ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
DEPARTMENT OF CIVIL ENGINEERING

In-depth Investigation into Engineering Characteristics of Jimma Soils

By

Jemal Jibril

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In-depth Investigation into Engineering Characteristics of Jimma Soils

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

By

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DECLARATION

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

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Place: Addis Ababa Institute of Technology

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LIST OF SYMBOLS, ABBREVIATIONS AND UNITS

Symbols	Designations	SI units
1D	One Dimensional	--
A	Activity	--
A	Corrected Cross Sectional Area	m^2
AD	Air Dry	--
A_o	Original Cross Sectional Area	m^2
a_v	Coefficient of Compressibility	m^2 /kN
AASHO	American Association of State Highway Officials	--
ASTM	American Society for testing and Materials	--
c_v	Coefficient of Consolidation	m^2/day,cm^2 /min
CBR	California Bearing Ratio	%
Cc	Compression Index	--
CEC	Cation Exchange Capacity	meq/100gm
Cc	Consistency Index	--
Cs	Swelling Index	--
CH	Inorganic Clays of High Plastic	--
CST	Controlled Stress Rate	--
Cu	Cohesion for Undrained Shear Strength	kN/m^2
E_s	Modulus of Deformation	kPa/MPa
e	Void ratio	--
FS	Free Swell	%
GI	Group Index	--
Gs	Specific Gravity	--
H	Height	m
k_v	Coefficient of Permeability	m/day, cm/min
LI	Liquidity Index	%
LL/w_L	Liquid limit	%
m	Mass	kg
MDD	Maximum Dry Density	kg/m^3 or gm/cc
NMC	Natural Moisture Content	%
OD	Oven Dry	--
OH	Organic Clays of Medium to High Plasticity	--
OMC	Optimum moisture content	%
OCR	Over Consolidation Ratio	--
P	Compressive Force	kN
p	Vertical applied pressure	kN/m^2
p_c	Preconsolidation Pressure	kN/m^2
p'_c	Effective Preconsolidation Pressure	kN/m^2
ps	Swelling Pressure	kN/m^2
PI/I_P	Plasticity Index	%
PL/P_L	Plastic limit	%
q_u	Unconfined Compressive Strength	kN/m^2

RH	Relative Humidity	%
S	Shear Strength	kN/m^2
SL	Shrinkage Limit	%
SP	Swelling Potential	--
S_c	Settlement due to Consolidation	cm,mm
S_t	Sensitivity of soils	---
T	Consolidation Time factor	--
t	Time	second
USCS	Unified Soil Classification System	--
U	Degree of Consolidation	%
u	Pore Water Pressure	kN/m^2
UU	Unconsolidated Undrained	--
UCST	Unconfined Compression Strength Test	--
W	Weight	kN
w	Water Content	%
Å	Angestron	10^{-10}m
ρ	Density	gm/cc
γ	Unit weight	kN/m^3
γ_w	Unit weight of water	kN/m^3
γ_d	Dry unit weight	kN/m^3
γ_s	Solid unit weight	kN/m^3
σ'_o	Effective over burden Pressure	kN/m^2
σ'_v	Effective Vertical Pressure	kN/m^2
ε	Axial Strain	%
ϕ	Angle of Internal Friction	^{o}
μ	Micron= 10^{-6}	--

ABSTRACT

Jimma is located in south-west Ethiopia and is classified as "Weyna Dega" with high degree of humidity. The topography is predominantly flat and rolling terrain. It is mainly covered with black, gray and red colored plastic clay soils.

Identification of soils type, classification of soils, and determination of compression parameters and estimation of the soils shear strength is done. Samples of 16 disturbed and 12 undisturbed soils are collected from 8 different test pits. The locations of the test pits are selected so that they can well represent soils found in Jimma town. Disturbed samples are used for grain size analysis, index property, free swell, geochemical and compaction tests whereas undisturbed samples are used for consolidation and undrained shear strength tests.

The study shows that the natural moisture content varies from 37% to 68%. The specific gravity varies from 2.58 to 2.82. The soils are categorized as fine-grained soils from which more than 90% of the particle sizes are smaller than 0.075mm. The liquid limits range from 53% to 108% and plastic limits vary from 27% to 41%. According to the Unified Soil Classification System, the soils are categorized as CH-clay with high plasticity (fat clay) with a potential of expansion. And as per AASHTO classification system these soils classified as plastic clay which are unfavorable for sub-grade construction.

The soil in the study area consists of expansive and lateritic residual soils. From geochemical tests, the silica sesquioxide ratio varies from 1.2 to 1.79 which proves that the presence of lateritic soils. Their Atterberg limits are considerably affected by sample preparation methods for air dried and oven dried and testing procedures such as for 5 minute and 30 minutes mixing time.

For black and gray soils their Atterberg limits, free swell and consolidation tests indicate that these soils are expansive. The free swell ranges from 80% to 130% and Swelling pressures vary from 135kPa to 260kPa. The activities of the soils fall within range of 0.58 to 1.43. From different classification schemes, the expansive soils of the area are rated medium to high to very high in their degree of expansiveness.

The maximum dry unit weight and optimum moisture content of the studied soils, from compaction test, range from 12.7 to 14.00kN/m^3 and from 30% to 37% respectively. For lateritic soils, pre-test drying temperature and test procedures also affect the result of the maximum dry densities and optimum moisture contents.

Compression parameters such as coefficient of compressibility extends from 106x10^{-4} to 241x10^{-4} m^2/kN, compression index ranges from 0.238 to 0.399, swelling index varies from

0.0393 to 0.0947, preconsolidation pressure varies from 230 to 300kPa and the OCR is much greater than 2. The coefficient of consolidation extends from 7.24×10^{-2} to $1.15 \times 10^{-1} cm^2/min$ and coefficient of permeability ranges from 3.64×10^{-7} to 8.24×10^{-7} cm/min.

The undrained shear strength of the studied soils ranges from 54 to 157kPa for undisturbed samples, 26 to 90kPa for disturbed samples. The sensitivity of the soils is less than 2, indicating that the soils are less sensitive to disturbance. Tangent modulus and secant modulus of deformation varies from 3750 to19000kPa and 2160 to 6300kPa respectively.

Comparison is made for expansive and lateritic soils investigated in this study with previously done researches in other parts of Ethiopia. The comparison shows that the engineering properties of these soils from the study area lie within the range given by different researchers in the other part of the country.

1 INTRODUCTION

1.1 General

Jimma is one of the largest towns in Southwestern Ethiopia, Oromiya National Regional State. It is predominantly covered with red, black and gray soils. The red colored soils are located on higher elevations and good drained condition. In contrast, the black and gray soils are found in the part of the town having flat topography and unfavorable drainage condition.

Tropical residual soils such as lateritic soils can have characteristics that are quite distinctly different from those of transported soils. Particles of residual soils often consist of aggregates or crystals of weathered mineral matter that breakdown and become progressively finer if the soil is further manipulated. Depending on soil forming factors such as climate, drainage, topography and parent rocks some red soils of the study area can be lateritic soils.

Damage due to soil swelling is very noticeable in ordinary and light weight structures such as buildings, roads, retaining walls and canal and landfill liners. Ethiopia is one of the countries with extensive coverage of expansive soil. Properties of the expansive soils vary from place to place due to topography, climate, geological history and formation. Therefore, it is important to make localized study for the different regions.

The safety of any civil engineering structures resting on soil foundations is extremely dependent on the detail investigation of the engineering properties of the soils. This investigation shall include grain size distribution, consistency limits, consolidation settlement and shear strength tests. The shear strength and estimation of settlement of soils are important aspects in many foundation engineering problems such as the bearing capacity of shallow, deep foundations and bridge foundations, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls.

1.2 Background of the Problem

The construction of civil engineering structures is developing fast in Jimma like other main towns of Ethiopia. Furthermore, Jimma is known for its production of Arabica Coffee which is the backbone of the country’s export economy. Several governmental institutions and private business centers are established in the town and around it. So the need for detail geotechnical investigation of the sub-surface condition of these soils has a paramount importance for the safe and economical design and construction activities.

The topography of the town is predominantly flat with poor drainage condition and the area is mainly covered with black clay soils and has surface and subsurface water which is mostly encloses the flat area. For this reason constructions could be sensitive for structural failure as a result of excessive consolidation settlement. For the expansive soils, because of change in moisture conditions, there could be a significant volume change problem at different seasons. This could affect the stability of light weight structures as a result of cyclic swell-shrink process.

Since Jimma is found in tropical region, residual tropical soils are abundantly found in the area. Research done before showed that the soils have lateritic behavior in some areas. Laterite soil distribution map depicts Jimma is included in the lateritic soil region [1]. Some residual soils are used as construction materials such as road sub-grade and selected backfill for dams, building foundations and road constructions. So to assess the strength of these soils conducting basic engineering investigations is vital.

Little is known about the engineering behavior of Jimma soils. Even though there is one previously done scientific research [2] around this area, detail engineering properties investigation has not been conducted so far. The study shows most of the classifications show that the soils are fine grained clay soils. However, further investigation of the engineering characteristics of soils is very essential to use as a scientific database for further research works and for preliminary design to determine the presumptive soils pressure [3].

Conventional soil test procedures and classification systems focus primarily on the properties of soil in its remolded state. This is often misleading for some lateritic residual soils, whose properties are likely to be strongly influenced by in situ structural characteristics derived from the original rock mass or developed as a consequence of weathering. Residual soils are also affected by pre treatment conditions during index property determination such as moisture content.

1.3 Objectives

General Objectives of the study

- To render comprehensive geotechnical properties of Jimma soils and to compile a database for further related works.

Specific Objectives

- To investigate the type, nature, inherent properties and mechanical behavior of soils found in Jimma Town.
- To investigate the expansive behavior of the soil found in the town.
- To verify the lateritic nature of Jimma soils by conducting basic tests.
- To classify the soils found in the area which can help for further studies.
- To enable the estimation of settlement and heave prediction using compression parameters.
- To determine the shear strength of the soils and to evaluate the suitability as a construction and foundation materials.
- To fill the gap in terms of determining the engineering properties of soils found in Ethiopia.

1.4 Scope of the Study

The study covers the investigation of soils' nature, type and classification found in the study area based on the conventional identification procedures and classification schemes. It also covers the exploration to determine different compression parameters and strength characteristics of these soils. The study focuses on identification, classification and comparison of works for expansive and lateritic based on the common laboratory experiments.

To achieve this, from eight test pits both disturbed and undisturbed samples are systematically collected from town sections on representative locations for useful comparison of the differences in the test results.

1.5 Methodology

In order to achieve the objectives of this thesis, literature reviews of many investigators are done. Necessary information about the geology, climatic condition and topography of the site are collected and analyzed. Reconnaissance study of the area is done by visiting the entire part of the town. The location of test pits is selected so that it can well represent the soil types (visually) found in the town. Roughly it includes red clay, gray and the black cotton soils, which are the main soil types found in the study area.

Once the locations of the test pits are selected, the excavation work is conducted by daily laborers using local digging equipment. This excavation is continued up to 1.5m and 2.5m depth then disturbed samples are taken by plastic bags. Undisturbed samples are taken from both depths. After the undisturbed samples are extracted both ends of steel tube are sealed with wax (melted candle) and tighten by polyethylene bags. Both the disturbed and undisturbed samples transported to the Addis Ababa University Geotechnical laboratory.

Undisturbed samples are used for one dimensional consolidation, undrained shear strength, natural moisture content and unit weight tests. Disturbed samples are used to conduct index property tests such as specific gravity, Atterberg limit, grain size analysis, compaction and free swell. ASTM procedures are followed for all tests.

Using Microsoft Office Excel and Word, grain size distribution curve, liquid limit graph, compaction curve, consolidation and unconfined compression tests are plotted. Other computer based softwares have been also used for presentation of results and report compilation.

1.6 Structure of the Study

This thesis work is divided in to six Chapters, each covering a specific topic of the research work. In the introductory Chapter the background of the problem, objective, scope, methodology of the thesis work and structure of the thesis are presented. Chapter two deals with a brief literature review which discusses about soils and their classification, testing procedures, description of index properties, consolidation parameters and shear strength parameters and their testing procedures. As well as it deals with the behavior of expansive and lateritic soils and corresponding tests and test procedures. Chapter three presents the study area, including information on topography, climate, and geology of the study area. The fourth Chapter deals with types of laboratory tests conducted and results obtained. Using the test results obtained, discussion, analysis and comparison are done in Chapter five. Chapter six presents the conclusions and recommendations drawn from the research. Finally, grain size distribution curves, index property test results, consolidation test results and classification schemes are given in the relevant Appendices.

2 LITRATURE REVIEW

2.1 Soils

2.1.1 General

Soil, from geotechnical engineering point of view, is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. The process of weathering of the rock decreases the cohesive forces binding the mineral grains and leads to the disintegration of bigger masses to smaller and smaller particles.

The behavior of a structure depends upon the properties of the soil materials on which the structure rests. The properties of the soil materials depend upon the properties of the rocks from which they are derived. A brief discussion of the parent rocks is, therefore, quite essential in order to understand the properties of soil materials [4].

In general, soil consists of the particles (solids), voids, water in some of the voids, and air taking up the remaining void space. At temperatures below freezing the pore water may freeze, with resulting particle separation (volume increase), when the ice in the voids melts particles close up (volume decrease) [5].

2.1.2 Formation and Mode of Deposition of Soils

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition.

Mechanical Weathering

Mechanical weathering of rocks to smaller particles is due to the action of such agents as the expansive forces of freezing water in fissures, due to sudden changes of temperature or due to the abrasion of rock by moving water or glaciers. Temperature changes of sufficient magnitude and frequency bring about changes in the volume of the rocks in the superficial layers of the earth's crust in terms of expansion and contraction. Such a volume change sets up tensile and shear stresses in the rock ultimately leading to the fracture of even large rocks. This type of rock weathering takes place in a very significant manner in arid climates where free, extreme atmospheric radiation brings about considerable variation in temperature at sunrise and sunset. Erosion by wind and rain is a very important factor and a continuing event. Cracking forces by growing plants and roots in voids and crevasses of rock can force fragments separate apart [4].

Chemical Weathering

Chemical weathering (decomposition) can transform hard rock minerals into soft, easily erodible matter. The principal types of decomposition are hydration, oxidation, carbonation, desilication and leaching. Oxygen and carbon dioxide which are always present in the air readily combine with the elements of rock in the presence of water [4].

Residual and Transported Soils

On the basis of origin of their constituents and deposition soils can be divided into two large groups. These are residual soils and transported soils.

Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. The depth of residual soils depends primarily on climatic conditions and the time of exposure. In some areas, this depth might be considerable. In temperate zones residual soils are commonly stiff and stable.

The engineering properties of residual soils vary considerably from the top layer to the bottom layer. Residual soils have a gradual transition from a relatively fine material near the surface to large fragments of stones at greater depth. The properties of the bottom layer resemble that of the parent rock in many respects. The thickness of the residual soil formation is generally limited to a few meters [4].

Transported soils are soils that are found at locations far removed from their place of formation. The transporting agencies of such soils are glaciers, wind and water. The soils are named according to the mode of transportation. *Alluvial* soils are those that have been transported by running water. The soils that have been deposited in quiet lakes are *lacustrine* soils. *Marine soils* are those deposited in sea water. The soils transported and deposited by wind are *Aeolian* soils. Those deposited primarily through the action of gravitational force, as in landslides, are *Colluvial* soils. *Glacial* soils are those deposited by glaciers. Many of these transported soils are loose and soft to a depth of several meters. Therefore, difficulties with foundations and other types of construction are generally associated with transported soils [5].

The engineering properties of transported soils are entirely different from the properties of the rock at the place of deposition. Deposits of transported soils are quite thick and are usually uniform. Most of the soil deposits with which a geotechnical engineer has to deal are transported soils [4].

2.1.3 Common Types of Soils that are generally used in Engineering Practice

Some of the important soils that are found in general practice are given below [6].

Sand, gravel, cobbles and boulders: are coarse-grained cohesion less soils. Grain size ranges are used to distinguish between them. Particles of size from 0.06 to 2mm are referred to as sand and those with a size from 2 to 60mm as gravel. Fragments with diameters more than 60mm and less than200mm are known as cobbles and those with a size greater than 200mm are categorized as boulders.

*Silt: i*s a fine grained soil, having particle size between 0.06mm to 0.002mm. Inorganic silt has little or no plasticity and is cohesion less .Organic silt contains an admixture of organic matter. It is somewhat plastic, highly compressible, cohesive and relatively impervious. It is a very poor foundation material because of compressibility.

Clay: is composed of microscopic particles of weathered rock within a wide range of water content and exhibits plasticity. Clay is a cohesive fine-grained soil and the particle size is less than0.002mm.Organic clay contains some finely divided organic matter and is usually dark grey or black in color. Organic clays are highly compressible when saturated and their dry strength is very high.

Black cotton soil: is a residual soil containing a high percentage of the clay mineral montimorillonite. The soil has high shrinkage and expansive characteristics. Its color varies from dark grey to black. Great care is required when structures are to be built on black cotton soil.

Peat is composed of fibrous particles of decayed vegetable matter. It is so compressible that it is entirely unsuitable to support any type of foundation.

Lateritic soil: is residual soil formed in tropical region. It is formed by decomposition of rock, removal of bases and silica, and accumulation of iron oxide and aluminum oxide. The presence of iron oxide gives this soil the characteristics red or pink color. The lateritic soil is soft and can be cut with a chisel when wet. However, it becomes hard after long exposure. Hardness is due to cementing action of iron oxide and aluminum oxide. A hard crust of gravel size particles, known as laterite, exists near the ground surface.

2.2 General Behavior of Fine Grained Soils

2.2.1 Particle Size and Shape

The size of particles as explained earlier may range from gravel to the finest size possible. Their characteristics vary with the size. Soil particles coarser than 0.075 mm are visible to the naked eye or may be examined by means of a hand lens. They constitute the coarser fractions of the soils. Grains finer than 0.075 mm constitute the finer fractions of soils. It is possible to distinguish the grains lying between 0.075 mm and 2μ ($1\ \mu$ = 1 micron = 0.001 mm) under a microscope. Grains having a size between 2 μ and 1 Å can be observed under a microscope but their shapes cannot be made out. The shape of grains smaller than 1Å can be determined by means of an electron microscope. The molecular structure of particles can be investigated by means of X-ray analysis [4].

Unlike coarse grained soils which are influenced by their unit weight at aggregated mass level, the properties of fine grained soils are significantly dictated by their behavior at mineralogical and particle level [8].

The coarser fractions of soils consist of gravel and sand. The individual particles of gravel, which are nothing but fragments of rock, are composed of one or more minerals, whereas sand grains contain mostly one mineral which is quartz. The individual grains of gravel and sand may be angular, sub-angular, sub-rounded, rounded or well-rounded. Gravel may contain grains which may be flat. Some sands contain a fairly high percentage of mica flakes that give them the property of elasticity [4].

Silt and clay constitute the finer fractions of the soil. Any one grain of this fraction generally consists of only one mineral. The particles may be angular, flake-shaped or sometimes needle-like [4].

Specific Surface

Soil is essentially a particulate system, that is, a system in which the particles are in a fine state of subdivision or dispersion. In soils, the dispersed or the solid phase predominates and the dispersion medium, soil water, only helps to fill the pores between the solid particles. The significance of the concept of dispersion becomes more apparent when the relationship of surface to particle size is considered. In the case of silt, sand and larger size particles the ratio of the area of surface of the particles to the volume of the sample is relatively small. This ratio becomes increasingly large as size decreases from 2μ which is the upper limit for clay-sized particles. A useful index of relative importance of surface effects is the *specific surface* of grain. The specific surface is defined as the total area of the surface of the grains expressed in square centimeters per gram or per cubic centimeter of the dispersed phase [4].

The shape of the clay particles is an important property from a physical point of view. The amount of surface per unit mass or volume varies with the shape of the particles. Moreover, the amount of contact area per unit surface changes with shape. It is a fact that a sphere has the smallest surface area per unit volume whereas a plate exhibits the maximum. Ostwald [4] has emphasized the importance of shape in determining the specific surface of colloidal systems. Since disc-shaped particles can be brought more in intimate contact with each other, this shape has a pronounced effect upon the mechanical properties of the system. The inter-particle forces between the surfaces of particles have a significant effect on the properties of the soil mass if the particles in the media belong to the clay fraction [4]. Table 2.1 shows the specific surface area of clay mineral.

Table 2. 1 Specific surface area of clay minerals [8]

Clay mineral	Specific surface (m^2/gm)
Kaolinite	10–20
Illite	80-100
Montimorillonite	800
Chlorite	5-50
Vermiculite	5-400
Halloysite	40

2.2.2 Composition and Structure of Clay Minerals

Clay minerals are essentially crystalline in nature though some clay minerals do contain material which is non-crystalline (for example Allophane). Two fundamental building blocks are involved in the formation of clay mineral structures. They are: Tetrahedral unit (Fig. 2.1) and octahedral unit (Fig. 2.2).The tetrahedral unit consists of four oxygen atoms (or hydroxyls, if needed to balance the structure) placed at the apices of a tetrahedron enclosing a silicon atom which combines together to form a shell-like structure with all the tips pointing in the same direction. The oxygen at the bases of all the units lie in a common plane [4].

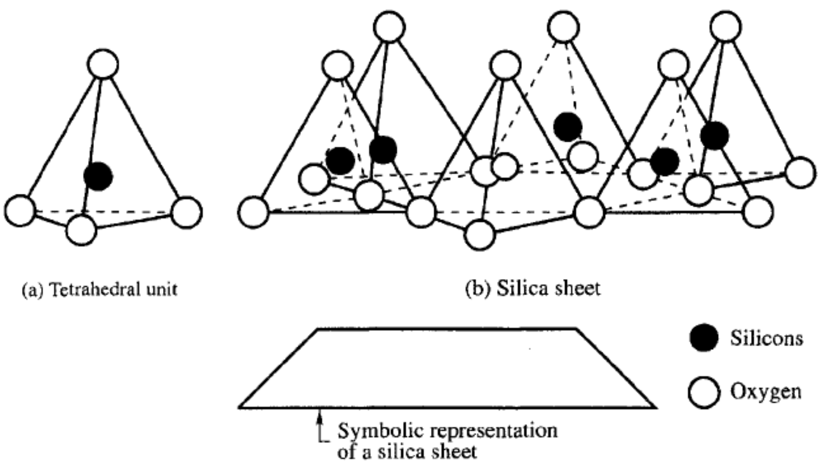

Figure 2. 1 Basic Structural units in the silicon sheet [8]

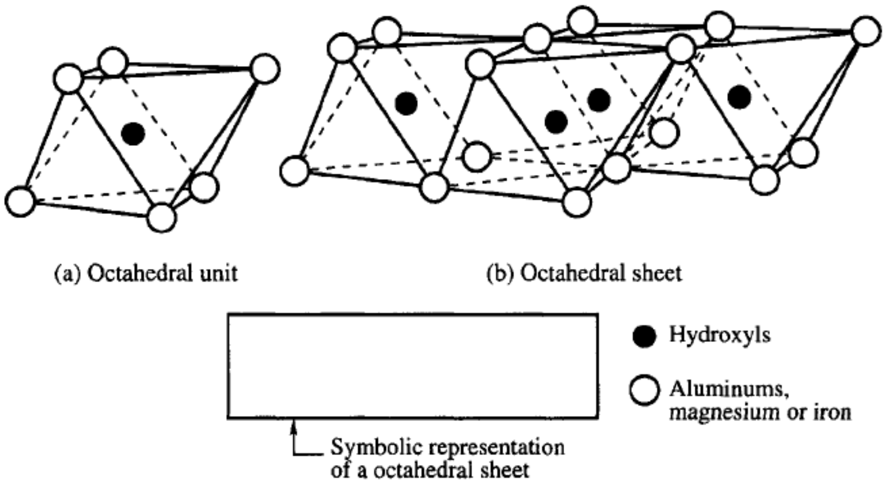

Figure 2. 2 Basic structural units in octahedral sheet-Gibssite sheet [8]

Formation of Minerals

The combination of two sheets of silica and gibbsite in different arrangements and conditions lead to the formation of different clay minerals as given in Table 2.2. In the actual formation of the sheet silicate minerals, the phenomenon of *isomorphous substitution* frequently occurs. Isomorphous (meaning same form) substitution consists of the substitution of one kind of atom for another [4].

Table 2.2 Common Clay minerals [4]

Name of mineral	**Structural formula**
I. Kaoline group - Kaolinite - Halloysite	 $Al_4Si_4O_{10}(OH)_8$ $Al_4Si_4O_6(OH)_{16}$
II. Montmorillonite group Montmorillonite	 $Al_4Si_4O_{20}(OH)_4nH_2O$
III. Illite group Illite	 $K_y(Al4Fe2.Mg_4.Mg_6)Si_{8-y}Al_y(OH)_4O_{20}$

Kaolinite Mineral

This is the most common mineral of the kaolin group. The building blocks of gibbsite and silica sheets are arranged as shown in Figure 2.3 to give the structure of the kaolinite layer. The structure is composed of a single tetrahedral sheet and a single alumina octahedral sheet combined in units so that the tips of the silica tetrahedrons and one of the layers of the octahedral sheet form a common layer. All the tips of the silica tetrahedrons point in the same direction and towards the center of the unit made of the silica and octahedral sheets. This gives rise to strong ionic bonds between the silica and gibbsite sheets. The thickness of the layer is about 7 Å (one angstrom = 10^{-8} cm) thick. The structure of Kaolinite mineral is presented in Figure 2.3 [4].

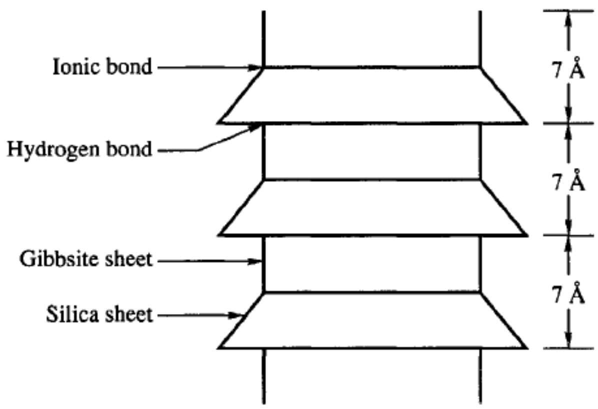

Figure 2. 3 Structure of Kaolinite layer

Montmorillonite Mineral

The structural arrangement of this mineral is composed of two silica tetrahedral sheets with a central alumina octahedral sheet. All the tips of the tetrahedral point in the same direction and toward the center of the unit. The silica and gibbsite sheets are combined in such a way that the tips of the tetrahedrons of each silica sheet and one of the hydroxyl layers of the octahedral sheet form a common layer. The atoms common to both the silica and gibbsite layer become oxygen instead of hydroxyls. The thickness of the silica-gibbsite-silica unit is about 10 Å. The structure of the mineral is presented in Figure 2.4 [4].

In stacking of these combined units one above the other, oxygen layers of each unit are adjacent to oxygen of the neighboring units, with a consequence that there is a weak bond and excellent cleavage between them. Water can enter between the sheets causing them to expand significantly and thus the structure can break. The soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics [4].

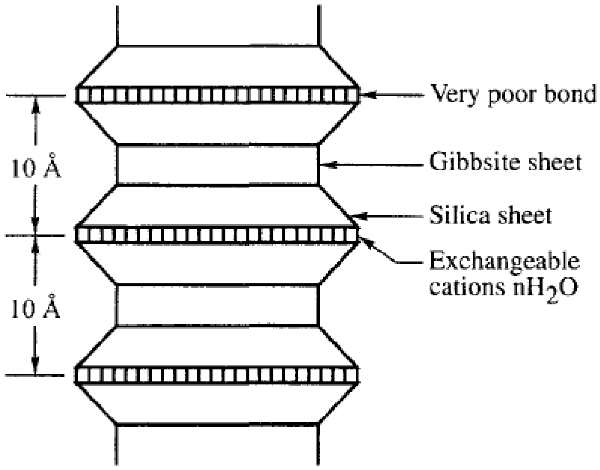

Figure 2. 4 Structure of Montimorillonite layer

Illite

The basic structural unit of illite is similar to that of montmorillonite except that some of thesilicons are always replaced by aluminum atoms and the resultant charge deficiency is balanced bypotassium ions. The potassium ions occur between unit layers. The bonds with thenonexchangeable K^+ ions are weaker than the hydrogen bonds, but stronger than the water bond of montmorillonite. Illite, therefore, does not swell as much in the presence of water as does montmorillonite. The lateral dimensions of illite clay particles are about the same as those of montmorillonite, 1000 to 5000 Å, but the thickness of illite particles is greater than that of montmorillonite particles, 50 to 500 Å. The structure of the mineral is presented in Figure 2.5 [4].

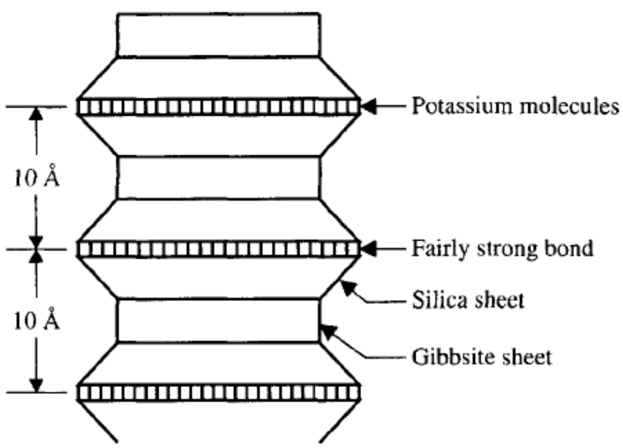

Figure 2. 5 Structure of Illite layer

2.2.3 Clay Particle-Water Relations

The behavior of a soil mass depends upon the behavior of the discrete particles composing the mass and the pattern of particle arrangement. In all these cases water plays an important part. The behavior of the soil mass is profoundly influenced by the inter-particle-water relationships, the ability of the soil particles to adsorb exchangeable cations and the amount of water present.

Adsorbed Water

The clay particles carry a net negative charge on their surface. This is the result of both isomorphous substitution and of a break in the continuity of the structure at its edges.

Minerals are said to have high or low surface activity, depending on the intensity of the surface charge. The surface activity depends not only on the specific surface but also on the chemical and mineralogical composition of the solid particle.

The negative charge on the surface of the soil particle, therefore, attracts the positive (hydrogen) end of the water molecules. More than one layer of water molecules sticks on the surface with considerable force and this attractive force decreases with the increase in the distance of the water molecule from the surface. The electrically attracted water that surrounds the clay particle is known as the *diffused double-layer of water*. The water located within the zone of influence is known as the *adsorbed layer* as shown in Fig. 2.6.The adsorbed water affects the behavior of clay particles when subjected to external stresses, since it comes between the particle surfaces. The forces associated with the adsorbed layers therefore play an important part in determining the physical properties of the very fine-grained soils, but have little effect on the coarser soils.

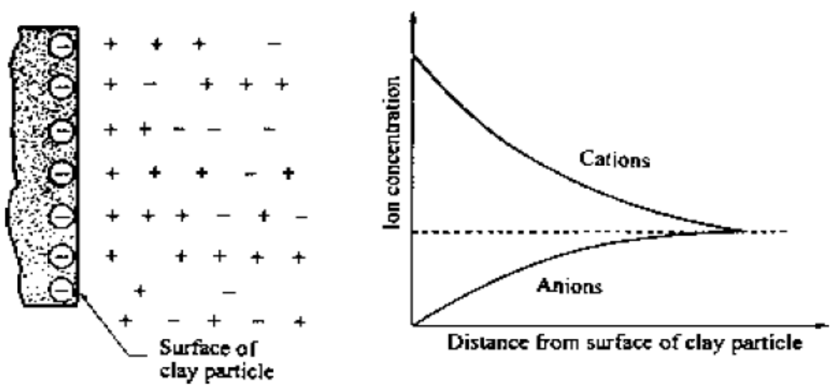

Figure 2. 6 Adsorbed water layer (diffuse double-layer) surrounding a soil particle [7].

Soils in which the adsorbed film is thick compared to the grain size have properties quite different from other soils having the same grain sizes but smaller adsorbed films. The most pronounced characteristic of the former is their ability to deform plastically without cracking when mixed with varying amounts of water. This is due to the grains moving across one another supported by the viscous inter-layers of the films. Such soils are called *cohesive soils*, for they do not disintegrate with pressure but can be rolled into threads with ease.

2.3 Index Properties of Soils

2.3.1 Moisture content

The water or moisture content of a soil material is defined as the ratio between the mass of water in the sample and the mass of solid material. It is expressed as percentage. For many materials, the water content is one of the most significant index properties used in establishing a correlation between soil behavior and its properties. In fine grained soils, the effect of water content on shear strength is highly significant. The water content of a material is used in expressing the phase relationship of air, water and solids in a given volume of material. In fine grained soils, the consistency of a given soil type depends on its water content. The water content of a soil along with its liquid and plastic limits is used to express its relative consistency termed as liquidity index.

Natural water content used to express the consistency of clay soil in its natural state. Consistency is a term used to indicate the degree of firmness of cohesive soils. The consistency of natural cohesive soil deposits is expressed qualitatively by such terms as very soft, soft, stiff, very stiff and hard. The physical properties of clays greatly differ at different water contents. A soil which is very soft at a higher percentage of water content becomes very hard with a decrease in water content. However, it has been found that at the same water content, two samples of clay of different origins may possess different consistency. Clay may be relatively soft while the other may be hard. Further, a decrease in water content may have little effect on one sample of clay but may transform the other sample from almost a liquid to a very firm condition. Water content alone, therefore, is not an adequate index of consistency for engineering and many other purposes.

2.3.2 Specific Gravity

Soil is a three-phase system comprising solid, liquid and gas. Many soil parameters like unit weight void ratio, porosity and water content relates the proportion of these phases with each other or to the total soil mass/volume but specific gravity of a soil is a property of soil solids only. Specific gravity of a soil is defined as the ratio of the mass in air of a given volume of soil solid to the mass in air of an equal volume of distilled water at stated temperature.

2.3.3 Grain Size Determination

Soil consists mostly of different sized soil particles as major constituent ingredient. The determination of the fractions of the particles will help to identify the soil type as well as to estimate many other engineering properties such as strength and permeability and also to identify whether the soil is suitable for construction projects such as highways, dams or as a backfill or for filter design.

Two methods are mostly used to determine grain size distribution are Sieve analysis for coarse grained portion of the soil (size coarser than 0.075mm) and Hydrometer analysis for fine grained portions (size finer than 0.075mm).

Soils are products of mechanical and chemical weathering and are found in a wide range of particle sizes and shapes. Simple sieve analysis can be used to differentiate the different size particles of coarse-grained soils. In the sieve analysis square holes between the wires of the sieve mesh provide a limiting size of particles retained on a particular sieve. However, it has to be noted that not all soil particles are spherical, cubical or of any regular shape. The sieve analysis does not provide any information on the shape of the soil grains regarding whether they are angular or rounded.

Statistical relationships have been established between grain size and significant soil properties. The suitability criteria for road airfield and embankment construction have been based on grain size distribution. The prediction of permeability can be done using grain size analysis. The proper gradation of filter material is established from particle size distribution. Grain size analysis is usually used in engineering soil classifications.

Simple sieve analysis is used for particles larger than 0.075mm while sedimentation analysis for particles smaller than 0.075mm. For soil sample that contains a measurable portion of their grains both coarser and finer than 0.075mm size combined analysis is required.

2.3.4 Atterberg limits

A fine-grained soil can exist in solid, semisolid, plastic, viscous or fluid state depending on its water content. The Swedish soil scientist Albert Atterberg originally defined seven “limits of consistency” to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid and plastic limits, are commonly used. (A third limit, called the shrinkage limit, is used occasionally.) Wide varieties of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the USCS or AASHTO system. The Atterberg limits are based on the moisture content of the soil. Atterberg proposed four states of soil and corresponding three boundaries as indicated in Figure 2.7.

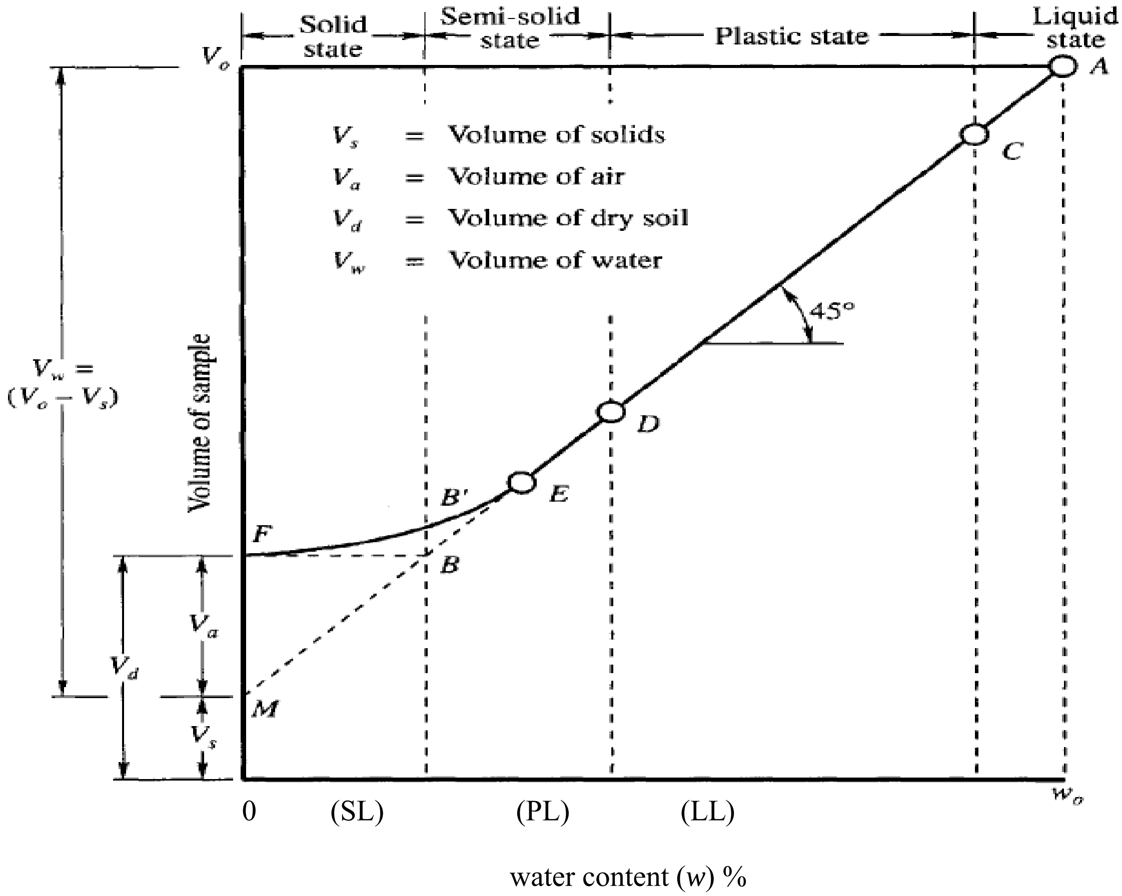

Figure 2. 7 Curve showing transition stages from the liquid to solid state

Definite reproducible values of these limits have to be obtained for engineering purposes. The liquid and plastic limits are widely used for engineering classification of fine-grained soil or fine portion of coarse-grained soil. The liquid limit, plastic limit, plastic index, and liquidity index of soils are used, either individual or together with other soil properties, in developing correlation such as shear strength, compressibility, permeability, and shrink and swell.

Liquid Limit (LL)

The liquid limit of a soil is the water content at the boundary between the liquid and plastic states. The water content at this boundary is arbitrarily defined as the water content at which, two halves of a soil pat placed in a brass cup, cut with standard groove, and dropped from a height of 1cm will undergo a groove closure of about 1.3 cm when the cup is dropped 25 times at the rate of 2 drops per sec.
In addition to being useful in identifying and classifying soils, the liquid limit can also be used to compute an approximate value of the compression index Cc for normally consolidated clays.

Plastic Limit (PL)

The plastic limit of a soil is the water content at the boundary between the plastic and semisolid state. The water content at this boundary is arbitrarily defined as the water content at which soil begins to crumble when rolled into threads of specified size (3.2mm).

Shrinkage Limit (SL)

The term shrinkage limit, expressed as water content in percent, is typically assumed to represent the amount of water required to fill the voids of a given cohesive soil at its minimum void ratio obtained by drying (usually oven). Thus, the concept shrinkage limit can be used to evaluate the shrinkage potential or possibility of development, or both, of cracks in earthworks involving cohesive soils. Data obtained from this test method may be used to compute the volumetric shrinkage and linear shrinkage.

2.3.5 Plasticity, Liquidity and Consistency Indexes

Plasticity Index (PI)

The range of water content between the liquid and plastic limits, which is an important measure of plastic behavior, is called the plasticity index.

$$PI = LL - PL \tag{2.1}$$

Where:

PI= Plasticity index
LL = liquid limit
PL= plastic limit

Plasticity index PI indicates the degree of plasticity of a soil. The greater the difference between liquid and plastic limits, the greater is the plasticity of the soil. A cohesion less soil has zero plasticity index. Such soils are termed as non-plastic. Soils possessing large values of LL and PI are said to be highly plastic or fat. Those with low values are described as slightly plastic or lean. Organic clays possess liquid limits greater than 50. The plastic limits of such soils are equally higher. Therefore soils with organic content have low plasticity indices corresponding to comparatively high liquid limits. Atterberg classifies the soils according to their plasticity indices as in Table 2.3

Table 2.3 Soil classifications according to plasticity index [4]

Plasticity Index	Plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium Plastic
>17	Highly plastic

Liquidity Index (LI)

The Atterberg limits are found for remolded soil samples. These limits as such do not indicate the consistency of undisturbed soils. The index that is used to indicate the consistency of undisturbed soils is called the liquidity index.
The liquidity index is expressed as:

$$LI = \frac{NMC - PL}{PI} \qquad (2.2)$$

Where: NMC = Natural moisture content

The liquidity index of undisturbed soil can vary from less than zero to greater than 1. The value of LI varies according to the consistency of the soil as in Table 2.4.
The liquidity index indicates the state of the soil in the field. If the natural moisture content of the soil is closer to the liquid limit the soil can be considered as soft, and the soil is stiff if the natural moisture content is closer to the plastic limit. There are some soils whose natural moisture contents are higher than the liquid limits. Such soils generally belong to the montmorillonite group and possess a brittle structure. A soil of this type when disturbed by vibration flows like a liquid. The liquidity index values of such soils are greater than unity. One has to be cautious in using such soils for foundations of structures [6].

Consistency Index (CI)

The consistency index indicates the consistency (firmness) of a soil. It shows the nearness of the water content of the soil from its plastic limit.
The liquidity index is expressed as:

$$CI = \frac{LL - NMC}{PI} \qquad (2.3)$$

A soil with a consistency index of zero is at the liquid limit. It is extremely soft and has negligible shear strength. On the other hand, a soil at a water content equal to the plastic limit has a consistency index of 1, indicating the soil is relatively firm. A consistency index of greater than 1 shows that the soil is relatively strong, as it is the semi-solid state. A negative value of consistency index is also possible, which indicates that the water content is greater than the liquid limit [6].

Table 2.4 Values of LI and CI according to consistency of soil [4]

Consistency	LI	CI
Semi-solid or solid state	Negative	>1
Very stiff sate (NMC=PL)	0	1
Very soft sate(NMC=LL)	1	0
Liquid state (when disturbed)	>1	Negative

2.3.6 Activity of Clay Soils

Skempton [6] considers that the significant change in the volume of a clay soil during shrinking or swelling is a function of plasticity index and the quantity of colloidal clay particles present in soil. The clay soil can be classified *inactive, normal* or *active* [4].

The *activity* of clay is expressed as

$$A = \frac{PI}{C} \tag{2.4}$$

Where: A= Activity C = Clay percent finer than 2μm

Table 2.5 gives the type of soil according to the value of A. The clay soil which has an activity value greater than 1.25 can be considered as belonging to the swelling type. The relationship between plasticity index and clay fraction is shown in Figure 2.8.

Table 2.5 Soil classification according to activity [6]

Activity, A	**Soil type**
<0.75	Inactive
0.75-1.25	Normal
> 1.25	Active

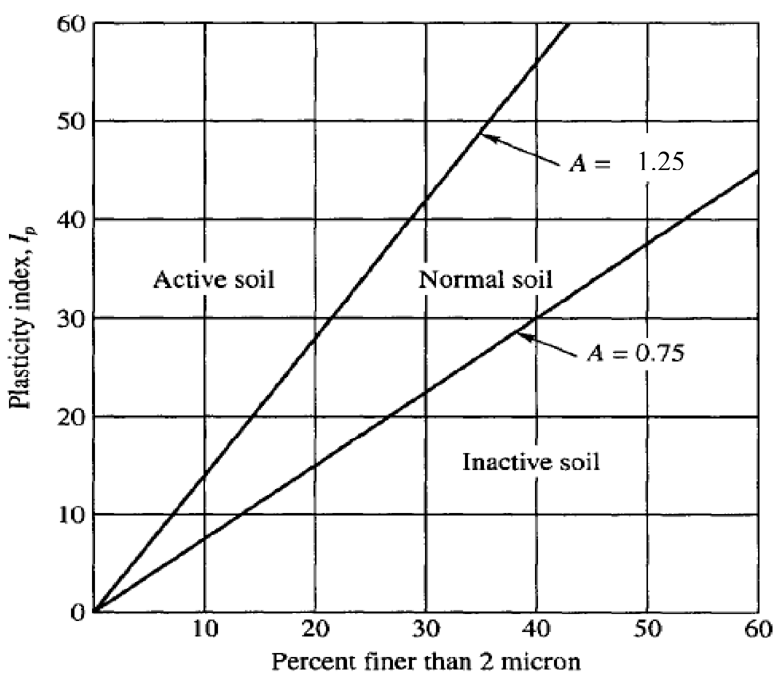

Figure 2. 8 Classification of soils according to their activity [6]

2.3.7 Soil Classification Systems

All widely used engineering soil classifications involve a combination of particle size and measures of plasticity and textural soil classifications. In addition to providing an orderly system for classification, the use of particle size and plasticity permits the Engineer to estimate the engineering properties of soils such as compaction, settlement, drainage, frost susceptibility, placement, excavation, and embankment characteristics. As grain size decreases, engineering problems associated with soils tend to increase. Also the difficulty with which particle-size distribution in a soil sample is determined also increases. As a result, the proportions and properties of the so-called fines (silt and clay sizes) present in a soil are evaluated by their plasticity rather than by more time-consuming sediment logical procedures. The measures of plasticity, the Atterberg limits, are directly applicable to design and construction uses of a soil, whereas strict size ranges and amounts are not.

The most widely used classification schemes are those that divide soils into an orderly, easily remembered system of groups, or classes, that have similar physical and engineering properties and that can be identified by simple and inexpensive tests. These groups ideally provide estimates of both the engineering characteristics and performance of soils for design and construction. The descriptions of soils within the groups of a given classification typically are represented by alphabetical or alphanumeric symbols for rapid identification in written material, graphic boring logs, and on engineering drawings. The continued use of a few engineering soil classifications is the result of the provision in each for the needs of the Civil Engineer as well as the adaptability of the classification to the variety of soils encountered in engineering practice.

Soils classification can be done in to two main ways.
First, Visual classification of soils (field classification method) - during excavation and sampling operations in the field classification has to be carried out quickly and without gradation analyses or Atterberg limits.
Second, laboratory classification of soils- this classification system is used after gradation analyses or Atterberg limit test is done in the laboratory. At the present time, two major soil classification systems are available for general engineering use. They are the unified soil classification system (USCS), and the American association of state highway and transport official (AASHTO) system. Both systems use simple soil properties such as grain-size distribution, liquid limit, and plasticity index of soil. Similarly, it is possible to use other classification systems depending on the type, size and texture of the soils. Such systems are International Classification System, Massachusetts Institute of Technology (MIT) classification system and Textural Classification System [6].

The soil identified in the field is done by conducting the following simple test. The sample is first spread on a flat surface. If more than 50% of the particle are visible to the naked eye (unaided eye), the soil is coarse-grained; otherwise fine-grained soils. The fine grained particles are smaller than 0.075mm size and are not visible to unaided eye. The fraction of the soil smaller than 0.075mm size, that is the clay and the silt fraction, is referred to as fines [6].

For the fine grained soils, the following tests shall be conducted. These are diletancy (reaction to shaking) test, toughness test and dry strength test as well as consistency test.

Here two most popular engineering soil classifications are presented: USCS and AASHTO classification systems.

Unified Soil Classification System (USCS)

The Unified Soil Classification System was developed cooperatively by the U.S. Army Corps of Engineers (USAE) and the U.S. Bureau of Reclamation (USBR). The USC classification was published in 1953. It has since been adopted by the American Society for Testing and Materials (ASTM) as the standard classification of soils for engineering purposes. The success of the USC is indicated by its routine use worldwide and its acceptance for international geotechnical communication.

The USC system is a textural-plasticity classification scheme. Soils are divided into two major groups, coarse-grained and fine-grained soils, using the No. 200 sieve as the size criterion. When more than half of the soil sample is larger than the No. 200 sieve, it is classified as coarse-grained and is further subdivided by sieving and gradation. When more than half of the soil sample is smaller than the No. 200 sieve, it is classified as fine-grained and is subdivided primarily based on liquid limit values and degree of plasticity. The presence of organic material is an additional classification factor for fine-grained soils. Paired letter symbols are used for each soil group in the USC system. The first symbol refers to the predominant particle size (with the exception of organics). The second symbol for coarse-grained soils refer to gradation for clean (little or no fines) soils and the presence of silt and clay-size particles for soils with appreciable amounts of fines. The second symbol for fine-grained soils subdivides on the basis of low (L) or high (H) plasticity.

Laboratory determination of liquid limit and plasticity indexes for a soil sample permits assignment of fine-grained soils (including the fine fraction of coarse-grained soils) to the proper group by use of the plasticity chart, or A-line diagram, as illustrated by Figure 2.9.

Field test procedures may be used to estimate the group to which a fine-grained soil should be assigned prior to more definitive laboratory testing. These tests are measures of crushing strength, dilatancy, and toughness, all measures of relative proportions of silt and clay sizes and plasticity.

The USC system includes typical soil names with the classification system. Soils that are intermediate between two groups may be identified symbolically by combined notation such as SM-ML and SC-CL. The table in Appendix C-1 and the tests required for soil-group assignment have been refined recently to formally include combinations. Figure 2.9 is also used to classify fine-grained soils from Atterberg limits using USC system notation [9].

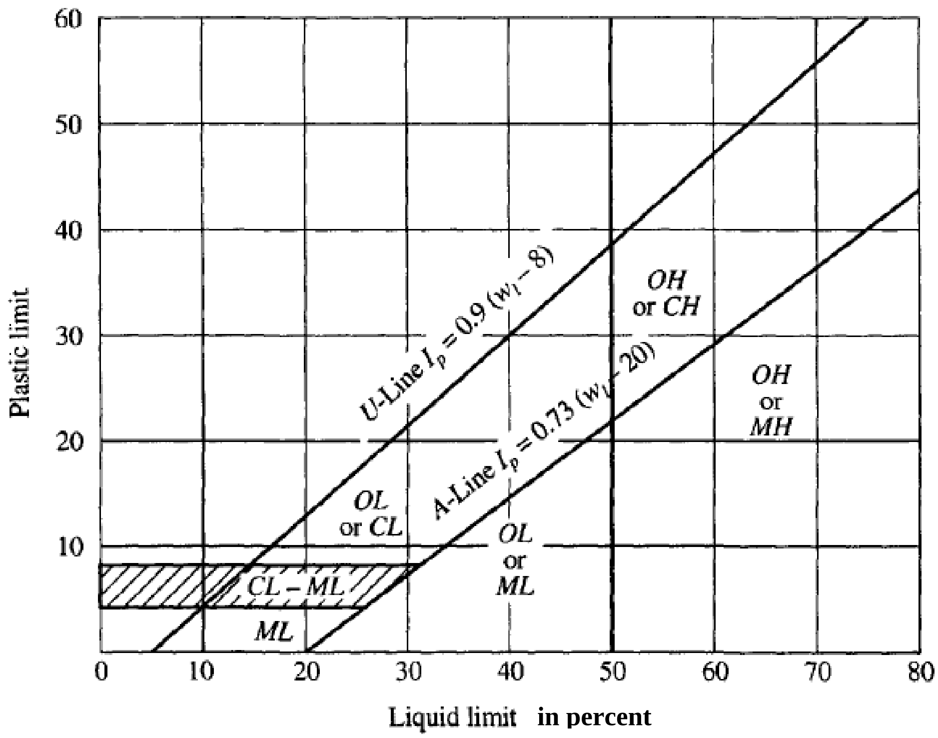

Figure 2. 9 Plasticity chart for classification of fine-grained soils [9]

The Unified Soil Classification System is now almost universally accepted and has been adopted by the American Society for Testing and Materials (ASTM). So this study also Unified Soil Classification System (USCS) was used for classification.

AASHTO Classification System

In addition to the USC system, an engineering soil classification was developed in 1928 by the U.S. Bureau of Public Roads, which is now called the American Association of State Highway and Transportation Officials (AASHTO). It is a textural-plasticity classification that uses sieved fractions and Atterberg limits for assignment of soils to seven main groups and several subgroups.

The classification is more specific than the USC system in the limits placed on size ranges and amounts and ranges of liquid limits and plasticity indexes for fines. As with the USC system, these limits are placed on groups within both the granular (coarse-grained) and silty/clay (fine-grained) soils as required by soil gradations. Rather than using the No. 4 sieve (4.75 mm) of the USC system as the upper limit of the sand-size range, the AASHTO classification uses the No. 10 sieve (2.0 mm) as the upper size limit of sand. However, the No. 200 sieve (0.075 mm) used in the USC system is retained to separate the finer fractions from sand.

The AASHTO system classifies soils into eight groups, A-1 through A-8. Appendix C-2 illustrates the current AASHTO soil classification system. Soil group A-8 is not shown, but is peat or muck based on a visual classification. Shown are groups A-1 through A-7 with two subgroups in A-7 for a total of 12 subgroups. The soil with the lowest number A-1, is the most suitable as a highway material or sub grade. The lower is the number of soil category; the more suitable is the soil. For example, the soil A-4 is better than the soil A-5.

Fine-grained soils are further rated for their suitability for highways by the group index (GI), determined as follows:

$$GI = (F-35)[0.2+0.005(LL-40)]+0.01(F-15)(PI-10) \tag{2.5}$$

Where: F = percentage by weight passing through sieve No.200 (size 0.075 mm), expressed as whole number; LL = liquid limit; and PI = plasticity index.

While calculating GI from the above equation, if any term in the parentheses becomes negative, it is dropped; not given a negative value. The group index is rounded off to the nearest whole number. If the computed value is negative, the group index is reported as zero. The group index is appended to the soil type determined from the classification table. For example, A-6 (15) indicates the soil type A-6, having a group index of 15. The smaller the value of the group index, the better is the soil in the category. A GI of zero indicates a good sub-grade, whereas a group index of 20 or greater shows a very poor sub-grade. The GI must be mentioned even when it is zero to indicate that the soil has been classified as per AASHTO system [10].

2.4 Laterite Soils

2.4.1General

Blight [11] describes laterite soils as highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical condition. The three major agencies of weathering are being physical, chemical and biological processes. In the process the parent rock and rock minerals break down, releasing internal energy and forming soils having a lower internal energy which are more stable. Physical processes increase surface area so that chemical attack increases. Biological weathering includes both physical and chemical actions.

Climate and topography influence the rate of weathering. Physical weathering is more pre-dominant in dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature. Topography on the other hand, controls the rate of weathering by partly determining the amount of available water and the rate at which it moves through the zone of weathering [11].

Laterites 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. Due to the presence of iron oxides lateritic soils are red in color ranging from light through bright to brown shades.

2.4.2 Formation of Laterite Soils

The lateritic soil formation involves three major processes which are identified as follows [12]:

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.

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

Desiccation /dehydration/: Dehydration (either partial or complete) alters the composition and distribution of the sesquioxide rich materials in a manner which is generally not reversible upon wetting. Dehydration also influences the formation processes of clay minerals. In the case of total dehydration, strongly cemented soils with a unique granular soils structure may be formed [11].

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.

Laterite occurs mostly in the tropical and sub-tropical regions with hot, humid climatic conditions. It has been suggested that a mean annual temperature of around 25°C is needed for their formation, and in seasonal situations there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold season, laterites do not develop freely. The minimum annual rainfall required for laterite formation is generally at least 750 mm [11].

2.4.3 Laterite and Lateritic Soils

Many conflicting definitions of laterite have been proposed in the technical literature. Buchinan is the earliest, and is based on the ability of a soft red material to harden on exposure to air. The term laterite became synonymous with the tropical weathering product of virtually all igneous rocks and was applied to red soils, weather hardening was involved or not [13].

A more precise definition is resulted in the application of chemical criteria to tropical weathered soils [13]. For classification of soils according to the degree of laterization gives a reasonable classification result. The degree of laterization is estimated by the silica-sesquioxide (S-S) ratio ($SiO2/(Fe2O3 + Al2O3)$). If an S-S ratio of 1.33 or smaller the soils is classified as Laterite soil; for an S-S ratio of 1.33 to 2.0 it is said to be lateritic soil and for an S-S ratio of 2.0 or higher = non-lateritic, tropical soil Rossiter [14].

2.4.4. Effect of Pre- treatment

Some lateritic soils show changes in physical properties when tested under different conditions. Laterites formed under continuously wet regions are likely to be characterized by high natural water contents, high liquid limits, and irreversible changes upon drying. Upon drying the plasticity decreases and grain size increases such that much of the clay sized material agglomerates to the size of silt [12]. The geotechnical behavior of soil altered up on drying due to alteration of clay minerals on dehydration and/or aggregation of fine particles to form larger particles. Important factor contributing to the close spacing of particles is the development of capillary stresses of significant magnitude. These capillary stresses lead to particle aggregation and reduce the available surface for interaction with water which is reflected in the reduction in plasticity characteristics [14].
Hence, index properties and engineering properties should be tested by simulating the actual condition.

2.4.5 The Self-Hardening Concept

The property of self-hardening under cycles of wetting and drying could be important for the performance of certain lateritic soils. Such property is less common in granular lateritic soils. At present self-hardening is not fully understood and cannot easily be predicted. The factors which influence the ability to self-harden may also affect the sensitivity of certain lateritic soils to test procedures. There is no doubt that some of these soils are sensitive to the way in which they are prepared for laboratory testing and to the actual testing method employed. This sensitivity can be attributed to three basic factors, described below [13].

Aggregation of clay-size particles

The sesquioxides within the fine fraction of tropical soils tend to coat the surface of individual soil particles. One reason for this is an electrical bonding between the negatively-charged kaolinite and the positively-charged hydrated oxides. The coating can reduce the ability of the clay minerals to absorb water. 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 sesqueoxide coatings, with an attendant increase in plasticity.

According to [11] the effect of drying prior to testing is attributed to either increased cementation due to oxidation of the iron and aluminum sesqueoxides or dehydration of Allophane; or both.

Irreversible changes in plasticity on drying

When laterites dry, soils which contain hydrated oxides of iron and aluminium may become less plastic (i.e. exhibit lower Atterberg Limit values). This is partly because dehydration of the sesquioxides creates a stronger bond between the particles, which is resistant to penetration by water. Drying is also accompanied by shrinkage, which brings the particles closer together, and the attractive forces become so strong that water no longer penetrates. The process cannot be reversed by re-wetting. The effect takes place during air-drying but becomes more pronounced on oven-drying at higher temperatures [11].

Loss of water of hydration on drying

The water of hydration in the sesquioxides of iron and aluminium may be driven off by oven-drying at l05°C, the standard temperature for testing temperate region soils. This water normally takes no part in the engineering performance of the material, but is reflected in the test results as higher moisture content [11].

2.4.6 Consideration on Index Property Tests for Lateritic Soils

To identify susceptibility of lateritic soils to the effect of clay mineral aggregation to drying and to re-wetting (dehydration of sesquioxides), bulk sample at its natural moisture content should be tested in laboratory. The following tests should be conducted [13].

Loss of water of hydration

Two test portions should be prepared for moisture content determinations. One should be oven dried at 105^{o}C until successive test weightings show that no further weight loss is taking place, and the moisture content should be then determined. The other sample should be air-dried or oven dried at no more than 50^{o}c until successive test weightings show no further weight loss, and the moisture content then determined. The two results should be compared. A.B. Fourier recommends the moisture variation 4-6% or more indicates that structural water is present [11]. If this is confirmed by repeated tests then the oven drying temperatures for the subsequent programme of tests should be changed to an appropriate value or reduce the difference from moisture content tested conventionally [11].

Disaggregation of clay-size particles on mixing

As A.B. Fourier [11] suggests five air dried test portions should be mixed with water to give the range of water contents suitable for liquid and plastic limit determinations. The mixing time should be no more than 5 minute, and the mixed samples should be left to cure overnight before testing. After determining the moisture content for each test point on a part of each test portion, the reminder should then be mixed for a further 25 minutes before again determining the liquid limit. A.B. Fourier suggests a "greater than 5%" difference between the liquid limits of the specimens 5 minutes and 30 minutes mixing times indicates a breakdown of the aggregations of clay sized particles within the material [11].

If this disaggregation is confirmed by further tests, the main test programme should include the following instructions. These are: Limit the mixing times (no more than 5 minutes) and use fresh material for each moisture content point in compaction tests, for liquid and plastic limit determinations.

One should consider that 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 0.425 mm sieve until the water runs clear. The material passing the sieve is collected and used for the Atterberg limit tests [11].

2.4.7 Laterites as Construction Materials

Concretionary laterites are valuable road pavement materials, widely used in the tropics as sub-base, base material and for gravel roads. Laterites are good material for embankment construction [11]. Laboratory testing to check the suitability of concretionary laterites to be used as road pavement materials should take into account how these, materials are affected by the testing procedures [13].

Some lateritic soils are sensitive to pretreatment and testing procedures. So laboratory testing should be simulated to site condition.

2.5. Expansive Soils

2.5.1 General

Expansive soils are mostly found in the arid and semi-arid regions of the world where poor internal drainage and low to moderate rainfall. The presence of montmorillonite clay in these soils imparts them high swell-shrink potentials [15]. Expansive soils which are also known as black cotton soils, clay soils which are rich in montimorillonite. Expansive soils are principally residual soils, derived from the weathering of basic volcanic rocks. These soils may be black or grey in color. They have a potential for heaving with an increase of moisture content and they shrink with a corresponding decrease of moisture content.

2.5.2 Clay Mineralogy of Expansive Soils

The parent materials of expansive soils may be classified into two groups. The first group comprises the basic igneous rocks such as basalt, dolerite sills and dykes, gabbros, etc., where feldspar and pyroxene minerals of the parent rocks decompose to form montmorillonite – the predominant mineral of expansive soil – and other secondary minerals. The second group comprises sedimentary rocks that contain montmorillonite, and break down physically to form expansive soils. There are indications that confirm that the expansive soils of Ethiopia are derived from both groups.

Particle size alone cannot determine clay minerals. The most important grain-level property of fine-grained soil is mineralogical composition. Clay minerals are essentially micro-crystalline, hydrous aluminum (occasionally magnesium or iron) sheets silicates are having a layered flaky structure. The magnitude of expansion depends upon the kind and amount of clay minerals present, their exchangeable ions, the electrolyte content of the aqueous phase, and the internal structure.

2.5.3. Formation of Clay Minerals

Clay minerals are formed by weathering .This includes disintegration, oxidation, hydration and leaching. The setting for the formation of Montimorillonite is extreme disintegration, strong hydration and restricted leaching. Magnesium, calcium, sodium, iron cations may accumulate in the system of leaching. The above conditions are favorable in semi-arid regions with relatively low rainfalls or highly seasonal moderate rainfall, particularly where evaporations exceed rainfall or rainfall with restricted leaching.

In clay minerals, the most common exchangeable cations are Ca++, mg++, H+, K+, NH4+, Na. The existence of such charges is indicated by the ability of the clay to absorb ions from the solutions. Cations (positive ions) are more ready absorbed than an ions (negative ions). Hence negative charge must be predominant on the clay surface. Cation exchange capacity (CEC) has major significance in determining clay mineral properties, particularly the conditions with which they adsorb water. Montimorillonite, on the other hand, have high cation exchange capacity.

2.5.4 Swell Pressure and Swell Potential

Swelling pressure is a very useful index of the trouble potential of an expansive soil. This pressure is the maximum force per unit area that needs to be applied over a swelling soil to prevent volume increase. A swelling pressure of less than 20 kPa may not be regarded as of much consequence. The swell potential of a soil, in comparison, is the magnitude of heave of a soil for given final water content and loading condition [16].

2.5.5. Identification

Investigation of expansive soils generally consists of two important phases. The first is the recognition and identification of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss tests and classification procedures that are commonly used to identify expansion potential.

Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include [15] usually have color of black or gray, Wide or deep shrinkage cracks, high dry strength and low wet strength, stickiness and low trafficability when wet, cut surfaces have a shiny appearance, appearance of cracks in nearby structures, arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature and etc.

Laboratory Identification

Methods to identify expansive soils in the laboratory can be categorized into three methods.

Mineralogical Identification:

It is generally believed that swelling potential of any clay can be evaluated by identification of constituent mineral of that clay. The techniques that can be used are [16]: X-ray diffraction, Differential thermal analysis, Dye adsorption, Chemical analysis and Electron microscope resolution. Using combinations of these methods, the different types of clay minerals present can be evaluated quantitatively.

Indirect Methods:

These methods include the index property, Potential Volume Change (require special PVC apparatus) and activity methods, which are valuable tools in evaluating the swelling property. However, none of the indirect methods should be used independently.

Direct Methods:

These methods offer the most useful data by direct measurement and tests are simple to perform and do not require complicated equipments. Testing should be performed on a number of samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer.

2.5.6 Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential as well as combinations to arrive at a rating.

Classification Using General Methods

Soils are classified in the general schemes: Unified Soil Classification System (USCS) and the American Association of State High way and Transportation Officials (AASHTO) method according to index properties. Soils rated CL or CH by the USCS, and A-6 or A-7 by AASHTO, can be considered potentially expansive [15].

Cation Exchange Capacity (CEC)

The CEC is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milli-equivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.6.

Table 2.6 Typical CEC values of basic clay minerals after Mitchell [15]

Clay Mineral	**CEC (meq/100 gm)**
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

Classification Using Soil Index Properties

Prediction of swelling potential using Atterberg limits is the most popular approach. Many of the procedures also include clay content.

Method of Chen:

Chen [15] presented a single index method for identifying expansive soils using only plasticity index as it is shown in Table 2.7.

Table 2.7 Expansive soil classification based on plasticity index [15]

Swelling Potential (SP)	Plasticity Index (PI)
Low	0 -15
Medium	10 -35
High	20 – 55
Very High	>55

The Van Der Merwe Method:

Defining activity as the ratio of the plasticity index to the percentage of the clay fraction(minus 2 micron size) present in the sample, two different types of charts were presented by different authors to identify the swelling potential of expansive soils. The first chart, presented as in Figure 2.10, known as the Activity chart, is a plot of plasticity index versus clay fraction. The second chart is presented by Seed et al [15], where activity versus clay fraction is presented in Figure 2.11.

The Van Der Merwe method, also called the South African method, gives a very practical approach to classify and estimate heave of expansive soils [15].

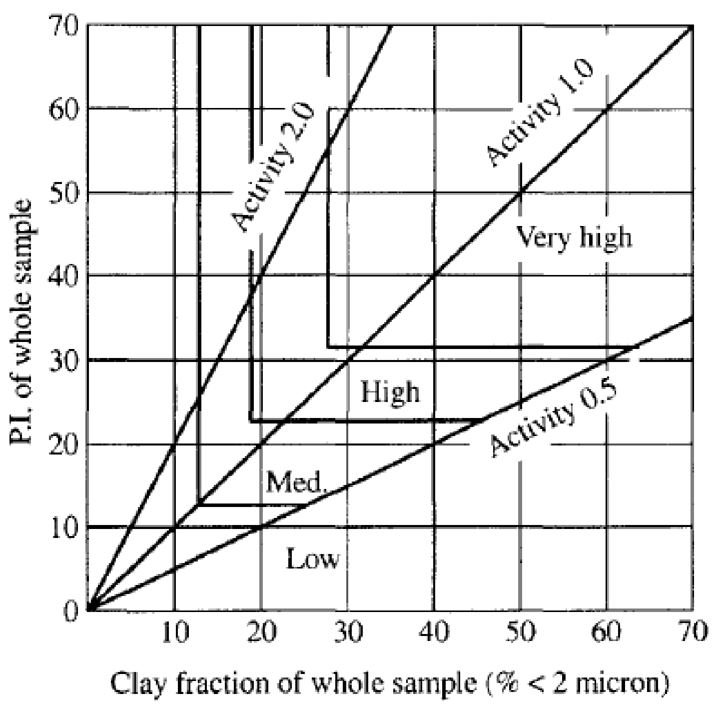

Figure 2. 10 Volume change potential classification for clay soils V.D. Merwe [15]

***Method of Seed et al.*:**

After an extensive study on swelling characteristics of remolded, artificially prepared and compacted clays, Seed et al., [15] have developed a chart based on activity and percent clay sizes and which is presented in Figure 2.11. The expansion was measured as the percentage of swell on soaking from 100% maximum density and optimum moisture content in a standard AASHTO compaction test under a surcharge of 1psi (6.9 kPa).

The activity here is defined as:

$$A_c = \frac{PI}{C - 5} \tag{2.6}$$

Where, A_c = Seed's activity number

C = the percentage of clay size finer than 0.002mm and

PI = the plasticity index.

Swelling potential can be estimated from the following formula as a function of plasticity index [15].

$$SP = 60kPI^{2.44} \tag{2.7}$$

Where: SP= swelling potential, k= 3.6x10-5 a factor for a clay content 8 to 65 percent

PI = plasticity index

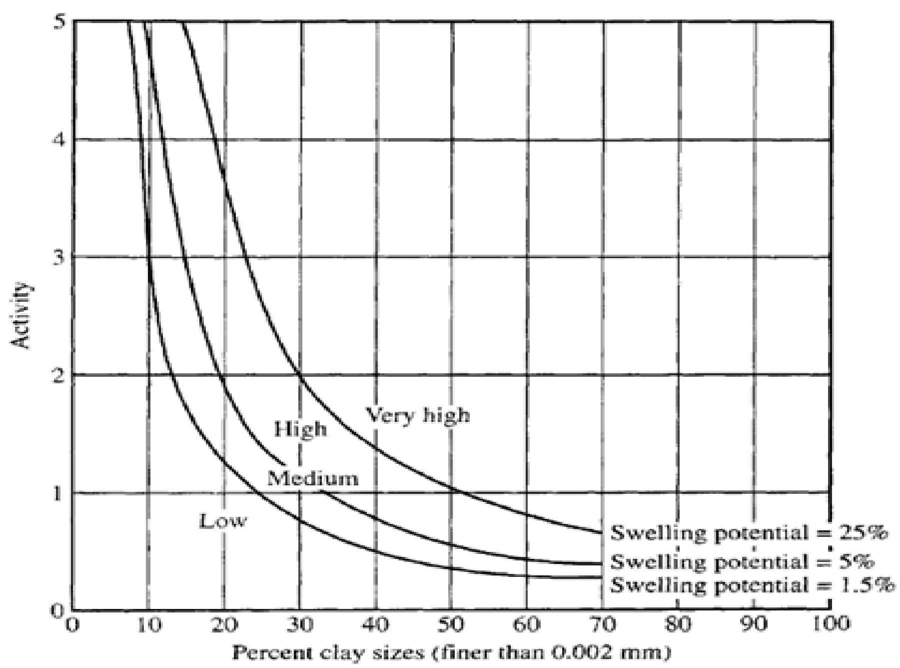

Figure 2. 11 Classification Chart for swelling potential Seed et al. [15]

Oedometer test for Swelling Soils (ASTM D 4546)

There are different available methods to determine swelling pressure of expansive soils. These are Different Pressure method, Swell-consolidtion method (Free swell method), Double Oedometer method and Constant Volume method. From these devised methods constant volume method is relatively easy to apply and gives reasonable swelling pressure over other methods [17].

The initial water content and void ratio should be representative of the in situ soil immediately prior to construction. In case where it is necessary to use disturbed soil samples, the soil sample should be compacted to the required field density and water content in a Proctor Compaction mould. The swell potential and swell pressure of a soil specimen can be determined by any of the three methods specified by ASTM Standards (ASTM D 4546-90-Method C, Standard Test Method for One Dimensional Swell or Settlement Potential of Cohesive Soils) [5].

A vertical pressure (σ'_v) equivalent to the estimated vertical in situ pressure or swell pressure is applied to the specimen. After completion of axial deformations under the vertical pressure (σ'_v), the specimen is inundated with water. Increments of vertical stress are applied to the wetted specimen to prevent any swell. Variations from the dial gage readings at the time the specimen is inundated at stress (P) shall be preferably kept within 0.005 mm and not more than 0.01 mm. The vertical pressure at which the wetted specimen shows no further tendency to swell defines the swell pressure of the specimen. The specimen is stepwise loaded following no further tendency to swell. The applied load increments should be sufficient to define the maximum curvature on the consolidation curve and to determine the slope of the virgin compression curve [5].

2.5.7. Mechanics of Swelling

If environment of an expansive soil has not been changed, swelling does not take place. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase due to moisture introduction. By far the most important element is the effect of water on expansive soils [16].

There must be a potential gradient, which can cause water migration, and a continuous passage through which water transfer can take place. With the introduction of water, volumetric expansion takes place. If pressure is applied to prevent expansion, the pressure required to maintain the initial volume is the swelling pressure. Thickness and location of potentially expansive layers in a profile considerably influence potential movement. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by non-expansive material or have got shallow depths. Water contents in the upper few meters of the expansive soil profile are affected by environmental factors and generally called zone of seasonal variation or the active zone. The mechanism of swelling is indicated in Figure 2.12 [16].

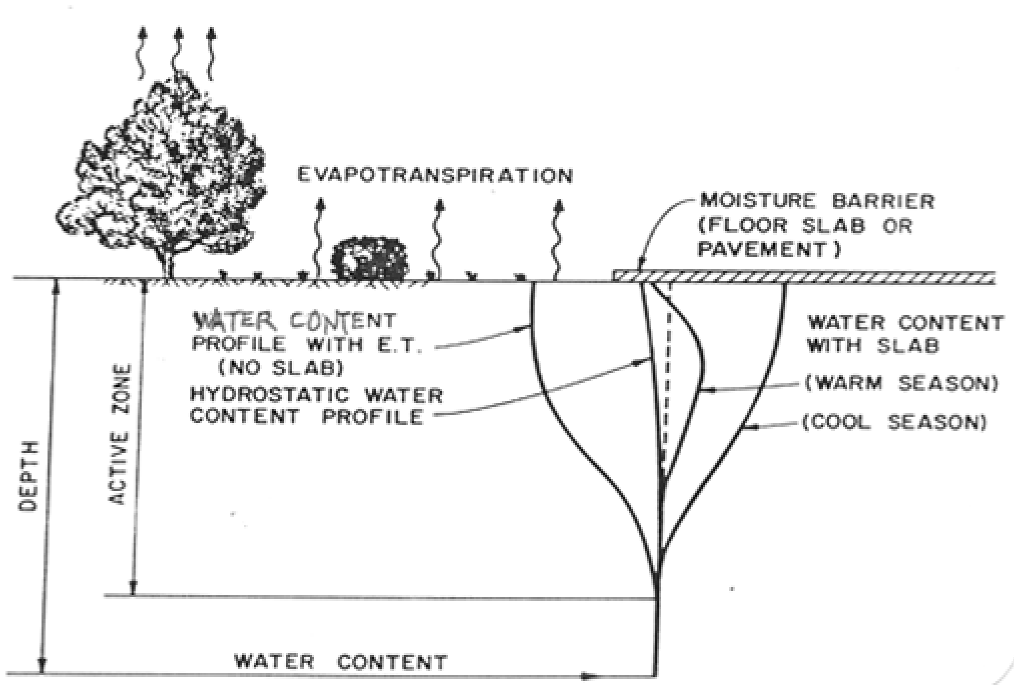

Figure 2. 12 Water content profiles in the active zone [16]

In fine-grained soils, capillary force is a significant means of water transfer. The height of water rise into the capillary fringe varies inversely with the radius of the capillary tube. In clean, coarse gravel, the capillary rise is insignificant. In clean sands, the rise is a few centimeters; in fine sands, the rise is 30 to 60 centimeters; in silt up to 3 to 4 meters; and in clay, a rise of more than 300 meters is theoretically possible [16].

It is well known that heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soils [16].

2.6 One Dimensional Consolidation of Soils

2.6.1 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. According to Terzaghi [18], “a decrease of water content of a saturated soil without replacement of the water by air is called a process of consolidation.”

When saturated clayey soils which have a low coefficient of permeability are subjected to a compressive stress due to a foundation loading, the pore water pressure will immediately increase; however, because of the low permeability of the soil, there will be a time lag between the application of load and the extrusion of the pore water and, thus, the settlement. This phenomenon is called consolidation [7].

2.6.2 The Standard One-Dimensional Consolidation Test

The 1-D test is used to obtain a compression parameter for the amount of settlement and the consolidation parameter c_v for the settlement rate estimate. The pre-consolidation pressure p_c and thus the OCR can also be determined from this test. The test is performed on an "undisturbed" soil sample that is placed in a consolidation ring available in diameters ranging from 45 to 115 mm. The sample height is between 20 and 30 mm; 20 mm is the most commonly used thickness to reduce test time [5].

Test Methods [5]

Controlled rate of strain (CRS) consolidation test: in this test, special equipment is used that can apply a very slow strain rate to develop the consolidation load. The load is applied as a constant strain but at a rate such that the excess pore pressure (measured at the sample base) is kept in a range of from 3 to 30 percent of the loading pressure developed by that strain rate.

Controlled stress test (CST): the standard consolidation test is a controlled stress test (CST), since a constant load increment is used for each stage of the test. A modified version of the test that applies the load through a controlled rate of strain (CRS) has been standardized as ASTM D 4186. And this test method is applied for this particular research work to conduct the consolidation test.

2.6.3 Coefficients of Compression

Compression parameters such as coefficients of compressibility (a_v), compression index (Cc), swelling index (Cs) and coefficient of volume compressibility (m_v) can be obtained from void ratio versus applied pressure curve of consolidation test [17].

2.6.4 Normally Consolidated and Over consolidated Clays

A clay is said to be normally consolidated if the present effective overburden pressure $\sigma'_0$is the maximum pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is said to be over consolidated if the layer was subjected at one time in its history to a greater effective overburden pressure, p'_c, than the present pressure, σ'_0. The ratio p'_c / σ'_0 is called the *over consolidation ratio* (OCR) [4].

2.6.5 Determination of the Coefficient of Consolidation and Coefficient of Permeability

The coefficient of consolidation c_v can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical.
There are two laboratory methods that are used in common for the determination of c_v. They are: Casagrande Logarithm of Time Fitting Method and Taylor Square Root of Time Fitting Method.

.Average vertical coefficient of permeability, *kv* can be estimated using the following formula [5].

$$k_v = c_v m_v \gamma_w \tag{2.8}$$

Where: c_v = vertical coefficient of consolidation
m_v= coefficient of volume compressibility
γ_w = unit weight of water

2.6.6 Correlation for Compression Index and Preconsolidation

A laboratory consolidation test takes a considerable amount of time and is both labor- and computation-intensive. Because of these factors a substantial effort has been undertaken to attempt to correlate the compression indexes to some other more easily determined soil index properties [4, 5].
A reliable estimate of the effective pre-consolidation pressure, p'_c is difficult without performing a consolidation test. There have been a few correlations given for p'_c of which one was given by Nagaraj and Srinivasa Murthy [5] for saturated soils preconsolidated by either overburden pressure or cementation and shrinkage.

2.7 Compaction of Soils

2.7.1 General

Compaction is pressing the soil particles close to each other by mechanical methods. During compaction air is expelled from the void space in the soil mass and, therefore, the mass density is increased. Action of a soil mass is done to improve its engineering properties. Compaction generally increases the shear strength, and hence the stability and bearing capacity. It is also useful in reducing the compressibility and permeability of the soil [5].

2.7.2 Factors Affecting Compaction

The increase in dry density of the soil achieved as a result of compaction depends upon the following factors. These are water content, compaction effort, soil type and method of compaction. And for lateritic soils method of sample manipulation and preparation can affect the compaction results [17].

2.7.3 Maximum Dry Density and Optimum Moisture Content

Dry unit weight

To assess the degree of compaction it is important to use the dry unit weight, γ_d, because we are interested in the weight of solid soil particles in a given volume, not the amount of solid, air and water in a given volume (which is the bulk unit weight) [17].

$$\gamma_d = \left[\frac{G_s \gamma_w}{G_s w + 1}\right] \quad \text{or} \quad \gamma_d = \frac{\gamma_{bulk}}{(1+w)} \tag{2.9}$$

Where:

γ_d= dry unit weigh of the soil sample
w = moisture content
Gs = specific gravity
γ_{bulk} = bulk unit weigh of the soil sample
γ_w = unit weigh of water

Optimum moisture content (OMC)

The optimum moisture content is the water content corresponding to maximum dry density which is obtained from dry density moisture content graph of compaction test.

2.8 Unconfined Shear Strength

2.8.1 General

The shear strength of soil is one of the most important aspects of geotechnical engineering. The bearing capacity of shallow and deep foundations, slope stability, retaining wall design and pavement design are all influenced by the shear strength of the soil. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads.
The shear strength is measured in terms of two soil parameters, cohesion or inter-particle attraction, and angle of internal friction, the resistance to inter particle slip. Grain crushing, resistance to rolling, and other factors are implicitly included in these two parameters. This behavior is well represented by the Mohr-Coulomb failure criterion [5].

2.8.2 Shear Strength of Cohesive soils

A characteristic of true clay is the property of cohesion, sometimes referred to as no load shear strength. Unconfined specimens of clay soil derive strength and firmness from cohesion as indicated in Figure 2.13.
The shear strength of saturated cohesive soil in undrained shear test (i.e. test in which change in volume is prevented) is derived entirely from cohesion. In such a case, the shearing strength is independent of magnitude of normal stress. However, in slow shear test, in which consolidation takes place, the shear strength of clay increases with normal stress [17].

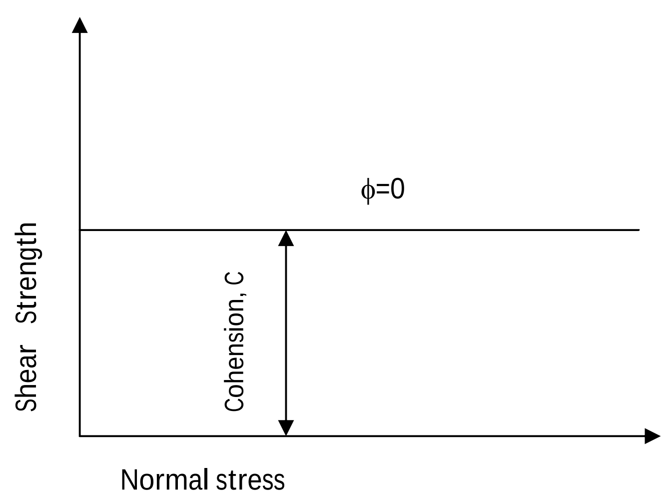

Figure 2. 13 Shear with no volume change (quick shear) [17]

It is well known that the shear strength of cohesive clay varies with its consistency. Clay which is at liquid limit has very little shear strength, whereas the same clay at lower moisture content may have considerable shear strength as indicated in Figure 2.14.

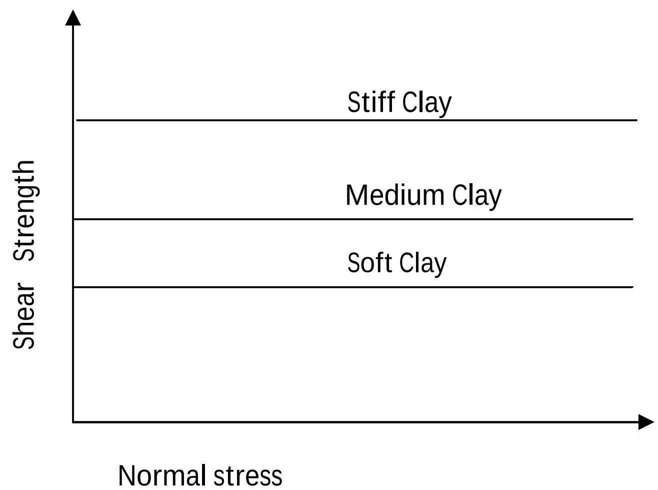

Figure 2. 14 Variation of cohesion with consistency [17]

The general relation between consistency and unconfined compression strength of clays is given in Table 2.8. From the table it can be seen that shear strength of clays is highly dependent on its consistency (i.e. natural water content). So in using this shear strength test result for design season where the site will have high water content should be taken into consideration.

Table 2. 8 Consistency and unconfined compression strength of clays [7]

Consistency	q_u (kN/m^2)
Very soft	0–24
Soft	24–48
Medium	48–96
Stiff	96–192
Very stiff	192–383
Hard	>383

2.8.3 Modulus of Deformation for UCS test

The modulus of a soil is one of the most difficult soil parameters to estimate because it depends on so many factors. Such factors are methods to calculate the modulus, the type of moduli, field conditions (water content, density, fabric of soil particles, preconsolidation state, cementation and etc.), loading process, and application of moduli [22].

Because soils do not exhibit a linear stress strain curve, many moduli can be defined from the triaxial test results (including unconfined compressive strength tests). The slope of the stress strain curve is not the modulus of the soil. However, the slope of the curve is related to the modulus and it is convenient to associate the slope of the stress strain curve to a modulus. If the slope is drawn from the origin to a point on the curve (Figure 2.15) the secant slope is obtained and the secant modulus $E_{s\,(s)}$ is calculated from it. One would use such a modulus for predicting the movement due to the first application of a load as in the case of a spread footing. If the slope is drawn as the tangent to the point considered on the stress strain curve then the tangent slope is obtained and the tangent modulus $E_{s\,(t)}$ is calculated from it. One would use such a modulus to calculate the incremental movement due to an incremental load as in the case of the movement due to one more story in a high-rise building [22].

It is often performed compression tests on soil samples, the data are always presented in terms of shear strength as the stress-strain relation. One can also use the relation to obtain stress-strain modulus or modulus of deformation, Es. Modulus of deformation is calculated from UCS (where lateral stresses are zero) stress-strain relation using the following formula [5].

$$E_s = q_u/\varepsilon \tag{2.10}$$

Where, q_u = unconfined compressive strength

ε = Axial strain

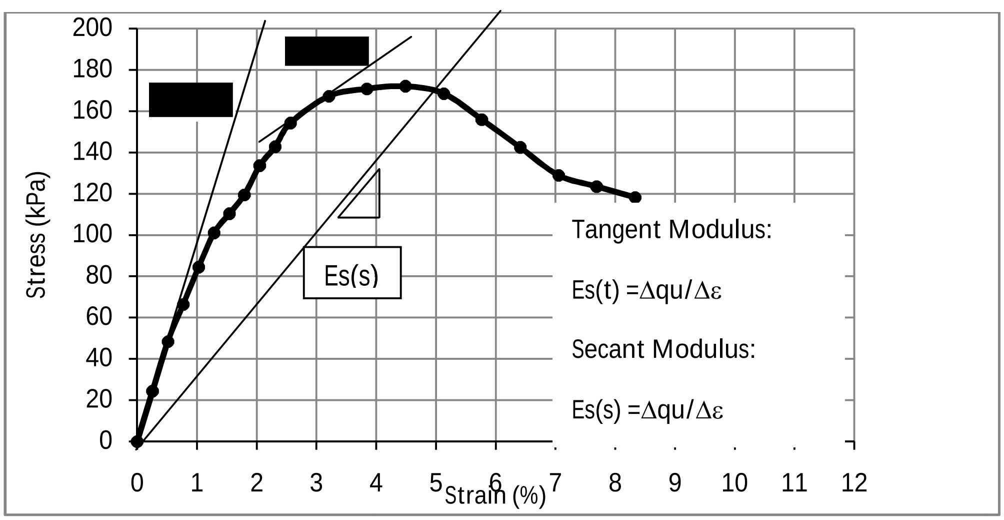

Figure 2. 15 Stress-strain curve for UCS test data to compute modulus of deformation *Es*

2.8.4 Sensitivity and Thixotropy of Clay Soils

Sensitivity

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

$$= \frac{\quad}{,} \qquad (2.11)$$

Where : q_u= unconfined compressive strength for undisturbed sample

$q_{u,r}$ = unconfined compressive strength for remolded sample

Table 2. 9 Soil Classification on the basis of sensitivity after Skempton and Northey [4]

Sensitivity (St)	Nature of Clay
1	Insensitive clays
1-2	Low-sensitive clays
2-4	Mediums clays
4-8	Sensitive clays
8-16	Extra-sensitive clays
>16	Quick clays

The clays that have sensitivity greater than 8 should be treated with care during construction operations because disturbance tends to transform them, at least temporarily, into viscous fluids. Such clays belong to the montimorillonite group and possess flocculent structure [4].

Thixotropy

Thixotropy is the regain of strength from the remolded state with time. All clays and other soils containing cementing agents exhibit thixotropic properties. When the strength gain is from pore-pressure dissipation, this is not thixotropy. However, piles driven into a soft clay deposit often have very little load-carrying capacity until a combination of aging/cementation (thixotropy) and dissipation of excess pore pressure (consolidation) occurs [5].

3 SAMPLE AREA DESCRIPTION

3.1 General

Jimma is the largest town in Southwestern Ethiopia located at latitude and longitude of $7^0$40’N and $36^0$50’E respectively in Oromiya National Regional State. It is 350km from Addis Ababa. The town has a rolling terrain with an elevation ranging from 1670m to 1770m above mean sea level.

Jimma is predominantly covered with red, black and gray soils. The red colored soils are found on rolling topography with higher elevation and well drainage condition. The black and gray soils, which cover the central and large part of the town, are found on flat topography of the town with lower elevation and unfavorable drainage condition.
The geographical location of the Town is shown on Figure 3.1.

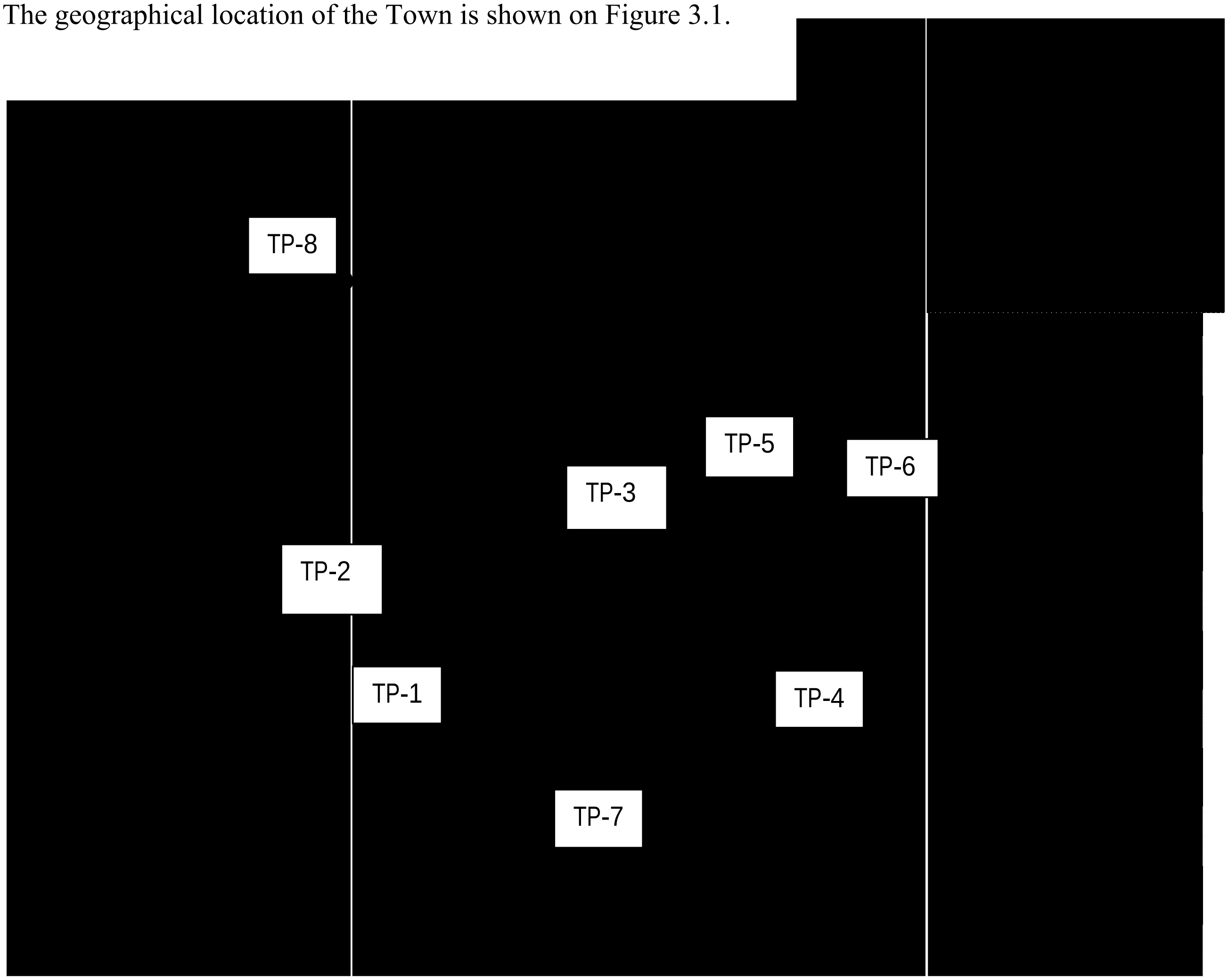

Figure 3. 1 Map of Jimma Town and location of Sample test-pits (Source: Google Map)

Samples of 16 disturbed and 14 undisturbed were collected from 8 test pits in one phase. The sampling location is selected so that it can be well represent soils found in Jimma town. Selection of these sampling locations also considers visual soil classification, economical importance of sampling area, non-uniformity of the sample locations and coverage of the section. During excavation for sampling, it was observed that the ground water table is found far below 2.5m depth. The sampling locations and their designations are presented in Table 3.1.

Table 3. 1 Sampling location and designation of sample test pits

Ser.No.	Location	Test Pit Designation	Elevation (m)	Description of Terrain	Colour Description
1	Merkato-1	TP-1	1716.28	Rolling	Red
2	Merkato-2 (Taxi Office)	TP-2	1719.63	Flat	Gray
3	Awetu	TP-3	1722.60	Rolling	Red
4	Technic sefer	TP-4	1713.85	Flat	Gray
5	Kebele-5	TP-5	1725.52	Flat	Black
6	Jimma university	TP-6	1748.40	Rolling	Red
7	Bacho Bore	TP-7	1714.94	Flat	Gray
8	Agri sefer	TP-8	1759.50	Rolling	Red

3.2 Climate

The climatic classification of Jimma Town is classified as “Wayna Dega” with a mean temperature of 23^{o}C. It has mean annual rainfall of 1500mm with maximum rainfalls from June to September. The area has a maximum temperature of 29^{o}C and a minimum temperature of 17^{o}C. According to the Meteorological data the mean monthly rainfall, mean daily maximum and minimum temperature and mean number of rain days are presented in Table 3.2.

Table 3. 2 Meteorological data of Jimma Town [2]

Month	Mean Temperature °C		Mean Total Rainfall (mm)	Mean Number of Rain Days
	Daily Minimum	Daily Maximum		
Jan	18	28	40	5
Feb	19	29	55	10
Mar	20	28	95	14
Apr	20	27	140	15
May	20	26	160	18
Jun	19	24	215	24
Jul	19	24	215	25
Aug	19	25	215	25
Sep	19	27	185	22
Oct	19	27	90	18
Nov	18	28	55	10
Dec	17	28	35	5

3.3 Geology

The main geologic formation of Jimma town is the Cenozoic tertiary volcanic rock of Nazareth Series and Jimma Volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the Trap series formation. Tuft and Alluvials are found in few amounts at different localities [19].

3.4 Soil Characteristics

Tropical Residual fine grained soils, like clays and silt-clays, developed mainly on basaltic bedrock represent the soils found in Jimma town. These soils are of two main types. The first type is red clay soil the color of which is the result of reduction of magnetic minerals. In flat lying location, massive dark silty clay soils (alluvial origin) formation have color ranging from gray to dark black. Even though most of the town is covered with soils colored from dark to gray clay soils, there are also red and yellow colored soils [11].

4 FIELD PROPERTIES AND LABORATORY TEST RESULTS

4.1 General

4.1.1 Visual description

The samples obtained from various locations their physical properties are different. The samples from flat area their color is vary from gray to black whereas from the sloped area it varies from brownish red to red. They have no odor at all. They are in moist and wet condition at the time of sample recovery. Their consistency varies from very soft to firm to hard from field consistency test method. The dry strength test shows that the soils strength varies from medium to very high. Diletancy test shows that the soils have none to slow reaction for shaking test. Their toughness ranges from medium to high under dry state. The detail result for each test pits and depths is presented under section 4.9 and it is summarized in Table 4.12.

4.1.2 Natural moisture content

The oven temperature 110ºc for water content determination is too high for certain clays of tropical residual soils such as lateritic soils. These soils contain loosely bound water of hydration or molecular water which can be lost at this high temperature, resulting in a change of the soil characteristics [5].

Moisture contents of the soil samples were determined in the laboratory according to ASTM D2216-92. Two set of samples were dried to a constant weight using oven dry at temperature of 105ºC and 50ºC and maximum relative humidity of 30% were used. The values of the moisture content variations are compared and summarized in Table 4.1.

As mentioned in section 2.4.6 moisture variations of 4 – 6 % or more indicates that loosely bound molecular water is present for residual lateritic soils. From the test results, one can see that the differences in moisture contents for some samples at 105ºc and 50ºc and 30% RH under consideration are between 4% -6%.

Table 4. 1 Natural moisture content and comparison for different testing temperatures.

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Colour description	Test conditions and natural moisture content, NMC (%)		Moisture content difference (%)
					OD[1]	AD[2]	
1	Merkato-1	Tp-1-1	1.5	Brownish Red	50.75	43.84	6.90
2		Tp-1-2	2.5	Gray	49.76	47.09	2.67
3	Merkato-2	Tp-2-1	1.5	Gray	49.70	44.24	5.46
4		Tp-2-2	2.5	Gray	51.21	47.09	4.12
5	Awetu	Tp-3-1	1.5	Red	42.26	36.98	5.28
6		Tp-3-2	2.5	Red	46.41	40.57	5.85
7	Teknik Sefer	Tp-4-1	1.5	Gray	40.88	38.83	2.05
8		Tp-4-2	2.5	Gray	47.83	44.34	3.49
9	Kebele-5	Tp-5-1	1.5	Black	65.58	60.67	4.91
10		Tp-5-2	2.5	Black	70.00	65.58	4.42
11	Jimma University	Tp-6-1	1.5	Red	41.67	37.10	4.57
12		Tp-6-2	2.5	Yellowish Red	46.55	42.70	3.86
13	Bacho Bore	Tp-7-1	1.5	Black	40.00	35.70	4.30
14		Tp-7-2	2.5	Gray	50.50	46.00	4.50
15	Agri Sefer	Tp-8-1	1.5	Red	42.50	37.10	5.40
16		Tp-8-2	2.5	Red	51.50	47.00	4.50

[1] OD is oven dried sample at a temperature of 105^{o}C

[2] AD is oven dried sample at a temperature of 50^{o}C and maximum Relative humidity, RH =30% [11].

4.2 Specific Gravity

In residual soils the specific gravity may be unusually high or low. It is thus essential that the specific gravity be determined in the laboratory using an accepted standard test procedure. The test was conducted according to ASTM D854-58, Standard Test for Specific Gravity of Soil Solids by Water Pycnometer, procedure. For test pits such as TP-3, TP-6 and TP-8 their natural moisture content were used since pretest drying of the sample tends to reduce the measured specific gravity as compared to the one with its natural moisture content [11]. The test result was summarized in Table 4.2.

Table 4. 2 Specific gravity test results

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Specific gravity, Gs
1	Merkato-1	Tp-1-1	1.5	2.71
2		Tp-1-2	2.5	2.69
3	Merkato-2	Tp-2-1	1.5	2.72
4		Tp-2-2	2.5	2.67
5	Awetu	Tp-3-1	1.5	2.75
6		Tp-3-2	2.5	2.81
7	Technic Sefer	Tp-4-1	1.5	2.64
8		Tp-4-2	2.5	2.58
9	Kebele-5	Tp-5-1	1.5	2.72
10		Tp-5-2	2.5	2.68
11	Jimma University	Tp-6-1	1.5	2.68
12		Tp-6-2	2.5	2.65
13	Bacho Bore	Tp-7-1	1.5	2.61
14		Tp-7-2	2.5	2.59
15	Agri Sefer	Tp-8-1	1.5	2.76
16		Tp-8-2	2.5	2.79

4.3 Grain Size Analysis

In this study wet sample preparation in accordance with ASTM D2217-85was applied. Mechanical analysis was used for the coarse sized soils by using set of sieve and whereas hydrometer analysis was used for fine grained soils. Here sodium hexametaphosphate is used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods are performed. Air dried samples were used for TP-3, TP-6 and TP-8 in order to avoid conglomeration of particles as a result of oven drying effect whereas oven dried samples were used for the rest of the test pits.

The sieve analysis results and the summarized combined grain size analysis curves are presented in Table 4.3 and Figure 4.1 respectively. The gradation curve for tests undertaken at depths 1.5m and 2.5m for each test pit are presented in Appendix-A.

Table 4. 3 Grain size analysis test results

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Percentage Amount of Particle Size (%)				% finer than 0.075mm
				Gravel	Sand	Silt	Clay	
1	Merkato-1	Tp-1-1	1.5	2	4	32	62	95
2		Tp-1-2	2.5	5	10	52	33	85
3	Merkato-2	Tp-2-1	1.5	0	5	43	52	95
4		Tp-2-2	2.5	2	7	51	40	91
5	Awetu	Tp-3-1	1.5	1	3	25	71	96
6		Tp-3-2	2.5	1	1	20	78	98
7	Technic Sefer	Tp-4-1	1.5	1	2	46	51	97
8		Tp-4-2	2.5	1	4	49	46	95
9	Kebele-5	Tp-5-1	1.5	0	1	40	59	99
10		Tp-5-2	2.5	0	1	41	57	99
11	Jimma University	Tp-6-1	1.5	1	6	57	36	93
12		Tp-6-2	2.5	14	25	42	19	61
13	Bacho Bore	Tp-7-1	1.5	0	4	43	53	95
14		Tp-7-2	2.5	2	6	50	42	92
15	Agri sefer	Tp-8-1	1.5	0	1	19	80	98
16		Tp-8-2	2.5	1	2	21	76	97

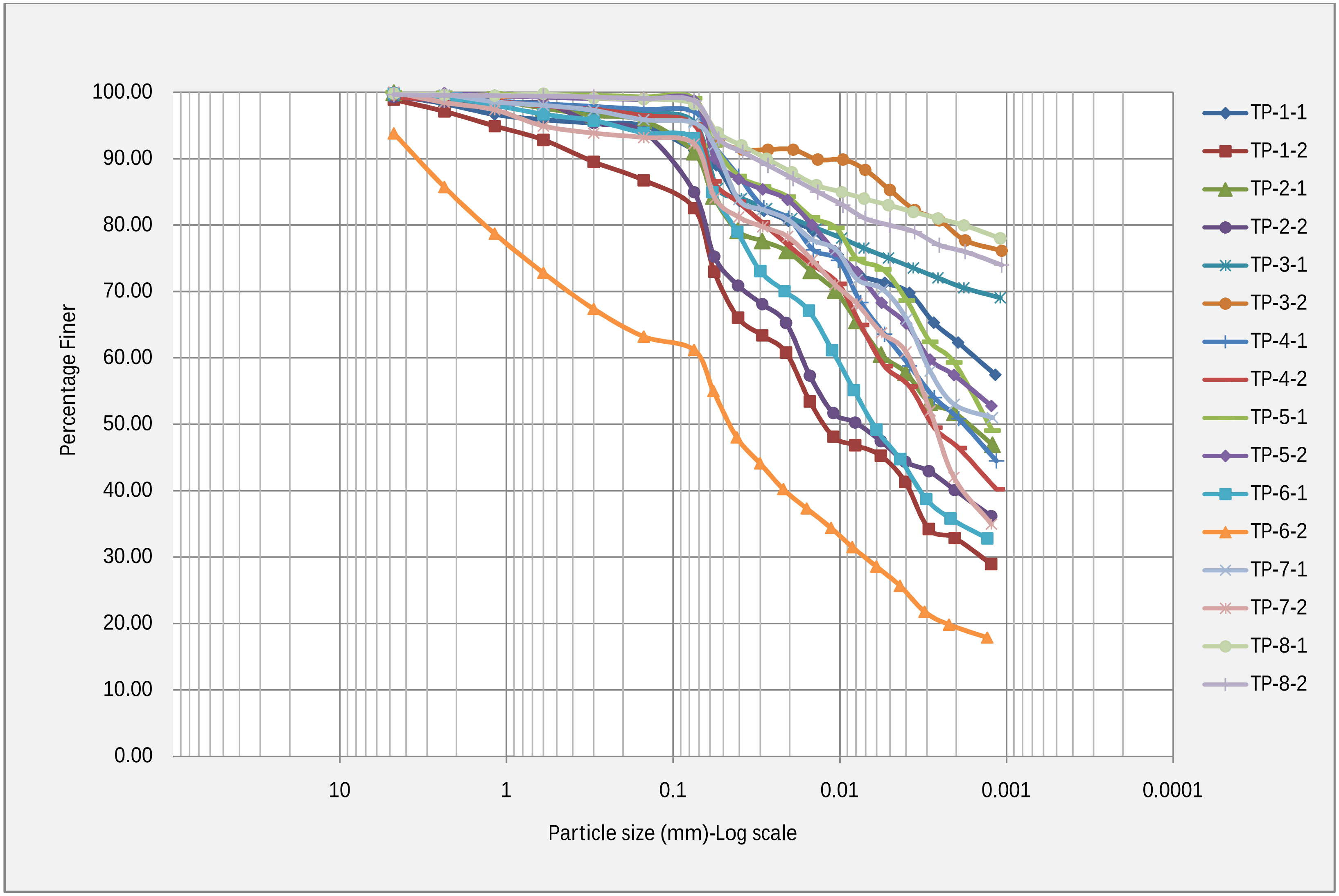

Figure 4. 1 Summary of combined grain size distribution curves from sieve and hydrometer analysis for 8 test pits at 1.5 and 2.5m depths

4.4 Atterberg Limits

Residual soil may change their property irreversibly upon drying. So, for presumed lateritic soils, to reduce pretest drying effect on the sample wet preparation was applied on the received samples (moist samples). The portions of the samples passing the No. 40(0.425mm) sieve were used for the preparation of the sample for this study as per the ASTM D4318-00, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

4.4.1 Conventional Test Procedures

Atterberg Limits were determined for air-dried (AD as per the ASTM D4318-00). The air-dried samples were prepared by spreading the specimen in the air for about 5-7 days. The temperature was about 25^{o}c. The test result is summarized in Table 4.4, while the detailed investigation is presented in Appendix-B.

Table 4. 4 Atterberg limit test results of Jimma Soils

Ser.No	Location	Test Pit designation	Depth of sampling (m)	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic index, PI (%)
1	Merkato -1	TP-1-1	1.5	85	35	50
2		TP-1-2	2.5	65	30	35
3	Merkato-2	TP-2-1	1.5	86	34	52
4		TP-2-2	2.5	72	32	39
5	Awetu	TP-3-1	1.5	74	27	46
6		TP-3-2	2.5	77	32	45
7	Teknik Sefer	TP-4-1	1.5	104	40	63
8		TP-4-2	2.5	92	35	57
9	Kebele-5	TP-5-1	1.5	97	38	59
10		TP-5-2	2.5	108	40	68
11	Jimma University	TP-6-1	1.5	64	35	28
12		TP-6-2	2.5	53	31	22
13	Bacho Bore	TP-7-1	1.5	101	39	62
14		TP-7-2	2.5	95	35	60
15	Agri sefer	TP-8-1	1.5	77	39	38
16		TP-8-2	2.5	80	41	39

4.4.2 Effect of Pre-treatment of the samples on Atterberg Limits

For the expected laterite soils that are recovered from three locations as indicated in Table 4.5, the effect of temperature on the Atterberg limit tests were investigated on samples that were oven dried (OD) and air-dried (AD). The test results are presented in Table 4.5.

Table 4. 5 Atterberg limit test results at different drying conditions

Ser. No	Location	Test Pit designation	Depth of sampling (m)	Test condition[3]	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic index , PI(%)	Change in LL(%)
1	Awetu	TP-3-1	1.5	AD	74	27	46	
				OD	63	21	42	11
2		TP-3-2	2.5	AD	77	32	45	
				OD	69	26	44	7
3	Jimma University	TP-6-1	1.5	AD	64	35	28	
				OD	58	31	28	5
4		TP-6-2	2.5	AD	53	31	22	
				OD	49	29	21	4
5	Agri sefer	TP-8-1	1.5	AD	77	39	38	
				OD	71	35	37	5
6		TP-8-2	2.5	AD	80	41	39	
				OD	75	37	34	5

[3] For both AD and OD soaked samples are used to enable uniform distribution of moisture throughout the sample.

4.4.3 Effect of Test Procedures on Atterberg Limits

As per the recommendations in section 2.4.6, five soil samples for each specimen were mixed with water to give the ranges of moisture contents. The soil specimens were left overnight for moisture equilibration, the next day the Atterberg Limits were conducted by mixing the specimens for not more than 5 minutes. The portion of these specimens was continuously mixed for 30 minutes before conducting the Atterberg limits. The 5 minutes and 30 minutes Atterberg limits mixing times were conducted. The difference of greater that 5% indicates that, there is a disaggregation of the clay sized particles up on manipulation [11]. The more the soil's structure is handled and disturbed, the finer the aggregates become in grading and the higher the Atterberg limit. The summarized test results are presented in Table 4.6.

For lateritic soils, the test should be conducted using the following instructions. These are: to limit the mixing times for Atterberg limit tests not more than 5 minutes and to use fresh material for each moisture content points for compaction tests [11].

Table 4. 6 Atterberg limit test results

Ser. No.	Location	Test Pit designation	Depth of sampling (m)	Mixing time (minute)	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic index, PI (%)	Change in LL (%) (for 5minute - 30minute)
1	Awetu	TP-3-1	1.5	5	71.5	33.3	38.1	
				30	76.7	35.4	44.3	5.2
2		TP-3-2	2.5	5	72.0	30.9	41.1	
				30	76.6	33.2	43.8	4.6
3	Jimma University	TP-6-1	1.5	5	60.9	31.3	29.6	
				30	66.6	33.3	33.3	5.7
4		TP-6-2	2.5	5	49.2	31.3	18.0	
				30	57.4	33.3	24.1	8.2
5	Agri sefer	TP-8-1	1.5	5	74.3	36.3	38.0	
				30	82.0	40.5	41.4	7.7
6		TP-8-2	2.5	5	76.3	38.3	38.0	
				30	83.1	40.7	42.5	6.8

4.5 Plasticity, Liquidity and Consistency Indices

All Plastic, liquidity and consistency indices can be obtained directly from liquid limit, plastic limit and natural moisture content of the soil. And these indices are calculated using the relations under section 2.3.5. Plasticity index PI indicates the degree of plasticity of a soil. The greater the difference between liquid and plastic limits, the greater is the plasticity of the soil. Cohesionless soil has zero plasticity index. Fat clays are highly plastic and possess a high plasticity index. A summarized test results are shown in Table 4.7.

Table 4. 7 Plasticity, Liquidity and Consistency indices

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Plastic index, PI (%)	Liquidity index, LI	Consistency index, CI	Description [4]
1	Merkato-1	Tp-1-1	1.5	50	0.17	0.83	Higly plastic/ Stiff/firm
2		Tp-1-2	2.5	37	0.47	0.53	" "
3	Merkato-2	Tp-2-1	1.5	52	0.20	0.80	Higly plastic/ Stiff/firm
4		Tp-2-2	2.5	39	0.38	0.62	" "
5	Awetu	Tp-3-1	1.5	38	0.25	0. 75	Higly plastic/ Stiff/firm
6		Tp-3-2	2.5	41	0.21	0.79	" "
7	Teknik Sefer	Tp-4-1	1.5	63	-0.02	1.02	Higly plastic/ Stiff/firm
8		Tp-4-2	2.5	57	0.16	0.84	" "
9	Kebele-5	Tp-5-1	1.5	59	0.43	0.57	Higly plastic/ Stiff/firm
10		Tp-5-2	2.5	68	0.38	0.62	Higly plastic/ " "
11	Jimma University	Tp-6-1	1.5	30	0.06	0.94	Higly plastic/ Stiff/firm
12		Tp-6-2	2.5	18	0.63	0. 37	" "
13	Bacho Bore	Tp-7-1	1.5	62	-0.06	1.06	Higly plastic/ Stiff/firm
14		Tp-7-2	2.5	60	0.19	0.81	" "
15	Agri Sefer	Tp-8-1	1.5	38	-0.05	1.08	Higly plastic/ Stiff/firm
16		Tp-8-2	2.5	39	0.15	0.85	" "

4.6 Activity of Jimma Soils

Skempton's colloidal activity is determined as the ratio of the plasticity index of the clay content to fines. He observed that, for a given soil, the plasticity index is directly proportional to the percent of clay-size fraction (i.e., percent by weight finer than 0.002 mm in size) [4]. Activity designated by "A_c" is defined as in section 2.3.6. The summarized result is presented in Table 4.8.

Table 4. 8 Degree of colloidal activity of Jimma soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Activity	Designation
1	Merkato-1	Tp-1-1	1.5	0.80	Normal
2		Tp-1-2	2.5	1.07	Normal
3	Merkato-2	Tp-2-1	1.5	1.00	Normal
4		Tp-2-2	2.5	0.98	Normal
5	Awetu	Tp-3-1	1.5	0.66	Inactive
6		Tp-3-2	2.5	0.58	Inactive
7	Teknik Sefer	Tp-4-1	1.5	1.27	Active
8		Tp-4-2	2.5	1.24	Active
9	Kebele-5	Tp-5-1	1.5	0.91	Normal
10		Tp-5-2	2.5	1.29	Active
11	Jimma University	Tp-6-1	1.5	0.78	Normal
12		Tp-6-2	2.5	1.16	Normal
13	Bacho Bore	Tp-7-1	1.5	1.17	Normal
14		Tp-7-2	2.5	1.43	Active
15	Agri Sefer	Tp-8-1	1.5	0.55	Inactive
16		Tp-8-2	2.5	0.59	Inactive

4.7 Geochemical Tests

Geochemical (oxide) tests are carried out to know quantitatively main oxides of the soil material. Almost all soils on earth contain some amount of colloidal oxides and hydroxides. The oxides and hydroxides of aluminium, iron and silicon are of greatest interest since they are the ones most frequently encountered. Iron and aluminium oxides coat mineral particles, or cement particles of soils together. They may also occur as distinct crystalline units, such as hematite, gibbsite and magnetite [20].

Geochemical tests were carried out at Geological Survey of Ethiopia Geochemical Laboratory, Addis Ababa. To obtain the percentage oxide composition of the soils under investigation Atomic Absorption Spectrometer and Colorometer Analysis methods were used. The degree of laterization of the soil samples can be evaluated based on silica -sesquioxides (s-s) ratio as detailed in section 2.3. The sesquioxide, designated as R2O3, is the combination of aluminium oxide (Al2O3) and Iron oxide (Fe2O3). The chemical formula SiO2 designates the silica. Accordingly unlaterized soils have s-s ratio greater than 2. For lateritic soils S-S ratio lie between 1.33 and 2 where as for true laterites the ratio is less than 1.33.

Lateritic soil has not under gone a considerable degree of laterization as compared to true laterite. True laterites are simply referred as laterites. The soil of such kind are highly laterized i.e., sesquioxides content are high. Mineralogy is the primary factor controlling the size, shape, and physical and chemical properties of soil mechanics. The most widely used technique to determine mineralogical composition is x-ray diffraction (XRD) [20].

However, for this study geochemical test is preferred over the other types of chemical tests. Because the geochemical test helps to identify the oxide compositions found in the soil and which is eventually used to compare silica versus sesquioxides ratio and to discover the behavior of lateritic nature of the soils.
Based on color criteria and soil classification, three representative samples (from TP-3-2, TP-5-2 and TP-6-1) are selected. One from red clay, one from black clay and one from silt soil.

The basic test methods employed during the conducting of geochemical test to obtain the percentage of oxide composition are: Absorption Spectrometer and Colorometer Analysis .The test results are shown in Table 4.9.

Table 4. 9 Oxide composition in percent for different test pits of Jimma soil

Ser.No	Location	Test Pit Designation	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI	Silica-Sesquioxide ratio(S/S)
1	Awetu	TP-3-2	42.32	23.77	14.12	0.56	0.64	<0.01	0.66	0.36	0.08	1.81	4.99	10.58	1.12
2	Kebele-5	TP-5-2	52.5	17.14	10.44	0.42	0.58	<0.01	0.62	0.04	0.06	1.78	7.41	8.21	1.90
3	Jimma University	TP-6-1	52.5	12.73	16.64	<0.01	0.28	<0.01	0.64	0.26	0.29	2.53	3.83	9.33	1.79

4.8 Test Results of Expansive Soils

4.8.1 Free Swell Test

According to Holtz and Gibbs [16], the free swell test is conducted by placing 10gm of dry soil passing the No 40 sieve into a graduated cylinder one filled with water and another with kerosene to 100 ml. Swelled volume of the soil in water is measured after 24 hours and the volume of soil in kerosene gives initial volume of the samples. Hence the free swell (FS) is computed as:

$$FS = \frac{V_f - V_o}{V_o} x100\% \tag{4.1}$$

Where:

FS = free swell in%

V_f = final volume of the soil in water

V_o = volume of soil in kerosene.

The Test results are presented in table 4.10

Table 4. 10 Free swell test results of Jimma soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Free Swell (%)	Remark
1	Merkato-1	Tp-1-1	1.5	50	Brownish Red
2		Tp-1-2	2.5	60	Yellowish Gray
3	Merkato-2	Tp-2-1	1.5	105	Gray
4		Tp-2-2	2.5	80	Gray
5	Awetu	Tp-3-1	1.5	50	Red
6		Tp-3-2	2.5	50	Red
7	Teknik Sefer	Tp-4-1	1.5	135	Light Gray
8		Tp-4-2	2.5	110	Gray
9	Kebele-5	TP-5-1	1.5	130	Black
10		TP-5-2	2.5	160	Black
11	Jimma University	TP-6-1	1.5	50	Red
12		TP-6-2	2.5	40	Yellowish Red
13	Bacho Bore	Tp-7-1	1.5	110	Black
14		Tp-7-2	2.5	100	Gray
15	Agri Sefer	Tp-8-1	1.5	50	Red
16		Tp-8-2	2.5	45	Red

4.8.2 Determination of Swelling Pressure

ASTM defines swelling pressure which prevents the specimen from swelling or that pressure which is required to return the specimen to its original state (void ratio, height) after swelling. Essentially, the method used for measuring swelling pressure is stress controlled method [5].

There are different methods to determine the swelling pressure of expansive soils. These are Different Pressure Method, Consolidation-swell Method, Constant volume Method and Double Oedometer Method. From these methods constant volume method is relatively easy and gives a reasonable swelling pressure result [17]. A test procedure that applied for this particular laboratory test is ASTM D4546-C as per ASTM D 2435 procedure which is *constant volume test or swell pressure test procedure*.

Stress controlled tests use the conventional Oedometer. The samples are placed in the consolidation ring trimmed to the height of the ring. The sample is maintained at constant height (volume) by adjustments in vertical pressure after the specimen is inundated in free water to obtain swell pressure. The stress required to maintain the sample to its original height is the zero volume change or swelling pressure. The test results of three locations are presented in Table 4.11.

Table 4. 11 Swelling pressure test results for Jimma expansive soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Natural moisture content, NMC (%)	Swelling pressure (kPa)
1	Merkato-2	TP-2-2	2.5	45.51	135
2	Technic Sefer	TP-4-2	2.5	40.52	210
3	Kebele-5	TP-5-2	2.5	61.20	180

4.9 Classification of Jimma Soils

Both field classification methods and laboratory classification systems are employed. Different field tests have been conducted during sampling process.

4.9.1 Conventional Classification System

These classifications are summarized in Table 4.12 to Table 4.15 and Figure 4.2 and Figure 4.3.

Table 4. 12 Physical descriptions and visual classification of Jimma soils [6]

Ser. No	Location	Test Pit designat -ion	Depth of sampling (m)	Color	Odor	Moisture condition	Consistency	Dry strength	Dilatancy	Toughness	Remarks
1	Merkato-1	Tp-1-1	1.5	Brownish Red	No	Moist	Firm	High	Slow	High	Clay with high plasticity
2		Tp-1-2	2.5	Yellowish Gray	No	Moist	Firm	Medium	Slow	Medium	Clay with high plasticity
3	Merkato-2	Tp-2-1	1.5	Gray	No	Wet	Very Soft	Medium	None	High	Clay with high plasticity
4		Tp-2-2	2.5	Gray	No	Wet	Soft	Medium	Slow	High	Clay with high plasticity
5	Awetu	Tp-3-1	1.5	Red	No	Moist	Firm	High	None	High	Clay with high plasticity
6		Tp-3-2	2.5	Red	No	Moist	Firm	Medium	None	High	Clay with high plasticity
7	Technic Sefer	Tp-4-1	1.5	Light Gray	No	Moist	Soft	Very High	None	High	Clay with high plasticity
8		Tp-4-2	2.5	Gray	No	Moist	Firm	High	None	High	Clay with high plasticity
9	Kebele-5	TP-5-1	1.5	Black	No	Wet	Very Soft	Very High	None	High	Clay with high plasticity
10		TP-5-2	2.5	Black	No	Wet	Soft	Very High	None	High	Clay with high plasticity
11	Jimma University	TP-6-1	1.5	Red	No	Moist	Hard	Medium	Slow	Medium	High plastic silt soil
12		TP-6-2	2.5	Yellowish Red	No	Moist	Hard	Medium	Slow	Medium	High plastic silt soil
13	Bacho Bore	Tp-7-1	1.5	Black	No	wet	Soft	High	None	High	Clay with high plasticity
14		Tp-7-2	2.5	Gray	No	Moist	Firm	High	Slow	High	Clay with high plasticity
15	Agri Sefer	Tp-3-1	1.5	Red	No	Moist	Firm	High	None	High	Clay with high plasticity
16		Tp-3-2	2.5	Red	No	Moist	Firm	Medium	None	High	Clay with high plasticity

Table 4. 13 Textural classification system of Jimma soils - Appendix Graph C-2[6]

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Percentage Amount of Particle Size (%)				Textural Classification System
				Gravel	Sand	Silt	Clay	
1	Merkato-1	Tp-1-1	1.5	2	4	30	65	Clay
2		Tp-1-2	2.5	5	10	46	39	Silty Clay
3	Merkato-2	Tp-2-1	1.5	0	4	43	52	Clay
4		Tp-2-2	2.5	2	7	49	42	Silty Clay
5	Awetu	Tp-3-1	1.5	1	3	22	74	Clay
6		Tp-3-2	2.5	1	1	19	79	Clay
7	Teknik Sefer	Tp-4-1	1.5	1	2	45	52	Clay
8		Tp-4-2	2.5	1	4	46	49	Silty Clay
9	Kebele-5	TP-5-1	1.5	0	1	39	60	Clay
10		TP-5-2	2.5	0	1	41	58	Clay
11	Jimma University	TP-6-1	1.5	1	6	55	38	Silty Clay
12		TP-6-2	2.5	14	25	31	30	Clayey Silt
13	Bacho Bore	TP-7-1	1.5	0	4	42	53	Clay
14		TP-7-2	2.5	2	6	50	42	Silty Clay
15	Agri Sefer	TP-8-1	1.5	0	1	18	80	Clay
16		TP-8-2	2.5	1	2	21	76	Clay

Table 4. 14 Classification of Jimma soils using USCS procedure [5]

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Liquid limit, LL (%)	Plastic index, PI (%)	Unified Soil Classification System (USCS)
1	Merkato-1	Tp-1-1	1.5	85	50	CH
2		Tp-1-2	2.5	65	35	CH
3	Merkato-2	Tp-2-1	1.5	86	52	CH
4		Tp-2-2	2.5	72	39	CH
5	Awetu	Tp-3-1	1.5	72	38	CH
6		Tp-3-2	2.5	72	41	CH
7	Teknik Sefer	Tp-4-1	1.5	104	63	CH
8		Tp-4-2	2.5	92	57	CH
9	Kebele-5	TP-5-1	1.5	91	54	CH
10		TP-5-2	2.5	108	68	CH
11	Jimma University	TP-6-1	1.5	61	30	MH
12		TP-6-2	2.5	49	18	MH
13	Bacho Bore	TP-7-1	1.5	101	39	CH
14		TP-7-2	2.5	95	35	CH
15	Agri Sefer	TP-8-1	1.5	74	38	CH
16		TP-8-2	2.5	76	39	CH

Where: CH=Inorganic clay with high plasticity (Fat clay),

MH= Silt with high plasticity [6].

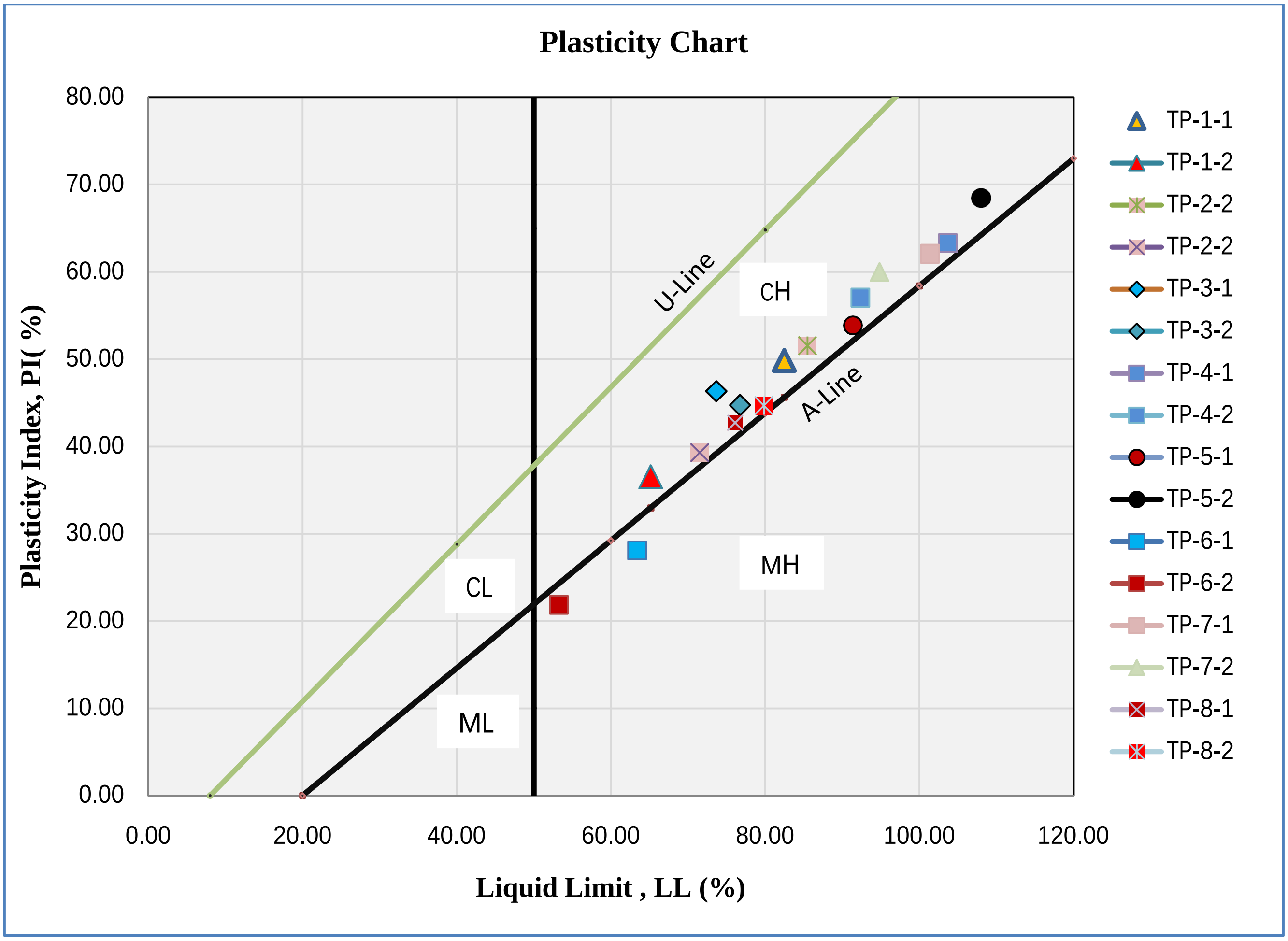

Figure 4. 2 Classification of the fine-grained Jimma soils using USCS plasticity chart

Table 4. 15 Classification using AASHTO procedure for Jimma Soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Liqid limit, LL (%)	Plastic index, PI (%)	AASHTO Classification	Remark
1	Merkato-1	Tp-1-1	1.5	85	50	A-7-5	Clayey soils
2		Tp-1-2	2.5	65	35	A-7-6	Clayey soils
3	Merkato-2	Tp-2-1	1.5	86	52	A-7-5	Clayey soils
4		Tp-2-2	2.5	72	39	A-7-5	Clayey soils
5	Awetu	Tp-3-1	1.5	72	38	A-7-6	Clayey soils
6		Tp-3-2	2.5	72	41	A-7-5	Clayey soils
7	Teknik Sefer	Tp-4-1	1.5	104	63	A-7-5	Clayey soils
8		Tp-4-2	2.5	92	57	A-7-5	Clayey soils
9	Kebele-5	TP-5-1	1.5	97	59	A-7-5	Clayey soils
10		TP-5-2	2.5	108	68	A-7-5	Clayey soils
11	Jimma University	TP-6-1	1.5	61	30	A-7-5	Clayey soils
12		TP-6-2	2.5	49	18	A-7-5	Clayey soils
13	Bacho Bore	Tp-7-1	1.5	101	62	A-7-5	Clayey soils
14		Tp-7-2	2.5	95	60	A-7-5	Clayey soils
15	Agri Sefer	Tp-8-1	1.5	74	38	A-7-5	Clayey soils
16		Tp-8-2	2.5	76	39	A-7-5	Clayey soils

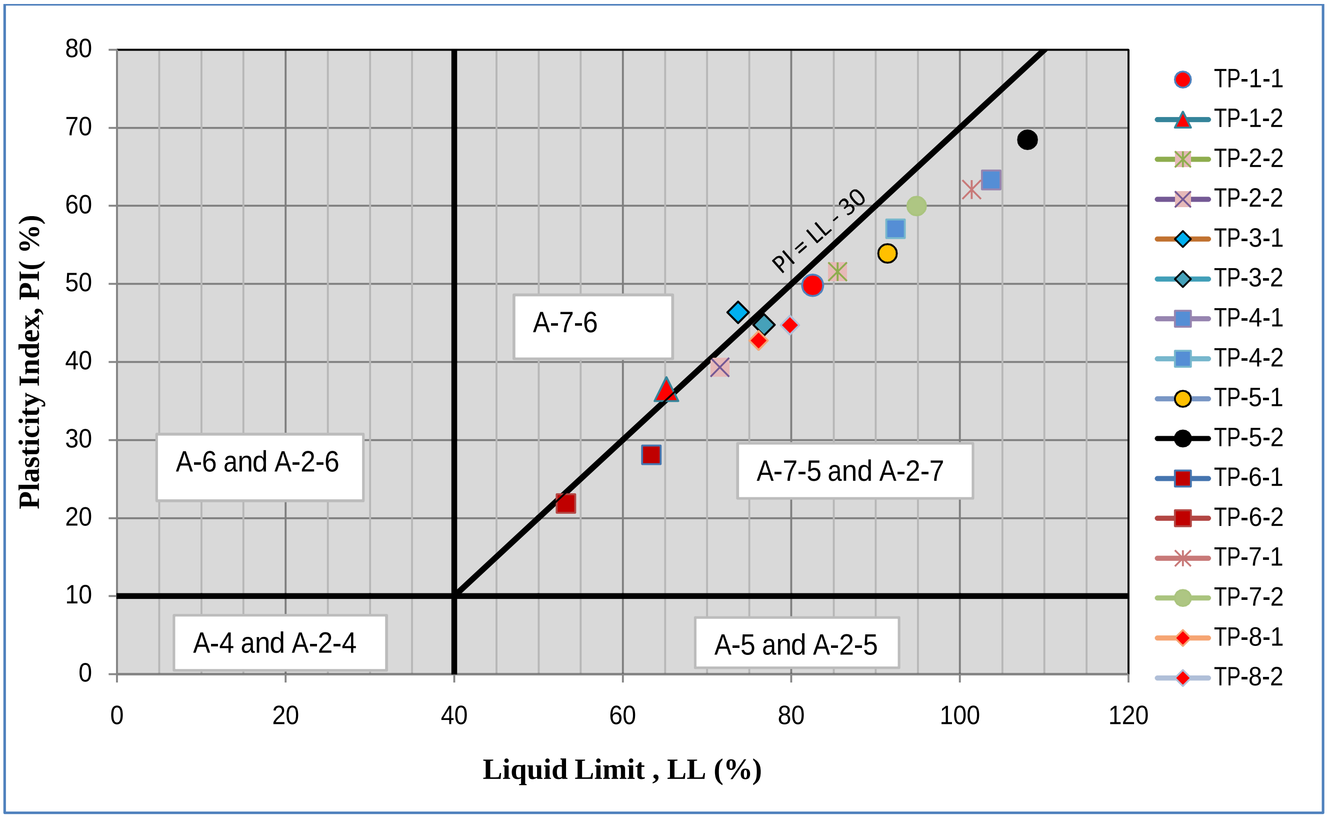

Figure 4. 3 ASHTO plasticity chart classification for the Jimma fine-grained soils

4.9.2 Classification for Expansive soils

Classification using Index properties

Here the classification is done based on the method proposed by Daksanamurty and Raman [15]. Anderson et al. and Chen [15] that uses plasticity index as its basis of classification. The result of Jimma expansive clay soils is presented in Table 4.16, Table 4.17 and Table 4-18.

Table 4. 16 Classification for degree of expansion using liquid limit criteria according to Daksanamurty and Raman [15]

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Liquid Limit (%)	Degree of Expansion -Daksanamurty and Raman [15]
1	Merkato-2	TP-2-1	1.5	85	Very high
2		TP-2-2	2.5	72	Very high
3	Teknik Sefer	TP-4-1	1.5	104	Very high
4		TP-4-2	2.5	92	Very high
5	Kebele-5	TP-5-1	1.5	97	Very high
6		TP-5-2	2.5	108	Very high
7	Bacho Bore	TP-7-1	1.5	63	High
8		TP-7-2	2.5	53	High

Table 4. 17 Classification for degree of (swelling potential) using plasticity index according to Anderson et al. and Chen [15]

Ser.No.	Location	Test Pit Designation	Depth of Sampling (m)	Plastic Index (%)	Degree of Expansion (Anderson et al.)	Degree of Expansion (Chen)
1	Merkato-2	TP-2-1	1.5	52	Very high	High
2		TP-2-2	2.5	39	High	High
3	Teknik Sefer	TP-4-1	1.5	63	Very high	Very high
4		TP-4-2	2.5	57	Very high	Very high
5	Kebele-5	TP-5-1	1.5	59	Very high	High
6		TP-5-2	2.5	68	Very high	Very high
7	Bacho Bore	TP-7-1	1.5	62	Very High	Very High
8		TP-7-2	2.5	60	Very High	Very High

Table 4. 18 Relationship between Swelling potential and Degree of swell, Seed et al. [15]

Ser.No.	Location	Test Pit Designation	Depth of Sampling (m)	Plasticity index, PI (%)	Swelling Potential, SP	Degree of Swell
1	Merkato-2	TP-2-1	1.5	52	48	Very High
2		TP-2-2	2.5	39	23	High
3	Teknik Sefer	TP-4-1	1.5	63	79	Very High
4		TP-4-2	2.5	57	59	Very High
5	Kebele-5	TP-5-1	1.5	54	59	Very High
6		TP-5-2	2.5	68	100	Very High
7	Bacho Bore	TP-7-1	1.5	62	75	Very High
8		TP-7-2	2.5	60	64	Very High

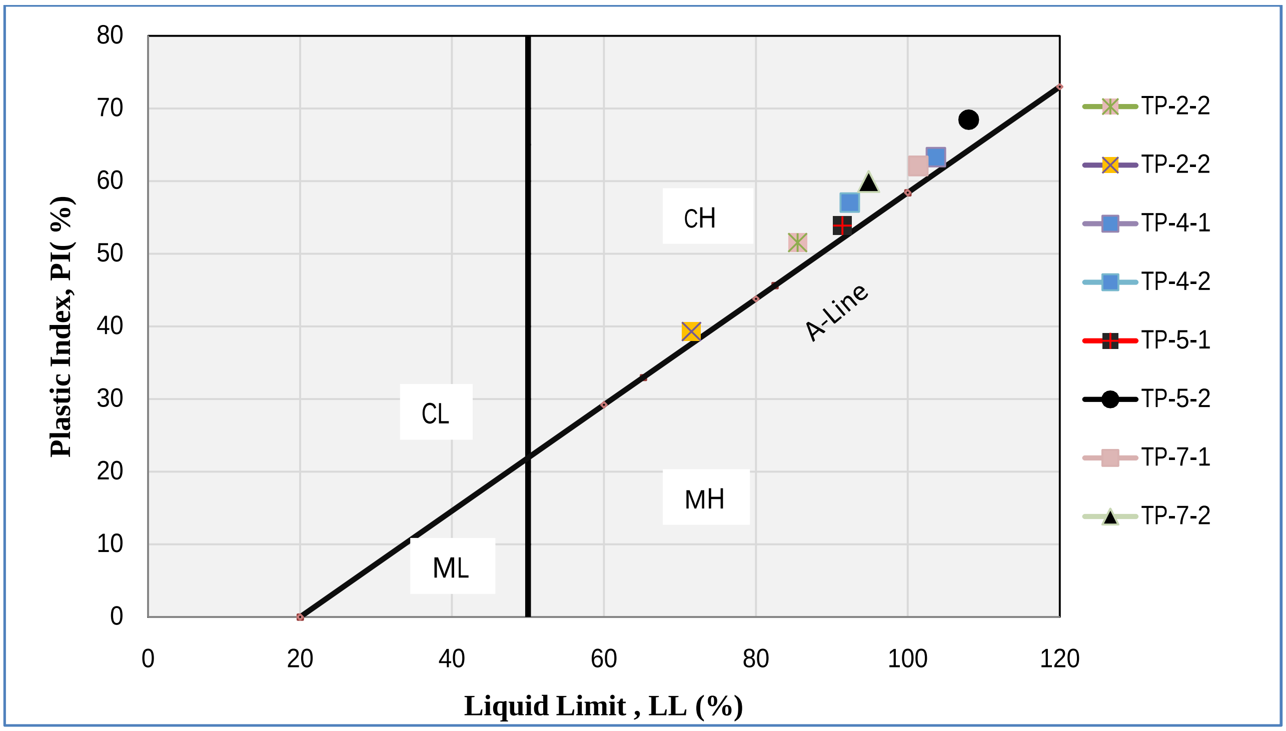

Figure 4. 4 USCS classification for Jimma expansive soils using plasticity chart

Classification based on Activity and Expansive soil charts

The location of the tested soils on the chart of Seed et al. and that of Merwe using the relation stated under section 2.5.6 are presented in Figure 4.5 and Figure 4.6 respectively.

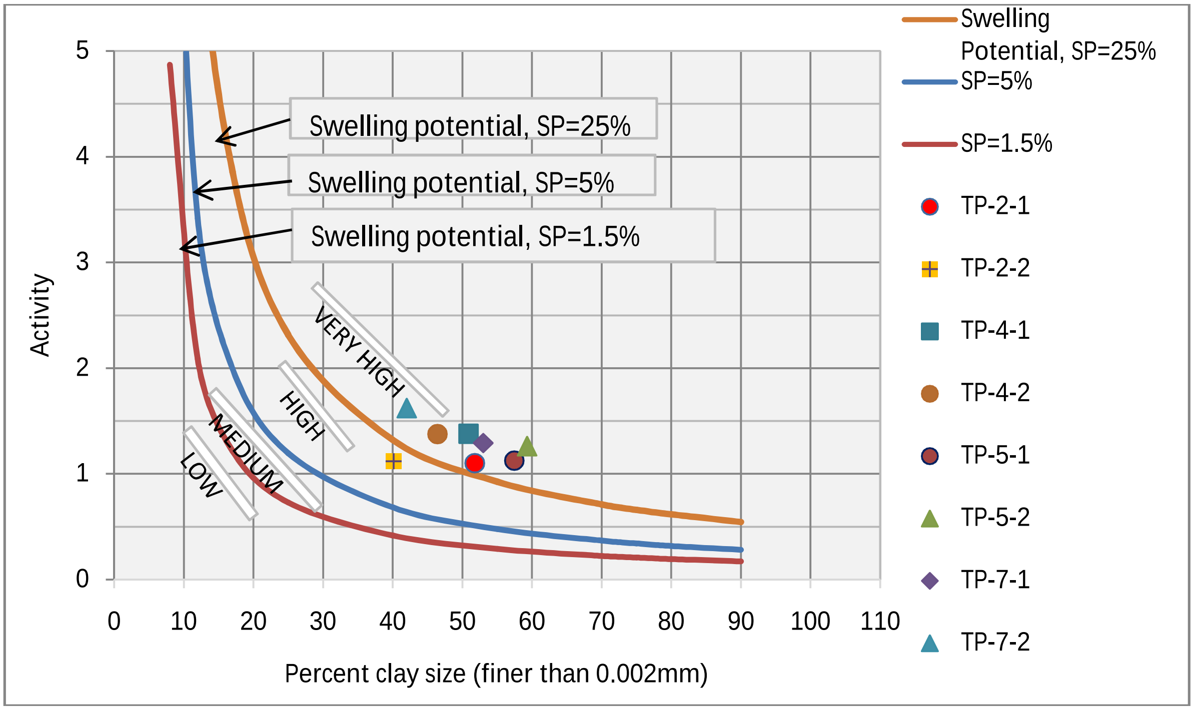

Figure 4. 5 Swelling potential classifications of Jimma soils using the method of Seed et al. [15]

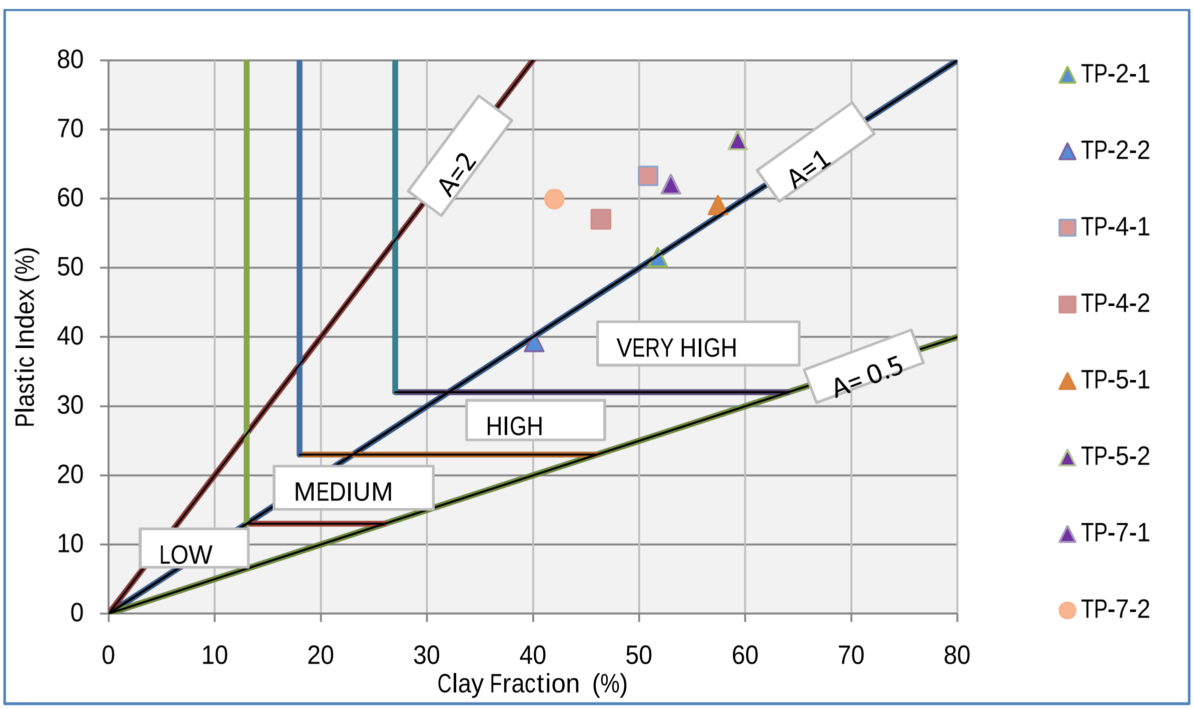

Figure 4. 6 Potential expansiveness classifications for Jimma clay soils using the Chart of V.D. Merwe [4]

4.10 Consolidation Test Results

4.10.1 General

The one dimensional consolidation test is used to obtain compression parameters to estimate the amount of settlement and consolidation parameter such as c_v is used to predict the rate of settlement of structures. The pre-consolidation pressure p_c and the OCR can be also determined from this test. Conventional consolidation test confines the soil laterally in a metal ring so that settlement and drainage can occur only in the vertical direction. These conditions are reasonably close to what occurs in situ for most loading cases.

4.10.2 Test Procedures and Methods

A one dimensional Consolidation test is conducted using conventional Oedometer apparatus. The type of standard consolidation test used in this case is a controlled stress test (CST) in which a constant load increment is used for each stage of the test. ASTM D2435 is employed to conduct the test. The results are plotted as void ratio *e* vs. *log p* [5].

The test is performed on an undisturbed soil sample that is placed in a consolidation ring available in diameters ranging from 45 to 115 mm. The sample height is between 20 and 30 mm; 20 mm is the most commonly used thickness to reduce test time. The larger diameter samples give better parameters, since the amount of disturbance (recovery, trimming, insertion into the test ring, etc.) is less for the larger samples. The consolidation test proceeds by applying a series of load increments (usually in the ratio of $\Delta p/p=1$ in a pressure range from about 25 to either 1600 kPa) to the sample and recording sample deformation by using either an electronic displacement device or a dial gauge at selected time intervals[5].

Based on location of test pits, soil conditions and type of soils (expansive or non-expansive) five representative test-pits are selected and undisturbed samples are used. For significant comparison of preconsolidation pressure against overburden pressure, the test was conducted on the samples from 2.5m depth. The other reason is, from design and construction practice in the area, most of the foundation of ordinary building structures placed at this depth.

The general outline of Terzaghi-Froehlich's theory of consolidation is employed for the double drainage condition of consolidation test [17]. Furthermore, to calculate the initial void ratio specific gravity (Gs), initial moisture content (NMC), bulk unit weight (γ_{bulk}) and dry unit weight (γ_d) are used.

4.10.3 Results of Consolidation Test

Pressure – void ratio curve

The pressure-void ratio curve can be obtained if the void ratio of the sample at the end of each increment of load is determined. The basic data used to determine this curve are natural moisture content, Specific gravity, density, cross sectional area and height of the sample, initial void ratio and applied loads. From these curve important parameters such as coefficient of compressibility (a_v), compression indexes (Cc), Swelling index (Cs) and preconsolidation pressure (p_c) are determined. The summary of test results is presented in Table 4.19 and Figure 4.7 below.

Table 4. 19 Summary of applied pressure and void ratio for five samples of Jimma soils

Test pit Designation	TP-1-2	TP-3-2
Pressure,kPa	**void ratio, e(%)**	**void ratio, e(%)**
Loading stage		
7	1.10	1.29
50	1.07	1.27
100	1.06	1.26
200	1.04	1.23
400	1.00	1.20
800	0.96	1.14
1600	0.88	1.06
Unloading stage		
1600	0.88	1.06
800	0.89	1.07
400	0.90	1.08
200	0.92	1.09
100	0.93	1.09
50	0.94	1.10
7	0.97	1.11

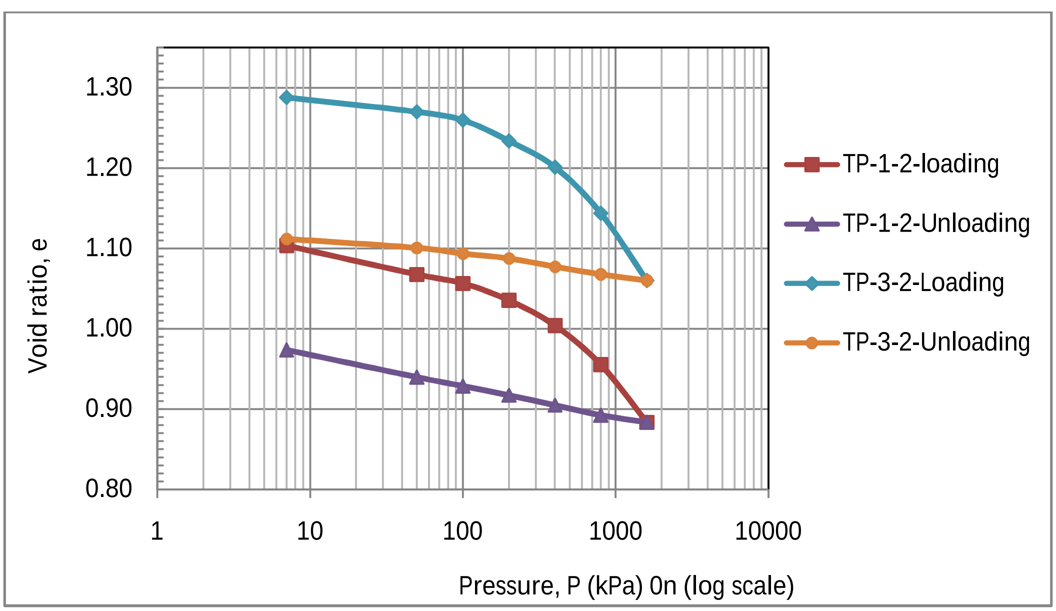

Figure 4. 7 Summarized void ratio and log pressure curve for five samples of Jimma soils

Compression Parameters and Preconsolidation pressure

Compression parameters such as coefficient of compressibility (a_v) is obtained from a plot of void ratio versus pressure and compression index (Cc), swelling index (Cs) and preconsolidation pressure (P_c) are obtained from void ratio versus log pressure curve.

There are a few graphical methods for determining the preconsolidation pressure based on laboratory test data. The earliest and the most widely used method was the one proposed by Casagrande [5]. The method involves locating the point of maximum curvature, on the laboratory e-log p curve of an undisturbed sample, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the preconsolidation pressure Pc. The summary of the results are tabulated as in Table 4.20 and the detail is presented in the Appendix D-I.

Coefficient of Consolidation

The coefficient of consolidation, c_v can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical.
There are two laboratory methods that are in common use for the determination of c_v. They are: Casagrande Logarithm of Time Fitting Method and Taylor Square Root of Time Fitting Method.
However, for the following reasons Taylor Square Root of Time Fitting Method is selected. First the time plot gives a straight line for the initial portion to indicate the point of corrected dial reading, second the square-root time curve is more suitable for soils exhibiting high secondary consolidation and third it is more convenient for a general case as it requires dial gauge readings covering a much smaller shorter period of time comparing with log-time method [6].

The coefficients of consolidation obtained from the test results are variable for different loadings. As a result Schneider [21] proposed characteristics value which is determined as follows:

$$c_{v,\,ch} = c_{v,\,mean} - 0.5c_{v,\,sd} \tag{4.2}$$

Where: $c_{v,\,ch}$ = *characteristic* coefficient of consolidation, $c_{v,\,mean}$ = *mean* coefficient of consolidation
$c_{v,\,sd}$ = *standard deviation for* coefficients of consolidation

Coefficient of Permeability, k_v

Average vertical coefficient of permeability, *kv* can be estimated using the following formula [5].

$$k_v = c_{v,ch}\, m_v \gamma_w \tag{4.3}$$

For coefficient of consolidation and permeability the summary of the results are tabulated as in Table 4.21while the detail of the calculation is presented in the Appendix D-II.

Table 4. 20 Summary for Oedometer test results for different test-pit of Jimma soils

Ser.No.	Location	Test Pit Designation	Initial moisture content (%)	Initial Void Ratio, e_o	Bulk unit weight , γ_b (kK/m^3)	Compression Index, Cc	Swelling Index, Cs[4]	Coefficient of compressibility,av (m^2/kN)	Preconsolidation Pressure, p_c (kPa)	Overburden Pressure, σo (kPa)	Over consolidation ratio, OCR
1	Merkato-1	TP-1-2	31.88	1.11	17.35	0.238	-0.0393	0.000106	230	43.38	5.3
2	Merkato-2	TP-2-2	45.51	1.28	17.15	0.336	-0.0790	0.000241	-	42.87	-
3	Awetu	TP-3-2	42.26	1.29	17.45	0.278	-0.0460	0.000164	300	43.63	6.9
4	Technic Sefer	TP-4-2	40.52	1.19	16.66	0.291	-0.0947	0.000172	-	41.66	-
5	Kebele-5	TP-5-2	56.23	1.26	17.21	0.399	-0.0830	0.000200	-	43.03	-

[4] The negative sign indicates the soil is in state of expansion (upward increase in volume)

Table 4. 21 Characteristic coefficient of consolidation and coefficient of permeability for five test pits of Jimma clay soils

Ser.No.	Location	Test Pit Design ation	Initial Void Ratio, e_o	Coefficient of compressibility, a_v (m^2/kN)	Coefficient of volume change,m_v (m^2/kN)	Coefficient of consolidation, $c_{v,ch}$ (cm^2/min)	Coefficient of permeability, $k_{v,\,ch}$ (cm/min)
1	Merkato-1	TP-1-2	1.11	1.06E-04	5.03E-05	$7.24x10^{-02}$	$3.64x10^{-07}$
2	Merkato-2	TP-2-2	1.28	2.41E-04	-	-	-
3	Awetu	TP-3-2	1.29	1.64E-04	7.15E-05	$1.15x10^{-01}$	$8.24x10^{-07}$
4	Technic Sefer	TP-4-2	1.19	1.72E-04	-	-	-
5	Kebele-5	TP-5-2	1.26	2.00E-04	-	-	-

4.11 Compaction Tests

4.11.1 General

The maximum dry density and optimum moisture content obtained are used to determine the strength to be attained during construction of road, foundation base, or retaining structures. For road construction the CBR value is obtained using the compaction test results. And these CBR results used to determine the thickness of the sub-grade layer of a road section. Maximum dry density and optimum moisture content are also used to compact the soil for foundation backfill and for earthen dam construction.

Particular objectives of this compaction test are: one, to evaluate the effect of sample preparation on lateritic soils, second, for general classification of the soils based on their dry density and optimum moisture content , third, anyone can conduct CBR test using compaction test results for evaluation of sub-grade of a road.

4.11.2 Test Procedures and Methods

ASTM D 698 – 91 (Standard Proctor Test) procedures are applied to conduct the compaction test. This method is employed since the soil 20 % or less by weight of particles retained on sieve No.4 (4.75mm).

The field unit weights of soils are also determined. This test is used to determine the in-place density of soils and to compare with the compacted dry density. Densities are calculated from unit weights measured from the laboratory divided by gravity due to the earth. To calculate overburden pressure at any depth the unit weight of the soil should be first determined. The test was conducted according to ASTM D 2937-00, Standard Test for Density of Soil in Place by the Drive-Cylinder Method, procedure.

All these tests are conducted on samples from 2.5m where relatively non-contaminated with surface materials and have good engineering quality in terms of grain size distribution, Atterberg limits and unconfined compression strength. The soils at this level exhibit coarser in size and relatively less in their plasticity behavior. Also for practical purpose such as building foundation placing, the testes shall be conducted from deeper depth. Compaction test results are summarized in Table 4.22 and Figure 4.8. The detail of the results is presented in Appendix E-1.

Table 4. 22 Maximum dry unit weight and optimum moisture content for Jimma soils

Ser.#	Location	Test pit designation	Field dry unit weight (kN/m^3)	Maximum dry unit weight (kN/m^3)	OMC (%)
1	Merkato-1	TP-1-2	13.16	13.75	30
2	Merkato-2	TP-2-2	11.78	13.25	35
3	Awetu	TP-3-2	12.27	13.30	34
4	Technik Sefer	TP-4-2	11.86	12.10	37
5	Kebele-5	TP-5-2	11.20	12.48	34
6	Jimma Univ.	TP-6-2	-	15.20	33
7	Becho Bore	TP-7-2	11.96	12.70	33
8	Agri Sefer	TP-8-2	12.62	14.00	35

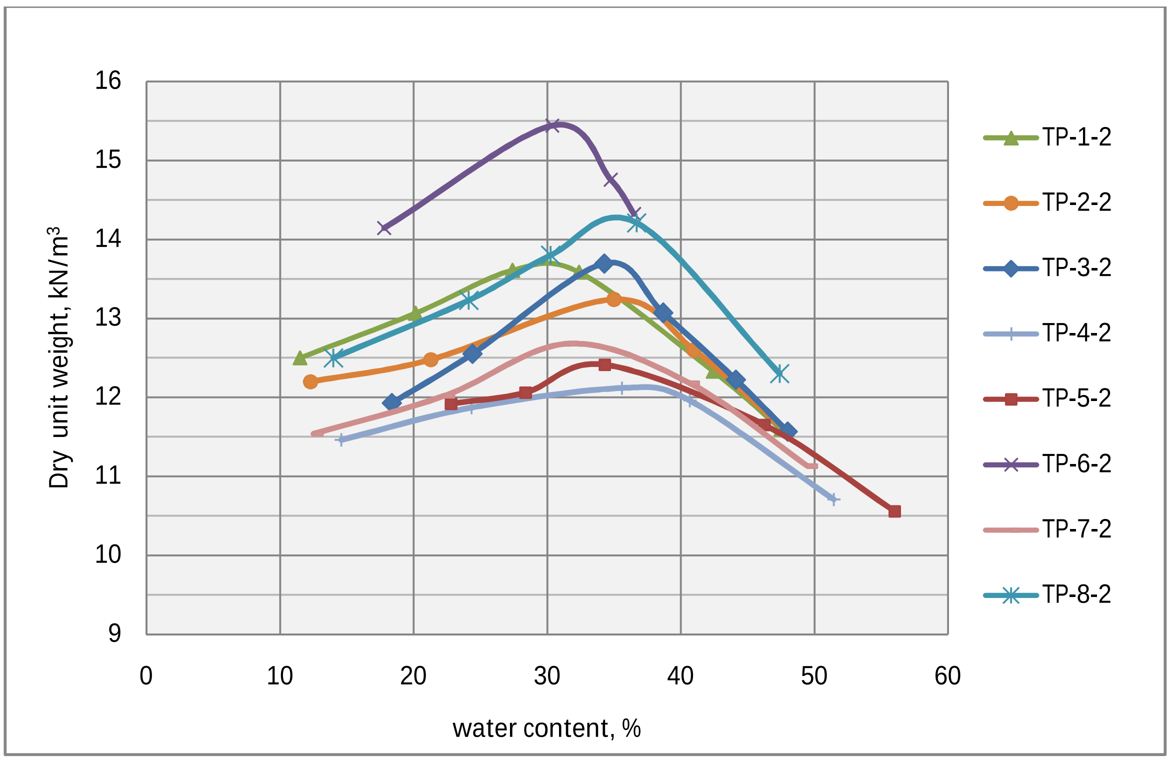

Figure 4. 8 Summarized compaction curve for Air-dried- fresh samples for 8 test pits

4.11.3 Effect of Sample Preparation Conditions

Air dried (AD) and oven dried (OD) pre-treatment conditions were examined on TP-3, Tp-6 and TP-8 to evaluate the effect of drying on compaction test values. Oven drying of wet soil samples were carried out at temperature of 105^{0}C. Air drying comprises both exposing the wet soil sample to sunlight energy and simply spreading on blue sheet inside laboratory. These testing conditions mainly affect lateritic soil type.

The values of maximum dry unit weight and optimum moisture contents for the lateritic soils exposed to different drying conditions are summarized in Table 4.23 and the curves are plotted in Figure 4.9 to Figure 4.11. The detail result is also presented in Appendix E-2.

Table 4. 23 Effect of sample drying on compaction test results of Jimma Lateritic soils

Ser.#	Location	Test Pit designation	Sample preparation condition	Optimum moisture content (%)	Maximum unit weight (kN/m^3)
1	Awetu	TP-3-2	AD	36	13.30
2	Awetu	TP-3-2	OD	33	13.80
3	Jimma University	TP-6-2	AD	33	15.20
4	Jimma University	TP-6-2	OD	32	15.40
5	Agri Sefer	TP-8-2	AD	35	14.00
6	Agri Sefer	TP-8-2	OD	33	14.50

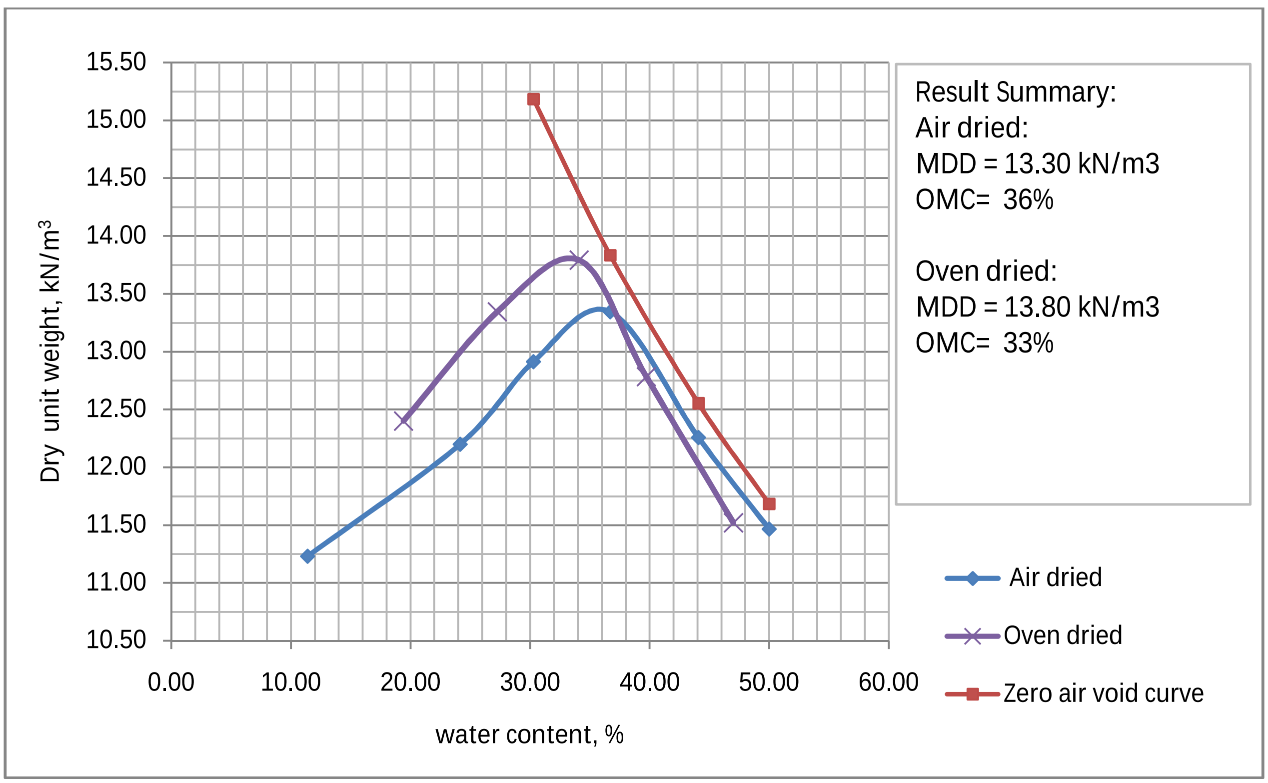

Figure 4. 9 Effect of sample drying on compaction curve for TP-3-2

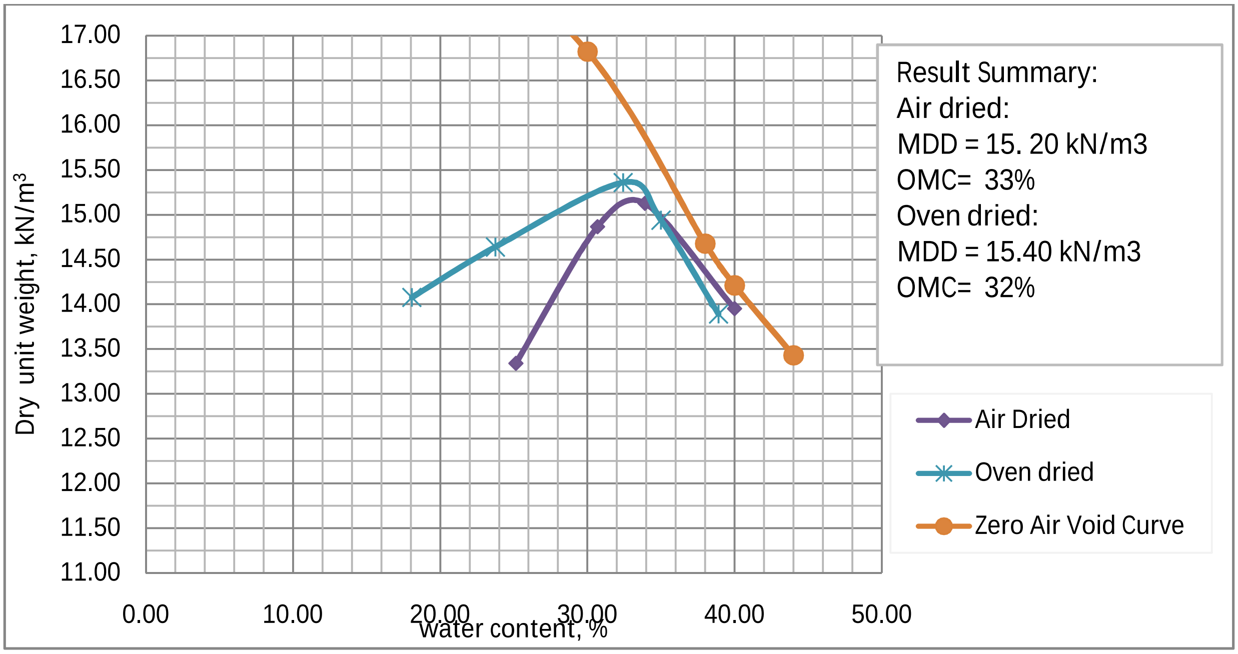

Figure 4. 10 Effect of Sample drying on compaction curve for TP-6-2

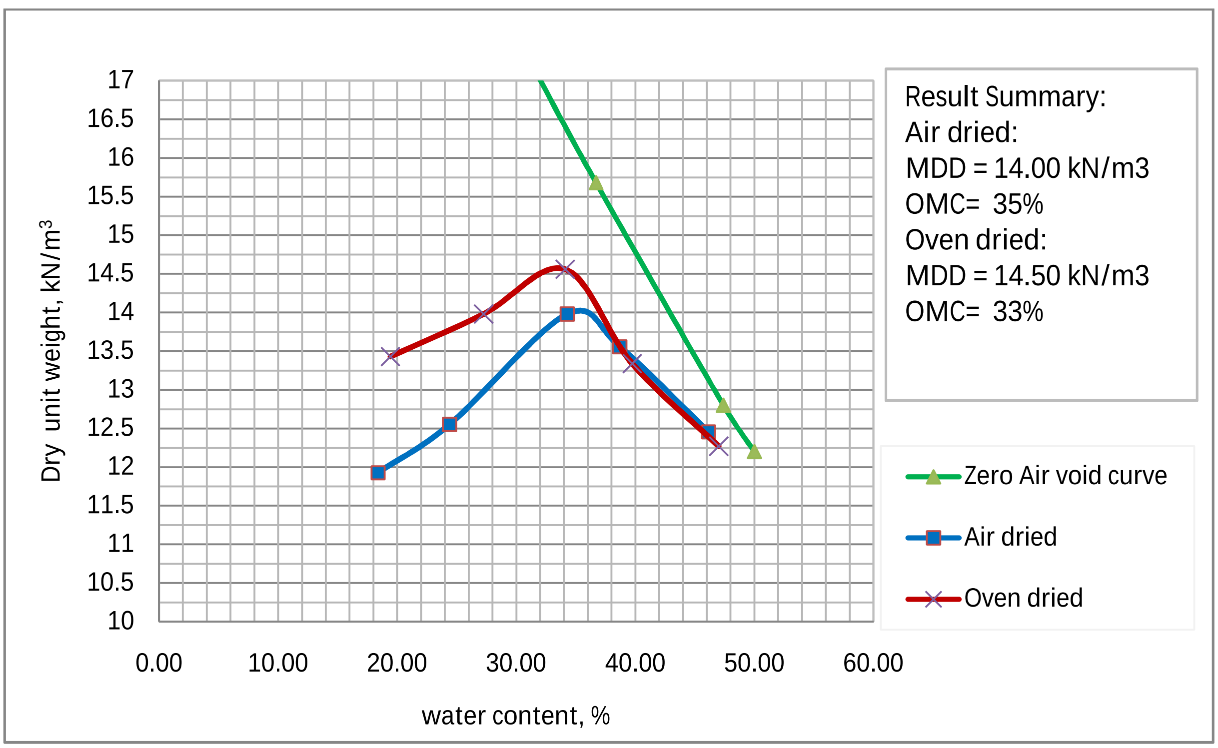

Figure 4. 11 Effect of Sample drying on compaction curve for TP-8-2

4.12 Unconfined Compression Strength Tests

4.12.1. General

There are three methods of laboratory shear strength test. These are direct shear test, triaxial compression test and unconfined compression test. For this study unconfined compression strength test was conducted.

The primary purpose of this test is to determine the unconfined compressive strength of cohesive soils on undisturbed sample, which is used to calculate the undrained shear strength of the material under investigation. It is used to determine the strength of remolded samples for cohesive soils. Unconfined strength of remolded samples is used to compute sensitivities of the soils. As well, it is to determine modulus of deformation of the soils from stress strain curve of the test result.

4.11.2 Test Procedures and Methods

ASTM D D2166 is employed to conduct the test on undisturbed samples whereas compaction test procedure in combination with ASTM D D2166 is used for remolded samples.

The strength of a soil determined by compression testing varies with extremes of the length to diameter ratio and the rate of strain. It is generally accepted that ratios of length to diameter of 2 to 2.5 are satisfactory. Similarly, satisfactory rates of strain are 0.5 to 2.0% per minute [5]. For the current investigation, length to diameter ratio of 2 (38mm diameter and 76mm height) and a strain rate of 2% per minute (1.52 mm per minute) was used.

Undrained shear strength is half of the ultimate shear stress of a soil which is obtained from shear stress versus shear strain curve at specified failure criteria condition [5]. This relation is well stated under section 2.8.
The objectives of conducting this strength test are four: one, to determine the unconfined shear strength of soils based on their natural moisture content and undisturbed sample state, second, to compare the shear strengths of undisturbed and remolded samples, third, to determine the sensitivity of the soils upon remolding, fourth, to determine modulus of deformation of the undisturbed soils from stress-strain curve.

4.12.3 Unconfined shear strength for undisturbed soil samples

For the undisturbed samples, the tests were carried on 6 test-pits at 1.5m and 2.5m depths, except one test pit-pit (where undisturbed sample was unable to recover due to its silt and sand content), to compare the strength of the soils across vertical depth. Modulus of deformation is determined from the stress-strain value at 2.5m depth level. This is for the rationale that the magnitude of settlement is calculated at the base of the foundation placed at this level. The current natural moisture content and field conditions were considered to determine the undrained shear strength of the soils. The test results are summarized in Table 4.24 and Figure 4.12. The detail is presented in Appendix F.

From stresses and strains relationships of the soil, one can compute *modulus of deformation* E_s from stress-strain curve. E_s is used to calculate the magnitude of settlement and the rate of settlement for soils under any applied loads.

One should be careful in determining modulus of deformation from stress-strain curve. There are different types of moduli which can be determined from stress-strain relation under variable conditions. In this study, based on their practical importance, tangent modulus and secant modulus are considered for undisturbed samples. Tangent modulus of deformation is calculated from slope of tangent line which is change of stress divided by change of strain developed at each point. According to the suggestion of different literatures [5] and for this particular condition, initial tangent modulus is considered to have a better estimation of the required practically applicable modulus. It is determined by drawing a tangent line from origin on a stress-strain curve.

Secant modulus of deformation is calculated from slope of line from origin to stress and strain corresponding to peak values [22].

For both, modulus of deformation is determined from stress-strain relation from UCS test using the formula shown in equation 4.4 [5].

$$E_s = \Delta q_u / \Delta \varepsilon \tag{4.4}$$

Where, q_u = unconfined compressive strength

ε = Axial strain

The results of modulus of deformations for tangent and secant are presented in Table 4.25. The detail is presented in Appendix F.

Table 4. 24 Summary of undrained shear strength for Jimma soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Color	NMC (%)	Unconfined compressive strength, q_u (kPa)	Undrained shear strength, $c_u = q_u/2$ (kPa)
1	Merkato-1	Tp-1-1	1.5	Brownish Red	51	167	84
2		Tp-1-2	2.5	Gray	33	115	58
3	Merkato-2	Tp-2-1	1.5	Gray	50	110	55
4		Tp-2-2	2.5	Gray	45	135	68
5	Awetu	Tp-3-1	1.5	Red	41	235	118
6		Tp-3-2	2.5	Red	42	315	158
7	Teknik Sefer	Tp-4-1	1.5	Gray	41	105	53
8		Tp-4-2	2.5	Gray	41	130	65
9	Kebele-5	TP-5-1	1.5	Black	66	85	43
10		TP-5-2	2.5	Black	56	108	54
11	Jimma University	TP-6-1	1.5	Red	42	286	143

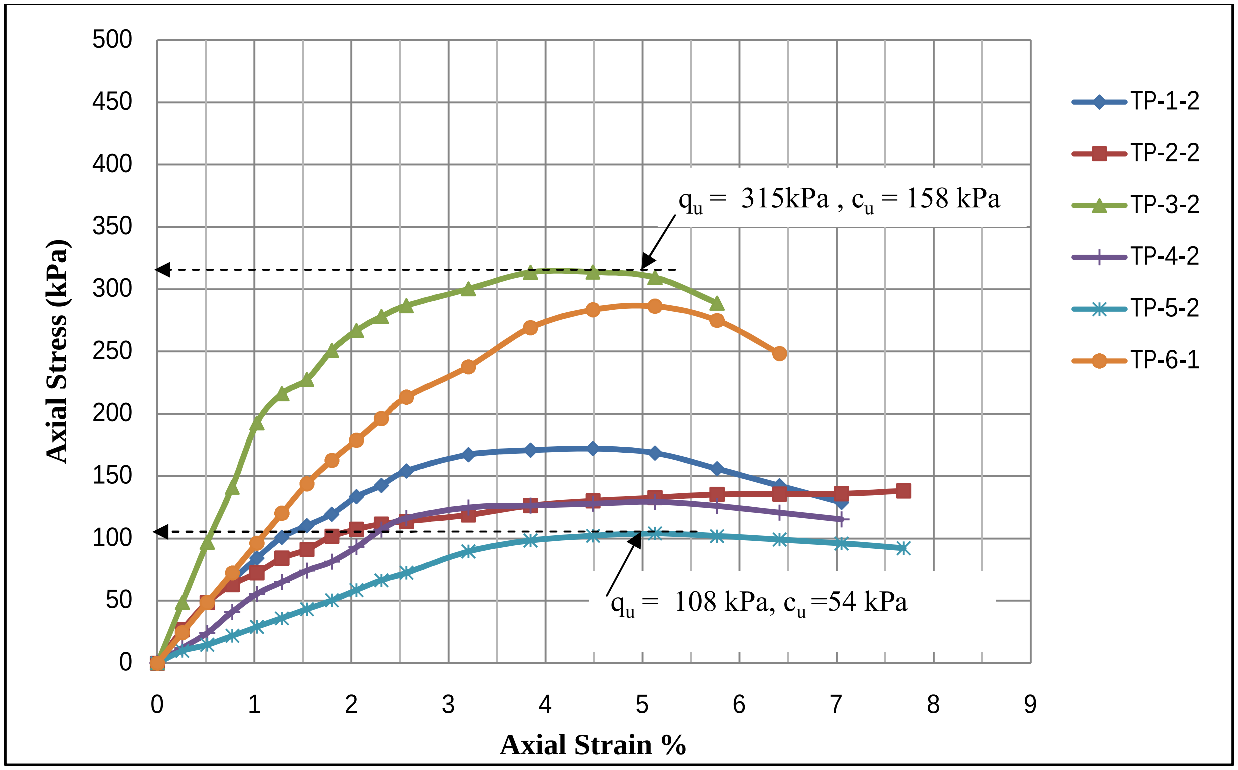

Figure 4. 12 Summary of stress-strain curve of UCS test for six undisturbed samples

Table 4. 25 Summary of modulus of deformation from UCS test for some Jimma soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Field density (kN/m^3)	Natural moisture content (%)	Tangent Modulus of deformation, $E_{s(t)}$ (kPa)	Secant Modulus of deformation, $E_{s(s)}$ (kPa)
1	Merkato-1	TP-1-2	2.5	17.34	33	6250	2300
2	Merkato-2	TP-2-2	2.5	17.12	45	10000	2700
3	Awetu	TP-3-2	2.5	17.44	42	19000	6300
4	Technic Sefer	TP-4-2	2.5	16.70	41	4375	2600
5	Kebele-5	TP-5-2	2.5	16.83	56	3750	2160
6	Jimma University	TP-6-1	2.5	17.98	42	9375	5720

4.11.4 Unconfined Shear Strength for Remolded Samples

This test is made on remolded samples using the field dry density and natural moisture content. This test is intended to evaluate the strength of compacted soil for engineering construction purposes such as foundation backfill, road sub-grade, embankment fill and earthen dam fill. The remolded shear strength is also use to evaluate the sensitivity of the clay soils. Four test pits are selected and the tests were conducted from samples taken at 2.5m depths where at this depth the undisturbed sample shows the soils have good strength. The samples are prepared using the concept of compaction tests. The results are summarized as in the Table 4.26. Comparisons for undisturbed and remolded sample are also presented in Figure 4.13 to Figure 4.16.

Table 4. 26 Undrained shear strength for remolded samples

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Color	NMC (%)	Unconfined compressive strength, $q_{u,r}$ (kPa)	Undrained shear strength, $c_u = q_{u,r}/2$ (kPa)
1	Merkato-1	Tp-1-2	1.5	Brownish Red	33	97	49
2	Awetu	Tp-3-2	2.5	Red	42	180	90
3	Kebele-5	TP-5-2	2.5	Black	56	52	26
4	Jimma University	TP-6-1	1.5	Red	42	156	78

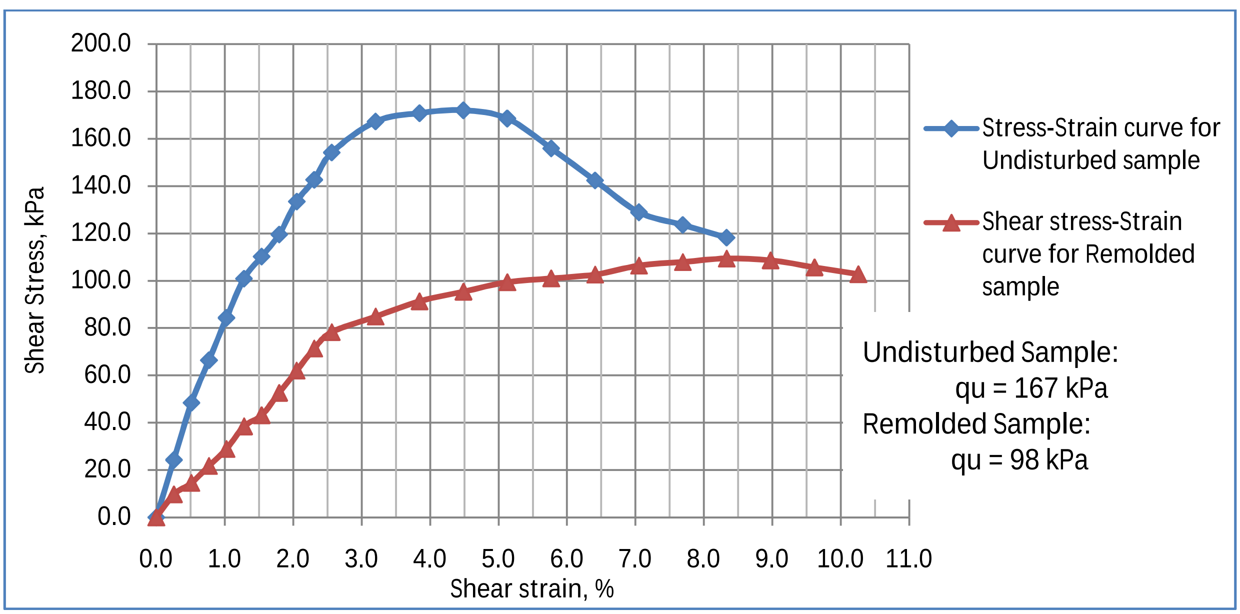

Figure 4. 13 Comparison of stress - strain curve for undisturbed and disturbed sample for TP-1-1

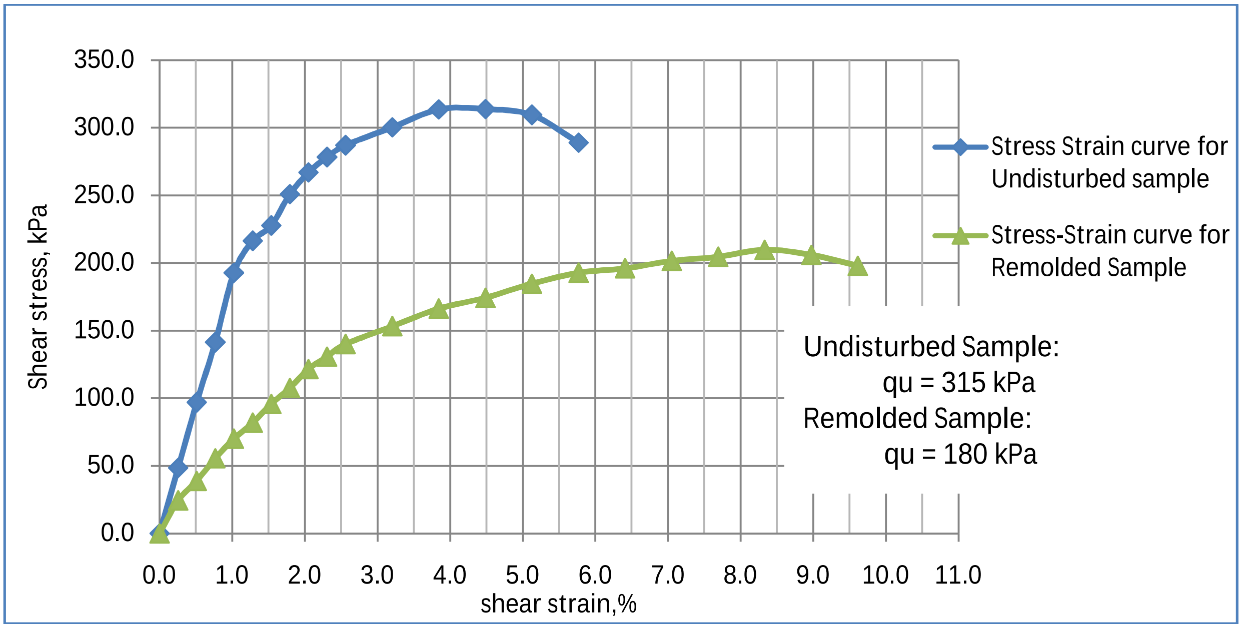

Figure 4. 14 Comparison of stress - strain curve for undisturbed and disturbed sample for TP-3-2

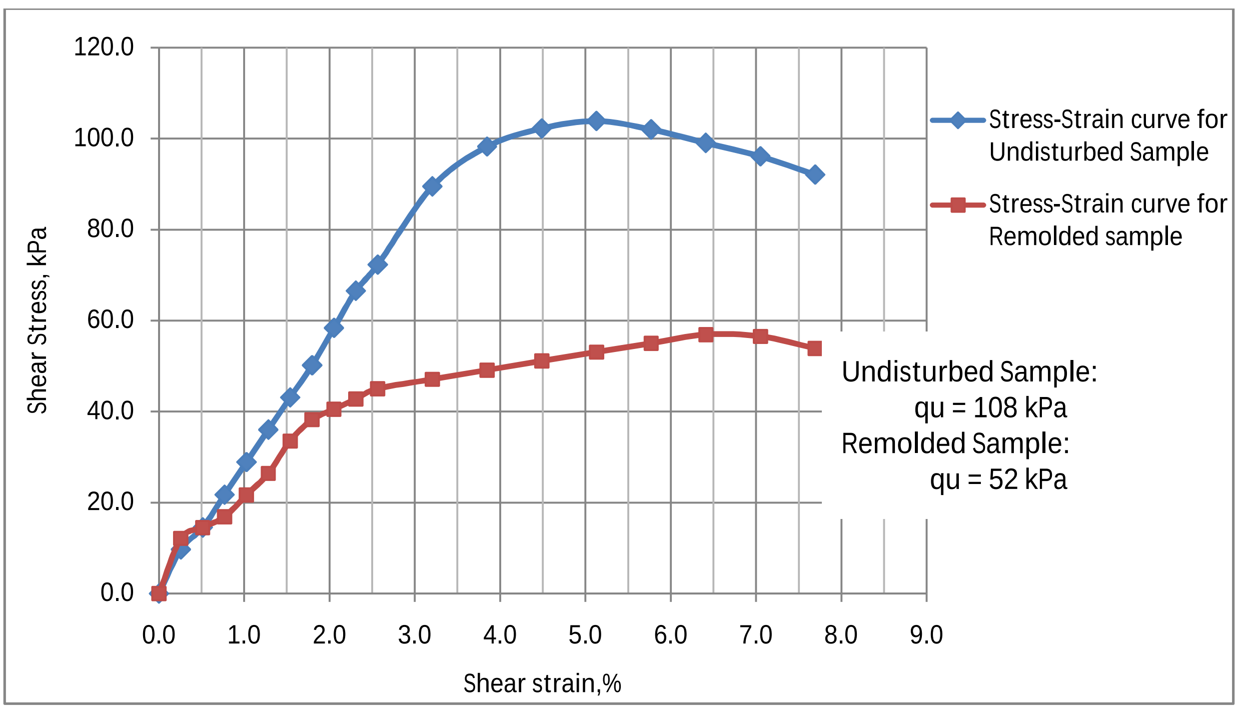

Figure 4. 15 Comparison of stress-strain curve for undisturbed and disturbed sample for TP-5-2

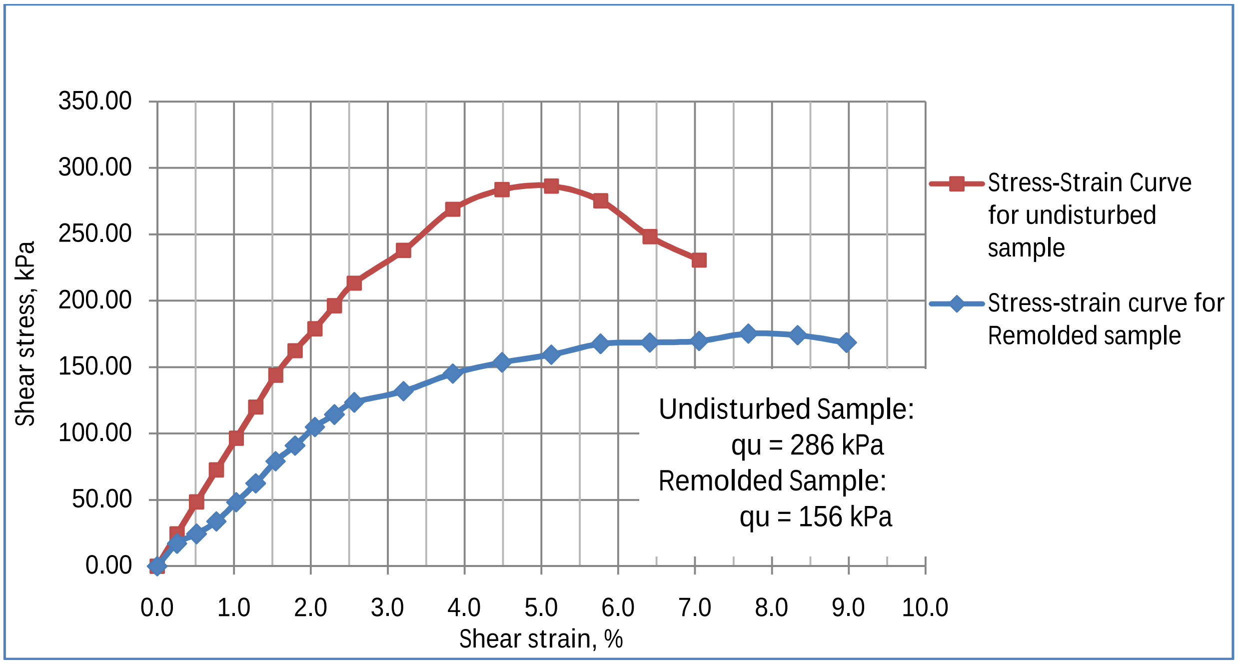

Figure 4. 16 Comparison of stress-strain curve for undisturbed and disturbed sample for TP-6-1

4.11.5 Sensitivity of clay soils

Remolding of an undisturbed sample of clay at the same water content alters its consistency, because of the destruction of its original structure. The degree of disturbance of undisturbed clay sample due to remolding can be expressed as (as under Section 2.8.3):

$$S_t = q_u/q_{u,r} \quad (4.4)$$

Where: q_u = Unconfined compressive strength for undisturbed sample

$q_{u,r}$ = Unconfined compressive strength for remolded sample

The results are presented in Table 4.27.

Table 4. 27 Sensitivity nature of Jimma clay soils

Ser.No.	Location	Test Pit designation	Depth of sampling (m)	Color	Unconfined compressive strength for undisturbed sample, q_u (kPa)	Unconfined compressive strength for remolded sample, $q_{u,r}$ (kPa)	Sensitivity, $S_t = q_u/q_{u,r}$	Remark
1	Merkato-1	Tp-1-2	1.5	Brownish Red	167	97	1.7	Low sensitive clay
2	Awetu	Tp-3-2	2.5	Red	310	180	1.7	Low sensitive clay
3	Kebele-5	TP-5-2	2.5	Black	103	52	2.0	Medium sensitive clay
4	Jimma University	TP-6-1	1.5	Red	286	156	1.8	Low sensitive clay

5 DISCUSSION ON TEST RESULTS

5.1 Assessments of Test Results

The points addressed in this research work are analyzed and discussed as follows.

Natural moisture content

From simple filed test, using the natural moisture content of the soils investigated, the state of moisture condition ranges from moist to wet. Likewise, their consistency varies from very soft to hard.

For tropical residual soils, a moisture variation of 4 – 6 % or more indicates that loosely bound molecular water is present. From the test results as indicated in Table 4.1, one can see that the differences in natural moisture contents for all red lateritic samples at 105ºc and 50 ºc and 30% relative humidity under consideration are between 4% - 6%. This shows there is considerable amount of structural water that could be destroyed by oven drying at a temperature of 105ºc and must be deducted from every moisture content determinations in Atterberg limit tests [11].

Specific Gravity

The specific gravity of most soils under investigation lies within a narrow range of 2.5- 2.85. But organic soil or soil containing porous particles such as diatomaceous earth show low specific gravity values such as 2.3 or less. On the other hand soils containing heavy substances such as iron may have values above 3. So, from the specific gravity value of Table 4.2, the soils can be categorized as inorganic soils since their Gs values are greater than 2.5 [4]. For some lateritic soils as it is investigated, their specific gravity is higher than others. This is due to the presence significant amount of iron-oxide in the mineral content of the lateritic soils [14].

Grain size analysis

From Table 4.3 and Figure 4.1, the results of grain size analysis at shallower depths show that more than 90% of the total mass pass through sieve size of 75µm. This indicates that almost all samples are fine-grained soil. From hydrometer analysis the size of more than 30% are less than 5µm (clay content as per ASTM boundaries criteria). This gives a hint the properties of these soils are highly affected by the presence of the clay content.

Effect of sample location, is another important factor. Residual soils formed at the same profile and locations show similar characteristics due to their mode of weathering, deposition and soil forming factors to which they are exposed. From the test results one can see that the soil has distinct characteristics where sampled at different locations.

From Table 4.3, the soils under investigation also show different characteristics along their depths. This is the fact that residual soils vary from highly weathered topsoil to unweathered (intact rock) which is stiff at the bottom.

For lateritic soils, according to pedological classification method based on particle size, for TP-3 and TP-8 the soil is classified as lateritic clay more than half of the particle size is less than 0.002mm and TP-6 can be classified as lateritic clayey silt [12].

Atterberg limits test

From the Atterberg limits test results of Table 4.4, in most cases the values decrease from 1.5m to 2.5m across vertical profile. This shows the fine content which affects the plasticity nature of the soil decreases from top to bottom of the strata.

From Table 4.5, one can also observe that the different treatments affect the Atterberg limits for soil samples fromTP-3, TP-6 and TP-8. The liquid limit and the plastic limit decrease with temperature increment and the increment is large value, normally 4 to 6 percent would be the acceptable range [11]. This is due to aggregation of fine particles into coarser particle after the soil dried. As a result a capillary stress is developed and sesquioxide coats among particles these inhibit the penetration of water between the particles [11, 14].Hence the temperature treatment conditions significantly affect the Atterberg values for Jimma lateritic soils.

Table 4.6 shows that the liquid limit is larger with significant amount for 30 minute mixing time than 5minute mixing time duration. Therefore, the greater the duration of mixing time (i.e. the greater the energy applied to the soil), the larger the resulting of liquid limit, and to a lesser extent, the larger the plasticity index. For this reason, the Atterberg limits tests should be conducted using fresh sample for each testing points and should be limited to only 5min mixing time duration.

From plasticity, liquidity and consistency indices of Table 4.7, it can be said that the soil in its natural state is highly plastic (fat clay) and firm consistency.

Activity of Soils

- From Table 4.8 of activity of soils, for TP-3 and TP-8 have inactive activity. Such types of soils are less prone to volume change as a result of shrinkage or swelling. The soils fromTP-1 and TP-6 their activity is Normal. But for gray and black soils from TP-2, TP-4, TP-5 and TP-7 their activity varies from normal to active. They show volume change upon wetting and drying due to high colloidal activity.

Activity values for soil samples from TP-3, TP-6 and TP-8 are less than 1.25. Activity greater than 1.25 is an indication of active soils. Laterite soils are inactive or normal due to the sesquioxides suppress the activity of the clay particles [14]. In general, the activities of Jimma soils vary from 0.55 to 1.43 and are categorized as inactive to active.

Geochemical tests

The test results in Table 4.9 show that the soils under investigation; TP-3-2 have silica – sesquioxide (S-S) ratio below 1.33 which is laterite. TP-6-1 has S-S ratio of between 1.33 and 2 which is in the state of laterization or lateritic. However, for TP5-2, though the S-S ratio is less than 2 it is not lateritic. It is black in colour, expansive in nature and the Atterberg limits are not affected by different testing conditions. This indicates that some Jimma soils are laterites, lateritic and non-literite residual soils.

Expansiveness tests

Samples obtained from flat areas of the town part such as TP-2, TP-4, TP-5, and TP-7 are measured to have free swell values of more than 50 %, Table 4.10. Soils having free swell value of 100% may exhibit substantial expansion in the field when wetted under light loading. It is also reported that, in extreme climates, clays with free swell values in the range of 50% have caused considerable damage through expansion [16].

The obtained swelling pressure of Jimma expansive soils is generally in excess of maximum dead load that can be exerted by most lightly loaded structures, i.e. ≅ 48 Kpa [15]. For test pit, TP-2-2, though the free swell is less than 100%, the swelling pressure is much significant to affect the stability of light weight structures under different moisture conditions. Thus the degree of expansion can be categorized as medium to high [15].

From the Plasticity chart classification, Figure 4.4, all the identified expansive soils lay above A-line which are clay with high plasticity.

According to V.D. Merwe and Seed et al. Charts, Figure 4.5 and Figure 4.6, the expansive soils of Jimma which are under investigation are clay soils with active activity as a result their expansiveness behavior varies from high to very high swelling potential.

Classification of soils

From visual observation, the dominant colors for Jimma soils are red, brownish red, black and gray. From the Table 4.12, the soils are classified as clayey and silty clay except test pit TP-6-2 which is silty soil according to Textural Classification System.

From grain size distribution curves, Figure 4.1, one can observe that for all samples more than 50% of the soils passes sieve No. 200(0.075mm) so that generally they can be classified as fine-grained soils. From Figure 4.2, using the plasticity chart of USCS, the soils lay in CH zone which is inorganic clays of high plasticity. And some soils lay below A-line, which are in MH region which is inorganic high plastic silts.

From the Table 4.15, applying AASHTO classification system, the soils are classified under A-7-5 with a group index GI of greater than 20. These types of soils are clay soils and they are not favorable for the construction of sub-grade of roads [5]. From the Plasticity chart of AASHTO classification, Figure 4.3, most of the soils lay within the region of A-7-5 and two results lay in the region of A-7-6. This shows that the soils within the group of A-7-5 their Plasticity Index is less than or equal to LL-30 or their Plastic limit is greater than 30% while for the group A-7-6 their plasticity index is greater than LL-30 or their Plastic limit is less than 30% . Both the soil groups which are classified as A-7-5 and A-7-6 are clayey soils.

Consolidation tests

From the plot of void ratio against applied pressure of Figure 4.7, these settlements (deformations of the soil samples) are not elastic, as one would have by compressing a column of steel or concrete, but rather are the vertical statistical accumulation of particle rolling, sliding, and slipping into the void spaces together with some particle crushing (or fracturing) at the contact points. This portion of the settlement is a state change and is largely non-recoverable upon any removal of load. Any elastic compression of the soil particles is a settlement component that is recoverable upon removal of load. Because this component is usually very small, the so-called elastic settlement recovery upon removal of the load is small [8].

From Table 4.20, the tested samples exhibit an over-consolidated state as a result the soils deform gradually before the applied load reaches the preconsolidation pressure stage. The over-consolidation ratio (OCR), which is preconsolidation ratio divided by vertical overburden pressure, is greater than 2. This shows that the soils have experienced higher effective vertical pressure in their past geological history and/or the soil might be preconsolidated and cemented by chemical and shrinkage action [5]. Their compressibility is relatively low at low stress levels [11]. However, the preconsolidation pressure is higher than expected for the topography like the study areas. This could be due to the additional factor of cementation property of the lateritic soils which enabled to increase their preconsolidation pressure.

The compression and swelling indices of the soils investigated vary from 0.238 to 0.399 and 0.0393 to 0.0947 respectively. Similarly, the preconsolidation pressure of the soils ranges from

210 to 300 kPa. A soil with larger magnitude of compression index indicates that soil has high degree of deformation when an external load acts against the soil mass.
For test pit TP-2-2, TP-4-2 and TP-5-2 their swelling index is higher than TP-1 and TP-3. This shows that the heave potential of the former soils is higher than the latter. So, one can estimate magnitude of a heave for expansive soils using swelling index results.

From square-root time fitting curve of Table 4.21, the coefficient of consolidation of the soils range from 7.24×10^{-3} to 1.15×10^{-1} cm^2/min. A soil cannot have a single coefficient of consolidation throughout loading stage. As the applied loading increases apparently c_v decreases since the particle comes together and at the same time the coefficient of permeability decreases. For the sake of convenience usually average c_v value is taken for the different loading applied. However, this may lead to wrong result. Different Codes and Authors suggest using characteristics value [21] which is a function of mean value less standard deviation of each coefficient of consolidation result.

The coefficient of permeability varies from 3.64×10^{-7} to 8.24×10^{-7} cm/min for the soils under investigation .This shows that the type of the soils are clay soils according to Unified Soil Classification System, which are within the range of 10^{-9} to 10^{-11} order-of-magnitude values in m/s [6]. These soils are also deemed as impervious because their respective coefficient of permeability is less than that 10^{-5} mm/s (10^{-8} m/sec).

Compaction tests

The maximum dry unit weight for the soils under investigation lies within the range of 12.70 to 14.00 (kN/m^3). And the optimum moisture contents are varies from 30% to 37%. As one observe the maximum dry density and optimum moisture content values indicate that the soils investigated can be categorized as fine grained soils with high amount of clay [27].

For expansive soils, care should be taken during compaction when it is used as fill material. Moisture variation should be prevented after compaction to avoid volume change of these soils. For such types of soils as their dry density increases by compaction correspondingly their swelling potential will rise too [15]. The relation between dry density and swelling pressure is direct proportional keeping other factors such as moisture content and applied surcharge loads remain constant [15].

At low water content (less than OMC) the repulsive forces between particles are smaller than the attractive forces, and as such the particles flocculate in a disorderly array. As the water content increases the repulsion between particles increases, permitting the particles to disperse, making particles arrange themselves in an orderly way. Beyond OMC the degree of particle parallelism increases, but the density decreases [20].

From the Table 4.22, a soil density in its natural state (dry field density) is less than the density obtained after compaction (maximum dry density). This is could be due to the reason that the effort applied to dense the soil during compaction activity is higher than the vertical overburden pressure exerted on the soil mass. The soil has lesser void ratio after compaction as it is compared to the field state.
To evaluate the effective and practical use of compaction test results, CBR test can be conducted using the maximum dry density and optimum moisture content obtained from the laboratory test [21].

From the test results which are shown in Table 4.23 and in Figures 4.9 to 4.11, one can see that oven drying samples gives the highest maximum dry densities and lowest optimum moisture content. The effect of drying temperature on compaction gives a significant difference between drying at 105^{o}c and air drying. The effect has been noticeable for the drier side of the optimum moisture content. Up on drying, lateritic soils become more cementious and irreversibly become harder [5].

Unconfined Compression Strength tests

From the Table 4.24 and Figure 4.12, the unconfined shear strength of Jimma soil varies from 85 kPa -315 kPa for the test-pits which are investigated. One can observe and analyze that under the given field condition, TP-5 has medium consistency (q_u is between 50kPa-100kPa), TP-1, TP-2 and TP-4 have stiff consistency (q_u is in the range of 100kPa-200kPa) whereas TP-3 and TP-6 has very stiff consistency (q_u in the range of 200kPa-400kPa) [4]. In TP-3 and TP-6 the amount of water content is lesser and their consistency is very stiff as a result they have failed nearly in shear at ultimate stress. Whereas the amount of water found in other test pits is higher and have failed in compression or bulge.

From the test results, the undrained shear strength of black and gray soils is less than that of red clay soils. This is possibly due to the fact that the cumulative contribution of field density, natural moisture content, preconsolidation state and cementious properties of the red clay soils are better than the black and gray soils. Besides that the field density and natural moisture content of red soils is relatively near to the compacted density and optimum moisture content than gray and black soils [5].

From Table 4.25, for the soils under investigation, the modulus of deformation for average tangent and secant modulus of deformation varies from 3750 to 19,000 kPa and 2160 to 6300 kPa respectively. A soil with lesser modulus of deformation indicates that the soil has lesser strength and is more deform as a load applied against soil mass than the soil with larger modulus of deformation.

From Figure 4.13 through Figure 4.15, one can observe from the stress-strain curve for undisturbed and disturbed samples that the strength of both soil samples has significant difference. This is due to the thixotropic nature of the clay soils. The remolded samples did not regain their full strength and cementation through time. The strength and cementation of the clay soils can increase with aging as a result of gradual reorientation of the absorbed molecules of water [5].

For all four samples, the nature of the stress-strain curves for the undisturbed samples, unlike for remolded samples, exhibit peak shear stresses at small strain level. For remolded samples, the shear stress increases gradually till the ultimate stress is reached. From the nature of the curves for undisturbed samples, it show that the soils are over-consolidated clays and stiff clays while for remolded soil samples since the natural soil structure is disturbed during remolding process it becomes normally-consolidated clay soil and softer clay [5].

The strain failure criteria for over consolidated clay soils (stiffer soils) are usually smaller and are between 2% to 5% while for normally consolidated clay soils (softer soils) the failure criteria is mainly within 10% to 16% of shear strain [5].

From Table 4.27, all the soils which are under investigation can be described as medium to low sensitive clays. These clays are said to be less sensitive to remolding and during construction operations less disturbance tends to transform to the soil mass.

5.2 Comparison of Test Results with Previous Works

5.2.1 Index Properties Comparison

Though little research works are done on Jimma soils, the comparison is made between the test results studied previously with the results of current research work.

Under this research work, specific gravity, grain size analysis, index properties, compaction, consolidation and unconfined compression strength tests are done. Comparisons are made with properties of soils that are previously studied on Jimma soils. These comparison elements are color, specific gravity, percentage of fines, density, liquid limit, plastic limit, plasticity index, classification according to UCS and undrained shear strength. The results are presented in Table 5.1, Table 5.2 and Table 5.3.

Some of Jimma residual soils found that they are lateritic in nature. This property was to some extent investigated in the previous research work [2].

5.2.2 Result Comparison with Expansive soils found in Ethiopia

Results of the present study on Jimma Soils are summarized together with ranges of values of expansive soils found in Ethiopia from Addis Ababa, Bahir Dar and Mekelle. The result is presented in Table 5.4.

5.2.3 Comparison with Lateritic Soils

For the soil under investigation, Index properties and Strength tests were studied and comparisons have been made with known lateritic soils of Nejo-Mendi, Asossa and Wolayita-Sodo. Their properties for the lateritic soils from Nejo-Mendi, Asossa, Wolayita-Sodo and Jimma soils are shown in Table 5.5.

5.2.4 Evaluation of Correlation Formulas against Laboratory Test Results

Evaluation of correlation is done to assess the applicability of previous empirical correlation formulas developed for other soils with the current laboratory test results of Jimma soils. This includes determining different soil parameters such as compression index, preconsolidation pressure and other soil coefficients. The comparisons are summarized in Table 5.6 and Table 5.7.

Table 5. 1 Comparison of index properties and classification of Jimma soils with the previous works [2]

Elements of comparison / Research work	Colors	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic index, PI (%)	Liquidity index, LI	UCS Classification	AASHTO Classification	Remarks
Previous	Black, Gray, Brown, Red	60 – 109	22 - 44	31 – 77	(-0.13)- (0.60)	CH	-	
Current	Black, Gray, Red	53 – 108	25 - 41	21 – 67	(-0.06) - (0.47)	CH, MH	A-7-5, A-7-6	The lowest and the highest values are taken

Table 5. 2 Comparison of specific gravity, density, percentage of fines, NMC and shear strength of Jimma soils with the previous research work test results [2]

Elements of comparison / Research works	Specific Gravity, Gs	Field Density,γ_b (kN/m^3)	Percentage Fines (%)	NMC (%)	Undrained Shear Strength, c_u (kPa)	Remarks
Previous	2.50 – 2.82	14.30 –17.80	65 – 96	31.0 – 64.7	32.2 – 324.6	The NMC is considered the water content at the time of UCS test done.
Current	2.58-2.81	16.75- 17.98	61 – 99	33 – 56	43 – 157	

Table 5. 3 Effect of pre-treatment of samples (Air drying and Oven drying conditions) on Atterberg limits for lateritic Jimma soils [2]

Elements for Comparison / Research works	Change in natural moisture content (%)	Change in liquid limit (%)	Change in plastic limit (%)	Remarks
Previous	4.2 – 5.8	-	-	
Current	4 – 7	5 – 8	3 – 7	

Table 5. 4 Comparison of property ranges of Jimma expansive soils with other expansive soils found in Ethiopia.

Item	Location			
	Jimma (current study, [2])	Addis Ababa* (Teklu et al. [23])	Bahr Dar [24]	Mekelle [23]
Sand, %	1-7	2 – 16	0.61- 17.1	3.8-19.2
Silt, %	42-51	9 - 46.40	10.23- 26.88	34.8-69.5
Clay, %	40-59	30 – 70	58.1-87	20.8-60.2
Liquid Limit, %	72-108	63 -108	78.5-112.05	48.6-89.7
Plasticity Index, %	36-68	45 - 76.4	46.46- 6.42	25.1-70.6
Specific Gravity, Gs	2.58-2.72	2.77-2.88	2.55-2.81	2.4-2.78
Free swell, %	80-160	51.8- 118	78-200	22-127
Swelling Pressure, kPa	135-210	36.9-420	80-520	50.2-262.9
Unconfined Compressive Strength(UCS), kPa	85-285.6	96.7-267.2		352.4-565.9

*Note: For Addis Ababa case, the finding for swelling pressure and UCS of two individuals is considered

Table 5. 5 Table comparison of lateritic soil properties for the Jimma and other parts of Ethiopia

Item \ Location	Nejo-Mendi	Asossa	Wolayita-Sodo	Jimma
Author	Abebaw [14]	Fekede [25]	Hana[26]	Current study
Soil Type	Lateritic	Lateritic	Lateritic	Lateritic
Clay Content, %	2.0-20.6	2.5- 60	48- 69.7	19-80
Activity	0.97- 0.98	0.62- 1.02	0.317- 0.488	0.55-1.16
Liquid Limit, %	48-67	41-72	48-71	53-80
Plasticity Index, %	17-27	20-48	19-30	22-46
Specific Gravity, Gs	2.78-3.03	2.19-2.94	2.61-2.97	2.65-2.81
Free swell, %	20-40	11-45	28-38	40-50
Unconfined Compressive Strength(UCS), kPa	165-553	-	215-385	286-314
Classification from Plasticity chart	MH	CH,SC,MH, CL&SM	MH	MH,CH
Silica to Sesquioxide ratio	0.35-0.41	-	1.07-1.50	1.12-1.79

Table 5. 6 Evaluation of correlations with laboratory results for compression index of Jimma Soils

Ser.No.	Location	Test Pit designation	Initial void ratio, e_o	Natural moisture content, NMC (%)	Liquid limit, LL (%)	Cc from Conso-lidation test	Terzaghi and Peck, [5], Cc = 0.007(LL-10)	Azzouz et al. [5], Cc= 0.37 (e_o + 0.003 LL + 0.0004 NMC - 0.34)	Hough [4], Cc = 0.3 (e_o-0.27)	Deviation from Hough (%)
1	Merkato-1	TP-1-2	1.11	32	65	0.238	0.386	0.360	0.251	5
2	Merkato-2	TP-2-2	1.28	46	72	0.336	0.431	0.435	0.304	10
3	Awetu	TP-3-2	1.29	42	77	0.278	0.467	0.443	0.306	10
4	Technic Sefer	TP-4-2	1.19	41	92	0.291	0.576	0.424	0.277	5
5	Kebele-5	TP-5-2	1.26	56	108	0.399	0.686	0.469	0.297	26

Table 5. 7 Evaluation of correlations with laboratory results for preconsolidation pressures of Jimma Soils

Ser.No.	Location	Test Pit Designation	Natural moisture content, NMC (%)	Liquid Limit, LL (%)	Overburden Pressure, σ_v (kN/m2)	Undrained shear strength, c_u (kpa)	Preconsolidation pressure from Laboratory test, P_c (kPa)	Preconsolidation pressure, P_c= 3.78c_u -2.9 (kPa)[5]	Deviation in (%)
1	Merkato-1	TP-1-2	32	65	43.38	60	230	224	3
2	Merkato-2	TP-2-2	46	72	42.87	68	-	252	-
3	Awetu	TP-3-2	42	77	43.63	157	300	591	49
4	Technic Sefer	TP-4-2	41	104	41.66	65	-	-	-
5	Kebele-5	TP-5-2	56	108	43.03	54	-	-	-

[5]Nagaraj and S. Murthy for soils pre-consolidated by cementation and shrinkage [5]

5.2.5 Discussion for Result comparison with the previous works

Table 5.4 shows that the black and gray expansive soils of Jimma lay within the range of indicative properties of expansive soils of Ethiopia [4]. In the other hand, Jimma expansive soils show similar properties with that of expansive soils found in Ethiopia.

From comparison Table 5.5 the lateritic soils of Jimma have similarity in properties with that of Wolaita-Sodo soils as it is compared to other lateritic soils of Ethiopia. From their engineering and geochemical test results, the soils of Mendi-Nejo are typical classified as Laterite soils while the others are lateritic or they are in state of laterization.

From Table 5.6, a comparison of compression index (Cc) is made on the results obtained from laboratory tests and from empirical correlation formulas which are developed by different authors [4, 5]. The Cc obtained from Hough's formula gave closer values with the laboratory test results, whereas the results from Terzaghi and Peck, Azzouz et al., gave higher values. This may be due to the fact that the correlations developed by these authors are applicable for the type of the soils from which the samples were taken. Such types of correlation are dependent on among other factors like sampling location and field conditions.

From Table 5.7, the preconsolidation pressure obtained from Nagaraj and Murthy [5] is relatively closer to the laboratory results exceptional for test pit TP-3-2. So these soils close to the correlation results may be preconsolidated from the consequence of cementation and shrinkage.

6 CONCLUSION AND RECOMMENDATIONS

6.1 Conclusions

Based on the test results obtained the following conclusions may be drawn:

- From visual observations and field tests, the soils of the area are classified as clay with high plasticity. According to Textural Classification System the soils studied are classified as silty-clay to clay. And from indices classification criteria these soils can be labelled as clay with high plastic and firm consistency.
  According to the Unified Soil Classification System (USCS) and AASHTO the soils are classified as CH (clay with high plasticity) and G-7-5(clayey soils) respectively.

- Mechanical grain size analysis shows that more than 90% of the total masses pass through sieve size of 75µm. This indicates that almost all soils can be taken as fine-grained soils. From hydrometer analysis, for most of the samples, more than 50% of their grain sizes are less than 5µm (clay size range as per ASTM boundaries criteria). This suggests that these types of soils are highly influenced by the presence of clay content.

- The specific gravity of the studied soils ranges between 2.58 to 2.82. This indicates that the soils are inorganic and for the lateritic soils investigated, their specific gravity is higher. This is due to the presence of significant amount of iron-oxide in the mineral content of the lateritic soils.

- The result of Atterberg limits and compaction tests for lateritic soils are affected by test procedures and sample treatment conditions. For conducting Atterberg limit tests, the mixing time of the sample should be limited to 5 minute and it should be air-dried.

- Geochemical tests indicate that the soils of TP-3 and TP-6 are laterites and lateritic respectively having variable concentration of iron oxide and aluminium oxide /sesqueoxide/. The degree of laterization of these soils ranges between1.12 to 1.79. Therefore, the lateritic soils found in Jimma need special attention during sample preparation and testing.

- Atterberg limit and swelling pressure tests show that Jimma Soils from sample TP-2, TP-4, TP-5 and TP-7 are found expansive. The degree of expansiveness varies from medium to very high. Their percentage of free swell extends from 80% to 130% which would have an adverse effect on light weight ordinary structures.

- From topographic distribution, expansive soils are found in the flat and poorly drained areas while lateritic soils are found in the slopy and well drained areas of the town part.

- The ratio of preconsolidation pressure with respect to overburden pressure shows the soils are over-consolidated with OCR greater than 2. The preconsolidation pressure of the soils ranges from 210 to 300 kPa. This indicates the soil of the study area is over consolidated residual soil. The compression and swelling indices of these soils varies from 0.238 to 0.399 and 0.0393 to 0.0947 respectively. The characteristic coefficient of consolidation extends from $1.15x10^{-1}$ to $7.47x10^{-3}cm^2/min$.

- The undrained shear strength varies from 54kPa to 157kPa for undisturbed samples whereas for remolded samples the strength varies from 26kPa to 90Kpa. This is due to the difference between cementation, ageing effect, inter-mineral bond and loading history of the soil for undisturbed and disturbed samples.

- Sensitivity test shows that the soils are medium to less sensitive. This indicates that remolding doesn't affect the soil strength.

- Tangent modulus and secant modulus of deformation for the undisturbed samples varies from 3750 to19000kPa and 2160 to 6300kPa respectively. These results may be used to calculate the Magnitude and rate of settlement of soils under different prescribed stresses on soil masses.

- Comparison made with the previously done research works show that the engineering properties of the soils of the study area lie within the range given by the different investigators in the country.

- From the evaluation of existing empirical formulas, some correlations agree with minor deviation from laboratory test results while for most of the correlations their values deviate significantly from the laboratory test results for soils of the study area. Hence, it can be concluded that before the adoption of these correlations proper verifications are required to taste their applicability for local soils.

6.2 Recommendations

- Further detailed investigation has to be carried out on a number of additional disturbed and undisturbed samples from different locations of the town to prepare a reliable geotechnical map of Jimma.
- The engineering properties of expansive soils and lateritic soils have to be studied in detail including their strength parameters.
- For a complete presentation of the engineering properties of the soil in the town dynamic properties of the soils should also be studied.

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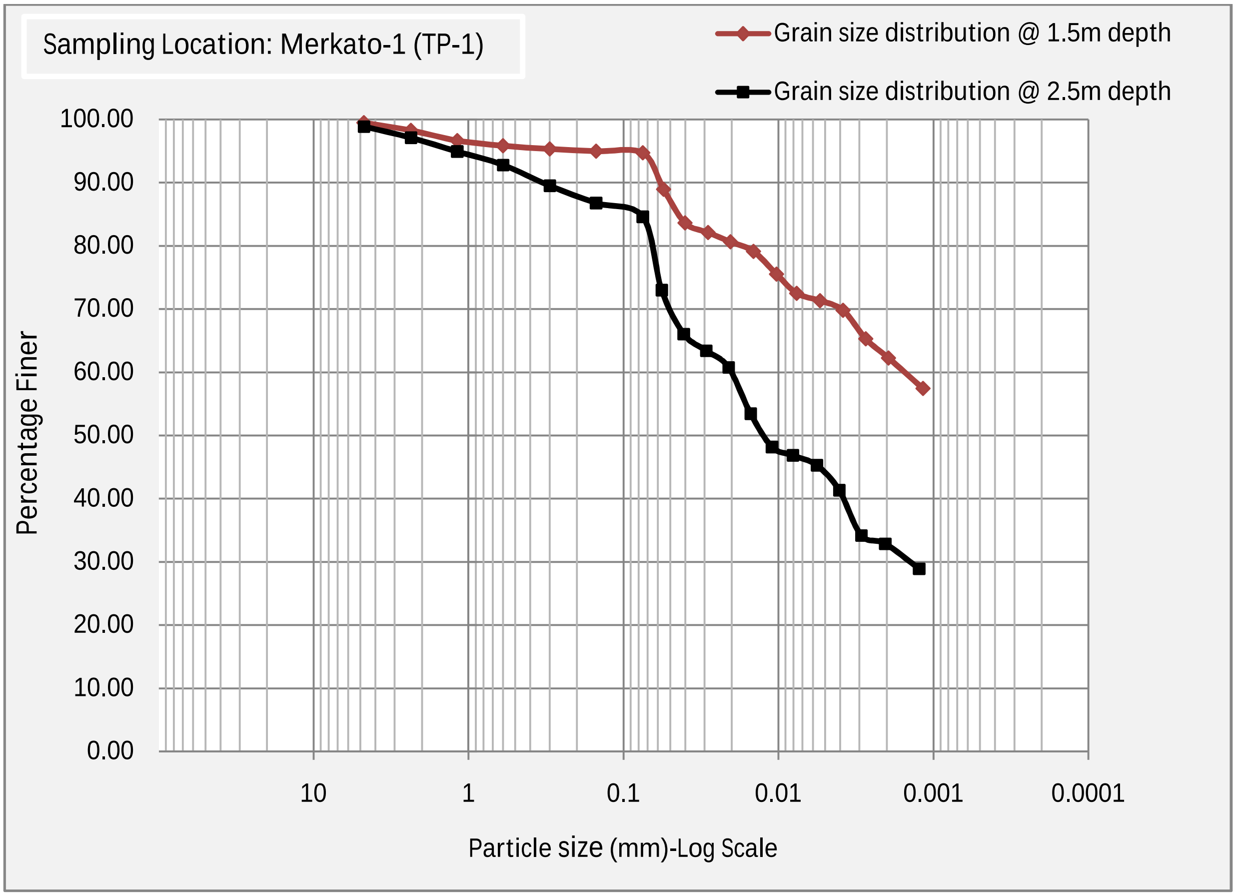
APPENDICES

Appendix– A

Grain Size Distribution Curves

Figure A.1 Combined grain size analysis test result for TP-1



Note: According to ASTM the following grain size boundaries are applied.

	Gravel	Sand	Silt	Clay	Colloids

75mm **4.75mm** **0.075mm** **0.005mm** **0.001mm**

Figure A.2 Combined grain size analysis test result for TP-2

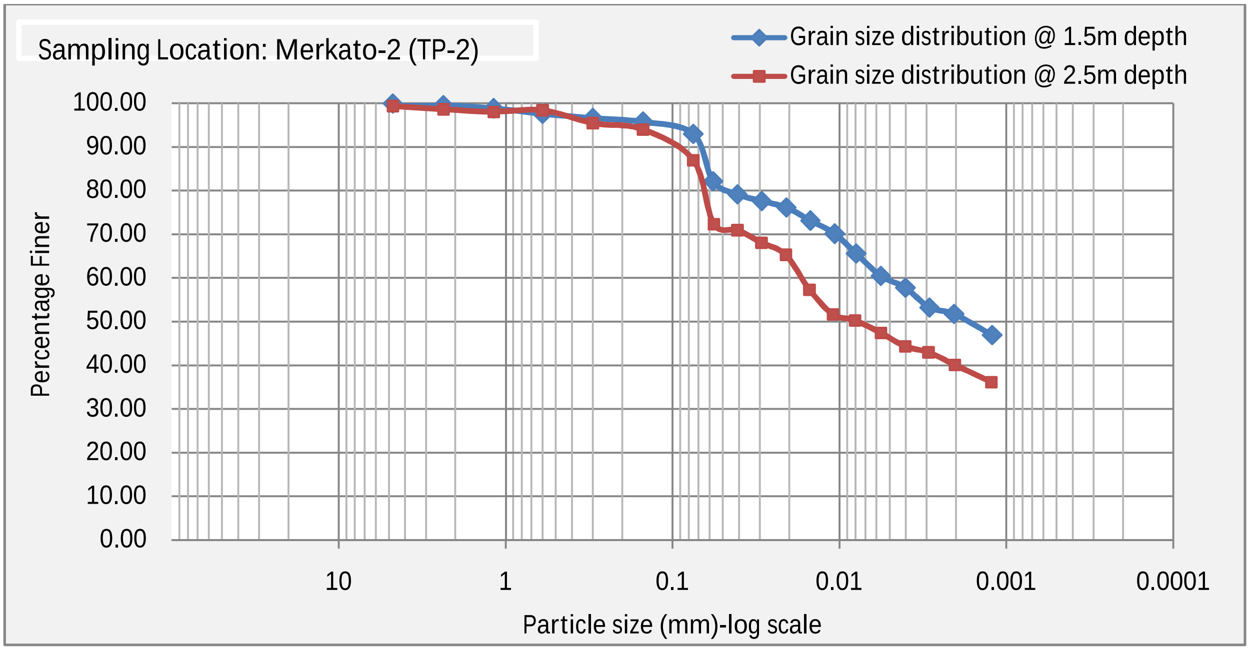


Figure A.3 Combined grain size analysis test result for TP-3

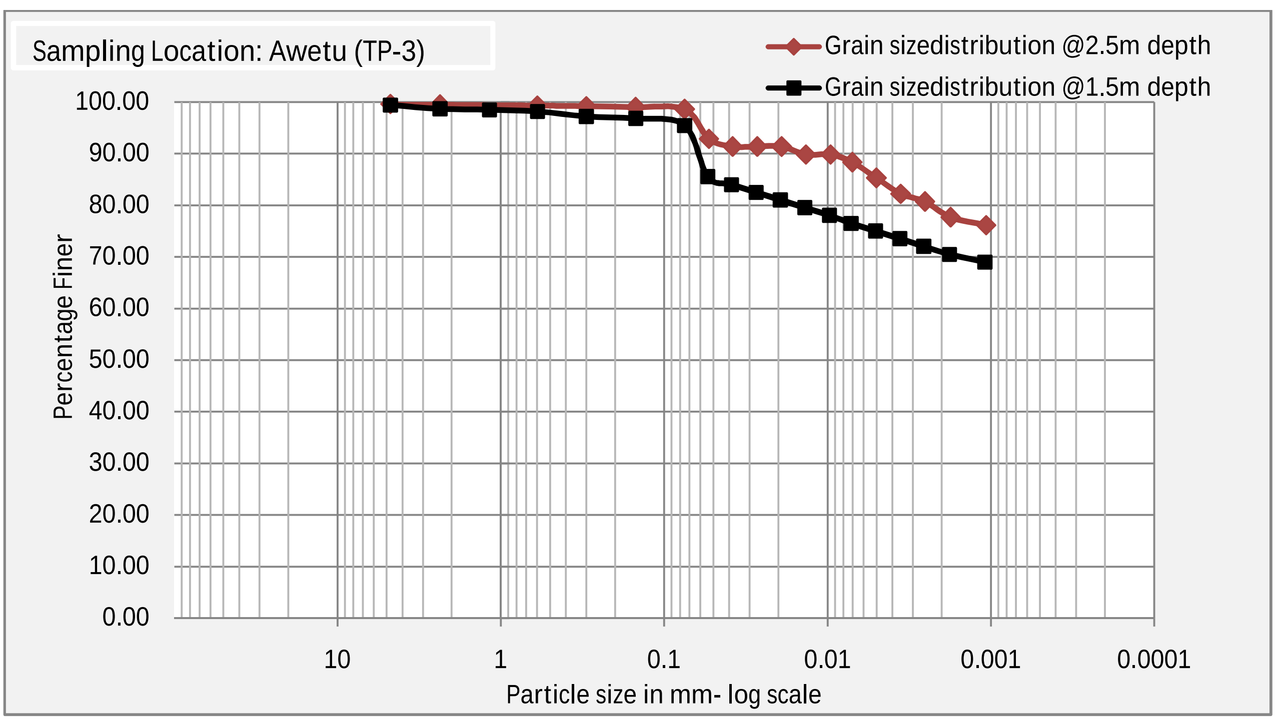


Figure A.4 Combined grain size analysis test result for TP-4

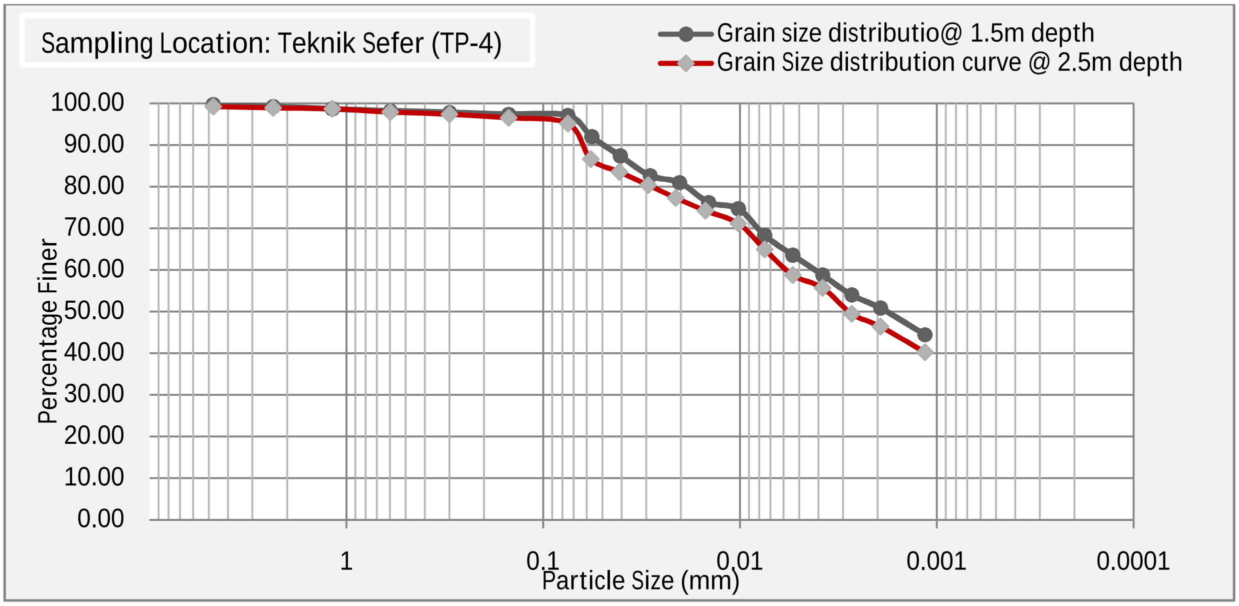


Figure A.5 Combined grain size analysis test result for TP-5

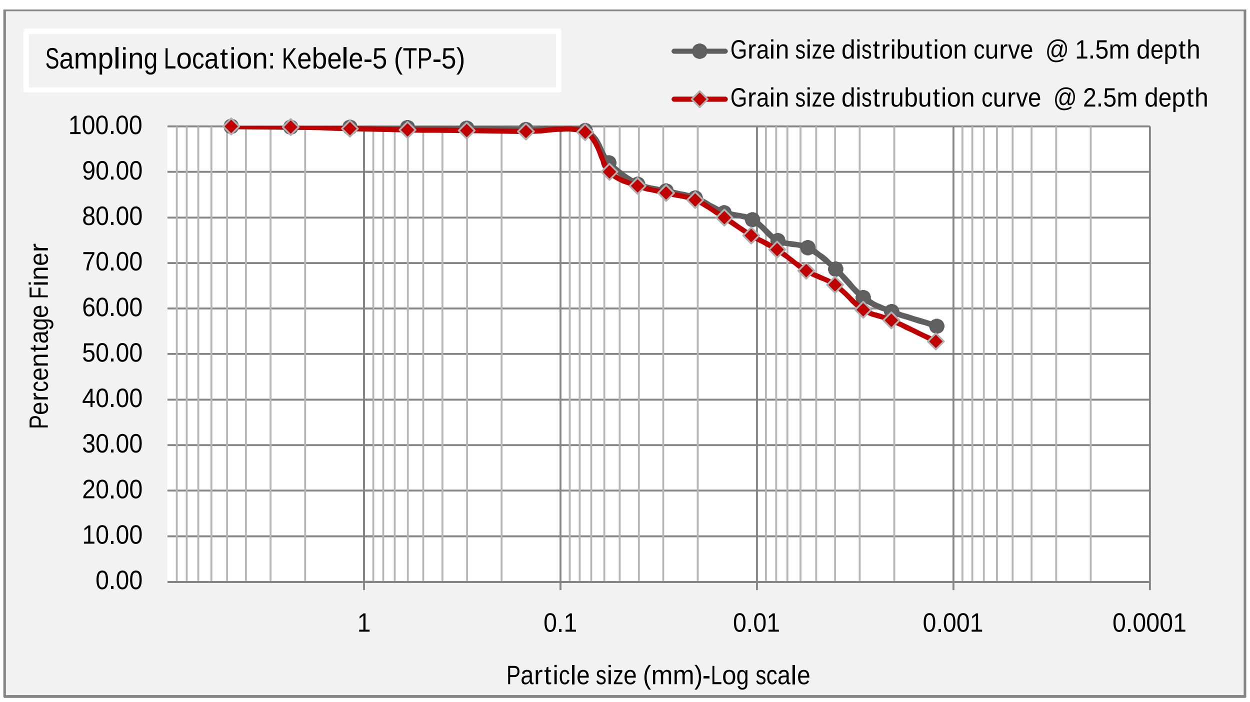


Figure A.6 Combined grain size analysis test result for TP-6

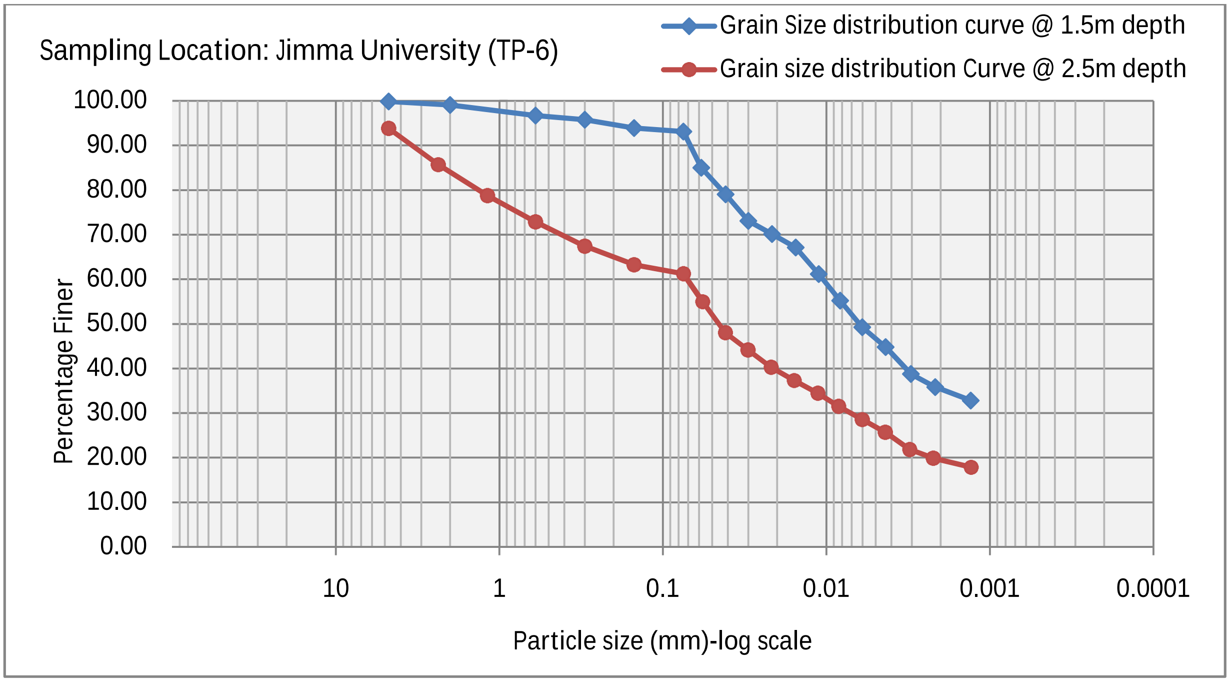


Figure A.7 Combined grain size analysis test result for TP-7

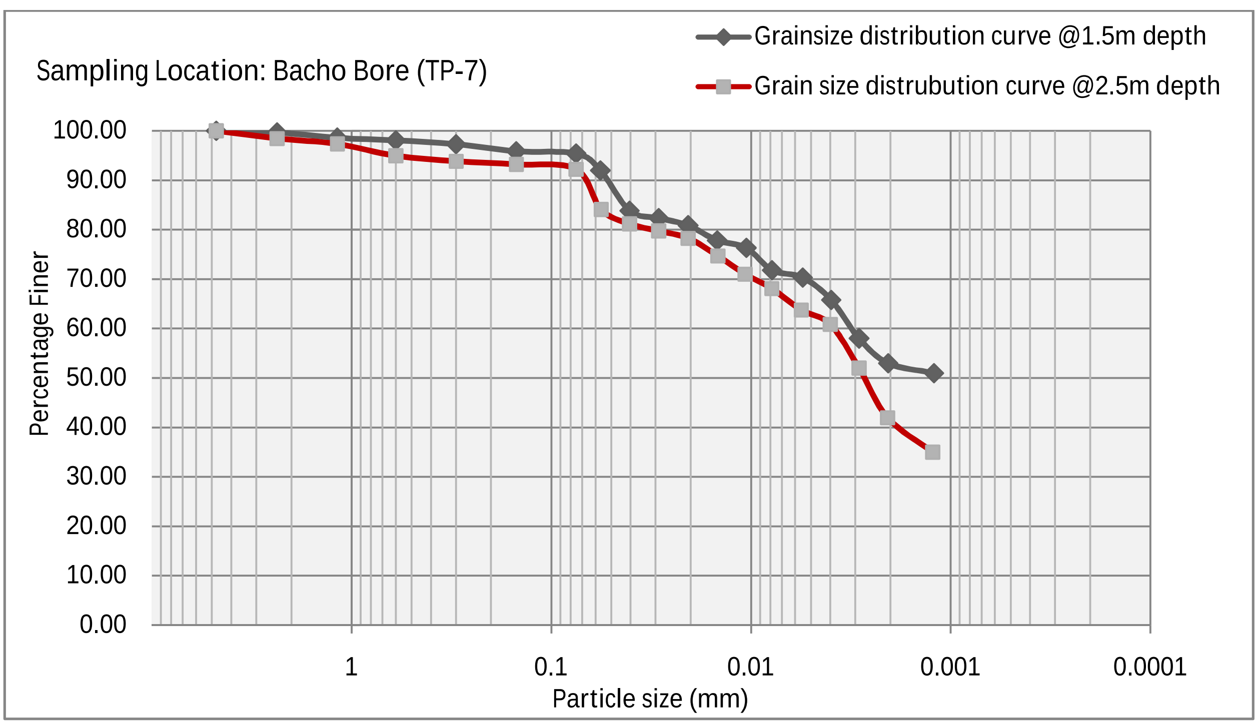
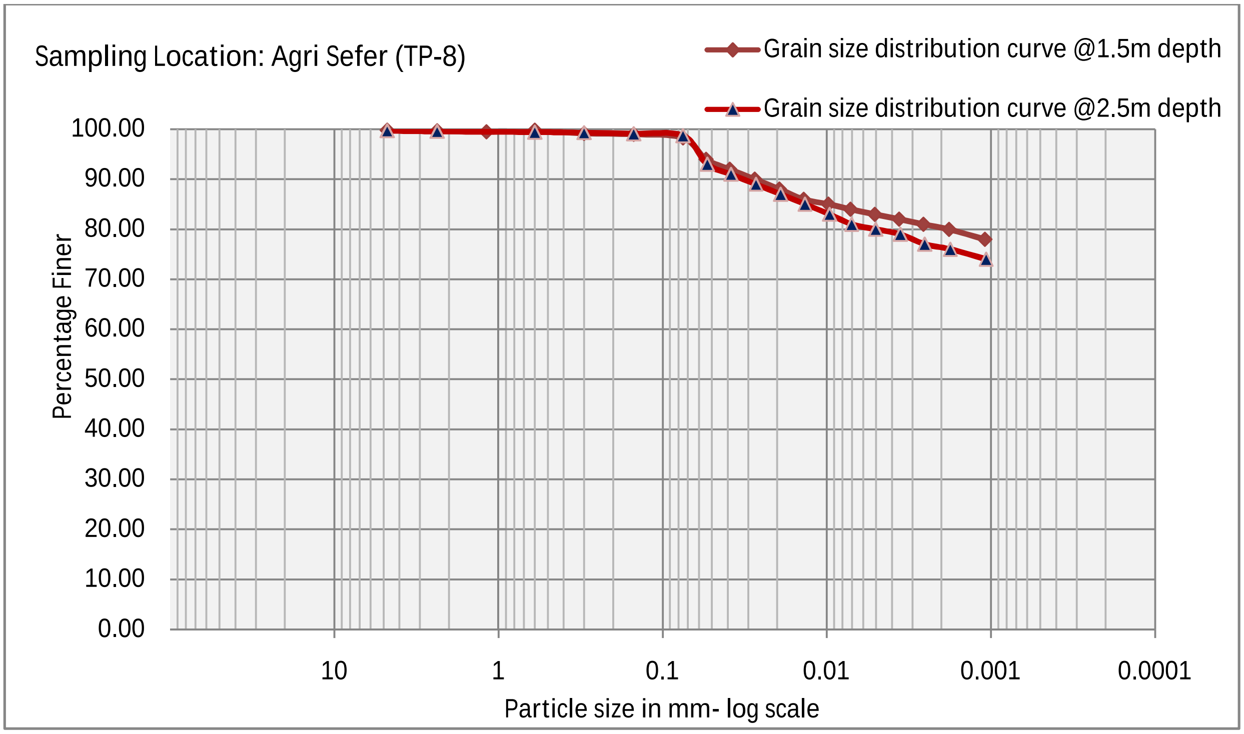


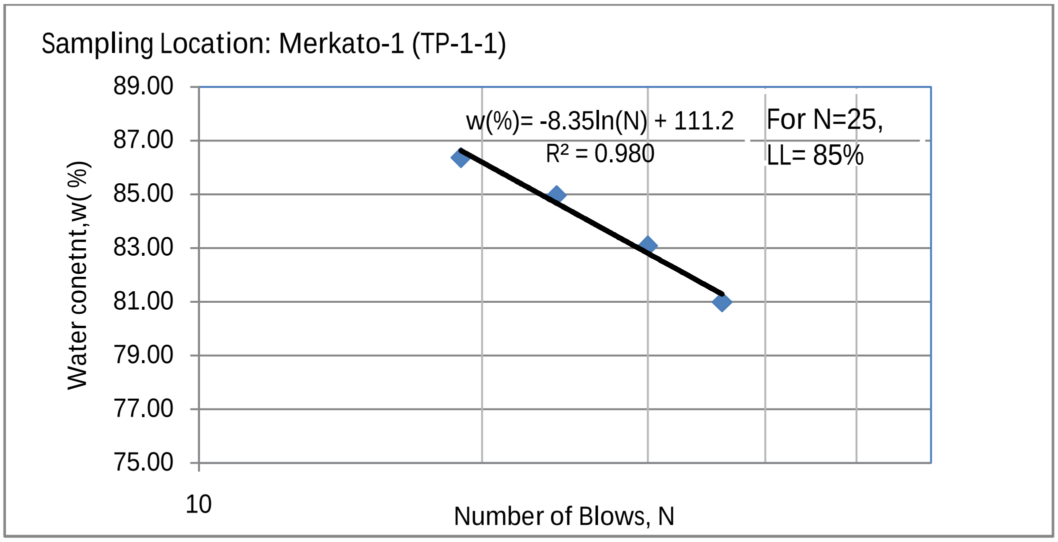
Figure A.8 Combined grain size analysis test result for TP-8



Appendix – B

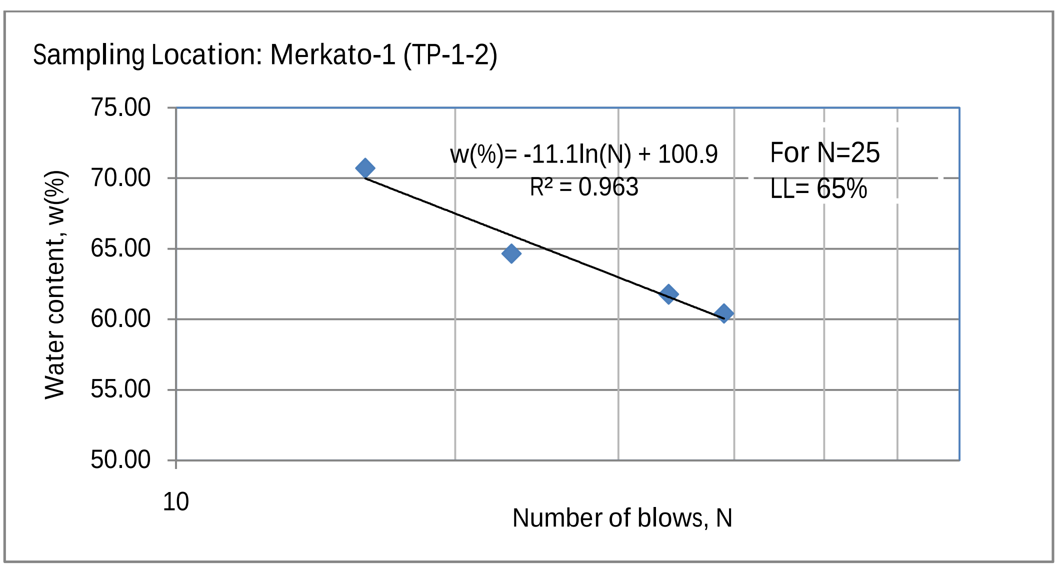
Atterberg Limit test Results

Figure B.1 Flow curve analysis and plastic limits for TP-1-1



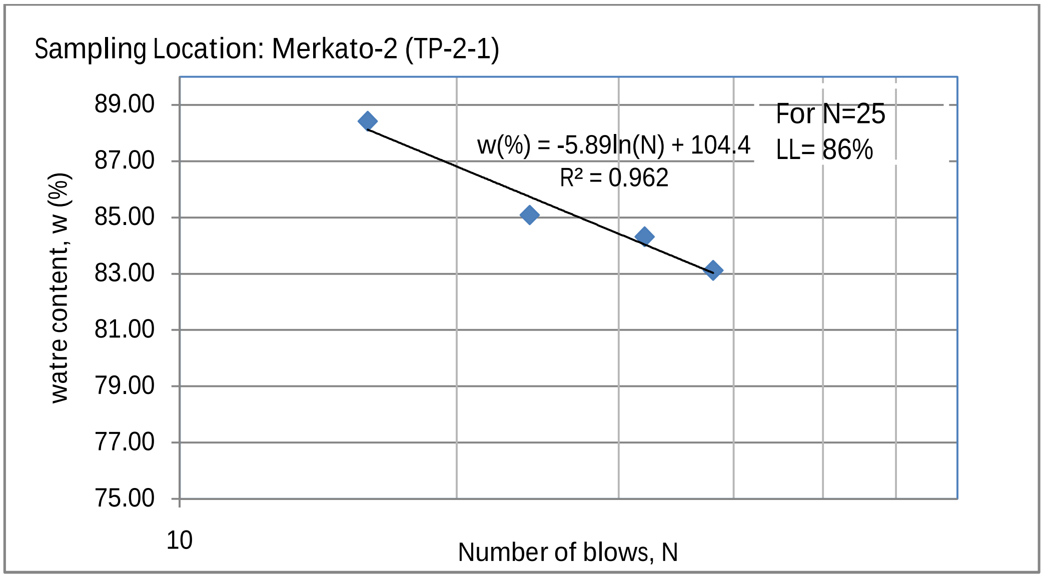
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 35\%}$

Figure B.2 Flow curve analysis and Plastic limit for TP-1-2



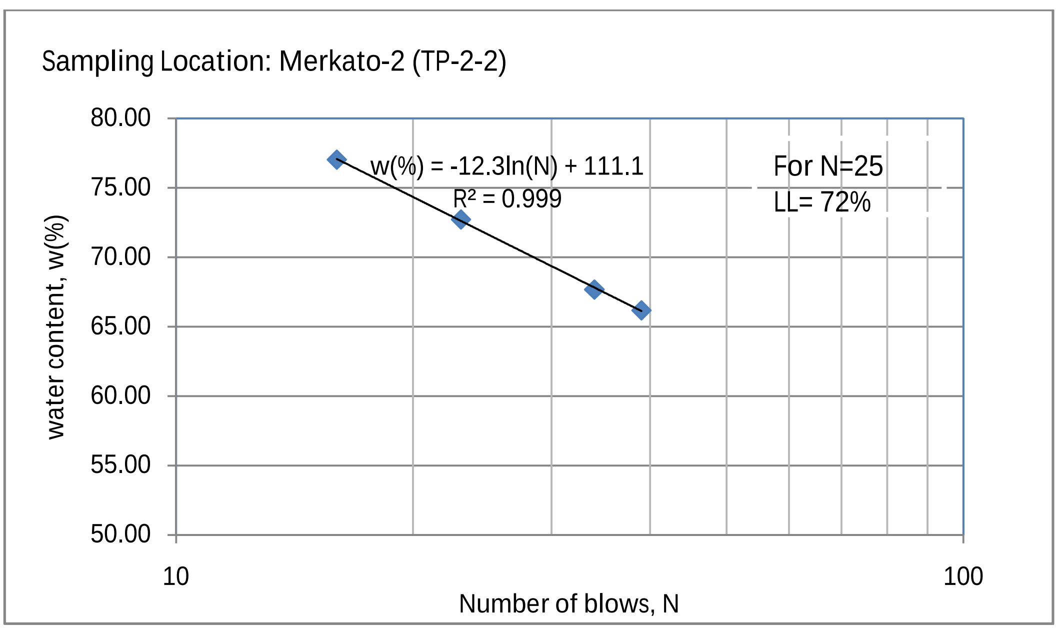
And plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 30\%}$

Figure B.3 Flow curve analysis and Plastic limit for TP-2-1



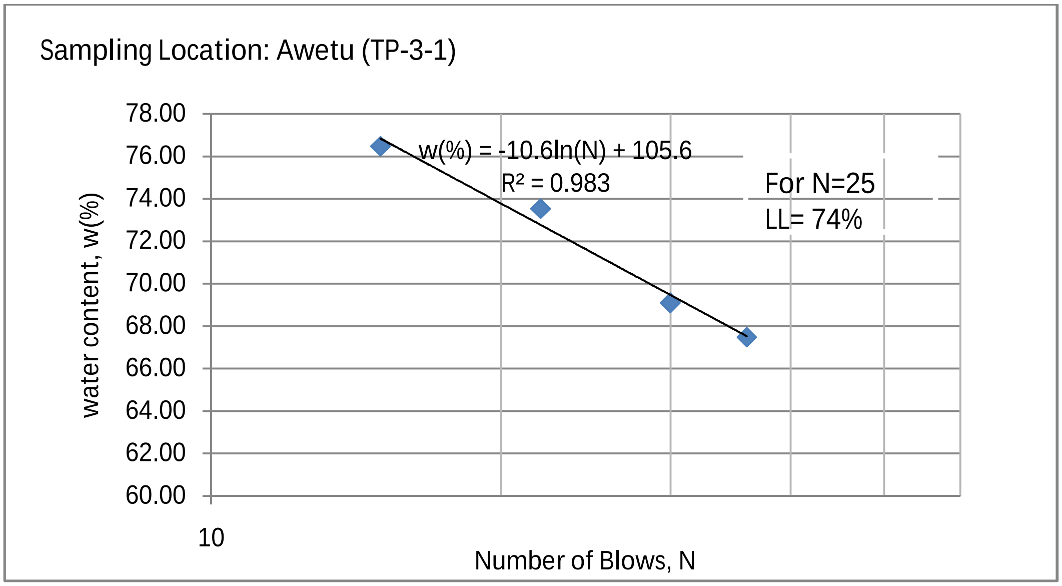
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 34\%}$

Figure B.4 Flow curve analysis and Plastic limit for TP-2-2



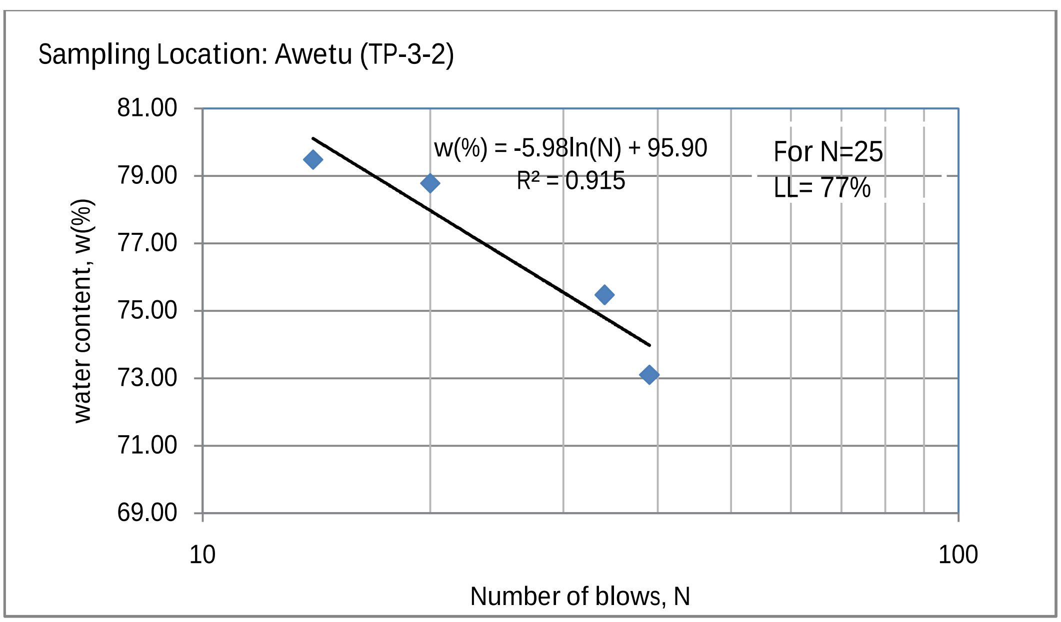
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 32\%}$

Figure B.5 Flow curve analysis and Plastic limit for TP-3-1



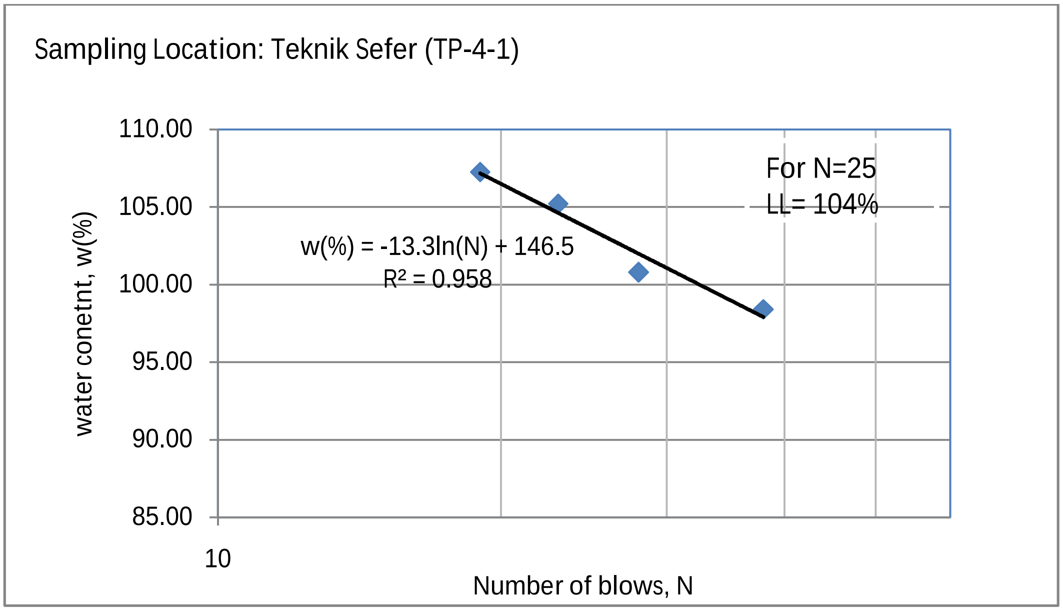
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 27\%}$

Figure B.6 Flow curve analysis and Plastic limit for TP-3-2



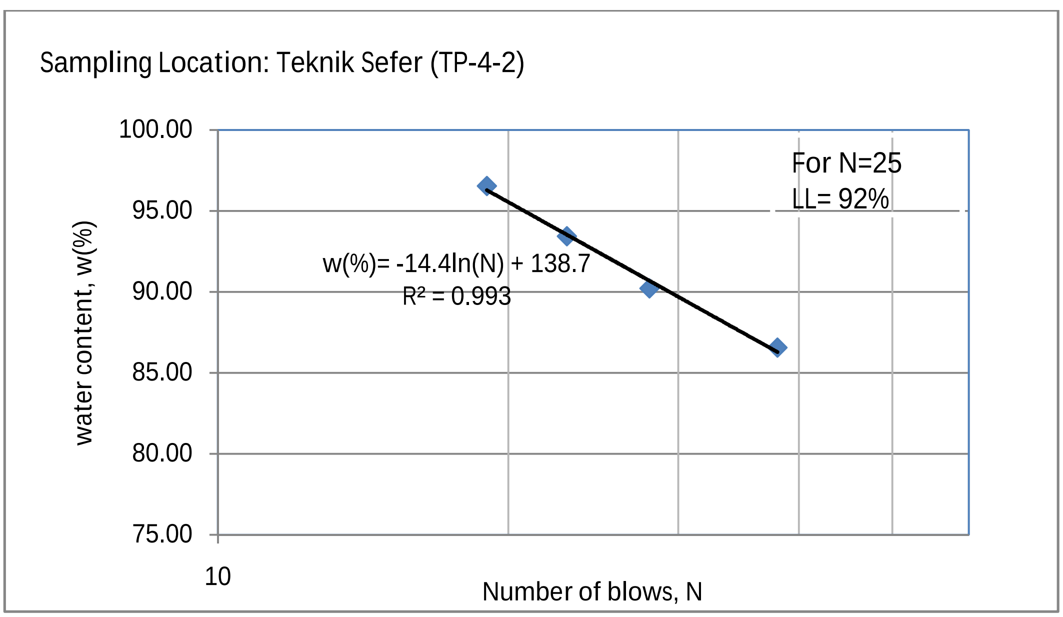
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 32\%}$

Figure B.7 Flow curve analysis and Plastic limit for TP-4-1



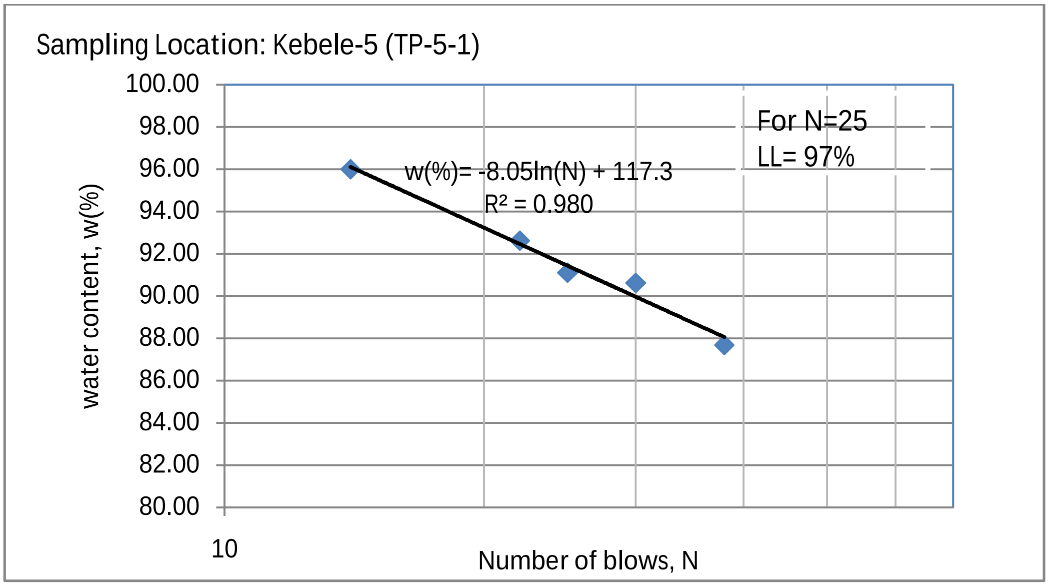
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg}}$ **= 40%**

Figure B.8 Flow curve analysis and Plastic limit for TP-4-2



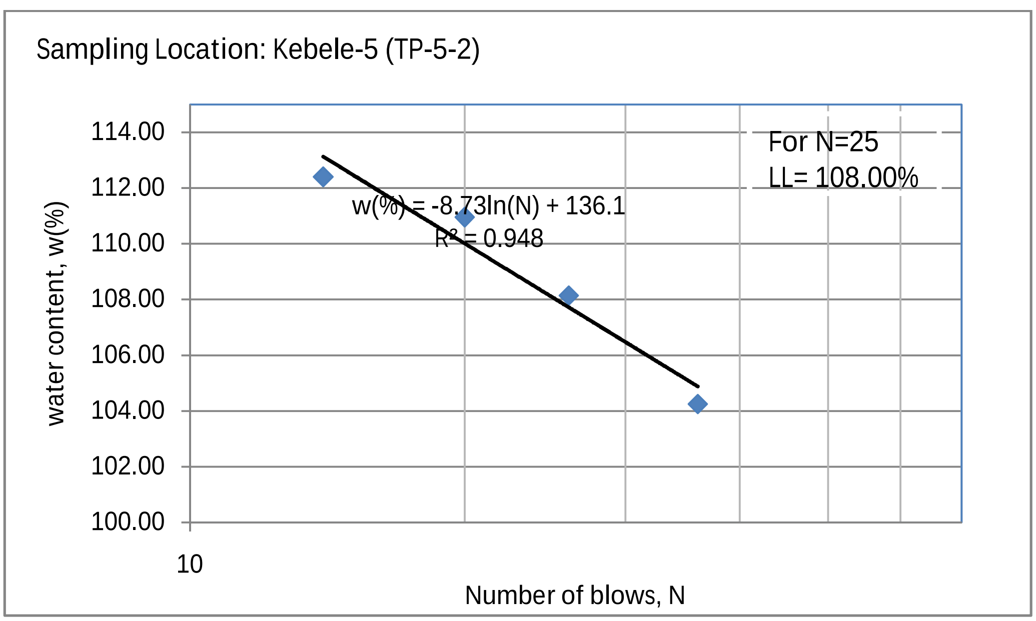
Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg}}$ **= 35%**

Figure B.9 Flow curve analysis and Plastic limit for TP-5-1



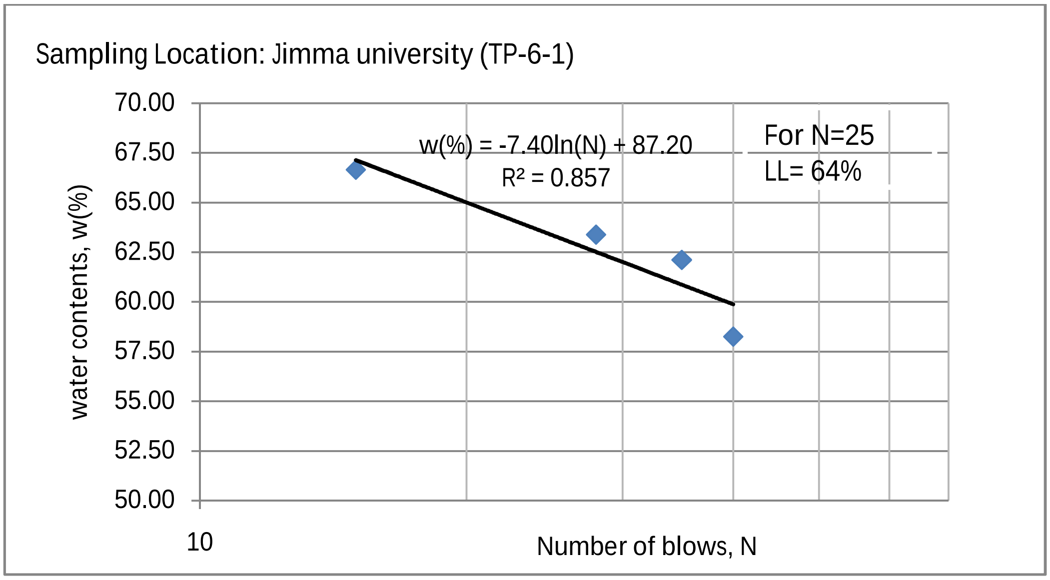
Plastic limit at 3mm rolling into thread, **PL_{avg} = 38%**

Figure B.10 Flow curve analysis and Plastic limit for TP-5-2



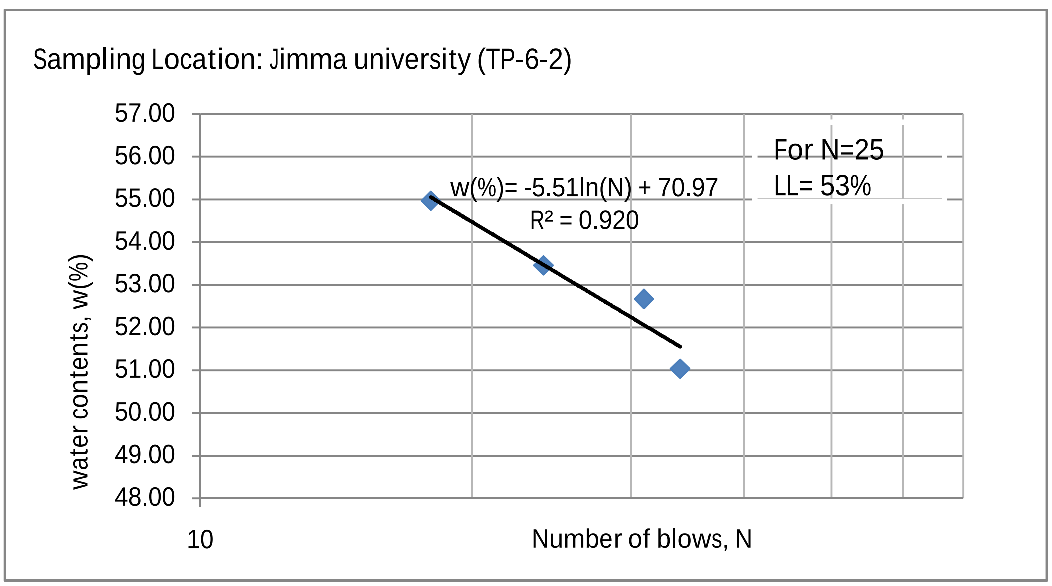
Plastic limit at 3mm rolling into thread, **PL_{avg} = 40%**

Figure B.11 Flow curve analysis and Plastic limit for TP-6-1



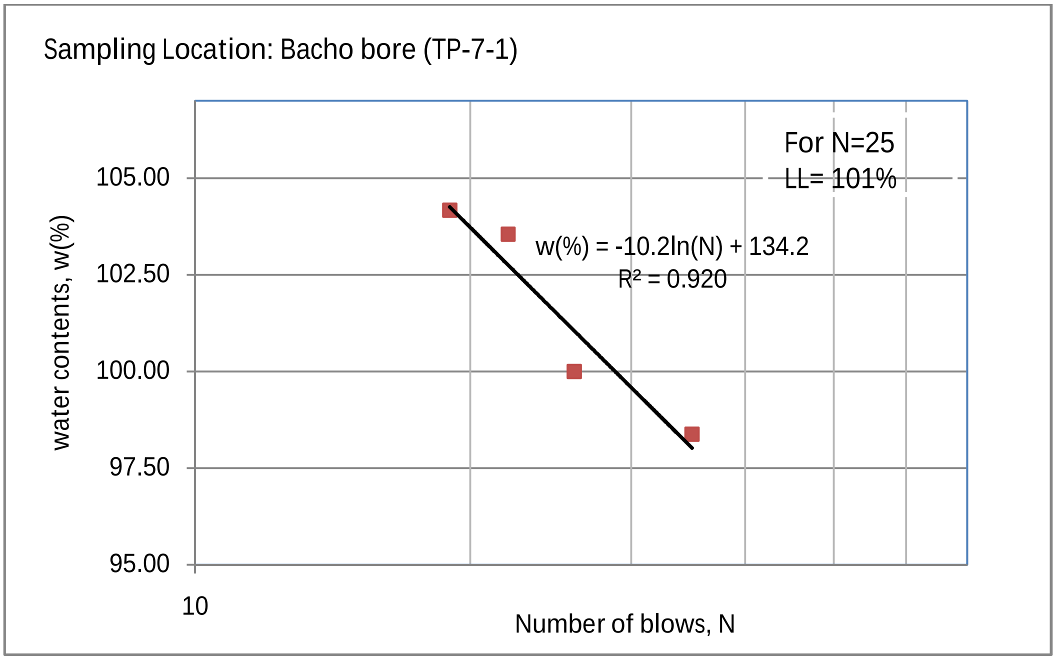
Plastic limit at 3mm rolling into thread, **PL_{avg} = 35%**

Figure B.12 Flow curve analysis and Plastic limit for TP-6-2



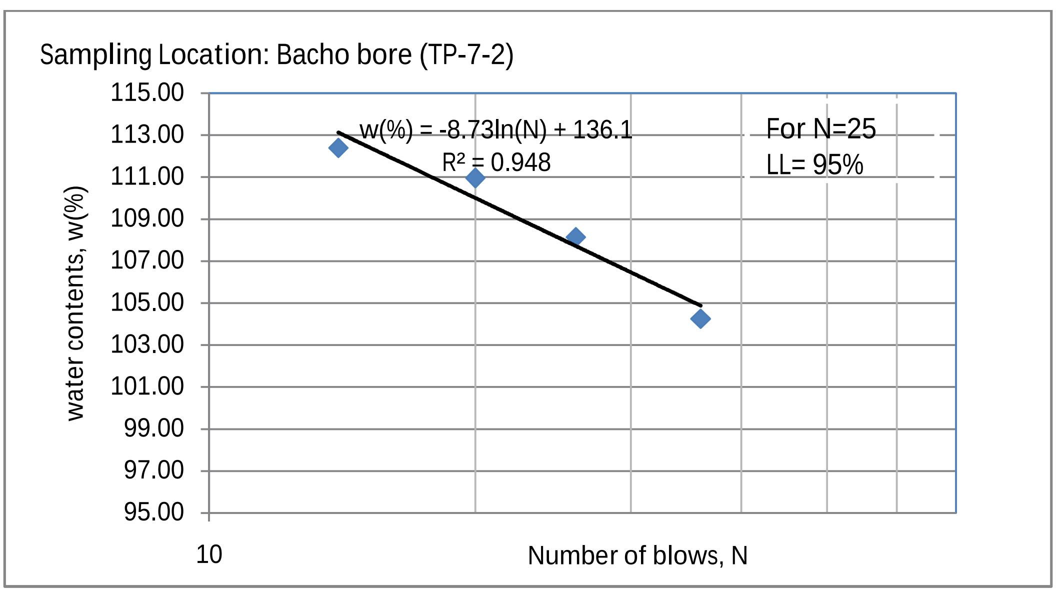
Plastic limit at 3mm rolling into thread, **PL_{avg} = 31%**

Figure B.13 Flow curve analysis and Plastic limit for TP-7-1



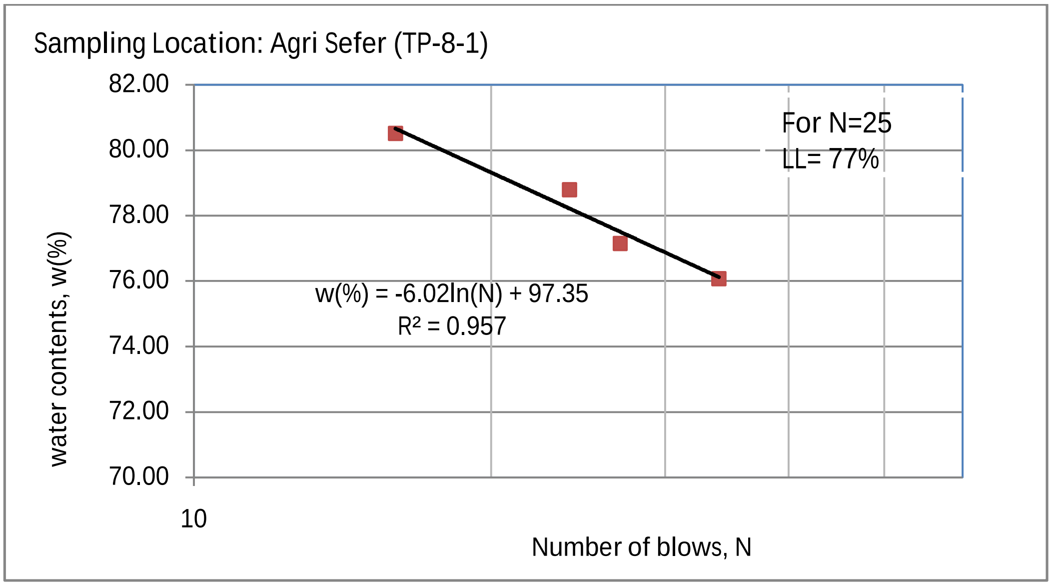
Plastic limit at 3mm rolling into thread, **PL_{avg} = 39%**

Figure B.14 Flow curve analysis and Plastic limit for TP-7-2



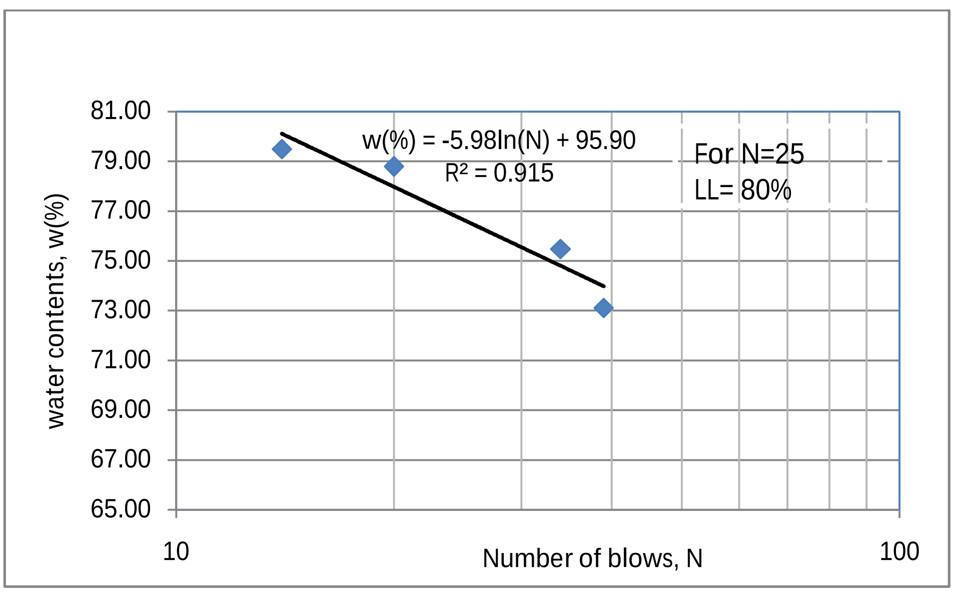
Plastic limit at 3mm rolling into thread, **PL_{avg} = 35%**

Figure B.15 Flow curve analysis and Plastic limit for TP-8-1



Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 39\%}$

Figure B.16 Flow curve analysis and Plastic limit for TP-8-2



Plastic limit at 3mm rolling into thread, $\mathbf{PL_{avg} = 41\%}$

Appendix –C

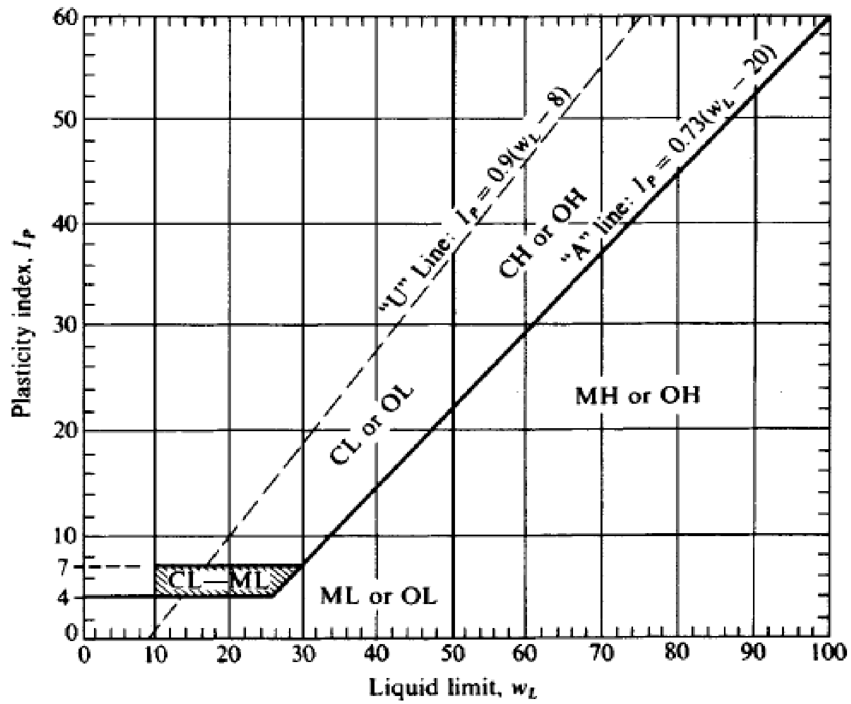
Classification Tables and Charts

Table C-1 Unified Soil Classification Sytem (USCS) ,Cassagrande [5]

Major divisions			Group symbols		Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentages of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP; More than 12% GM, GC, SM, SC; 5 to 12 % *Borderline* cases requiring dual symbols†	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{10}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting C_u or C_c requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM*	*d*	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or I_P less than 4	Limits plotting in hatched zone with I_P between 4 and 7 are *borderline* cases requiring use of dual symbols.
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with I_P greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting C_u or C_c requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM*	*d*	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or I_P less than 4	Limits plotting in hatched zone with I_P between 4 and 7 are *borderline* cases requiring use of dual symbols.
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with I_P greater than 7	

….table continued

Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50%)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	For all soils plotting nearly on "A" line use dual symbols, i.e., $I_P = 29.5$, $w_L = 60$ gives CH-OH or CH-MH. When w_L is near 50 use CL/CH, ML/MH. Take "nearly on" as ±2 percent.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		OL	Organic silts and organic silty clays of low plasticity	
	Silts and clays (Liquid limit greater than 50%)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		CH	Inorganic clays of high plasticity, fat clays	
		OH	Organic clays of medium to high plasticity, organic silts	
	Highly organic soils	Pt	Peat and other highly organic soils	



A-Chart

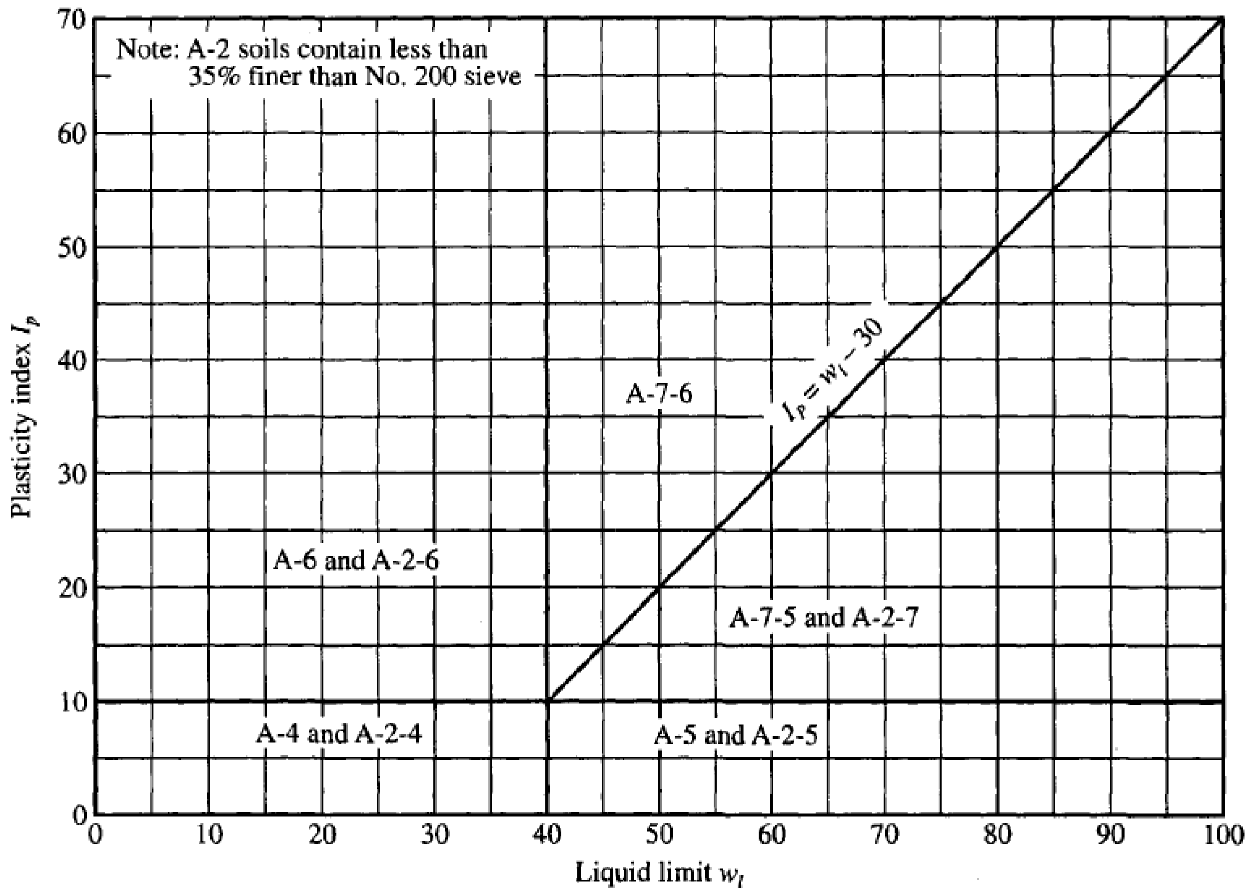
*Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when w_L is 28 or less and the I_P is 6 or less; suffix u used when w_L is greater than 28.

†Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

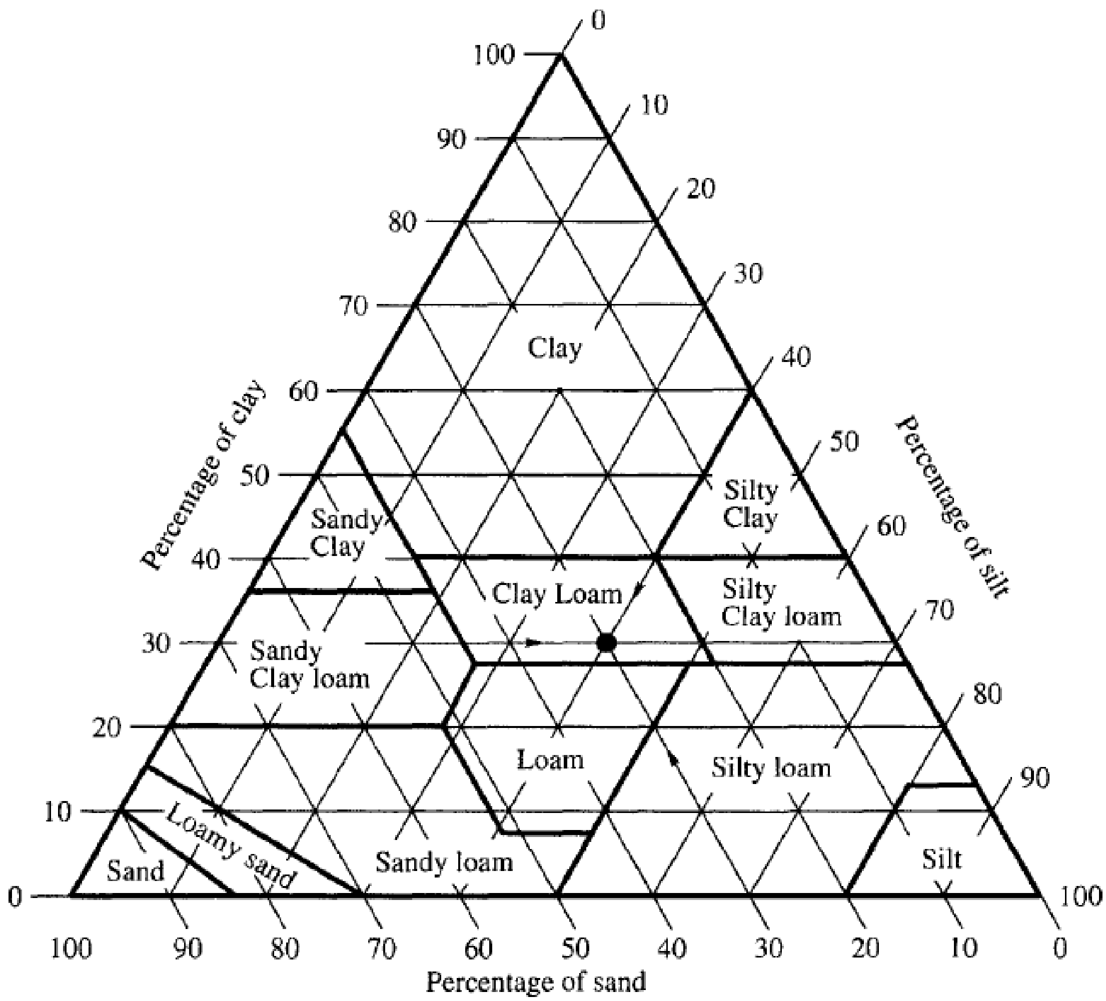
Table C-2 AASHTO soil classification scheme [5]

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	*A*-1		*A*-3	*A*-2				*A*-4	*A*-5	*A*-6	*A*-7
	A-1-*a*	*A*-1-*b*		*A*-2-4	*A*-2-5	*A*-2-6	*A*-2-7				*A*-7-5 *A*-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments—gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

Graph C-1 Chart for use in AASHTO soil classification system



Graph C-2 Chart for Textural soil classification system [4]



Appendix – D
One Dimensional Consolidation Results

D-I-Compression Parameters, Pre-consolidation and Swelling Pressure

Location: Merkato-1(TP-1-2)

Figure D-1-1- Void ratio Vs log p curve for TP-1-2

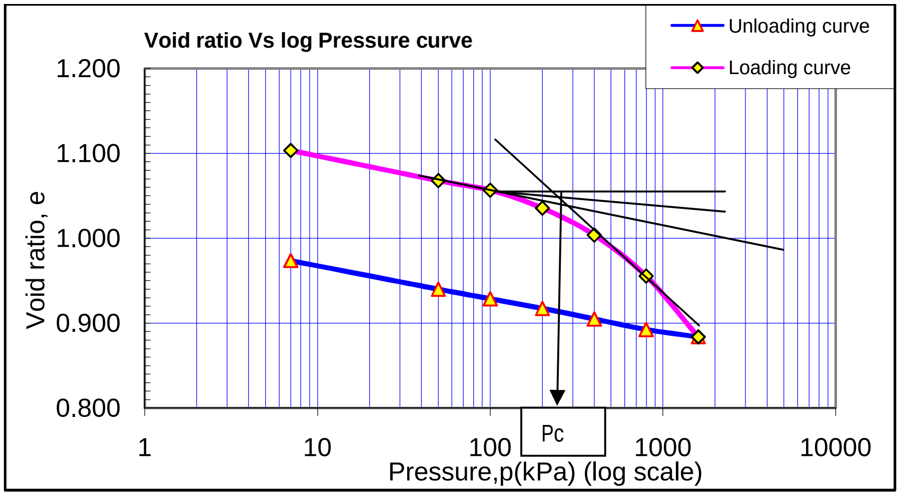


Figure D-1-2 Void ratio Vs p curve for TP-1-2

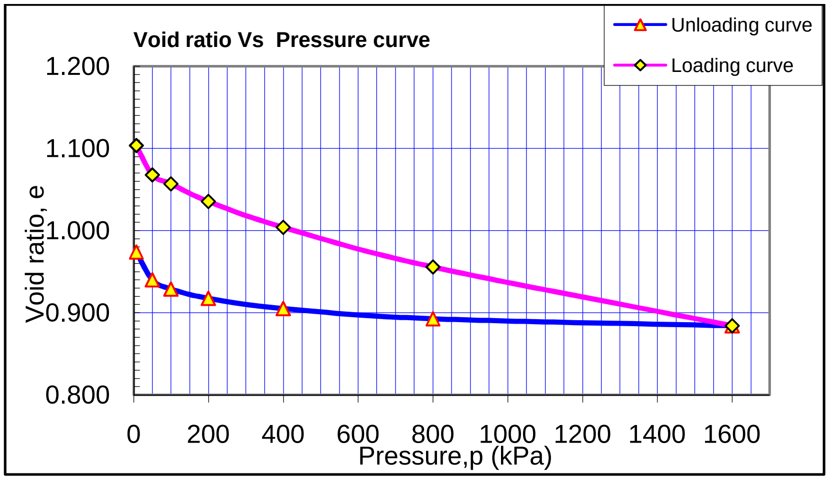


Table D-1-3 Compression parameters for TP-1-2

Compression Index, Cc	0.238
Swelling Index, Cs	-0.0393
Preconsolidation Pressure, pc,kPa	230
Coefficient of compressibility,a_v, m^2/kN	-0.000106

Location: Awetu (TP-3-2)

Figure D-3-1- Void ratio Vs log p curve for TP-3-2

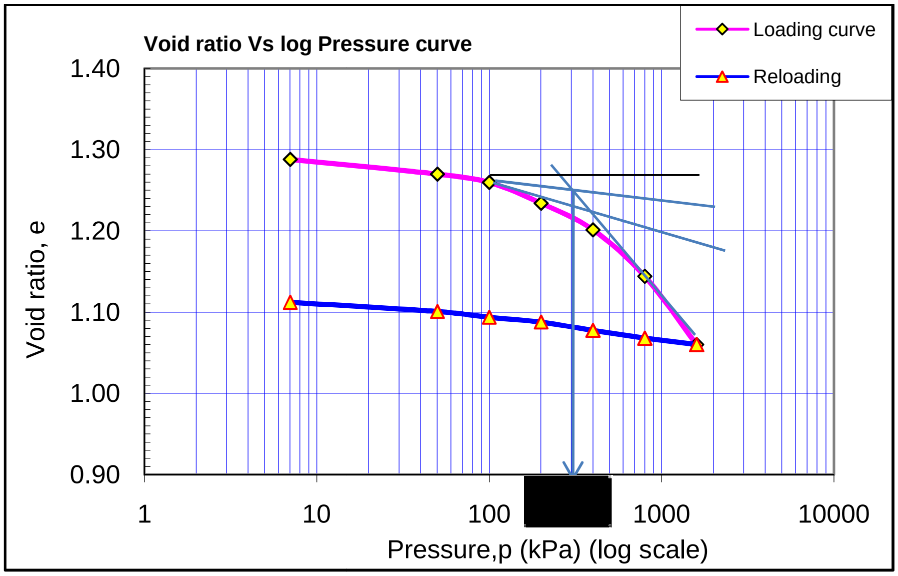


Figure D-3-1- Void ratio Vs p curve for TP-3-2

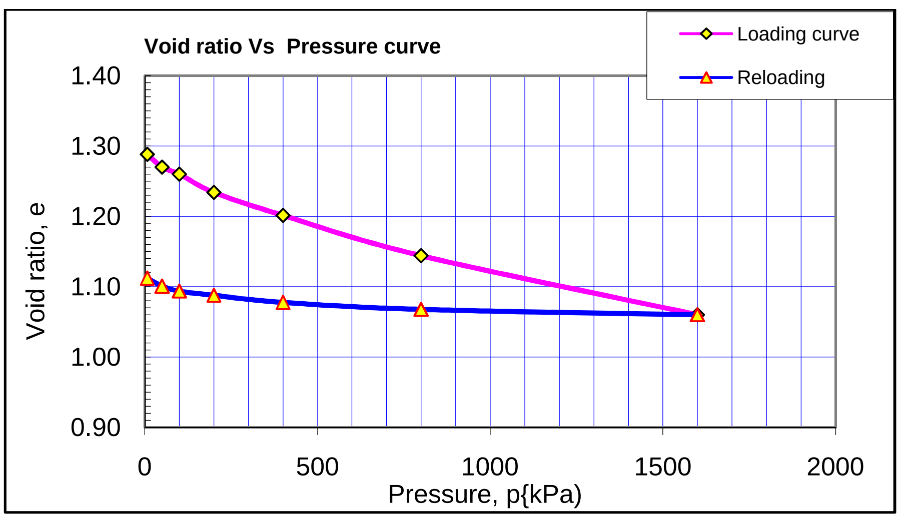


Table C-3-3 Compression parameters for TP-3-2

Compression Index, Cc	0.278
Swelling Index, Cs	-0.0460
Ppreconsolidation Pressure, pc,kPa	300
Coefficient of compressibility, av, m^2/kN	-0.000164

D-II-Coefficient of Consolidation and Permeability

Table D-II-Characteristic Coefficient of Consolidation and Permeability

Applied Load(kPa)	Coefficient of consolidation, c_v (cm^2/min)	
	TP-1	TP-3
50	1.92E-01	2.93E-01
100	5.30E-02	1.47E-01
200	1.16E-01	1.47E-01
400	8.82E-02	1.36E-01
800	4.19E-02	1.25E-01
1600	1.47E-01	1.08E-01
Characteristic Coefficients		
$c_{v,\ mean}$	1.06E-01	1.60E-01
$c_{c,\ sd}$	6.81E-02	8.95E-02
$c_{v,ch}$(cm^2/min)	7.24E-02	1.15E-01
$k_{v,\ ch}$(cm/min)	3.64E-07	8.24E-07

Appendix – E
Compaction Test Results

E-1 Dry unit weight and Optimum moisture content determination for all Samples

Sample Location: Merkato-1 (TP-1-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-1 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.206	4.724	1.482	15.7
2	6.361	4.724	1.637	17.34
3	6.422	4.724	1.698	17.99
4	6.413	4.724	1.689	17.89
5	6.401	4.724	1.568	16.7

Note: Volume of mold for all tests (Vm) in cc = **944**

Table E-1-2 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
60	15.4	60.1	52.6	7.5	37.2	11.5	12.5
J1.5	15.6	63	52.8	10.2	37.2	20.16	13.07
L2	15.7	62.7	51.2	11.5	35.5	27.42	13.61
E3	15.3	81.4	61.7	19.7	46.4	32.39	13.58
A1	15.5	76.5	65.2	11.3	40.6	42.46	12.45

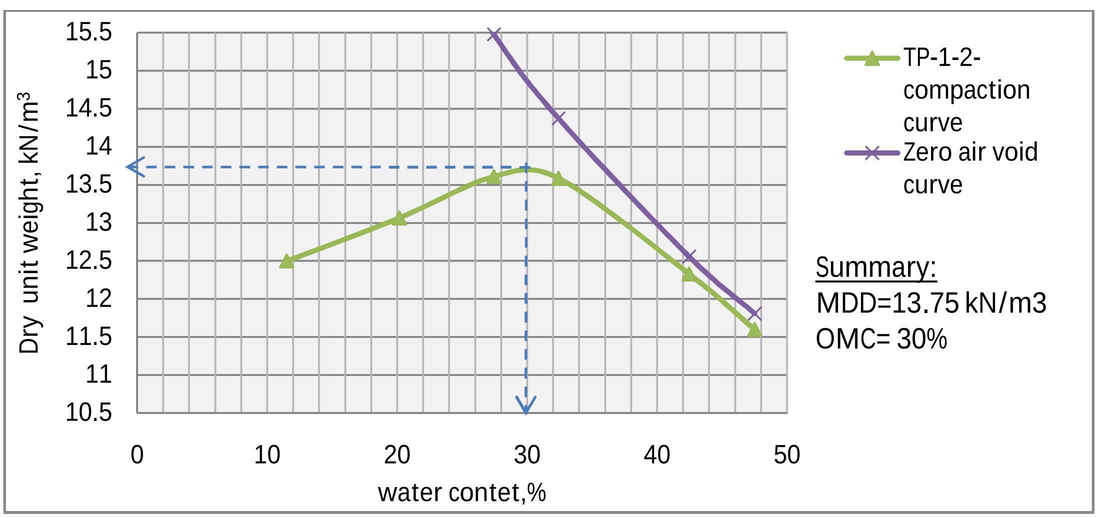


Figure E-1-1 Compaction curve for TP-1-2 for Air dried sample

Sample Location: Merkato-2 (TP-2-2)

Test Condition:_(Air Dried and Fresh sample)

Table E-1-3 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.206	4.724	1.482	15.70
2	6.361	4.724	1.637	17.34
3	6.422	4.724	1.698	17.99
4	6.413	4.724	1.689	17.89
5	6.21	4.724	1.486	15.74

Table E-1-4 Bulk unit weight determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	water content (%)	Dru unit weight of compacted soil (kN/m^3)
MA12	15.4	60.1	52.6	7.5	37.2	12.3	12.2
MA23	15.6	63	52.8	10.2	37.2	21.30	12.48
MA15	15.7	62.7	51.2	11.5	35.5	35.00	13.24
D4	15.3	81.4	61.7	19.7	46.4	41.00	12.6
D5	15.8	85.8	64.7	21.1	48.9	48.00	11.56

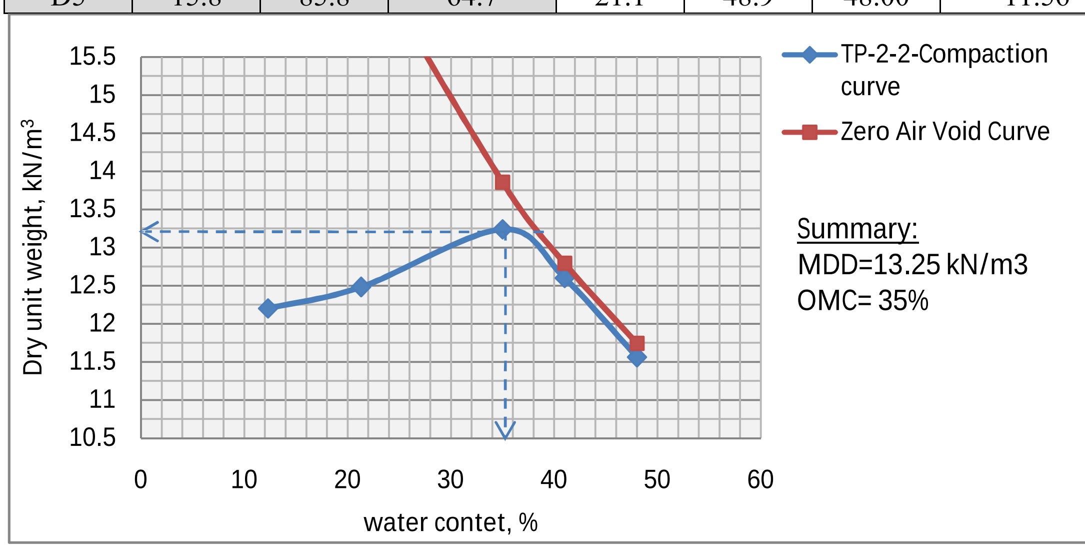


Figure E-1-2 Compaction curve for TP-2-2 for Air dried sample

Sample Location: Awetu (TP-3-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-5 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.035	4.724	1.311	13.89
2	6.103	4.724	1.379	14.61
3	6.312	4.724	1.588	16.82
4	6.446	4.724	1.722	18.24
5	6.391	4.724	1.667	17.66

Table E-1-6 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
T15	15.6	58.6	54.2	4.4	38.6	11.40	11.23
N2	15.5	62.3	53.2	9.1	37.7	24.14	12.20
MA16	15.3	59.2	49	10.2	33.7	30.27	12.91
50	15.3	53.3	43.1	10.2	27.8	36.69	13.35
A1	15.3	59.1	45.7	13.4	30.4	44.08	12.26

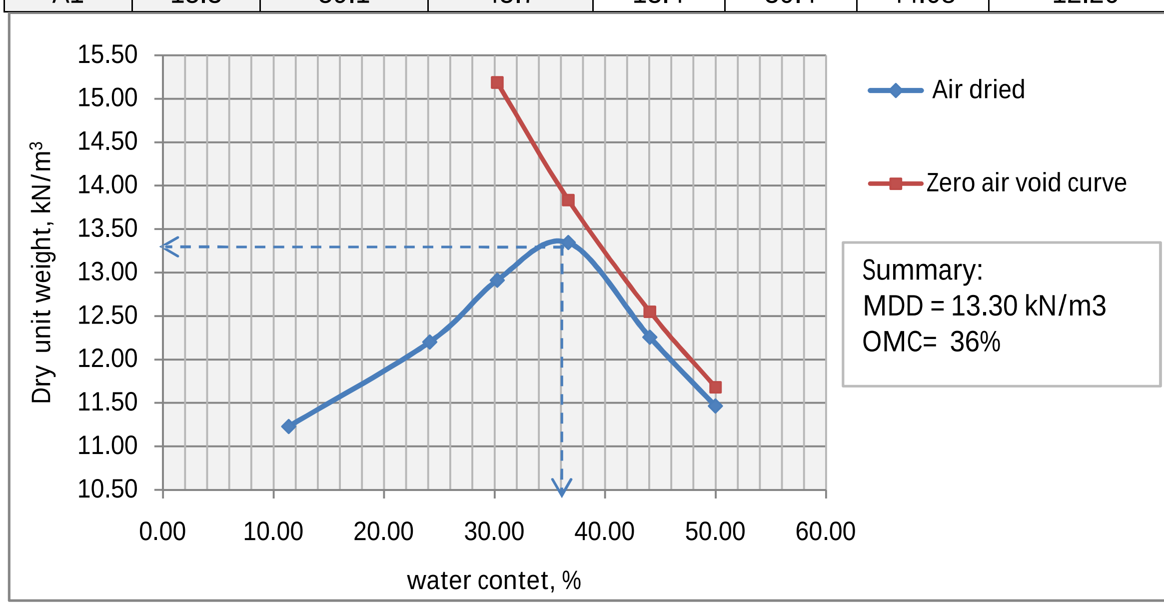


Figure E-1-3 Compaction curve for TP-3-2 for Air dried sample

Sample Location: Teknik Sefer (TP-4-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-7 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.137	4.724	1.413	14.97
2	6.186	4.724	1.462	15.49
3	6.298	4.724	1.574	16.67
4	6.333	4.724	1.609	17.04
5	6.249	4.724	1.525	16.15

Table E-1-8 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
E3	15.3	65.9	56.5	9.4	41.2	15	11.46
MA16	15.2	69	57.1	11.9	41.9	24.34	11.67
MA29	15.5	80.1	63.6	16.5	48.1	35.60	12.04
MA28	15.7	84.3	62.6	21.7	46.9	40.67	11.96
60	15.5	79.7	55.5	24.2	40	51.50	10.70

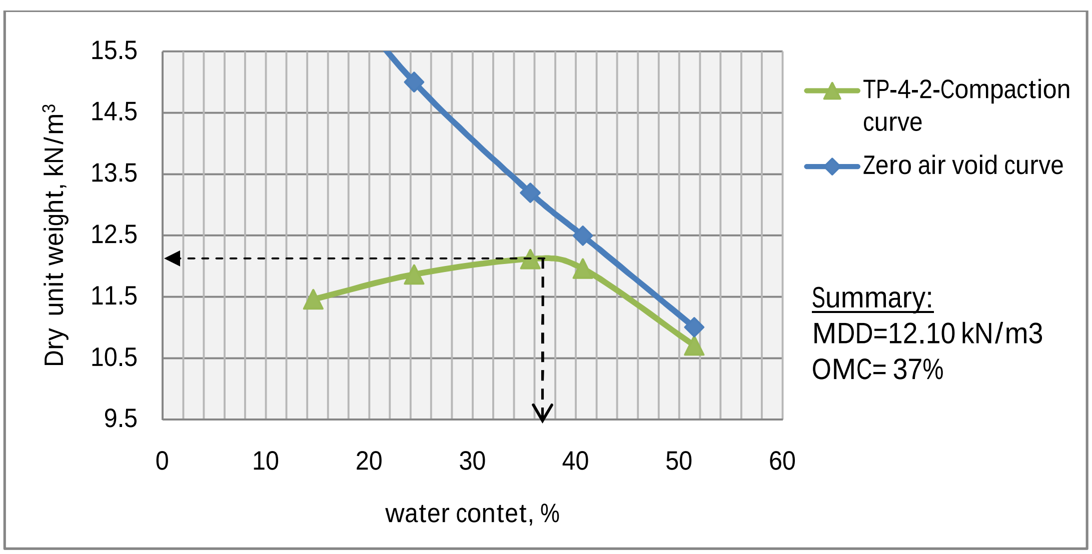


Figure E-1-4 Compaction curve for TP-4-2 for Air dried sample

Sample Location: Kebel -5 (TP-5-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-9 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.137	4.724	1.413	14.97
2	6.186	4.724	1.462	15.49
3	6.298	4.724	1.574	16.67
4	6.333	4.724	1.609	17.04
5	6.249	4.724	1.525	16.15

Table E-1-8 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
E3	15.3	65.9	56.5	9.4	41.2	22.82	11.92
MA16	15.2	69	57.1	11.9	41.9	28.40	12.06
MA29	15.5	80.1	63.6	16.5	48.1	34.30	12.41
MA28	15.7	84.3	62.6	21.7	46.9	46.27	11.65
60	15.5	79.7	55.5	24.2	40	56.00	10.56

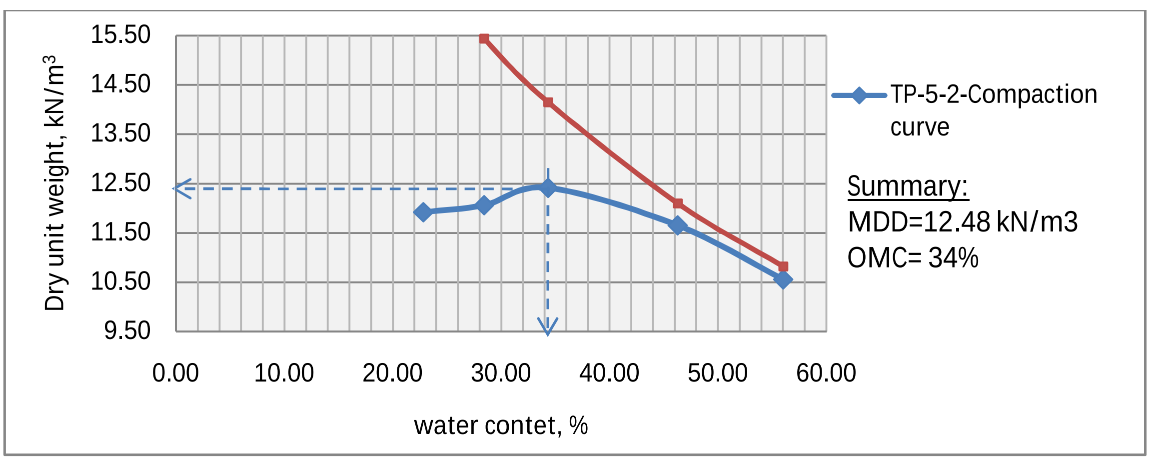


Figure E-1-5 Compaction curve for TP-5-2 for Air dried sample

Sample Location: Jimma University (TP-6-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-9 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.300	4.724	1.576	16.69
2	6.558	4.724	1.834	19.43
3	6.636	4.724	1.912	20.25
4	6.568	4.724	1.844	19.53
5	6.468	4.724	1.744	18.47

Table E-1-10 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
1f	15.6	82.8	69.3	13.5	53.7	25.14	13.34
MA15	15.5	74.7	60.8	13.9	45.3	30.68	14.87
D2	15.4	84.9	67.3	17.6	51.9	33.91	15.13
B4	15.4	83.3	63.9	19.4	48.5	40.00	13.95
B7	15.4	89.3	65.9	23.4	50.5	46.34	12.62

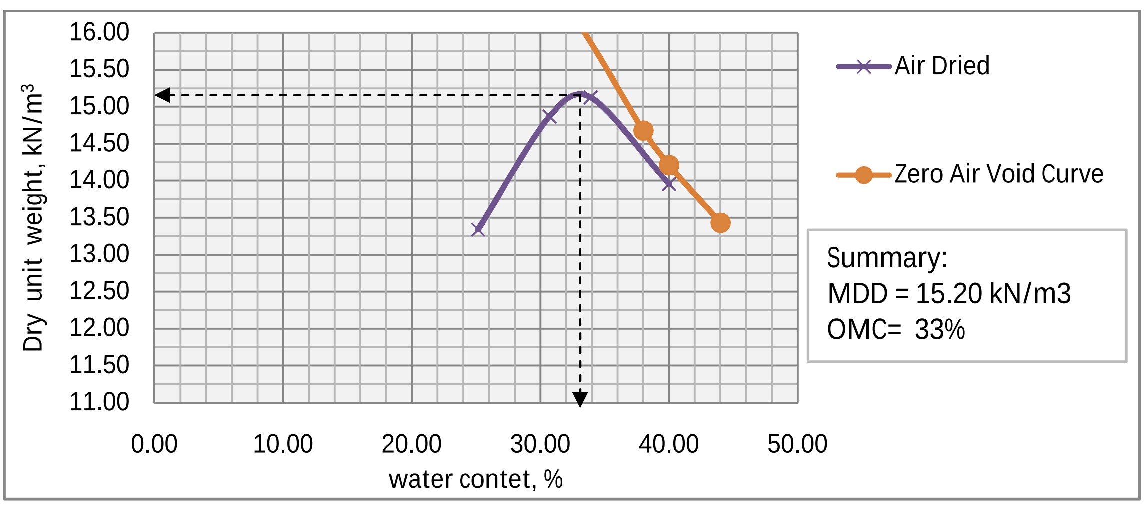


Figure E-1-6 Compaction curve for TP-6-2 for Air dried sample

Sample Location: Bacho Bore (TP-7-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-11 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.137	4.724	1.413	14.97
2	6.186	4.724	1.462	15.49
3	6.298	4.724	1.574	16.67
4	6.333	4.724	1.609	17.04
5	6.249	4.724	1.525	16.15

Table E-1-12 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
E3	15.3	65.9	56.5	9.4	41.2	14.21	11.50
MA16	15.2	69	57.1	11.9	41.9	24.34	11.67
MA29	15.5	80.1	63.6	16.5	48.1	35.60	12.22
MA28	15.7	84.3	62.6	21.7	46.9	39.67	12.22
60	15.5	79.7	55.5	24.2	40	47.50	11.44

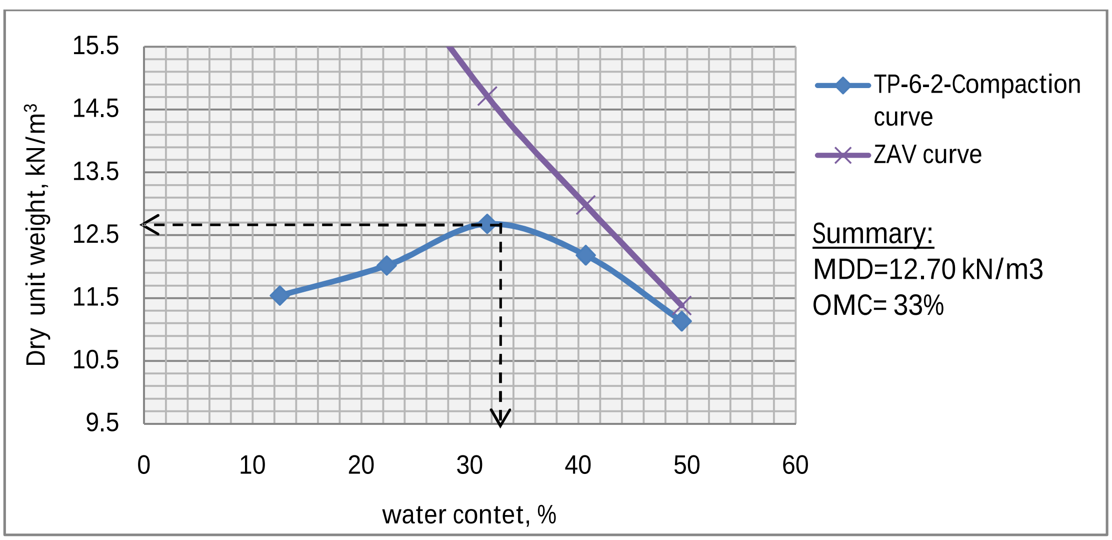


Figure E-1-7 Compaction curve for TP-7-2 for Air dried sample

Sample Location: Agri Sefer (TP-8-2)

Test Condition: (Air Dried and Fresh sample)

Table E-1-13 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.057	4.724	1.333	14.12
2	6.198	4.724	1.474	15.61
3	6.46	4.724	1.736	18.39
4	6.435	4.724	1.711	18.13
5	6.387	4.724	1.663	17.62

Table E-1-14 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
MA29	15.6	58.6	54.2	4.4	38.6	14.00	12.50
60	15.5	62.3	53.2	9.1	37.7	24.14	13.23
MA28	15.3	59.2	49	10.2	33.7	30.27	13.80
50	15.3	53.3	43.1	10.2	27.8	36.69	14.21
G3-2	15.3	59.1	45.7	13.4	30.4	47.40	12.30

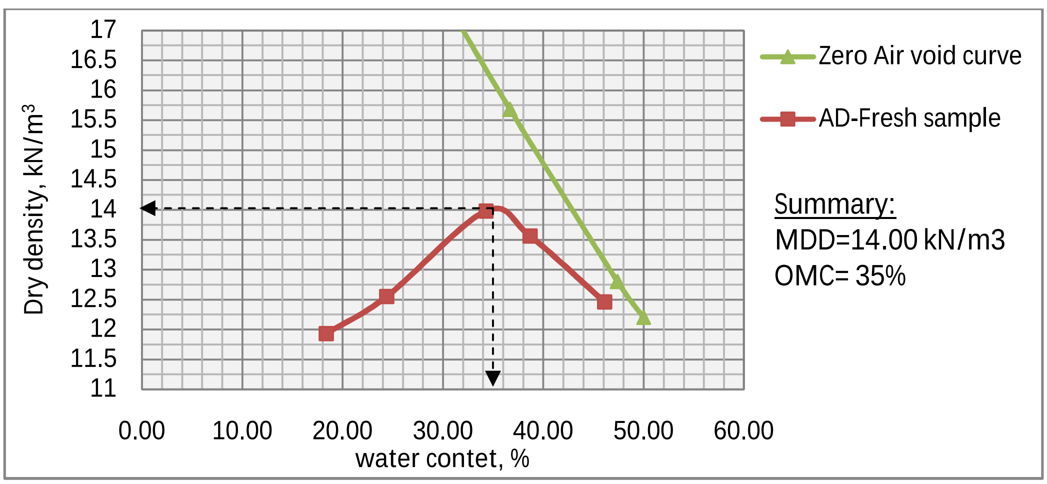


Figure E-1-8 Compaction curve for TP-8-2 for Air dried sample

E-2 Effect of Sample preparation for lateritic soils

Sample Location: Awetu (TP-3-2)

Test Condition: (Oven dried sample)

Table E-2-1 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.077	4.724	1.353	14.33
2	6.327	4.724	1.603	16.98
3	6.47	4.724	1.746	18.50
4	6.41	4.724	1.686	17.86
5				

Table E-2-2 Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
G3-2	15.4	81.2	70.5	10.7	55.1	19.42	12.40
D2	15.6	69.3	57.8	11.5	42.2	27.25	13.34
E3	15.3	79	62.8	16.2	47.5	34.11	13.79
MA29	15.5	95.7	72.9	22.8	57.4	39.72	12.78
						47.00	11.52

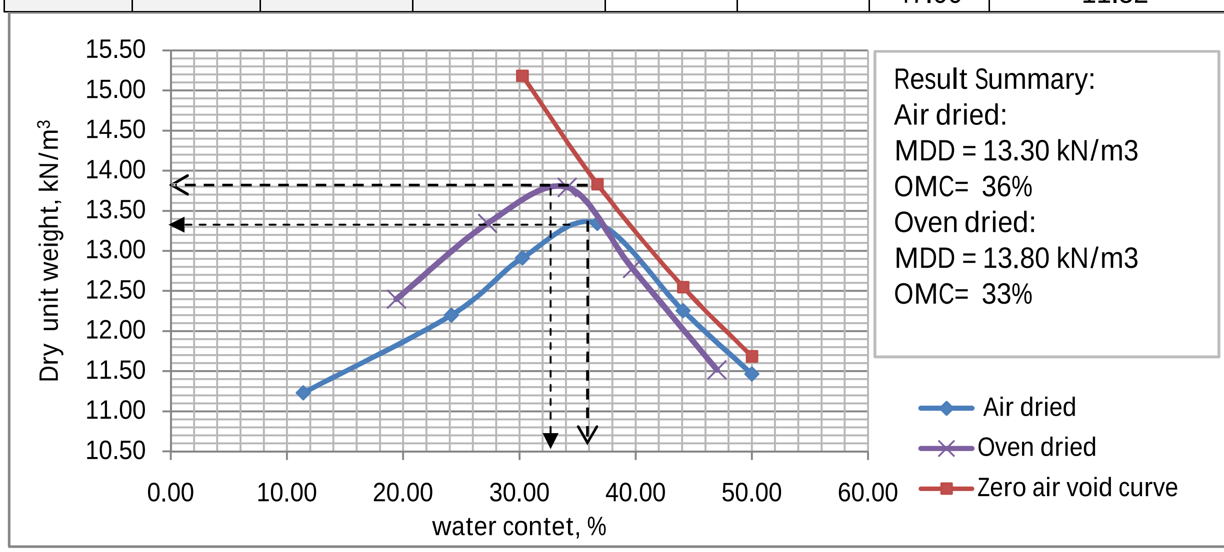


Figure E-2-1 comparison of compaction curves for TP-3-2 for Air dried and Oven dried samples

Sample Location: Jimma University (TP-6-2)

Test Condition: (Oven dried sample)

Table E-2-3 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.293	4.724	1.569	16.62
2	6.434	4.724	1.71	18.11
3	6.644	4.724	1.92	20.34
4	6.600	4.724	1.876	19.87
5	6.650	4.724	1.926	20.40

Table E-2-4Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
MA28	15.5	81.5	71.4	10.1	55.9	18.07	14.08
B4	15.4	81.6	68.9	12.7	53.5	23.74	14.64
MA15	15.4	111.4	87.9	23.5	72.5	32.41	15.36
MA16	15.2	110.4	85.9	24.5	70.7	35.00	14.94
A5	15.2	115.4	88.6	26.8	73.4	38.90	13.89

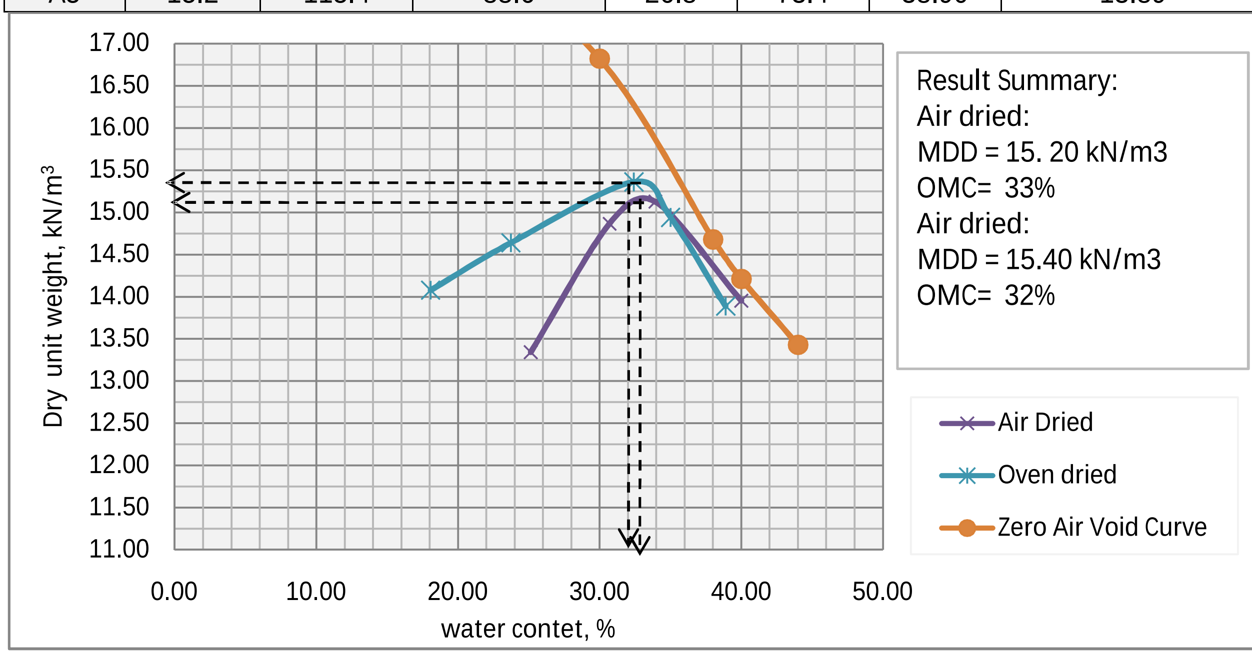


Figure E-2-2 Comparison of compaction curves for TP-6-2 for Air dried and Oven dried samples

Sample Location: Agri Sefer (TP-3-2)

Test Condition: (Air dried and Oven dried sample)

Table E-2-5 Bulk unit weight determination

Group	Mass of moist specimen + mold	Mass of mold	Mass of moist specimen	Moist unit weight of compacted specimen (kN/m^3)
1	6.077	4.724	1.353	14.33
2	6.327	4.724	1.603	16.98
3	6.47	4.724	1.746	18.50
4	6.41	4.724	1.686	17.86
5	6.42	4.724	1.696	17.97

Table E-2-6Moisture content determination

Can no.	Mass of can	Mass of soil + can	Mass of dried sample + can	Mass of water	Mass of Dry soil	Water content (%)	Dry unit weight of compacted soil (kN/m^3)
G3-2	15.4	81.2	70.5	10.7	55.1	19.42	13.43
D2	15.6	69.3	57.8	11.5	42.2	27.25	13.98
E3	15.3	79	62.8	16.2	47.5	34.11	14.56
MA29	15.5	95.7	72.9	22.8	57.4	39.72	13.34
C4	15.5	98.7	76.9	21.8	61.4	47.00	12.27

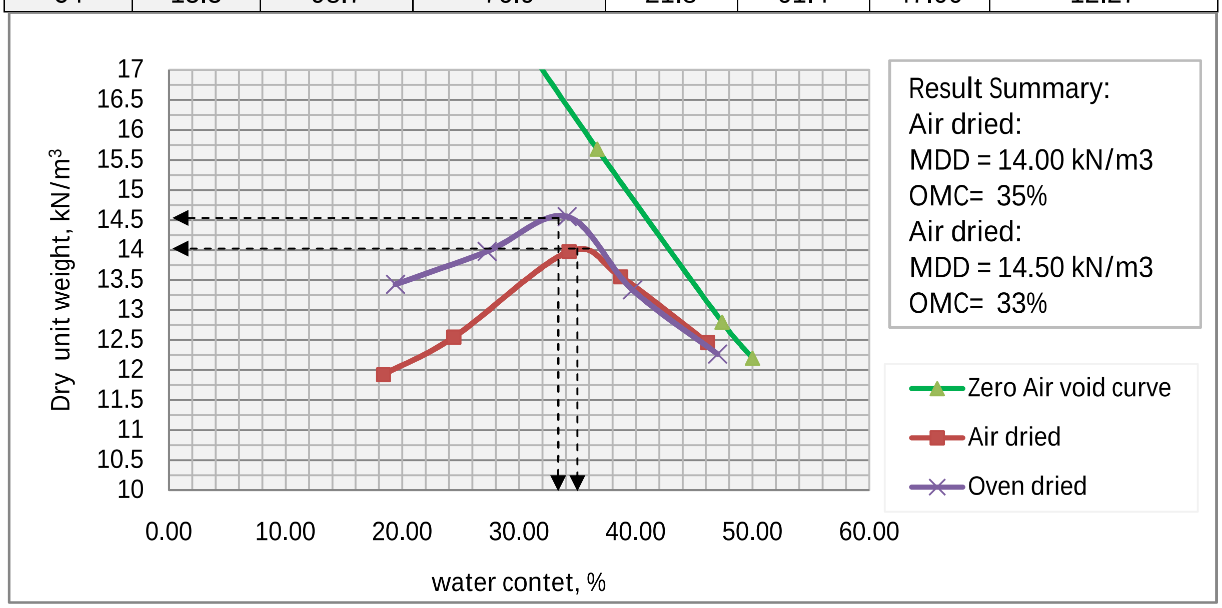


Figure E-2-3 Comparison of compaction curves for TP-8-2 for Air dried and Oven dried samples

Appendix – F
Unconfined Compression Results

F-1Unconfined Compressive Strength and Undrained Shear Strength

Sample Location: Merkato-1 (TP-1 at 1.5 and 2.5m depths)

Sample Type: Undisturbed

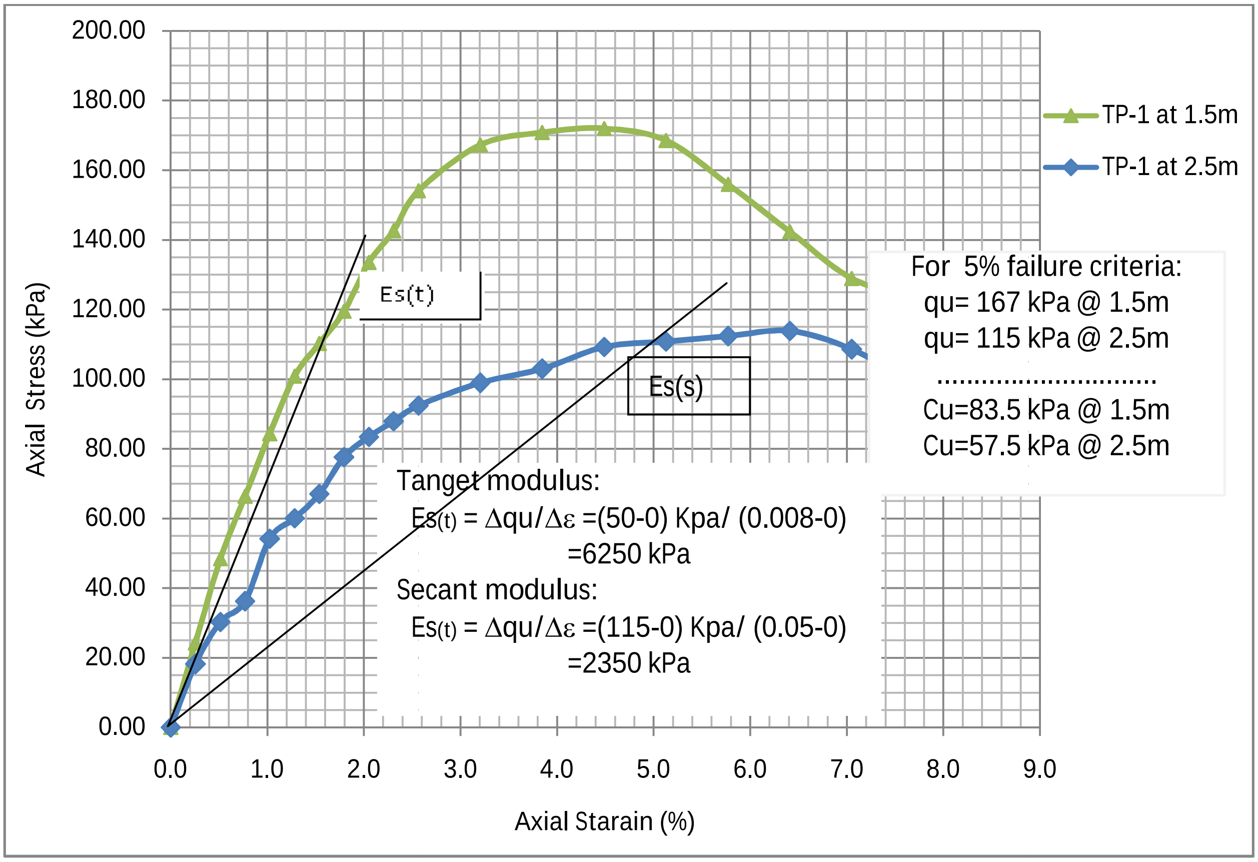


Figure F-1-1 Stress-Strain curve for TP-1 at 1.5 and 2.5m on undisturbed samples

Sample Location: Merkato-2 (TP-2 at 1.5 and 2.5m depths)

Sample Type: Undisturbed

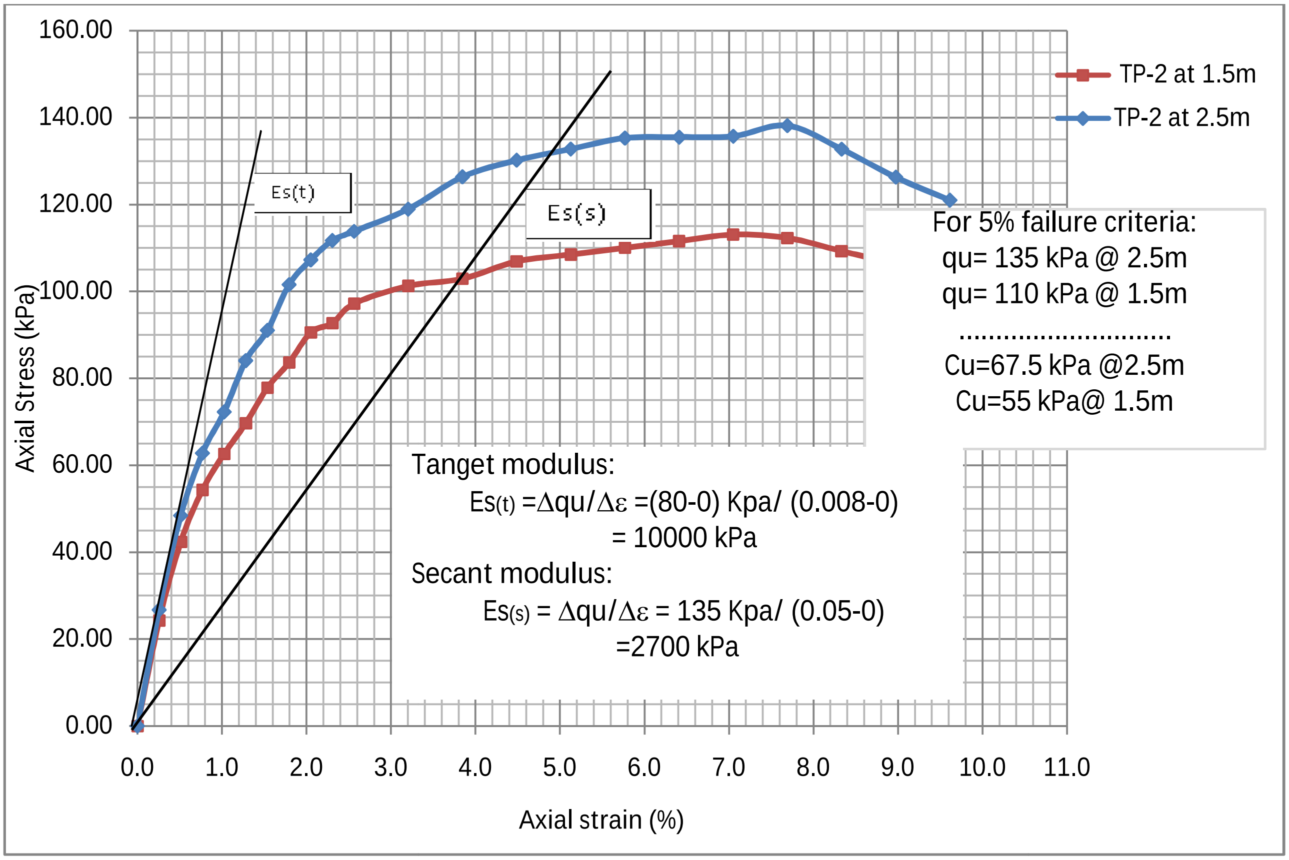


Figure F-1-2 Stress-Strain curve for TP-2 at 1.5 and 2.5m on undisturbed samples

Sample Location: Awetu (TP-3 at 1.5 and 2.5m depths)

Sample Type: Undisturbed

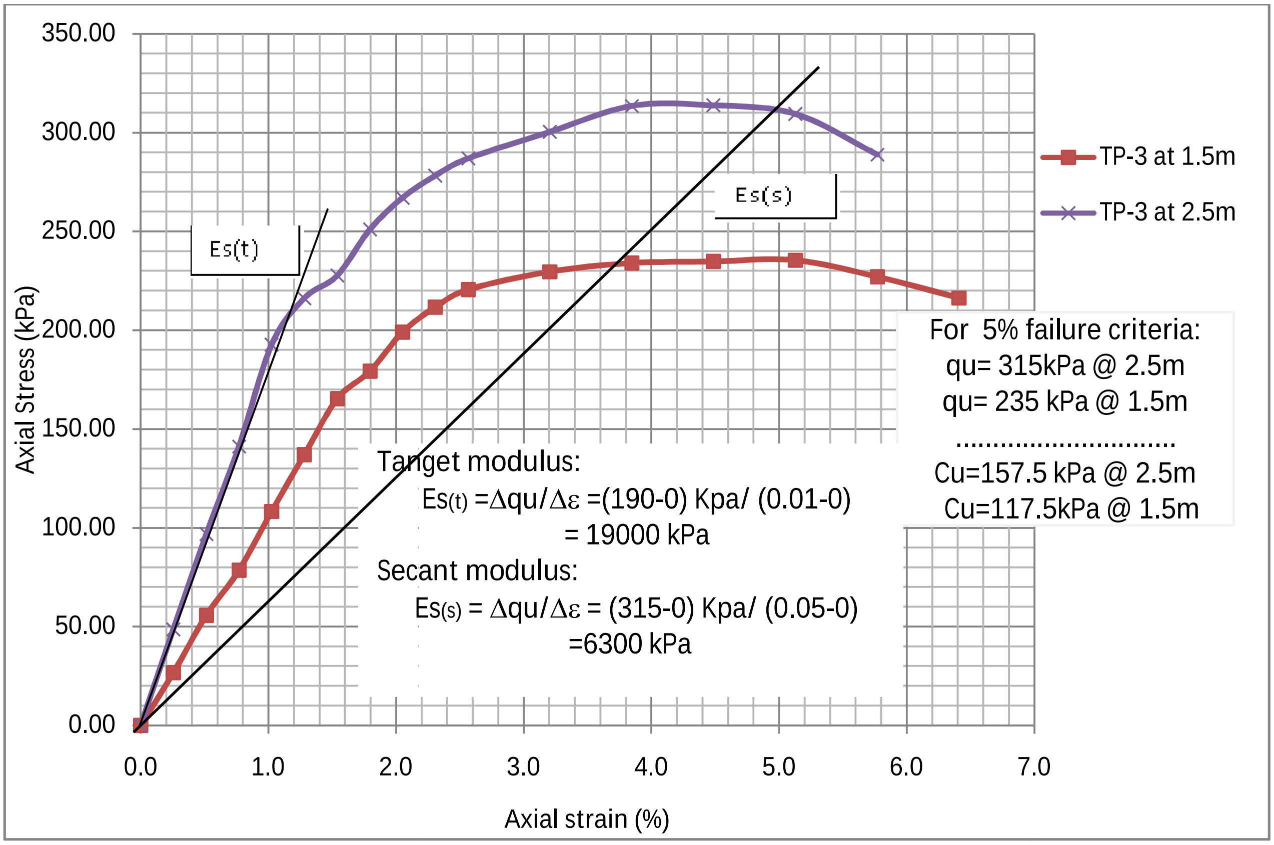


Figure F-1-3 Stress-Strain curve for TP-3 at 1.5 and 2.5m on undisturbed samples

Sample Location: Teknik Sefer (TP-4 at 1.5 and 2.5m depths)

Sample Type: Undisturbed

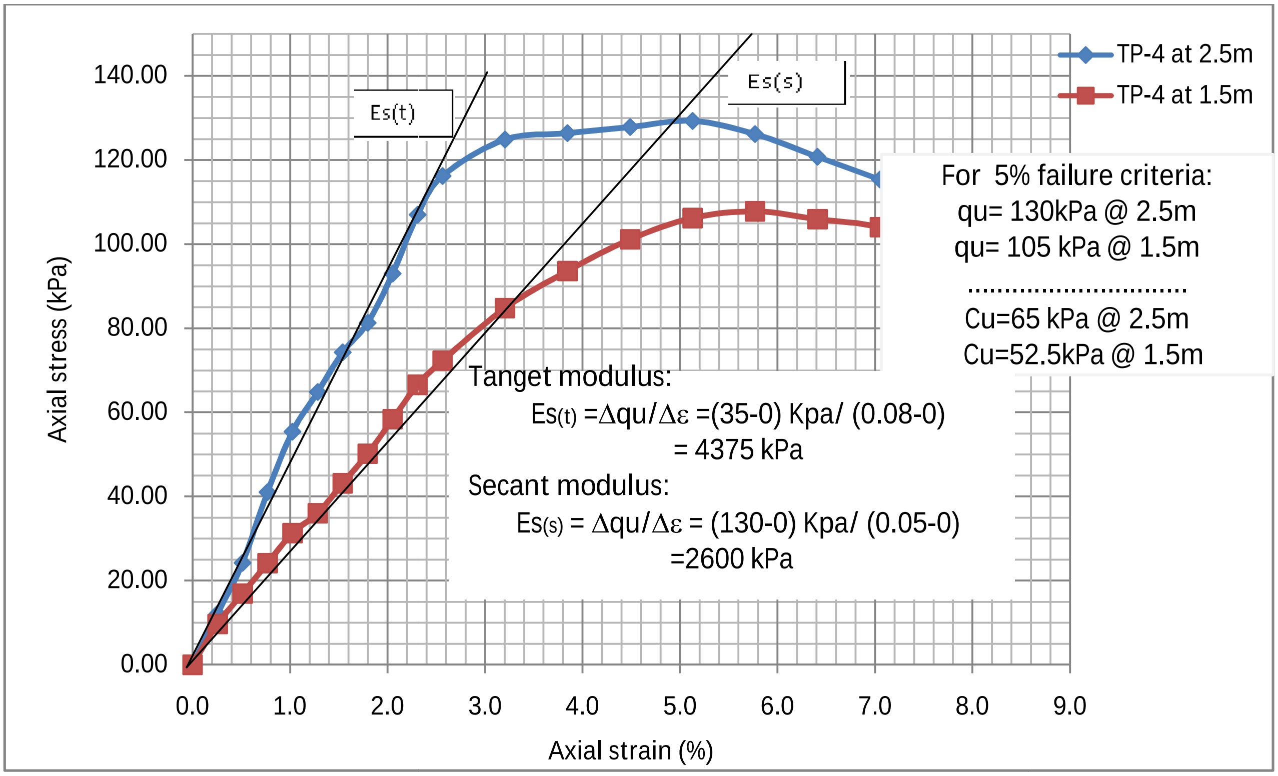


Figure F-1-4 Stress-Strain curve for TP-4 at 1.5 and 2.5m on undisturbed samples

Sample Location: Kebele 5 (TP-5 at 1.5 and 2.5m depths)

Sample Type: Undisturbed

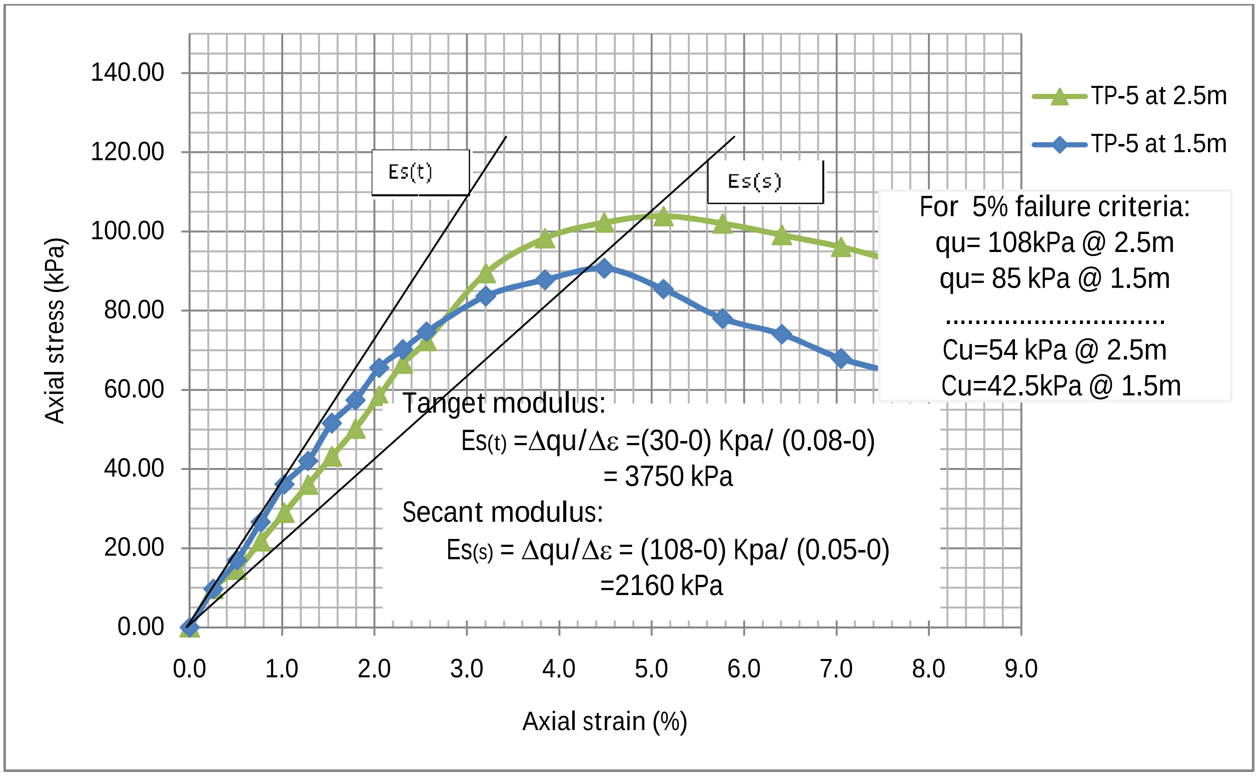


Figure F-1-5 Stress-Strain curve for TP-5 at 1.5 and 2.5m on undisturbed samples

Sample Location: Jimma University (TP-6 at 1.5m depth)

Sample Type: Undisturbed

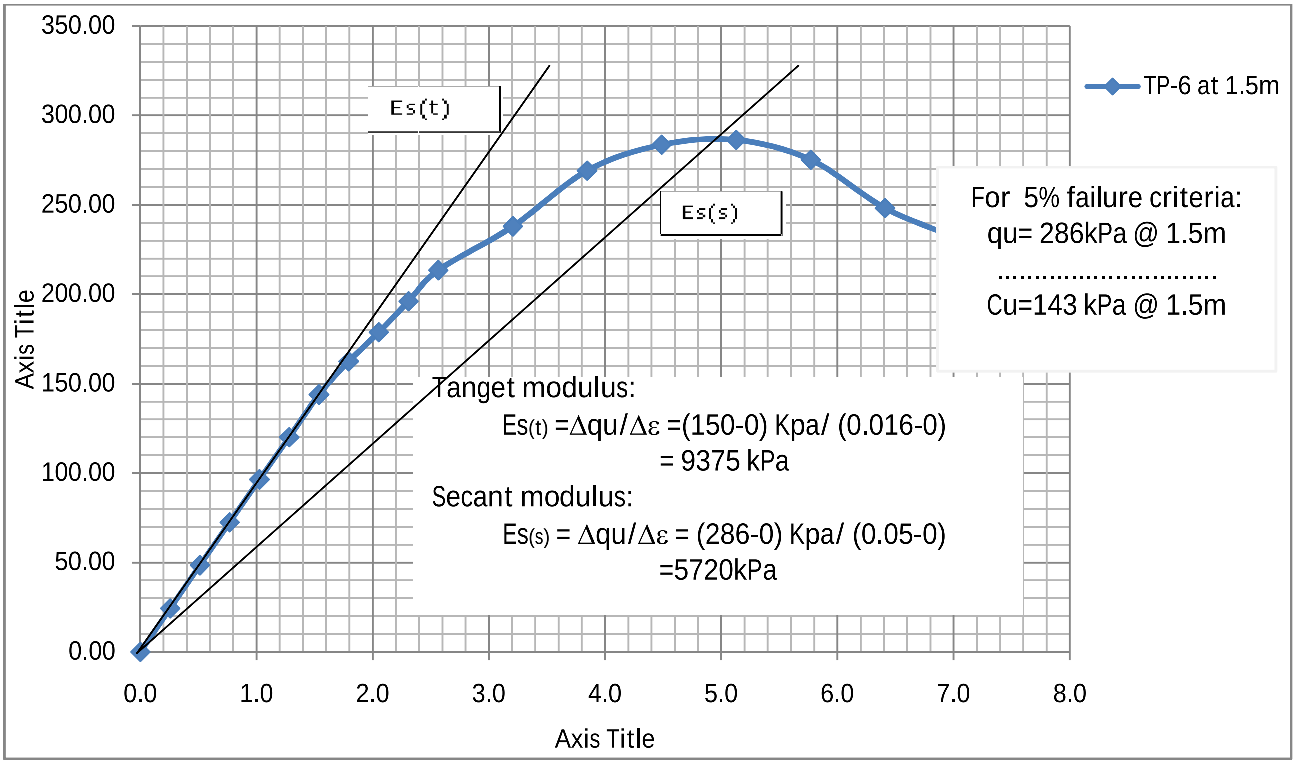


Figure F-1-6 Stress-Strain curve for TP-6 at 1.5 on undisturbed samples