ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

Consolidation and Settlement behavior of Residual lateritic soils of Western Ethiopia (The case of Tongo-Begi-Mugi Road Project, Contarct-1, Tongo-Gidami)

A thesis submitted to the school of Graduate Studies of Addis Ababa
University in Partial fulfillment of the Requirements for the

Degree of Master of Science in Civil Engineering

By

Tiruneh Adugna Melaku

Advisor: Dr.-Ing Samuel Tadesse

March, 2015

ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

(Geotechnical Engineering Program)

Consolidation and Settlement Behavior of Residual Lateritic Soils of Western Ethiopia(the case of Tongo-Begi-Mugi Road Project,Contract-1,Tongo-Gidami)

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Abstract

Compressibility of soils is one of the basic Engineering properties of Soils, and it mostly occurs through the process of consolidation. Consolidation is a time dependent process in which expulsion of water from a soil mass occurs due to the application of stress on it. Consequently, there exist change in volume in the vertical direction of the soil mass and this difference in thickness results the settlement of the structures constructed above the soil. On the other hand, degree of consolidation and extent of settlement varies with types of soils. Therefore, on this study an effort was made to reach the unique behaviour of residual laterite soils with regard to consolidation and settlement properties focusing on the western Ethiopia which is rich of such soils.

To achieve the intended objective of the research, papers, journals and different books have been scrutinized under literature review, samples from the research area were collected, and then laboratory tests for index properties and primary consolidation have been carried out using disturbed and undisturbed samples respectively.

From the test results, it has been observed that index properties of lateritic soils greatly are affected by pre-test treatment and manipulation, and in the case of consolidation and settlement behaviours, laterite soils at intermediate and high stress levels depict very similar behavior to transported soils, but at the initial phase the lateritic soils behave as if overconsolidated due to their cementation bond whereas transported soils show overconsolidation due to the past stress history.

Table of Contents

List of Tables

List of Figures

Symbols and Abbreviations

Designation	Description
σ	total stress
σ'	effective stress
u, u_w	pore water pressure
u_a	pore air pressure
t	time
V	volume
C_v	coefficient of consolidation
m_v	coefficient of volume compressibility(Volume change)
k_z	hydraulic conductivity
γ_w	Specific Gravity of water
G_s	Specific Gravity of soil
Z	depth
H_{dr}	Drainage Path
T_v	Time factor
U	degree of consolidation
a_v	coefficient of compressibility
e	void ratio
Δ	Delta, A Greek letter used as change
C_c	Compression index
S_c	Consolidation Settlement
H_o	initial Sample Height

CIRIA	Construction Industry Research and Information Association
ERA	Ethiopian Roads Authority
UNICONE	United Consulting Engineers
HCE	Highway Consulting Engineers
ASTM	American Society of Testing and Materials
AASHTO	American Association of State Highway and Transport Officials
USCS	Unified Soil Classification System
AD	Air Dried
AR	As Received
OD	Oven Dried
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
TP	Test Pit

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1. Introduction

1.1 General

When structures (high-rise buildings, dams, roads on high fills or embankments, etc…) are built on soils, they transfer loads to the sub soil through the foundation. Consequently, the soil will be compressed due to the imposed stress (load). The compression of the soil mass due to the imposed stress may be almost immediate or time dependent according to the permeability characteristics behavior of the soil. If the permeability of the soil is high, flow of water out of the soil voids is almost instantaneous, whereas, if the permeability is low the pore water dissipates gradually. This gradual flow of water out of the soil mass and the gradual transfer of the external stress from the pore water to soil skeleton (soil grains) is called consolidation. It is the time dependent process involving drainage, compression and stress transfer. Rate and degree of consolidation therefore, depends on the type of soils, the stress (loading) history and the permeability of the soil. Based on their formation process soils may be classified as transported and /or residual types. Consolidation for transported soils can be evaluated based on principles of classical soil mechanics, where as for residual soils the case is different due to its formation process.

Residual tropical soils are formed from in-situ tropical weathering and decomposition of rocks which are transported from their original location. The permeability of residual soils depends on the macro and micro structures of the soils, implies that it is difficult to determine consolidation and settlement behavior of residual soils using the principle of classical soil mechanics. Basically this thesis focuses on the consolidation and settlement behavior of laterite soils, which are classified under residual soils according to Wesley L.D. and Irfan T.Y. and (Blight, 1997). In Ethiopia, laterite soils are found extensively in western part of the country especially in western Wellega zone and most of Benshangul Gumuz region because these areas are suitable for the formation of laterite soils with respect to topography, temperature and climate.

1.2 Statement of the Problem

Laterite soils are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical conditions. Thus, in conjunction with their way of formation residual laterite soils show behaviors different from transported soils .They are highly affected by pre-test preparation and manipulations during the test. Countries located in tropical zones, for example, Ghana, Nigeria, Malaysia, India and Indonesia; use lateritic soils as good construction materials for roads and air fields. In Ethiopia these soils are treated as transported soils.

Thus, the soil test and the test results are judged based on classical soil mechanics principles so that the soil couldn’t fulfill the requirements specified on the Standards. As a result finding construction material far away from the construction site will be recommended and this is cumbersome and extravagant especially countries like Ethiopia in terms of cost. So expecting to be part of the solution, in this research work one of the basic engineering properties of soils, which is consolidation and settlement behavior of laterite soils of Western Ethiopia will be investigated. Understanding these properties is vital for the construction of high-fill embankment roads, high-rise buildings and earth dams on laterite soils.

1.3 Objective of the research

Main Objective:

The main objective of this research is:

- ✓ to investigate the consolidation and settlement behavior of residual lateritic soils of Western Ethiopia (The case of Tongo-Begi-Mugi Road Project, Contract 1- Tongo-Gidami)
- ✓ Comparing and analyzing the result with the consolidation and settlement characteristics of transported soils as a whole and laterite soils of the tropical world.

Specific Objectives:

- ✓ Investigate index properties of laterite soil
- ✓ Examining this index properties in line with transported soils
- ✓ Investigate consolidation of laterite soils in remolded state

1.4 Research Methodology

Before soil sample is collected site visit has been held which helps to find out the exact location of laterite soils for sampling. Soil sample collection has been conducted with great care since laterite soils are sensitive to change when exposed to air, and aggregation of clay particles, irreversible change in plasticity on drying and loss of water hydration on drying. In addition to this to address such sensitivity characteristics index tests in different conditions such as oven dried, air dried, and just the sample as received keeping its natural ,moisture content as it is, has been conducted. Seven disturbed samples for soil classification and Atterberg limit tests and seven undisturbed block samples to do consolidation test were collected from five test pits.

1.5 Limitation of the Thesis

Consolidation and settlement behavior of the soil is one of the fundamental elements that have to be seen in detail during foundation design based on laboratory and in-situ soil test results of several trials but due to unavailability of excavating machine (equipment), the number of test pits is limited to five. In addition, reference materials on consolidation and settlement behavior of laterite soils is scarce or is easy to say none, so most of the referenced materials are on transported soils. Geochemical and/or mineralogical tests shouldn’t done due to financial constraint but fortunately the consultant for the aforementioned road project has done the test during construction material investigation and the soil has been identified as a laterite soil and used as surface wearing coarse for Tongo-Gidami road stretch(UNICONE,2007).

2. Literature Review

2.1 General

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. When saturated fine grained (clayey) soils are exposed to external loads pore water pressure will be developed and extrusion of pore water needs a time lag due to low permeability of the soil. Such a time dependent expelling of water out from the soil is called consolidation. But the rate of consolidation varies with the soil type. Thus, it is predictable that residual laterite soils may show different characteristics on consolidation test results due to their formation mechanism. Before we are trying to discuss this behavior, it is necessary to express the origin and formation of Residual soils in general and laterite soils and their special engineering characteristics in particular which helps us a general guide line for the issue we have raised.

2.2 Origin and formation of Residual Lateritic Soils

Residual soil by itself is a soil-like material derived from the in-situ weathering and decomposition of rock which has not been transported from its original location. They are formed directly by the physical and chemical weathering of the rock underlying them. They can have characteristics that are quite distinctively different from those of transported soils. As an example the permeability of transported soil can be usually related to its granulometry where as permeability of residual soils is usually governed by its micro and macro-fabric and joints and by superimposed features such as slicken siding, termite or other bio-channels (Blight, 1997).

Particles of residual soils often consist of aggregates or crystals of weathered mineral matters that breakdown and become progressively finer if the soil is manipulated (Blight, 1997).

As explained earlier, Residual soils are formed by the in situ weathering of rocks through physical, chemical and biological processes.

Physical process: includes the effect of such mechanical process as stress release by erosion, differential thermal strain and ice and salt crystallization pressures, abrasion, expansion, and contraction which comminute the rock expose fresh surface to chemical attack and increase the permeability of the material to the percolation of chemical reactive fluids. Physical weathering produces end products consisting of angular blocks, cobbles, gravel, sand, silt and even clay sized rock flour. The mineral constituents of all these products are exactly like those of the original rock.

Chemical processes: chiefly hydrolysis, cation exchange and oxidation alter the original rock minerals to form stable clay minerals, Mitchell, 1976 on (Blight, 1997). The chemical changes operating in primary minerals of the rocks in temperate or semi-tropical zones tend to produce end products consisting of clay minerals predominately represented by Kaolinite and occasionally by Halloysite and by hydrated or dehydrous Oxides of Iron and Aluminum.

Chemical weathering is favored by warm humid climates, by the process of vegetation and by gentle slope. Thus, tropical and subtropical regions of low relief with abundant rainfall and high temperature are the most susceptible to chemical alterations. Deep, strongly leached red, brown and yellow profiles are manifestations of the effects of sever chemical weathering (Lyon Association, 1971). Chemical processes tend to predominate in the weathering of igneous rocks, whereas physical weathering dominates weathering of sedimentary and metamorphic rocks. However, physical and chemical weathering are so closely interrelated that one process never proceeds without some contribution by the other (Blight, 1997). Under conditions favorable to tropical weathering the weathering processes may be so intense and may continue so long that even the clay minerals which are primarily hydrous aluminum silicates, are destroyed, in the continued weathering, the silica is leached and the reminder consists merely of aluminum oxides such as gibbsite or of hydrous iron oxide such as limonite or goethite derived from the iron .This process is known as laterization.

The extent to which a residual soil has been laterized may be measured by the ratio of silica (SiO2), remaining in the soil to the amount of iron (Fe_2O_3) and Aluminum (Al_2O_3) that has accumulated. The silica: sesquioxide ratio:

$\frac{SiO2}{R2O2} = \frac{SiO3}{FeO3+Al2O3}$ is served as a basis for classification of residual laterite soils.

Therefore, according to this degree of laterization of laterite soils is classified as:

S-S ratio >2, unlaterized soil

S-S ratio 1.33 – 2, Lateritic soil

S-S ratio <1.33, true laterite

2.2.1 Factors influencing the Rate of Weathering

2.2.1.1 Climate

Climate exerts a considerable influence on the rate of weathering. Physical Weathering is more predominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and by temperature.

Climate has a further effect on the properties of tropical residual soils. In sub humid tropical and subtropical areas ,water tables are often deeper than 5 to 10 m and the effects of unsaturation, desiccation and seasonal or longer term rewetting have to be taken into account in geotechnical design.

2.2.1.2 Topography

Topography controls the rate of weathering by partly determining the amount of available water for each zone of weathering(G.E.Blight, 1997). Precipitation will tend to run off hills and accumulate soils in valleys and hollows. The slope angle controls the amount of water available to move downward through the weathering zone. On steep slopes run-off is greater than infiltration, erosion is active, and conditions are generally not suitable for the development of deep weathering, conversely on flatter slopes run-off is not so marked only limited erosion takes place, and long uninterrupted period of weathering can occur, producing deep soil profile. On level ground however, where drainage is impeded and the ground is waterlogged black montmorillinite soils dominate at the expense of red soils (CIRIA, 1988). For a deep residual soil profile to develop, the rate at which weathering advances into the earth's crust must exceed the rate of removal of the products of weathering by erosion.

Topography has a strong and fairly consistent influence on the weathering process, and thus on the type of clay minerals formed, especially in the wet tropics. In hilly and mountainous areas, the soil is well drained and seepage flow has a strong downward component. This leads to the formation of low-activity clay minerals, especially Kaolinite.

In volcanic areas, as noted above, the minerals Allophane and Halloysite may be formed initially before ending up as Kaolinite. Soils containing these minerals generally have good engineering properties. As (Vaughan, 1985) states, with some caution, "Residual soils are generally quite well behaved."

In wide, flat areas, drainage of any sort is much more limited, and moisture movement occurs primarily as a result of seasonal changes. Water is lost during dry periods from evaporation and the soil takes up moisture again during periods of rainfall. This environment tends to produce montmorillinite and associated high-activity clay minerals (smectites). Soils containing these minerals normally have poor or highly undesirable geotechnical properties.

2.3 Pedological and Lithological Classification of Residual Soils

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

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

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

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

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

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

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

1. Group A: Soils without a strong mineralogical influence
2. Group B: Soils with strong mineralogical influence derived from clay minerals also commonly found in transported soils.
3. Group C: Soils with a mineralogical influence deriving from clay minerals only found in residual soils. Group C is further classified in sub groups as:
    a) Halloysitic soils
    b) Allophanic soils
    c) Soils influenced by the presence of sesquioxides:

The principal role of sesquioxide appears to be acting as cementing agents bind the other mineral constituents in to clusters or aggregations. With sufficient concentrations of sesquioxides hard concretionary materials commonly known as Laterites are formed. Therefore, let’s see in detail laterite soils specifically, since they are the focus of the thesis.

2.4 Laterite Soils

Lateritic soils according to (Blight, 1997) are highly weathered and altered residual soils, low in silica, that contain a sufficient concentration of the sesquioxide of iron and aluminium, formed by the in-situ weathering and decomposition of rocks in the tropical and sub-tropical regions with hot, humid climatic conditions and according to (CIRIA, 1995) laterite in its form is a highly-weathered natural material formed by the concentration of the hydrated oxides of iron or aluminium. This concentration may be by residual accumulation or by solution, movement and chemical precipitation. In all cases it is the result of secondary physico-chemical processes and not of the normal primary processes of sedimentation, metamorphism, or volcanism. The progress of weathering from kaolinite to sesquioxides involves the leaching out of the silica-based minerals and the concentration of aluminum and iron compounds known as sesquioxides. With time, these compounds act as cementing agents, forming “concretions” and the material becomes non-plastic, sandy gravel. This material is known as laterite, and the weathering process that produces it is termed laterization. This laterization process can occur in both volcanic and non-volcanic soils, although it appears to be more common in the former one. In cooler climates, the weathering action may not progress as far as the formation of laterite (Wesley, 2010).

On the other hand, Laterite Soils are defined as the product of gradual disintegration weathering of the bedrock under tropical condition of high temperature and rainfall. They may be found directly on the rock i.e. they are formed at in-situ or transported into another location (Alexander and Cady, 1962). The composition and mineralogy of the parent rock account for the variation in properties and behavior of soils from place to place. Lateritic soils are reddish tropical soils that have gained a wide range of utilization as construction material in roads, houses, airfield pavement and landfill for foundations. Pedogenic factors also known as soil-forming factors such as climate, vegetation, drainage, chemical and mineralogical composition and degree of weathering control engineering properties of soils. The study of parent rock factor on the engineering properties of lateritic soils developed over different rock types can be determined by keeping other factors constant. These parent materials are of importance in the early stages of weathering since they supply the starting material (Mesida, 1985).

Two aspects of the parent rock affect the formation of Laterites. One is the availability of iron and aluminium minerals. These are more readily available in basic rocks. The other is the quartz content of the parent rock. Where quartz is a substantial component of the original rock, it may remain as quartz grains. Laterite profiles occur on flat slopes in the terrain where runoff is limited. On the level ground, where drainage is poor, expansive clays dominate at the expense of Laterites (Makasa, 1998; Charman, 1995).

From the above discussions, we see that three major processes can be identified during the formation of Laterites which are summarized as follows:

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.

Laterization: leaching occurs under appropriate conditions where combined silica and bases and the relative accumulation or enrichment of Oxides and Hydroxides of Sesquioxides (Fe_2O_3, Al_2O_3, and TiO_2) exists. The soil conditions under which the various elements are rendered soluble and removed through leaching or combination with other substances depend mainly on the pH of the ground water and the drainage conditions (Makasa, 1998; Dibisa, 2008; Million, 2009).

Desiccation: desiccation or dehydration involves partial or complete dehydration (sometimes involving hardening) of the Sesquioxide rich materials and secondary minerals. The dehydration of colloidal hydrated Iron oxide involves loss of water and the concentration and crystallization of the amorphous Iron colloids into dense crystals, in the sequence; Limonite, Goethite with Hematite to Hematite (Makasa, 1998).

Dehydration may be caused by climatic changes, upheaval of the land, or may also be induced by human activities, for example by clearing of forests.

Lateritic soils have specific features or characteristics that are not adequately covered by conventional methods of soil classification such as the Unified Soil Classification System.

Among these features are the following (Blight, 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 such as the Unified Soil Classification System.
- 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.
- Conventional soil classification systems focus primarily on the properties of the soil in its remolded state(disturbed state); this is often misleading with residual soils, whose 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.

Formation also consists of leaching out of free silica and bases and accumulation of oxides of iron, aluminum or both, and this process is termed as laterization. Sesqueoxides (R_2O_3) is the combined name for Iron oxide (Fe_2O_3) and Aluminum oxide (Al_2O_3) the following classes of soils could be possible based on laterization

S-S ratio >2, unlaterized soil

S-S ratio 1.33 – 2, Lateritic soil

S-S ratio < 1.33 true laterite.

The principal role of the sesqueoxides appears to be act as cementing agents which bind the other mineral constituents into clusters or aggregations. With sufficient concentration of sesquioxides, the hard concretionary materials commonly known as laterite are formed. The silica/alumina ratio (SiO_2/Al_2O_3) and the silica/sesquioxide ratio have both used as indicators of degree of laterization.

Laterites are rich in sesquioxides (Fe_2O_3 or Al_2O_3 both) and low in bases and primary silicates but may contain appreciable amounts of quartz and kaolinite. Due to the presence of iron oxides lateritic soils are red in color ranging from light through bright to brown shades. 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. Thus Laterization requires three stages to complete, which are as described below:

- The first state (decomposition) is characterized by physic-chemical break down 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.
- The second stage (Laterization) involves the breaching under appropriate of combined silica and bases and the relative accumulation or enrichment from outside sources of oxides and hydroxides of sesquioxides (mainly Al_2O_3, $Fe_2O_3$and TiO_2)
- The third state (dehydration O_2 desiccation) involves partial or complete dehydration. (Sometimes involving hardening) of the sesquioxide rich materials and secondary minerals (Hegde, R. A. and, Daware, S.N., 2010)

Laterites are grouped under group of soils with a strong mineralogical influence derived from clay minerals only found in residual soils (Blight, 1997). Soil engineering properties under this group are highly influenced by the presence of sesquioxides. The sesquioxides within the fine fraction of tropical soils tend to coat the surface of individual soil particles. It can also cause a physical cementation of adjacent grains, thus producing aggregated particles of coarser size. Both factors reduce plasticity, but intensive remolding of the soil breaks down the aggregations and the sesquioxides coatings which is called disaggregation, results increase in plasticity. The effect of disaggregation of clay size particles up on test manipulation are checked by executing Atterberg limit tests following different testing procedures.

2.4.1 Effect of Pre-test treatment on Index properties of Residual Laterite soils

2.4.1.1 Pre-test Drying on Moisture Content

Drying causes partial or complete dehydration of the clay minerals and can change them and their properties irreversibly. Residual soils are particularly prone to changes in properties caused by drying and exposure to air. Even air-drying at ambient temperature can cause changes that cannot be reversed by re-wetting, even if the re-wetted soil is allowed to mature for long periods. Apart from the relatively well known effect of drying on index properties of residual soils, drying also affects the composition, compressibility and shear strength characteristics of residual laterite soils (G.E.Blight).

The conventional test for determination of moisture content (between 105 and 110˚ C) for laterite soils is not only driven off free water but also may remove the crystallized water within the soil structure when it above temperature. Therefore, to see the effect of pre test drying two test specimens should be prepared for the determination of moisture content. One specimen should be oven dried at 105˚c until successive weighing show no further loss of mass. The moisture content should then be calculated in the normal way. The second sample should be air dried or oven dried at a temperature of not more than 50˚c with a maximum relative humidity of 30% until successive weighing show no further loss of mass. The two moisture content results should be compared; a significant difference (4-6% moisture content obtained by oven drying at 105˚c) indicates that ‘structural’ water is present. This water forms parts of the soil solids, and should therefore be excluded from the calculation of moisture content (G.E.Blight, 1997).

2.4.1.2 Atterberg limit

2.4.1.2.1 Pre-test drying effect on Atterberg Limit

On Atterberg limit tests air drying of the soil sample before carrying out the test has been observed to result in a decrease in the liquid limit and plasticity index. Effect of drying prior to testing may be attributed to increase cementation due to oxidation of the iron and aluminium sesquioxides. Therefore, in order to have meaningful result, Atterberg limit tests should be performed without any form of drying prior to carrying out the test. If some form of drying is unavoidable, for example, if the water content exceeds the plastic limit, this should be noted on the laboratory reports (G.E.Blight).

2.4.1.2.2 Effect of duration and method of Mixing

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

In order to address this problem the following procedure is recommended.

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

On the following day the liquid limit should be determined with a minimum of further mixing. A sub-sample from each of the specimens used in the test should be used for the determination of moisture content, using the above procedure. The remainder of each specimen should then be mixed continuously for a further 25 minutes before again determining the Liquid Limit. A significant difference (of >5% of the liquid limit obtained) between the liquid limit from tests using 5 and 30 minutes 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 425μm sieves until the water runs clear. The material passing the 425µm sieve is collected and used for Atterberg Limit test (Blight, 1997).

2.4.1.3 Particle Size Distribution

The particle size distribution of residual laterite soils may be affected by certain aspects of sample preparation.

2.4.1.3.1 Effect of drying: the most widely effect of drying is to reduce the percentage of the clay fraction (finer than 2µm).It is accordingly recommended that drying of the soil prior to testing be avoided. The soil sample should be split into two sub-samples, one for determining the moisture content (in order to calculate initial dry mass), and the other for the particle size distribution test. The later sample should be immersed in a solution of dispersant such as dilute alkaline sodium hexametaphosphate, and then, washed through the standard nest of sieves.

2.4.1.3.2 Chemical Pre-treatment:

This should be avoided wherever possible. Pre-treatment with hydrogen peroxide is only necessary to eliminate carbonates or sesquioxides, and then pre-treatment with hydrochloric acid is used.

2.4.1.3.3 Sedimentation: It is essential to achieve complete dispersion of fine particles prior to carrying out a sedimentation test. The use of alkaline sodium hexametaphosphate is suggested.

2.4.1.4 Specific Gravity

The specific gravity (also called the particle density) (G_s) in residual soils may be unusually high or unusually low(G.E.Blight ,1997). The soil to be used in this test should be at its natural moisture content.Pre-test drying of the soil should be avoided as this tends to reduce the measured specific gravity as compared with natural moisture content sample.

2.5.3 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. In this test, a uniaxial vertical compressive load is applied on a thin sample, which is constrained from expanding horizontally by a rigid ring. A disk of soil is enclosed in a stiff metal ring and placed between two porous stones in a cylindrical container filled with water, as shown in Figure 2.1 metal load platen mounted on top of the upper porous stone transmits the applied vertical stress (vertical total stress) to the soil sample. Both the metal plates and the upper porous stone can move vertically inside the ring as the soil settles under the applied vertical stress. A series of known vertical pressures is applied to the sample using a weight and lever system that forms part of the loading frame of the oedometer. As each pressure increment is applied, readings of vertical compression are taken at regular time intervals until movement ceases; the next load is then applied. The measurements made provide a picture of both compression versus stress behavior and compression versus time behavior. Once the applied pressure reaches the vertical effective stress that acted on the sample in the field, water is added to the oedometer cell to ensure no water is lost by evaporation Wesley (2010).

Compression of laterally confined specimen is measured in terms of change of thickness. However, the test result is presented graphically in the form of pressure versus void ratio curve. This essentially means that the change in thickness has to be converted to change in void ratio.

The two conventions used in plotting the curve (i.e. e-p curve)

1. the use of natural scale for both co-ordinates
2. Plotting the void ratio on a natural scale and the applied pressure on a logarithmic scale.

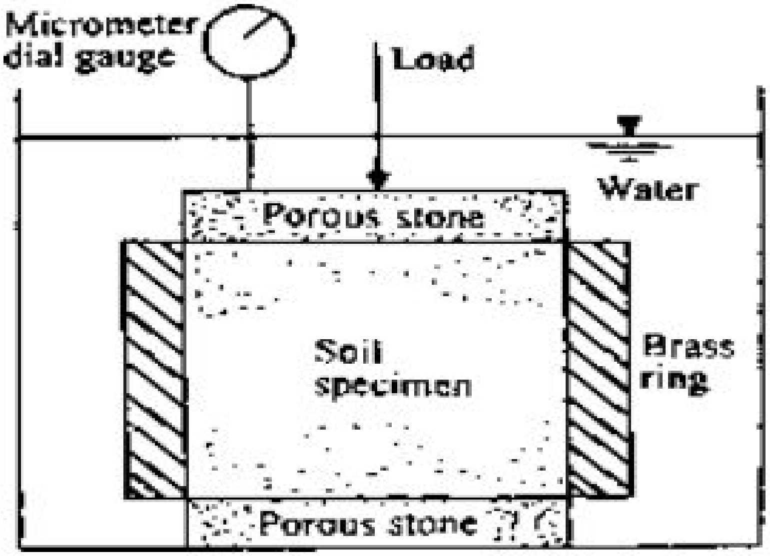


Figure 2.1 Consolidometer from B.Das(2008)

2.5.3.1 The e-log P curve

Figure 2.2.a shows a typical deformation versus log “t” graph. The graph consists of three distinct parts B.Das (2008):

- Upper curved portion (stage I). This is mainly the result of precompression of the specimen.
- A straight-line portion (stage II). This is referred to as primary consolidation. At the end of the primary consolidation, the excess pore water pressure generated by the incremental loading is dissipated to a large extent.
- A lower straight-line portion (stage III). This is called secondary consolidation. During this stage, the specimen undergoes small deformation with time. In fact, there must be immeasurably small excess pore water pressure in the specimen during secondary consolidation.

Note that at the end of the test, for each incremental loading the stress on the specimen is the effective stress. Once the specific gravity of the soil solids, the initial specimen dimensions, and the specimen deformation at the end of each load has been determined, the corresponding void ratio can be calculated. A typical void ratio versus effective pressure relation plotted on semi logarithmic graph paper is shown in Figure 2.2.b.

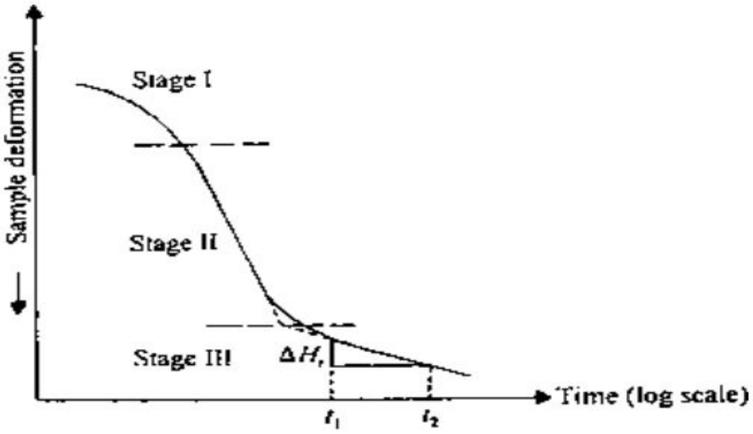


Figure 2.2.a Deformation versus Time (log scale)

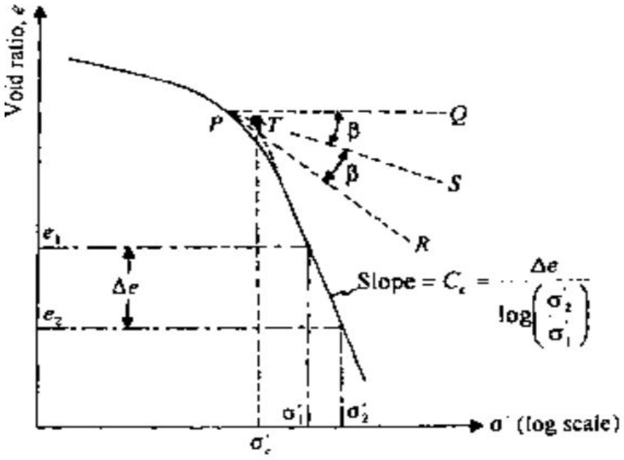


Figure 2.2.b Void ratio (e) versus logσ′, B.Das (2008)

The test is used to get compression index(C_c),recompression index(C_r),coefficient of consolidation(C_v), Modulus of volume compressibility(m_v)and secondary compression index(C_α) and using these coefficients one can estimate the settlement of a structure built on the soil.

2.5.3.2 Preconsolidation Pressure

The preconsolidation pressure is the maximum pressure to which an overconsolidated soil had been subjected in the past (Arora, 2004).In Figure 2.2.b, it can be seen that the upper part is curved; however, at higher pressures, e and logσ′ bear a linear relation. A soil in the field at some depth has been subjected to a certain maximum effective past pressure in its geologic history. This maximum effective past pressure may be equal to or less than the existing effective overburden pressure at the time of sampling. The reduction of effective pressure in the field may be caused by natural geologic processes or human processes. During the soil sampling, the existing effective overburden pressure is also released, which results in some expansion. When this specimen is subjected to consolidation test, a small amount of compression (that is, a small change in void ratio) will occur when the effective pressure applied is less than the maximum effective over-burden pressure in the field to which the soil has been subjected in the past. When the effective pressure on the specimen becomes greater than the maximum effective past pressure, the change in the void ratio is much larger, and the e –log σ′ relationship is practically linear with a steeper slope. The upper part is curved because when the soil specimen was obtained from the field, it was subjected to a certain maximum effective pressure. During the process of soil exploration, the pressure is released.

In the laboratory, when the soil specimen is loaded, it will show relatively small decrease of void ratio with load up to the maximum effective stress to which the soil was subjected in the past.

This is represented by the upper curved portion in Figure 2.2.b. If the effective stress on the soil specimen is increased further, the decrease of void ratio with stress level will be larger. This is represented by the straight-line portion in the e versus log σ′ plot.

The effect can also be demonstrated in the laboratory by unloading and reloading a soil specimen, as shown in Figure 2.2.c

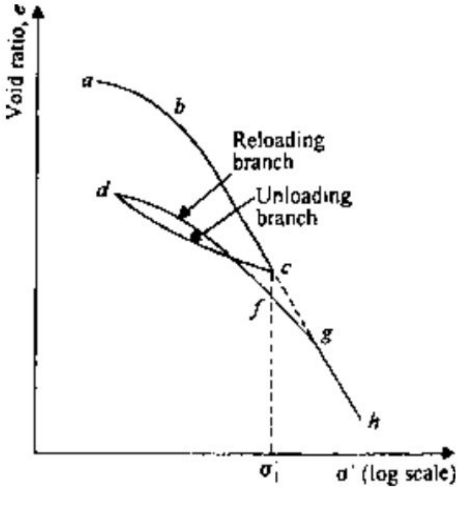


Figure 2.2.c Plot of void ratio versus effective pressure showing unloading and reloading branches (B.Das, 2008)

In Fig.2.2.c, cd is the void ratio–effective stress relation as the specimen is unloaded, and dfgh is the reloading branch. At‘d’ the specimen is being subjected to a lower effective stress than the maximum stress σ’ to which the soil was ever subjected. So ‘df’ will show a flatter curved portion. Beyond point ‘f’, the void ratio will decrease at a larger rate with effective stress, and ‘gh’ will have the same slope as ‘bc’.

Based on the above explanation, we can now define the two conditions of a soil, B.Das (2008):

1. Normally consolidated. A soil is called normally consolidated if the present effective overburden pressure is the maximum to which the soil has ever been subjected, i.e., present≥ past maximum.

2. Overconsolidated. A soil is called overconsolidated if the present effective overburden pressure is less than the maximum to which the soil was ever subjected in the past, i.e., present< past maximum.

In Figure 2.2.c, the branches ab, cd and df are the overconsolidated state of a soil, and the branches bc and fh are the normally consolidated state of a soil.

In the natural condition in the field, a soil may be either normally consolidated or overconsolidated. A soil in the field may become overconsolidated through several mechanisms

2.5.3.3 Coefficient of Consolidation (C_v)

It is used to calculate the rate of consolidation of a given clay layer. For a given load increment, the coefficient of consolidation, C_v can be determined from laboratory observation of time versus dial reading. The curve between dial gauge reading and time t obtained in the laboratory by testing the soil sample is similar in shape to the theoretical curve between the degree of consolidation (U) and the time factor (T_v) obtained from the consolidation theory. This similarity between the laboratory curve and the theoretical curve is used for the determination of the coefficient of consolidation (C_v) of the soil. Two methods to determine C_v

2.5.3.3.1 Square –root of time fitting Method

The method devised by Taylor, utilizes the theoretical relationship between U and $\sqrt{Tv}$ .The relationship is linear up to the value of U equal to about 60%.It has been further established that U=90%, the value of $\sqrt{Tv}$ is 1.15 times the value obtained by the extension of the initial straight line portion .At the point of 90% consolidation, the value of T=0.848

Therefore, $C_v = \frac{0.848 Hdr^2}{t_{90}}$,where H_{dr} = drainage path------------------(2.17)

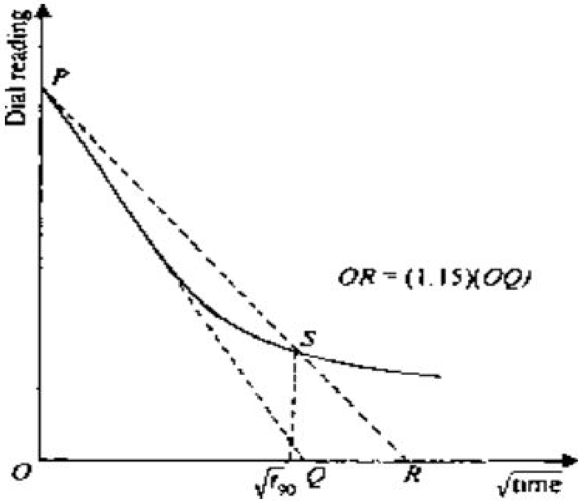


Figure 2.3 Root-time methods for determination of Cv

2.5.3.3.2 Logarithm of Time Fitting Method

The method is given by Casagrande uses the theoretical curve between U and logT_v. The curve consists of three parts

- An initial portion which is parabolic in shape
- A middle portion which is almost linear
- The last portion to which the horizontal axis is an asymptote. It is observed that the point of intersection of the tangent drawn at the point of inflexion on the curve and the asymptote of the linear portion gives the value of 100% consolidation.

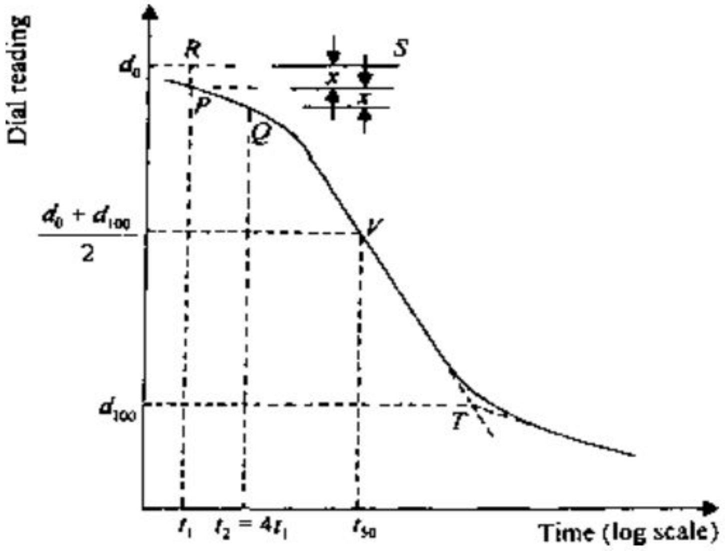


Figure 2.4 Logarithm of time to determine Cv

2.5.3.4 Compression Index (C_c)

The slope of the void ratio (e) versus logσ′ plot for normally consolidated soil is referred to as the compression index, C_c.

$$C_c=\frac{e1-e2}{log\sigma_2'-log\sigma1'}=\frac{\Delta e}{\log(\sigma2'/\sigma1')}\text{--} (2.18)$$

For undisturbed normally consolidated clay Terzaghi and Peck (1967) gave correlation for the compression index as;

$$C_c=0.009(LL-10),\quad \text{(B.Das, 2008)} \text{-----------------------------------} (2.19)$$

2.5.4 Computation of the amount of Settlement

Settlement of a structure is its vertical, downward movement due to volume decrease of a soil on which it is built. In other words the settlement is the gradual sinking of a structure due the compression of the soil below (Arora, 2004).During settlement analysis, prediction of the magnitude of settlement that a structure which is founded above a buried clay layer is likely undergo and the rate at which the settlement shall take place should done.

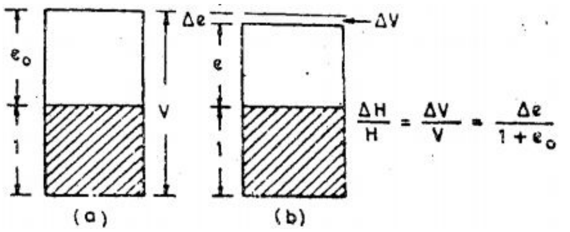


Figure 2.5 a) initial state of sample b) after compression

Referring to Fig.2.5 the volume of solids V_s is assumed as unity the void volume equal to e_o which is the void ratio before compression. If e_2 is the void ratio after the completion of primary consolidation, the decrease in void ratio is $\Delta e=e_o-e_2$.Then ΔH, the change in height of the consolidating layer or its settlement is given by the equation:

$$\frac{\Delta H}{H_o}=\frac{\Delta e}{1+e_o}$$

$$\Delta H=Sc=\frac{\Delta e}{1+e_o}H_o\text{--} (2.20)$$

When the consolidation test results are plotted between void ratio and effective stress arithmetically, the slope of the curve for the pertinent stress range, that is the coefficient compressibility a_v, can be used in settlement computation, $a_v=\frac{\Delta e}{\Delta\sigma'}$ and thus from equation (2.20)

$$Sc=\left(\frac{a_v}{1+e_o}\right)H_o\Delta\sigma'\text{---} (2.21)$$

In equation (2.21) the terms within the brackets are given the name coefficient of volume change or the coefficient of volume compressibility m_v,

$$m_v = \frac{a_v}{1+e_v} = \frac{\Delta e}{\Delta \sigma'}\left(\frac{1}{1+e_o}\right) \text{ --(2.22)}$$

Combining with equation (2.21)

$$Sc = m_v H_o \Delta\sigma' \text{ -- (2.23)}$$

Thus, it can be seen that the magnitude of settlement is a function of soil compressibility m_v, boundary condition H_o(thickness)and the loading condition $\Delta\sigma'$.If e is plotted against $\log\sigma'$,the slope of the virgin compression curve (straight line portion) is called the compression index,Cc.

$$C_c = \frac{\Delta e}{\log(\sigma'/\sigma' o)} \text{ --- (2.24)}$$

$$\text{Since, } C_v = \frac{K}{m_v \gamma_w} \text{ »} K = C_v m_v \gamma_w \text{--- (2.25)}$$

Combining equation (2.24) and (2.25)

$$Sc = \frac{\Delta e}{1+e_o} H_o = C_c \frac{H_o}{1+e_o} \log(\sigma'/\sigma'_o)$$

Hence, σ'_o is the present effective overburden Pressure and $\sigma' = \sigma'_o +$additional stress $\Delta\sigma'$ induced by an imposed load. Thus,

$$Sc = Cc \frac{Ho}{1+e_o} \log \frac{\sigma'_o + \sigma'}{\sigma'_o} \text{ --- (2.26)}$$

Where, C_c= Coefficient of Compressibility index,

H_o=Height of clay layer,

e_o =initial void ratio

2.5.5 Consolidation behavior of Residual Laterite soils

The formation of residual soils does not involve the processes of sedimentation and consolidation (aggregation) that are essential components of sedimentary soil formation. Even if the tropical soils are mostly found on unsaturated states, the weathering process may goes too far in depth up to 15m or more in thickness. Thus after some depth the soil may be on saturation state and if there is such a situation, to construct an earth dam or high-rise building or high fill embankment roads on it, or in order to use these soils as a construction material, one should expect that the consolidation process will be run when load is imposed, weather it is immediate or time dependent. However, all residual soils behave as if overconsolidated (G.E.Blight, 1997). Their compressibility is relatively low at low stress levels. However, once a threshold yield stress or equivalent preconsolidation stress has been exceeded, the compressibility increases. In most cases, the stress range will be such that the soil will remain within the pseudo-overconsolidated range of behavior (G.E.Blight, 1997). Cohesive residual soils, although products of markedly different geological processes are often characterized by e-log σ curves similar to those of transported clays of moderate to high sensitivity. They may appear to be preloaded, but the apparent preconsolidation pressure is a consequence of residual cohesive bonds between the particles rather than of effective pressure produced by a pressure overburden or by desiccation (R.B.Peck et al, 1974).Likewise, according to (Malomo S., and Ogusanwo O., 1983) the results of consolidation tests of laterite soils generally indicates that they are overconsolidated. However, the overconsolidation phenomenon is not due to the previous history of pressure but mainly due to cementation and the cementation in laterite soils is by means of iron and/or aluminium sesquioxide. The greater the interparticle bonding and cementation, the greater is the resistance to compression in the recompression range, and the more abrupt is the transition from recompression to compression, because major interparticle slip begins the process of destructuration at the preconsolidation pressure (Terzaghi et al,1996).

To assess the consolidation behavior of residual laterite soils, it is common to use oedometer and triaxial tests in the laboratory and standard penetration test, the pressure meter test, and plate loading test in the field (G.E.Blight, 1997). Consolidometer (Oedometer) tests are conducted for this thesis work.

2.5.5.2 Effect of Remolding on consolidation Behaviors of Laterite Soils

In addition to these two groups of natural soils (Transported and Residual), there is a third group of soils that are no longer natural and are therefore of much less importance to geotechnical engineers. These are soils that have been disturbed and/or remolded so that they no longer retain important characteristics of their undisturbed in situ state. This group includes soils prepared by sedimentation from artificial slurry, which are often used to investigate sedimentary soil behavior, and also compacted soils (Wesley, 2010).

The term destructured is frequently used these days to designate these soils and has a slightly different meaning from the term remolded. The term “destructured” means that the soil has been manipulated in such a way that bonds between particles or any other structural effects are destroyed, but the particles themselves are not altered. “Remolding” is a somewhat vague term but is generally taken to mean that the soil has been thoroughly manipulated, and any special characteristics associated with its undisturbed state are no longer present. With residual soils, thorough remolding may well completely destroy some particles as well as destroy the structure of the material (Wesley, 2010).

The properties of remolded soils are thus not governed by any form of structure, as is normally the case with most undisturbed soils, regardless of whether they are residual or sedimentary. Compacted clays may be an exception to this statement to a small extent, as it is possible that the compaction process does create some form of structure. For example, compacted clays tend to have a higher permeability in the horizontal direction than in the vertical direction, due to the horizontal layering effect produced by the compaction method. They may also have a greater stiffness in the vertical direction than the horizontal direction (Wesley, 2010).

3. Description of Sample Area

Project Location

Figure 3.1 Location Map of Sampling Area(Project Area)

Tongo-Begi-Mugi Road project is approximately 153km length and located in the far west part of Ethiopia in which 92.4% of the project lies within Oromia National regional state in western Wellega zone while the remaining 7.6% is in Benshangul Gumuz National regional state. The road is part of the "Local Network West" which connects Benshangul Gumuz, Gambela and Oromia regions to Addis Ababa and central regions of Ethiopia. The road passes through potentially rich agricultural areas for both rain-fed and irrigated agriculture as well as coffee growing. The project is intended to construct new road of gravel standards except the Begi-Gidami section, an existing gravel surfaced road built in rural road standard. The client of the project is Ethiopian Roads Authority (ERA). United consulting Engineers (UNICONE) in Joint Venture with Highway consulting Engineers (HCE) and Hamada Consulting Engineers conducted the detail Engineering Design and tender document preparation, which were in addition awarded the consultancy contract for the supervision services. Currently the road is under construction by Berhe Hagos General Contractor. The Ethiopian Federal Government has financed the project. The first lot of the project is almost completed and the wearing course of the road is covered by the red and reddish brown laterite soil.

But the primary interest of the thesis is investigating the consolidation and settlement behavior of laterite soil when it is used as a construction and foundation material such as for bridge foundation, high-rises buildings and on earth dams. Before dealing with this behavior it is important to see the topography, geological formation and climate of the area with regard to the actual situation whether it enhances development of laterite soils or restricts laterite formation.

3.1 Topography, Climate and Geology

The research area is dominantly rolling terrain to flat terrain which is suitable for the drain off water (well drained), which in turn enhances leaching out of silica and facilitates laterization. Most of the area is covered by grass, bushes and sparsely growing trees.

Climate of this targeted area and most of Benshangul Gumuz region is hot with seasonal rainfall ranging from 1600-2000mm annually, the mean period of onset of the summer ("Kiremit") rain is May and cessation is November. The whole area is characterized by dry winter and wet summer conditions with annual rainfall varying between 900mm and 2090mm as per the record at Gidami (UNICONE, 2007). And the temperature varies from 15˚ C to 35˚C and has valuable contribution for fast chemical reaction which plays a significant role for the development of laterite soils.

3.1.1 Geological Setting and Soil Characteristics

The geological make up of the area includes igneous rocks like basalt and granite and alluvial deposits. Igneous rocks are mainly quaternary undifferentiated sediments confined within low lying valleys light brown to reddish clayey soils (laterite) (UNICONE, 2007).Most of the soil starting from Nekemt along the road Nekemt--Nejo--Assosa ,Nejo-Jarso-Begi and Tongo-Begi-Gidami is red in color,coverd by grass(Savannah grass) with sparsely growing trees.

The soil formation factors such as topography, amount of rain, climate and parent rock seem to favor for Laterites formation. Generally formation of laterite soils favors rolling slope with good water runoff, distinct rainy season having warm summer. The minimum annual rainfall required for laterite formation is generally at least 750 mm (Lyon Association, 1971).

Laterite soils are formed by the in-situ weathering and decomposition of rocks under tropical condition. They are rich in sesquioxides (secondary oxides of iron, aluminum or both) and low in bases and primary silicates but may contain appreciable amounts of quartz and kaolinite. Soil engineering properties under this group are highly influenced by the presence of sesquioxides.

Sesquioxides appear to act as cementing agents which bind the other mineral constituents into clusters or aggregations. The color of pure laterite soils is red.

4. Laboratory Test Results and Analysis

Soil samples for this thesis work were collected from vicinity of Tongo-Begi-Gidami Road project which is part one (Contract-I) of Tongo-Begi-Mugi Road construction Project. Before excavation and collection of soil samples, site visit was made to get the most laterite soil representative site. Since laterite soils are infertile in nature they are easily identified

Table 4.1 sampling location

Serial/No	Test Pit Designation	Sampling Station	Offset	Sampling Depth(m)
1	TP1	5km back of 0+000	200m,RHS	1.2
2	TP2	3km back of 0+000	300m,LHS	1.5
3	TP3	11+700	1.5km,LHS	0.7
4	TP4-1	20+140	1.2km,RHS	0.7
	TP4-2			1.5
5	TP5-1	20+200	0.5km,LHS	0.7
	TP5-2			1.5

The test pits were dug by labor force. It was hard to dig more than five test pits having a depth of 2m (Table 4.1). A total of fourteen fresh samples were collected with seven disturbed and seven undisturbed (Block samples were dug out, since it was a dry season, it is too difficult to sample using sampler tubes) samples to run index properties and consolidation tests. All the collected samples were collected and tied with a plastic bag to prevent loss of natural moisture content. Proper care had been taken as much as possible. All samples were reddish brown in color. After the sample is transported to the project lab, the natural moisture content was determined immediately so as to prevent loss of moisture.

4.1. Index Properties

4.1.1 Effect of Temperature on Moisture Content Determination

The oven temperature 105˚c for water content determination is too high for certain clays and tropical soils. These soils contain loosely bound water of hydration or molecular water which can be lost at this high temperature resulting a change in the soil characteristics (Bowels, 1978,Zelalem,2005).This effect was checked using different oven temperatures. Drying oven temperatures of 105˚c and 50˚c with maximum relative humidity (RH) 30% were used to dry the samples. Two specimens from each sample were taken for moisture content determination.

One set of specimens 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. The values of the moisture content variations are compared and summarized in Table 4.2. As mentioned in section 2.4.1.1 moisture variations 4 - 6 % or more indicates that loosely bound molecular (structural) water is present. One can see the test results for this thesis work, that the differences in moisture contents for all sites under consideration are below 4% which means that the soil under investigation does not contain structural water. Hence for the subsequent tests those will be executed for this thesis work can use samples dried at oven temperature of 105˚c if necessary.

Table 4.2 Moisture content comparison by different oven temperatures

S/No.	Designation of test Pits	Sampling Depth(m)	Oven Tempreture(˚C)		Difference(%)
			105˚C	50˚C,RH=30%	
1	TP1	1.2	23.41	22.88	0.53
2	TP2	1.5	29.16	27.24	1.92
3	TP3	0.7	25.37	23.65	1.72
4	TP4-1	0.7	25.44	23.11	2.33
5	TP4-2	1.5	24.62	23.89	0.73
6	TP5-1	0.7	24.22	22.23	1.99
7	TP5-2	1.5	20.21	17.71	2.5

4.1.2 Grain size Analysis

The size of the particles that constitute soils may vary from that of boulders to clay. Grain size analysis is an attempt to determine the relative proportions of different grain sizes which make up a soil mass. Coarse soil particles are easily tested through nest of sieve mechanically. Finer soils on the other hand, follow hydrometer (Sedimentation) method. Here sodium hexametaphosphate is used as a dispersing agent. For soils comprising of coarser and finer size particles, both mechanical and hydrometer testing methods should necessary to be done.

The usual grain size testing and analysis method (adopted for temperate zone soils) consists of first drying the soil sample brought from the test pit and then pulverizing it before letting run through a nest of sieves. The soil passing the No. 10 sieve (fine soils) was separated by hydrometer analysis. Then, the result obtained was expressed by a plot of percent finer (passing) by weight against size of soil particles (Particle Diameter) in millimeters on a log scale. But in case of laterite soil pretest treatment has a significant effect on in grain size analysis, thus, both the dry and wet preparation have been done in order to compare and judge the test result.

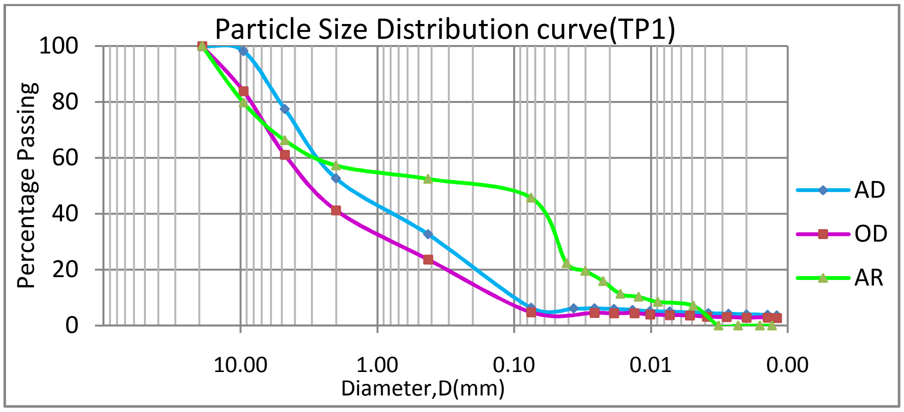
4.1.2.1 Dry Preparation

The soil sample brought from field was first air dried or oven dried at 105˚c and then pulverized before it was screened through the nest of sieves. During the preparation procedure, the sample was divided in to two portions using sieve No.10 (2mm). One portion contains only particles retained on No.10 sieve, while the other portion contains only particles passing 2mm sieve. Coarser particles were screened through a nest of sieves. Whereas soil particles passing the No. 10 sieve was 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 D 422-63). Apart from the method of preparation of the soil samples, sieve analysis tests were carried out essentially in accordance to ASTM D 422-63. 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 in side oven at temperature of 50˚c with maximum relative humidity 30% for at least 5 days. The room temperature was about 20˚c±2. The oven dried samples were prepared by drying the soils overnight at 105˚c.

4.1.2.2 Wet Preparation

Wet preparations were used for the as-received test samples (AR) i.e. soil samples which contain all their natural moisture are tested without any form of pre-test drying. The test samples were prepared in accordance with ASTM D 2217-85, Procedure B. This procedure provides that the samples be kept at moisture content equal to or greater than the natural moisture content. During the sample preparation procedure the test samples as received from the field were soaked until the coating material is fully softened. It is important that the fines adhering to the coarse particles should be removed and the fracturing of weak coarse particles should be prevented (Chairman, 1995; Lyon Associates, 1971, G.E.Blight, 1997). The soaked samples were washed in the manner stated in ASTM D 2217-85 Section 6.1.2 and Lyon Associates (1971), using No.10 (2.00-mm) & No.200 (0.075-mm) sieves. After washing, the materials retained on the No.10 (2.00-mm) & No.200 (0.075-mm) sieves were dried in an oven at a temperature of 105˚C overnight. Then the dried materials were used for the particle-size analysis using nest of sieves. The dry weights used in the computations for the gradation were calculated by using the wet weight and moisture contents of the test samples before sieving as recommended in (Lyon Associates, 1971). In addition to this, the hydrometer analysis was used in the determination of particle-sizes less than sieve No. 200 (0.075-mm) in accordance with the procedures stated in ASTM D 422-63.

Classification:



Fig,4.1 Sample TP1

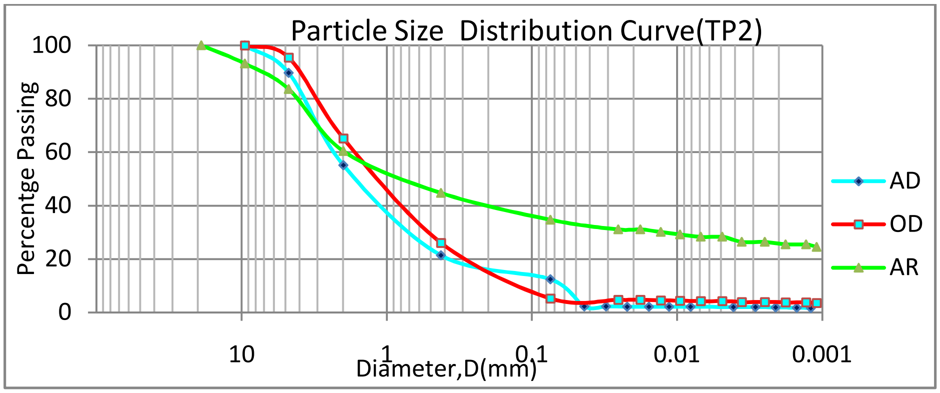


Fig.4.2 Sample TP2

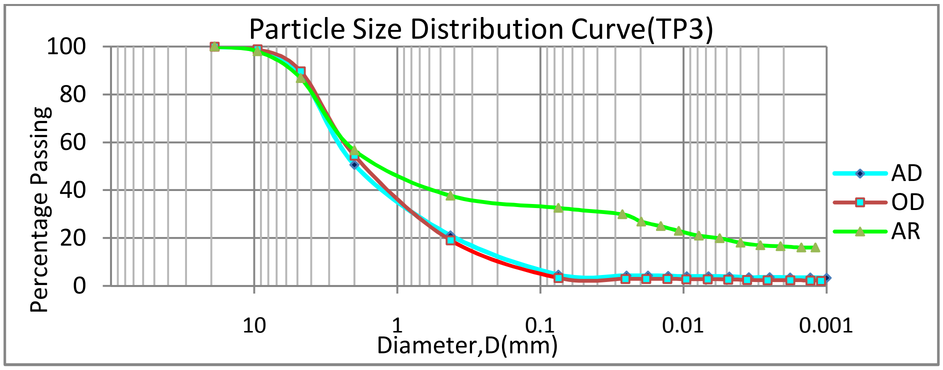


Fig.4.3 Sample TP3

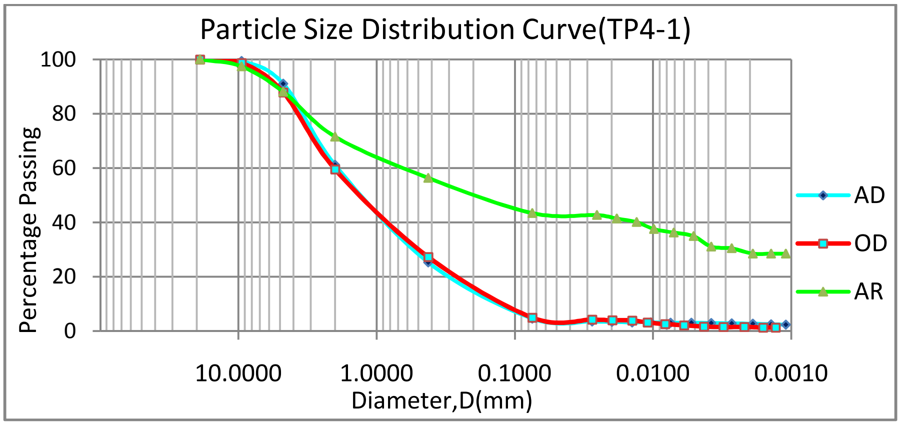


Fig.4.4a Sample TP4-1

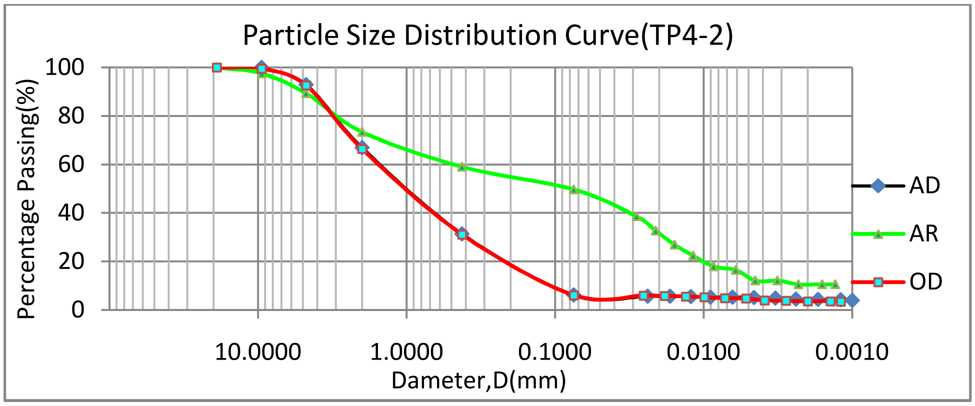


Fig.4.4b Sample TP4-2

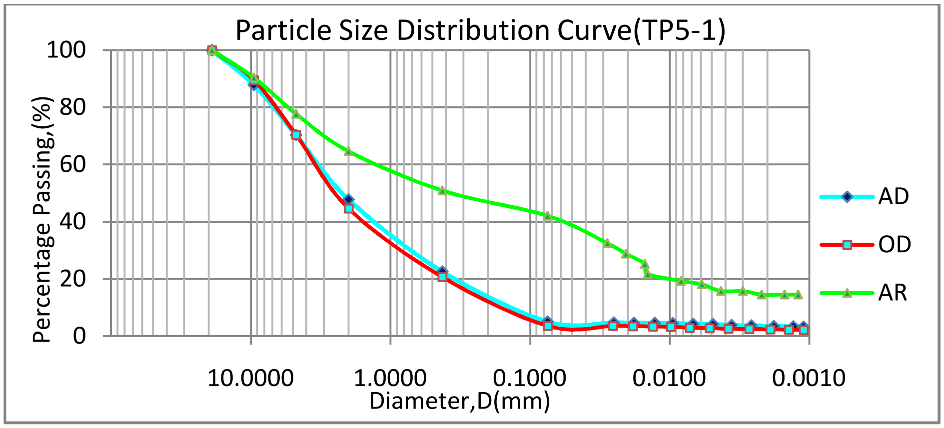


Fig.4.5a Sample TP5-1

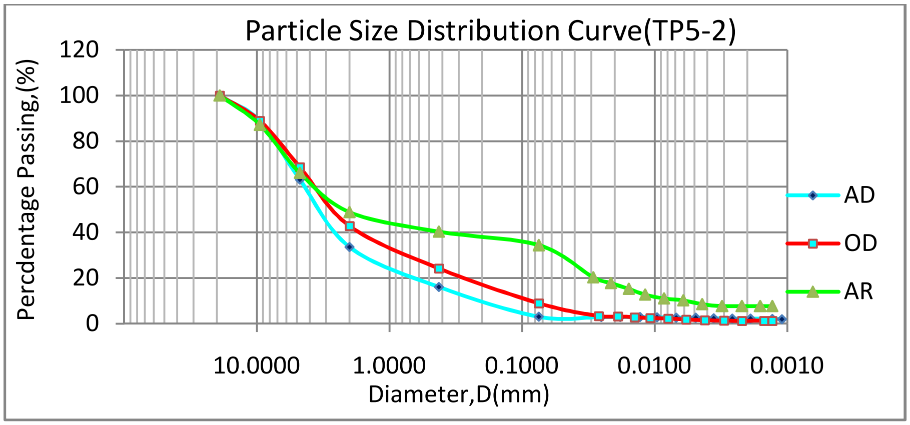


Fig.4.5b Sample TP5-2

Figure 4.1-4.5b Particle Size Distribution curve at different testing/pretreatment condition laterally

To see the variation of laterite soils along the profile, two test pits TP4 and TP5 were selected and the particle size distribution has done at different testing conditions.

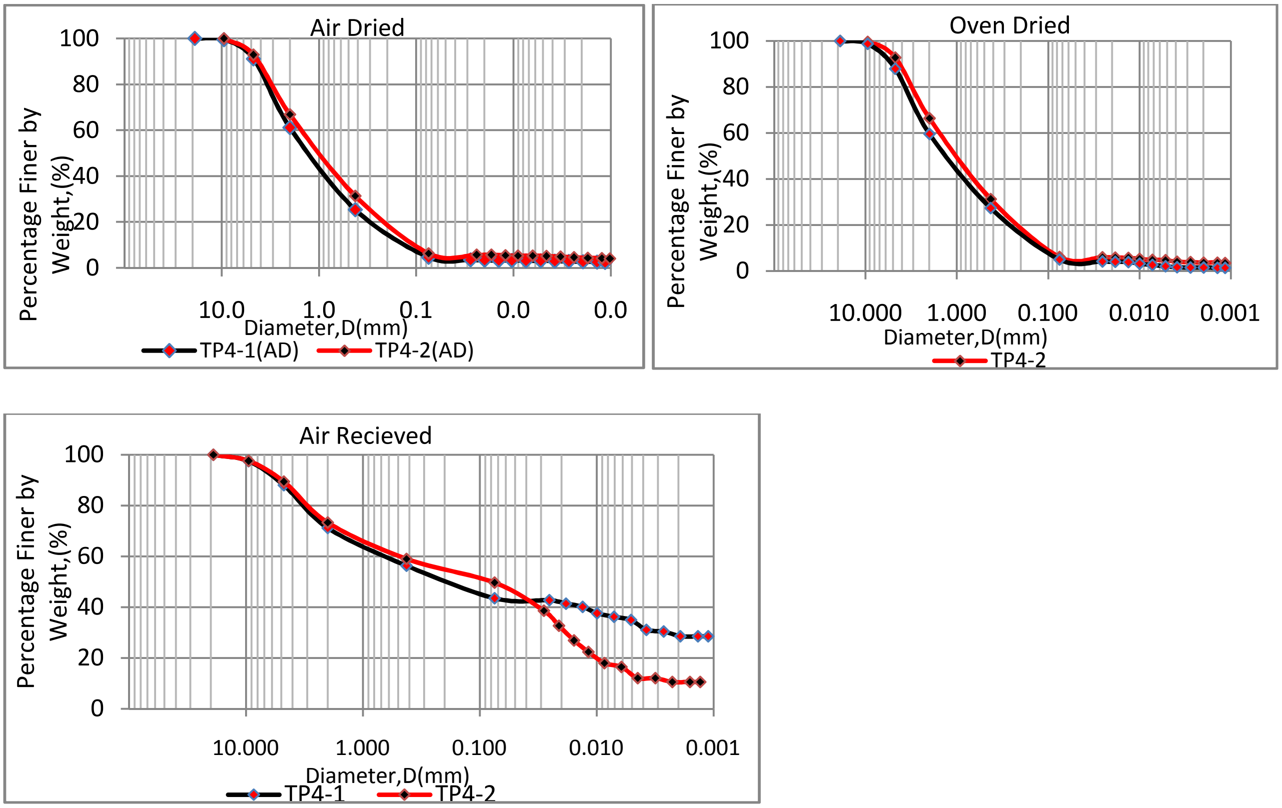
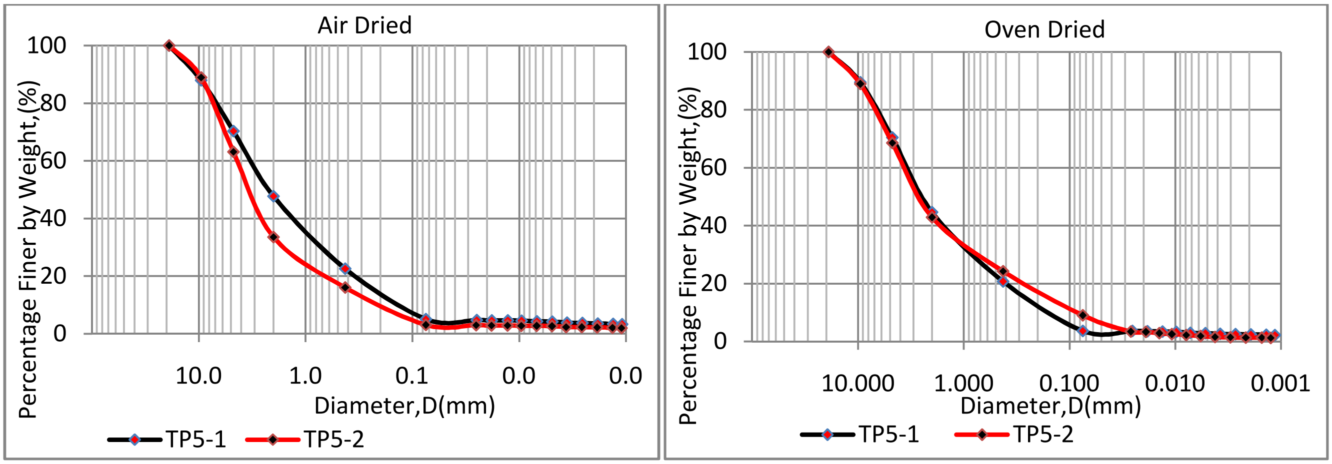


Figure 4.6 TP4-1 and TP4-2 Particle distribution curve along the profile at different testing condition



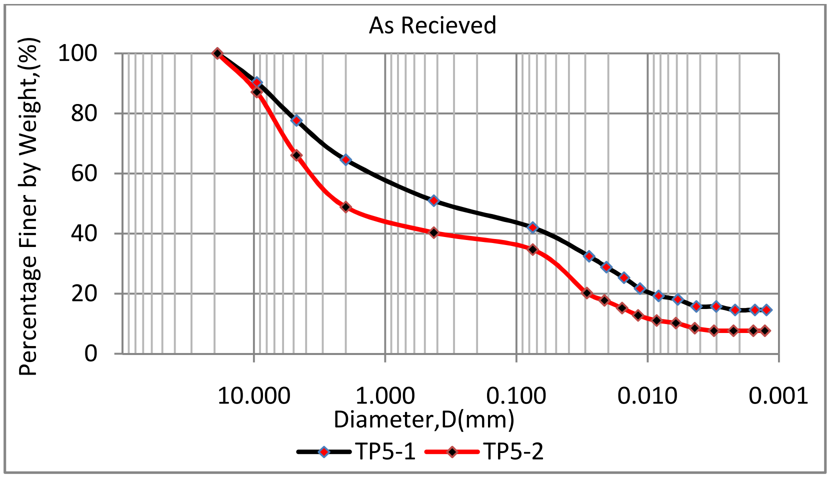


Figure 4.7 TP5-1 and TP5-2 particle distribution curve along the profile at different testing condition

Test Result and Discussion

Although there are different methods of classification of soils, the Unified Soil Classification System (USCS) later modified by American Society for Testing and Materials (ASTM: D2487-98) and the American Association of State Highway and Transport Officials (AASHTO) system are used for this thesis work for the reason that these two classification systems take particle size distribution and Atterberg Limit into consideration notwithstanding that AASHTO classification is used only for road constructions.

Classification according to USCS (ASTM: D2487-98)

The original form of this system was proposed by Casagrande in 1942 for use in the airfield construction works undertaken by the Army Corps of Engineers during World War II. In cooperation with the U.S. Bureau of Reclamation, this system was revised in 1952.At present; it is used widely by engineers (ASTM Test Designation D-2487). This system classifies soils into two broad categories:

1. Coarse-grained soils that are gravelly and sandy in nature with less than 50% passing through the No.200 sieve. The group symbols start with a prefix of G or S, G stands for gravel or gravelly soil, and S for sand or sandy soil.

2. Fine-grained soils are with 50% or more passing through the No. 200 sieve. The group symbols start with prefixes of M, which stands for inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils according to B.Das (2010). Therefore, according to this classification system:

- Gravel 75mm - 4.75mm
- Sand 4.75mm - 0.075mm
- Silt less than No.200 (0.075mm) and nonplastic or very slightly plastic
- Clay less than No.200 (0.075mm) exhibit plasticity or putty- like properties.

Classification according to AASHTO

The AASHTO soil classification system is used to determine the suitability of soils for earthworks, embankments, and road bed materials (subgrade—natural material below a constructed pavement; subbase—a layer of soil above the subgrade; and base—a layer of soil above the subbase that offers high stability to distribute wheel loads). Similar to USCS this system classifies soils in two broad class but according to AASHTO, granular soils are soils in which 35% or less are finer than the No. 200 sieve (0.075 mm). Silt-clay soils are soils in which more than 35% are finer than the No. 200 sieve.

▪ Gravel	75mm - No.10 (2mm)
▪ Sand	No.10 (2mm) - No.200 (0.075mm)
▪ Silt	No.200 (0.075mm) - 0.002mm
▪ Clay	less than 0.002mm

On the other hand Lyon Association sets average grain size classification for Laterite soils (Lyon, 1971)

- Lateritic Clays < 0.002 mm
- “ Silts = 0.002 ~ 0.06 mm
- “ Sands = 0.06 ~ 2 mm
- “ Gravels = 2 ~ 60 mm

But this classification system is nearly similar to AASTO classification system

Table4.3 Classification according to AASHTO and USCS (ASTM D 2487-98)

Sample Designation	Testing Condtion	Liquid Limit	Plasticity Index	**Percentage of particle Size**				**Classification According to AASHTO**	**Percentage of particle Size**			**Classification According to USCS (ASTM:D 2487-98)**
				Gravel	Sand	Silt	Clay	Group	Gravel	Sand	Silt and Clay	
TP1	OD	52	16.53	58.80	36.42	1.93	2.85	A-2-7	38.85	56.37	4.78	Well Graded sand with Gravel
	AD	54.2	22.46	47.40	46.20	2.59	3.81	A-2-7	22.50	71.10	6.40	Poorly Graded Sand with Silt and Gravel
	AR	65.6	23.53	42.77	11.51	39.10	6.62	**A-7-5**	33.72	20.56	45.72	Poorly Graded Gravel with Silt
TP2	OD	55.4	28.58	34.83	59.89	0.90	4.38	A-2-7	4.49	90.23	5.28	Well Graded sand with Silt
	AD	56.4	24.57	44.90	42.69	10.64	1.67	A-2-7	10.20	77.38	12.42	Silty sand
	AR	65.8	27.27	39.58	25.70	9.24	25.48	A-2-7	16.32	48.96	34.72	Silty sand with Gravel
TP3	OD	56.3	27.05	45.53	51.09	0.67	2.71	A-2-7	10.20	86.42	3.38	Well Graded sand with Silt
	AD	59	25.03	44.90	42.69	10.64	1.67	A-2-7	11.75	83.57	4.68	Well Graded sand
	AR	66	23.22	43.34	24.10	16.03	16.53	A-2-7	13.17	54.27	32.56	Silty sand
TP4-1	OD	52	23.64	40.43	54.59	3.60	1.38	A-2-7	12.15	82.87	4.98	Well Graded sand with Silt
	AD	58.8	23.41	38.82	56.60	2.02	2.56	A-2-7	8.90	86.52	4.58	Well Graded sand
	AR	62.8	27.16	28.85	27.68	14.91	28.56	**A-7-5**	11.89	44.64	43.47	Silty sand
TP4-2	OD	53.5	22.91	33.60	60.33	2.42	3.65	A-2-7	7.28	86.65	6.07	Well Graded sand with Silt
	AD	57	23.75	33.05	60.66	2.09	4.20	A-2-7	7.11	86.60	6.29	Silty sand
	AR	97	28.52	26.62	23.65	39.10	10.63	**A-7-6**	10.61	39.66	49.73	Silty sand
TP5-1	OD	54	25.09	55.33	41.06	1.33	2.28	A-2-7	29.58	66.81	3.61	Well Graded sand with Gravel
	AD	58	26.27	52.23	42.68	1.68	3.41	A-2-7	29.62	65.29	5.09	Well Graded sand with Silt
	AR	59.2	28.08	35.39	22.50	27.55	14.56	**A-7-5**	22.31	35.58	42.11	Silty sand with Gravel
TP5-2	OD	51.5	23.91	57.19	33.83	7.68	1.30	A-2-7	31.44	59.58	8.98	Well Graded Sand with Silt and Gravel
	AD	54.8	25.08	66.41	30.57	0.95	2.07	A-2-7	36.84	60.14	3.02	Well Graded sand with Gravel
	AR	68.8	24.68	51.16	14.15	26.97	7.72	A-2-7	33.91	31.40	34.69	Silty Gravel with sand

Laterite soils behave different shape in Particle Size distribution curve, when we are doing the test at different testing conditions. When a soil sample is oven dried at 105˚c or even air dried before test, this increases cementation due to oxidation of iron and aluminum sesquioxide. To observe effect of pretest drying seven samples from five test pits were collected and the test has done on three testing conditions, i.e. oven dried at 105˚c; air dried or oven dried at 50˚c and as received from the field and the test result is as shown previously from Fig.4.1-4.5b. In order to specify profile or in depth variation of five test pits two test pits (TP4 & TP5) were selected and two samples from each test pit from 0.7m and 1.5m depth is taken and grain size and other index property tests were done .As said on earlier figures from 4.1-4.5b shows the shape of particle size distribution curve in different testing conditions. In all samples the curves in oven dry and air dry case have nearly similar in shape and particle size distribution and the constituent of the soil is more of gravel and sand whereas in the wet sieve analysis the particle distribution curve is different from the two cases and show relatively well graded distribution. It is easily understood that pre-test oven drying and/or air dry causes laterite soils to form high cementation bond. But wet sieving method helps to wash coated particle on soil grains, prevent formation of cementation bond due to drying and breakage of soil grains during pulverization but we have to understand that classification systems do not take into account the properties of intact materials as found in nature. Since the foundation materials of most engineering structures are undisturbed, the properties of intact materials only determine the soil behavior during and after construction. The classification of a soil according to any of the accepted systems does not itself enable detailed studies of soils to be dispensed with altogether. Solving flow, compression and stability problems merely on the basis of soil classification can lead to disastrous results. However, soil classification has been found to be a valuable tool to the engineer. It helps the engineer by giving general guidance through making available in an empirical manner the results of field experience (Murthy, 2007).

4.1.3 Specific Gravity

Specific gravity is the ratio of the mass of a unit volume of a material at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of a soil is used in calculating the phase relationships of soils (that is, the relative volumes of solids to water and air in a given volume of soil).The term solid particles is typically assumed to mean naturally occurring mineral particles that are not readily soluble in water. Therefore, the specific gravity of materials containing extraneous matter (such as cement, lime, and the like), water-soluble matter (such as sodium chloride), and soils containing matter with a specific gravity less than one, typically require special treatment or a qualified definition of their specific gravity(ASTM, D854-98). The specific gravity tests were carried out under different testing conditions(air dried, oven dried and as received pre treatment conditions) and the test result is attached in the Appendix-C and summarized as seen in Table 4.4.

Table 4.4 Specific Gravity at different testing conditions

S/No.	Test Pit Designation	Specific Gravity at different testing conditions		
		Air Dried	Oven Dried at 105°C	As received
1	TP1	3.10	3.09	3.05
2	TP2	2.75	2.91	2.94
3	TP3	2.89	2.89	2.92
4	TP4-1	2.89	3.00	3.08
5	TP4-2	3.01	3.01	3.07
6	TP5-1	3.00	3.00	3.03
7	TP5-2	3.11	3.07	3.03

The specific gravity of laterite soils is unusually high or unusually low during different pretreatment conditions as seen on Table 4.4. Air dried and oven dried test results of TP1, TP3, TP4-1, TP4-2 are relatively similar than the as received. All test results are in the range between 2.75 to 3.10 and according to Morin W.J., and Todor P.C. (Lyon Association, 1971) specific gravities of lateritic soils may vary 2.6 - 3.4. Such a high specific gravity test value is due to its high iron content.

4.1.4 Atterberg Limit

4.1.4.1 General

The water contents corresponding to the transition from one state to another are termed as Atterberg Limits and the tests required determining the limits are the Atterberg Limit Tests. These tests indicate the range of the plastic state (plasticity is defined as the property of cohesive soils which possess the ability to undergo changes of shape without rupture) and other states (Murthy, 2007).As stated under section 2.4.1 Atterberg limits of laterite soils is highly affected by pretreatment and duration of mixing times.

4.1.4.2 Test Procedure

In order to consider and observe pretreatment and mixing time effect, the soil sample is prepared in three conditions as oven dried at 105˚C, Air dried 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 for at least 5 days until they were dried thoroughly, and as received (keeping the soil sample on its natural moisture content before test).The room temperature during the test varies between 20- 23˚C which is at the project site(on contract:2,Gidami-Mugi having 'Woina Dega' climate, at main camp laboratory room ,that is why the temperature is lowered with relative to research site). For air and oven-dried test samples, portion of the dried soil samples passing No. 40 (0.425mm) sieve were kept wet for a period of 24 hrs for moisture content equilibration before carrying out Atterberg limit tests whereas in the case of wetly prepared soil samples the moist portion, i.e. wet soil portion which was passed while it was washed on No. 40 (0.425mm) sieve were kept drying, i.e. reducing the moisture content, until the mass reaches a putty-like consistency (such as 15-25 drops of the cup in the liquid limit test). Finally, Atterberg limit tests were carried out by reducing the moisture content (from wet to dry point) (Lyon Associates, 1971, G.E Blight, 1997, Million, 2009).

Table 4 .5 Atterberg Limit at different testing condition and mixing time

Testing condition	Representative Sample	Mixing Time	Liquid Limit(LL)	Plastic Limit(PL)	Plasticity Index(PI)	PI Difference
AR	TP1	5 min	65.6	42.1	23.5	7.9
		30 min	71.5	40.0	31.5	
	TP2	5 min	65.8	38.5	27.3	2.9
		30 min	71.2	41.1	30.1	
AD	TP1	5 min	54.2	31.7	22.5	2.5
		30 min	63.8	38.8	25.0	
	TP4-1	5 min	58.8	35.4	23.4	1.8
		30 min	67.8	42.6	25.2	
OD	TP4-2	5 min	53.5	30.6	22.9	5.4
		30 min	61.2	32.9	28.3	
	TP5-1	5 min	54	28.9	25.1	8.1
		30 min	64.9	31.7	33.2	

Table 4.6 Atterberg Limit at different Testing condition, Mixing time=5min, recommended by Blight (1997)

Sample Designation	Mixing Time	Liquid Limit(LL)			Plastic Limit(PL)			Plasticity Index(PI)		
		AR	AD	OD	AR	AD	OD	AR	AD	OD
TP1	5 min	65.6	54.2	52	42.1	31.7	35.4	23.5	22.5	16.6
TP2	5 min	65.8	56.4	55.4	38.5	31.8	26.8	27.3	24.6	28.6
TP3	5 min	66	59	56.3	42.8	34.0	29.2	23.2	25.0	27.1
TP4-1	5 min	62.8	58.8	52	35.6	35.4	28.4	27.2	23.4	23.6
TP4-2	5 min	57	57	53.5	28.5	33.2	30.6	28.5	23.8	22.9
TP5-1	5 min	59.2	58	54	31.1	31.7	28.9	28.1	26.3	25.1
TP5-2	5 min	68.8	54.8	51.5	44.1	29.7	27.6	24.7	25.1	23.9

4.1.4.3 Test Result and Discussion

During Atterberg limit test, in order to investigate the effect of manipulation some samples are done for 5minute and 25 minute mixing time. If there is a significant difference (>5%) in Atterberg limit result this indicates that the aggregation of clay size particles are broken down due to manipulation G.E Blight (1997). From laboratory test results (Table 4.5 and Table 4.6) one can see that the soils in these areas are sensitive to handling and disturbance /manipulation/. The more the soil is disturbed, the finer the aggregates become and the higher the Atterberg limit. As the greater duration of mixing (i.e., the greater the energy applied to the soil prior to testing) the larger the resulting liquid limit, and to a lesser extent, the larger the plasticity index. This result is in agreement with compiled notes of Fourie A.B in Blight (1997) and previous researchers (Zelalem, 2005; Million, 2008).

4.1.4.3 Effect of mixing times on Atterberg Limits

In general it is believed that, the greater the energy applied and the longer mixing time will result in more extensive breaking down of cemented bonds between clay clusters and within peds and thus the formation of greater portions of fine particles. This effect can be seen on the extent to which liquid limit value increases for an increment of mixing time.

Excessive manipulation during testing leads to crumbling of the soil structure and disaggregating; these effects produce fines which in turn result in higher liquid limit values. Hence, the mixing time should be kept to a minimum, generally about 5 minutes for each point in the plasticity tests for Laterite soils (Lyon Associates, 1971). To address this problem, liquid limit and plastic limits of seven samples which were collected from the research area, Tongo-Begi-Gidami, were checked by conducting different testing procedures on air-dried, oven-dried and as-received test samples as discussed previously. The difference in the liquid limit for 5 minutes and 30 minutes mixing times is shown together with the drying conditions in Table 4.2 & 4.3. ASTM D 421-85 and D 4318-95 test procedures were adopted for AD and OD test samples while ASTM D 2217-85 procedure B with a modification of testing procedure when carrying out the liquid limit tests. This is done by carrying out the test from wet point to dry point while the wet soil paste getting drier and drier rather than carrying out from dry to wet points. This procedure was recommended by different authors (Lyon Associates, 1971; Blight, 1997 and Charman, 1995) during testing of lateritic soils for the AR test samples conditions. As can be seen in Table 4.2 significant differences were observed in between the liquid limit values obtained from the specimens mixed for 5min and 30min, this indicates that the cementation of the clay particles and/or laterization of the soils were broken down up on manipulation /longer mixing times. Hence, the greater duration of mixing, i.e. the greater the energy applied to the soil prior to testing, the larger the resulting liquid limit, and to a lesser extent, the larger the plasticity index. This is in agreement with compiled notes of Fourie A.B (Blight, 1997). One can also observe from the test results shown in the tables that the soils in the research areas are disintegrated by elongation of mixing time. Thus, when Atterberg limit tests are carrying out for such soils it is recommended to limit the mixing times not more than 5 minutes; and the soils should be broken-down by soaking in water, and not by drying and grinding as can be done conventionally for temperate zone soils. Hence, the testing Schedule for Laterites should include the following instructions as recommended by Fourie A.B in Blight (1997). That is:

a) Limit the mixing times to not more than 5 minutes

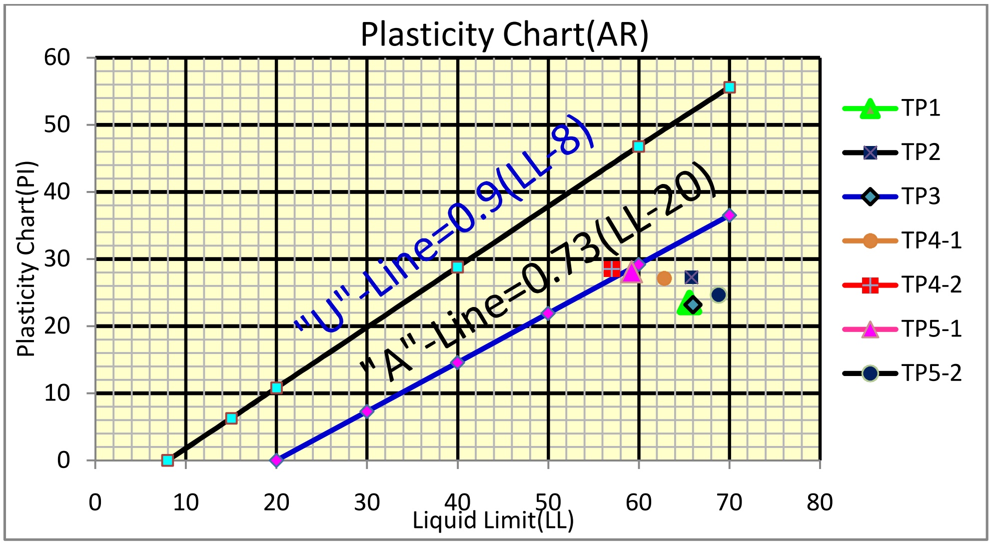
b) Make use of fresh soil for each moisture content point in Atterberg Limit tests, as well as in compaction tests.

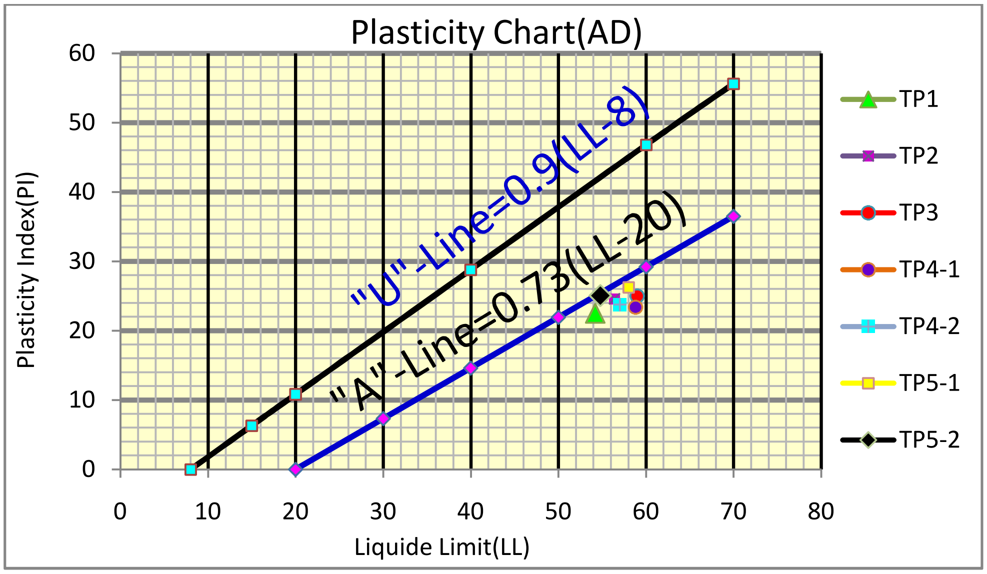
In addition to these, the soils should be broken-down by soaking in water, not by drying and grinding, Lyon Associates (1971), G.E.Blight (1997).

4.1.4.4 Plasticity Chart

The plasticity chart is intended as a means of dividing or classifying fine-grained soils into groups expected to have similar engineering properties. The Plasticity index (PI) is plotted against the Liquid Limit (LL), so a soil can be represented by a single point on this chart. There is a dividing line separating soils with clay characteristics from those with silt characteristics; this line is known as the A-line. Casagrande established the A-line by conducting Atterberg limit tests on a wide range of soils at the same time examining their engineering characteristics. This enabled him to establish the line separating the two fine-grained soils as silt and clay. In addition to a division based on the A-line, Casagrande also used a vertical division at LL = 50 to divide clays and silts into two further groups those of high LL and those of low LL. Regardless of whether or not it is used for systematic classification of soils, the plasticity chart is a very useful indicator of the likely engineering properties of fine-grained soils. The Atterberg limit values by themselves are not particularly reliable as indicators of soil behavior but become very useful when plotted on the plasticity chart. Soils that plot well above the A-line generally have poor engineering properties. They are likely to be of high compressibility and low shear strength and display shrink/swell behavior.

On the other hand, soils that plot below the A-line tend to have the opposite characteristics and be good engineering materials. Along with this general trend will be another trend dependent on the LL. For a given position in relation to the A-line, the higher the LL the less desirable will be the engineering characteristics. These are general trends followed by most soils, though there are exceptions. The position the soil occupies on the plasticity chart can be regarded as a good indicator of the intrinsic properties of the soil. The plasticity chart is plotted for different testing conditions i.e. oven dried at 105˚c, air dried and as received samples. For the As received sample only TP4-2 is above the A-Line near to it. All the remaining test samples are below the A-line. When we see air dried samples all the samples are below the A-line. In the case of Oven dried samples (at 105˚c) TP2 is above the A-line, TP3, TP4-1 and TP5-2 are just on the A-Line and the rest are below the A-Line. But the governing test for laterite soils is the As Received condition so that the test result in this testing condition reveals that the soil has good engineering property.


Plasticity Chart(AR)
Plasticity Chart(PI)
Liquid Limit(LL)
"U"-Line=0.9(LL-8)
"A"-Line=0.73(LL-20)
0
10
20
30
40
50
60
70
80
TP1
TP2
TP3
TP4-1
TP4-2
TP5-1
TP5-2


Plasticity Chart(AD)
Plasticity Index(PI)
Liquide Limit(LL)
"U"-Line=0.9(LL-8)
"A"-Line=0.73(LL-20)
0
10
20
30
40
50
60
70
80
TP1
TP2
TP3
TP4-1
TP4-2
TP5-1
TP5-2

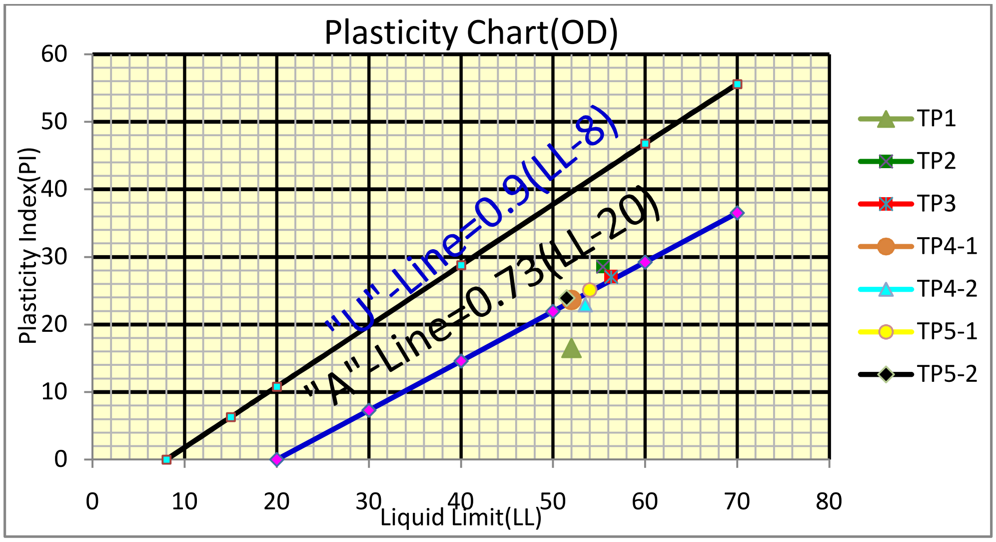


Figure 4.8a-c Plasticity Chart at different testing conditions

4.2 Consolidation and Settlement Behavior of Laterite Soils

4.2.1 Test Procedure

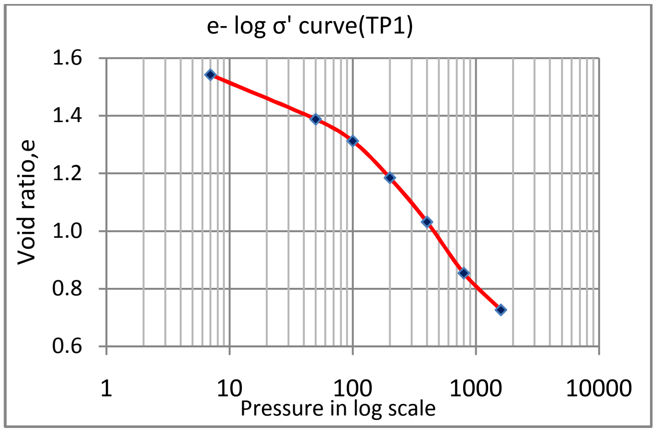
To measure both magnitude and rate of settlement, consolidation tests are carried out in a device known as an oedometer (consolidometer), a diagrammatic view of which is shown in Figure 2.5. An undisturbed sample of the soil is trimmed into a circular ring having a diameter of about 50 mm and a thickness of 20 mm (20 mm is the most commonly used thickness to reduce test time but above 20mm possibly to use). The porous stone allow water to drain from top and bottom of the sample when vertical pressure is applied. The sample is then placed between the porous stones in a holding cell and set up in a loading frame. A series of known vertical pressures is then applied to the sample using a weight and lever system that forms part of the loading frame of the oedometer. As each pressure increment is applied, readings of vertical compression are taken at regular time intervals until movement ceases usually a load kept for 24hrs; the next load is then applied. The measurements made provide a picture of both compression versus stress behavior and compression versus time behavior. Once the applied pressure reaches the vertical effective stress that acted on the sample in the field, water is added to the oedometer cell to ensure no water is lost by evaporation. The standard load increment shall consists of load increment ratio of one which is obtained by doubling the pressure on the soil to obtain values of approximately 12,25,50,100,200,400 kPa etc…,and change in height of a specimen on a dial gauge is recorded at 0.1,0.25,0.5,1,2,4,8,15,30,60,120,……..1440 minutes just immediately after the application of each load increment. A total of nine consolidation tests have done of which seven undisturbed, and two remolded samples to see effect of remolding on consolidation behavior of Laterite soils. As it is seen on the consolidation curve(Figure 4.14) even if in actual cases remolding of laterite soils causes the breakdown of the bond between particles (cementation bond), loss of natural state and compressibility will be high to the reverse, remolding of samples prevents ease of dissipation of water from the soil sample. Basically the permeability of residual soils including laterites is related to their formation and depends on the macro and micro structures of the soil. During remolding this macro and micro structure will be destroyed due to compaction and this by far blocks small pores, minimizes permeability of the soil and has an effect on the consolidation test result. Consequently, in some cases relative to undisturbed samples, remolded samples are less compressible. In one dimensional consolidation test the direction of water is assumed as one directional (vertical), on the other hand in remolded samples the compaction process is done layer by layer and this may create small void space between layers so that expulsion of water may occur laterally whilst the sample is laterally confined and this minimizes the compressibility of the soil sample (Budhu, 2011; ASTM D-2435-96; Das, 2008).

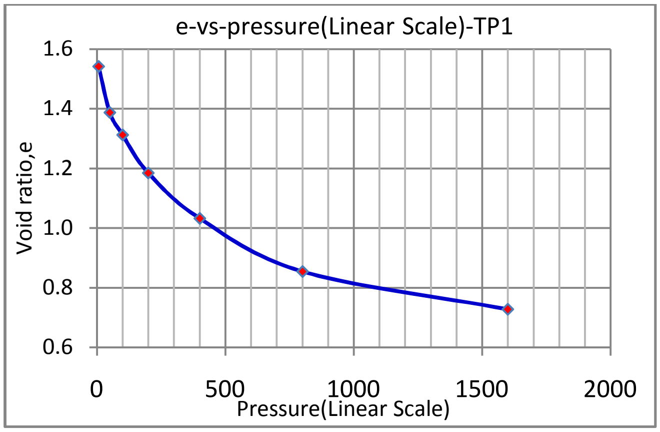
4.2.2 Test Result and Discussion

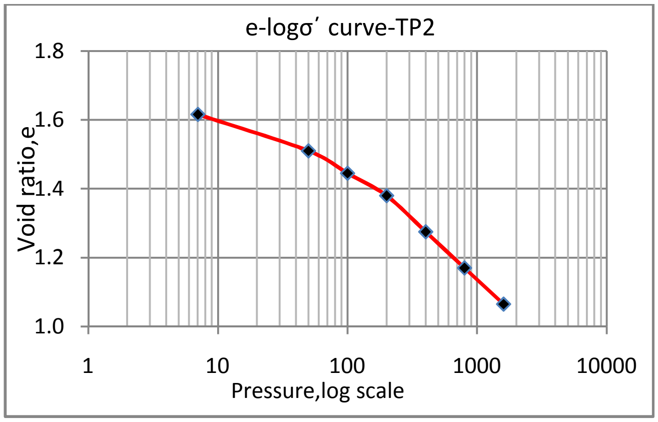
After the consolidation test has been done, the void ratio versus effective stress graph is plotted using the semi-logarithmic scale and /or linear scale. From Figure 4.9 one can observe that the shape of the consolidation curves of all the test samples show preconsolidation pressure. But this preconsolidation pressure is not derived from the stress history of the soil as in the case of transported soils but due to the strong cementation behavior of residual laterite soils. (Malomo S., and Ogusanwo O., 1983) clearly stated that the results of consolidation tests of laterite soils generally indicate that they are overconsolidated; however, the overconsolidation phenomenon is not due to the previous history of pressure but mainly due to cementation.

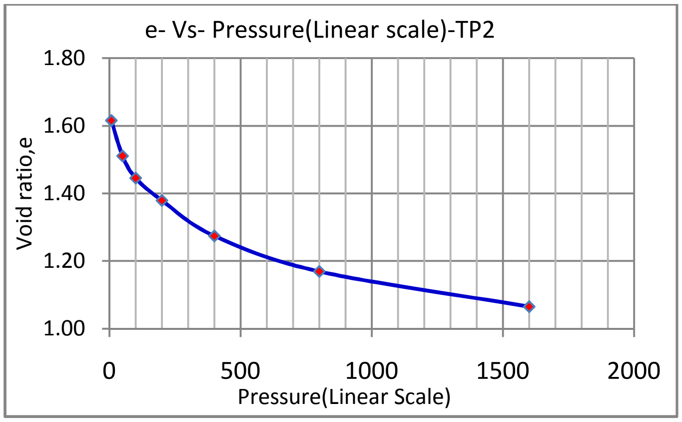
In addition according to Terzaghi et al.(1996),the greater the interparticle bonding and cementation of this soils, the greater is the resistance to compression in the recompression range, and the more abrupt is the transition from recompression to compression, because major interparticle slip begins the process of destructuration at the preconsolidation pressure. Likewise, as per R.B.Peck et al. (1974) cohesive residual soils, although products of markedly different geological processes are often characterized by e-log σ curves similar to those of transported clays of moderate to high sensitivity; they may appear to be preloaded, but the apparent preconsolidation pressure is a consequence of residual cohesive bonds between the particles rather than of effective pressure produced by a pressure overburden or by desiccation.

Therefore, the preconsolidation pressure in laterite soil is vividly due its cementation behavior developed during laterization process. The consolidation curves of TP1, TP2, and TP4-1and TP4-2 (Figure 4.9) are identical in shape and show continuously compressing behavior after the preconsolidation pressure whereas TP3, TP5-1 and TP5-2 (Figure 4.9) at lower and intermediate loading show the same shape as TP1, TP2, TP4-1 and TP4-2 (Figure 4.9) but at higher loading their compressibility increase and finally minimized. In the logarithmic plot at the initial pressures the soil behaves as stiff material due to their cementation bond and when the pressure increases, the compressibility also increases continuously. Similarly, when the consolidation test result is plotted using linear scale, all the soil samples show continuous deformation as per the pressure increases especially in lower and middle stress levels and rather decreases at higher stress levels.


e- log σ' curve(TP1)
Void ratio,e
1.6
1.4
1.2
1.0
0.8
0.6
1
10
100
1000
10000
Pressure in log scale


e-vs-pressure(Linear Scale)-TP1
Void ratio,e
1.6
1.4
1.2
1.0
0.8
0.6
0
500
1000
1500
2000
Pressure(Linear Scale)


e-logσ′ curve-TP2
Void ratio,e
1.8
1.6
1.4
1.2
1.0
1
10
100
1000
10000
Pressure,log scale


e- Vs- Pressure(Linear scale)-TP2
Void ratio,e
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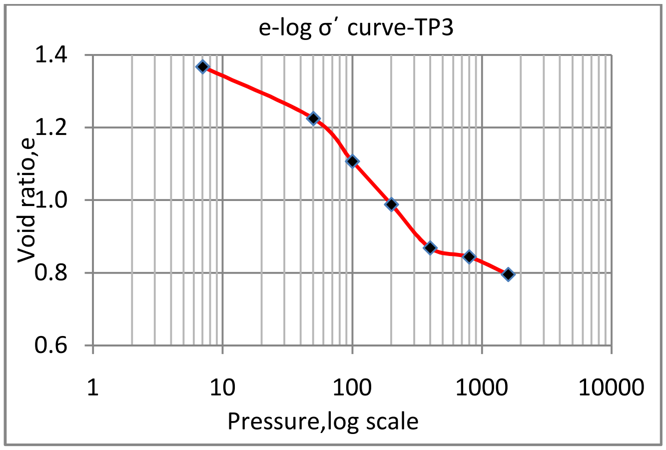
1.60
1.40
1.20
1.00
0
500
1000
1500
2000
Pressure(Linear Scale)


e-log σ′ curve-TP3
Void ratio,e
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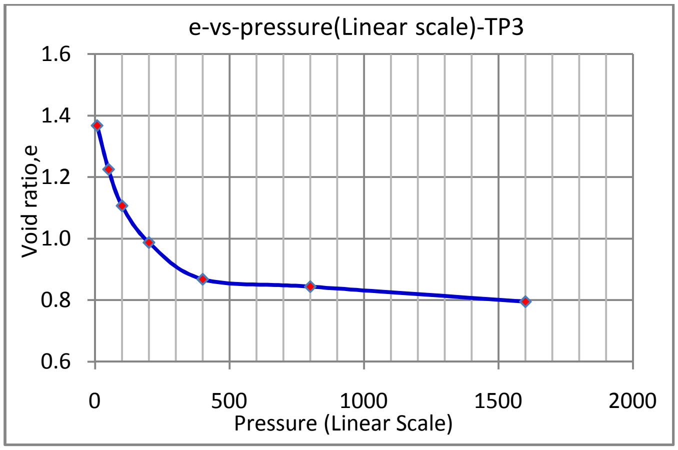
1.2
1.0
0.8
0.6
1
10
100
1000
10000
Pressure,log scale


e-vs-pressure(Linear scale)-TP3
Void ratio,e
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1.4
1.2
1.0
0.8
0.6
0
500
1000
1500
2000
Pressure (Linear Scale)

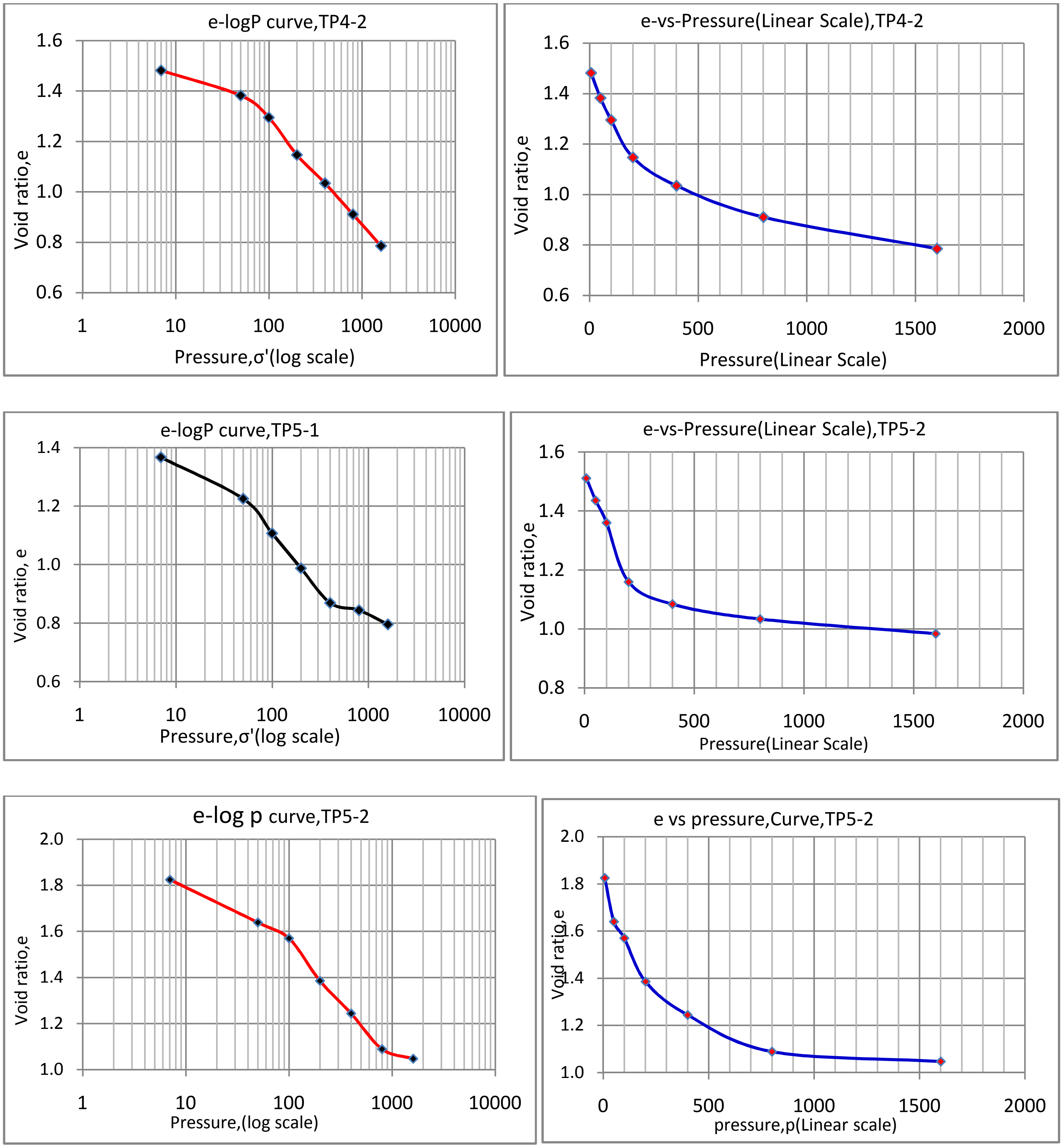
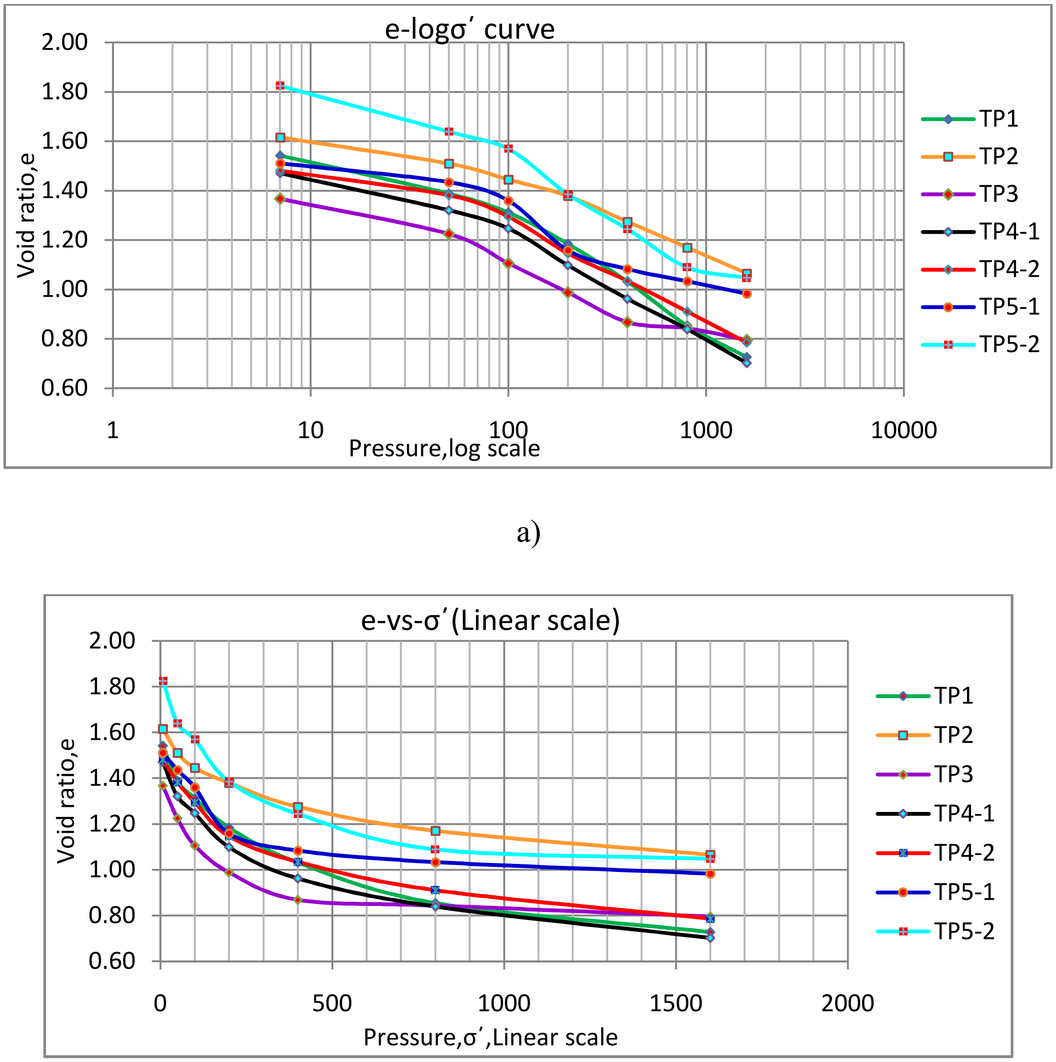


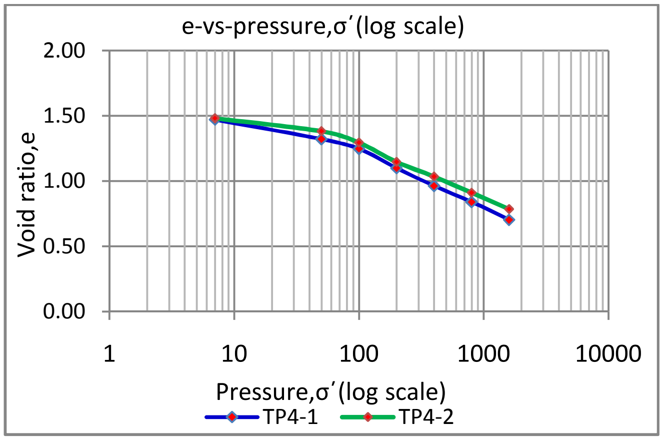
Figure 4.9 Consolidation test result using a) log scale and b) linear scale for all test samples

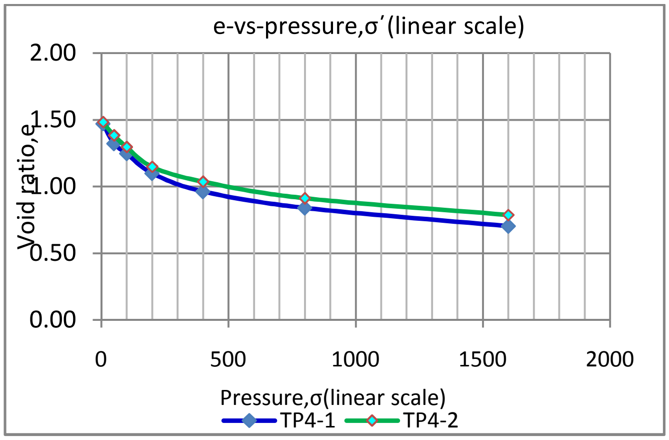


a)

b)

Figure 4.10a) shape of consolidation curve in logarithmic scale for all test samples b) Shape of consolidation curve in linear scale for all test





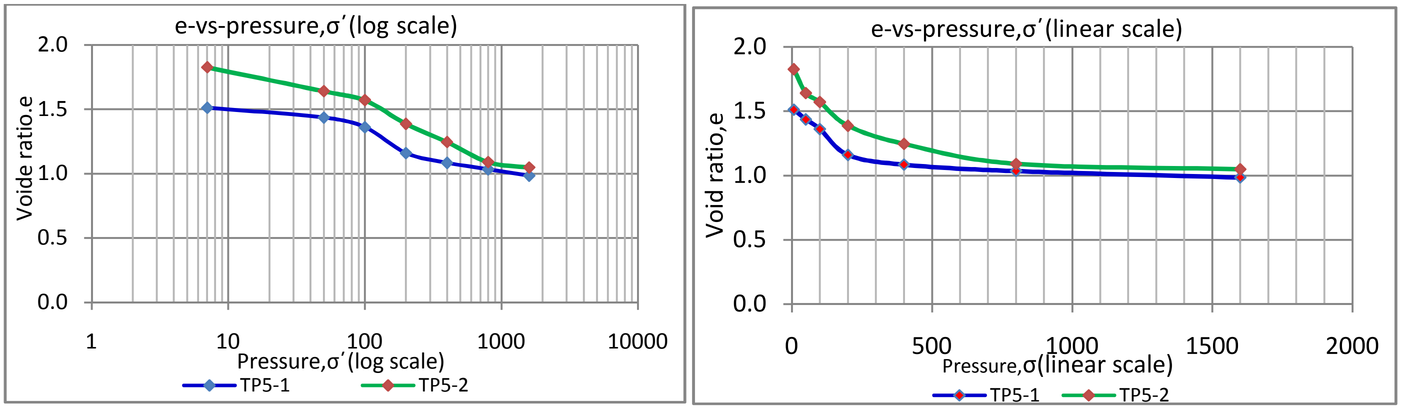


Figure 4.11 Consolidation test result along the profile both in a) logarithmic and b) linear scale

From the graph one can see that the consolidation curve of TP4-1 and TP5-1(Figure 4.11) are below TP4-2 and TP5-2 (Figure 4.11) respectively and shows that as the depth increases weathering is minimized and vice-versa. The chemical tests, which were done previously around the research area vividly, depict that degree of weathering/laterization decreases as the depth increases Zelalem (2005), Dibisa (2008). Thus, TP4-1 and TP5-1 samples have high degree of laterization or strong cemented bond and as a result of this they are stiffer than TP4-2 and TP5-2.

Determination of Preconsolidation Pressure (σ'_c): Preconsolidation pressure is the maximum effective overburden pressure to which the soil at the sample depth has been subjected in the past. The preconsolidation pressure from an e -versus $\log\sigma'_c$ plot is generally determined by a graphical procedure suggested by Casagrande (1936), as shown in Figure 2.6b B.Das (2008). The steps are as follows:

1. Visually determine the point P (on the upper curved portion of the e versus log p plot) that has the maximum curvature.
2. Draw a horizontal line PQ.
3. Draw a tangent PR at P.
4. Draw the line PS bisecting the angle QPR.
5. Produce the straight-line portion of the e versus $\log\sigma'$ plot backward to intersect PS at T.
6. The effective pressure corresponding to point T is the preconsolidation pressure σ'_c.

Thus according to the above procedure the preconsolidation pressure for the different samples is determined graphically as presented in Fig 4.12

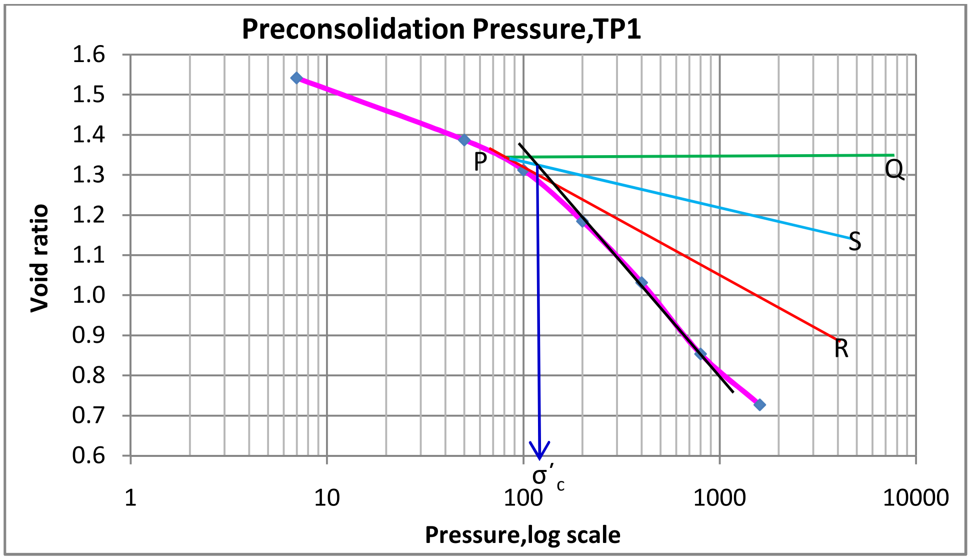


Fig.4.12a

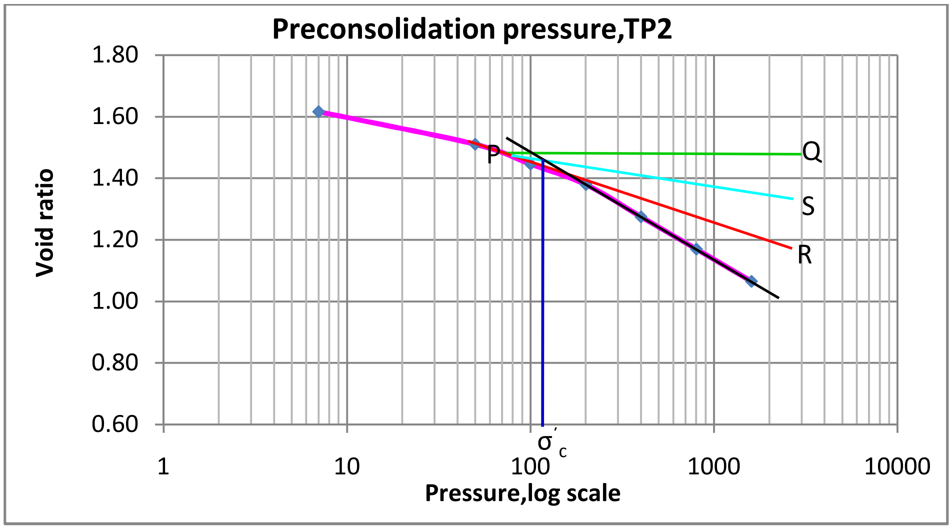


Fig.4.12b

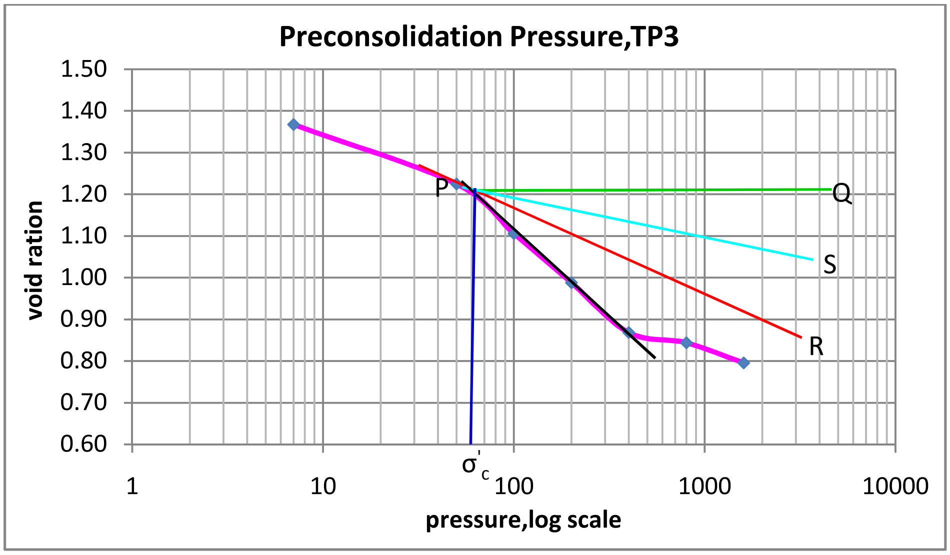


Fig.4.12c

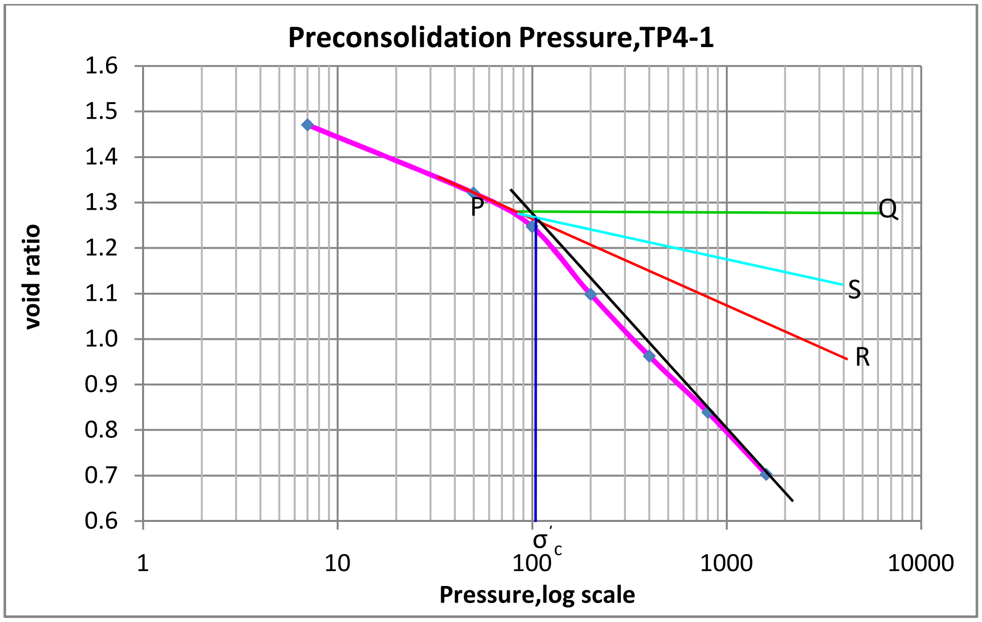


Fig.4.12d

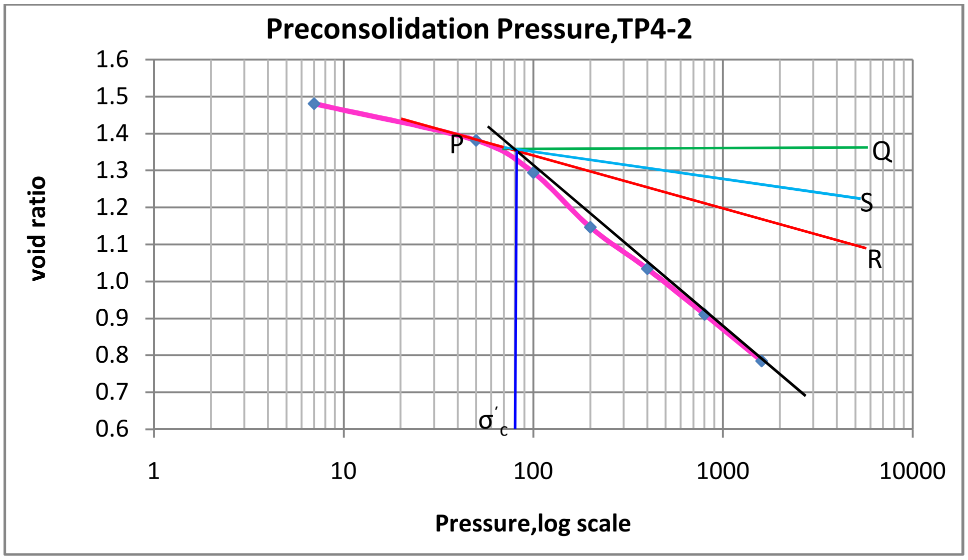


Fig.4.12e

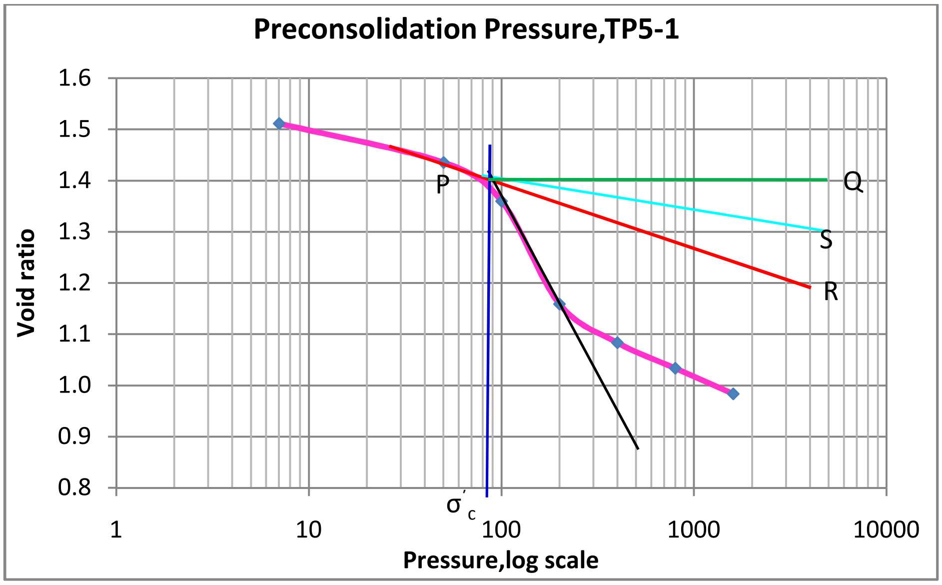


Fig.4.12f

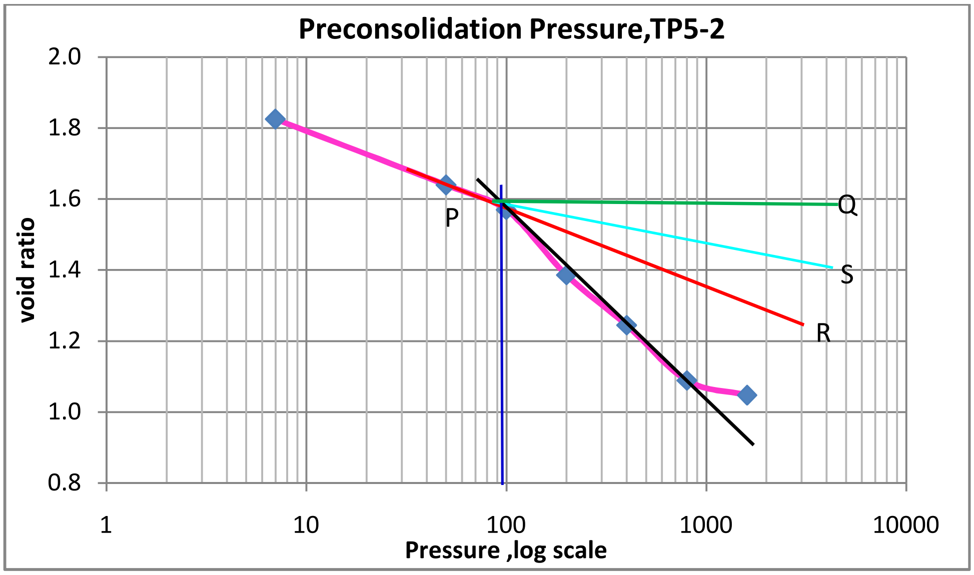


Fig.4.12g

Figure 4.12 Determination of Preconsolidation Pressure

As stated briefly under section 2.5.5 and section 4.2.2 the consolidation test result of laterite soils is undoubtedly shows overconsolidation but this is not due to the stress history of the soil rather the degree of cementation bond of laterite soils. Initially when the load is applied the soil resists the load due to its strong concretionary material developed (iron and/or aluminum sesquioxide) but as the load increase this cementation bond starts to break and the specimen becomes continuously compressible. Therefore, that is why the preconsolidation pressure has been observed at the initial phase of the consolidation curve.

Table 4.7 Preconsolidation Pressure

Test Pits	TP1	TP2	TP3	TP4-1	TP4-2	TP5-1	TP5-2
Preconsolidation Pressure(kPa)	≈110	≈105	60	≈100	80	≈85	≈100

4.2.2.1a Coefficient of consolidation (C_v)

This is a coefficient containing the physical constant of a soil affecting its rate of volume change. It indicates the combined effects of permeability and compressibility for a given void ratio range. 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 two popular methods that can be used to calculate the coefficient of consolidation (C_v). Taylor (1942) proposed one method, called the Square - Root of time method and Casagrande and Fadum on (Budhu, 2011) proposed the other method, called the Log -Time method. The Square-Root of time method utilizes the early time response, which theoretically should appear as a straight line in a plot of square root of time versus displacement gauge reading.

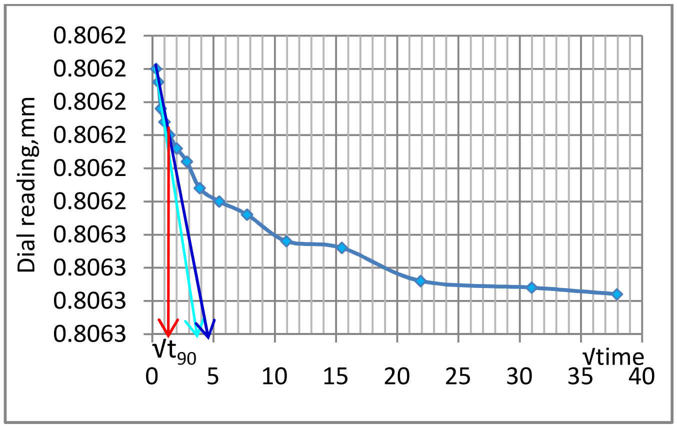
Square –Root of Time Fitting Method

The method devised by Taylor, utilizes the theoretical relationship between degree of consolidation (U) and the square root of time factor ($\sqrt{Tv}$).The relationship is linear up to the value of U equal to about 60%.It has been further established that U=90%, the value of $\sqrt{Tv}$ is 1.15 times the value obtained by the extension of the initial straight line portion .At the point of 90% consolidation, the value of T=0.848

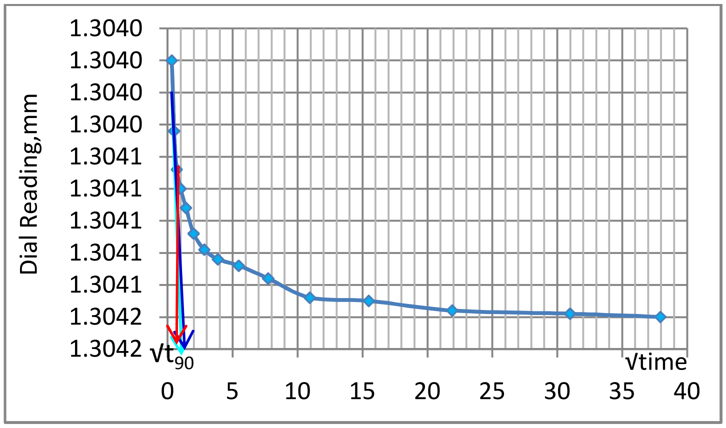
Therefore, $C_v = \frac{0.848Hdr^2}{t_{90}}$, where H_{dr} = drainage path

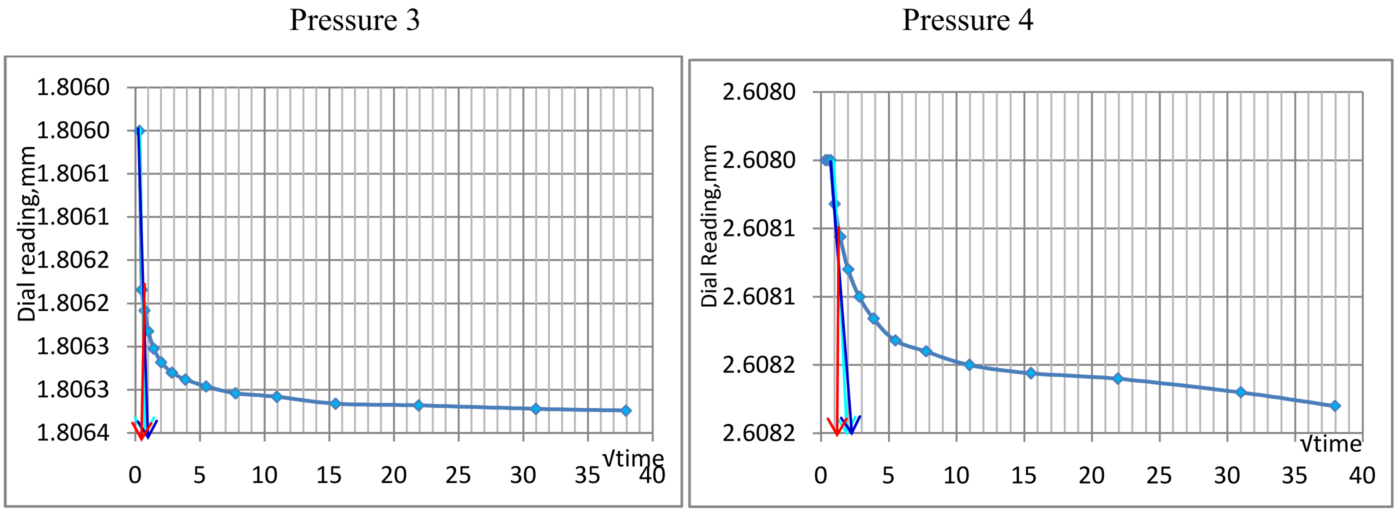
The dial reading versus square root of time is plotted for each load increment and from the plot the square root time for 90% consolidation is obtained and C_v for each load increment has been found using the above equation.

Pressure 1



Pressure 2





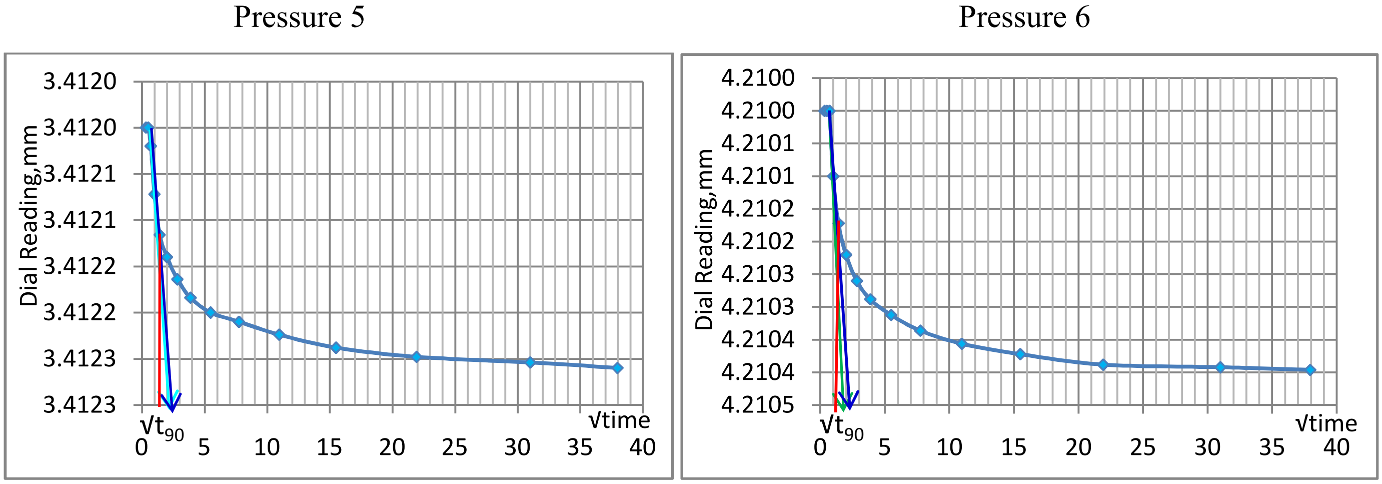


Figure 4.12 Square-time fitting method of determination of Cv

It is obvious that one value of coefficient of consolidation, C_v is obtained for each stress increment. C_v, obtained from square root time curve fitting method can be plotted as ordinate with the average effective stress as the abscissa. From the curve the approximate C_v value for any given stress increment can be read off and used as in the time-settlement computation (G.Ranjan and A.S.R. Rao, 2000).

Table 4.8 Cv value for each test sample as per stress increment

Pressure,σ′ (Kpa)	TP1	TP2	TP3	TP4-1	TP4-2	TP5-1	TP5-2
	C_v(cm^2/min)	C_v(cm^2/min)	C_v(cm^2/min)	C_v(cm^2/min)	C_v(cm^2/min)	C_v(cm^2/min)	C_v(cm^2/min)
50	0.00848	0.00707	0.0106	0.00848	0.007067	0.014133	0.00848
100	0.0106	0.00942	0.00163	0.007709	0.000499	0.003262	0.004988
200	0.00848	0.01696	0.00848	0.005653	0.004038	0.012114	0.0106
400	0.00848	0.00652	0.00047	0.000785	0.007709	0.006057	0.001663
800	0.00707	0.00652	0.00058	0.001073	0.00265	0.012114	0.003262
1600	0.00339	0.00771	0.00471	0.005653	0.007067	0.007067	0.000785

4.2.2.1b Compressibility Index (C_c)

The slope of the straight portion of the e-logσ′ plot for normally consolidated soil is referred to as the compression index C_c (Figure 4.13).Thus, $C_c = \frac{e1-e2}{\log\sigma'1-\log\sigma'2} = \frac{\Delta e}{(\log\sigma'1/\log\sigma'2)}$,Residual soils doesn't show normally consolidation behavior rather acting as overconsolidated, while considering as a normally consolidated soil the index of compression from the test result is as shown in table 4.9.

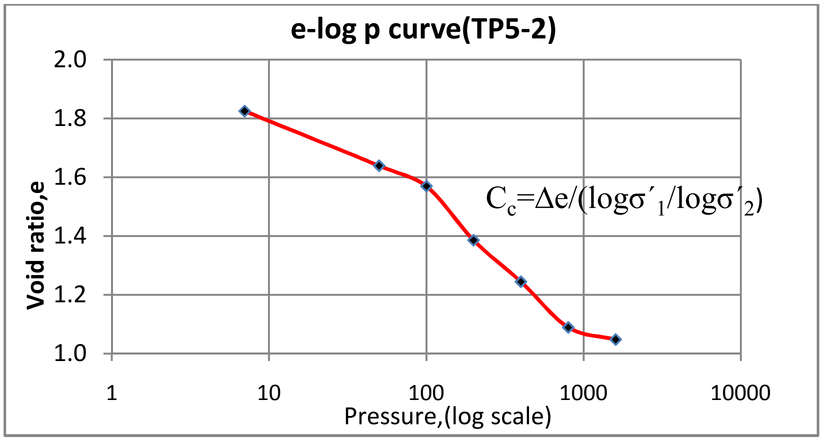


Figure 4.13 Determination of compression index, Cc

4.2.2.1c Coefficient of Compressibility (a_v)

When the consolidation test results are plotted between void ratio and effective stress arithmetically, the slope of the curve for pertinent stress range, that is, the coefficient of compressibility a_v can be used for the computation of settlement.

$a_v = \frac{\Delta e}{\Delta \sigma'}$, the value of a_v is as shown on table 4.8

4.2.2.1d Coefficient of Volume compressibility (volume change)

m_v can be determined as : $m_v = \frac{av}{1+eo}$

Table 4.9 Coefficients determined from consolidation test

Coefficients	**Designation of Test Pits**						
	TP1	TP2	TP3	TP4-1	TP4-2	TP5-1	TP5-2
$C_c=\Delta e/\log(\sigma'/\sigma'_o)$	0.366	0.348	0.395	0.439	0.4	0.459	0.493
$a_v=\Delta e/\Delta\sigma'$	0.0001585	0.0001305	0.0000608	0.0000608	0.0001570	0.0000833	0.000494
e_o	1.54	1.48	1.37	1.47	1.48	1.51	1.82
$m_v=a_v/(1+e_o)$	0.0000624	0.0000526	0.0000257	0.0000246	0.0000633	0.0000332	0.0001752

4.3 Settlement Computation

Settlement of the soil can be calculated in different ways as stated in section 2.5.4 by using the coefficients found from the consolidation test result. But in this research work only two methods will be incorporated, using compressibility index (C_c) and coefficient of volume compressibility (m_v).

Using equations (2.23) and (2.26), one can compute the settlement of a structure as follows.

Table 4.10 Settlement Calculation using compressibility index, Cc

Test Pit	C_c	e_o	H_o(m)	σ'_o(kPa)	σ'_f(kPa)	$\Delta\sigma'$(kPa)	$(\sigma'_o+\Delta\sigma')/\sigma'_o$	$Sc(m)=C_c (H_o/1+e_o)\log(\sigma'_o+\Delta\sigma')/(\sigma'_o)$
TP1	0.366	1.54	1.2	17	1617	1634	97.118	0.3436
TP2	0.348	1.48	1.5	18	1618	1636	91.889	0.4132
TP3	0.395	1.37	0.7	8.68	1608.68	1617.36	187.332	0.2651
TP4-1	0.439	1.47	0.7	8.75	1608.75	1617.5	185.857	0.2823
TP4-2	0.4	1.48	1.5	18.75	1618.75	1637.5	88.333	0.4708
TP5-1	0.459	1.51	0.7	8.47	1608.47	1616.94	191.902	0.2923
TP5-2	0.375	1.82	1.5	18.15	1618.15	1636.3	91.154	0.3909

Table4.11 Settlement Calculation using coefficient of compressibility, m_v

Test Pit	H_o(m)	m_v	$\Delta\sigma'$(kPa)	$Sc(m)=H_o m_v \Delta\sigma'$
TP1	1.2	0.0000624	1634	0.122
TP2	1.5	0.0000526	1636	0.129
TP3	0.7	0.0000257	1617.36	0.029
TP4-1	0.7	0.0000246	1617.5	0.028
TP4-2	1.5	0.0000633	1637.5	0.155
TP5-1	0.7	0.0000332	1616.94	0.038
TP5-2	1.5	0.0001753	1636.3	0.430

Table 4.10 and Table 4.11 clearly depict that as the depth increases the amount of settlement increases. This by far shows that, as the depth increases degree of laterization decreases because weathering action also decreases.

4.4 Effect of Remolding on consolidation behavior of Laterite soils

The soil is remolded on its natural moisture content using a standard compaction test. The soil is compacted in three layers with 25 numbers of blows for each layer. Then, a representative sample has been taken using a consolidation ring during extrusion of the sample from compaction mold. After determining the initial moisture content and specific gravity of remolded sample and following the usual procedure, the consolidation test was done.

The consolidation curve (Figure 4.14) of the test result of the remolded samples (Samples from TP4-2 and TP5-1) are very similar to the test result of the undisturbed samples. In TP4-2 the void ratio of the remolded sample is somewhat higher than the undisturbed one, whereas in TP5-1 the void ratio of the remolded sample is less than the undisturbed specimen and implies in TP4-2 the specimen after remolding loses its soil fabric, becomes more compressible.

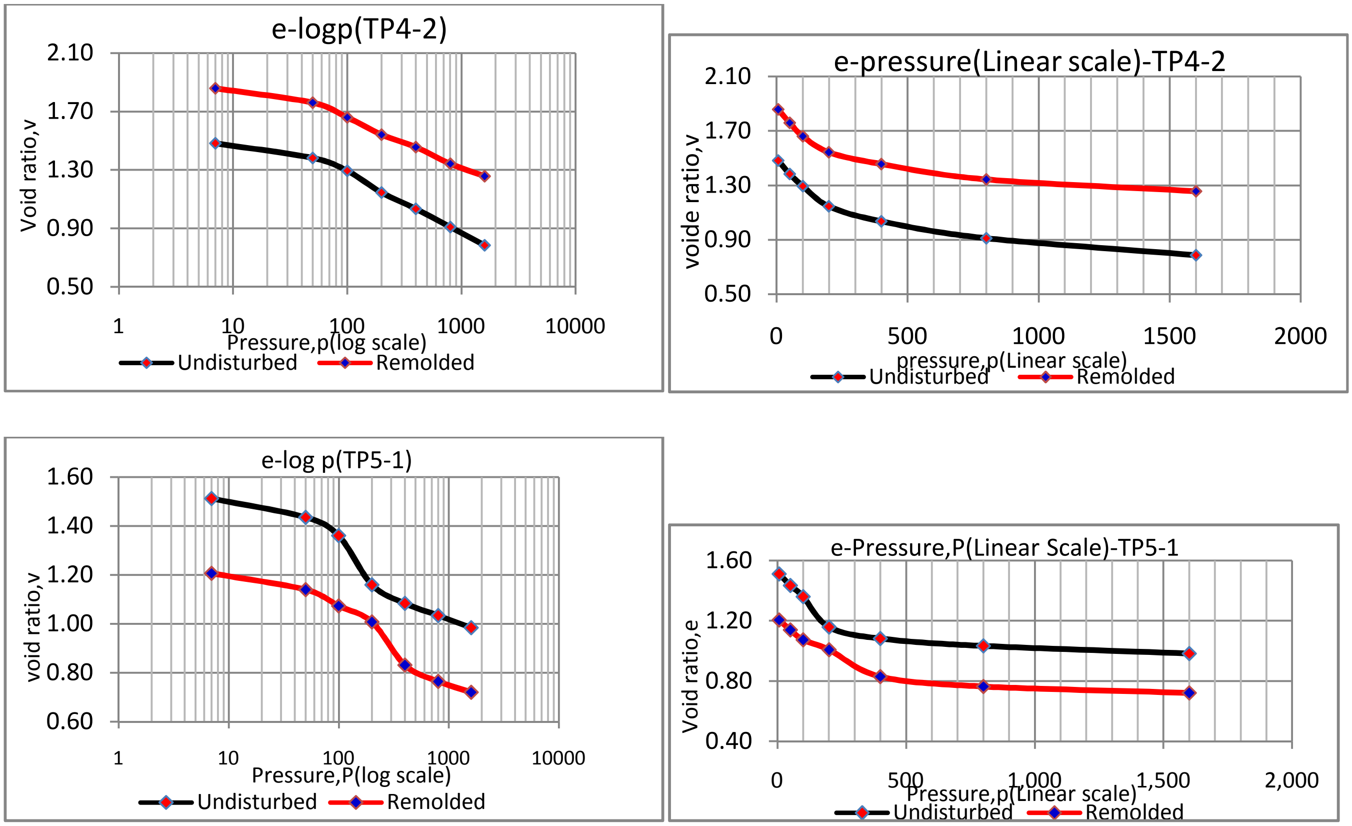


Figure 4.14 Consolidation curve for Undisturbed and Remolded samples in a) log scale and b) linear scale

4.5 Comparison of Consolidation and Settlement behavior of Laterite Soils of Western Ethiopia with Sedimentary and Other Laterite Soils.

As stated earlier from the stands of different Authors, the most significant differences in behavior between residual and sedimentary soils are probably those associated with their consolidation characteristics. The conventional understanding or interpretation of the consolidation behavior of sedimentary soils is based on their formation mode, namely deposition in sea or lake, followed by compression due to self-weight. Which means the consolidation behavior of sedimentary soils is highly influenced by the stress history. Whereas in residual laterite soils stress history has not significant value since they are formed by the in-situ weathering of rocks, thus, their formation process is the only determining factor in consolidation behavior or in any of the properties of laterite soils. In sedimentary soils till the applied stress passes the previous maximum stress the consolidation curve is almost constant and when it passes the soil compresses continuously.

When a soil is stressed to a level greater than the maximum stress to which it was ever subjected in the past, perhaps some kind of a breakdown may occur in a soil structure, resulting in a much higher compressibility.

According to this research test result, the consolidation curve of laterite soils observed is very similar to sedimentary soils regardless of their formation process. However, in the case of sedimentary soils until the stress passes the past maximum stress the consolidation curve is almost constant and when the applied stress passes the maximum stress the soils compresses continuously and preconsolidation pressure is clearly observed on the initial phase of consolidation curve. But the preconsolidation pressure observed at the initial phase of the consolidation curve for laterite soils is due to load bearing capacity of the strong cementation bond developed through time by means of Iron and/or Aluminium Sesquioxide not by the past maximum effective stress as stated in section 4.2.2, and when the applied load increases it breaks this bond and the soil becomes continuously compressible. Therefore, though the consolidation curve of laterite soils of the study site is similar to the very common consolidation curve of transported soils, this similarity couldn’t come from the similarity in their behavior.

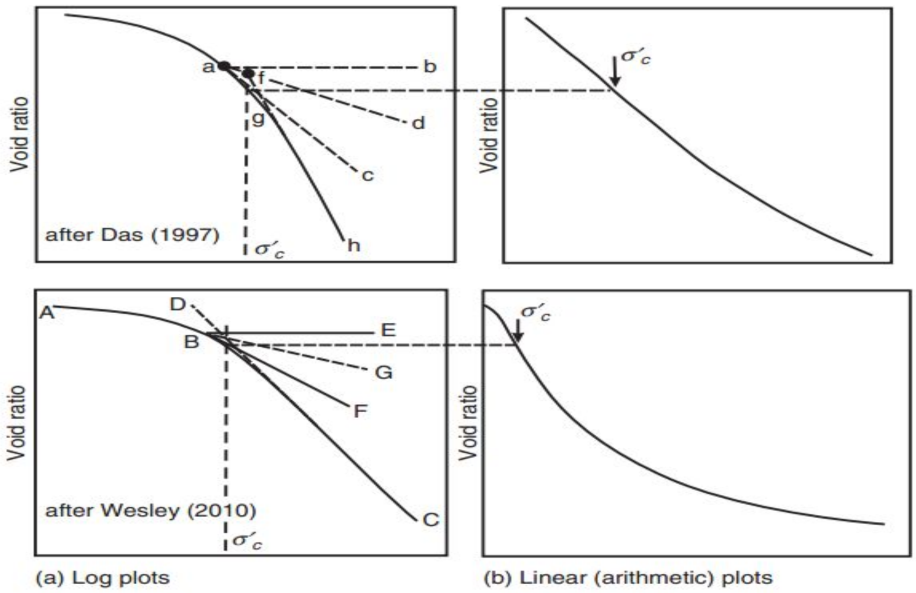


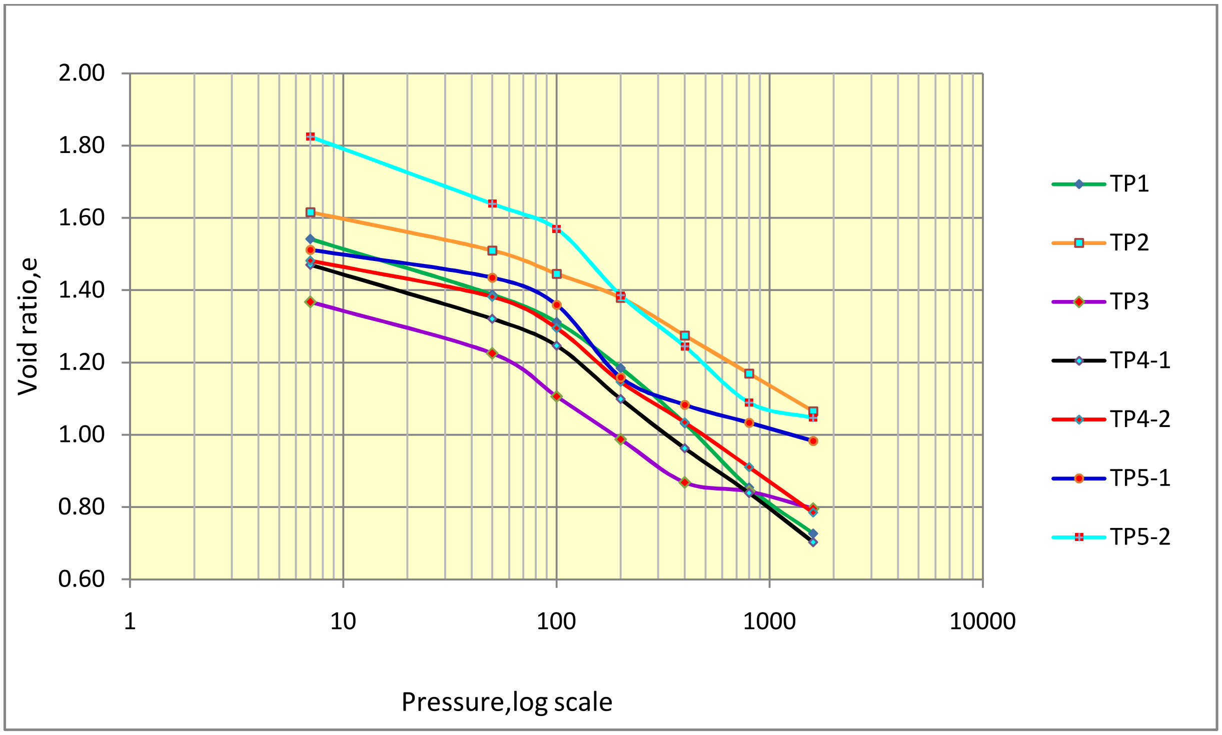
Figure 4.15 Oedometer test results plotted using both logarithmic (log) and linear scales of pressure for sedimentary soils (Wesley, 2010)

From figure 4.15, Wesley (2010), and figure 4.16, one can understand that:

1. The shape of the curves on the log plots and linear plots suggests that the soil shows an initial zone of low compressibility followed by a steady transition to a zone of higher compressibility. This low compressibility is due to the stress history for the case of transported soils, and due to cementation bond developed through weathering action for laterite soils.

2. This behavior in turn suggests that there is a preconsolidation pressure separating these zones in both soil types.

Generally the consolidation behavior of laterite soils at intermediate and high stress levels is very similar to transported soils, but at the initial phase the laterite soils behave as if overconsolidated due to their cementation bond whereas the transported soils shows overconsolidation due to the past stress history.



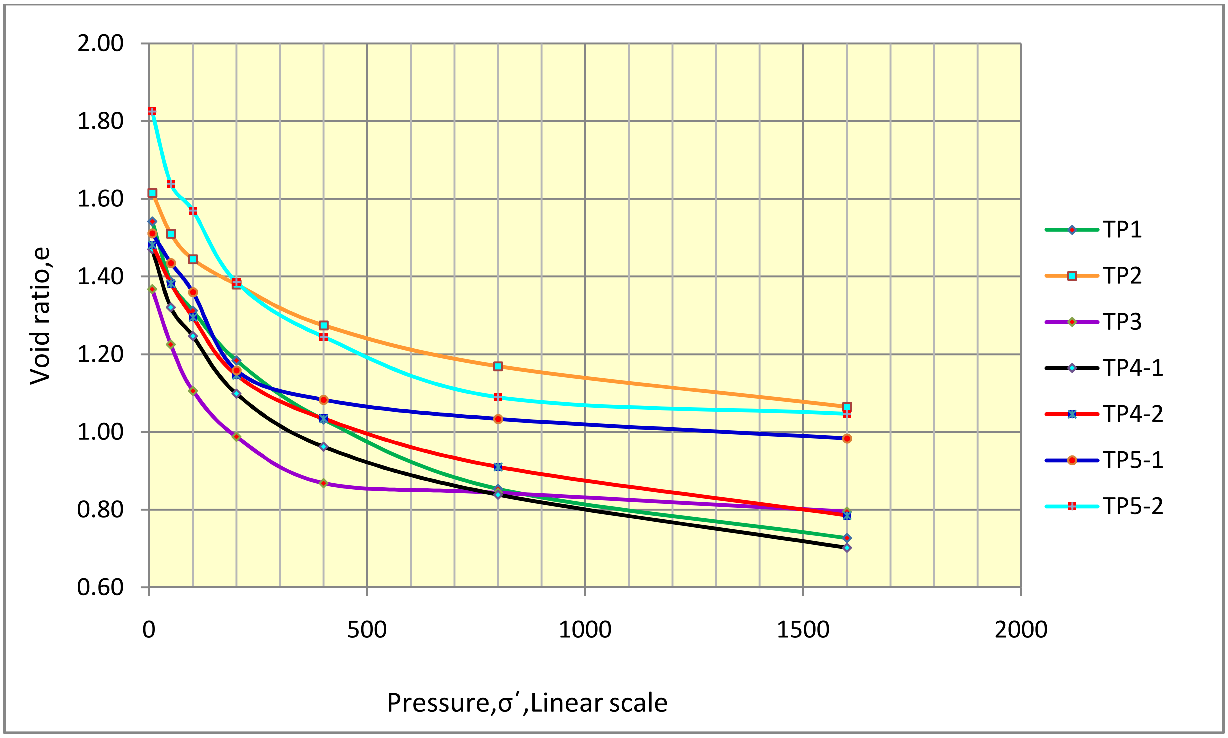


Figure 4.16 shape of consolidation curve in logarithmic scale for all test samples and shape of consolidation curve in linear scale for all tests

5. Conclusions and Recommendations

5.1 Conclusions

- For the research area mixing time for Atterberg limit tests should be limited to 5 minutes and fresh samples should be used for each specific test.
- Due to their unusual behavior laterite soils fall below the A-line, though they have high plasticity index, and this indicates that they have good engineering properties regardless of plasticity nature, but further investigation regarding to permeability, consolidation and shear strength properties should be done.
- Specific gravity of the laterite soil for this site varies from 2.8-3.1
- Consolidation test result of laterite soils at intermediate and high stress levels is very similar to transported soils, but at the initial phase the laterite soils behave as if overconsolidated due to their cementation bond whereas the transported soils shows overconsolidation state due to the past stress history.
- The consolidation test result curve of remolded soil samples behave similar to undisturbed samples test result shape, but the curve falls above the undisturbed one as remolding the soil sample breaks the soil fabrics and this by far makes the soil more compressible.
- Settlement behavior of laterite soils increase in line with depth since weathering action decreases apparently with depth and the cementation bond decrease as well.

5.2 Recommendations

- During classification testing, it is better to use wet sieving method in order to prevent desiccation due to drying, grinding of soil grains during pulverization, possible to wash out coated particles on the surface especially of gravels.
- Pretest treatment of laterite soils for Atterberg limit test gives unpredictable result which doesn’t represent or doesn’t show the real properties of laterite soils, thus pretest manipulation should be avoid
- In this regarded using laterite soils as a foundation material is not recommended since all the results show high degree of compressibility and settlement.
- Degree of laterization/cementation decreases as the depth increases but a geochemical test shall be done specifically in line with the consolidation test.

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APPENDIX-A

TEST RESULT OF NATURAL MOISTURE CONTENT

Determination of Natural Moisture Content at 50˚C

TP1

Trial No	**1**	**2**
Container No	**1**	**B**
Mass of container, g	15	7.31
Mass of container + Wet soil, g	77.45	64.35
Mass of container + Dry soil, g	66.196	53.39
Mass of water, g	11.25	10.96
Mass of dry soil, g	51.20	46.08
Water content, %	21.98	23.78
Ave. moisture content,% =	22.88	

TP2

Trial No	**1**	**2**
Container No	**2**	**E**
Mass of container, g	14.65	7.31
Mass of container + Wet soil, g	81.29	78.25
Mass of container + Dry soil, g	66.872	63.23
Mass of water, g	14.42	15.02
Mass of dry soil, g	52.22	55.92
Water content, %	27.61	26.86
Ave. moisture content,% =	27.24	

TP3

Trial No	**1**	**2**
Container No	**7**	**A**
Mass of container, g	14.93	7.34
Mass of container + Wet soil, g	73.02	72.64
Mass of container + Dry soil, g	61.77	60.31
Mass of water, g	11.25	12.33
Mass of dry soil, g	46.84	52.97
Water content, %	24.02	23.28
Ave. moisture content,% =	23.65	

TP4-1

Trial No	**1**	**2**
Container No	**99**	**C**
Mass of container, g	14.14	7.39
Mass of container + Wet soil, g	75.64	72.52
Mass of container + Dry soil, g	64.153	60.23
Mass of water, g	11.49	12.29
Mass of dry soil, g	50.01	52.84
Water content, %	22.97	23.26
Ave. moisture content,% =	23.11	

Determination of Natural Moisture Content at 50˚C

TP4-2

Trial No	**1**	**2**
Container No	**102**	**D**
Mass of container, g	15.09	7.34
Mass of container + Wet soil, g	69.01	67.00
Mass of container + Dry soil, g	58.356	55.78
Mass of water, g	10.65	11.22
Mass of dry soil, g	43.27	48.44
Water content, %	24.62	23.16
Ave. moisture content,% =	23.89	

TP5-1

Trial No	**1**	**2**
Container No	**101**	**84**
Mass of container, g	14.97	7.26
Mass of container + Wet soil, g	77.59	74.88
Mass of container + Dry soil, g	66.273	62.50
Mass of water, g	11.317	12.38
Mass of dry soil, g	51.30	55.25
Water content, %	22.06	22.41
Ave. moisture content,% =	22.23	

TP5-2

Trial No	**1**	**2**
Container No	**9**	**F2**
Mass of container, g	15.02	7.356
Mass of container + Wet soil, g	86.53	76.33
Mass of container + Dry soil, g	75.385	66.33
Mass of water, g	11.15	10.00
Mass of dry soil, g	60.37	58.97
Water content, %	18.46	16.96
Ave. moisture content,% =	17.71	

Determination of Natural Moisture Content at 105˚C

TP1

Trial No	**1**	**2**
Container No	**3**	**51**
Mass of container, g	14.44	9.93
Mass of container + Wet soil, g	84.42	78.36
Mass of container + Dry soil, g	70.63	65.89
Mass of water, g	13.79	12.47
Mass of dry soil, g	56.19	55.96
Water content, %	24.54	22.28
Ave. moisture content,% =	23.41	

TP2

Trial No	**1**	**2**
Container No	**4**	**21**
Mass of container, g	14.97	7.33
Mass of container + Wet soil, g	82.08	74.66
Mass of container + Dry soil, g	66.29	60.12
Mass of water, g	15.79	14.54
Mass of dry soil, g	51.32	52.79
Water content, %	30.77	27.54
Ave. moisture content,% =	29.16	

TP3

Trial No	**1**	**2**
Container No	**10**	**48**
Mass of container, g	14.59	7.26
Mass of container + Wet soil, g	71.45	70.25
Mass of container + Dry soil, g	59.9	57.55
Mass of water, g	11.55	12.70
Mass of dry soil, g	45.31	50.29
Water content, %	25.49	25.26
Ave. moisture content,% =	25.37	

TP4-1

Trial No	**1**	**2**
Container No	**6**	**53**
Mass of container, g	14.44	7.32
Mass of container + Wet soil, g	78.81	76.45
Mass of container + Dry soil, g	66.28	61.88
Mass of water, g	12.53	14.57
Mass of dry soil, g	51.84	54.56
Water content, %	24.17	26.70
Ave. moisture content,% =	25.44	

Determination of Natural Moisture Content at 105˚C

TP4-2

Trial No	**1**	**2**
Container No	**44**	**X**
Mass of container, g	14.52	9.61
Mass of container + Wet soil, g	82.71	78.96
Mass of container + Dry soil, g	69.06	65.44
Mass of water, g	13.65	13.52
Mass of dry soil, g	54.54	55.84
Water content, %	25.03	24.21
Ave. moisture content,% =	24.62	

TP5-1

Trial No	**1**	**2**
Container No	**100**	**12**
Mass of container, g	14.97	7.37
Mass of container + Wet soil, g	78.39	75.74
Mass of container + Dry soil, g	66.39	62.02
Mass of water, g	12.00	13.72
Mass of dry soil, g	51.42	54.66
Water content, %	23.34	25.10
Ave. moisture content,% =	24.22	

TP5-2

Trial No	**1**	**2**
Container No	**20**	**O**
Mass of container, g	14.39	7.39
Mass of container + Wet soil, g	90.61	76.33
Mass of container + Dry soil, g	77.88	64.66
Mass of water, g	12.73	11.67
Mass of dry soil, g	63.49	57.28
Water content, %	20.05	20.38
Ave. moisture content,% =	20.21	

Moisture content Difference,%

Test Pit	50˚C	105˚C	Difference
TP1	22.88	23.41	0.53
TP2	27.24	29.16	1.92
TP3	23.65	25.37	1.72
TP4-1	23.11	25.44	2.33
TP4-2	23.89	24.62	0.73
TP5-1	22.23	24.22	1.99
TP5-2	17.71	20.21	2.50

All the values are less than 5%

APPENDIX-B

SOIL CLASSIFICATION TEST RESULT (Gradation and hydrometer test result)

Air Dried

TP1		TP2		TP3		TP4-1		TP4-2		TP5-1		TP5-2	
Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass
19.000	100.000	9.500	100.000	19.000	100.000	19.000	100.000	9.500	100.000	19.000	100.000	19.000	100.000
9.500	98.200	4.750	89.800	9.500	98.320	9.500	99.460	4.750	92.890	9.500	87.900	9.500	88.970
4.750	77.500	2.000	55.110	4.750	88.250	4.750	91.100	2.000	66.950	4.750	70.380	4.750	63.160
2.000	52.600	0.425	21.420	2.000	50.650	2.000	61.180	0.425	31.390	2.000	47.770	2.000	33.590
0.425	32.600	0.075	12.420	0.425	21.120	0.425	25.330	0.075	6.290	0.425	22.610	0.425	16.050
0.075	6.400	0.044	2.250	0.075	4.680	0.075	4.580	0.024	5.710	0.075	5.090	0.075	3.020
0.037	6.080	0.031	2.250	0.025	4.340	0.028	3.540	0.017	5.710	0.025	4.780	0.025	2.960
0.026	6.080	0.022	2.140	0.018	4.340	0.020	3.400	0.012	5.520	0.018	4.630	0.018	2.870
0.019	5.890	0.016	2.140	0.013	4.200	0.014	3.260	0.009	5.330	0.013	4.630	0.013	2.870
0.014	5.510	0.011	2.140	0.010	4.050	0.010	3.260	0.006	5.330	0.010	4.480	0.010	2.780
0.010	5.140	0.008	2.140	0.007	4.050	0.008	3.120	0.005	5.140	0.007	4.330	0.007	2.690
0.007	4.950	0.004	2.020	0.005	3.910	0.005	3.120	0.003	4.950	0.005	4.170	0.005	2.600
0.004	4.380	0.003	2.020	0.004	3.620	0.004	2.980	0.002	4.580	0.004	3.870	0.004	2.330
0.003	4.190	0.002	1.900	0.003	3.620	0.003	2.840	0.002	4.390	0.003	3.710	0.003	2.250
0.0020	4.000	0.002	1.780	0.002	3.480	0.002	2.700	0.0012	4.200	0.002	3.560	0.002	2.160
0.0014	3.810	0.001	1.670	0.0013	3.480	0.0014	2.560	0.0010	4.010	0.0013	3.410	0.0013	2.070
0.0012	3.630			0.0010	3.340	0.0011	2.420			0.0011	3.260	0.0011	1.980

Oven Dried

TP1		TP2		TP3		TP4-1		TP4-2		TP5-1		TP5-2	
Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass	Sieve Size(mm)	% Pass
19.000	100.000	9.500	100.000	19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000
9.500	83.890	4.750	95.510	9.500	98.910	9.500	98.700	9.500	99.600	9.500	89.560	9.500	88.920
4.750	61.150	2.000	65.170	4.750	89.800	4.750	87.850	4.750	92.720	4.750	70.420	4.750	68.560
2.000	41.200	0.425	26.120	2.000	54.470	2.000	59.570	2.000	66.400	2.000	44.670	2.000	42.810
0.425	23.570	0.075	5.280	0.425	19.140	0.425	27.340	0.425	31.170	0.425	20.660	0.425	24.250
0.075	4.780	0.026	4.720	0.075	3.380	0.075	4.980	0.075	6.070	0.075	3.610	0.075	8.980
0.026	4.550	0.018	4.720	0.026	3.030	0.028	4.220	0.026	6.000	0.026	3.570	0.026	3.320
0.019	4.410	0.013	4.560	0.018	2.930	0.020	4.070	0.018	5.820	0.018	3.460	0.019	3.220
0.013	4.410	0.010	4.400	0.013	2.930	0.014	3.920	0.013	5.640	0.013	3.350	0.014	2.790
0.010	3.990	0.007	4.230	0.010	2.820	0.011	3.170	0.010	5.280	0.010	3.250	0.011	2.470
0.007	3.700	0.005	4.230	0.007	2.820	0.008	2.570	0.007	4.920	0.007	2.920	0.008	2.150
0.005	3.560	0.004	3.910	0.005	2.720	0.006	2.120	0.005	4.730	0.005	2.820	0.006	1.830
0.004	3.140	0.003	3.910	0.004	2.510	0.004	1.670	0.004	4.010	0.004	2.600	0.004	1.510
0.003	3.000	0.002	3.750	0.003	2.410	0.003	1.530	0.003	3.830	0.003	2.490	0.003	1.410
0.002	2.850	0.0013	3.750	0.002	2.410	0.0022	1.530	0.002	3.650	0.002	2.390	0.0022	1.300
0.0014	2.850	0.0011	3.590	0.0013	2.310	0.0016	1.380	0.0014	3.650	0.0014	2.280	0.0015	1.300
0.0012	2.710			0.0011	2.200	0.0013	1.380	0.0012	3.470	0.0011	2.170	0.0013	1.190

As Received

TP1		TP2		TP3		TP4-1		TP4-2		TP5-1		TP5-2	
Sieve Size(mm)	**% Pass**	**Sieve Size(mm)**	**% Pass**	**Sieve Size(mm)**	**% Pass**	**Sieve Size(mm)**	**% Pass**	**Sieve Size(mm)**	**% Pass**	**Sieve Size(mm)**	**% Pass**	**Sieve Size(mm)**	**% Pass**
19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000	19.000	100.000
9.500	79.770	9.500	93.230	9.500	98.000	9.500	97.470	9.500	97.580	9.500	90.380	9.500	87.210
4.750	66.280	4.750	83.680	4.750	86.830	4.750	88.110	4.750	89.390	4.750	77.690	4.750	66.090
2.000	57.230	2.000	60.420	2.000	56.660	2.000	71.550	2.000	73.380	2.000	64.610	2.000	48.840
0.425	52.460	0.425	44.790	0.425	37.660	0.425	56.340	0.425	59.040	0.425	50.960	0.425	40.310
0.075	45.720	0.075	34.720	0.075	32.560	0.075	43.470	0.075	49.730	0.075	42.110	0.075	34.400
0.041	22.270	0.026	31.100	0.027	29.890	0.026	42.710	0.028	38.680	0.028	32.470	0.029	20.300
0.030	19.510	0.018	31.100	0.020	26.920	0.018	41.430	0.021	32.770	0.021	28.890	0.021	17.780
0.022	15.830	0.013	30.160	0.014	24.940	0.013	40.140	0.016	26.870	0.015	25.310	0.016	15.270
0.017	11.220	0.010	29.230	0.011	22.960	0.010	37.570	0.012	22.440	0.014	21.730	0.012	12.750
0.012	10.300	0.007	28.290	0.008	20.980	0.007	36.280	0.009	18.010	0.008	19.340	0.009	11.070
0.009	8.460	0.005	28.290	0.006	19.990	0.005	34.990	0.006	16.530	0.006	18.150	0.006	10.230
0.005	7.080	0.004	26.420	0.004	18.010	0.004	31.140	0.005	12.110	0.004	15.760	0.004	8.560
0.003	0.003	0.003	26.420	0.003	17.020	0.003	30.490	0.003	12.110	0.003	15.760	0.003	7.720
0.0023	0.002	0.002	25.480	0.0021	16.530	0.002	28.560	0.0023	10.630	0.0022	14.560	0.0022	7.720
0.0016	0.002	0.0013	25.480	0.0015	16.030	0.0014	28.560	0.0016	10.630	0.0015	14.560	0.0016	7.720
0.0013	0.001	0.0011	24.540	0.0012	16.030	0.0011	28.560	0.0013	10.630	0.0012	14.560	0.0013	7.720

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APPENDIX-C

SPECIFIC GRAVITY TEST RESULT

Specific Gravity at 50C°

TP1,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	166	164.8
Temperature, T_x (^{o}c)	23.0	23.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.1	147.8
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.999	0.9993
Specific gravity of soil at 20°c.	3.084	3.123
Average specific gravity of soil.	3.1	

TP2,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.3	165
Temperature, T_x (^{o}c)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.3	149.2
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	2.777	2.716
Average specific gravity of soil.	2.7	

TP4-2,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	164.6	165.7
Temperature, T_x (^{o}c)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x)	148.1	148.9
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	1	0.9996
Specific gravity of soil at 20°c.	2.940	3.048
Average specific gravity of soil.	3.0	

TP5-1,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	164.6	165.5
Temperature, T_x (^{o}c)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x)	148.1	149.2
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	2.940	2.872
Average specific gravity of soil.	2.9	

TP3,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.3	164.2
Temperature, T_x (^oc)	23.0	23.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	148.9	147.9
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.905	2.872
Average specific gravity of soil.	2.9	

TP4-1,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.5	165.5
Temperature, T_x (^oc)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.1	149.2
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	2.906	2.872
Average specific gravity of soil.	2.9	

TP5-2,50C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.9	166.3
Temperature, T_x (^oc)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.1	149.2
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	3.048	3.163
Average specific gravity of soil.	3.1	

Specific Gravity at 105C°

TP1,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	164.6	164.4
Temperature, T_x (^{o}c)	21.0	21.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	148	147.9
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9991	0.9992
Specific gravity of soil at 20°c.	2.974	2.939
Average specific gravity of soil.	2.956	

TP2,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	164.3	165.6
Temperature, T_x (^{o}c)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	147.9	149.2
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	2.906	2.906
Average specific gravity of soil.	2.9	

TP3,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.5	165.4
Temperature, T_x (^{o}c)	23.0	23.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.2	149.1
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9993	0.9993
Specific gravity of soil at 20°c.	2.872	2.872
Average specific gravity of soil.	2.9	

TP4-1,110C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.7	165
Temperature, T_x (^{o}c)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.3	148.1
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	2.906	3.085
Average specific gravity of soil.	3.0	

TP4-2,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.9	166
Temperature, T_x (^oc)	21.0	21.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149.2	149.1
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9991	0.9992
Specific gravity of soil at 20°c.	3.009	3.084
Average specific gravity of soil.	3.0	

TP5-1,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	165.7	166
Temperature, T_x (^oc)	21.0	21.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	149	149.1
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9991	0.9992
Specific gravity of soil at 20°c.	3.009	3.084
Average specific gravity of soil.	3.0	

TP5-2,105C°

Determination No.	1	2
Pycnometer No.	P1	P2
Weight of pycnometer + soil + water, W_{pws} (g)	164.6	164.9
Temperature, T_x (^oc)	22.0	22.0
Weight of pycnometer + water at T_x , W_{pw}(at T_x) (g)	147.9	147.9
Weight of dry soil , w_s (gm)	25	25
Correction factor , K	0.9996	0.9996
Specific gravity of soil at 20°c.	3.011	3.124
Average specific gravity of soil.	3.1	

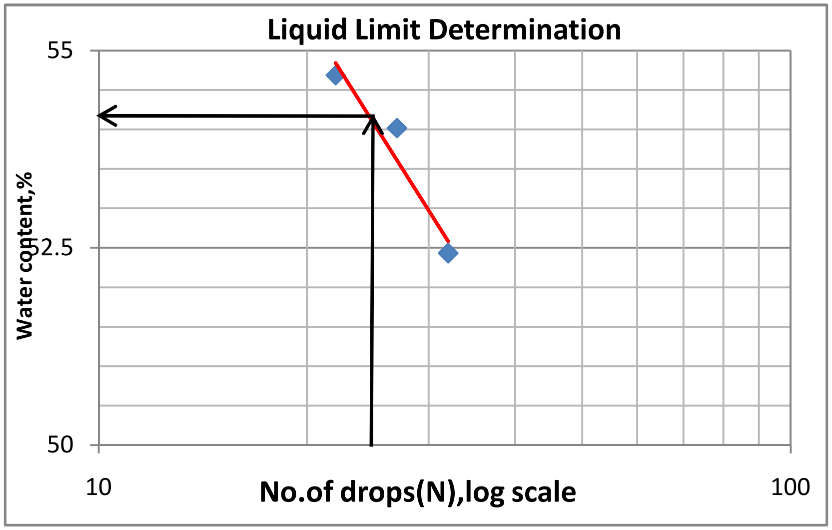
APPENDIX-D

ATTERBERG LIMIT TEST RESULT

Atterberg Limit at 50C° (Mixing Time =5 minutes)

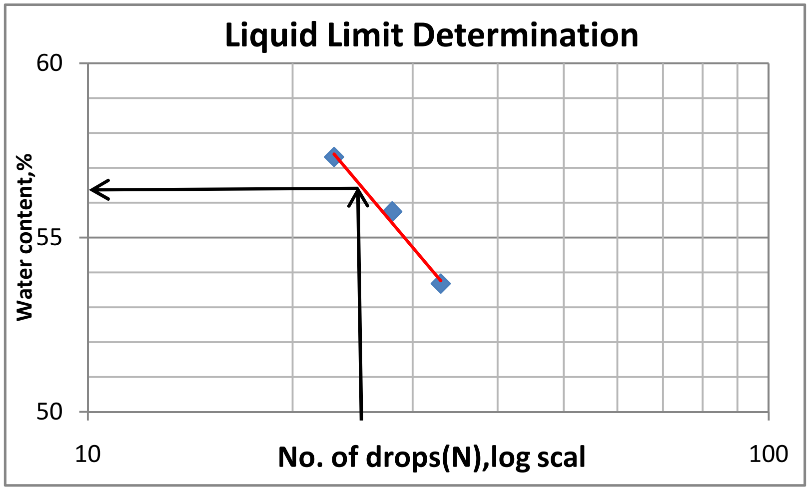
TP1

Trial No.	Liquid Limit			Plastic Limit	
	1	2	3	1	2
Container No.	102	10	1	11	101
Mass of container, g	15.087	14.984	15.004	15.159	14.971
Mass of container+Wet Soil,g	25.463	28.864	27.973	17.951	17.848
Mass of container+Dry Soil,g	21.894	23.996	23.388	17.283	17.15
Mass of water,g	3.569	4.868	4.585	0.668	0.698
Mass of dry soil ,g	6.807	9.012	8.384	2.124	2.179
Water content ,%	52.431	54.017	54.688	31.450	32.033
No. of drops(N)	32	27	22	31.742	



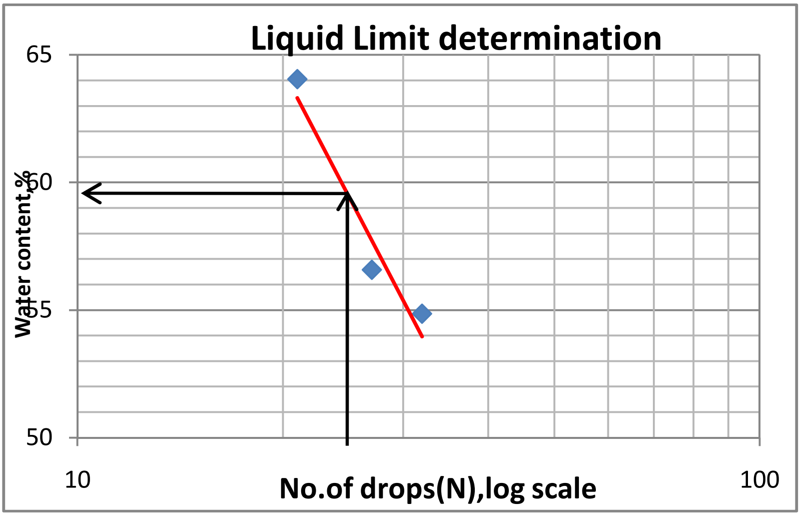
TP2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	B	10	F	51	21
Mass of container, g	7.301	7.307	7.317	9.931	7.331
Mass of container+Wet Soil,g	16.519	16.01	17.497	14.103	11.028
Mass of container+Dry Soil,g	13.299	12.895	13.788	13.095	10.136
Mass of water,g	3.22	3.115	3.709	1.008	0.892
Mass of dry soil ,g	5.998	5.588	6.471	3.164	2.805
Water content ,%	53.685	55.744	57.317	31.858	31.800
No. of drops(N)	33	28	23	31.829	



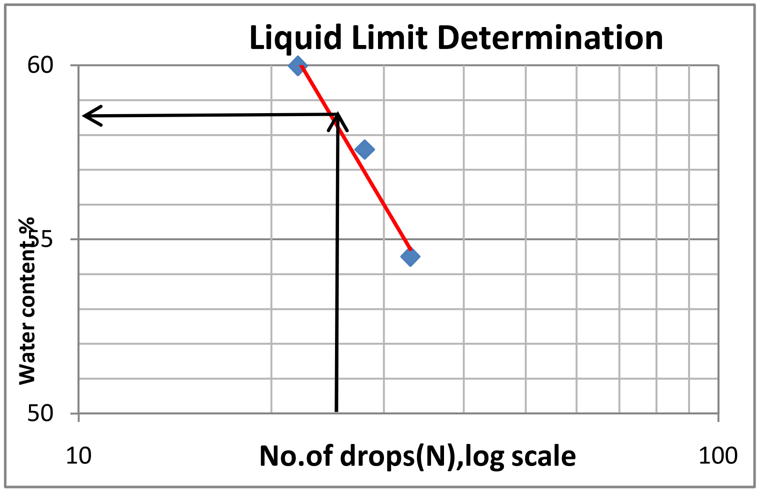
TP3

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	C	85	48	53
Mass of container, g	7.287	7.314	7.287	7.263	7.318
Mass of container+Wet Soil,g	17.937	17.592	17.657	11.151	11.222
Mass of container+Dry Soil,g	14.164	13.878	13.608	10.168	10.229
Mass of water,g	3.773	3.714	4.049	0.983	0.993
Mass of dry soil ,g	6.877	6.564	6.321	2.905	2.911
Water content ,%	54.864	56.581	64.056	33.838	34.112
No. of drops(N)	32	27	21	33.975	



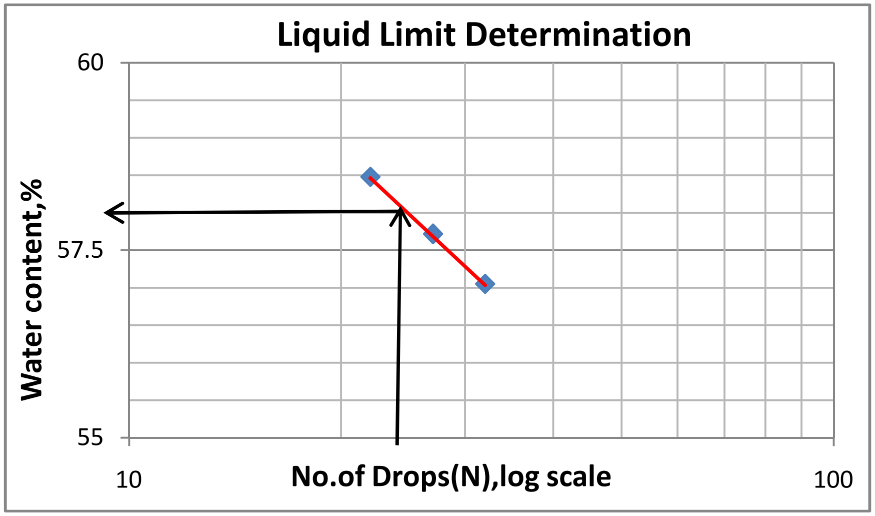
TP4-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	B	D	X	12
Mass of container,g	7.359	7.312	7.4	9.605	7.365
Mass of container+Wet Soil,g	21.629	21.003	20.971	14.671	12.079
Mass of container+Dry Soil,g	16.595	16	15.883	13.35	10.844
Mass of water,g	5.034	5.003	5.088	1.321	1.235
Mass of dry soil ,g	9.236	8.688	8.483	3.745	3.479
Water content ,%	54.504	57.585	59.979	35.274	35.499
No. of drops(N)	33	28	22	35.386	



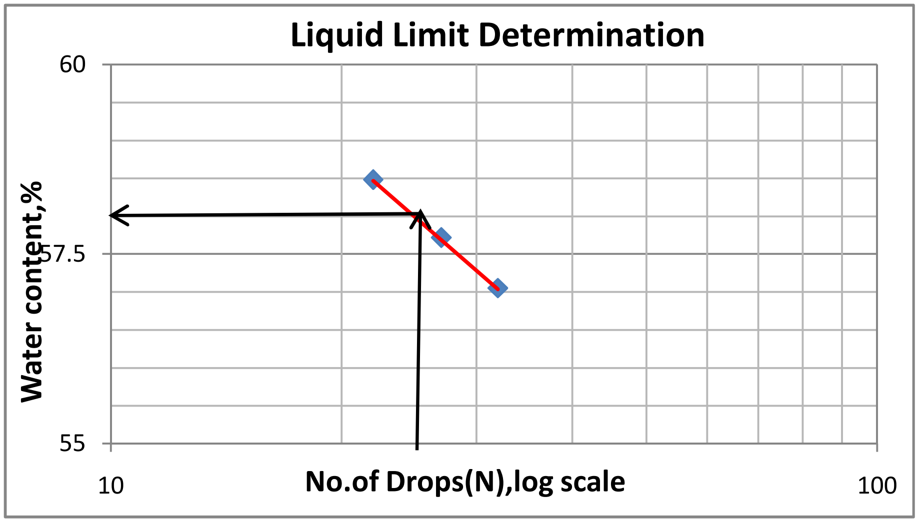
TP4-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	B	X	F2	12
Mass of container, g	7.338	7.304	9.603	7.356	7.367
Mass of container+Wet Soil,g	21.136	20.316	23.453	12.212	12.09
Mass of container+Dry Soil,g	16.247	15.614	18.35	11.005	10.907
Mass of water,g	4.889	4.702	5.103	1.207	1.183
Mass of dry soil ,g	8.909	8.31	8.747	3.649	3.54
Water content ,%	54.877	56.582	58.340	33.078	33.418
No. of drops(N)	31	26	22	33.248	



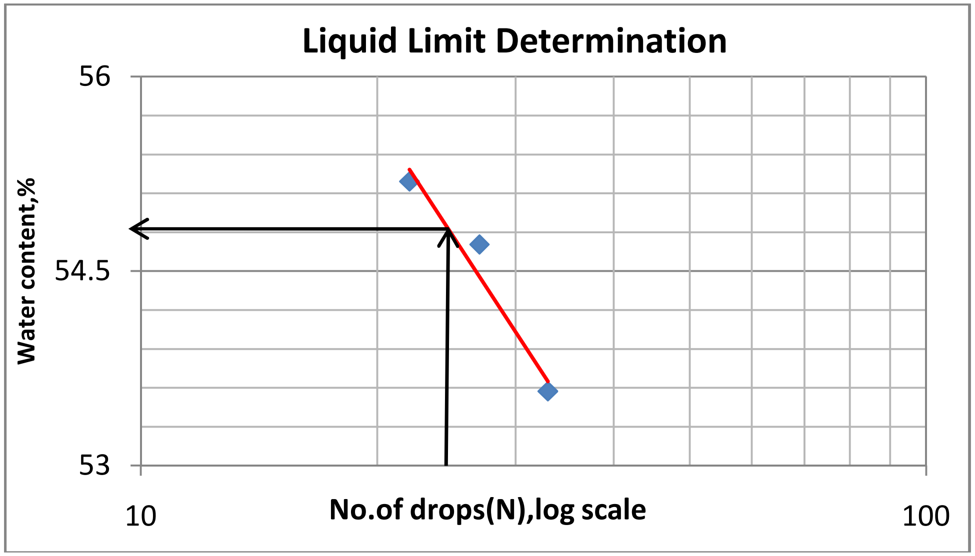
TP5-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	5	9	10	16	102
Mass of container, g	13.965	15.009	15	14.914	15.096
Mass of container+Wet Soil,g	25.268	28.773	33.165	20.308	19.473
Mass of container+Dry Soil,g	21.162	23.736	26.462	19.031	18.401
Mass of water,g	4.106	5.037	6.703	1.277	1.072
Mass of dry soil ,g	7.197	8.727	11.462	4.117	3.305
Water content ,%	57.052	57.717	58.480	31.018	32.436
No. of drops(N)	32	27	22	31.727	



TP5-2

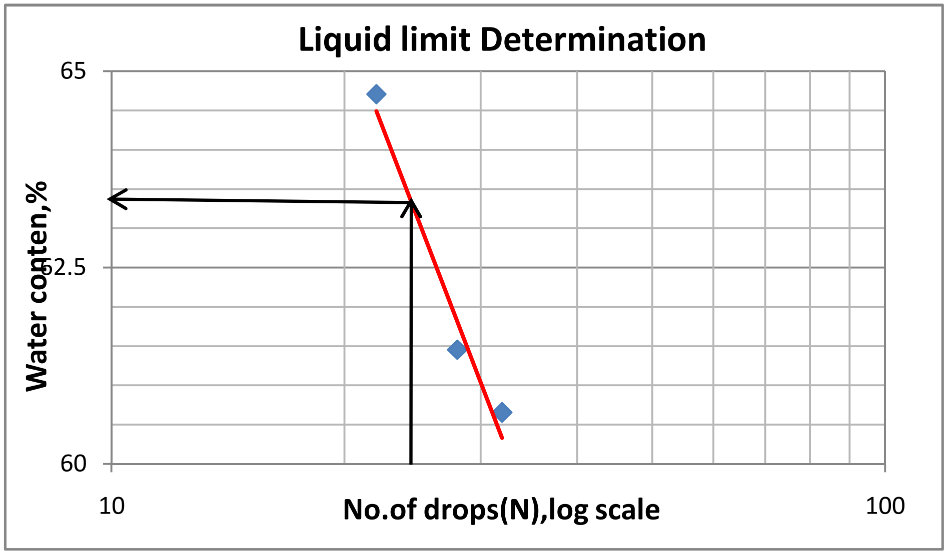
	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	E	zero 7	47	O	54
Mass of container, g	7.31	7.254	9.834	7.385	7.327
Mass of container+Wet Soil,g	18.398	16.541	20.955	11.473	11.313
Mass of container+Dry Soil,g	14.53	13.257	17	10.537	10.399
Mass of water,g	3.868	3.284	3.955	0.936	0.914
Mass of dry soil ,g	7.22	6.003	7.166	3.152	3.072
Water content ,%	53.573	54.706	55.191	29.695	29.753
No. of drops(N)	33	27	22	29.724	



Mixing time additional 25 Minutes(Total 30 Minute mixing time)

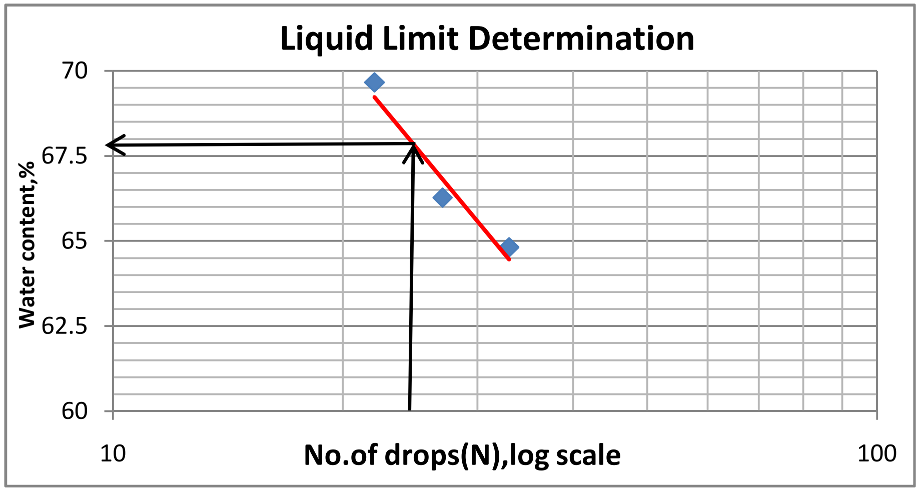
TP1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	16	9	99	84	F2
Mass of container, g	14.908	15.016	14.142	7.255	7.357
Mass of container+Wet Soil,g	26.528	28.47	29.786	11.422	11.276
Mass of container+Dry Soil,g	22.141	23.349	23.64	10.274	10.187
Mass of water,g	4.387	5.121	6.146	1.148	1.089
Mass of dry soil ,g	7.233	8.333	9.498	3.019	2.83
Water content ,%	60.653	61.454	64.708	38.026	38.481
No. of drops(N)	32	28	22	38.253	



TP4-1

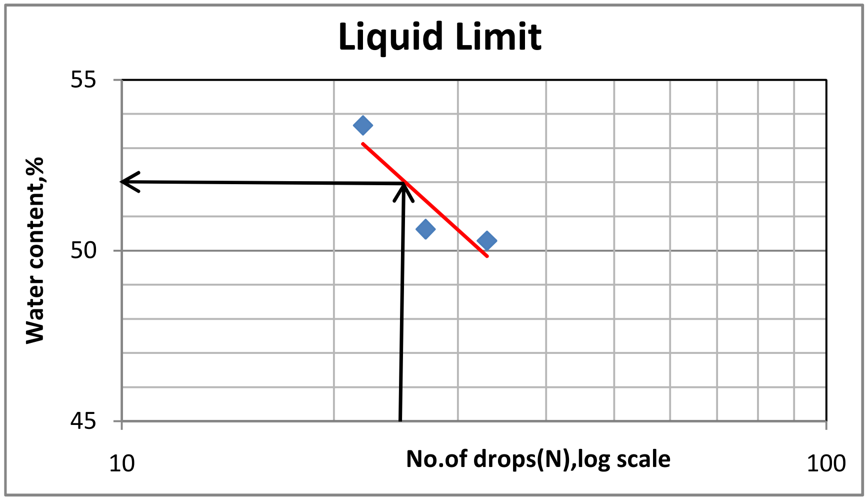
	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	7	45	8	2	5
Mass of container, g	14.937	15.089	15.155	14.65	13.965
Mass of container+Wet Soil,g	29.237	32.156	32.155	19.5	19.2
Mass of container+Dry Soil,g	23.613	25.353	25.175	18.049	17.64
Mass of water,g	5.624	6.803	6.98	1.451	1.56
Mass of dry soil ,g	8.676	10.264	10.02	3.399	3.675
Water content ,%	64.822	66.280	69.661	42.689	42.449
No. of drops(N)	33	27	22	42.569	



Atterberg Limit at 105C° (Mixing Time =5 minutes)

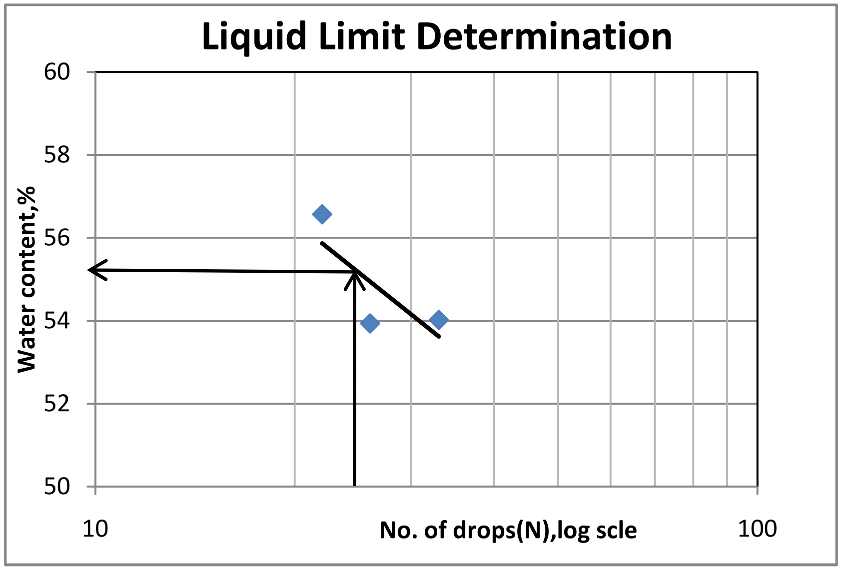
TP1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	51	F2	85	B	80
Mass of container, g	9.93	7.355	7.285	7.301	9.831
Mass of container+Wet Soil,g	21.66	19.736	15.729	12.557	14.461
Mass of container+Dry Soil,g	17.735	15.575	12.78	11.06	13.362
Mass of water,g	3.925	4.161	2.949	1.497	1.099
Mass of dry soil ,g	7.805	8.22	5.495	3.759	3.531
Water content ,%	50.288	50.620	53.667	39.824	31.124
No. of drops(N)	33	27	22	35.474	



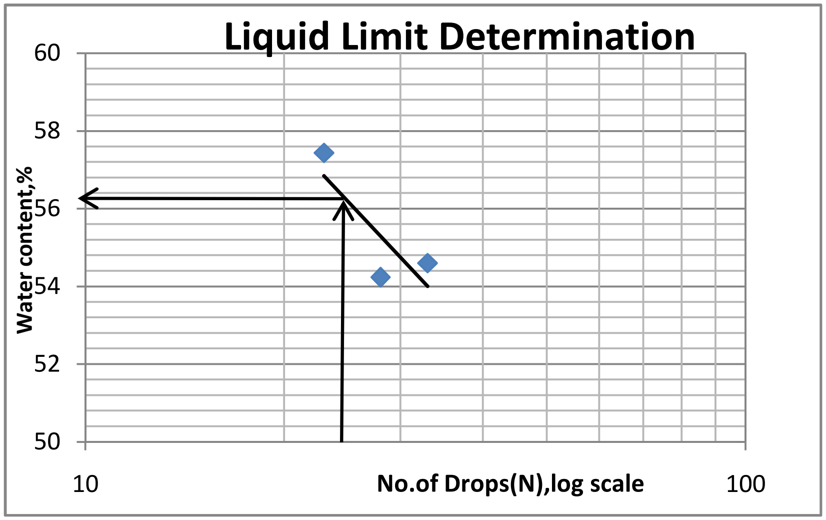
TP2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	B	F	21	53	F2
Mass of container, g	7.303	7.318	7.333	7.318	7.356
Mass of container+Wet Soil,g	16.187	14.75	16.763	10.077	10.001
Mass of container+Dry Soil,g	13.071	12.146	13.356	9.494	9.441
Mass of water,g	3.116	2.604	3.407	0.583	0.56
Mass of dry soil ,g	5.768	4.828	6.023	2.176	2.085
Water content ,%	54.022	53.935	56.566	26.792	26.859
No. of drops(N)	33	26	22	26.825	



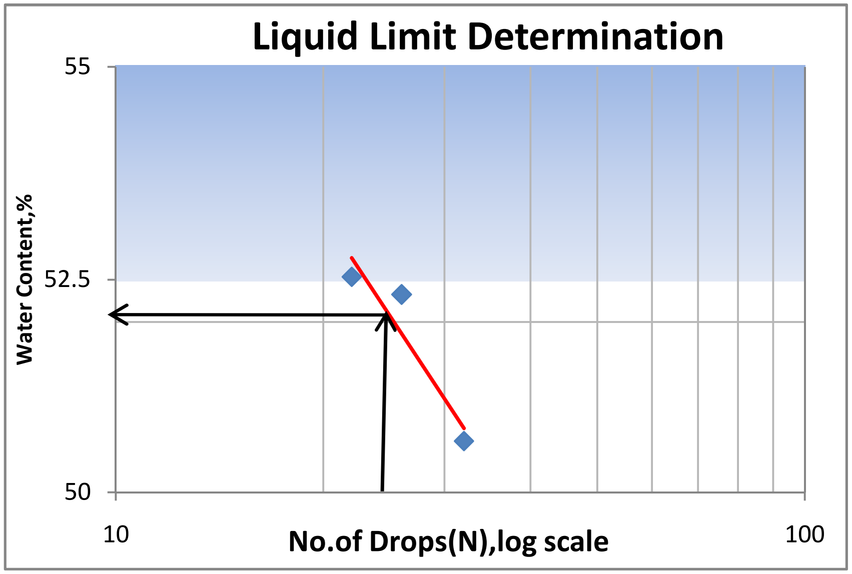
TP3

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	B	D	E	O
Mass of container, g	7.334	7.304	7.374	7.308	7.383
Mass of container+Wet Soil,g	24.068	25.93	25.937	11.735	11.497
Mass of container+Dry Soil,g	18.158	19.226	19.165	10.729	10.57
Mass of water,g	5.91	6.704	6.772	1.006	0.927
Mass of dry soil ,g	10.824	11.922	11.791	3.421	3.187
Water content ,%	54.601	56.232	57.434	29.407	29.087
No. of drops(N)	33	28	23	29.247	



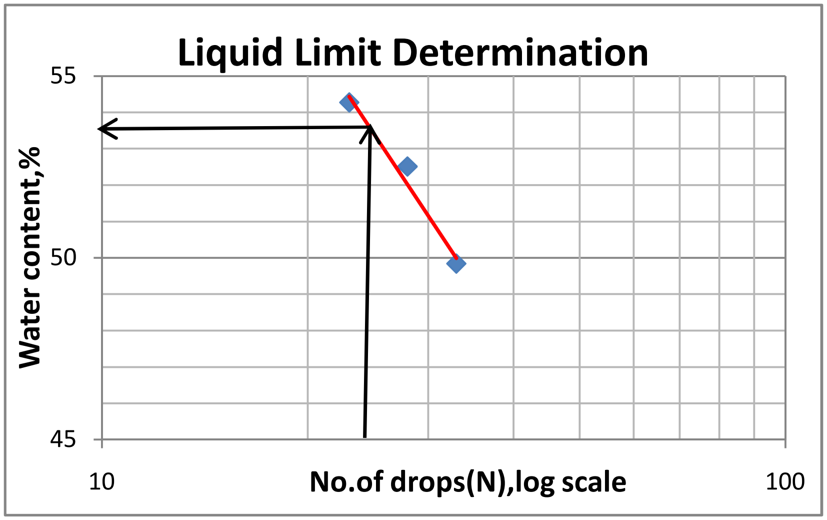
TP4-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	X	54	85	80	51
Mass of container, g	9.605	7.328	7.287	9.837	9.932
Mass of container+Wet Soil,g	16.721	12.804	15.15	13.076	12.984
Mass of container+Dry soil,g	14.33	10.923	12.442	12.357	12.313
Mass of water,g	2.391	1.881	2.708	0.719	0.671
Mass of dry soil ,g	4.725	3.595	5.155	2.52	2.381
Water content ,%	50.603	52.323	52.532	28.532	28.181
No. of drops(N)	32	26	22	28.357	



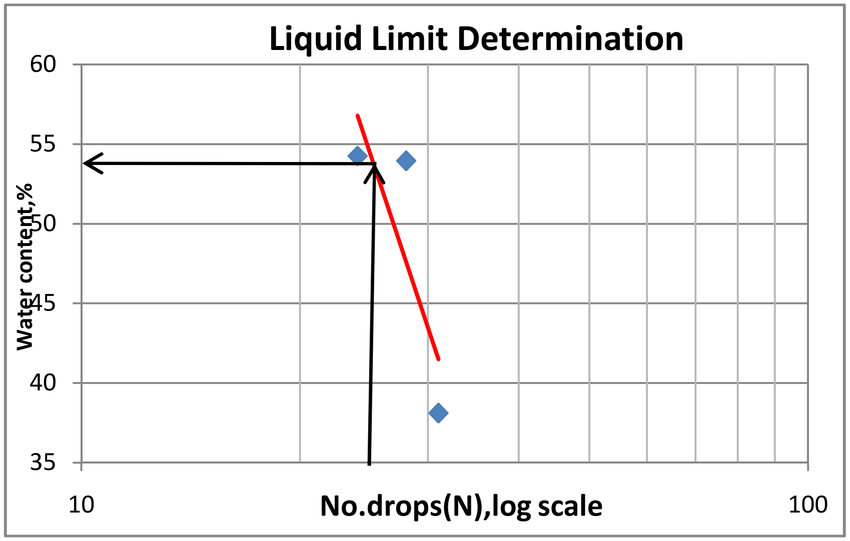
TP4-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	22	16	8	11	1
Mass of container,g	14.986	14.92	15.114	14.986	14.948
Mass of container+Wet Soil,g	25.027	25.076	24.934	18.08	16.125
Mass of container+Dry Soil,g	21.687	21.579	21.479	17.739	15.739
Mass of water,g	3.34	3.497	3.455	0.341	0.386
Mass of dry soil ,g	6.701	6.659	6.365	2.753	0.791
Water content ,%	49.843	52.515	54.281	12.386	48.799
No. of drops(N)	33	28	23	30.593	



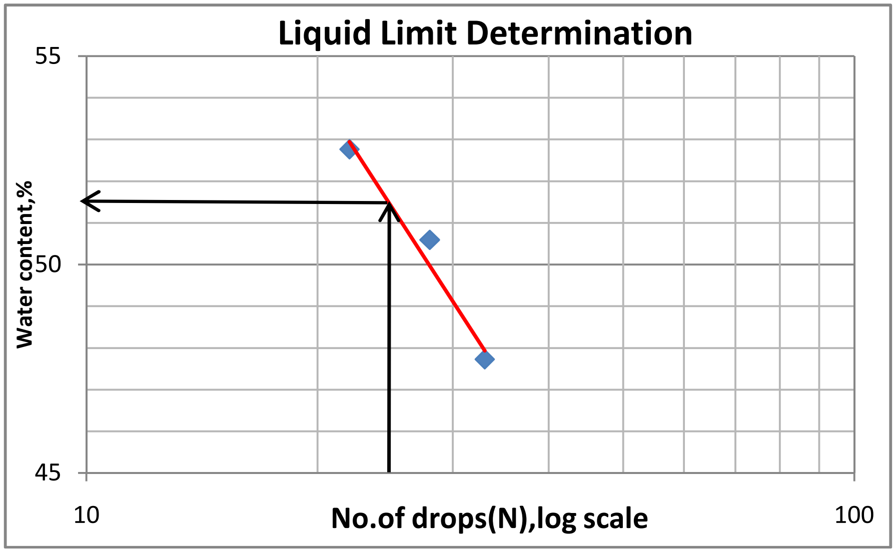
TP5-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A3	10	19	45	3
Mass of container,g	12.632	15.169	15.015	15.098	23.464
Mass of container+Wet Soil,g	26.19	23.051	28.016	18.438	26.682
Mass of container+Dry Soil,g	22.449	20.289	23.443	17.675	25.974
Mass of water,g	3.741	2.762	4.573	0.763	0.708
Mass of dry soil ,g	9.817	5.12	8.428	2.577	2.51
Water content ,%	38.107	53.945	54.260	29.608	28.207
No. of drops(N)	31	28	24	28.908	



TP5-2

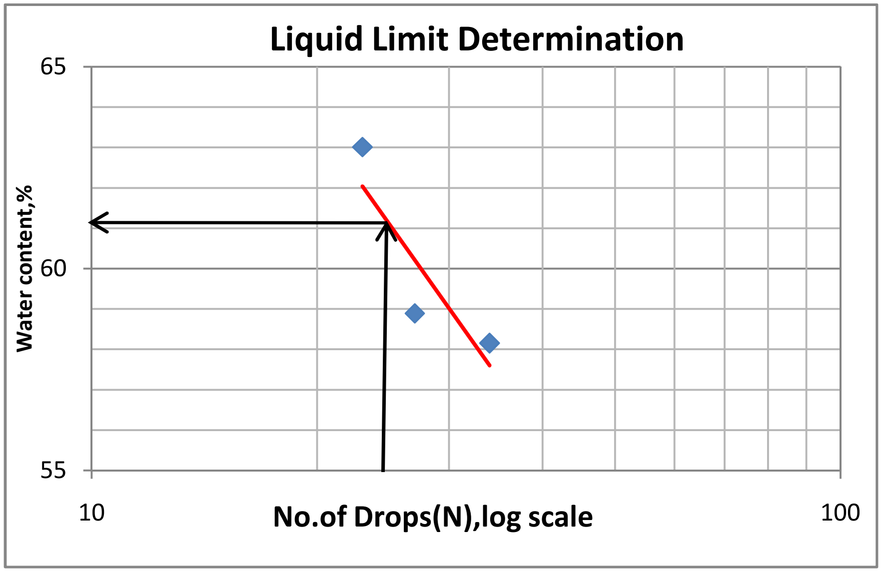
	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	21	53	84	F	X
Mass of container,g	7.33	7.317	7.254	7.314	9.604
Mass of container+Wet Soil,g	15.514	17.863	17.905	11.477	13.792
Mass of container+Dry Soil,g	12.87	14.32	14.226	10.583	12.88
Mass of water,g	2.644	3.543	3.679	0.894	0.912
Mass of dry soil ,g	5.54	7.003	6.972	3.269	3.276
Water content ,%	47.726	50.593	52.768	27.348	27.839
No. of drops(N)	33	28	22	27.593	



Mixing time additional 25 Minutes(Total 30 Minute mixing time)

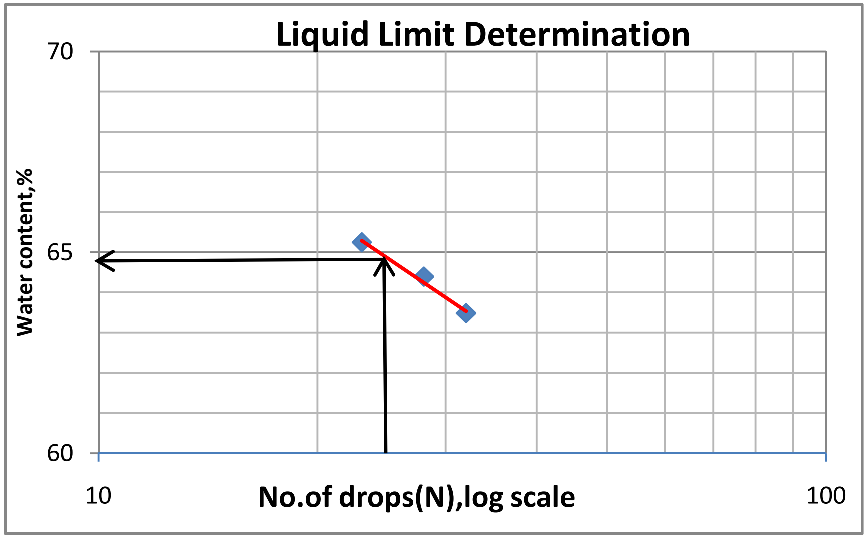
TP4-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	E	B	A	11	1
Mass of container,g	7.314	7.307	7.339	14.986	14.948
Mass of container+Wet Soil,g	16.841	14.176	15.985	18.08	16.125
Mass of container+Dry Soil,g	13.338	11.63	12.643	17.629	15.739
Mass of water,g	3.503	2.546	3.342	0.451	0.386
Mass of dry soil ,g	6.024	4.323	5.304	2.643	0.791
Water content ,%	58.151	58.894	63.009	17.064	48.799
No. of drops(N)	34	27	23	32.931	



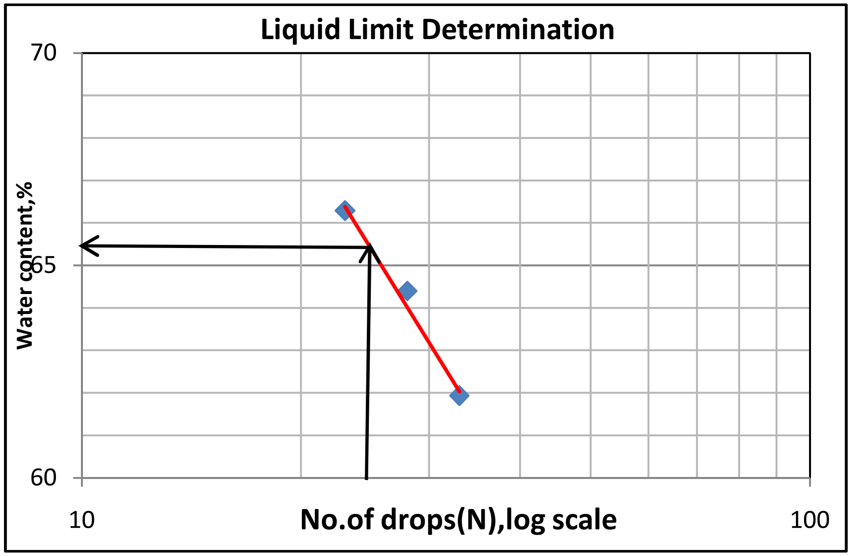
TP5-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	84	D	O	45	3
Mass of container,g	7.26	7.34	7.39	15.10	23.46
Mass of container+Wet Soil,g	14.67	14.70	15.33	18.44	26.68
Mass of container+Dry Soil,g	11.79	11.82	12.19	17.64	25.90
Mass of water,g	2.88	2.89	3.14	0.80	0.78
Mass of dry soil ,g	4.54	4.48	4.81	2.54	2.44
Water content ,%	63.49	64.40	65.25	31.44	31.89
No. of drops(N)	32.00	28.00	23.00	31.66	



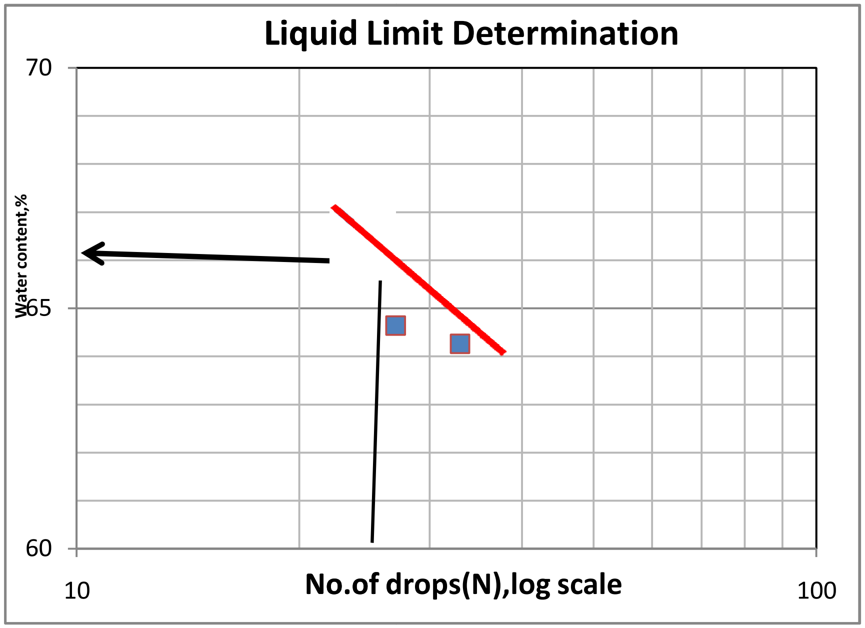
TP1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	B	X	12	F2
Mass of container,g	7.338	7.305	9.603	7.363	7.355
Mass of container+Wet Soil,g	20.343	24.181	26.689	10.353	10.447
Mass of container+Dry Soil,g	15.369	17.57	19.878	9.466	9.533
Mass of water,g	4.974	6.611	6.811	0.887	0.914
Mass of dry soil ,g	8.031	10.265	10.275	2.103	2.178
Water content ,%	61.935	64.403	66.287	42.178	41.965
No. of drops(N)	33	28	23	42.071	



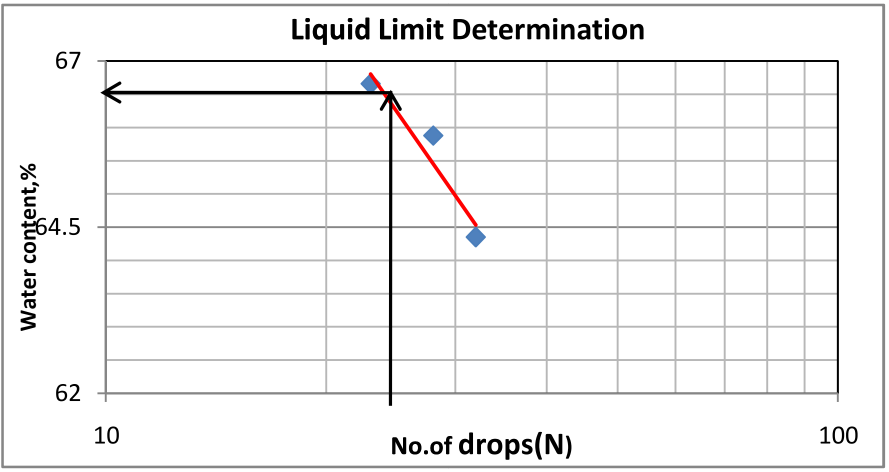
TP2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	47	21	10	O	84
Mass of container,g	9.83	7.328	7.304	7.388	7.253
Mass of container+Wet Soil,g	29.149	23.178	25.204	11.667	11.509
Mass of container+Dry Soil,g	21.591	16.955	18.031	10.474	10.328
Mass of water,g	7.558	6.223	7.173	1.193	1.181
Mass of dry soil ,g	11.761	9.627	10.727	3.086	3.075
Water content ,%	64.263	64.641	66.869	38.658	38.407
No. of drops(N)	33	27	23	38.532	



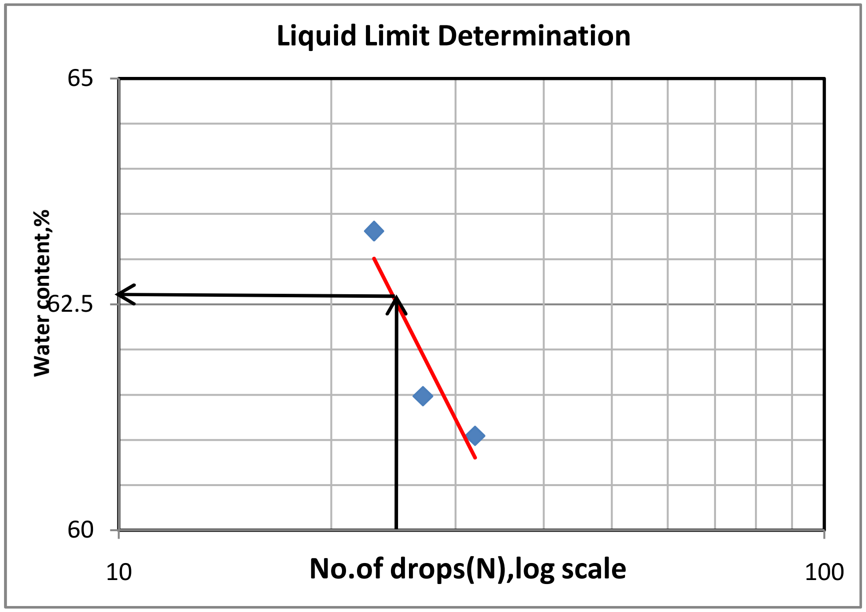
TP3

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A$^+$	E	F	O	12
Mass of container,g	7.286	7.312	7.315	7.386	7.366
Mass of container+Wet Soil,g	21.992	20.083	22.299	11.541	11.326
Mass of container+Dry Soil,g	16.234	15.011	16.306	10.205	10.232
Mass of water,g	5.758	5.072	5.993	1.336	1.094
Mass of dry soil ,g	8.948	7.699	8.991	2.819	2.866
Water content ,%	64.350	65.879	66.656	47.393	38.172
No. of drops(N)	32	28	23	42.782	



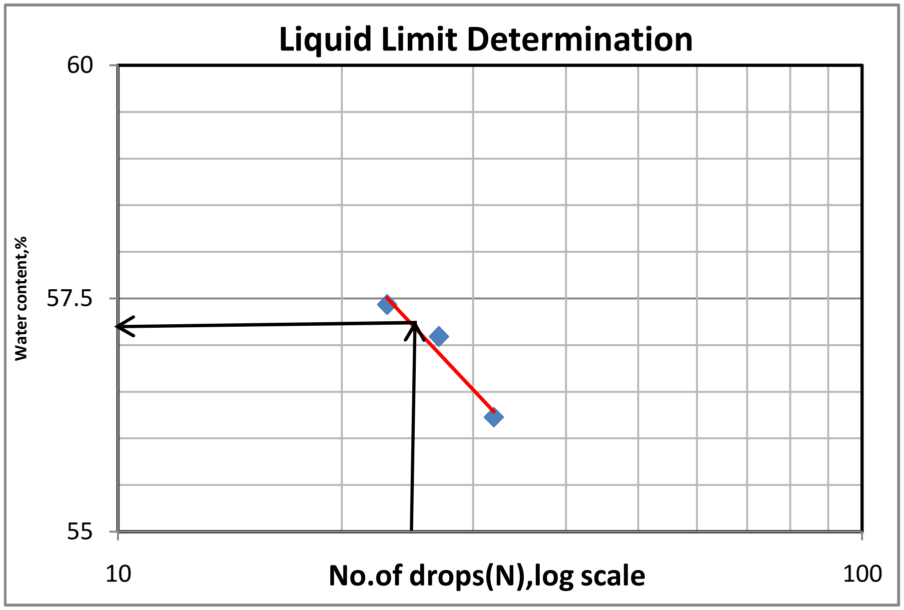
TP4-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	F2	21	80	85
Mass of container,g	7.333	7.352	7.327	9.83	7.285
Mass of container+Wet Soil,g	29.224	24.786	32.323	14.722	12.369
Mass of container+Dry Soil,g	21.345	18.45	23.204	13.632	11.248
Mass of water,g	7.879	6.336	9.119	1.09	1.121
Mass of dry soil ,g	14.012	11.098	15.877	3.802	3.963
Water content ,%	56.230	57.091	57.435	28.669	28.287
No. of drops(N)	32	27	23	28.478	



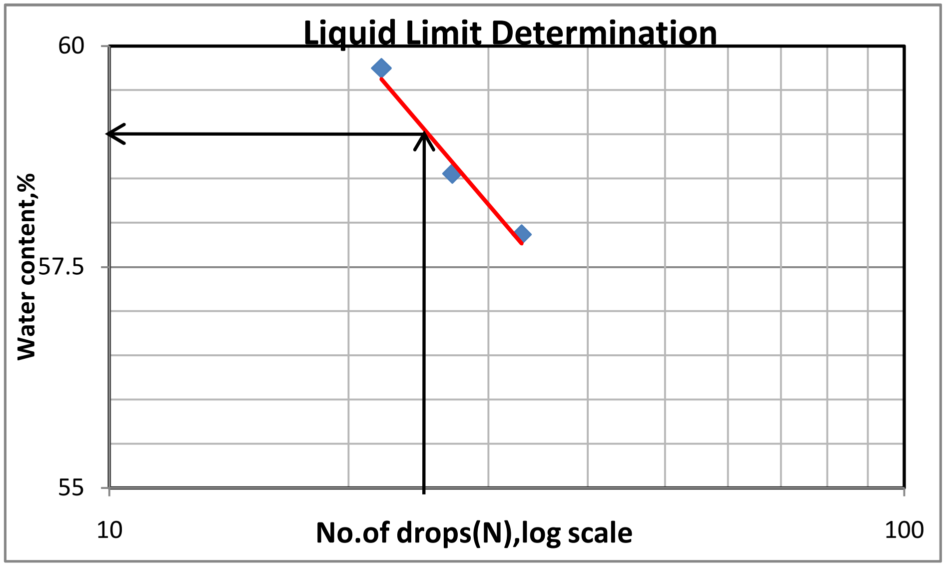
TP4-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	A	F2	21	80	85
Mass of container,g	7.333	7.352	7.327	9.83	7.285
Mass of container+Wet Soil,g	29.224	24.786	32.323	14.722	12.369
Mass of container+Dry Soil,g	21.345	18.45	23.204	13.632	11.248
Mass of water,g	7.879	6.336	9.119	1.09	1.121
Mass of dry soil ,g	14.012	11.098	15.877	3.802	3.963
Water content ,%	56.230	57.091	57.435	28.669	28.287
No. of drops(N)	32	27	23	28.478	



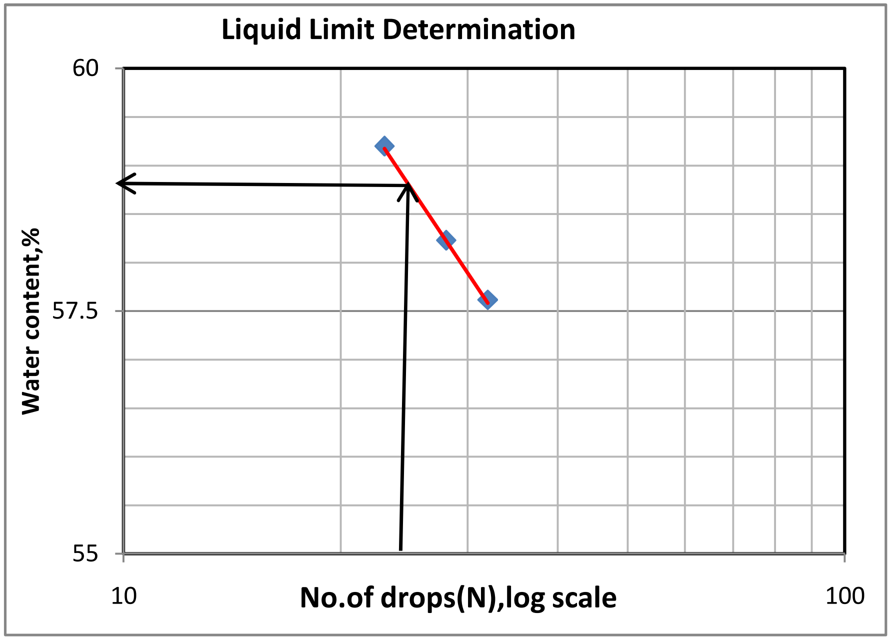
TP5-1

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	47	48	51	X	54
Mass of container,g	9.83	7.262	9.929	9.602	7.325
Mass of container+Wet Soil,g	27.68	26.062	29.947	13.8	11.748
Mass of container+Dry Soil,g	21.137	19.119	22.46	12.8	10.702
Mass of water,g	6.543	6.943	7.487	1	1.046
Mass of dry soil ,g	11.307	11.857	12.531	3.198	3.377
Water content ,%	57.867	58.556	59.748	31.270	30.974
No. of drops(N)	33	27	22	31.122	



TP5-2

	Liquid Limit			Plastic Limit	
Trial No.	1	2	3	1	2
Container No.	B	X	O	10	12
Mass of container,g	7.306	9.602	7.383	7.304	7.365
Mass of container+Wet Soil,g	27.016	30.553	30.161	11.835	12.019
Mass of container+Dry Soil,g	19.811	22.843	21.691	10.465	10.577
Mass of water,g	7.205	7.71	8.47	1.37	1.442
Mass of dry soil ,g	12.505	13.241	14.308	3.161	3.212
Water content ,%	57.617	58.228	59.198	43.341	44.894
No. of drops(N)	32	28	23	44.117	



APPENDIX-E

CONSOLIDATION TEST RESULT

Time-Settlement Data

TP1

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	1.21004	1.80609	2.80812	4.00820	5.40824	6.41000
15sec	1.21040	1.80619	2.80818	4.01020	5.41000	6.41001
30sec	1.21202	1.80623	2.80825	4.01028	5.41036	6.41001
1min	1.21204	1.80626	2.80829	4.01034	5.41202	6.41002
2min	1.21206	1.80628	2.80832	4.01038	5.41208	6.41004
4min	1.21208	1.80630	2.80834	4.01202	5.41212	6.41006
8min	1.21209	1.80633	2.80838	4.01205	5.41215	6.41007
15min	1.21210	1.80635	2.81000	4.01207	5.41217	6.41009
30min	1.21211	1.80636	2.81002	4.01209	5.41220	6.41012
1hr	1.21212	1.80637	2.81004	4.01211	5.41222	6.41015
2hr	1.21213	1.80638	2.81005	4.01212	5.41225	6.41018
4hr	1.21214	1.80639	2.81007	4.01214	5.41226	6.41021
8hr	1.21215	1.80639	2.81008	4.01216	5.41228	6.41023
16hr	1.21215	1.80640	2.81009	4.01217	5.41229	6.41026
24hr	1.21216	1.80640	2.81009	4.01218	5.41230	6.41030

TP2

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	0.80620	1.30400	1.80600	2.60624	3.41028	4.20820
15sec	0.80620	1.30404	1.80618	2.60632	3.41040	4.20832
30sec	0.80621	1.30407	1.80621	2.60800	3.41202	4.21000
1min	0.80622	1.30408	1.80623	2.60803	3.41207	4.21010
2min	0.80622	1.30409	1.80625	2.60806	3.41212	4.21017
4min	0.80622	1.30411	1.80627	2.60808	3.41214	4.21022
8min	0.80623	1.30412	1.80628	2.60810	3.41216	4.21026
15min	0.80624	1.30412	1.80629	2.60812	3.41218	4.21029
30min	0.80624	1.30413	1.80630	2.60813	3.41220	4.21031
1hr	0.80624	1.30414	1.80630	2.60814	3.41221	4.21034
2hr	0.80625	1.30415	1.80631	2.60815	3.41222	4.21036
4hr	0.80625	1.30415	1.80632	2.60816	3.41224	4.21037
8hr	0.80626	1.30416	1.80632	2.60816	3.41225	4.21039
16hr	0.80627	1.30416	1.80632	2.60817	3.41225	4.21039
24hr	0.80627	1.30416	1.80632	2.60818	3.41226	4.21040

TP3

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	1.202	2.20404	3.20608	4.21216	4.4221	4.82812
15sec	1.20204	2.204044	3.20612	4.2122	4.42214	4.82822
30sec	1.20206	2.204052	3.206148	4.21221	4.42216	4.82836
1min	1.20208	2.20406	3.2062	4.21225	4.42234	4.83002
2min	1.202092	2.204092	3.206284	4.21238	4.424	4.830156
4min	1.202112	2.204116	3.206312	4.214	4.42405	4.830244
8min	1.202128	2.204132	3.206336	4.21403	4.42408	4.8303
15min	1.202136	2.204148	3.206344	4.21405	4.4241	4.830344
30min	1.202148	2.204156	3.20636	4.21407	4.42412	4.830368
1hr	1.202152	2.204168	3.20637	4.21408	4.42414	4.830396
2hr	1.20216	2.204172	3.206378	4.21409	4.42415	4.832008
4hr	1.202168	2.20418	3.206388	4.2141	4.42416	4.832038
8hr	1.20217	2.20419	3.206392	4.21412	4.42417	4.832048
16hr	1.202178	2.204194	3.206396	4.21412	4.42418	4.832054
24hr	1.202184	2.204196	3.206398	4.21412	4.42418	4.83206

TP4-1

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	1.20832	1.80628	3.00832	4.1102	5.1082	6.21436
15sec	1.21	1.80636	3.01	4.11024	5.10829	6.214392
30sec	1.21002	1.808	3.01004	4.11029	5.10836	6.216076
1min	1.21005	1.80806	3.01008	4.11032	5.11	6.216144
2min	1.21008	1.80809	3.01011	4.11035	5.11005	6.216184
4min	1.21009	1.80811	3.01013	4.11038	5.11008	6.216224
8min	1.21012	1.80813	3.01015	4.112	5.11011	6.21626
15min	1.21014	1.80814	3.01017	4.11202	5.11013	6.216284
30min	1.21015	1.80815	3.01018	4.11204	5.11014	6.216304
1hr	1.21015	1.80818	3.01019	4.11205	5.11016	6.21632
2hr	1.21016	1.80819	3.01021	4.11206	5.11017	6.216332
4hr	1.21017	1.8082	3.01022	4.11207	5.11018	6.216344
8hr	1.21019	1.80821	3.01022	4.11208	5.11019	6.216352
16hr	1.2102	1.80822	3.01023	4.11209	5.11021	6.216356
24hr	1.2102	1.80822	3.01023	4.11209	5.11022	6.21636

TP4-2

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	0.80016	1.5023	2.70004	3.60016	4.60013	5.61028
15sec	0.800168	1.502328	2.700048	3.6002	4.60014	5.612
30sec	0.800188	1.502344	2.700056	3.60024	4.60022	5.612132
1min	0.800208	1.502398	2.700064	3.60025	4.60024	5.612196
2min	0.800216	1.504	2.700072	3.60026	4.60027	5.612248
4min	0.800228	1.504016	2.700084	3.60027	4.60028	5.612296
8min	0.800236	1.504032	2.700092	3.60028	4.6003	5.612328
15min	0.80024	1.504044	2.7001	3.60028	4.60032	5.612352
30min	0.800248	1.50407	2.700108	3.60029	4.60033	5.61238
1hr	0.800252	1.504084	2.700116	3.6003	4.60034	5.614
2hr	0.800256	1.504085	2.70012	3.6003	4.60035	5.614008
4hr	0.800264	1.5041	2.700124	3.60031	4.60036	5.614028
8hr	0.800268	1.504108	2.700136	3.60031	4.60038	5.61404
16hr	0.80027	1.504116	2.70014	3.60031	4.60038	5.61405
24hr	0.800272	1.504124	2.700148	3.60032	4.60038	5.61406

TP5-1

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	0.6061	1.206	2.80608	3.408	3.806	4.204
15sec	0.60616	1.20612	2.80616	3.408	3.8061	4.204004
30sec	0.60617	1.20617	2.8062	3.40801	3.80612	4.204008
1min	0.60619	1.20621	2.80623	3.40806	3.80614	4.204036
2min	0.6062	1.20622	2.80626	3.4081	3.80615	4.20406
4min	0.60622	1.20624	2.80628	3.40814	3.80616	4.204084
8min	0.60623	1.20625	2.80629	3.40816	3.80617	4.204108
15min	0.60624	1.20626	2.80631	3.40818	3.80618	4.204128
30min	0.60626	1.20626	2.80632	3.40819	3.80618	4.204144
1hr	0.60627	1.20627	2.80633	3.4082	3.80619	4.204156
2hr	0.60628	1.20628	2.80635	3.40821	3.8062	4.204168
4hr	0.6063	1.20628	2.80636	3.40823	3.8062	4.204184
8hr	0.60632	1.20629	2.80636	3.40824	3.80621	4.204192
16hr	0.60632	1.2063	2.80637	3.40826	3.80621	4.204198
24hr	0.60633	1.2063	2.80637	3.40828	3.80621	4.204204

TP5-2

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	1.31404	1.806	3.108	4.108	5.20618	5.50408
15sec	1.31406	1.80602	3.10806	4.10808	5.20629	5.50416
30sec	1.314086	1.806056	3.108096	4.10812	5.20636	5.504264
1min	1.314104	1.806092	3.108128	4.10821	5.20804	5.504308
2min	1.31412	1.80612	3.108156	4.10825	5.20809	5.504348
4min	1.314136	1.806144	3.108174	4.10828	5.20813	5.506
8min	1.314148	1.806164	3.10819	4.1083	5.20817	5.506036
15min	1.314158	1.806176	3.108204	4.10832	5.20819	5.506064
30min	1.314168	1.806188	3.108212	4.10834	5.20821	5.506088
1hr	1.314176	1.8062	3.10822	4.10836	5.20823	5.506112
2hr	1.314184	1.806208	3.108232	4.10837	5.20824	5.506134
4hr	1.314192	1.80622	3.10824	4.10838	5.20826	5.506152
8hr	1.3142	1.806238	3.108248	4.10839	5.20827	5.506172
16hr	1.314208	1.806242	3.108256	4.10839	5.20828	5.50618
24hr	1.314216	1.806244	3.10826	4.1084	5.20829	5.506188

TP4-2,Remolded Sample

Time	Dial Gauge Reading,mm					
	1kg	2kg	4kg	8kg	16kg	32kg
6sec	1.40202	1.60208	2.20416	2.80432	3.60238	4.00236
15sec	1.40204	1.6021	2.20423	2.80437	3.604	4.00402
30sec	1.40205	1.60212	2.20426	2.80602	3.60412	4.004088
1min	1.40207	1.60215	2.20429	2.80608	3.60421	4.004172
2min	1.40209	1.60217	2.20433	2.80615	3.60433	4.004288
4min	1.40211	1.60219	2.20436	2.80621	3.60604	4.006024
8min	1.40212	1.60221	2.20439	2.80625	3.60615	4.206134
15min	1.40213	1.60223	2.20601	2.80628	3.60621	4.2062
30min	1.40214	1.60224	2.20603	2.80631	3.60625	4.206246
1hr	1.40215	1.60225	2.20604	2.80633	3.60628	4.206268
2hr	1.40216	1.60226	2.20607	2.80635	3.6063	4.206288
4hr	1.40217	1.60229	2.20608	2.80637	3.60632	4.206312
8hr	1.40218	1.60232	2.20609	2.80638	3.60633	4.206332
16hr	1.40219	1.60233	2.2061	2.80639	3.60634	4.206348
24hr	1.40219	1.60234	2.20611	2.8064	3.60635	4.206364

TP5-1,Remolded

Time	1kg	2kg	4kg	8kg	16kg	32kg
6sec	0.6061	1.20428	1.80608	3.40608	4.00432	4.402
15sec	0.606156	1.00612	1.80616	3.40634	4.0061	4.40238
30sec	0.606172	1.206168	1.8062	3.408008	4.00612	4.404008
1min	0.606188	1.206208	1.806228	3.408064	4.00614	4.404036
2min	0.606204	1.206224	1.806256	3.408104	4.006152	4.40406
4min	0.606216	1.20624	1.806276	3.408136	4.006164	4.404084
8min	0.61558	1.206248	1.806292	3.40816	4.006172	4.404108
15min	0.60624	1.206256	1.806312	3.408176	4.00618	4.404128
30min	0.60626	1.20626	1.806324	3.408192	4.006184	4.404144
1hr	0.606272	1.206272	1.806334	3.408204	4.006192	4.404156
2hr	0.60628	1.20628	1.806348	3.408212	4.006196	4.404168
4hr	0.606304	1.206284	1.80636	3.408228	4.006202	4.404184
8hr	0.606316	1.206292	1.806364	3.408244	4.006208	4.404192
16hr	0.606322	1.206296	1.80768	3.40826	4.006211	4.404198
24hr	0.606328	1.2063	1.806372	3.408276	4.006214	4.404204