
Mass of dry soil, g	5.20	4.85	4.80	5.70	6.15	5.80	
Water content, %	55.77	60.82	66.67	68.42	26.83	27.59	
No of blows	33	26	21	18	-----	-----	

Liquid Limit, % = 62 Plastic Limit, % = 27 PI, % = 35



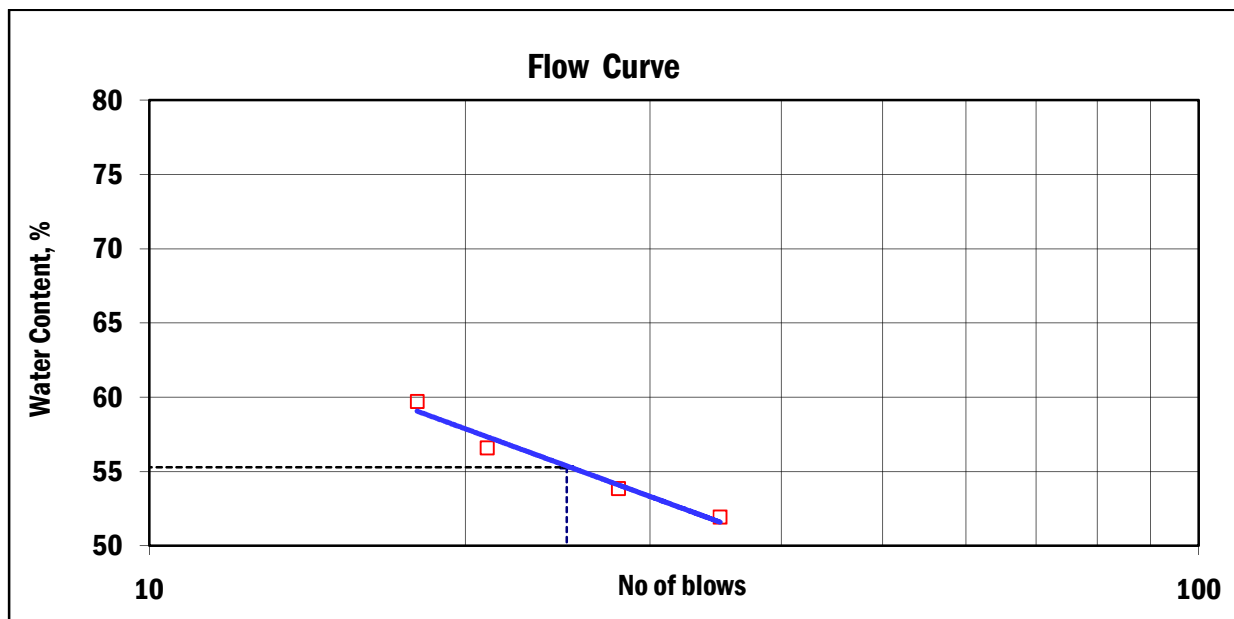
Sample No :	TP5-1(50)				Sample Depth, m :	1.5	
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	
Container No	163	143	65	91	105	155	
Mass of container, g	33.60	33.70	33.40	33.40	33.30	33.60	
Mass of container + Wet soil, g	47.80	46.00	43.90	42.80	40.80	41.00	
Mass of container + Dry soil, g	42.40	41.30	39.80	39.00	39.10	39.20	
Mass of water, g	5.40	4.70	4.10	3.80	1.70	1.80	
Mass of dry soil, g	8.80	7.60	6.40	5.60	5.80	5.60	
Water content, %	61.36	61.84	64.06	67.86	29.31	32.14	
No of blows	34	29	22	16	-----	-----	

Liquid Limit, % = 64 Plastic Limit, % = 31 PI, % = 33



Sample No :	TP10-1(50)				Sample Depth, m :	1.5	
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	
Container No	A1	A2	A3	F1	A302	A115	
Mass of container, g	33.00	32.80	32.80	32.90	33.30	33.60	
Mass of container + Wet soil, g	40.90	40.80	44.70	43.60	41.75	42.80	
Mass of container + Dry soil, g	38.20	38.00	40.40	39.60	39.60	40.70	
Mass of water, g	2.70	2.80	4.30	4.00	2.15	2.10	
Mass of dry soil, g	5.20	5.20	7.60	6.70	6.30	7.10	
Water content, %	51.92	53.85	56.58	59.70	34.13	29.58	
No of blows	35	28	21	18	-----	-----	

Liquid Limit, % = 55 Plastic Limit, % = 32 PI, % = 23



3. GRAIN SIZE ANALYSIS TESTS

The determination of grain-size distribution in a soil, called grain-size analysis, is made by a screening

Process (sieve analysis) for coarse grained soils (for particle sizes larger 75- μm), and by settling/sedimentation process in water (i.e. wet mechanical analysis), using a hydrometer, for fine-grained soils (for particle sizes smaller than 75- μm). When both processes are used on the same sample, the test is called combined grain size analysis.

Because of the tendency of some lateritic soils to form aggregations while drying at higher temperatures, a sieve analysis should be made on the sample after the fines are removed by washing. In addition to this, relatively weak coarse particles may also cause problems in grading analysis as they become finer during sample preparation i.e. during pulverization. Hence, it is important that the sample preparation and testing procedures should not fracture the coarse particles. The Grain size Analysis Test is carried out in laboratory as per the procedures of (ASTM D 422-63).

Preparation of Test Sample

1) Two samples shall be selected by the method of quartering or by use of a sample splitter (one for the gradation test and the other for moisture content).

2) The gradation sample shall weigh, in its natural moisture conditions, not less than the amount indicated in the following table.

Table A- 5 Minimum Weight of gradation sample (Lyon Associates, 1971)

Nominal Maximum sieve size	Approximate Minimum Weight of Sample, kg
No. 4 (4.75-mm)	1
½-in. (14.25-mm)	2
¾-in. (19.0-mm)	3
1-in. (25.0-mm)	4
1½-in. (37.5-mm) or over	5

3) The moisture sample shall weigh at least 30 per cent of the gradation sample.

4) Weigh the gradation sample in its wet condition and place in a container. Add sufficient water to the sample in the container to cover it.

- 4) Prepare a nest of two sieves with a No. 200 (0.075-mm) sieve on the bottom and a No. 10 (2.0-mm) sieve on the top.
- 6) Transfer the sample to the nested sieves and wash with running water. When the sample is larger than can be handled at one time on the nested sieves, wash a portion of the sample and transfer it to a container in which it is to be dried (see Note 3.1).
- 7) Dry and combine the washed materials retained on the nested No. 10 & No. 200 sieves to a constant weight at a temperature not to exceed 110°C .

Note 3.1:

Tapping of the sieves expedites the washing procedure (see Note 3.2).

Note 3.2:

The dry weight to be used in the computation for the gradation shall be computed using the initial wet weight and the moisture content of the sample, i.e. the dry weight is equal to wet-weight divided by $(1 + \text{moisture content})$.

Test Procedures

Sieve Analysis of portion retained on No. 10 & No. 200 sieves

- 1) Transfer dry sample to set of sieves (coarse sieves at top, fine sieves at bottom) with the sieve pan on bottom. Place cover on top and shake the stack vigorously with a horizontal rotating motion. The sieves may be jarred occasionally by dropping lightly on several thick nesses of magazines. Do not drop directly on work bench or table, this will damage sieves. The shaking period depends on the amount of fine material in the sample, but should not be less than 15 minutes for most fine-grained soils (see Note 3.3).
- 2) Transfer material from largest screen to pan of balance and weight to nearest 5 grams. Record size of largest particle. Place material in separate container and save until test is completed. For sample weighting less than 50 grams, weigh each fraction on a balance to nearest gram.
- 3) Repeat procedure for each successively smaller screen size. Individual stone particles caught in the wire screens should not be forced through screens but should be removed by hand and included with fraction before weighing. The finer sieves should be inverted over the scale pan and brushed clean.

Note 3.3:

Large samples containing many coarse stones should first be screened through the portable screen set. The number and size of screens and sieves to be used depends on the type of soil tested and the purpose of the test.

Sample calculation

Table A- 6 Sieve analysis data sheet

Depth of Sampling: 3m**Sample designation TP5-2 (50°c)**

Sieve Analysis				Total mass of sample, 500g			
Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%) (5/Ms)*100	Cum. Percentage Retained (%)	Perc. Passing (%)
1	2	3	4	5=4-3	6	7	8=100-(7)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	716.0	716.0	0.0	0.0	0.0	100.0
1/2"	12.5	584.0	584.0	0.0	0.0	0.0	100.0
3.8"	9.5	586.0	586.0	0.0	0.0	0.0	100.0
No 4	4.75	435.5	435.5	0.0	0.0	0.0	100.0
No 8	2.36	440.1	440.8	0.7	0.1	0.1	99.9
No 10	2	442.4	442.6	0.2	0.04	0.18	99.8
No 16	1.18	402.6	405.1	2.5	0.5	0.7	99.3
No 30	0.6	379.7	400.1	20.4	4.1	4.8	95.2
No 50	0.3	338.0	365.1	27.1	5.4	10.2	89.8
No 100	0.15	340.7	352.9	12.2	2.4	12.6	87.4
No 200	0.075	302.7	308.9	6.2	1.2	13.9	86.1
pan	--	470.7	470.8	0.1	0.0	13.9	-----

Where ms= total soil weight

- Percentage retained on any sieve = $\frac{\text{weight of soil retained}}{\text{total soil weight}} * 100\%$
- Cumulative percentage retained on any sieve = Sum of percentage retained on all any coarser sieves.

- Percentage finer than any sieve size = 100 percent minus cumulative percentage retained

Hydrometer analysis

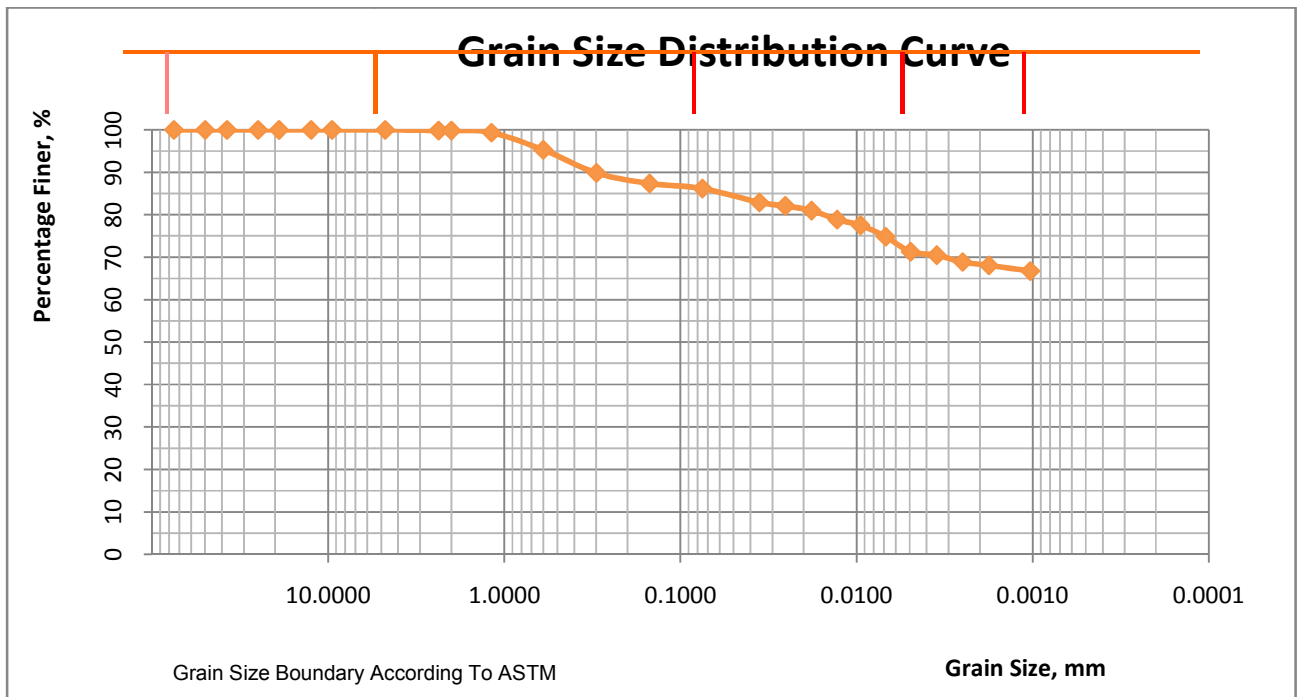
Laboratory testing procedure required for the analysis of grain sizes less than 0.075mm in diameter (passing No.200 sieve) is not included in the set. This test can be done using the step-by-step test procedures described in ASTM D 422-63.

- 1.50 gm of soil passing sieve No. 200 was taken.
- 2.Weight of 5 gm dispersing agent was weighted.
- 3.The soil and dispersing agent was placed in a bottle half filled with distilled water.
- 4.The suspension was mixed thoroughly 10 minute by shaking and let the soil soak for 12h.
- 5.The suspension placed in a graduated jar and enough water was added to bring the level to 1000 cm³
- 6.The soil and water was mixed in the graduated cylinder by placing the palm of hand over the open end graduated cylinder was turning up side down and back for a period of 1 min
- 7.The graduated cylinder was placed on the table and the stop watch was started immediately.
- 8.The hydrometer reading and test temperature at total elapsed time of 2, 4, 8, 15, 30, 60, 120, 240, 480, 960 and 1440 min was taken.

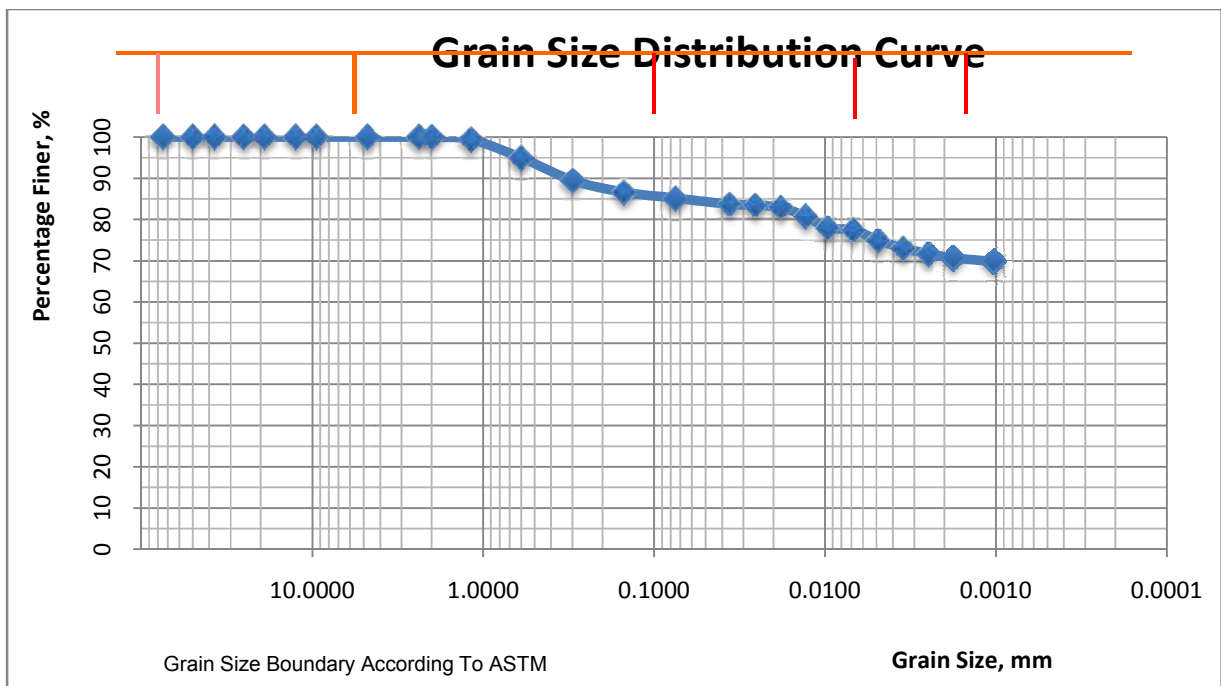
Hydrometer Analysis

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Test Temperature, °C		
						Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
3/4	1.0335	-0.0027	1.0308	7.44	0.01307	0.0356	96.21	82.87
1	1.0332	-0.0027	1.0305	7.52	0.01307	0.0253	95.27	82.07
2	1.0328	-0.0027	1.0301	7.62	0.01307	0.0180	94.02	80.99
4	1.0320	-0.0027	1.0293	7.84	0.01307	0.0129	91.52	78.84
8	1.0315	-0.0027	1.0288	7.97	0.01307	0.0095	89.96	77.49
15	1.0305	-0.0027	1.0278	8.23	0.01307	0.0068	86.84	74.80
30	1.0299	-0.0027	1.0272	8.39	0.01307	0.0049	84.96	73.19
60	1.0292	-0.0027	1.0265	8.58	0.01307	0.0049	82.78	71.30
120	1.0289	-0.0027	1.0262	8.66	0.01307	0.0035	81.84	70.50
240	1.0283	-0.0027	1.0256	8.81	0.01307	0.0025	79.96	68.88
480	1.0280	-0.0027	1.0253	8.89	0.01307	0.0018	79.03	68.07
1440	1.0275	-0.0027	1.0248	9.03	0.01307	0.0010	77.47	66.73

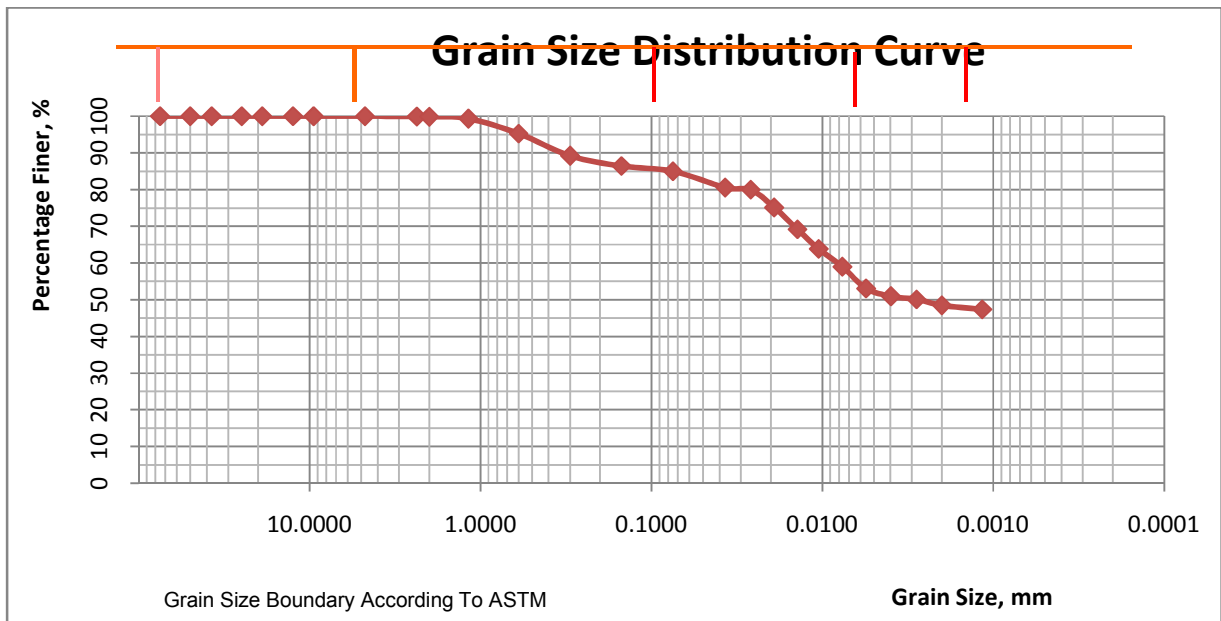
Figure A - 6 Grain-Size distribution curve



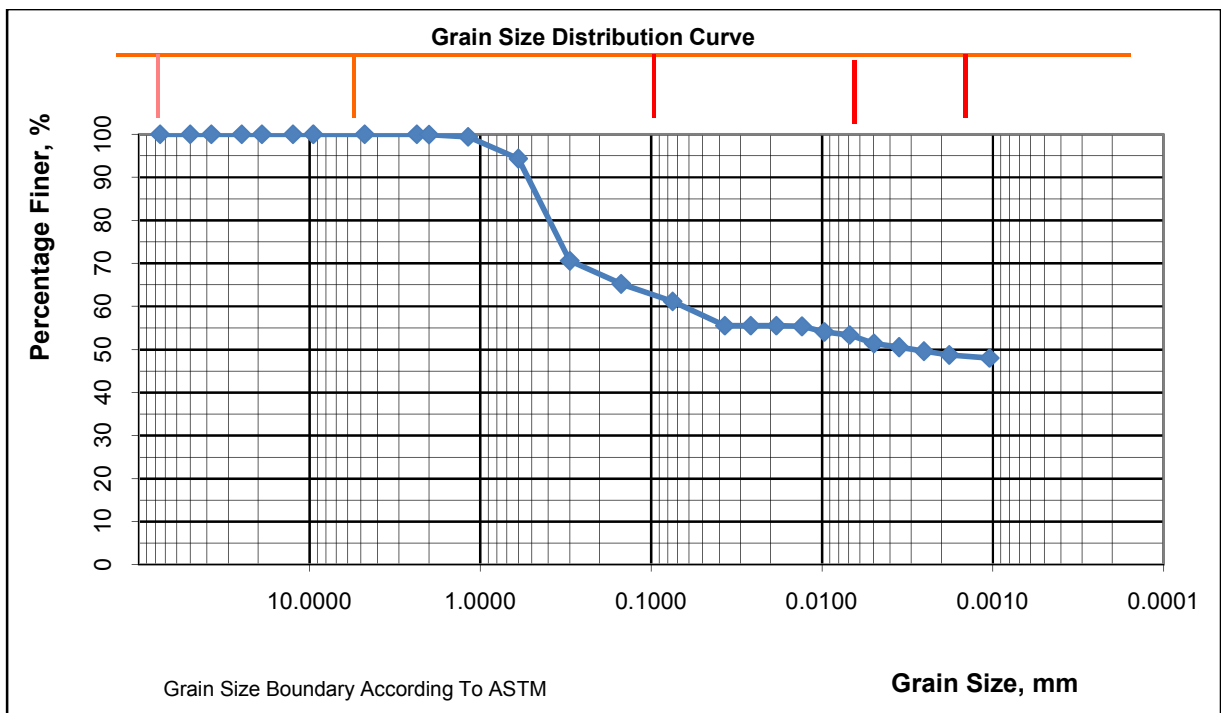
TPI-1(105 °c) ----- depth 1.5 m



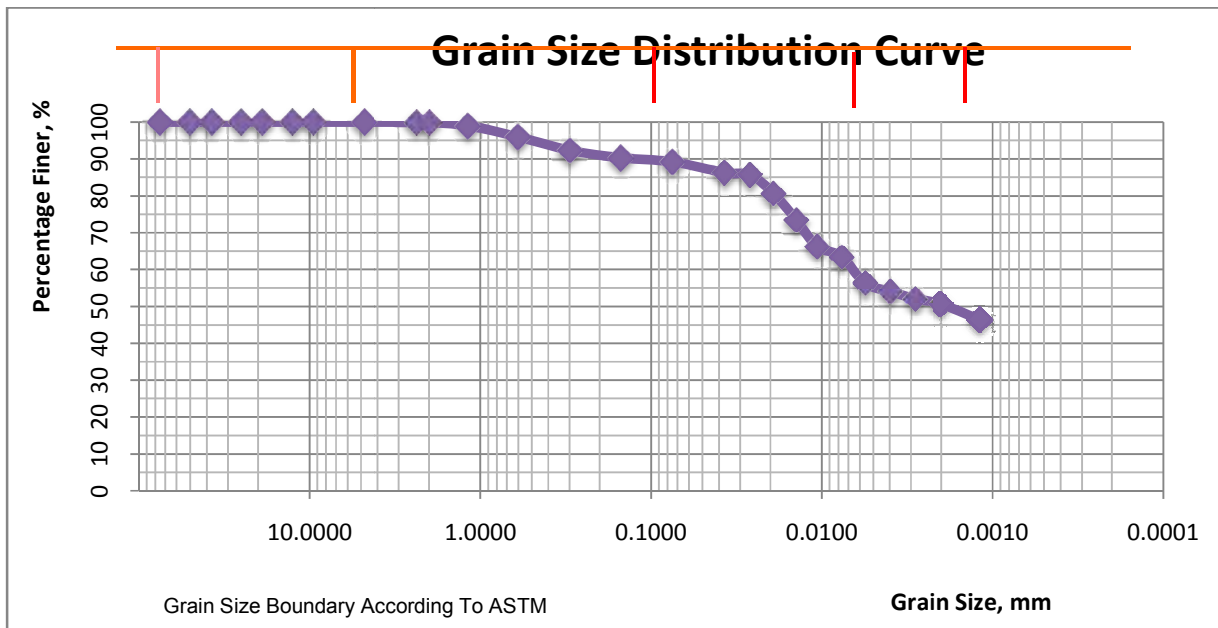
TPI-2(105 °c) ----- depth 3 m



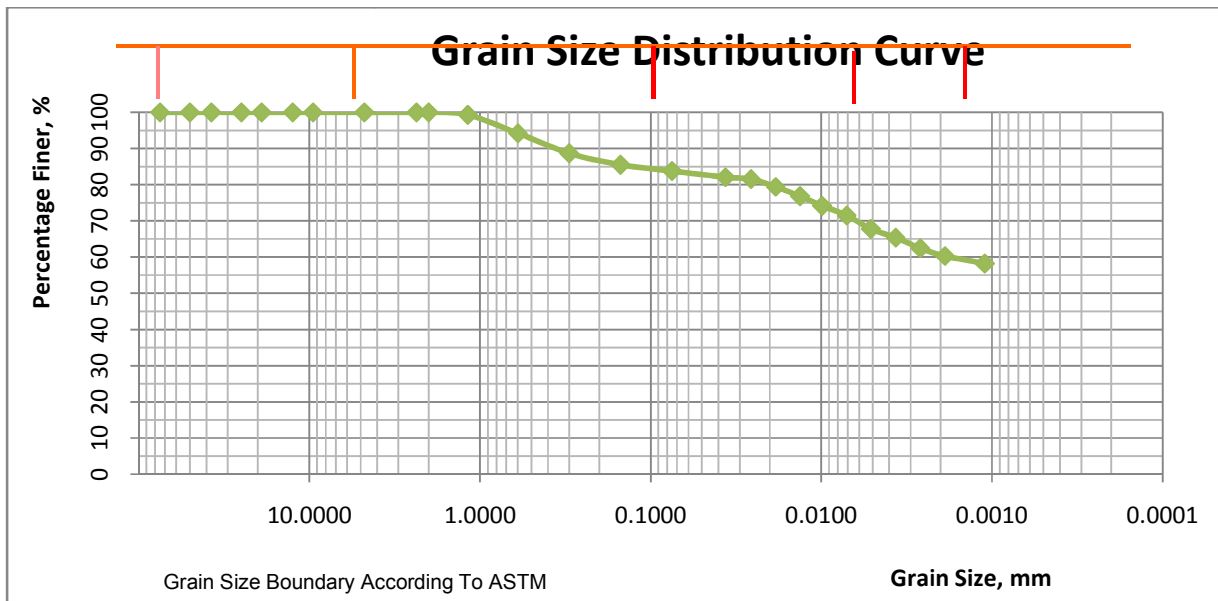
TP2-1(105 °c)----- depth 1.5 m



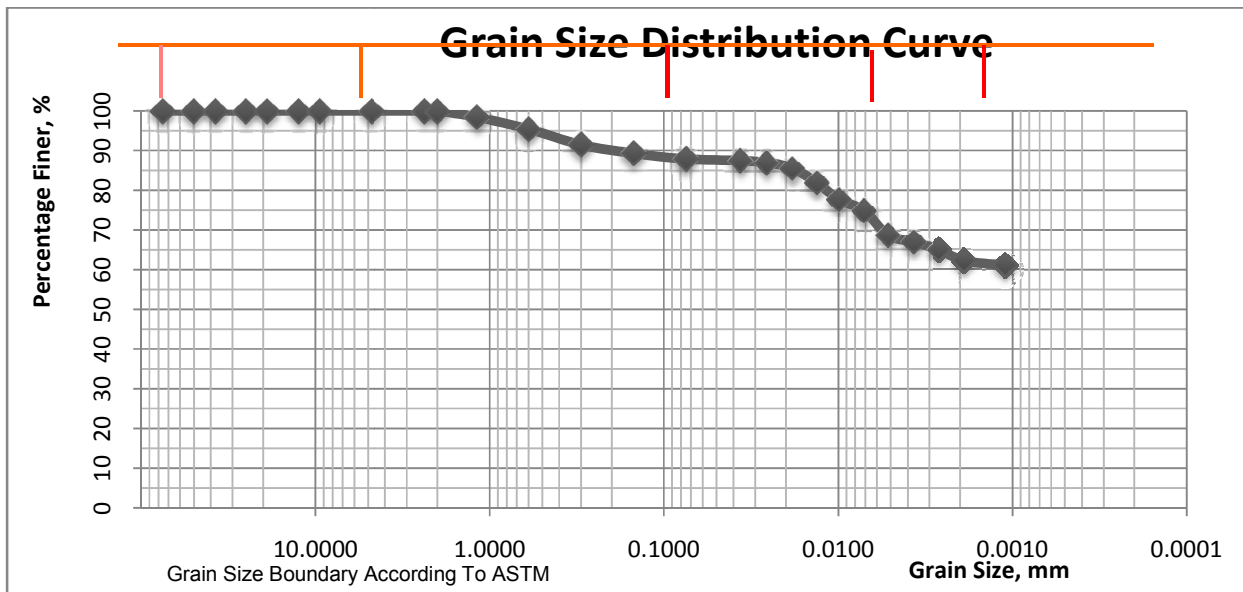
TP2-2(105 °c)----- depth 3 m



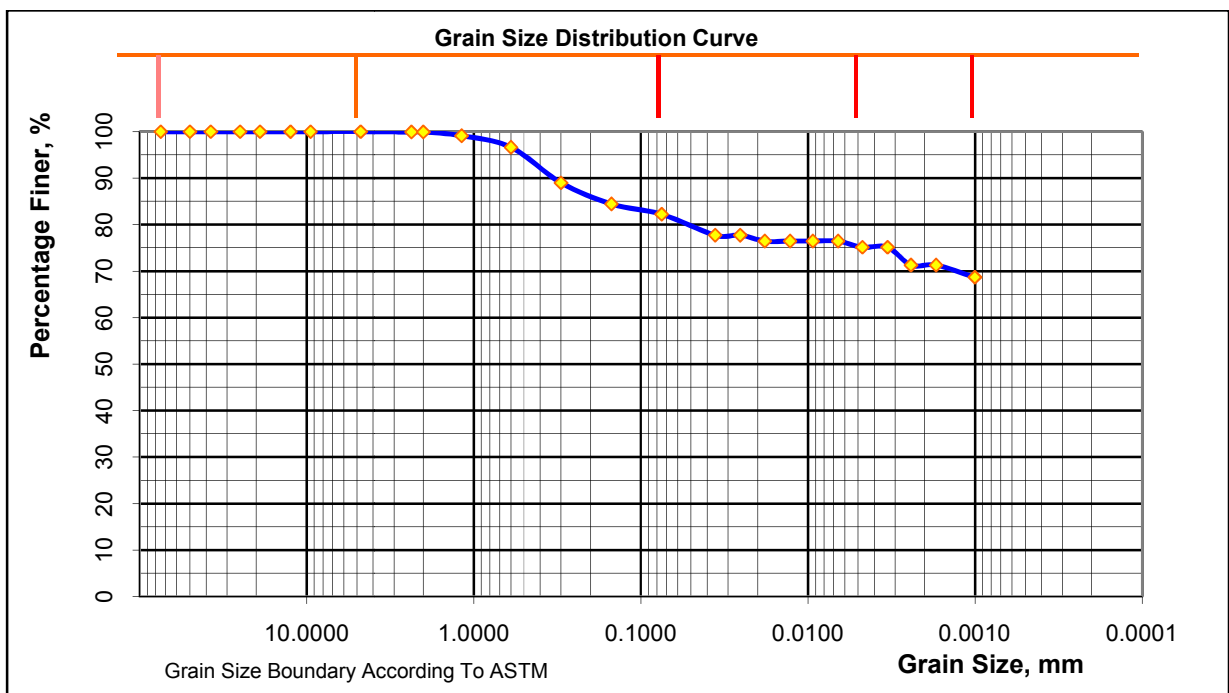
TP3-1(105 °c)----- depth 1.5 m



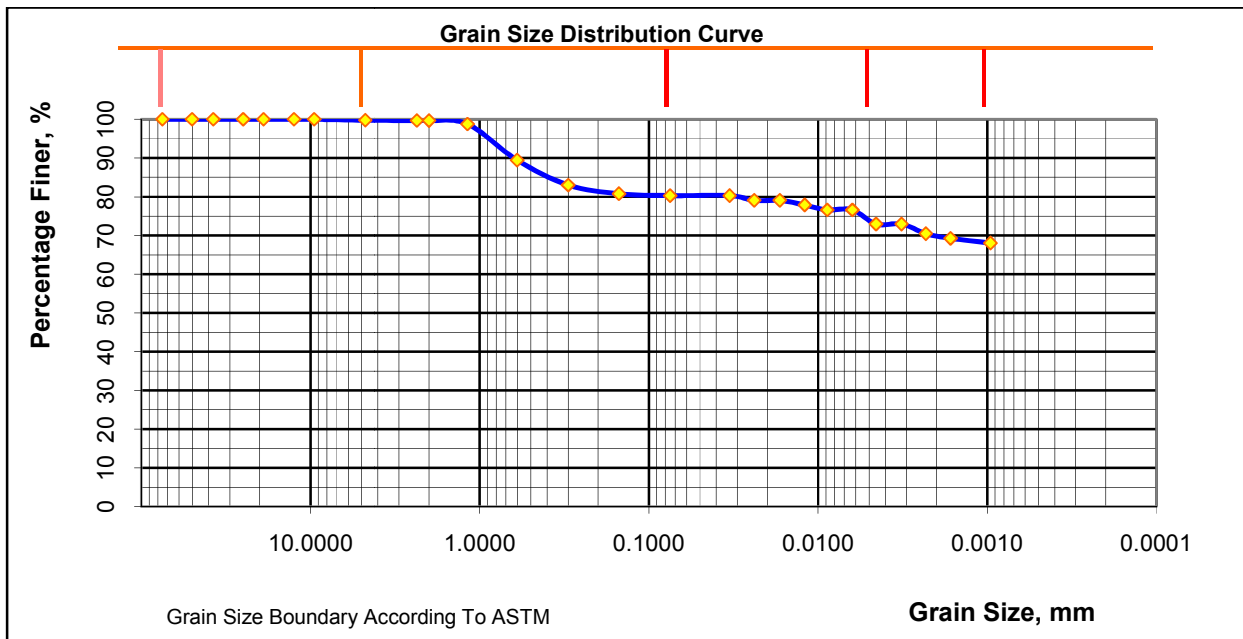
TP3-2(105 °c)----- - depth 3 m



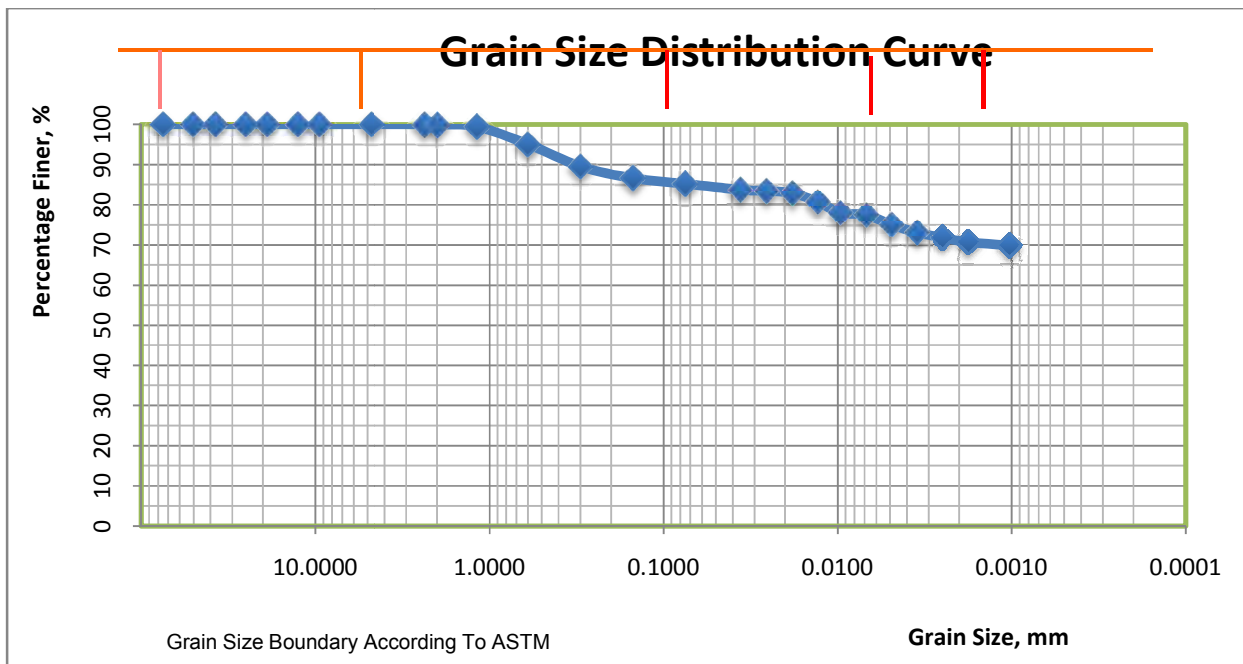
TP4-1(105 °c)----- depth 1.5 m



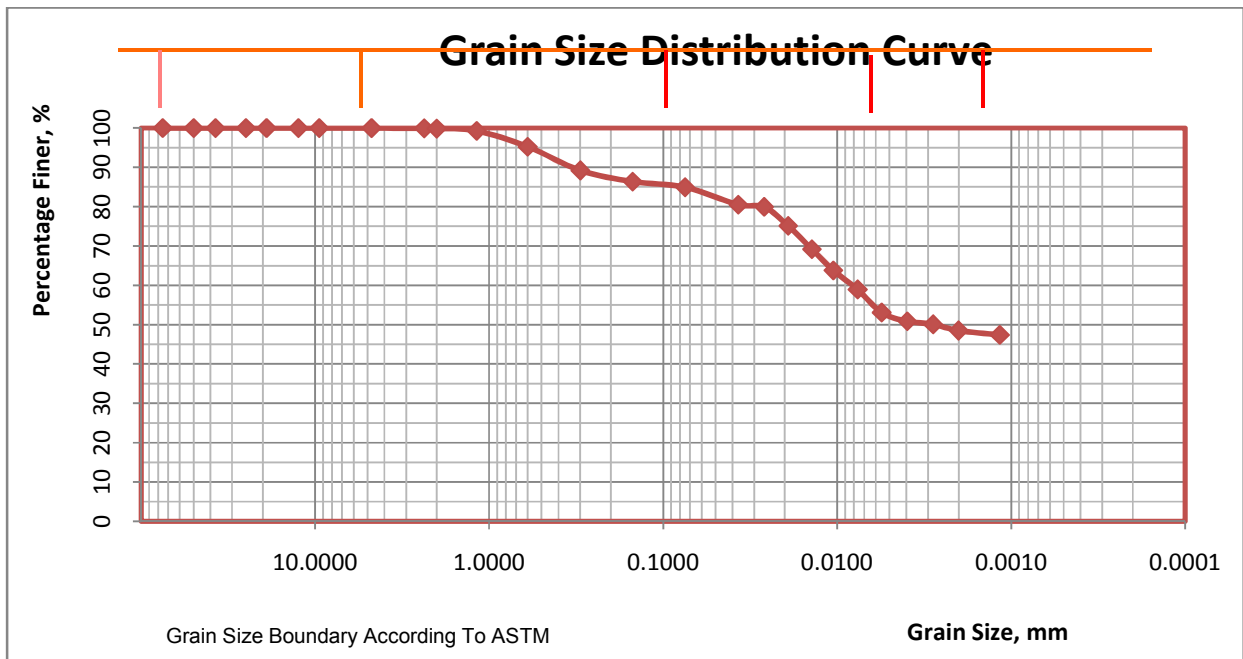
TP4-2(105 °c)----- depth 3m



TP5-1(105 °c)----- depth 1.5 m



TP5-2(105 °c)----- depth 3m



4. SPECIFIC GRAVITY TEST

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer.

Sample calculation

Table A- 8 Specific gravity determination data sheet

Depth of Sampling: 3m

Sample designation TP3-2 (50°C)

[A] Calibration of pycnometer

Pycnometer No.	P1	P2
Weight of dry, clean pycnometer, w_p (g)	45.26	49.86
Weight of pycnometer + water, w_{pw} (g)	144.81	149.55
Observed temperature of water, T_i (°C)	21.8	20.4

[B] Specific Gravity Determination

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	160.63	165.41
Temperature, T_x (°C)	21.3	21.2
Weight of pycnometer + water at T_x , $W_{pw}(atT_x)$ (g)	144.83	149.53
Weight of dry soil, w_s (gm)	25	25
Conversion factor, K	0.9998	0.9998
Specific gravity of soil at 20°C.	2.72	2.74
Average specific gravity of soil .		2.73

TP1-1(105 °C)

<i>Pycnometer No.</i>	<i>P20</i>	<i>P41</i>
<i>Weight of dry, clean pycnometer, w_p (g)</i>	45.35	49.88
<i>Weight of pycnometer + water, w_{pw} (g)</i>	165.6	169.1
<i>Observed temperature of water, T_i (oc)</i>	21.8	21.7

<i>Determination No.</i>	<i>1</i>	<i>2</i>
<i>Pycnometer No.</i>	<i>P20</i>	<i>P41</i>
<i>Weight of pycnometer + soil + water, W_{pws} (g)</i>	165.6	169.1
<i>Temperature, T_x (°c)</i>	21.8	21.7
<i>Weight of pycnometer + water at T_x, $W_{pw}(atT_x)$ (g)</i>	149.80	153.40
<i>Weight of dry soil, w_s (gm)</i>	25	25
<i>Conversion factor, K</i>	0.9996	0.9996
<i>Specific gravity of soil at 20°C.</i>	2.72	2.69
<i>Average specific gravity of soil.</i>	2.70	

TP1-2(105 °C)

<i>Pycnometer No.</i>	<i>P9</i>	<i>P10</i>
<i>Weight of dry, clean pycnometer, w_p (g)</i>	49.38	45.57
<i>Weight of pycnometer + water, w_{pw} (g)</i>	149.9	143.3
<i>Observed temperature of water, T_i (oc)</i>	20.3	20.2

<i>Determination No.</i>	<i>1</i>	<i>2</i>
<i>Pycnometer No.</i>	<i>P9</i>	<i>P10</i>
<i>Weight of pycnometer + soil + water, W_{pws} (g)</i>	165.8	158.9
<i>Temperature, T_x (°c)</i>	23.2	23.1
<i>Weight of pycnometer + water at T_x, $W_{pw}(atT_x)$ (g)</i>	149.83	143.23
<i>Weight of dry soil, w_s (gm)</i>	25	25
<i>Conversion factor, K</i>	0.9996	0.9998
<i>Specific gravity of soil at 20°C.</i>	2.77	2.68
<i>Average specific gravity of soil.</i>	2.72	

TP1-2(50°C)

<i>Pycnometer No.</i>	<i>P4</i>	<i>P3</i>
<i>Weight of dry, clean pycnometer, w_p (g)</i>	43.2	47.6
<i>Weight of pycnometer + water, w_{pw} (g)</i>	142.9	146.9
<i>Observed temperature of water, T_i (oc)</i>	24	24

<i>Determination No.</i>	<i>1</i>	<i>2</i>
<i>Pycnometer No.</i>	<i>P4</i>	<i>P3</i>
<i>Weight of pycnometer + soil + water, W_{pws} (g)</i>	158.3	163.4
<i>Temperature, T_x (°c)</i>	23	23.2
<i>Weight of pycnometer + water at T_x, $W_{pw}(atT_x)$ (g)</i>	142.90	146.90
<i>Weight of dry soil, w_s (gm)</i>	25	25
<i>Conversion factor, K</i>	0.9993	0.9993
<i>Specific gravity of soil at 20°C.</i>	2.60	2.94
<i>Average specific gravity of soil.</i>		2.77

5. Free swell

The test is performed by slowly pouring 10cm³ of dry soil which has passed the No. 40 (0.425mm) sieve in to 100 cm³ graduated cylinder filled with distilled water. After 24 hours, final volume of the suspension is read. Hence, free swell is defined as.

TP1-1(50^o)...1.5m

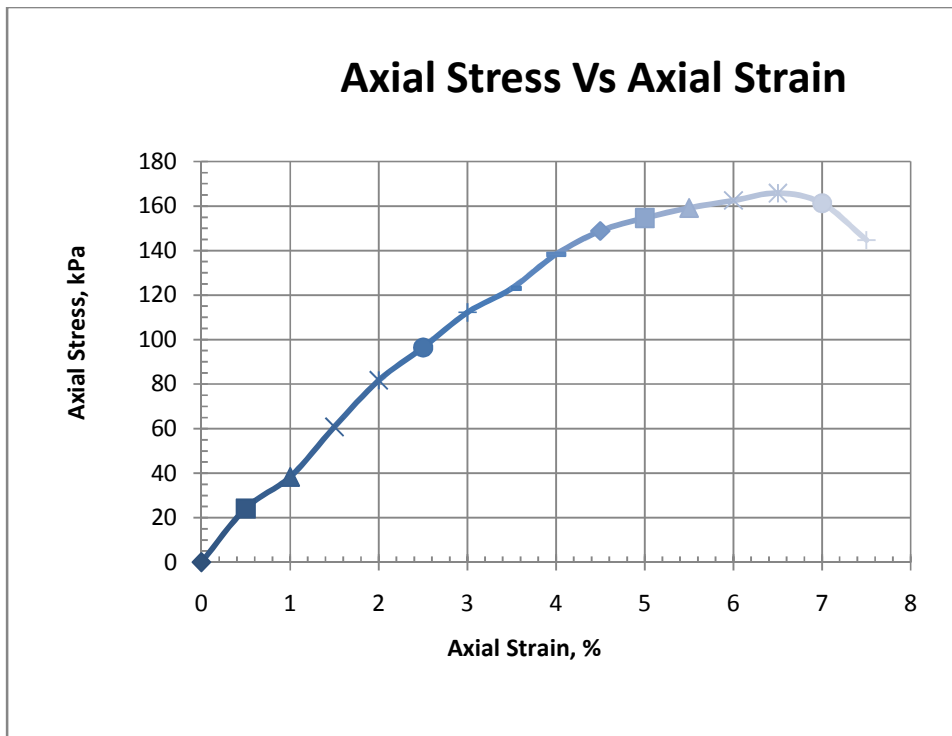
<i>Initial Volume (cc)</i>	<i>Final Volume</i>		<i>Average Final Volume (cc)</i>	<i>Free Swell (%)</i>
	<i>Sample No.1 (cc)</i>	<i>Sample No.2 (cc)</i>		
10.0	12.9	12.0	12.5	25

TP1-2(50^o)...3 m

<i>Initial Volume (cc)</i>	<i>Final Volume</i>		<i>Average Final Volume (cc)</i>	<i>Free Swell (%)</i>
	<i>Sample No.1 (cc)</i>	<i>Sample No.2 (cc)</i>		
10.0	12.5	14.0	13.3	33

6. Unconfined Compression Test

Test Pit Depth , m		3.00	Ring calibration factor, KN/div		0.00133
Diameter of sample ,mm		38	Moisture content ,%		32.75
Length of sample,mm		76	Wet unit weight ,KN/m ³		17.7
Axial	Axial	Proving Ring	Axial	Corrected	Axial
Deformation	Strain	Reading	Load	Area	Stress
[mm]	[%]	[div]	[kN]	[m ²]	[kPa]
0	0.00	0	0.0000	0.00113	0
0.5	0.66	20	0.0276	0.00114	24.18
1	1.32	32	0.0442	0.00115	38.43
1.5	1.97	51	0.0704	0.00116	60.84
2	2.63	69	0.0952	0.00116	81.76
2.5	3.29	82	0.1132	0.00117	96.51
3	3.95	96	0.1325	0.00118	112.21
3.5	4.61	106	0.1463	0.00119	123.05
4	5.26	120	0.1656	0.00120	138.35
4.5	5.92	130	0.1794	0.00121	148.83
5	6.58	136	0.1877	0.00121	154.61
5.5	7.24	141	0.1946	0.00122	159.17
6	7.89	145	0.2001	0.00123	162.52
6.5	8.55	149	0.2056	0.00124	165.81
7	9.21	146	0.2015	0.00125	161.31
7.5	9.87	132	0.1822	0.00126	144.78



Unconfined Compressive
Strength(q_u), kPa = 166

Undrained Shear
Strength (c_u), kPa = 83

Unconfined Compression Strength (q_u) KN/m^2

Sample type = remolded clay

Mass of sample = 155.57g

Length of sample = 36mm

Diameter of sample = 38mm

Field density = 1.77g/cm³

Natural moisture content (ω) = 32.75

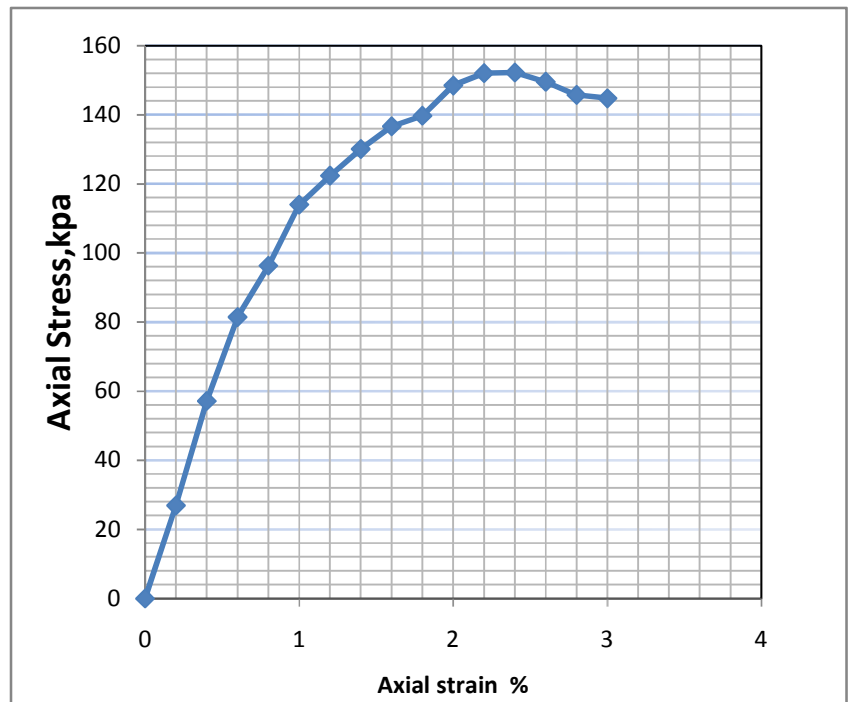
Mass of Mold, g = 3128.6

axial strain,%	axial stress,kpa
0	0
0.2	26.90
0.4	57.17
0.6	81.45
0.8	96.32
1	114.00
1.2	122.36
1.4	130.09
1.6	136.63
1.8	139.70
2	148.46
2.2	152.04
2.4	152.20
2.6	149.52
2.8	145.72
3	144.76

Mass of mold+ Compacted Soil, g = 4800

Mass of Compacted soil, g =1671.4

Volume of Mold,cm³ = 944



Appendix-B

Detail calculations of consolidation test results

Test Sample-----1

Sample Designation-----TP2-2

Sample depth-----3.0 m

Table 1 Data before commencement of the consolidation test:

Area of the consolidation ring cm ²	19.6
Volume of the consolidation ring cm ³	39.20
Height of Sample ,mm	20
Mass of soil(Ms)	47.1
Sample type	Undisturbed
Seating Load ,kpa	7
initial moisture content%	30.34
Specific gravity	2.68

Table 2 Determination of Initial Moisture Content

	A16	85
Wt. of can	32.6	33.5
Wt. of can+wet soil	115	116.1
Wt. of can+oven dry soil	95.4	97.3
Wt. of water	19.6	18.8

Wt of dry soil	62.8	63.8
Moist. Content	31.21	29.47
Average Moisture%	30.34	

Table 3 Determination of Dry Weight of Specimen

Oven Dry wt.+ Ring	101.30
Wt. of ring	54.20
Dry wt. of Specimen, gm	47.1

Table 4 Dial gauge reading for each incremental loading

Time(min.)	□time	Dial Guage Reading, mm						
		7	50	100	200	400	800	1600
		[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]
0	0.00	14.935	14.925	14.815	14.637	14.188	13.435	12.478
0.25	0.50	-	14.850	14.728	14.318	13.578	12.660	11.640
0.50	0.71	-	14.849	14.715	14.302	13.568	12.630	11.609
1	1.00	-	14.848	14.702	14.290	13.541	12.609	11.582
2	1.41	-	14.843	14.692	14.279	13.529	12.590	11.550
4	2.00	-	14.841	14.689	14.265	13.512	12.572	11.522
8	2.83	-	14.839	14.680	14.255	13.501	12.558	11.500
15	3.87	-	14.838	14.673	14.245	13.490	12.542	11.480
30	5.48	-	14.834	14.670	14.233	13.480	12.530	11.462
60	7.75	-	14.831	14.661	14.225	13.471	12.519	11.450
120	10.95	-	14.828	14.655	14.218	13.468	12.509	11.439
240	15.49	-	14.822	14.650	14.208	13.455	12.500	11.425

480	21.91	-	14.819	14.640	14.198	13.445	12.490	11.417
1440	37.95	14.925	14.815	14.637	14.188	13.435	12.478	11.401
Cummulative Dial Guage Reading At The End Of Each Cosecutive Unloading								
Dial Guage Reading, mm								
1600	800	400	200					
[kPa]	[kPa]	[kPa]	[kPa]					
11.457	11.521	11.59	11.789					

Compression index .Cc

Dry mass of the soil (M_d)= $M_s/(1+w)$ = 47.1

Height of solid (H_s) = $\frac{M_d}{(G_s * A * p_w)} = 47.1 / (2.68 * 19.6 * 1) = 8.96\text{mm}$

where

G=specific gravity of the solids

A =Area of the consolidation ring

p_w = density of water 1g/cm³

H_s = 8.96mm

Height of void (H_v) = $H - H_s = 20\text{mm} - 8.96\text{mm} = 11.03\text{mm}$

Initial void ratio (e_o) = $H_v/H_s = 11.03\text{mm}/8.96\text{mm} = 1.23$

For loading of 7 kpa

initial reading=14.935

Final dial reading=14.925

$\Delta H_1 = 14.935 - 14.925 = 0.01$

$$\Delta e_1 = \Delta H_1 / H_s = 0.01 / 8.96 = 0.001$$

$$e_1 = e_0 - \Delta e_1 = 1.23 - 0.001 = 1.229$$

For loading of 50 kpa

initial reading=14.925

Final dial reading=14.815

$$\Delta H_1 = 14.925 - 14.815 = 0.11$$

$$\Delta e_2 = \Delta H_2 / H_s = 0.11 / 8.96 = 0.012$$

$$e_3 = e_2 - \Delta e_3 = 1.229 - 0.012 = 1.217$$

For loading of 100kpa

initial reading=14.815

Final dial reading=14.637

$$\Delta H_2 = 14.815 - 14.637 = 0.178$$

$$\Delta e_4 = \Delta H_4 / H_s = 0.178 / 8.96 = 0.0198$$

$$e_4 = e_3 - \Delta e_4 = 1.217 - 0.0198 = 1.1972$$

For loading of 200kpa

initial reading=14.637

Final dial reading=14.188

$$\Delta H_3 = 14.637 - 14.188 = 0.449$$

$$\Delta e_3 = \Delta H_3 / H_s = 0.449 / 8.96 = 0.0501$$

$$e_3 = e_2 - \Delta e_3 = 1.1972 - 0.0501 = 1.1471$$

For loading of 400kpa

initial reading=14.188

Final dial reading=13.435

$$\Delta H_4 = 14.188 - 13.435 = 0.753$$

$$\Delta e_4 = \Delta H_4 / H_s = 0.753 / 8.96 = 0.084$$

$$e_4 = e_3 - \Delta e_4 = 1.1471 - 0.084 = 1.0631$$

For loading of 800kpa

initial reading=13.435

Final dial reading=12.478

$$\Delta H_5 = 13.435 - 12.3478 = 1.0872$$

$$\Delta e_5 = \Delta H_5 / H_s = 1.087 / 8.96 = 0.121$$

$$e_5 = e_4 - \Delta e_5 = 1.0631 - 0.121 = 0.9421$$

For loading of 1600kpa

initial reading=12.478

Final dial reading=11.401

$$\Delta H_6 = 12.478 - 11.401 = 1.077$$

$$\Delta e_6 = \Delta H_6 / H_s = 1.077 / 8.96 = 0.120$$

$$e_6 = e_5 - \Delta e_6 = 0.8221$$

For Un loading of 1600kpa

initial reading=12.478

Final dial reading=11.401

$$\Delta H_6 = 12.478 - 11.401 = 1.077$$

$$\Delta e_6 = \Delta H_6 / H_s = 1.077 / 8.96 = 0.120$$

$$e_6 = e_5 - \Delta e_6 = 0.8221$$

For Un loading of 800kpa

initial reading=11.457

Final dial reading=11.521

$$\Delta H_8 = 11.457 - 11.521 = 0.064$$

$$\Delta e_8 = \Delta H_8 / H_s = 0.0064 / 8.96 = 0.0073$$

$$e_8 = e_7 + \Delta e_8 = 0.8221 + 0.0073 = 0.8294$$

For Unloading of 400kpa

initial reading=11.521

Final dial reading=11.590

$$\Delta H_8 = 11.59 - 11.521 = 0.07$$

$$\Delta e_8 = \Delta H_8 / H_s = 0.07 / 8.96 = 0.0078$$

$$e_8 = e_7 + \Delta e_8 = 0.8294 + 0.0078 = 0.8372$$

For Unloading of 200kpa

initial reading=11.789

Final dial reading=11.59

$$\Delta H_9 = 11.789 - 11.59 = 0.199$$

$$\Delta e_9 = \Delta H_9 / H_s = 0.199 / 8.96 = 0.022$$

$$e_9 = e_8 + \Delta e_9 = 0.8373 + 0.022 = 0.8593$$

Table 5 Summary of Applied pressure Vs void ratio for loading and unloading

Applied pressure (P kPa)	ΔH_i (mm)	Δe_i	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	0.00	0.00	20	11.04	1.23
7	0.01	0.001	19.99	11.03	1.229
50	0.11	0.012	19.88	10.92	1.217
100	0.178	0.0198	19.702	10.74	1.197
200	0.449	0.0501	19.253	10.29	1.147
400	0.753	0.084	19.50	9.54	1.063
800	1.087	0.121	17.413	8.45	0.942
1600	1.077	0.12	16.33	7.37	0.82
Unloading					
1600	1.077	0.12	16.33	7.37	0.82
800	0.064	0.0073	16.394	7.43	0.895
400	0.07	0.0078	16.404	7.44	0.903
200	0.199	0.022	16.663	7.703	0.925

Test Sample-----2

Sample Designation-----TP5-2

Sample depth-----3.0 m

Table 6 Data before commencement of the consolidation test

Area of the consolidation ring cm^2	19.6
Volume of the consolidation ring cm^3	39.20
Height of Sample ,mm	20
Mass of soil(Ms)	47.1
Sample type	Undisturbed
Seating Load ,kpa	7
initial moisture content%	37.05
Specific gravity	2.68

Table 7 Determination of Initial Moisture Content

	A16	85
Wt. of can	163	77
Wt. of can+wet soil	135.4	134.8
Wt. of can+oven dry soil	107.8	107.6
Wt. of water	27.6	27.2
Wt of dry soil	74.3	73.6

Moist. Content	37.15	36.96
Average Moisture%	37.05	

Table 8 Determination of Dry Weight of Specimen

Oven Dry wt.+ Ring	102.50
Wt. of ring	54.20
Dry wt. of Specimen, gm	48.3

Table 9 Dial gauge reading for each incremental loading

Time(min.)	time	Dial Guage Reading, mm						
		7	50	100	200	400	800	1600
		[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]
0	0.00	14.950	14.945	14.860	14.795	14.608	14.215	13.783
0.25	0.50	-	14.891	14.829	14.698	14.315	13.930	12.985
0.50	0.71	-	14.889	14.826	14.690	14.308	13.913	12.949
1	1.00	-	14.888	14.822	14.683	14.299	13.899	12.911
2	1.41	-	14.884	14.820	14.679	14.291	13.888	12.881
4	2.00	-	14.882	14.819	14.671	14.282	13.875	12.855
8	2.83	-	14.880	14.817	14.661	14.273	13.862	12.831
15	3.87	-	14.879	14.815	14.655	14.268	13.851	12.811
30	5.48	-	14.878	14.813	14.646	14.260	13.842	12.795
60	7.75	-	14.875	14.811	14.640	14.251	13.830	12.779
120	10.95	-	14.872	14.809	14.631	14.242	13.819	12.759
240	15.49	-	14.869	14.807	14.623	14.236	13.810	12.751

480	21.91	-	14.867	14.797	14.618	14.227	13.799	12.738
1440	37.95	14.945	14.860	14.795	14.608	14.215	13.783	12.721
Cummulative Dial Guage Reading At The End Of Each Consecutive Unloading								
Dial Guage Reading, mm								
1600	800	400	200	100	50	7		
[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]		
12.721		12.921		13.260		13.295		

Table 10 Summary of Applied pressure Vs void ratio for loading and unloading

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H _v (mm)	Void Ratio, E
Loading					
7	14.950	0.00	20.00	11.13	1.26
7	14.945	0.00	20.00	11.13	1.25
50	14.860	-0.09	19.91	11.04	1.25
100	14.795	-0.15	19.85	10.98	1.24
200	14.608	-0.34	19.66	10.79	1.22
400	14.215	-0.73	19.27	10.40	1.17
800	13.783	-1.17	18.83	9.97	1.12
1600	12.721	-2.23	17.77	8.90	1.00
Unloading					
1600	12.721	-2.23	17.77	8.90	1.00
400	12.921	-2.03	17.97	9.10	1.03
100	13.260	-1.69	18.31	9.44	1.06
7	13.295	-1.66	18.35	9.48	1.07

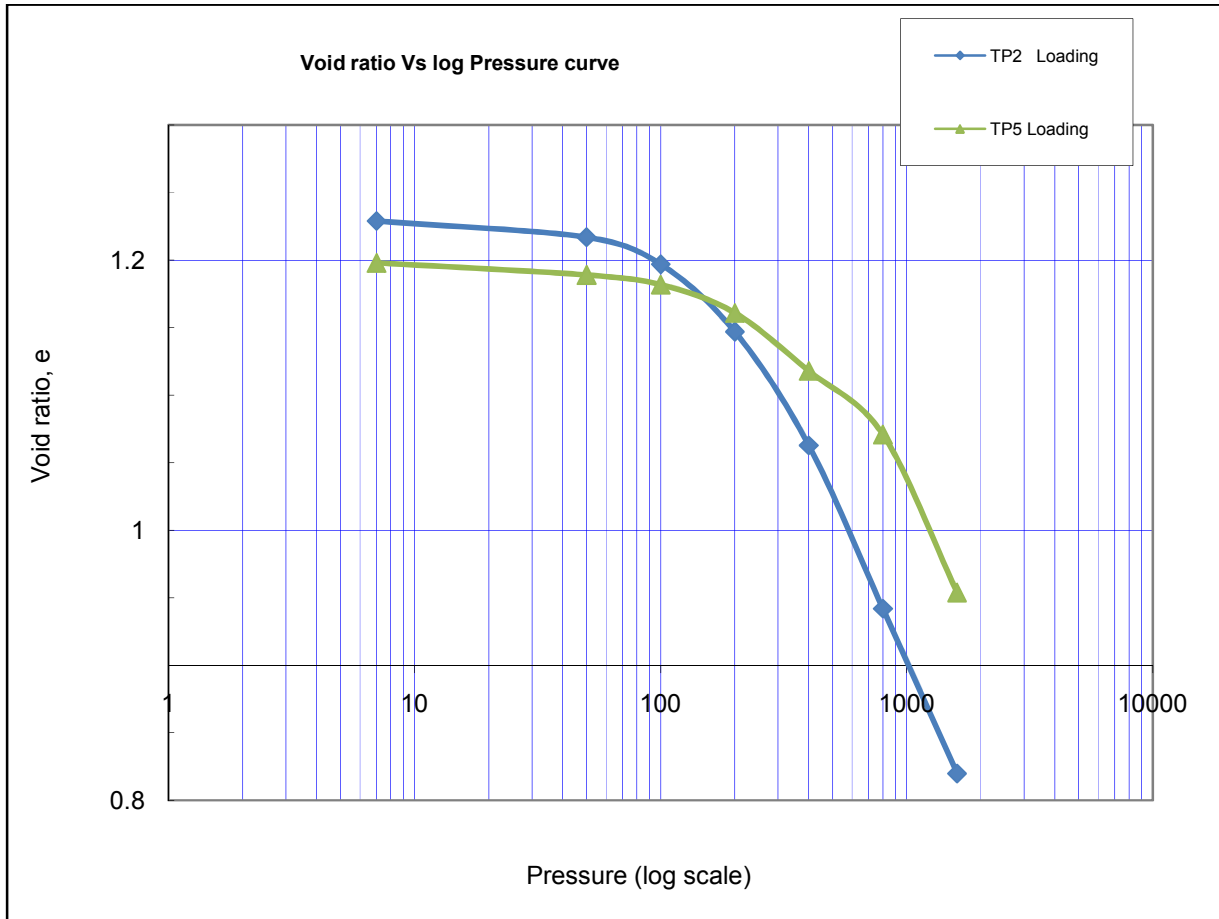
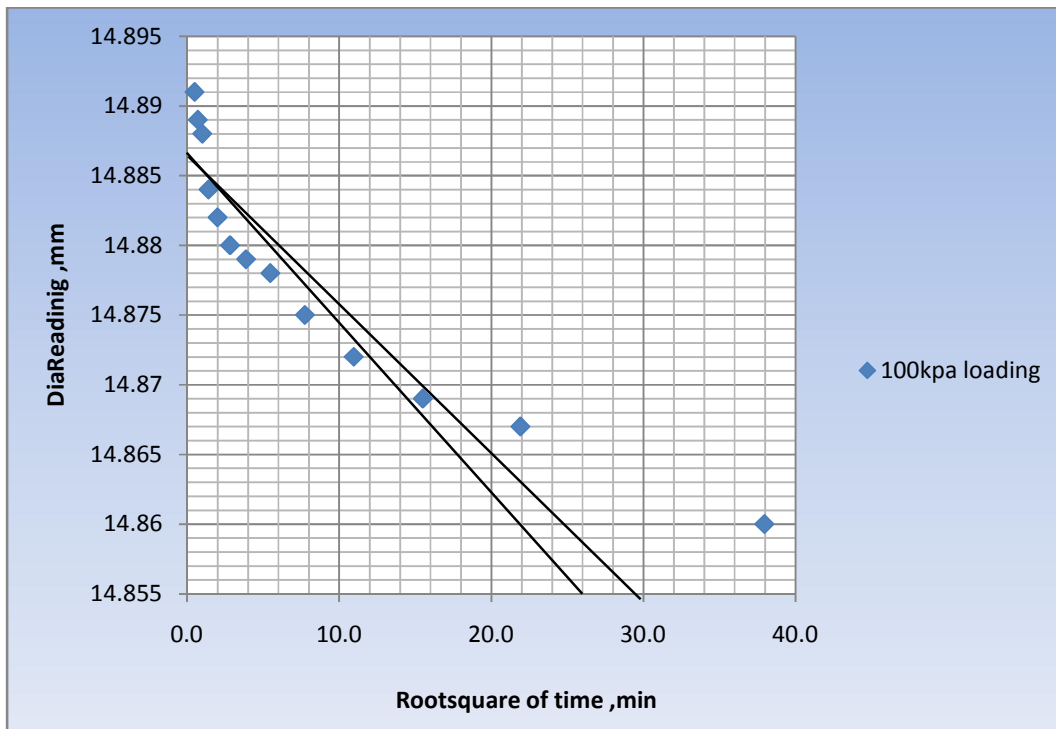


Fig 11 Plot of vertical effective stress Vs void ratio on semi-log scale

Coefficient of consolidation

Typical plots of Dial reading versus Square root of time of sample-TP5-2 (P=100Kpa)



$$C_v = \frac{0.848H^2}{t_{90}}$$

t_{90} = time elapsed corresponding to 90 % consolidation as read from the laboratory curve

H= the distance of the drainage path (half of the total thickness)

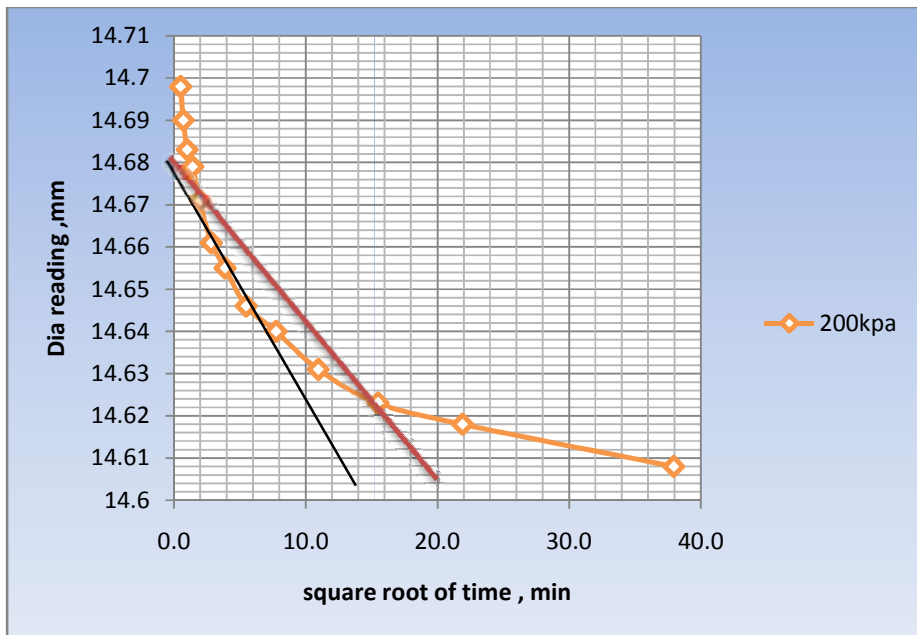
$$t_{90} = 15.8$$

$$= (15.8)^2 = 249.64 \text{ min} = 249.64 * 60 \text{ sec} = 14978.4 \text{ sec}$$

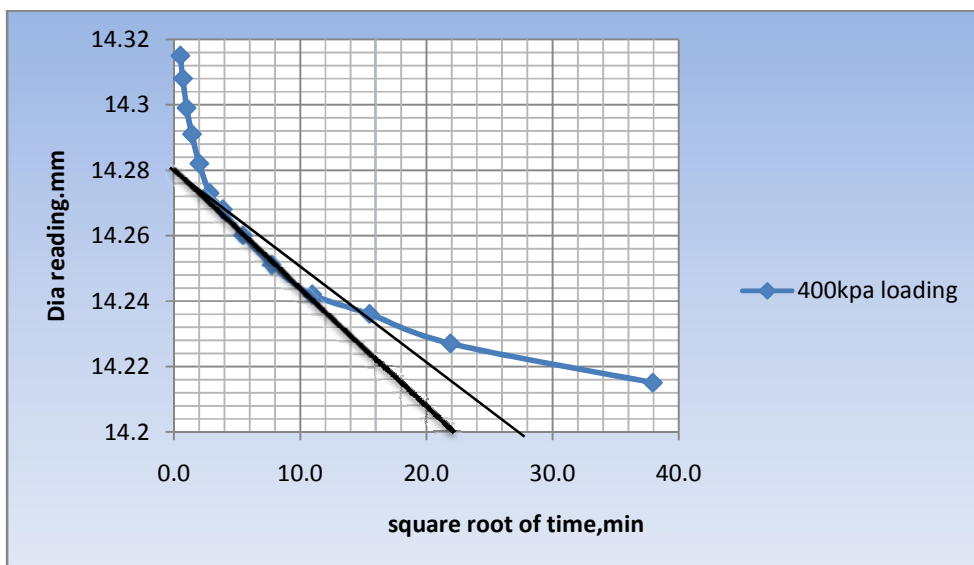
$$H = 1 \text{ cm}^2$$

$$C_v = 0.848 * 1^2 \text{ cm}^2 / (14978.4) \text{ sec} = 3.56 * 10^{-3} \text{ cm}^2/\text{sec}.$$

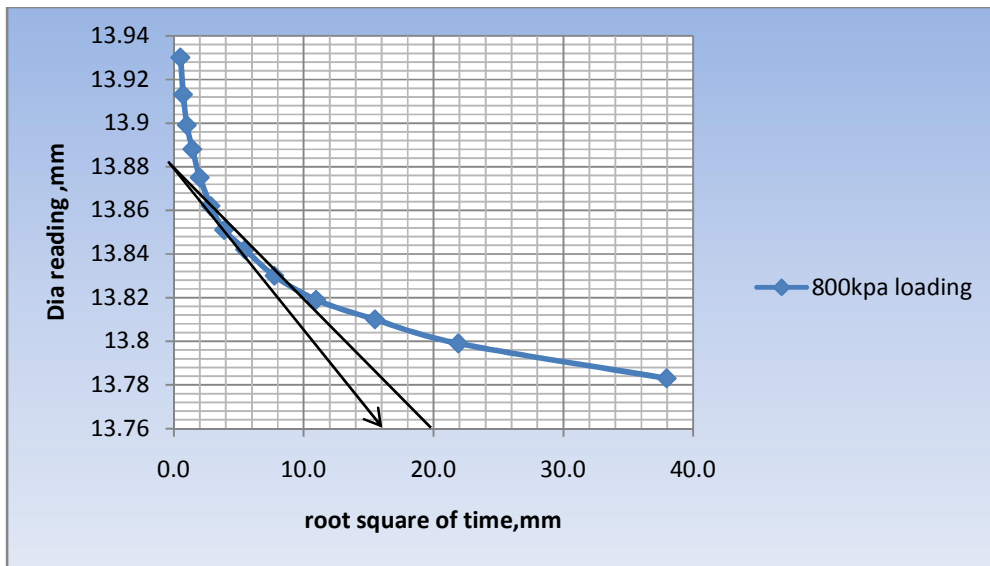
Typical plots of Dial reading versus Square root of time of sample-TP5-2 (P=200Kpa)



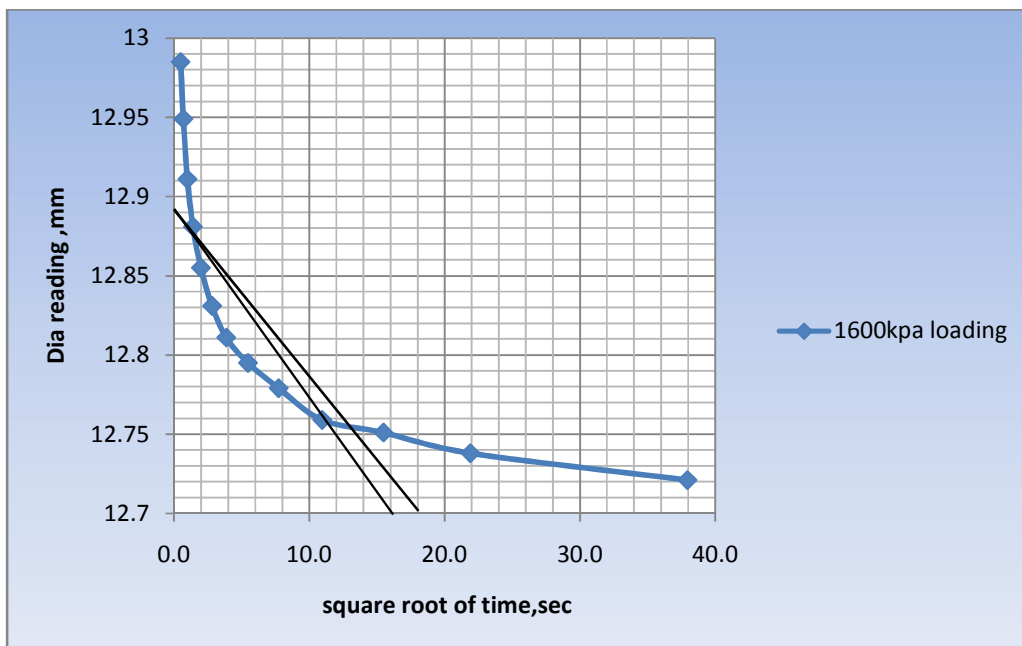
Typical plots of Dial reading versus Square root of time of sample-TP5-2 (P=400Kpa)



Typical plots of Dial reading versus Square root of time of sample-TP5-2 (P=800Kpa)



Typical plots of Dial reading versus Square root of time of sample-TP5-2 (P=1600Kpa)



Coefficient of Permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test. The Coefficient of permeability can be obtained from the following relationship

$$K = \frac{C_v * a_v * \gamma_w}{(1 + e_o)}$$

where: K=Coefficient of permeability

CV= Coefficient of consolidation

av=Coefficient of compreseblity

= Unit weight of water

e_o= void ratio

For loading of 100kpa (TP5-2)

Void ratio (e)= 1.24

from figure 4.13= C_v=3.56*10⁻³ Cm²/sec

unit weight of water(γ_w)=10KN/m³

From figure 4.12 (e-p) graph a_v= $\frac{1.18-1.06}{400-1200}$

$$a_v = 15 * 10^{-5} \text{ m}^2/\text{KN}$$

$$k = \frac{3.56 * 10^{-3} \text{ Cm/sec}^2 * 15 * 10^{-5} \text{ m}^2/\text{KN} * 10 \text{ KN/m}^3}{(1 + 1.24)}$$

$$k = 2.38 * 10^{-8} \text{ cm/sec}$$

