ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

INVESTIGATION INTO SOME OF THE ENGINEERING PROPERTIES OF SOILS FOUND IN WOLISO TOWN

BY:

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"A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in civil engineering (Geotechnical Engineering)"

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SYMBOLS AND ABBREVIATIONS

AASHTO	-	American Association of Highway and Transportation Officials
ASTM	-	American Society for Testing Materials standard
Cc	-	Compression index
Cs	-	swelling index
C_V	-	Coefficient of consolidation
e	-	Void ratio
K	-	Coefficient of Permeability
LL	-	Liquid limit
MH	–	highly plastic Inorganic Elastic silt
CH	–	highly plastic Inorganic clay
OCR	-	Over-consolidation ratio
P_C	–	Pre-consolidation pressure
Po	-	Over burden pressure
PI	-	Plastic Index
PL	-	Plastic limit
SL	-	Shrinkage limit
N	-	Number of blows
TP	–	Test pit
USCS	-	Unified Soil Classification System
UCS	-	Unconfined compressive strength
γ_d	-	Dry unit weight
γ_w	–	Unit weight of water
q_u	-	Unconfined compressive strength
S_u	-	Undrained shear strength

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Abstract

Investigation of the ground conditions is used for the economical design of the sub structural elements. It is also necessary to obtain sufficient information on type, characteristics and distributions of soil and rock underlying a site for proposed structures.

The overall aim of this study is to come up with reliable results for some of the engineering properties of soils found in Woliso town in order to understand clearly the nature of the soil so that suitable foundation can be recommended for better design and construction of civil engineering structures in the town. To achieve these objective disturbed and undisturbed samples from different parts of the town were collected and laboratory tests were done on the collected samples. The undisturbed samples were taken from the test pits by mechanically driving a Shelby tube. For all tests the apparatus and the procedures used for analysis were done according to American Society for Testing Materials (ASTM) standard.

The natural moisture content of soil in the study area ranges from 25.5% - 66%. Expansive and non-expansive soils were investigated in the study area. The average specific gravity of the expansive soils is 2.68 and a non-expansive soil is 2.74.

The grain size analysis test result showed that the dominant proportion of soil particle in the research area is clay. For expansive soils the clay content ranging from 55 - 80%, silt ranging from 13.8 – 28.2%, sand ranging from 4.5 – 9.7% and gravel ranging from 0.1 – 11.4% and for non-expansive soils the clay content ranging from 42.5 – 85%, silt ranging from 12.8 – 51.1%, sand ranging from 2.2 – 7.5% and gravel ranging from 0 – 0.4% .

Atterberg Limit test results in the research area showed a liquid limit of expansive soils ranging from 70.5 - 126%, plastic limit ranging from 31.2 - 44% and plasticity index ranging from 39.3 – 81.7% and for non-expansive soils the liquid limit ranging from 56.2 – 61.1%, plastic limit from 30.3 – 34.4% and plasticity index ranging from 26.2 – 29.7%.

According to Unified Soil Classification System, expansive and non-expansive soils in the study area are categorized as highly plastic inorganic clay and silt soils respectively.

The unconfined compressive strength of the soils in the study area ranges from 75 - 540kN/m^2 and undrained shear strength ranges from 37.5 - 270 kN/m^2. Unconfined compressive strengths show that the approximate consistencies of the soil ranges from medium to hard.

One-dimensional consolidation test showed that the area under investigation is over consolidated in its natural state, have compression index ranging from 0.11-0.62, coefficient of consolidation from 1.25-17.17 (x10^{-3})cm^2/min, coefficient of permeability from 0.15-10.63 (x10^{-8} cm/sec).

1. Introduction

1.1. Background of the problem

In the field of civil engineering, nearly all projects are built on, or into, the ground. Thus, during the planning, design, and construction of foundations, embankments, tunnel and earth-retaining structures, geotechnical engineers must study the properties of soils, such as origin, grain-size distribution, permeability, compressibility, shear strength and load-bearing capacity.

Soil investigation is an essential part of the design and construction of a proposed structural system (buildings, dams, roads and highways, etc.). Soils are identified, observed, and recovered during investigation of a proposed site. Usually soil investigations are conducted only on a fraction of a proposed site because it would be prohibitively expensive to conduct an extensive investigation of a whole site. One then makes estimates and judgments based on information from a limited set of observations, and from field and laboratory test data that will have profound effects on the performance and costs of structures constructed at a site.

Investigation of the sub-surface conditions at a site is prerequisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies of the proposed project. Public building officials may require soil data together with the recommendations of the geotechnical consultant prior to issuing a building permit, particularly if there is a chance that the project will endanger the public health or safety or degrade the environment.

The aim of investigation is to provide maximum information that is useful in the design and construction of the project at a minimum cost.

Insufficient geotechnical investigations, faulty interpretation of results or failure to portray results in a clearly understandable manner may contribute to inappropriate designs; delays in construction schedules, costly construction modifications, and use of substandard material, environmental damage to the site and even failure of a structure. Therefore, to obtain information on type, characteristics and distributions of a soil, geotechnical investigations should be done on soil and rock underlying (and sometimes adjacent to) a site of proposed structures.

For developing countries like Ethiopia which is developing at high growth rate the construction industry is also growing rapidly. Detailed geotechnical investigation on the engineering property of soil is very essential.

Woliso is one of the fast growing towns in Oromia regional state. Due to its location from Addis Ababa, (114km), and its topographical location, Woliso is preferred by investors. Thus, there are many ongoing constructions and as stated by administration the town has a vision to become one of the developed cities in the country in every aspect. However, the engineering property of the soil in the town is not studied. This research is therefore directed to the study of the basic engineering properties of soils i.e. investigating the index property and consolidation as well as swelling characteristics of the soil.

1.2. Objectives of the study

The overall aim of this thesis is to investigate the engineering properties of soils found in Woliso town in order to understand clearly the nature of the soil so that suitable foundation can be recommended for better design and construction of civil engineering structures in the town.

The specific objectives of this thesis are:-

- To investigate some of the engineering and index properties of the Woliso soil including: natural moisture content, specific gravity, bulk density, consistency limits, grain size analysis and free swell.
- To classify the soil.
- To determine the shear strength characteristics of the soil.
- To investigate the expansive potential of the soil.
- To determine the consolidation characteristics of soils in the town.

1.3. Methodology

To achieve the above mentioned objectives six sampling areas were selected which represent all types of soils found in the town. From the selected sampling areas test pits were excavated to a depth of around three meters for a visual observation of the sub-surface condition and to take representative samples. Disturbed and undisturbed samples of soils were collected for laboratory testing. The undisturbed samples were taken from the test pits by manually driving a Shelby tube, which is a thin-walled sampling tube, slowly in to the desired depth. While preparing the tube length and diameter were adjusted to make sample extrusion easy. An inside clearance ratio of about 1% was provided for tip relief of the soil and to reduce the friction between the soil and inner edge of the sampling tube during the sampling process. A thin film of oil was applied at the cutting edge and inside the sampler to reduce the friction between the soil and metal tube during sampling operations and to make sample extrusion easy. The purpose of having a low area ratio and a sharp cutting end is to slice into the soil with as little disruption and displacement of the soil as possible. Undisturbed soil samples recovered from the test pits were kept within the sampling tube. The soil sampling tube was tightly sealed with wax thoroughly sealed in containers to prevent a loss of moisture during transportation to the laboratory.

Then, from disturbed and undisturbed samples collected the following laboratory tests were done.

- ✓ Specific gravity test
- ✓ Atterberg limit tests
- ✓ Grain size analysis
- ✓ Free swell test
- ✓ Unconfined compression test
- ✓ One-dimensional consolidation test

1.4. Scope of the Study

Twelve samples of disturbed and undisturbed soil from six pits are to be collected. The scope of this study is limited to investigating the index properties, shear strength, swelling properties and consolidation characteristics. Due to the budget constraint, the depth of investigation in this research is limited to the maximum depth of three meters and the number of samples taken was limited to twelve. Consolidation test is done only for four samples. The permeability coefficient of the soil in the research area is calculated from consolidation test results.

1.5. Structure of the Thesis

This thesis work is divided in to six Chapters, each covering a specific topic of the research work. In this introductory Chapter the background of the problem, objective, methodology and scope of the thesis work and structure of the thesis are presented.

Chapter two deals with a brief literature review. Chapter three deal with description of the study area in which this research is done. The fourth chapter deals with in-situ properties with sample description and the types of laboratory tests conducted and results obtained. The discussion on the laboratory results obtained from this work is covered in chapter five. Chapter six covers the conclusions and recommendations drawn from the research. Detail calculation of Atterberg limits, the unconfined compression test, consolidation test results and meteorological data are included in the appendices.

2. Literature review

2.1. Soil formation

Soil 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. Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by (physical) mechanical disintegration, and/or chemical decomposition. 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. When rock surface gets exposed to atmosphere for an appreciable time, it disintegrates or decomposes into small particles and thus soils are formed. Physical weathering involves reduction of size without any change in the original composition of the parent rock. The main agents responsible for this process are exfoliation, unloading, erosion, freezing, and thawing. Chemical weathering causes both reductions in size and chemical alteration of the original parent rock. The main agents responsible for chemical weathering are hydration, carbonation, and oxidation. Often, chemical and physical weathering takes place in concert (Geotechnical Engineering by V.N.S Murthy).

2.2. General types of soils

It has been discussed earlier that soil is formed by the process of physical and chemical weathering. The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) to the largest possible (boulders). This implies that all the weathered constituents of a parent rock cannot be termed soil. According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials (Geotechnical Engineering by V.N.S Murthy).

2.2.1. Residual and Transported Soils

On the basis of origin of their constituents, soils can be divided into two large groups these are Residual soils, and Transported soils.

i. ***Residual soils*** are those that remain at the place of their formation as a result of chemical weathering of parent rocks and may be found on level rock surfaces where the action of elements has produced a soil with little tendency to move. Residual soils can also occur whenever the rate of breakup of the rock exceeds the rate of removal. 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. Residual soils include topsoil and laterites. Laterites are formed by chemical weathering under warm, humid tropical conditions when the rain water leaches out the soluble rock material leaving behind the insoluble hydroxide of iron and aluminum, giving them their characteristic red-brown color. An important characteristic of residual soil is that the sizes of grains are indefinite. For example, when a residual sample is sieved, the amount passing any given sieve size depends greatly on the time and energy expended in shaking, because of the partially disintegrated condition.

ii. ***Transported soils*** are soils that are found at locations far removed from their place of formation. The transporting agents of such soils are glaciers, wind and water. These soils include gravels, sands, silts and clays. As a stream or river loses its velocity it tends to some of the particles that it is carrying, dropping the larger, heavier particles first. Hence, on higher reaches of a river gravel and sand are found whilst on the lower parts silts and clays predominate. Common descriptive terms such as gravels, sands, silts, and clays are used to identify specific textures in soils. One can refer to these soil textures as soil types; that is, Sands and gravels are grouped together as coarse-grained soils. Clays and silts are fine-grained soils. To characterize fine-grained soils, one need further information on the types of minerals present and their contents. The response of fine-grained soils to loads, known as the mechanical behavior, depends on the type of predominant minerals present.

Many of these transported soils are loose and soft to a depth of several hundred feet. Therefore, difficulties with foundations and other types of construction are generally associated with transported soils (Geotechnical Engineering by V.N.S Murthy).

2.3. Soil structure

Soil structure is defined as the geometric arrangement of soil particles with respect to one another. Among the majority factors that affect the structure of the soil are the shape, size, and mineralogical composition of soil particles, and the nature and composition of soil water. The structure of soils that is formed by natural deposition can be altered by external forces.

2.3.1. Structures in Cohesionless Soil

The structures generally encountered in Cohesionless soils can be divided into two major categories: single grained and honeycombed. In single-grained structures soil particles are in stable positions, with each particle in contact with the surrounding ones. The shape and size distribution of the soil particles and their relative positions influence the denseness of packing; thus, a wide range of void ratios is possible.

In honeycombed structure relatively fine sand and silt form small arches with chains of particles. Soils that exhibit a honeycombed structure have large void ratios, and they can carry an oridinary static load. However, under a heavy load or when subjected to shock loading, the structure breaks down, which results in a large amount of settlement.

2.3.2. Structures in cohesive soils

As described previously the particles in cohesive soils are flaky and sheet-like and retain a significant amount of adsorbed water on their surfaces. The ability of sheet-like particles, to slide relative to one another, gives a cohesive soil the property known as plasticity. The structure of cohesive soils is highly complex. Macrostructures of clay soils are summarized below

Dispersed structures are formed by settlement of individual clay particles. All the particles are oriented more or less parallel to one another.

Flocculent structures are formed by settlement of flocs of clay particles. Clays that have flocculent structures are light weight and possess high void ratios. Clays formed in the sea are highly flocculent. Most of the deposits formed from freshwater possess an intermediate structure between dispersed and flocculent.

Domains are aggregated or flocculated submicroscopic units of clay particles.

Clusters are formed from group of domains. They can be seen under light microscope.

Peds are formed from group of clusters. They can be seen without microscope. Groups of peds are macrostructural features along with joints and fissures (Principles of Geotechnical Engineering 5th edition by Braja M. Das)

2.4. Soil particle size and shape

Irrespective of the origin of soil, the sizes of particles in general, that make up soil, vary over a wide range. Soils are generally called gravel, sand, silt, or clay, depending on the predominant size of particles within the soil. 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, constitutes the finer fractions of soils. The shape of grains smaller than 1μ can be determined by means of an electron microscope. The molecular structure of soil particles can be investigated by means of X-ray analysis.

The shape of particles present in a soil mass is equally as important as the particle size distribution because it has significant influence on the physical properties of a given soil. However, not much attention is paid to particle shape because it is more difficult to measure. The individual grains of gravel and sand may be angular, sub-angular, sub-rounded, rounded or well-rounded. Silt and clay constitute the finer fractions of the soil. Silt and clay constitute the finer fractions of the soil. The particles may be angular, flake-shaped or sometimes needle-like (Elements of Soil mechanics seventh edition by G.N. Smith and Ian G. N. Smith).

2.5. Soil mineralogical composition

Soil minerals are inorganic particles which are derived from weathered parent material and decayed plants and animals. Gravels are pieces of rocks with occasional particles of quartz, feldspar and other minerals. Sand particles are made of mostly quartz and feldspar. Silts are the microscopic soil fractions that consist of very fine quartz grains and some flake-shaped particles that are fragments of micaceous minerals. Clays are mostly flake-shaped microscopic and submicroscopic particles of mica and other minerals. Clays are defined as those particles "which develop plasticity when mixed with a limited amount of water" (Grim, 1953). Clay minerals are almost always the result of chemical weathering of rock particles and are hydrates of aluminum, iron or magnesium silicate combined to create sheet-like structures. These sheets are built from two basic units, the tetrahedral unit of silica and the octahedral unit of the hydroxide of aluminum, iron or magnesium.

The three main groups of clay minerals are as follows:

i. Kaolinite group

This is the most dominant part of residual clay deposits and is made up from large stacks of alternating single tetrahedral sheets of silicate and octahedral sheets of aluminum. Kaolinites are very stable with strong structure and absorb little water. They have low swelling and shrinkage responses to water content variation.

ii. Illite group

This mineral consists of a series of single octahedral sheets of aluminum sandwiched between two tetrahedral sheets of silicon. Illites tend to absorb more water than kaolinites and have higher swelling and shrinkage characteristics.

iii. Montmorillonite group

This mineral has a similar structure to Illite group but, in the tetrahedral sheets, some of the silicon is replaced by iron, magnesium and aluminum. Montmorillonite exhibit extremely high water absorption, swelling and shrinkage characteristics (Elements of Soil mechanics seventh edition by G.N. Smith and Ian G. N. Smith).

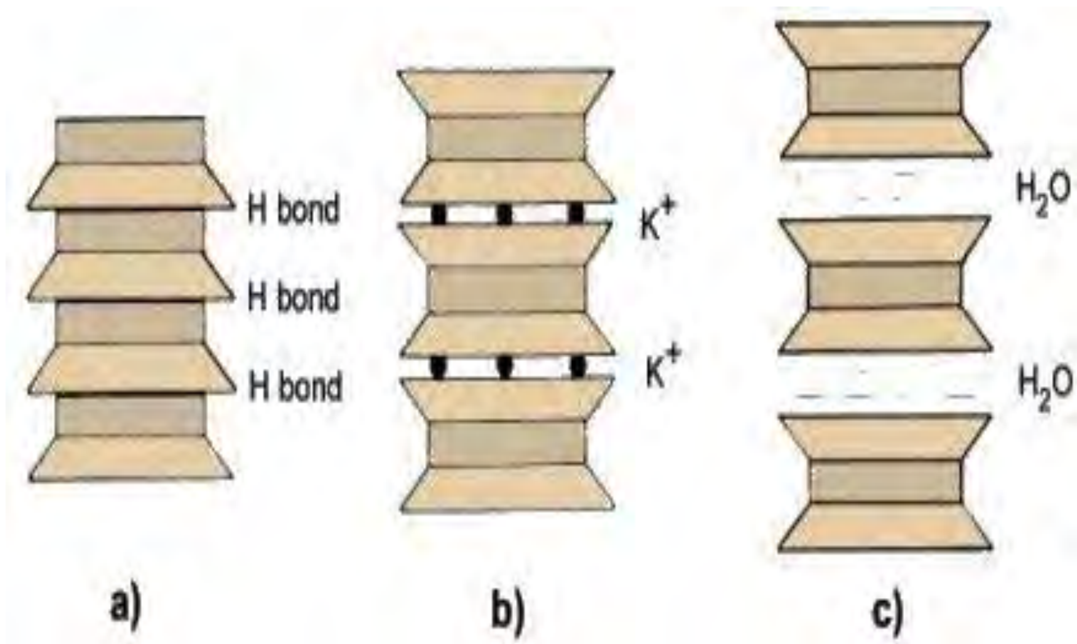

Fig 2.1 Structure of the main clay minerals: (a) kaolinite, (b) Illite and (c) Montmorillonite, based on combined sheets (from Craig, 1990)

3. Description of the study area

3.1. General

The town of Woliso is one of the highly populated towns in Oromia national regional state. It is located at 8°32'00" N latitude and 37°59'00" E longitude and located at a distance of 114 km from Addis Ababa, the capital city of Ethiopia. The altitude of Woliso is about 1900 meter above sea level with annual rain fall of 1200ml and temperature of 18-27^{0}c. Woliso was founded in 1927. Since then the town has passed through different social, economic and political reforms. At the moment the town of Woliso has area coverage of 2,225.25 hectare and a population 53,000. In 2003 the town was put under reform by urban proclamation 65/2003 of Oromia Regional sate. The town is serving as a seat to Woliso Wereda and South West Shoa Zone Administration. The town has its own administrative structure which is led by mayor. And is sub divided into four urban Kebeles.

Ethnically the population is composed of almost every nation and nationalities of Ethiopia. Because of its economic and political importance Woliso is becoming the center of different governmental and private investment sectors.

The vital factor that makes Woliso an ideal town is the life style of its population. Regardless of ethnic and religious diversity, the population lives in harmony and tolerance. Industriousness and hospitality are the basic assets of people of Woliso, making it unforgettable by anyone who spends a while in Woliso.

3.2. Climate

3.2.1. Temperature

Woliso town, with an altitude of 1900 meters above sea level has average maximum, average minimum and average monthly temperature of 25.5^{0}c, 13.5^{0}c and 19.5^{0}c respectively. Average temperature of Woliso town is 19.5^{0}c which is categorized as 'woina dega' or sub-tropical climatic region. From Fig 3.1 and 3.2 one can observe that the highest temperatures are during months January, February, March, April and May whereas July and August have low temperatures.

3.2.2. Rainfall

The records of National Meteorological Service Agency from woliso observatory substation show that the mean annual rainfall for five years i.e. from 2008 to 2012 at an altitude of 2058m above mean sea level, latitude of 8^{0} 33' and longitude of 37^{0}59' is 1092.76mm. As shown in Fig 3.3, more than 50% of the rainfall is in the months of June, July, August and September followed by long dry season.

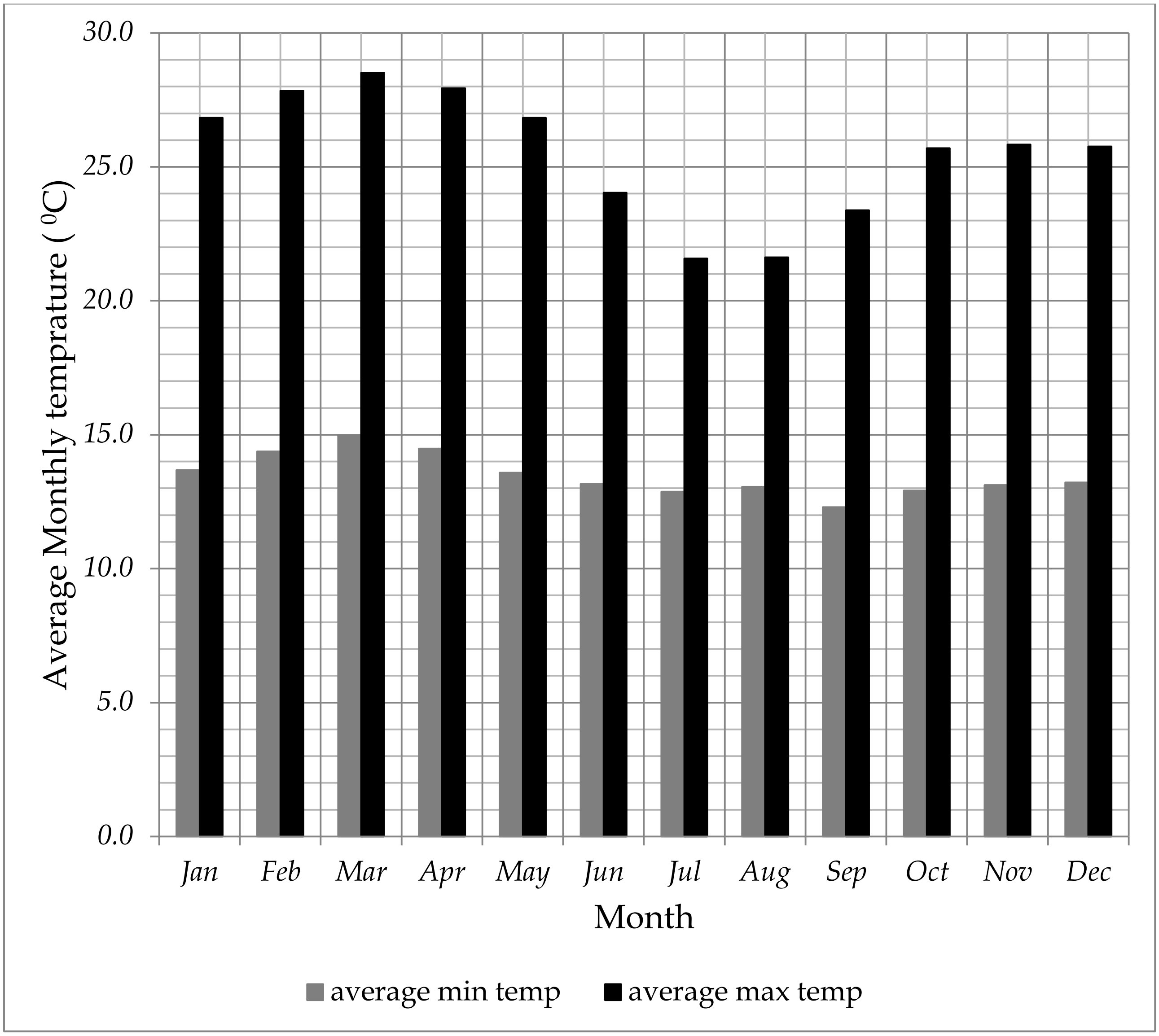

Fig 3.1 Average maximum and minimum monthly temperature distribution of Woliso, (2008 – 2012 G.C.)

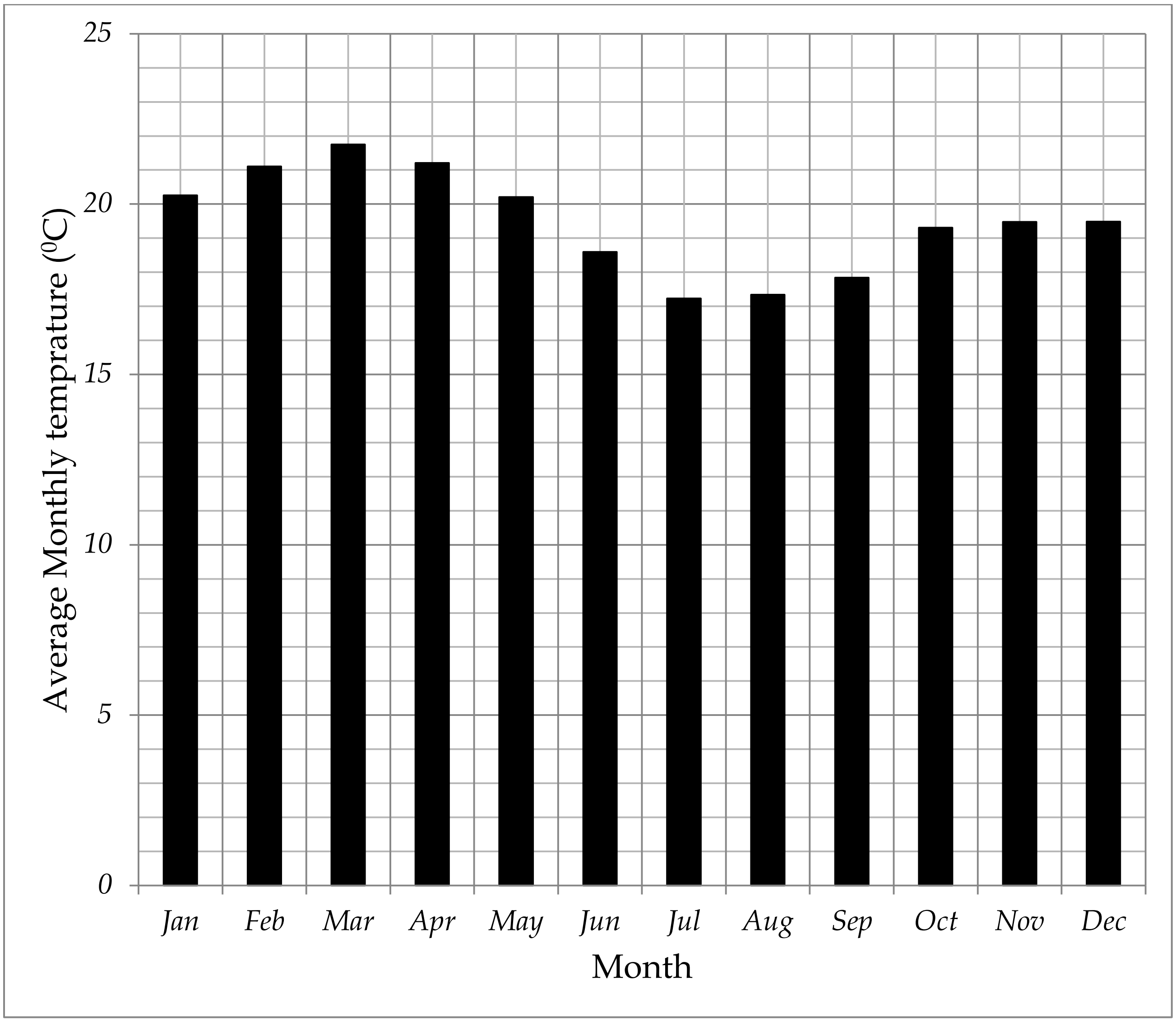

Fig 3.2 Average monthly temperature distribution of Woliso, (2008 – 2012 G.C.)

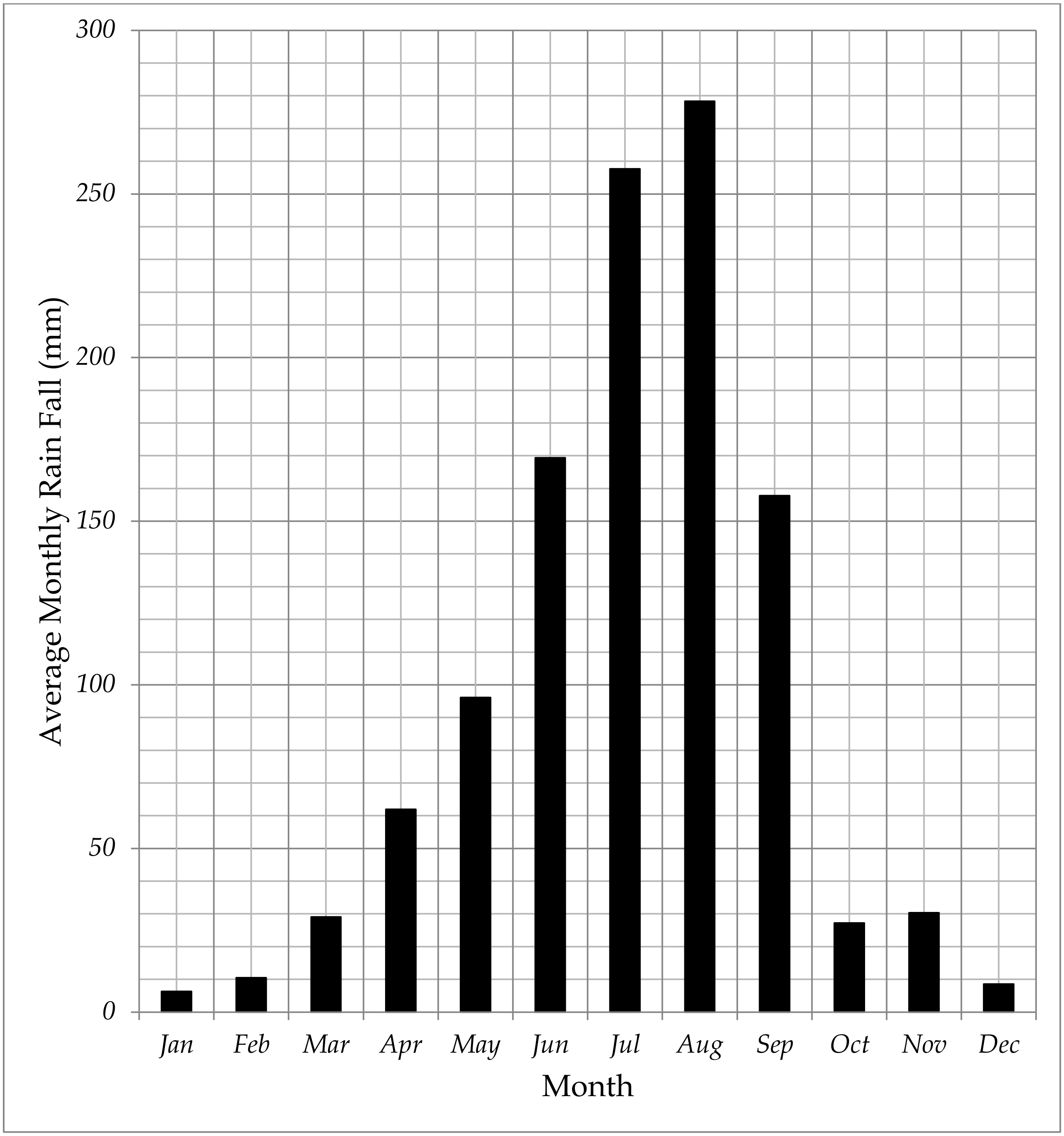

Fig 3.3 Mean monthly Rainfall distribution of woliso town (2008 – 2012 G.C.)

4. In-situ properties and laboratory test results

4.1. In-situ properties

4.1.1. Identification of soil in the study area

The soil specimens for this study were collected from Woliso town. Prior to sampling, visual site investigations and information from residents and construction firms were collected to consider the different soil types and to take representative samples evenly in the whole town. Accordingly, six sampling areas were selected from different locations of the town which are supposed to represent all types of the soils found in the town from preliminary site investigation. By visiting the ongoing construction areas and information from construction firms there are two types of soils found in the study area, which are red clay and black cotton soil. Pits were excavated to the maximum depth of three meters. Disturbed and undisturbed samples were collected for this work and taken to laboratory for testing. Each tests listed on objectives were done for all samples taken except for one dimensional consolidation test is done only for four samples at a depth of three meters for TP-1, TP-2, TP-3 and TP-4. The location of test pits is shown on map of Woliso town in Fig 4.1.

Table 4.1 sampling depth and location of sampling area

Test pit	Sampling depth(m)	Pit location	Observed color
TP-1	1.5	Around stadium	Black
	3		Black
TP-2	1.5	Near prison	Black
	3		Grayish
TP-3	1.5	Around catholic church	Black
	3		Grayish
TP-4	1.5	Around negash lodge	Reddish
	2.7		Reddish
TP-5	1.5	Around Woliso Health Center	Reddish
	2.8		Reddish
TP-6	1.5	Around university	Black
	3		Grayish

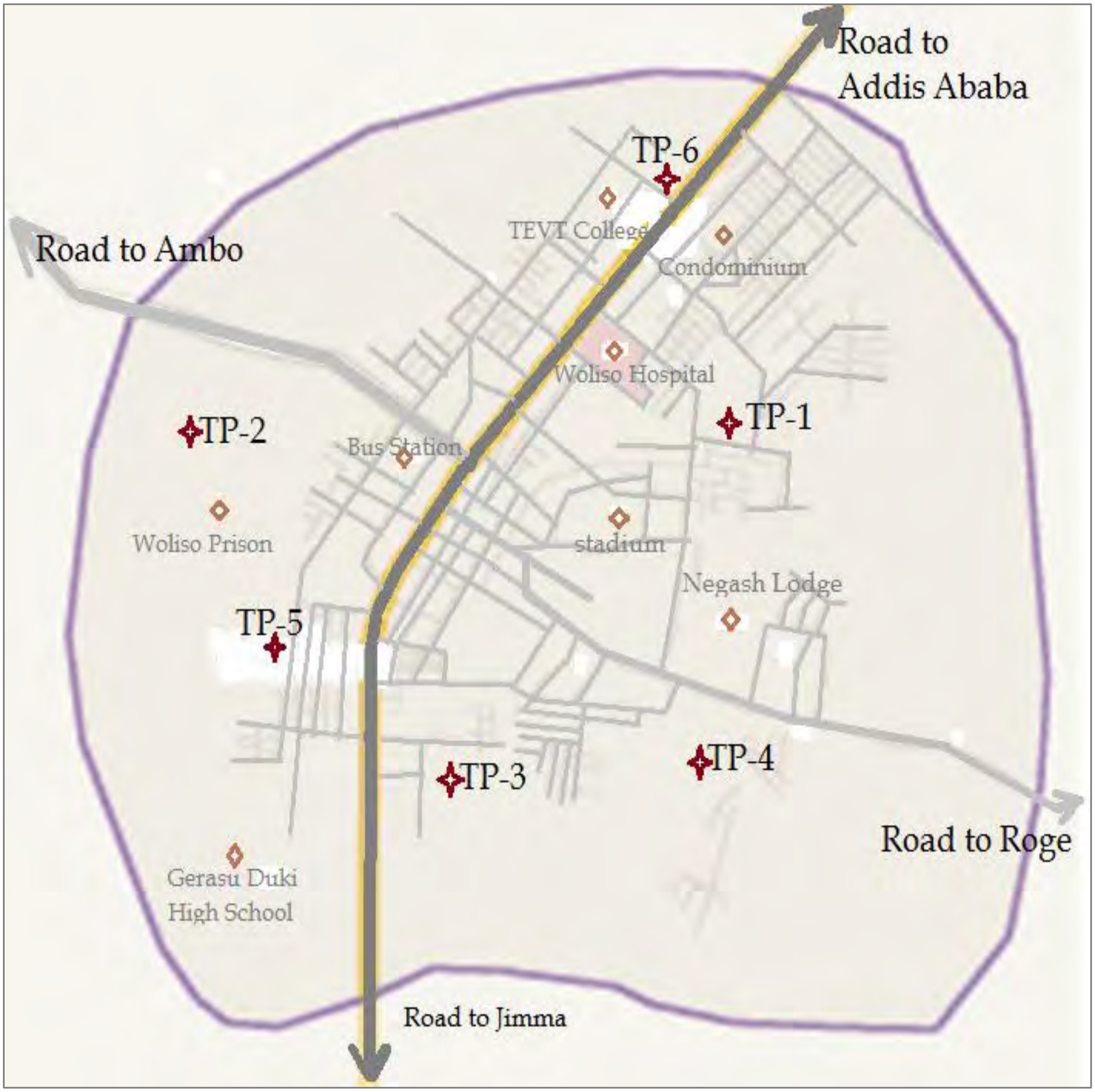

Fig 4.1 Test pit locations on map of woliso town

4.1.2. In-situ properties Description

From each test pits disturbed and undisturbed samples were taken to laboratory. Disturbed samples are used for performing classification tests. Undisturbed samples are used to determine in-situ properties of the soil such as natural moisture content, in situ density, shearing resistance and stress-deformation characteristics of the soil.

4.1.2.1. Natural moisture content

Natural moisture content of the soil is determined in the laboratory by using undisturbed sample. In the laboratory this test is done by referring ASTM D 2216-98 standard. A test specimen is first weighed in its natural or wet state is then dried in oven at a temperature of 105°C to a constant mass for 24 hrs. The loss of mass due to drying is considered to be water. The water content is calculated using the mass of water and the mass of the dry specimen. The natural moisture content of each sample is summarized in Table 4.2.

Table 4.2 Natural moisture content of samples

Test pit	Sampling depth(m)	Natural moisture content (%)
TP-1	1.5	31.2
	3	29.4
TP-2	1.5	34
	3	66.5
TP-3	1.5	37.6
	3	40.0
TP-4	1.5	27.2
	2.7	35.4
TP-5	1.5	36.5
	2.8	34
TP-6	1.5	25.5
	3	32.9

4.2. Index properties

4.2.1. General

Index properties are basic for distinguishing soils. Index properties may be divided into two main categories namely, soil grain properties and soil aggregate properties. The soil grain properties are the properties of the individual grains as expressed by size, shape, and mineralogical characteristics. The soil aggregate properties are the properties of the soil mass as a whole. The most significant aggregate property of cohesion less soils is the relative density, whereas that of cohesive soils is the consistency (Soil Mechanics by Alemayehu Teferra and Mesfin Leikun).

Index tests are the most basic types of laboratory tests performed on soil samples. Index tests include the water content (also known as moisture content), specific gravity tests, unit weight determinations, and particle size distributions and Atterberg limits, which are used to classify the soil (Soil Mechanics and Foundation Engineering by Dr.K.R Arora)

4.2.2. Specific gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the laboratory. The specific gravity of a soil is used in calculating the phase relationships of soils.

The specific gravity of the minerals affects the specific gravity of soils derived from them. Most of the values fall within a range of 2.6 to 2.9. The specific gravity of solids of light-colored sand, which is mostly made of quartz, may be estimated to be about 2.65; for clayey and silty soils, it may vary from 2.6 to 2.9 (Principles of Geotechnical Engineering 5th edition by Braja M.Das).

According to ASTM D 854-98, two procedures for performing specific gravity are provided. These are Method-A, procedures for oven dried specimen and Method-B, procedure for moist specimen. For specimens of organic soils and highly plastic, fine grained soils, Procedure B shall be the preferred method. Procedure A is used to prepare soil samples for plasticity tests and particle-size analysis when the coarse-grained particles of a sample are soft and pulverize readily or when the fine particles are very cohesive and tend to resist removal from the coarse particles. In this research specific gravities are determined using method A due to high cohesive behavior of the soil specimens it was not workable to sieve the sample in moist condition. The test results are summarized in table 4.3. From the table one can observe that the specific gravities of the soils range from 2.65 to 2.76, which is within the above specified range.

Table 4.3 specific gravity of samples

Test pit	Sampling depth(m)	Specific gravity	Water used for testing
TP-1	1.5	2.65	Tap water
	3	2.66	"
TP-2	1.5	2.66	"
	3	2.68	"
TP-3	1.5	2.75	"
	3	2.72	"
TP-4	1.5	2.75	"
	2.7	2.76	"
TP-5	1.5	2.73	"
	2.8	2.73	"
TP-6	1.5	2.65	"
	3	2.67	"

4.2.3. Bulk unit weight

The bulk unit weight (also known as the total unit weight) is the natural in situ unit weight of the soil; therefore it should only be obtained from undisturbed soil specimens. The first step in the laboratory is to determine the bulk density using undisturbed sample. The bulk density is the ratio of total mass of the soil in natural state to the total volume of the soil sample. The in-place density is used to determine density of compacted soils used in the construction of structural fills, highway embankments, or earth dams. Typical values for bulk unit weight are 17 to 20 kN/m^3.
The bulk unit weight and density of soil samples taken from woliso town are summarized in Table 4.4.

Table 4.4 Bulk Unit weight and Density of samples from woliso town

Test pit	Sampling depth(m)	Bulk Density (g/cm³)	Bulk U nit Weight(kN/m³)
TP-1	1.5	1.78	17.8
	3	1.88	18.9
TP-2	1.5	1.76	17.6
	3	1.71	17.1
TP-3	1.5	1.81	18.1
	3	1.76	17.6
TP-4	1.5	1.72	17.2
	2.7	1.79	17.9
TP-5	1.5	1.79	17.9
	2.8	1.86	18.6
TP-6	1.5	1.75	17.5
	3	1.70	17

4.2.4. Particle size analysis

4.2.4.1. General

Particle size analysis, also known as Mechanical analysis, is one of the oldest and most common forms of soil analysis. It expresses quantitatively the proportions, by mass, of various sizes of particles present in a soil. It provides the basic information for revealing the uniformity or gradation of the materials within established size ranges and for textural classifications. The results of a mechanical analysis are not equally valuable in different branches of engineering. The size of the soil grains is of importance in such cases as construction of earth dams or railroad and highway embankments, where earth is used as a material that should satisfy definite specifications. In foundations of structures, data from mechanical analyses are generally illustrative; other properties such as compressibility and shearing resistance are of more importance.
Particle size analysis is done in two stages: (i) Sieve Analysis, (ii) Hydrometer Analysis.
The normal method adopted for separation of particles in a fine grained soil mass is the hydrometer analysis and for the coarse grained soils the sieve analysis. The test method covers the quantitative determination of the distribution of particle sizes in soils (Soil Mechanics and Foundation Engineering by Dr.K.R Arora)

The data are presented on a semi-log plot of percent finer vs. particle diameters and combined with the data from a sieve analysis of the soil sample retained on the No.200 sieve. The principal value of the hydrometer analysis appears to be to obtain the silt and clay fraction.
The combined grain size distribution curve for particles is shown in figure 4.2 and 4.3. It can be seen that the soils for almost all samples are fine grained. The gradation of soils in the study area varies significantly (Table 4.5). From the grain size analysis results, clay content range from 42.5 – 85%, silt fraction range from 12.8 – 51.1%, sand fraction range from 2.2 – 9.7% and gravel content range from 0.0 – 11.4%.

Table 4.5 summery of grain size analysis result

Test pit	Sampling depth(m)	Percent amount of particle size			
		Gravel	Sand	Silt	Clay
TP-1	1.5	0.1	7.1	18.8	74
	3	3.5	9.7	16.8	70
TP-2	1.5	1.5	9.7	14.8	74
	3	1	5.2	13.8	80
TP-3	1.5	6.4	6.4	22.2	65
	3	1	4.5	22.5	72
TP-4	1.5	0.4	7.5	40.6	51.5
	2.7	0	6.4	51.1	42.5
TP-5	1.5	0	2.2	12.8	85
	2.8	0.1	4.1	16.8	79
TP-6	1.5	7.7	7.3	16	69
	3	11.4	5.4	28.2	55

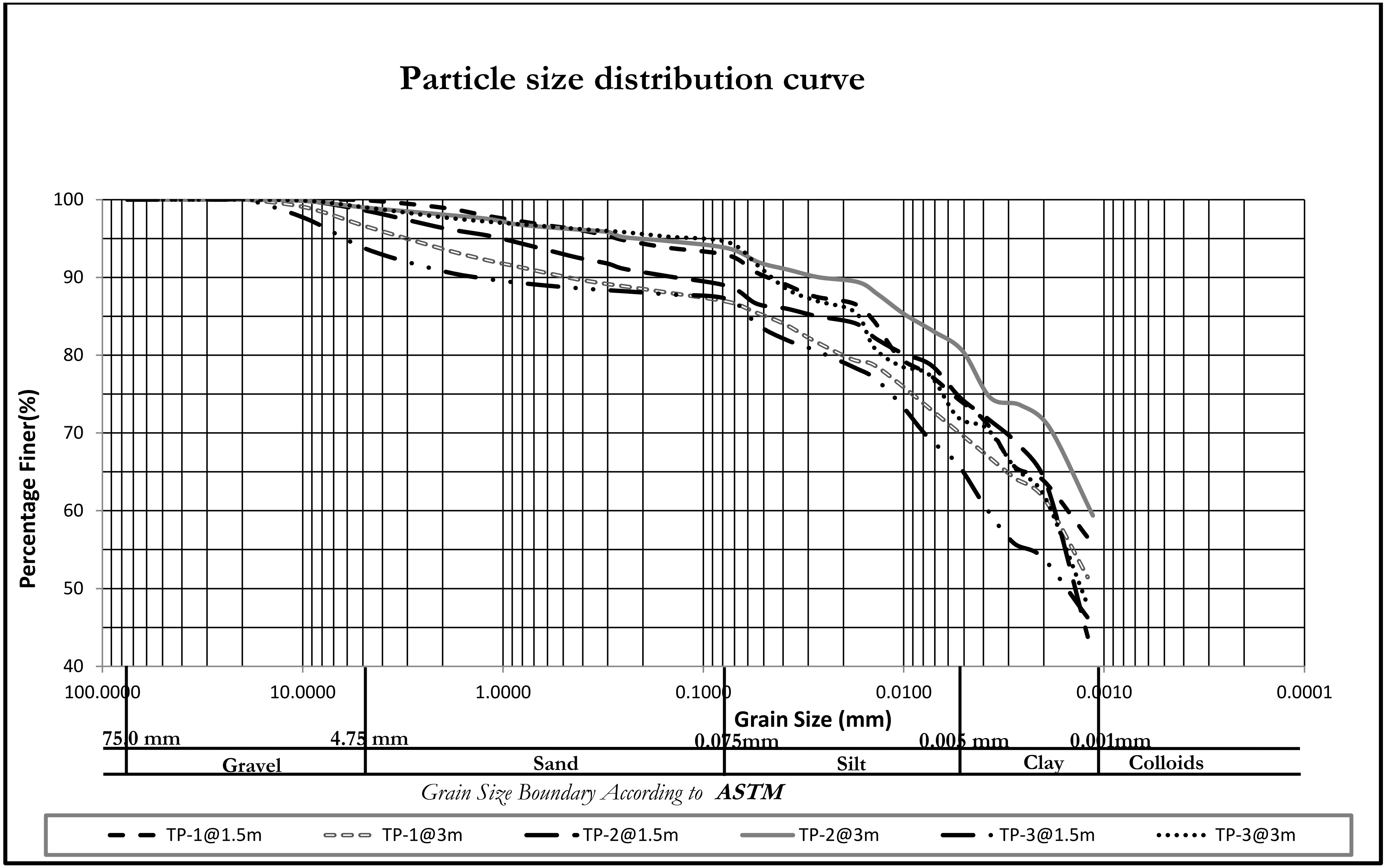

Fig 4.2 Grain size distribution curve for samples from TP-1 to TP-3

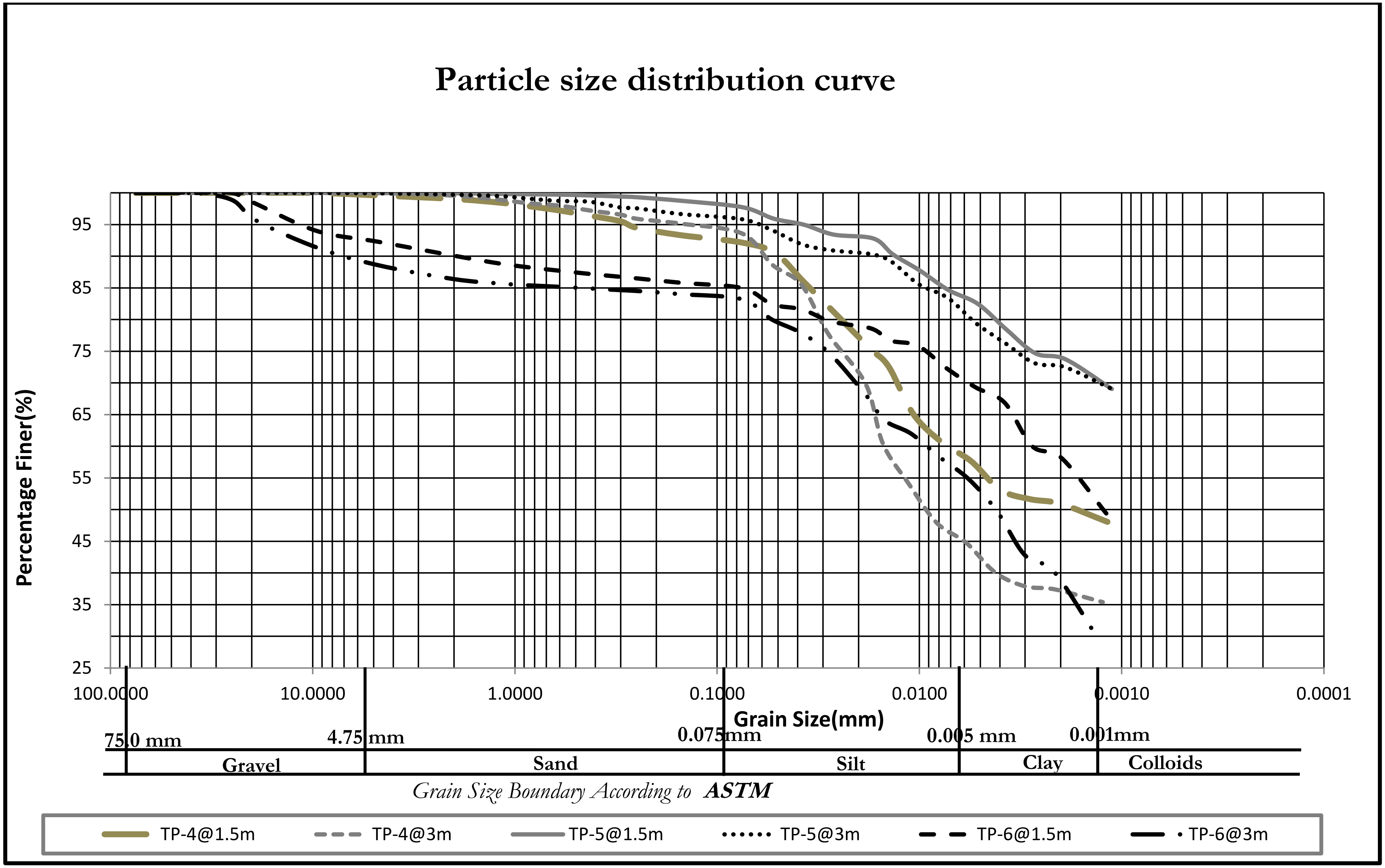

Fig 4.3 Grain size distribution curve for samples from TP-4 to TP-6

4.2.5. Atterberg limits

Atterberg limits or consistency limits are water contents at which the soil changes from one state to the other. Soil consistency is a term used to describe the degree of firmness of soil and is expressed by such terms as soft, firm or hard. It usually applies to fine grained soils whose condition is affected by changes in moisture content. Consistency limits are very important index properties of fine grained soils. As the consistency of soil changes, its engineering properties also change. Such soil properties as shearing strength and bearing capacity vary significantly with consistency. The Swedish scientist, Atterberg, established the four states of soil consistency (fig 4.4) which are called the liquid, the plastic, the semi-solid, and the solid states. He also proposed a series of tests for determining the boundaries known as Atterberg limits between the physical states of soil. Each boundary or limits is defined by the water content that produces a specified consistency.

A soil containing high water content is in a liquid state. It offers no shearing resistance and can flow like liquids. As the water content is reduced, the soil becomes stiffer and starts developing resistance to shear deformation. At some particular water content, the soil becomes plastic. The water content at which the soil changes from the liquid state to the plastic state is known as liquid limit. The soil in the plastic state can be moulded into various shapes. As the water content is reduced, the soil passes from the plastic state to semi-solid state where it stops behaving as a plastic. It cracks when moulded. The water content at which the soil becomes semi-solid is known as the plastic limit. The water content at which the soil changes from semi-solid state to solid state is known as shrinkage limit.

states	limit	consistency	Volume change
Liquid ……………….LL Plastic ……………….PL Semi-solid ……………….SL Solid	 Liquid limit………… Plastic limit……….... Shrinkage limit……..	Very soft Soft Stiff Very stiff Extremely stiff Hard	↑ Decrease in volume ↓ Constant volume

Fig 4.4 States of soil consistency and Atterberg limits

A typical plot of water content against the log of blows was made as shown in Fig. 4.5. Within the range of 5 to 40 blows, the plotted points lie almost on a straight line. The curve so obtained is known as a '*flow curve*'. The water content corresponding to 25 blows is termed the *liquid limit.*

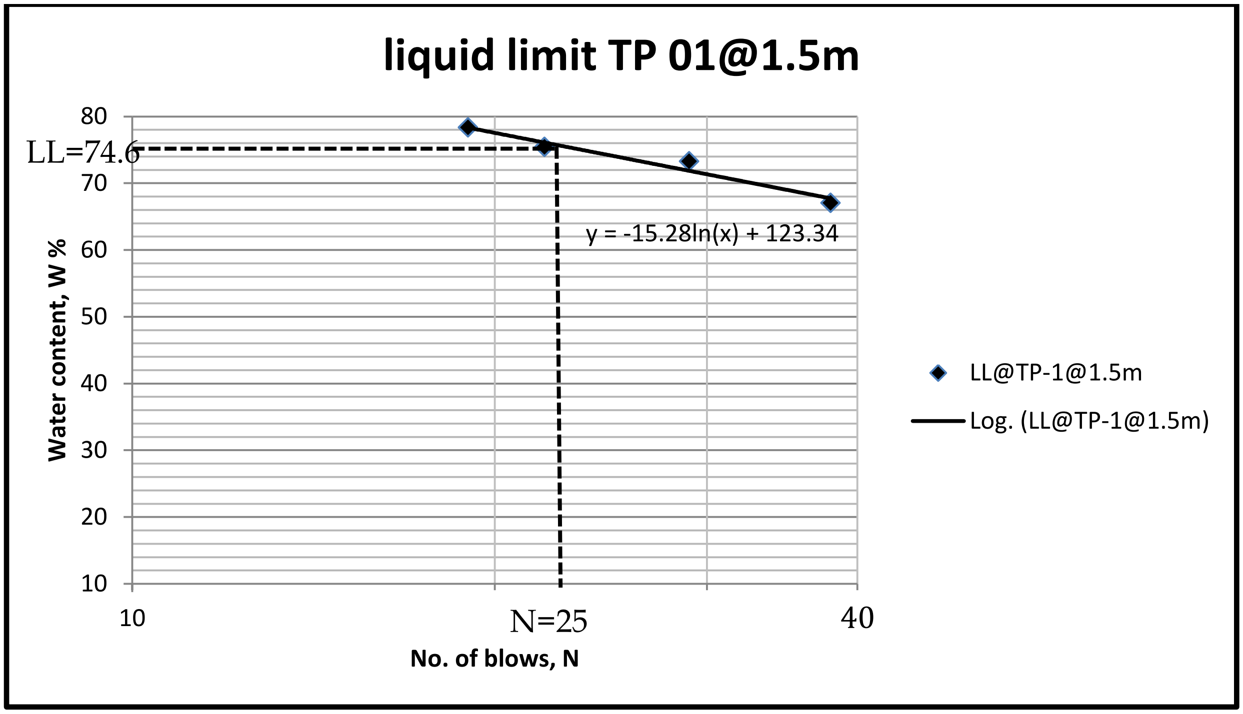

Fig 4.5 Typical water content against log of number of blows

Table 4.6 Summery of Atterberg limits of the study area

Test pit	Sample depth(m)	Testing condition	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Clay % finer than 0.002µm	Activity
TP-1	1.5	Oven dried	74.6	31.8	42.8	60	0.86
	3	"	70.5	31.2	39.3	54	0.89
TP-2	1.5	"	103.6	41.8	61.8	57	1.31
	3	"	126.1	44.4	81.7	67	1.43
TP-3	1.5	"	81.8	36.3	44	53	1.06

	3	"	93.8	39.2	54.6	61	1.07
TP-4	1.5	Oven dried	60	31.4	28.5	52	0.71
		Air dried	61.1	31.4	29.7	-	-
	2.7	Oven dried	56.5	30.3	26.2	50	0.72
		Air dried	60.4	31.5	28.9	-	-
TP-5	1.5	Oven dried	57.3	32.2	25.1	74	0.39
		Air dried	55.7	30.8	24.9	-	-
	2.8	Oven dried	56.2	30.6	25.7	72	0.41
		Air dried	58.7	34.4	24.3	-	-
TP-6	1.5	Oven dried	90.7	36.9	53.8	58	1.12
	3	"	81.9	34.7	47.2	42	1.47

From the above table one can see that TP-4 & TP-5 have low activity this implies that the samples taken are non-expansive soils.

4.2.6. Free swell

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

$$\text{Free swell} = \frac{\text{Final volume - Initial volume of the soil}}{\text{Initial volume}} * 100\% \quad \ldots\ldots\ldots\ldots 4.1$$

Free swell test results for the study area by using oven dried samples are summarized in table 4.7. From the test result one can see that the free swell of the soil under investigation ranges from 40% to 250%. Those soils having free swell less than 50% (TP-4 &TP-5) are considered as non-expansive, with free swell between 50%-100% are considered as marginal and soils having free swell greater than 100% are considered as expansive. Therefore TP-4 and TP-5 are non-expansive and TP-1, TP-2, TP-3 and TP-6 are expansive.

Table 4.7 free swell of soils in the study area

Test pit	Sampling depth(m)	Test condition	Water used for testing	Free swell (%)
TP-1	1.5	Oven dried	Tap water	80
	3	"	"	110
TP-2	1.5	"	"	217.5
	3	"	"	250
TP-3	1.5	"	"	155
	3	"	"	147.5
TP-4	1.5	"	"	40
	2.7	"	"	34.5
TP-5	1.5	"	"	47.5
	2.8	"	"	47.5
TP-6	1.5	"	"	155
	3	"	"	147.5

4.3. Classification of the soils

4.3.1. General

Soil classification system is the arrangements of soils into different groups such that, the soils in particular group have similar behavior. Since there are a wide variety of soils covering the earth, it is desirable to classify the soils into broad groups of similar behavior. It is more convenient to study the behavior of groups than that of individual soils. The main purpose of soil classification is to make possible estimation of soil properties by association with soils of the same class whose properties are known and to provide the Engineer with accurate method of soils description. A classification system thus provides a common language between engineers dealing with soils. A soil is classified according to index properties, such as particle size and plasticity characteristics. Grouping of soils on the basis of certain definite principles would also help the engineer to rate the performance of a given soil either as a sub-base material for roads and airfield pavements, foundations of structures, etc. Many systems are in use that is based on grain size distribution and Atterberg limits of soil. The systems that are quite popular amongst engineers are the AASHTO Soil Classification System and the Unified Soil Classification System (USCS). These methods are among the widely used classification systems in our country and the classification results are shown in Table 4.9 and 4.10.

Average grain size classification according to ASTM, 1998

Gravel	76.2mm – 4.75mm
Sand	4.75mm – 0.075mm
Coarse sand	4.75mm – 2mm
Medium sand	2mm - 0.425mm
Fine sand	0.425 – 0.075
Silt size	0.074 to 0.005 mm
Clay size	< 0.005 mm
Colloids	< 0.001 mm

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

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

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

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

4.3.2. Classification of soils based on Unified Soil Classification System (USCS)

The unified soil classification system is the most popular system for use in all types of engineering problems involving soils and shall be used when precise classification is required.

The Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil into three major divisions: coarse-grained soils, fine grained soils, and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the coarse-grained soils and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used. The peaty soils are readily identified by color, odor, spongy feel and fibrous texture.

According to unified soil classification system as shown in table 4.8 below the Black soils of the study area fall under CH region, which shows that the soils are inorganic clays of high plasticity and the red soils fall under MH region, which are inorganic silts of high plasticity. According grain size analysis result red colored soils have high clay fraction thus it is better to classify the soil as red clay rather than silt.

The distinction between the inorganic and organic soils is made by visual appearance and odor by USC method. In the ASTM method more specifically defined by comparison of the oven-dried liquid limit (LL) and air-dried liquid limit (LL′). If oven dried value LL<0.75LL′ and the appearance and odor indicates "organic" then the soil is classified as organic; otherwise, inorganic. In this case the distinction done by using both methods and the result indicates that the soils are classified as inorganic clays of high plasticity based on unified soil classification method and ASTM method.

Table 4.8 Classification of soils based on USC classification system

Test pit	Sampling depth(m)	Percent amount of particle size				LL (%)	PI (%)	Classification according to UCS
		Gravel	Sand	Silt	Clay			
TP-1	1.5	0.1	7.1	18.8	74	74.6	42.8	CH
	3	3.5	9.7	16.8	70	70.5	39.3	CH
TP-2	1.5	1.5	9.7	14.8	74	103.6	61.8	CH
	3	1	5.2	13.8	80	126.1	81.7	CH
TP-3	1.5	6.4	6.4	22.2	65	81.8	45.5	CH
	3	1	4.5	22.5	72	93.8	54.6	CH
TP-4	1.5	0.4	7.5	40.6	51.5	61.1	29.7	MH
	2.7	0	6.4	51.1	42.5	60.4	28.9	MH
TP-5	1.5	0	2.2	12.8	85	55.7	24.9	MH
	2.8	0.1	4.1	16.8	79	57.7	25.7	MH
TP-6	1.5	7.7	7.3	16	69	90.7	53.8	CH
	3	11.4	5.4	28.2	55	81.9	47.2	CH

4.3.3. Plasticity Chart

The information provided in the plasticity chart is of great value and is the basis for the classification of fine grained soils in the unified soil classification system. The important feature of this chart is the empirical A-line that is given by equation PI=0.73(LL-20). A-line separates the inorganic clays from the inorganic silts. Inorganic clay values lie on or above the A-line, and values for inorganic silts lie below the A-line. Organic silts plot in the same region (below the A-line and with LL ranging from 30 to 50) as the inorganic silts of medium compressibility. On the chart, organic clays plot in the same region as inorganic silts of high compressibility (below the A-line and LL greater than 50).

The U-line lies above the A-line. The U-line is the upper limit of the relationship of the plasticity index to the liquid limit for any currently known soil. The equation for the U-line can be given by equation PI=0.9(LL-8) (Principles of Geotechnical Engineering 5th edition by Braja M. Das).

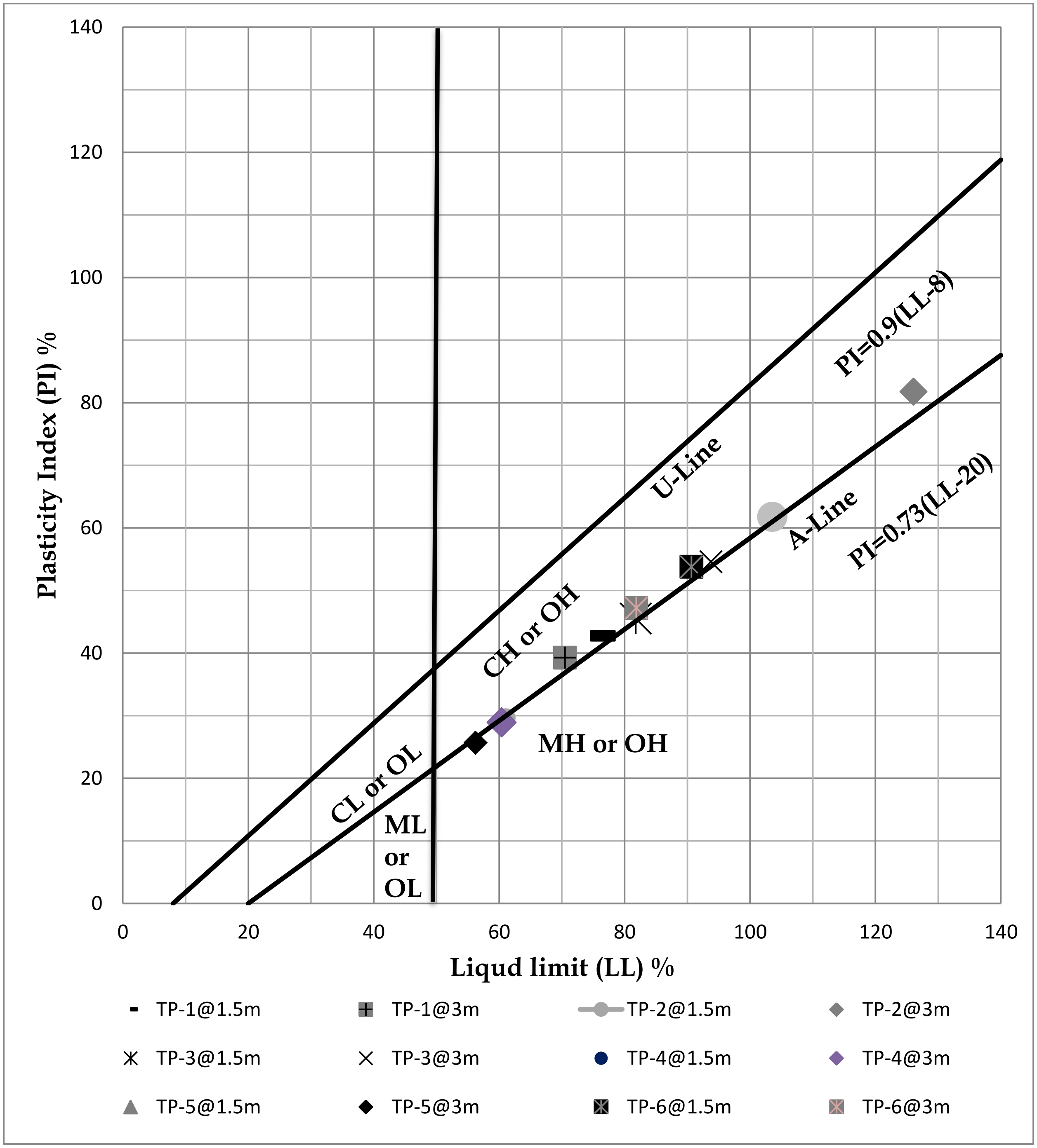

Fig 4.6 plasticity chart for the study area according to unified soil classification system

4.3.4. Classification of soils using AASHTO classification system

American Association of State Highway and Transportation Official (AASHTO) classification system is useful for classifying soils for highways. The particle size analysis and the plasticity characteristics are required to classify a soil. The classification system is a complete system which classifies both coarse-grained and fine-grained soils. In this system the soils are divided into 7 types, designated as A-1 to A-7. The soils A-1 and A-7 are further subdivided into two categories, and the soil A-2, into four categories. To classify a soil, its particle size analysis is done, and plasticity index and liquid limit are determined. The soil with the lowest number, A-1, is the most suitable as a highway material or subgrade. Figure 4.7 shows a plot of the range of the liquid limit and the plasticity index for soils that fall into groups A-2, A-4, A-5, A-6, and A-7.

To evaluate the quality of a soil as a highway subgrade material, one must also incorporate a number called the group index (GI) with the groups and subgroups of the soil. The smaller the value of the group index, the better is the soil in that category. A group index of zero indicates a good subgrade, whereas index of 20 or greater shows a very poor subgrade (Soil Mechanics and Foundation Engineering by Dr. K.R. Arora).

The group index is given by the equation

$$GI = 0.2(F-35) + 0.005(F-35)(LL-40) + 0.01(F-15)(PI-10) \qquad \text{……4.2}$$

Where F = percent passing No.200 sieve

If $F < 35$, use $F-35 = 0$

LL = liquid limit, in percent

PI = plasticity index, in percent

From AASHTO Classification system results shown in table 4.9 and Fig 4.7 one can observe that all the samples collected fall under A-7-5, which are clayey soils with group index ranging from 31 to 98. The group index results indicate that generally the soils of the study area are very poor for highway subgrade material.

Classification of soils of Woliso town according to AASHTO for the collected samples is shown in fig 4.7 below.

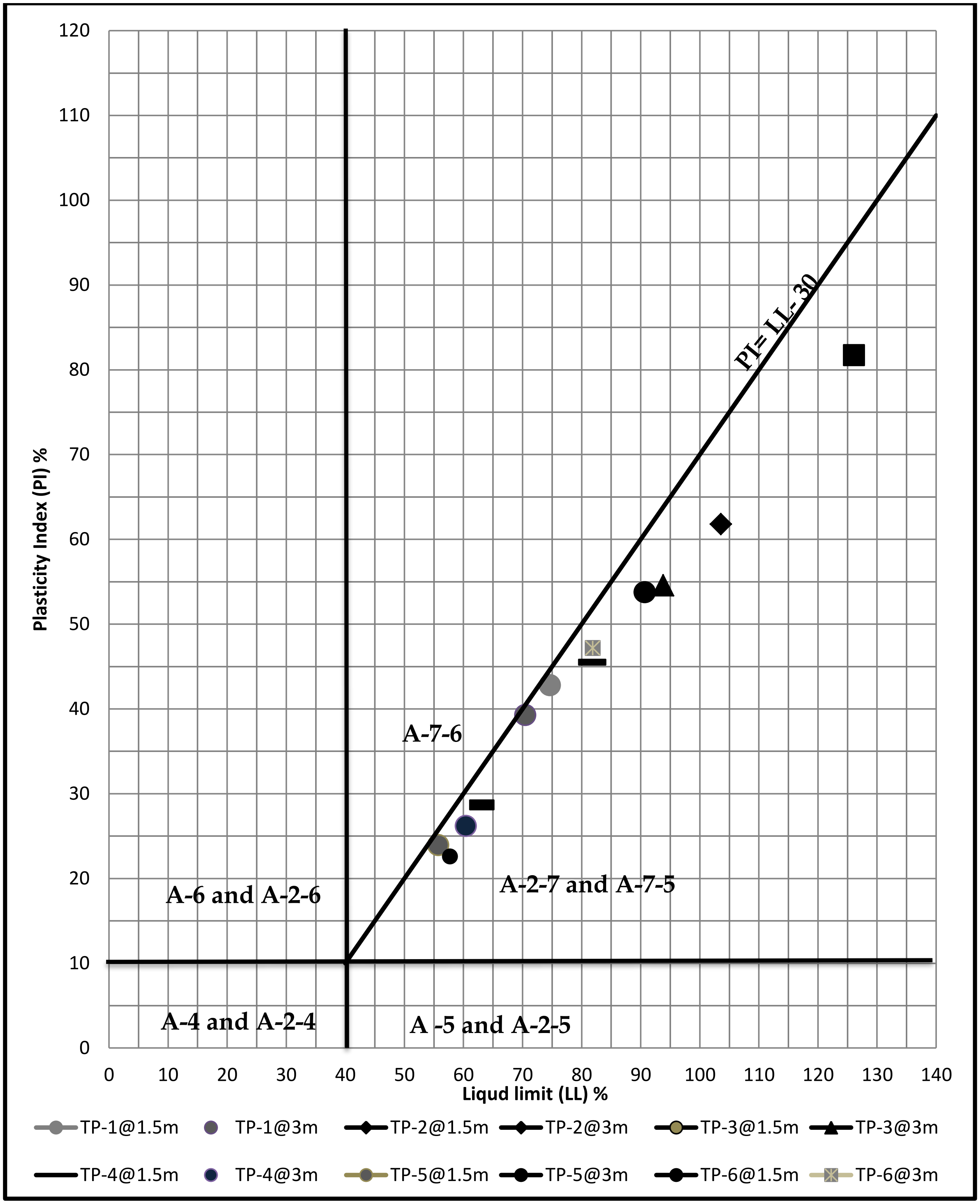

Fig 4.7 plasticity chart used for classifying soils according to AASHTO classification system

Table 4.9 Classification of soils based on AASHTO classification system

Test pit	Sampling depth (m)	Sieve analysis Percent passing			LL (%)	PI (%)	Group index	Group classification	Usual types of significant constituent materials	General rating as sub-grade	Classification according to AASHTO
		No.10	No.40	No.200							
TP-1	1.5	98.96	96.25	95	74.6	42.8	50	A-7-5	Clayey soil	Poor	A-7-5(50)
	3	93.66	89.76	87	70.5	39.3	41	"	"	"	A-7-5(41)
TP-2	1.5	96.36	94.92	93	103.6	61.8	72	"	"	"	A-7-5(72)
	3	98.44	97.85	96	126.1	81.7	98	"	"	"	A-7-5(98)
TP-3	1.5	90.81	88.62	87	81.8	45.5	49	"	"	"	A-7-5(49)
	3	97.73	96.24	95	93.8	54.6	66	"	"	"	A-7-5(66)
TP-4	1.5	99.69	99.28	98	61.1	29.7	37	"	"	"	A-7-5(37)
	2.7	99.97	99.87	99	60.4	28.9	34	"	"	"	A-7-5(34)
TP-5	1.5	99.94	99.6	98	55.7	24.9	31	"	"	"	A-7-5(31)
	2.8	99.72	99.24	98	57.7	25.7	31	"	"	"	A-7-5(31)
TP-6	1.5	90.08	87.2	85	90.7	53.8	55	"	"	"	A-7-5(55)
	3	86.39	84.91	83	81.9	47.2	46	"	"	"	A-7-5(46)

4.3.5. Activity

Because the plasticity of soil is caused by the adsorbed water that surrounds the clay particles, one can expect that the type of clay minerals and their proportional amounts in a soil will affect the liquid and plastic limits. Skempton (1953) observed that the plasticity index of a soil increases linearly with the percentage of clay-size fraction (% finer than 2μm by weight) present. The correlations of PI with the clay-size fractions for different clays plot separate lines (fig 4.8). This difference is due to the diverse plasticity characteristics of the various types of clay minerals. On the basis of these results, Skempton defined a quantity called *activity*, which is the slope of the line correlating PI and % finer than 2μm. This activity may be expressed as:

$$A = \frac{PI}{\%\ of\ clay-size\ fraction, by\ weight} \quad \ldots\ldots\ldots\ldots\ldots 4.3$$

Activity is used as an index for identifying the swelling potential of clay soils. Typical values of activities for samples taken for this study are given in table 4.6. From the chart shown below (fig 4.8) one can see that most of the samples from expansive soils fall on the very highly expansive region.

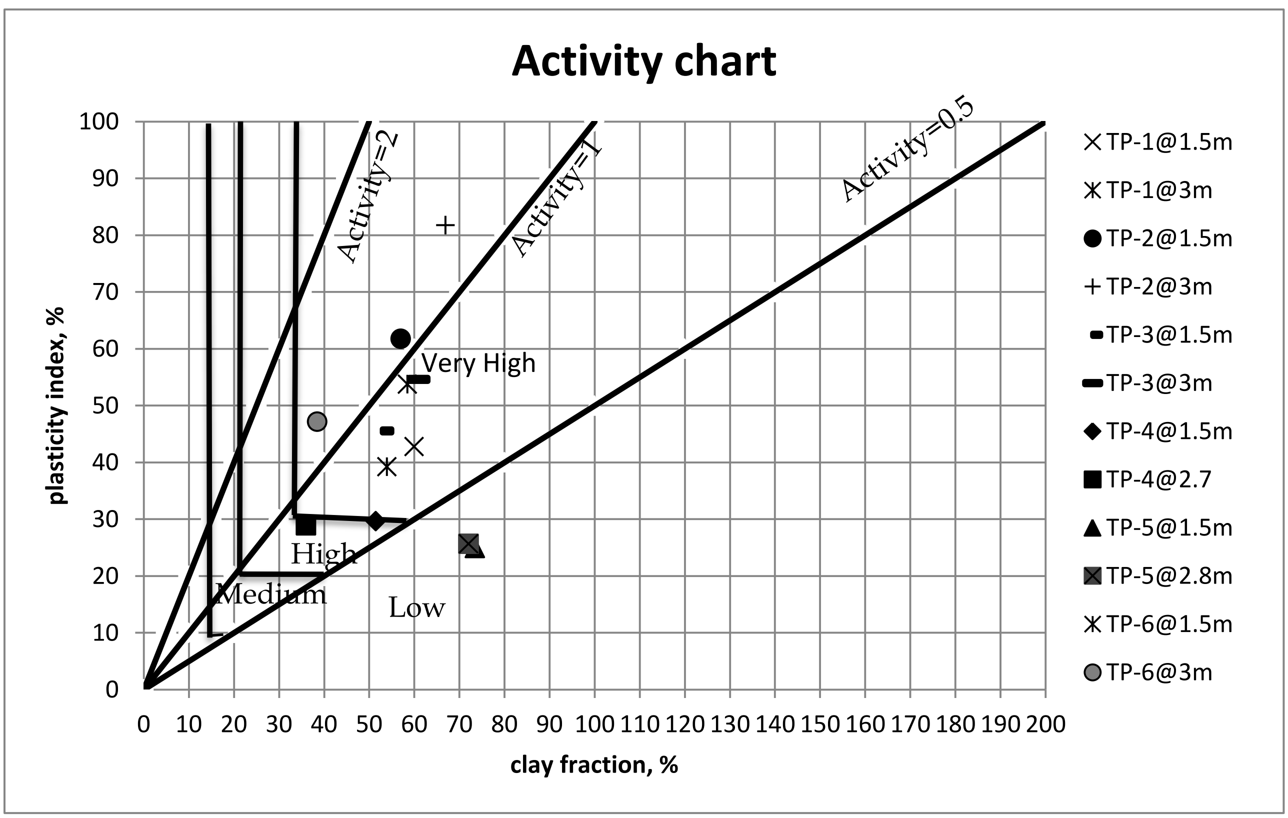

Fig 4.8 Activity chart for soils in the study area

4.4. Shear strength of soils

4.4.1. General

One of the most important and basic engineering properties of soil is its shear strength or ability to resist sliding along internal surfaces within a mass. The stability of a cut, the slope of an earth dam, the foundations of structures, the natural slopes of hillsides and other structures built on soil depend upon the shearing resistance offered by the soil along the probable surfaces of slippage. There is hardly a problem in the field of engineering which does not involve the shear properties of the soil in some manner or the other. The safety of any geotechnical structure is dependent on the strength of the soil. If the soil fails, a structure founded on it can collapse, endangering lives and causing economic damage. Shear strength of a soil is the internal frictional resistance of a soil to shearing forces. Shear strength of a soil is the property that enables a soil to remain in equilibrium when its surface is not level. Shear strength is required to make estimates of the load-bearing capacity of soils and the stability of geotechnical structures, and in analyzing the stress–strain characteristics of soils (Soil Mechanics and Foundation Engineering by Dr. K.R. Arora).

Laboratory shear strength tests can generally be divided into two categories:

1. Shear Strength Tests Based on Total Stress. The purpose of these laboratory tests is to obtain the undrained shear strength (S_u) of the soil or the failure envelope in terms of total stresses (total cohesion, *c*, and total friction angle, Φ). These types of shear strength tests are often referred to as ‘‘undrained’’ shear strength tests.

2. Shear Strength Tests Based on Effective Stress. The purpose of these laboratory tests is to obtain the effective shear strength of the soil based on the failure envelope in terms of effective stress (effective cohesion, *c*′, and effective friction angle, Φ′). These types of shear strength tests are often referred to as ‘‘drained’’ shear strength tests (Soil Mechanics and Foundations by Robert W. Day)

The shear strength of the soil can be defined as (Mohr-Coulomb failure law):

$$\tau_f = c' + \sigma' \tan\Phi' \quad \text{..........} 4.4$$

Where τ_f = shear strength of the soil

c' = effective cohesion

σ_n = effective normal stress on the shear surface

Φ' = effective friction angle

For this study total stress analysis is used. The total stress parameters, such as the undrained shear strength (s_u), determined from an unconfined compression test.

4.4.2. Unconfined compressive strength

The unconfined compression test is a special type of a triaxial compression test in which the all-round pressure is zero. The axial load is increased rapidly until the soil sample fails, that is, it cannot support any additional load. At failure, the total minor principal stress is zero and the total major principal stress is σ_1. The loading is applied quickly so that the pore water cannot drain from the soil; the sample is sheared at constant volume. Because of the absence of any confining pressure in the test, a premature failure through a weak zone may terminate an unconfined compression test. The tests are carried out only on saturated samples which can stand without any lateral support. The test is, therefore, applicable to cohesive soils only. The test is an undrained test and is based on the assumption that there is no moisture loss during the test. The purpose of this test is to determine the undrained shear strength of saturated clays quickly. The unconfined compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils.

Unconfined compressive strength testing was performed by straining the specimen at a constant axial strain rate of between 0.5-2.0%/min. During the test, σ_1 was plotted versus ε_1 to identify qu (Fig. 4.10). For stiff clays, qu is defined as the peak of the $\sigma_1 - \varepsilon_1$ curve. For soft clays, qu is defined as σ_1 at a strain level of 15%. Since as summarized on table 4.11 below the consistency of the soil ranges from medium to hard. Therefore q_u is defined as the peak of the $\sigma_1 - \varepsilon_1$ curve.

For stiffer specimens, a failure plane may be apparent within the specimen, oriented at an angle of approximately 45 degrees (Fig. 4.9). Softer specimens were less likely to exhibit a distinct failure plane, and are more likely to demonstrate “barreling” behavior.

Fig 4.9 – Typical appearance of failed specimens after unconfined compressive strength testing

The undrained shear strength can be determined from the Mohr's circle as: $Su = \frac{Pz}{2A} = \frac{1}{2}\sigma 1 = \frac{qu}{2} = Cu$…………………………4.5 (Appendix-C)

Where, $A = A_o = (1-\varepsilon_1)$ (no volume change i.e. $\varepsilon_p = 0$).

q_u = unconfined compressive strength

The results from unconfined compression tests can be used to:

- Estimate the short-term bearing capacity of fine-grained soils for foundations.
- Estimate the short-term stability of slopes.
- Compare the shear strengths of soils from a site to establish soil strength variability quickly and cost-effectively (the UC test is cheaper to perform than other triaxial tests).
- Determine the stress–strain characteristics under fast (undrained) loading conditions.

As summarized in table 4.10, below unconfined compressive strength of the study area ranges from 75 to 540 kN/m^2 and undrained shear strength ranges from 37.5 to 270 kN/m^2.

The consistency of the soils is also summarized in the table based on the general relationship of consistency and unconfined compressive strength of clays (as stated by Braja M. Das Advanced Soil Mechanics third edition, 2008) and it range from medium to hard.

Table 4.10 unconfined compressive strength, undrained shear strength and consistency of soils of the study area

Test pit	Sample depth(m)	Moisture content, W_c (%)	Unconfined compressive strength, q_u (kN/m^2)	Undrained shear strength, S_u (kN/m^2)	consistency
TP-1	1.5	31.2	210	105	Very stiff
	3	29.4	540	270	Hard
TP-2	1.5	34.0	370	185	Very stiff
	3	66.5	100	50	Stiff
TP-3	1.5	37.6	190	95	Stiff
	3	40.0	220	110	Very stiff
TP-4	1.5	27.2	450	225	Hard
	2.7	35.4	150	75	Stiff
TP-5	1.5	36.5	75	37.5	Medium
	2.8	34.0	85	42.5	Medium
TP-6	1.5	25.5	190	95	Stiff
	3	32.9	140	70	Stiff

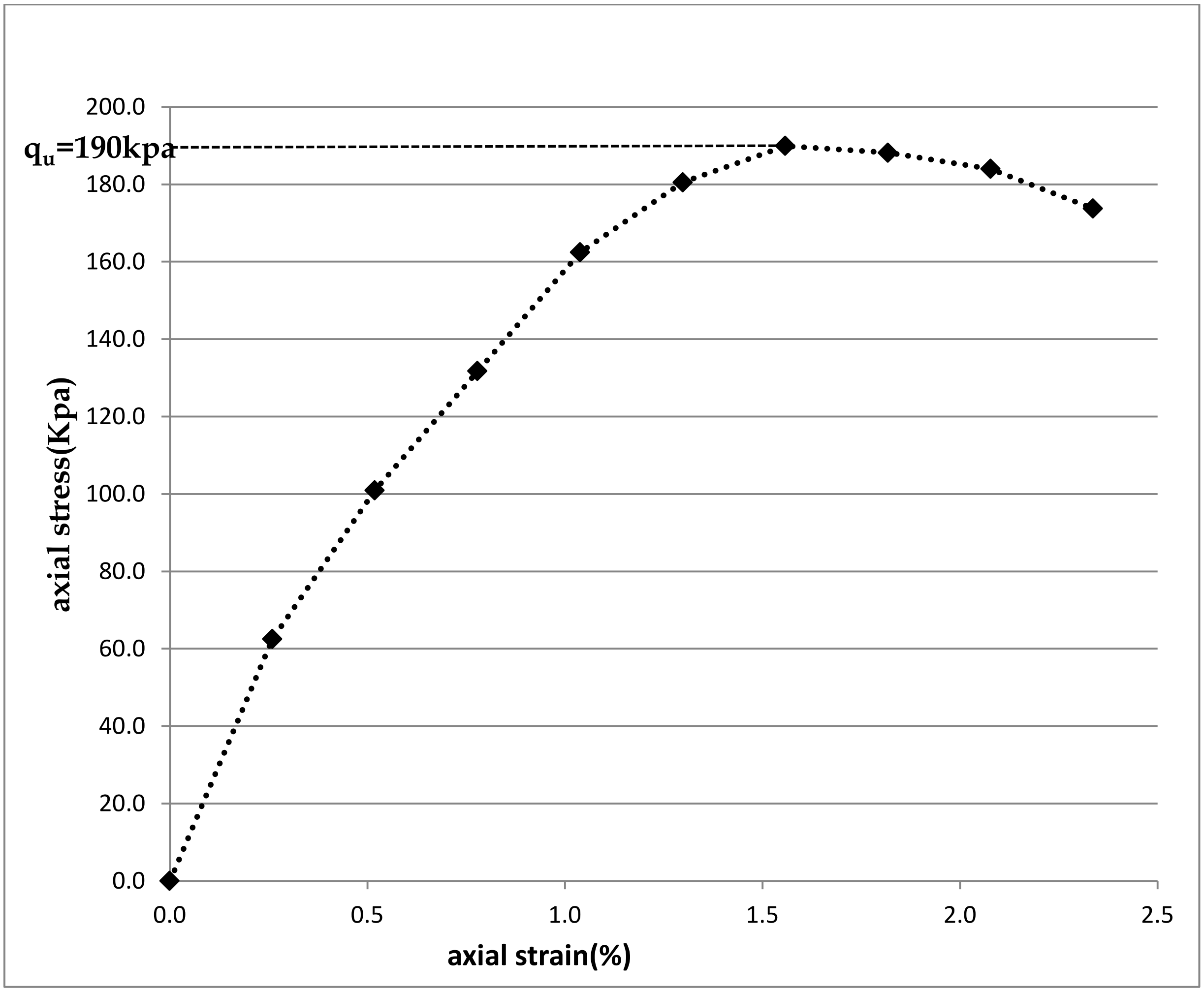

Fig 4.10 Typical $\sigma - \varepsilon$ curve for determining q_u for TP-3 at 1.5m

4.5. Consolidation

4.5.1. General

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

The compression of the soil mass due to the imposed stresses may be almost immediate or time dependent according to the permeability characteristics of the soil. Cohesionless soils which are highly permeable are compressed in a relatively short period of time as compared to cohesive soils which are less permeable. The compressibility characteristics of a soil mass might be due to any or a combination of the following factors:

- Compression of the solid matter.
- Compression of water and air within the voids.
- Escape of water and air from the voids.

It is quite reasonable and rational to assume that the solid matter and the pore water are relatively incompressible under the loads usually encountered in soil masses. The change in volume of a mass under imposed stresses must be due to the escape of water if the soil is saturated. But if the soil is partially saturated, the change in volume of the mass is partly due to the compression and escape of air from the voids and partly due to the dissolution of air in the pore water. The compression of a saturated soil under a steady static pressure is known as *consolidation*. It is entirely due to expulsion of water from the voids (Soil Mechanics and Foundation Engineering by Dr. K.R. Arora).

Consolidation may be due to one or more of the following factors:

- External static loads from structures.
- Self-weight of the soil such as recently placed fills.
- Lowering of the ground water table.
- Desiccation.

A study of the compressibility of soils is necessary to be able to forecast the settlement of structures on different type of soils. If the settlement is not kept to a tolerable limit, the desired use of the structure may be impaired and the design life of the structure may be reduced. Structures may settle uniformly or non-uniformly. The latter condition is called differential settlement and is often the crucial design consideration.

The total compression of saturated clay strata under excess effective pressure may be considered as the sum of

- ✓ Initial or Immediate consolidation,
- ✓ Primary consolidation, and
- ✓ Secondary consolidation.

The portion of the settlement of a structure which occurs more or less simultaneously with the applied loads is referred to as the *initial* or *immediate* settlement. This settlement is due to the immediate compression of the soil layer under undrained condition and is calculated by assuming the soil mass to behave as an elastic soil. For saturated soils, the initial consolidation is mainly due to compression of solid particles.

If the rate of compression of the soil layer is controlled solely by the resistance of the flow of water under the induced hydraulic gradients, the process is referred to as *primary* consolidation. The portion of the settlement that is due to the primary consolidation is called primary consolidation settlement or compression.

The third part of the settlement is due to *secondary* consolidation or compression of the clay layer. This compression is supposed to start after the primary consolidation ceases that is after the excess pore water pressure approaches zero. It is assumed caused due to the plastic readjustment of the solid particles and the adsorbed water to the new stress system (Geotechnical Engineering by V.N.S. Murthy).

The plot of typical void ratio versus logarithm of pressure and pressure for expansive and non-expansive soils is shown in fig 4.11, 4.12, 4.13 & 4.14 respectively. From these figures one can see that the void ratio of expansive soils is higher than expansive soils this is due to the high compressibility of the expansive soils. Soils become highly compressive when there are high void spaces between the soil particles.

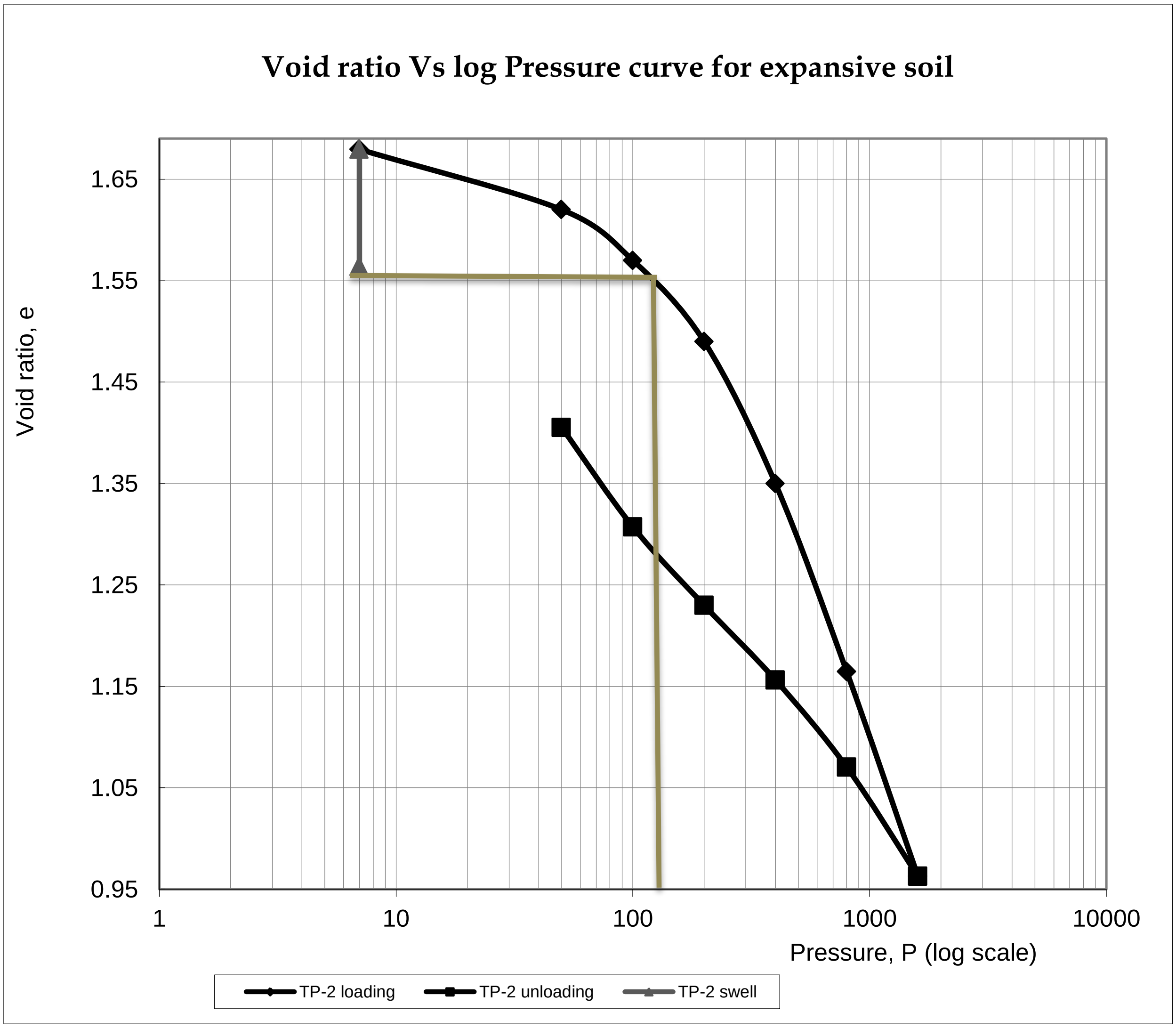

Fig 4.11 Plot of typical vertical effective stress Vs void ratio curve for expansive soils (TP-2) on semi-log scale

Note: the actual applied load intensities to the soil specimen is the weight times the loading arm ratio. In most consolidation loading devices, the arm ratio is equal to ten. Therefore, the weight value is multiplied by ten to obtain the load on the specimen. The vertical effective stress applied to the soil specimen is obtained by dividing the load by the specimen cross-sectional area.

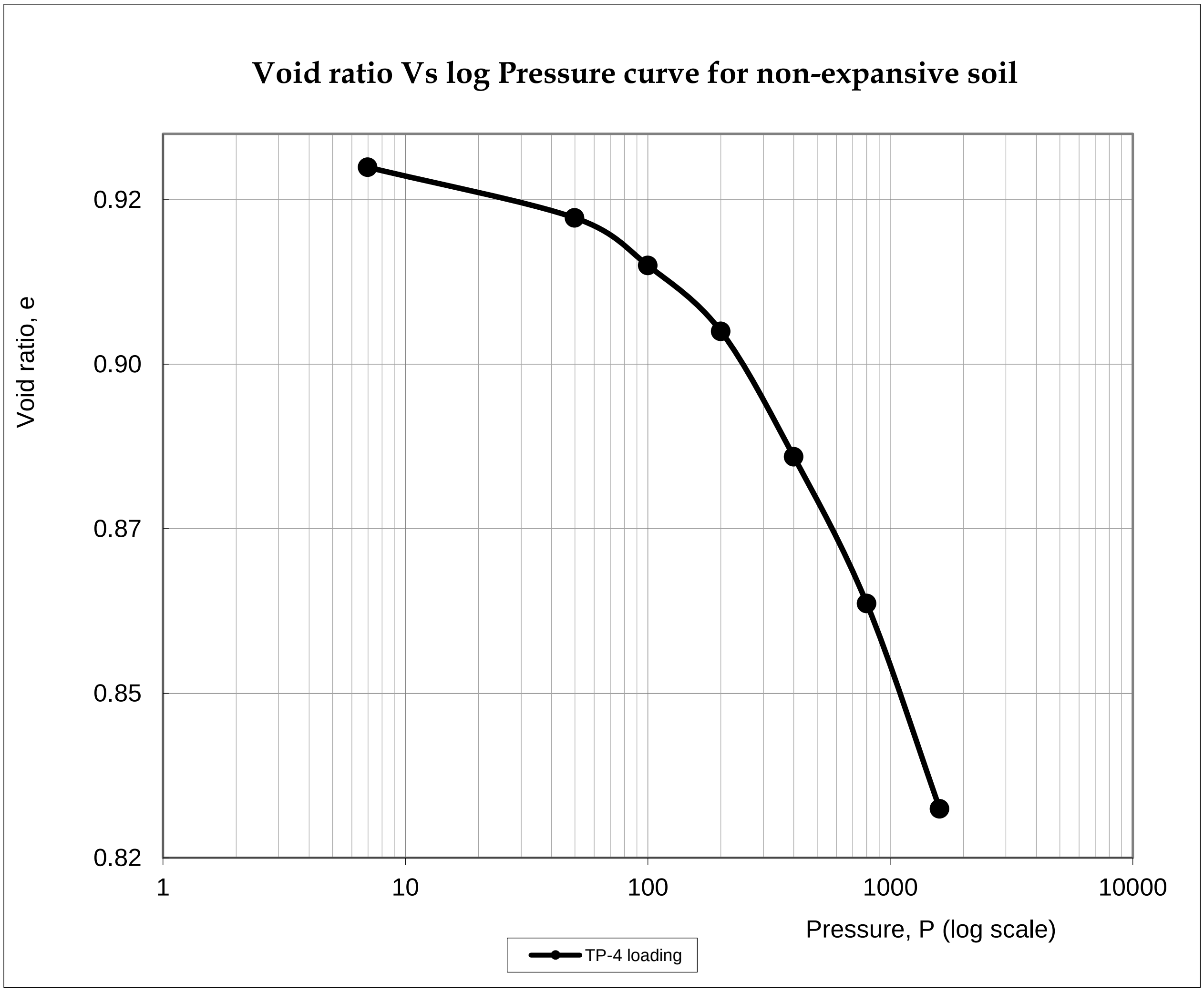

Fig 4.12 Plot of typical vertical effective stress Vs void ratio curve for non-expansive soils (TP-4) on semi-log scale

From figure 4.11, 4.12, 4.13 & 4.14 one can see that the void ratios for expansive soils are higher than non-expansive this implies that expansive soils are highly compressible than expansive soils.

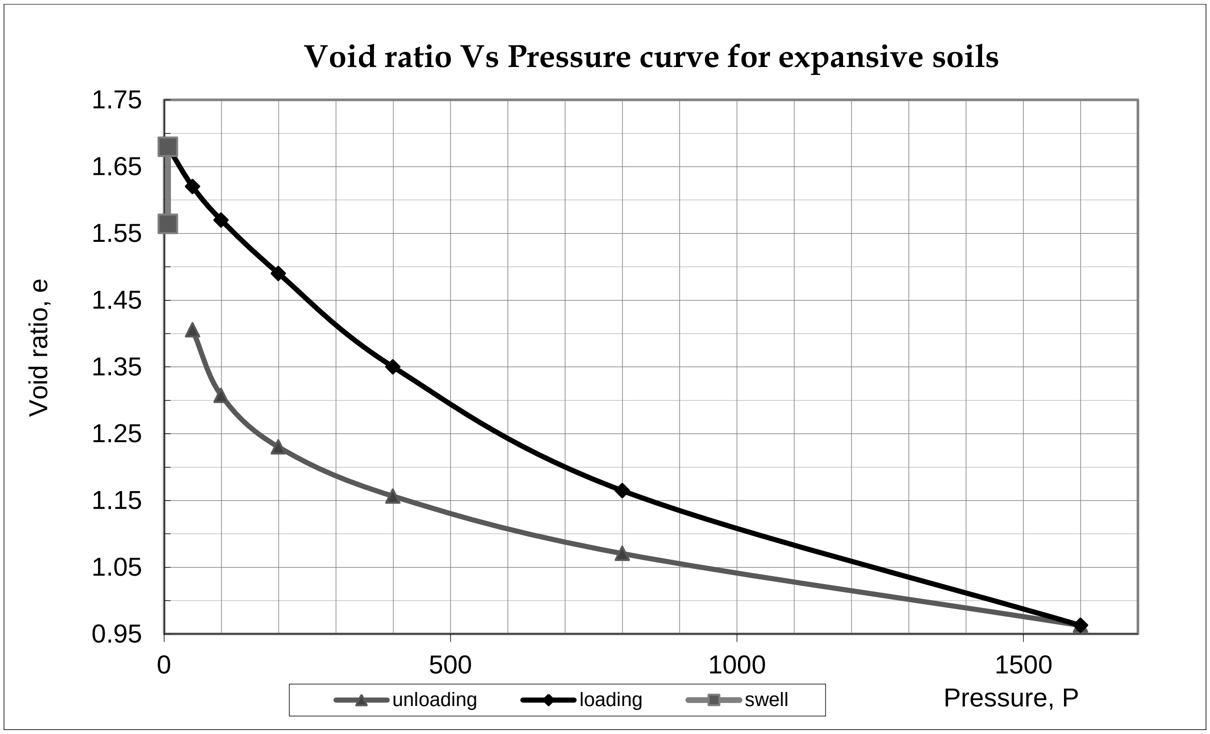

Fig 4.13 Plot of typical vertical effective stress Vs void ratio curve for expansive soils (TP-2) on linear scale

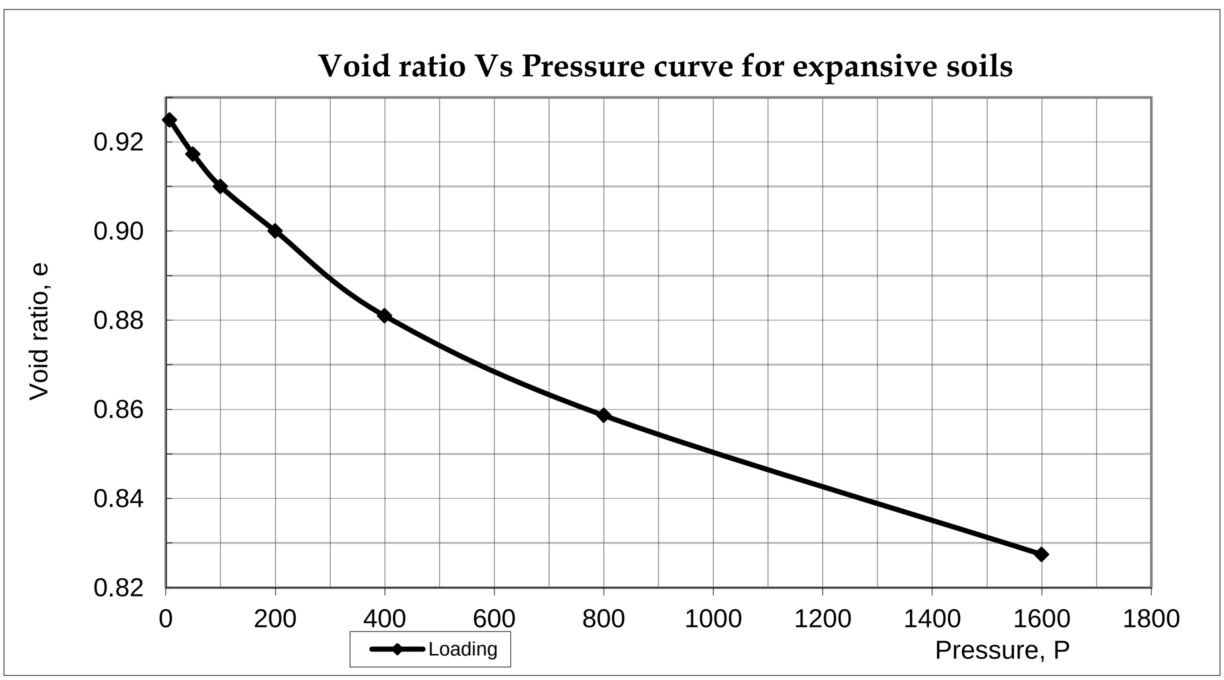

Fig 4.14 Plot of typical vertical effective stress Vs void ratio curve for non-expansive soils (TP-4) on linear scale

4.5.2. Pre-consolidation pressure

A soil in the field at some depth has been subjected to a certain maximum effective past pressure in its geologic history. This maximum effective past pressure may be equal to or less than the existing effective overburden pressure (P_0) at the time of sampling. The reduction of effective pressure in the field may be caused by natural geologic process or human processes. During soil sampling, the existing effective overburden pressure is also released. This results in some expansion. When this specimen is subjected to a consolidation test, small amount of compression will occur when the effective pressure applied is less than the maximum effective overburden pressure in the field to which the soil has been subjected in the past (Geotechnical Engineering by V.N.S. Murthy).

The two basic definitions of clay based on stress history are:

- ✓ *Normally consolidated*; whose present overburden pressure is the maximum pressure that the soil was subjected to it in the past.
- ✓ *Over-consolidated*; whose present effective overburden pressure is less than that which the soil experienced in the past. The maximum effective past pressure is called the *pre-consolidation* pressure.

Several methods have been proposed for determining the value of the maximum consolidation pressure. These are field method and graphical procedure based on consolidation test results.

The field method is based on geological evidence. The geology and physiography of the site may help to locate the original ground level. The overburden pressure in the clay structure with respect to the original ground level may be taken as the pre-consolidation pressure P_c. Usually the geological estimate of the maximum consolidation pressure is very uncertain. In such instances, the only remaining procedure for obtaining an approximate value of *pc* is to make an estimate based on the results of laboratory tests or on some relationships established between *pc* and other soil parameters.

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log *p* curve of an undisturbed sample as shown in Fig. 4.15. From *B,* a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure P_c. Typical curve for determining pre-consolidation pressure using Casagrande method is shown on fig 4.16.

The relative amount of pre-consolidation is usually reported as the over-consolidation ratio (OCR) defined as:

$$OCR = \frac{Pc}{Po} \quad \text{..........} 4.6$$

If OCR =1, the soil is normally consolidated soil. If OCR >1 the soil is over consolidated soil. The over-consolidation ratio of soils has been observed to decrease with depth, eventually reaching a value of 1 (normally consolidated state) (Geotechnical Engineering by V.N.S. Murthy).

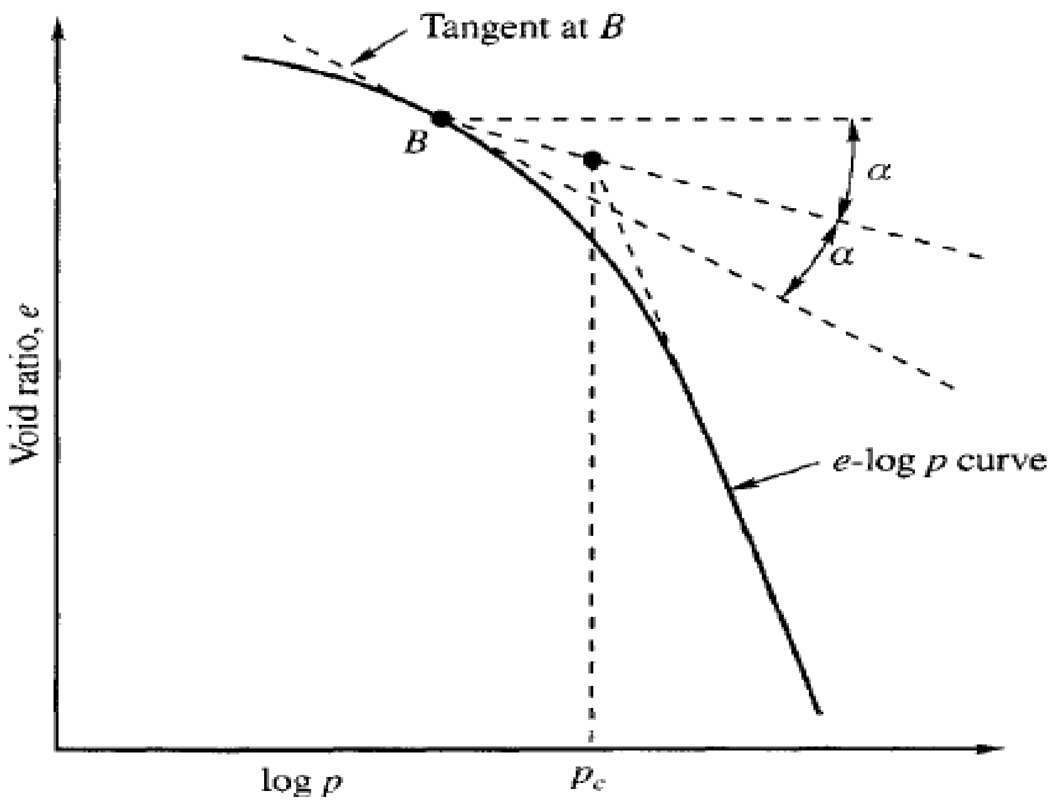

Fig 4.15 method of determining P_C by Casagrande method

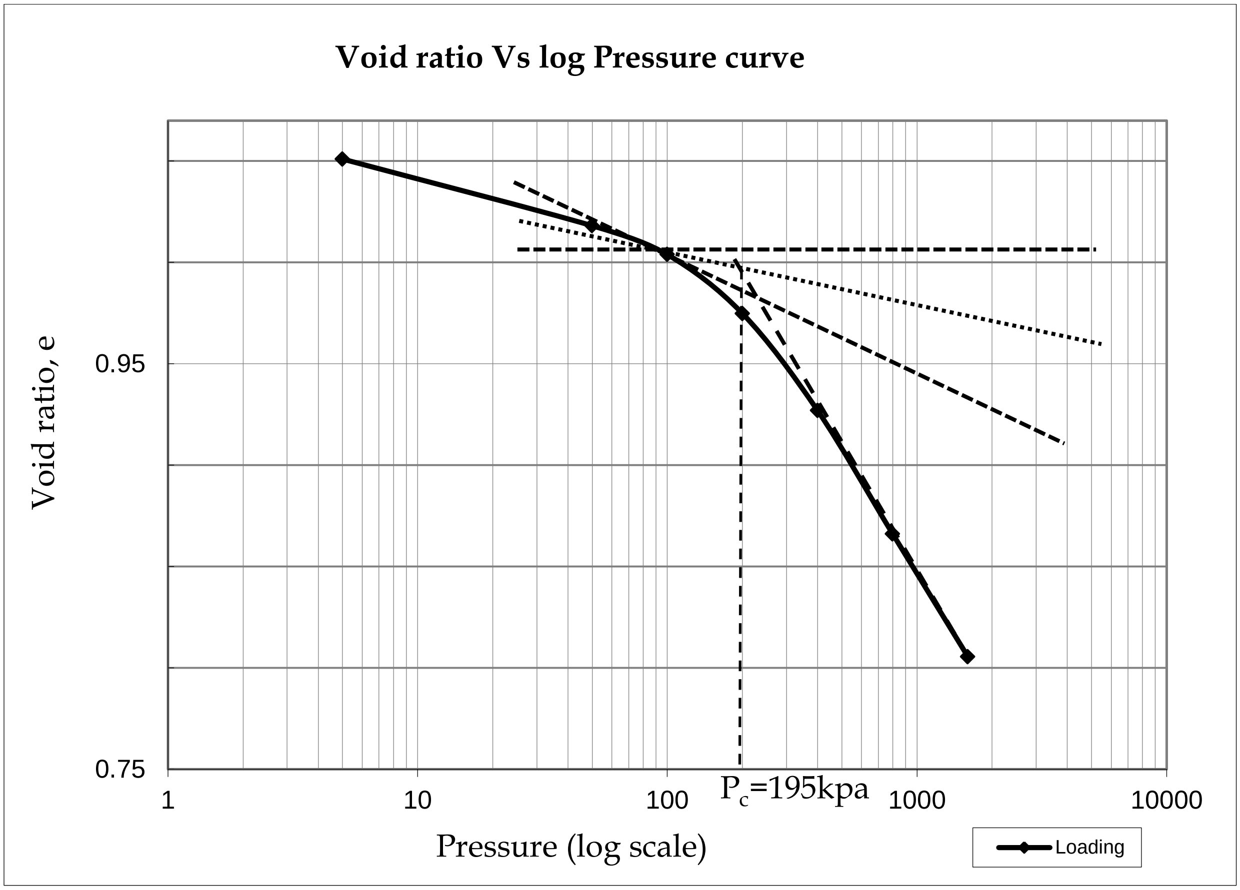

Fig 4.16 typical e-log p curve to determine P_c for TP-3

4.5.3. Compression index (C_C) and swelling index (C_S)

The compression index (Cc) and swelling index (C_S) can be determined by graphic construction from laboratory results for void ratio and pressure, which is equal to the slope of the linear portion of the void ratio versus log pressure loading curve and unloading curve respectively. Thus:

$$Cc = \frac{e_1 - e_2}{log\sigma_2' - log\sigma_1'} \quad \text{.................................} 4.7$$

$$C_S = \frac{e_1 - e_2}{log\sigma_2' - log\sigma_1'} \quad \text{.................................} 4.8$$

There are several empirical expressions for compression index and swelling index determination is also used for approximate calculation in the absence of laboratory consolidation data.

Compression index is extremely useful for determination of field settlement caused by consolidation.

4.5.4. Swelling pressure

Swelling pressure is defined as the pressure which prevents the specimen from swelling or that pressure which is required to return the specimen back to its original state (void ratio, height) after swelling (ASTM D 4546-96).

Basically, the methods of measuring swelling pressure can be either strain controlled or stress controlled.

Strain controlled method is based on the principle of controlling the strain that is developed as water is added. In such a test, modification to the conventional Oedometer is required to allow the control of strain during testing and measurement of the resulting loads.

For stress controlled tests, the conventional Oedometer is used. The samples are placed in the consolidation ring trimmed to the height of the ring. After loading with a standard load of 7kpa, water is added to the sample. When swelling of the sample is ceased, the vertical stress is increased until the sample is compressed back to its original height. The stress required to compress the sample to its original height is the zero volume change swelling pressure or simply swelling pressure.

Swelling pressure is an integral soil property, hence whether determining it through strain or stress controlled, the result is expected to be the same (Chen, 1988).

To investigate swelling pressure of Woliso expansive soils, the method followed is the stress controlled test using undisturbed samples taken from three expansive soil samples.

The swelling pressure is determined graphically from void ratio versus logarithm of pressure curve (fig 4.12). Horizontal line was drawn from the end of swell curve towards the loading curve and then the swelling pressure was read directly by projecting downward on the abscissa.

Swelling pressure values measured are summarized in Table 4.12. The resulting swelling pressure ranges from 84 to 140kpa.

The obtained swelling pressure of Woliso expansive soils is mostly in excess of maximum dead load that can be exerted by most lightly loaded structures, i.e. ≈48 Kpa. Thus the degree of expansion can be categorized as Medium (Chen, 1988).

4.5.5. Coefficient of consolidation

For a given load increment, the coefficient of consolidation C_V can be determined from the laboratory observations of time versus dial reading. There are two laboratory methods that are presently in common use for determination of the coefficient of consolidation. One of them is the *logarithm-of-time method* proposed by Casagrande and Fadum (1940), and the other is the *square-root-of-time method* suggested by Taylor (1942). The general procedures for obtaining C_V by the two methods are described below (Fundamentals of Geotechnical Engineering by Braja M. Das third edition).

i. Logarithm-of-time fitting method

In the logarithm of time fitting method, the displacement gage readings for each load increment are plotted against the times (log scale). The procedure for determining C_V using logarithm of time method is shown in figure 4.17 and a typical curve obtained is shown in Figure 4.18. The theoretical early time settlement response in a plot of logarithm of times versus displacement gage readings is a parabola. The straight line portion of primary and secondary consolidation to intersect at A. the ordinate of A is represented by d_{100} that is, the deformation at the end of 100% primary consolidation. Compression in excess of the above estimated 100% primary consolidation is defined as secondary compression.

The initial curved portion of the plot of deformation versus log t is approximated to be parabola on the natural scale. Times t_1 and t_2 on the curved portion such that $t_2=4t_1$, let the difference of specimen deformation during time (t_2-t_1) be equal to x. A horizontal line DE was drawn such that the vertical distance BD is equal to x. the deformation corresponding to the line DE is d_0 (that is, deformation at 0% consolidation).

The ordinate of point F on the consolidation curve represents the deformation at 50% primary consolidation, and its abscissa represents the corresponding time (t_{50}). The deformation, d_{50}, corresponding to 50% primary consolidation is equal to the average of the deformations corresponding to the 0 and 100% deformations. Then calculation was repeated for different load increments using equation 4.9 and an average value of C_V for the desired load range is determined and the results are summarized using table 4.13 (Fundamentals of Geotechnical Engineering by Braja M. Das third edition).

$$C_V = 0.197 * \frac{{H_{dr}}^2}{t_{50}} \text{..........................} 4.9$$

Where H_{dr} =drainage path

For specimens drained at both top and bottom, H_{dr}, equals one–half the average height of the specimen during consolidation:

$$H_{dr} = \frac{H_{AV}}{2} = \frac{H_0+H_f}{4} = \frac{H_i-d_{50}}{2}$$

Where H_{AV} is the average height and H_0 and H_f are the initial and final heights, respectively, under the current loading.

d_{50}= Compression of sample up to 50% consolidation.

H_i=Initial height of the specimen

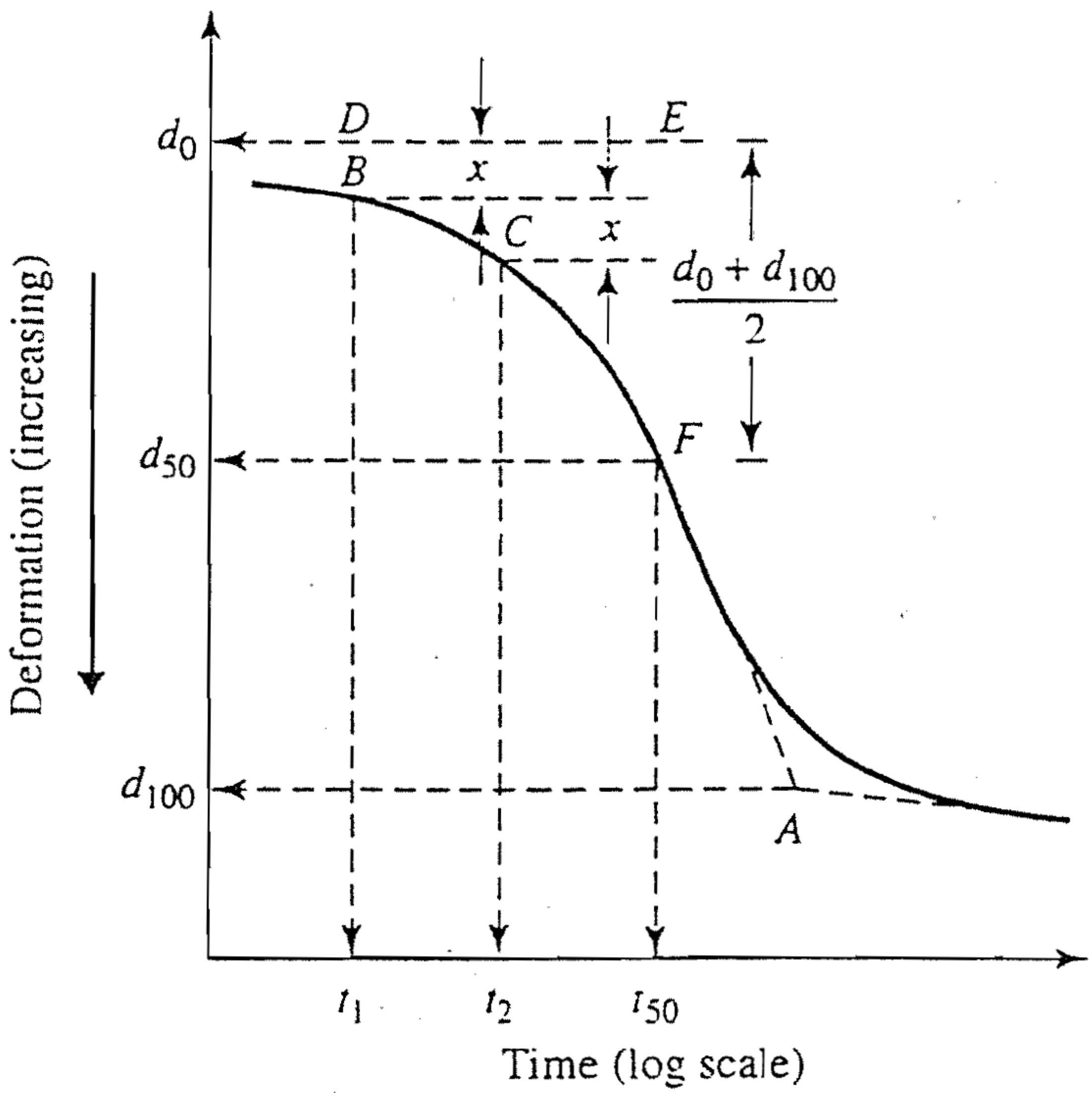

Fig 4.17 Logarithm of time fitting method for determining coefficient of consolidation

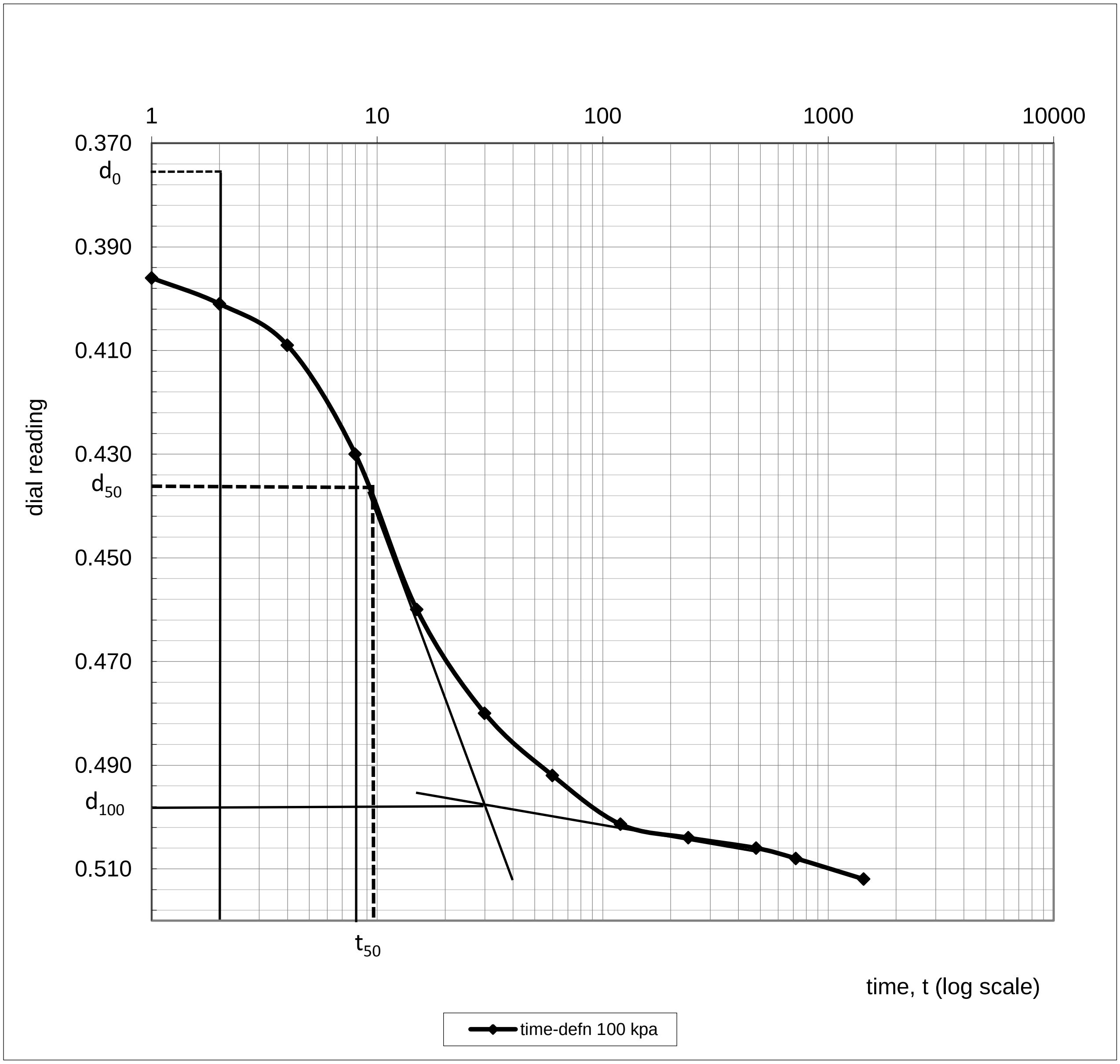

Fig 4.18 Typical curve for determining coefficient of consolidation using

Logarithm of time fitting method

Table 4.11 Summary of consolidation test results

Test pit	Depth (m)	Natural moisture content (%)	Total unit weight(kN/m³)	Pressure P(kpa)	Void ratio	Compression index (Cc)	Swelling index (Cs)	Over burden pressure (kpa)	Pre-consolidation pressure (kpa)	Over consolidation ratio (OCR)	Swelling pressure (kpa)
TP-1	3	29.4	18.9	5	1.08	0.230	_	56.6	145	2.56	84
				50	1.04						
				100	1.01						
				200	0.97						
				400	0.91						
				800	0.86						
				1600	0.79						
TP-2	3	66.5	17.1	5	1.62	0.617	0.299	51.2	130	2.54	140
				50	1.55						
				100	1.49						
				200	1.41						
				400	1.29						
				800	1.15						

				1600	0.96						
TP-3	3	40	17.6	5	1.05	0.209	0.13	52.9	195	3.68	94
				50	1.02						
				100	1.00						
				200	0.97						
				400	0.93						
				800	0.87						
				1600	0.81						
TP-4	2.7	35.4	17.9	5	0.92	0.112	–	48.4	250	5.16	–
				50	0.91						
				100	0.90						
				200	0.89						
				400	0.88						
				800	0.85						
				1600	0.82						

ii. Square root of time fitting method

In this method, the dial readings for each load increment are plotted against the square root of time as shown in the procedure using figure 4.19 and the typical curve obtained is shown in figure 4.20.

First straight line which best fits the early portion of the laboratory curve is drawn. Next a straight line is drawn which at all points has abscissa 1.15 times as great as those of the first line. The intersection of this line and the laboratory curve was taken as the 90 percent *(R_{90})* consolidation point. Its value may be read and is designated as t_{90}.

Usually the straight line through the early portion of the laboratory curve intersects the zero time line at a point *(do)* differing somewhat from the initial point (d_i). This intersection point is called the *corrected zero point.* If one-ninth of the vertical distance between the corrected zero point and the 90 per cent point was set off below the 90 percent point, the point obtained is called the "*100 percent primary compression point*" *(d_{100}).* The compression between zero and 100 per cent point is called "*primary compression*". At the point of 90 percent consolidation, the value of T = 0.848. The equation of C_V may now be written as (Fundamentals of Geotechnical Engineering by Braja M. Das third edition):

$$C_V = 0.848\,\frac{H^2 dr}{t_{90}} \qquad \text{4.10}$$

Usually, the square root of time fitting method yields higher values of coefficient of consolidation than the logarithm of time fitting method. The same as the logarithm of time method an average value of C_V for the desired load range is determined and the results from each case are summarized in table 4.13 below.

4.5.6. Coefficient of permeability (Hydraulic conductivity)

The hydraulic conductivity of soils depends on several factors: fluid viscosity, pore size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of soil saturation. In clayey soils, structure plays an important role in hydraulic conductivity. Other major factors that affect the hydraulic conductivity of clays are the ionic concentration and the thickness of layers of water held to the clay particles.

The coefficients of permeability of soils vary according to their type, textural composition, structure, void ratio and other factors. Therefore, no single value can be assigned to a soil purely on the basis of soil type.

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Sometimes it can be estimated from one dimensional consolidation test. Having established coefficient of consolidation (C_V), coefficient of permeability can be determined from the following relationship (Investigation on some of the engineering properties of soils in Adama town, Ethiopia by Dagnachew Debebe):

$$K = C_V * \gamma_w * \frac{a_v}{1+e_0} \qquad \text{.................................4.11}$$

Where: C_V=coefficient of consolidation

a_V= coefficient of compressibility= $\frac{\Delta e}{\Delta \sigma_v}$

e_0= initial void ratio

γ_w=unit weight of water =9.81kN/m^3

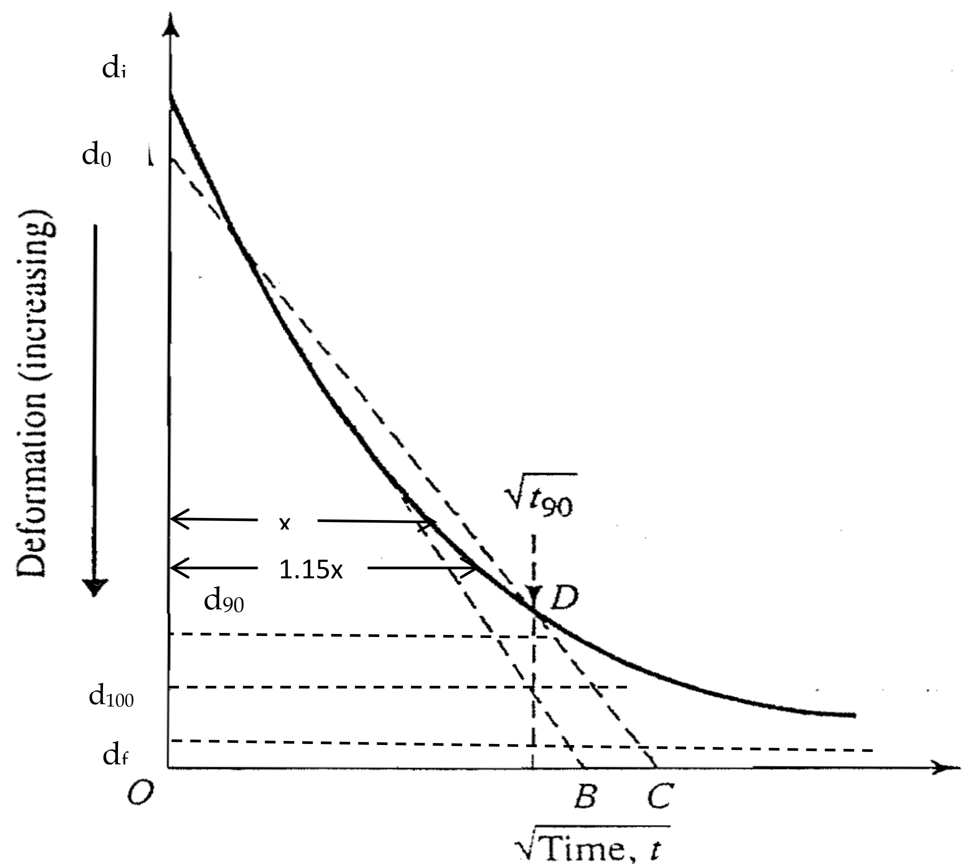

Fig 4.19 Square root of time fitting method for determination of coefficient of consolidation

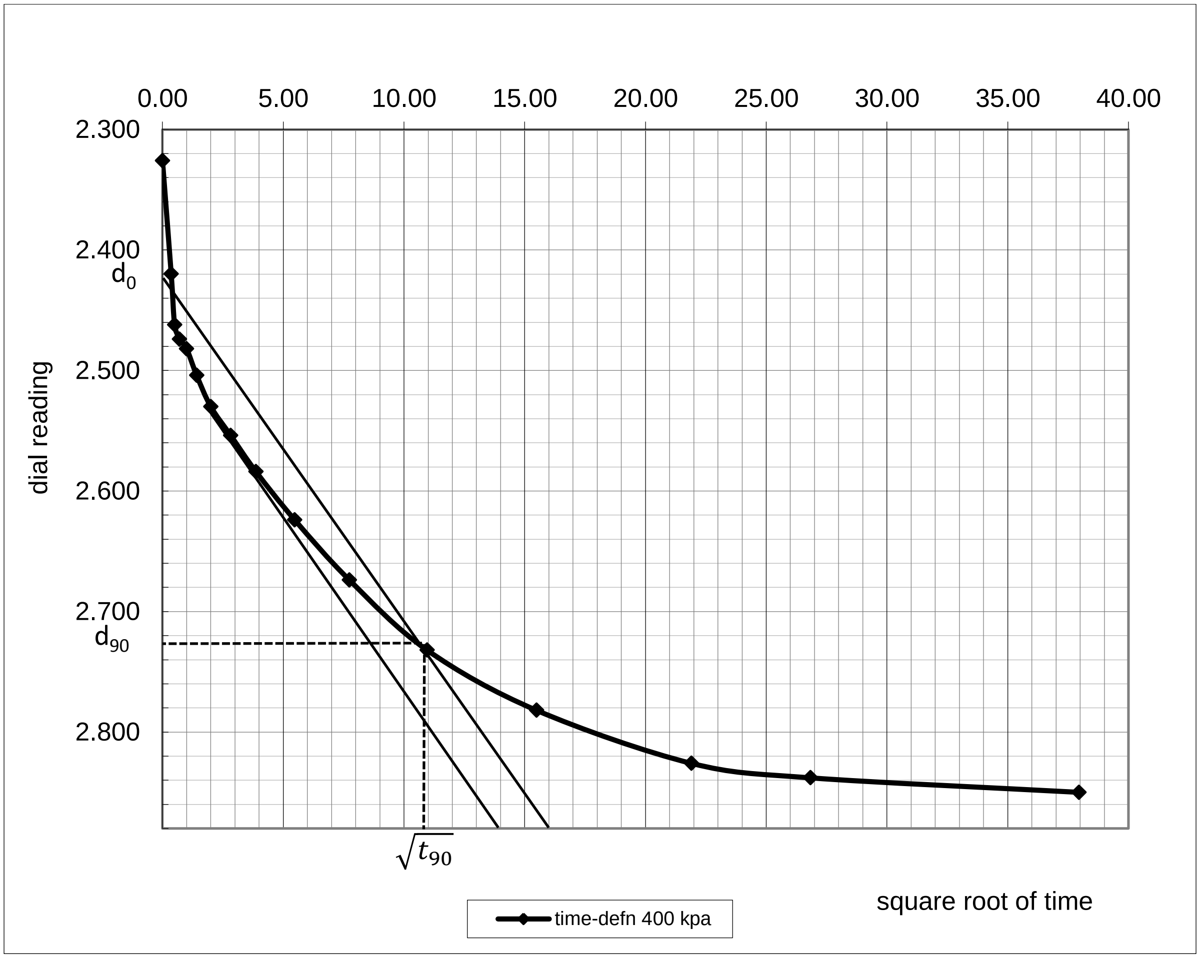

Fig 4.20 Typical curve for determination of coefficient of consolidation

Using Square root of time fitting method

Using equation 4.11 above, the coefficient of permeability as a function of coefficient of consolidation, coefficient of compressibility and void ratio was calculated for each load increment and shown in Table 4.13 below. According to Terzaghi and Peck (1967), the classification of soil according to hydraulic conductivity k is as follows:

High degree of permeability, k is over 0.1 cm/s, Medium degree of permeability, k is between 0.1 and 0.001 cm/s, Low permeability, k is between 0.001 and 1×10^{-5} cm/s, Very low permeability, k is between 1×10^{-5} and 1×10^{-7} cm/s, Practically impermeable, k is less than 1×10^{-7} cm/s.

The coefficient of consolidation used for this calculation is the average of logarithm of time and square root of time method. The range of values of coefficient of permeability lies between 10^{-9} and10^{-10} cm/s, which indicates that the soils are practically impervious or have low permeability.

Table 4.12 Summary of coefficient of consolidation, coefficient of compressibility and coefficient of permeability

Test pit	Sample depth(m)	Pressure P (kpa)	Void ratio (e)	Coefficient of consolidation (C_v) log t method 10^{-3}(cm^2/min)	Coefficient of consolidation (C_v) $\sqrt{t}$ method 10^{-3}(cm^2/min)	Coefficient of compressibility (a_v) 10^{-4} (m^2/kN)	Coefficient of permeability (k) 10^{-8}(cm/s)
TP-1	3	5	1.09	-	-	-	-
		50	1.05	8.84	17.17	8	8.39
		100	1.02	2.97	14.46	3	2.11
		200	0.97	4.12	16.1	2.5	2.04
		400	0.92	3.71	6.43	1.25	0.70
		800	0.87	3.11	5.21	1.25	0.42
		1600	0.80	4.36	7.17	0.31	0.15
TP-2	3	5	1.62	-	-	-	-
		50	1.55	3.63	16.02	14	9.4
		100	1.49	6.4	15.55	12	10.63
		200	1.41	2.23	2.5	8	1.53
		400	1.29	2.56	2.6	6	1.25
		800	1.16	1.25	2.34	3.25	0.47

		1600	0.96	6.03	4.72	2.50	1.08
TP-3	3	5	1.05	-	-	-	-
		50	1.02	3.75	4.65	2.4	0.78
		100	1.00	6.66	7.18	2.8	1.56
		200	0.97	6.5	7.76	2.9	1.67
		400	0.93	3.11	8.33	2.4	1.11
		800	0.87	1.91	1.75	1.5	0.22
		1600	0.81	3.61	2.04	0.71	0.16
TP-4	2.7	5	0.925	-	-	-	-
		50	0.917	5.02	3.91	2	0.72
		100	0.91	15.7	12.67	2	2.29
		200	0.90	7.76	8.88	1	0.67
		400	0.88	7.35	5.07	1	0.5
		800	0.86	7.2	8.07	0.5	0.31
		1600	0.83	11.65	12.5	0.375	0.37

5. Discussions on laboratory test results

From the laboratory tests expansive and non-expansive soils were obtained. The natural moisture content of the samples ranges from 25.5 – 66.5%. The water content is higher below a depth of three meters in some parts of the town around areas where sample TP-2 and TP-3 is taken. This indicates that the underground water level in some parts of the town is around three meters. The specific gravity of expansive soils ranges from 2.65 – 2.75 and for non-expansive soils ranges from 2.73 – 2.76. The bulk density and unit weight of soils in the study area ranges from 1.7 – 1.88 g/cm^3 and 17 – 18.9 kN/m^3 respectively. The grain size analysis result indicate that the dominant proportion of soil particle in the research area is clay, which has clay content ranging from 42.5 – 80%, silt fraction 16 – 59%, sand fraction 1 – 9.7% and gravel content from 0.0 – 11.4%.

The Atterberg limit of the soil samples on air dried and oven dried samples have high values. The liquid limit of expansive soil in the research area ranges from 70.5 – 126%, plastic limit ranges from 31.2 – 44.1% and plasticity index ranges from 39.3 – 81.7% and for non-expansive soils liquid limit ranges from 56.2 – 61.1%, plastic limit ranges from 30.3 – 34.4% and plasticity index ranges from 26.2 – 29.7%. High Atterberg limit values of expansive soils show high swelling potential of the soils. It has been found that both liquid limit and plastic limits depend upon the type and amount of clay in the soil. However, the plasticity index depends mainly on the amount of clay. Soils with plasticity index of more than 35% have very high swelling potential (Seed, Woodward, and Lundgren (1 – 42)). One can see that the expansive soils of the study area have high swelling potential.

Free swell test results for oven dried samples at a temperature of 105 ^{O}C shows that the free swell of the soil under investigation ranges from 34.5% to 250%. Those soils having a free swell less than 50% are considered as low in degree of expansion, free swell between 50%-100% are considered as marginal and soils having free swell greater than 100% are considered as expansive. Therefore most of the soil samples under investigation are expansive. The black soil samples taken show high expansive behavior with free swell ranging from 80 – 250% but the red clay soils are non-expansive with free swell ranging from 34.5 – 47.5% this may be due to kaolinite minerals that are very stable with strong structure and absorb little water.

The swelling pressure of some of expansive soils of the study area ranges from 84 to 140kpa. The obtained swelling pressure of Woliso expansive soils is in excess of maximum dead load that can be exerted by most lightly loaded structures. Thus the degree of expansion can be categorized as Medium (Chen, 1988). Comparing the result with the free swell test, the lower swelling pressure is due to higher initial moisture content of the specimen prior to testing. The initial moisture content of the expansive soils controls the amount of swelling. Clays with lower initial moisture content will have higher swelling potential.

Plasticity chart of the study area according to Unified Soil Classification System shows that the black soil under investigation lies above the A-line which is inorganic clay with high plasticity and the red soil falls below the A-line which is inorganic silts of high plasticity. According grain size analysis result red colored soils have high clay fraction thus it is better to classify the soil as red clay rather than silt. The distinction between the inorganic and organic soils is made by visual appearance and odor by USC method and ASTM method. In the ASTM method more specifically by comparison of the oven-dried liquid limit (LL) and air-dried liquid limit (LL′). If oven dried value LL<0.75LL′ and the appearance and odor indicates "organic" then the soil is classified as organic; otherwise, inorganic.

In this study the distinction was done by using both methods and the result indicates that the soils are classified as inorganic clays and silts of high plasticity based on unified soil classification method and ASTM method.

Classifications of soils in the study area based on AASHTO Classification system is classified in group A-7-5, which are clayey soils with group index ranging from 31 to 98. The group index results indicate that generally the soils of the study area are very poor for highway subgrade material. The smaller the value of the group index, the better is the soil in that category for highway subgrade material. A group index of zero indicates a good subgrade, whereas group index of 20 or greater shows a very poor highway subgrade material.

Shear strength characteristics of the soil was determined by using unconfined compressive strength test. During the test, σ_1 was plotted versus ε_1 to identify qu. For stiff clays, qu is defined as the peak of the $\sigma_1 - \varepsilon_1$ curve. For soft clays, qu is defined as σ_1 at a strain level of 15%. Since the consistency of the soil under investigation ranges from medium to hard q_u is defined as the peak of the $\sigma_1 - \varepsilon_1$ curve.

Unconfined compressive strength and undrained shear strength of expansive soils of the study area ranges from 100 to 540 kN/m^{2}and 50 to 270 kN/m^2 respectively and for non-expansive soils it ranges from 75 – 450kN/m^2 and 37.5 – 225kN/m^2.

From the plot of void ratio versus pressure for expansive and non-expansive samples one can see that the void ratio of expansive soils is higher than the non-expansive samples this is due to the high compressibility of expansive soils. Soils become highly compressive when there are high void spaces between the soil particles. The soil under investigation has a pre-consolidation pressure of 130 - 250kPa. Over-consolidation ratios of the soils are more than one, so the soil in the study area is over consolidated in its natural state.

The compression index of the soils is calculated from the straight line portions of the e-logp curve. The compression index, Cc, ranges from 0.112-0.617. In the absence of laboratory consolidation data, C_C is often used for an approximate calculation of primary consolidation in the field.

The coefficient of consolidation (Cv) of the samples were determined using logarithm of time fitting method and square root of time fitting method. Then C_V using logarithm of time and square root method is determined for each incremental load and an average value of C_V for the desired load range was determined. The coefficient of permeability of soil under investigation which is calculated from the test results of consolidation shows that range of values of coefficient of permeability lies between 10^{-8} and 10^{-9} cm/s, which indicates that the soils are practically impervious or have low permeability.

6. Conclusions and Recommendation

6.1. Conclusions

From this study black and red clay soils were investigated in Woliso town which are expansive and non-expansive respectively. The specific gravity of soils in the study area ranges from 2.65 to 2.76.

Grain size analysis tests revealed that, starting from few centimeters below the ground level to the depth of investigation which is three meters, the soil in Woliso town is black and red clay soils.

Most of the soil samples in town have free swell value of greater than 50%. This shows the soil in the study area ranges from non-expansive to highly expansive with free swell value ranging from 34 - 250%. The black clay soil samples (TP-1, TP-2, TP-3 & TP-6) show high expansive behavior with free swell value ranging from 110 – 250% but the red clay samples (T-4 & TP-5) are non-expansive with a free swell ranging from 34.5 – 47.5%. And the swelling pressure shows that the swelling potential of the soils is medium which ranges from 84 to 140kpa.

The soils in the study area have high Atterberg limit values which show the soils are highly plastic. From Atterberg limit test results the liquid limit of black clay soils ranges from 70.5 - 126%, plastic limit ranges from 31.2 - 44% and plasticity index ranges from 39.3 - 82%. High values of consistency limits indicate the presence of high clay content. For red clay soils the liquid limit ranges from 56.2 – 61.1%, plastic limit from 30.3 – 34.4% and plasticity index ranges from 26.2 – 29.7%.

The unconfined compressive strength of the soils in the study area range from 75 to 540kN/m^2 and undrained shear strength value range from 37.5 to 270kN/m^2. Based on unconfined compressive strength the approximate consistency of the soil ranges from medium to hard.

As determined from the one-dimensional consolidation test conducted on undisturbed soil samples, compression index, Cc, ranges from 0.11-0.62, coefficient of consolidation, C_V, from 1.25-17.17 (x10^{-3})cm^2/min, coefficient of permeability, k, from 0.15-10.63 (x10^{-8} cm/sec). This indicates that since the soil in the study area is clayey it has low coefficient of permeability or the soil is impervious.

6.2. Recommendation.

1. In this research samples of soil were collected only from six test pits, by increasing the number of sampling area and depth of sampling further detailed investigation has to be carried out on disturbed and undisturbed soil samples.

2. Some of the basic engineering properties of the soil in this study are obtained from laboratory tests; detailed in situ investigation has to be carried out because soil properties at in situ stresses are critical.

3. Establishing the interaction between swelling potential of the expansive soils, climate and moisture content monitoring have to be an area for further research.

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Appendix-A

(Particle Size Analysis procedures and sample calculation)

A-1. Test Procedures

The procedures listed in ASTM standard with designation D1140-97 was followed to run this test. The distribution of particle sizes larger than 75µm (retained on the No. 200 sieve) was determined by sieving, while the distribution of particle sizes smaller than 75µm was determined by a sedimentation process, using a hydrometer to secure the necessary data.

Soil finer than the 75µm (No.200) sieve can be separated from larger particles much more efficiently and completely by wet sieving than the use of dry sieving. For this study wet sieve analysis was used for all of the samples collected from the site for accurate determination of particles finer than 75µm in soil prior to dry sieving.

Prior to wet sieving, some of the samples were air dried and the rest were oven dried. After drying, a sample of 1kg was weighed and then soaked for 24 hours, in order to avoid the finer material from adhering to the larger particles. Then the sample of the soil was washed over 75µm (No.200) sieve. Dry sieve analysis was done on samples of soil retained on sieve No. 200, after oven drying it for 24 hours by using a series of sieves. This set consists of the following sieves:

3⁄8-in. (9.5-mm)
No. 4 (4.75-mm)
No. 10 (2.00-mm)
No. 16 (1.18-mm)
No. 20 (850-µm)
No. 30 (600-µm)
No. 40 (425-µm)
No. 50 (300-µm)
No. 60 (250-µm)
No. 100 (150-µm)
No. 200 (75-µm)

For Hydrometer analysis a sample of 50grams was soaked for 24 hours by adding dispersing agent. The test was performed as per the standard for obtaining an estimate of the distribution of soil particle sizes from the No. 200 (0.075 mm) sieve to around 0.01 mm.

The actual hydrometer reading and test temperature was taken for 0.5, 1, 2, 5, 8, 15, 30, 60, 120, 240, 480, 1440 minutes.

Table –A-1 Combined wet sieve and dry sieve analysis result for TP-1 at 1.5m

Sieve opening (mm)	Mass of sieve (g)	Mass of sieve with retained soil (g)	Mass of retained soil (g)	Percentage retained, %	Cumulative percentage retained, %	cumulative Percentage passing, %
9.5	456.2	456.2	0	0.00	0.00	100
4.75	430.6	431.6	1	0.10	0.10	99.9
2	557.8	567.2	9.4	0.94	1.04	98.96
1.18	372.1	382.9	10.8	1.08	2.12	97.88

0.85	344.7	350.6	5.9	0.59	2.71	97.29
0.6	324.4	330.3	6.1	0.61	3.32	96.68
0.425	305.6	310.1	5.3	0.53	3.85	96.15
0.3	302.5	305.4	7.98	0.80	4.65	95.352
0.25	454.1	455.2	5.6	0.56	5.21	94.792
0.15	277.7	281.9	9.78	0.98	6.19	93.814
0.075	272.2	280.2	9.87	0.99	7.17	92.827
Pan	256.2	1202.5	928.3	92.83	100.00	
Total			1000.00	100.00		

Table A-2 Hydrometer analysis result for TP-1 at 1.5m

		at 1.5m			Test Temperature,		19^0 C	
Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer	Percent Finer for Combined analysis
0.5	1.0320	-0.0029	1.0291	7.8354	0.01382	0.0547	93.47	90.67
1	1.0315	-0.0029	1.0286	7.9676	0.01382	0.0390	91.87	89.11
2	1.0310	-0.0029	1.0281	8.0999	0.01382	0.0278	90.26	87.55
5	1.0307	-0.0029	1.0278	8.1792	0.01382	0.0177	89.30	86.62
8	1.0300	-0.0029	1.0271	8.3644	0.01382	0.0141	87.05	84.44
15	1.0285	-0.0029	1.0256	8.7612	0.01382	0.0106	82.23	79.76
30	1.0278	-0.0029	1.0249	8.9463	0.01382	0.0075	79.98	77.58
60	1.0268	-0.0029	1.0239	9.2109	0.01382	0.0054	76.77	74.47
120	1.0258	-0.0029	1.0229	9.4754	0.01382	0.0039	73.56	71.35
240	1.0240	-0.0029	1.0211	9.9515	0.01382	0.0028	67.78	65.74
480	1.0234	-0.0029	1.0205	10.1102	0.01382	0.0020	65.85	63.87
1440	1.0210	-0.0029	1.0181	10.7451	0.01382	0.0012	58.14	56.40

Table –A-3 combined wet sieve and dry sieve analysis result for TP-1 at 3m

Sieve opening (mm)	Mass of sieve (g)	Mass of sieve with retained soil (g)	Mass of retained soil (g)	Percentage retained, %	Cumulative percentage retained, %	cumulative Percentage passing, %
9.5	463.2	473.6	10.4	1.04	1.04	98.96
4.75	430.6	455.2	24.6	2.46	3.5	96.5
2	557.8	586.2	28.4	2.84	6.34	93.66
1.18	372.1	387	14.9	1.49	7.83	92.17
0.85	344.7	352.4	7.7	0.77	8.6	91.4
0.6	324.4	332.9	8.5	0.85	9.45	90.55
0.425	305.6	313.5	7.9	0.79	10.24	89.76
0.3	302.5	308.7	6.2	0.62	10.86	89.14
0.25	454.1	456.8	2.7	0.27	11.13	88.87
0.15	277.7	286	8.3	0.83	11.96	88.04
0.075	272.2	284.1	11.9	1.19	13.15	86.85
Pan	256.2	1124.7	868.5	86.85	100	0
Total			1000			

Table A-4 Hydrometer analysis result for TP-1 at 3m

At 3m					Test Temperature,		19^0 C	
Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer	Percent Finer for Combined analysis
0.5	1.0310	-0.0029	1.0281	8.10	0.01378	0.0555	90.06	85.55
1	1.0305	-0.0029	1.0276	8.23	0.01378	0.0395	88.45	84.03
2	1.0298	-0.0029	1.0269	8.42	0.01378	0.0283	86.21	81.90
5	1.0290	-0.0029	1.0261	8.63	0.01378	0.0181	83.65	79.46
8	1.0288	-0.0029	1.0259	8.68	0.01378	0.0144	83.00	78.85
15	1.0280	-0.0029	1.0251	8.89	0.01378	0.0106	80.44	76.42
30	1.0270	-0.0029	1.0241	9.16	0.01378	0.0076	77.24	73.37
60	1.0260	-0.0029	1.0231	9.42	0.01378	0.0055	74.03	70.33
120	1.0250	-0.0029	1.0221	9.69	0.01378	0.0039	70.83	67.29
240	1.0240	-0.0029	1.0211	9.95	0.01378	0.0028	67.62	64.24
480	1.0232	-0.0029	1.0203	10.17	0.01378	0.0020	64.93	61.68
1440	1.0198	-0.0029	1.0169	11.06	0.01378	0.0012	54.16	51.45

Table –A-5 combined wet sieve and dry sieve analysis result for TP-2 at 1.5m

Sieve opening (mm)	Mass of sieve (g)	Mass of sieve with retained soil (g)	Mass of retained soil (g)	Percentage retained, %	Cumulative percentage retained, %	cumulative Percentage passing, %
9.5	463.3	465.1	1.8	0.18	0.18	99.82
4.75	428.4	441.2	12.8	1.28	1.46	98.54
2	400.9	422.7	21.8	2.18	3.64	96.36
1.18	372.3	378.9	9.94	0.99	4.634	95.366
0.85	344.9	347.6	8.7	0.87	5.504	94.496
0.6	324.5	327.1	9.85	0.99	6.489	93.511
0.425	292.4	294.9	9.61	0.96	7.45	92.55
0.3	302.6	305.1	7.8	0.78	8.23	91.77
0.25	454.3	455.4	6.6	0.66	8.89	91.11
0.15	277.8	282.8	9.7	0.97	9.86	90.14
0.075	272.4	282	12.87	1.29	11.147	88.853
Pan	256.6	1187.6	888.5	88.85	99.997	0.003
Total			1000.00	100.00		

Table A-6 Hydrometer analysis result for TP-2 at 1.5m

		at 1.5m			Test Temperature,	18^0 C		
Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Percentage Finer	Percent Finer for Combined analysis
0.5	1.0300	-0.0027	1.0273	8.3644	0.01325	0.0542	87.49	86.62
1	1.0298	-0.0027	1.0271	8.4173	0.01325	0.0384	86.85	85.98
2	1.0295	-0.0027	1.0268	8.4967	0.01325	0.0273	85.89	85.03
5	1.0292	-0.0027	1.0265	8.5760	0.01325	0.0174	84.93	84.08
8	1.0286	-0.0027	1.0259	8.7347	0.01325	0.0138	83.00	82.17
15	1.0280	-0.0027	1.0253	8.8934	0.01325	0.0102	81.08	80.27
30	1.0275	-0.0027	1.0248	9.0257	0.01325	0.0073	79.48	78.68
60	1.0262	-0.0027	1.0235	9.3696	0.01325	0.0052	75.31	74.56
120	1.0253	-0.0027	1.0226	9.6076	0.01325	0.0037	72.43	71.70
240	1.0243	-0.0027	1.0216	9.8722	0.01325	0.0027	69.22	68.53
480	1.0227	-0.0027	1.02	10.2954	0.01325	0.0019	64.10	63.46
1440	1.0187	-0.0027	1.016	11.3535	0.01325	0.0012	51.28	50.76

Appendix-B

(Atterberg limit test results and charts)

B.1. Test procedure

Atterberg limits for the collected samples were performed using air dried and oven dried samples. Procedures used were according to ASTM D 4318-98 standard test method for liquid limit, plastic limit and plasticity index. The Casagrande Liquid Limit Device was used for determining the liquid limits of the soils. A dry soil specimen of 150-200grams passing the 425μm (No.40) sieve was used for each test. To check whether drying will have effect on red colored soils; an air dried sample was used and for each trial fresh sample was used by fixing the mixing time to be five minutes. But from the table one can observe that the effect of air drying and oven drying of the specified samples is insignificant. For the oven dried samples conventional way of testing procedure was used. By varying the water content of the sample the tests were repeated. At least four tests were carried out for each sample by adjusting the water contents in such a way that the number of blows required to close the groove may fall within the range of 5 to 40.

<u>TP-1 @ 1.5m</u>

Data before commencement of test

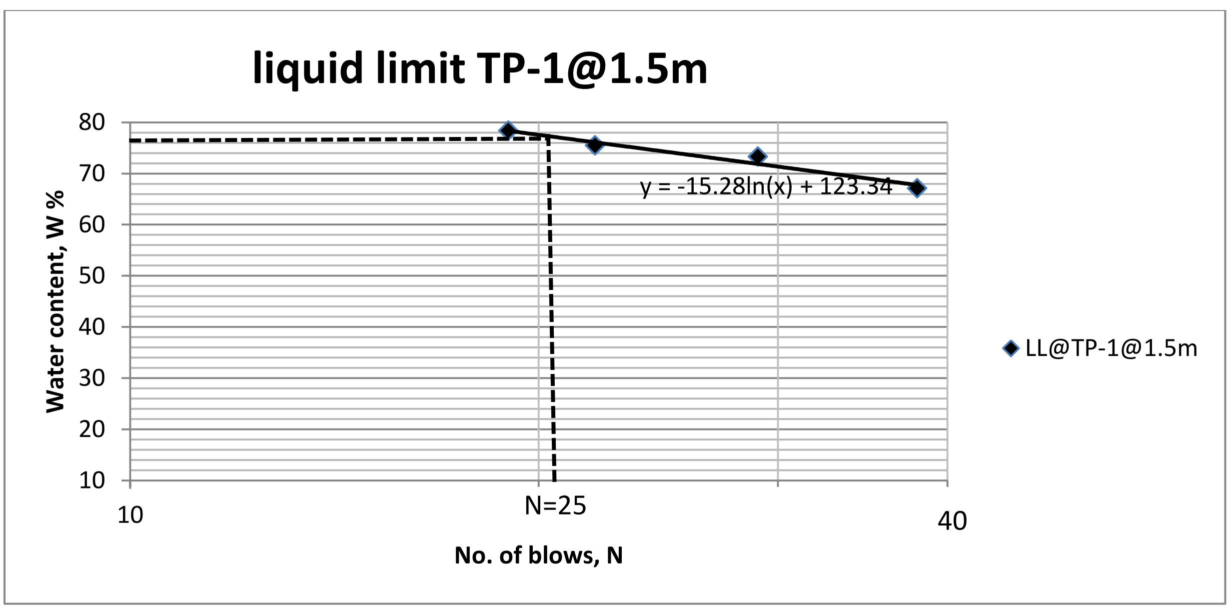
Sample type ………………………..……disturbed

Sampling depth……………………………1.5m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...31.2%

	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
Container No	H1	B1	C15	40	C6	Z1	D3
Mass of container, g	22.1	22.1	20.4	22.2	20.5	21.5	22.2
Mass of container + Wet soil, g	52	53.5	47.6	58.4	30.05	28.6	29.8
Mass of container + Dry soil, g	40	40	36.1	42.5	27.7	26.8	28.1
Mass of water, g	12	13.5	11.5	15.9	2.35	1.8	1.7
Mass of dry soil, g	17.9	17.9	15.7	20.3	7.2	5.3	5.9
Water content, %	67.0391 1	75.4189 9	73.2484 1	78.325123 2	32.63889	33.9622 6	28.8135 6
No of blows	38	22	29	19		**PL**	**31.8**



From the above plot the liquid limit is **74.6%**

TP-1 @ 3m

Data before commencement of test

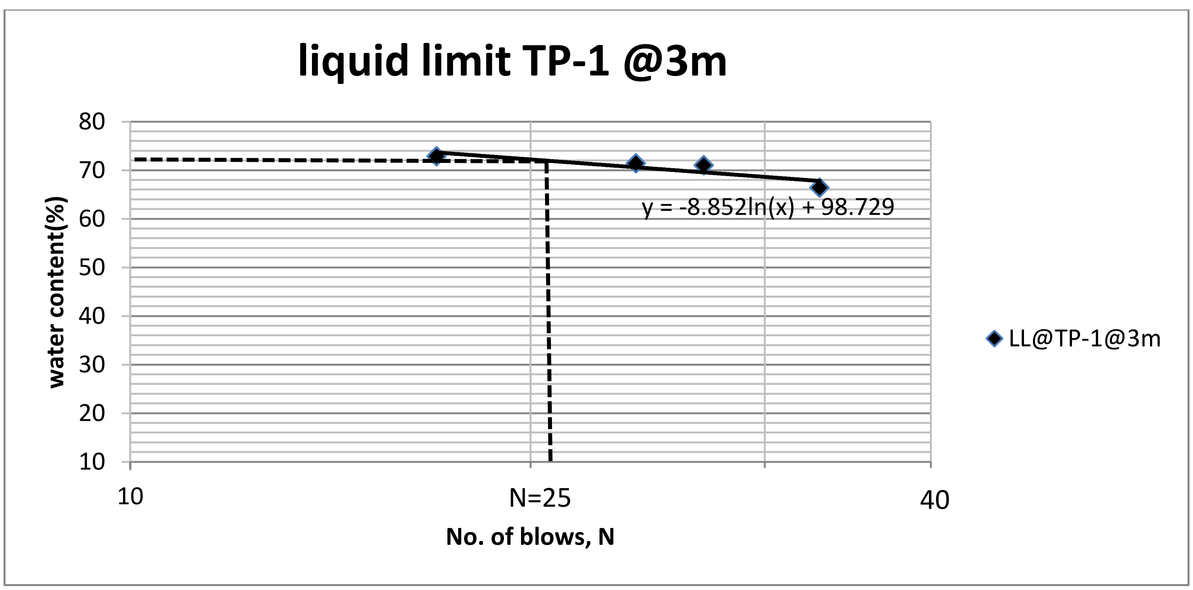
Sample type ……………………………..……disturbed

Sampling depth……………………………3m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...29.4%

	liquid limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
Container No	B1	C15	IT4	H1	T13	H7	B1
Mass of container, g	22.2	21.7	22.2	22.1	21.8	21.3	22
Mass of container + Wet soil, g	64.2	64.2	58.8	62.8	30	31.3	30.6
Mass of container + Dry soil, g	46.5	46.5	44.2	45.9	28.3	29.2	28.1
Mass of water, g	17.7	17.7	14.6	16.9	1.7	2.1	2.5
Mass of dry soil, g	24.3	24.8	22	23.8	6.5	7.9	6.1
Water content, %	72.83951	71.37097	66.36364	71.0084	26.15385	26.58228	40.98361
No of blows	17	24	33	27		**PL**	**31.2**



The liquid limit of the specimen from the above plot is **70.5%**

<u>TP-2 @ 1.5m</u>

Data before commencement of test

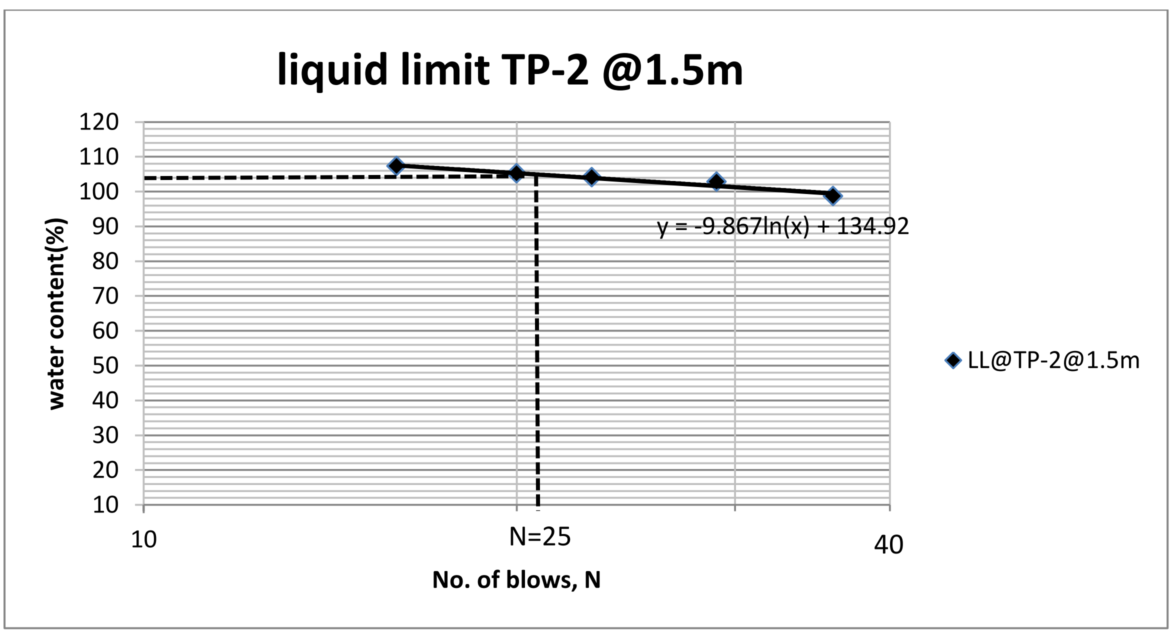
Sample type ……………………………..……disturbed

Sampling depth…………………………….1.5m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...34%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	R1	N3	H1	E1	R3	C27	IT6	74	M
Mass of container, g	22	22.4	22	22.2	22.2	21.4	21.7	22	20.8
Mass of container + Wet soil, g	53.2	58.5	57.7	61.8	64.7	29.6	30.1	32.1	32.7
Mass of container + Dry soil, g	37.7	40.2	39.4	41.6	42.7	27.2	27.6	29.2	29.1
Mass of water, g	15.5	18.3	18.3	20.2	22	2.4	2.5	2.9	3.6
Mass of dry soil, g	15.7	17.8	17.4	19.4	20.5	5.8	5.9	7.2	8.3
Water content, %	98.726	102.81	105.17	104.12	107.32	41.379	42.373	40.278	43.373
No of blows	36	29	20	23	16			**PL**	**41.9**



The liquid limit of the sample is **103.6%**

TP-2 @ 3m

Data before commencement of test

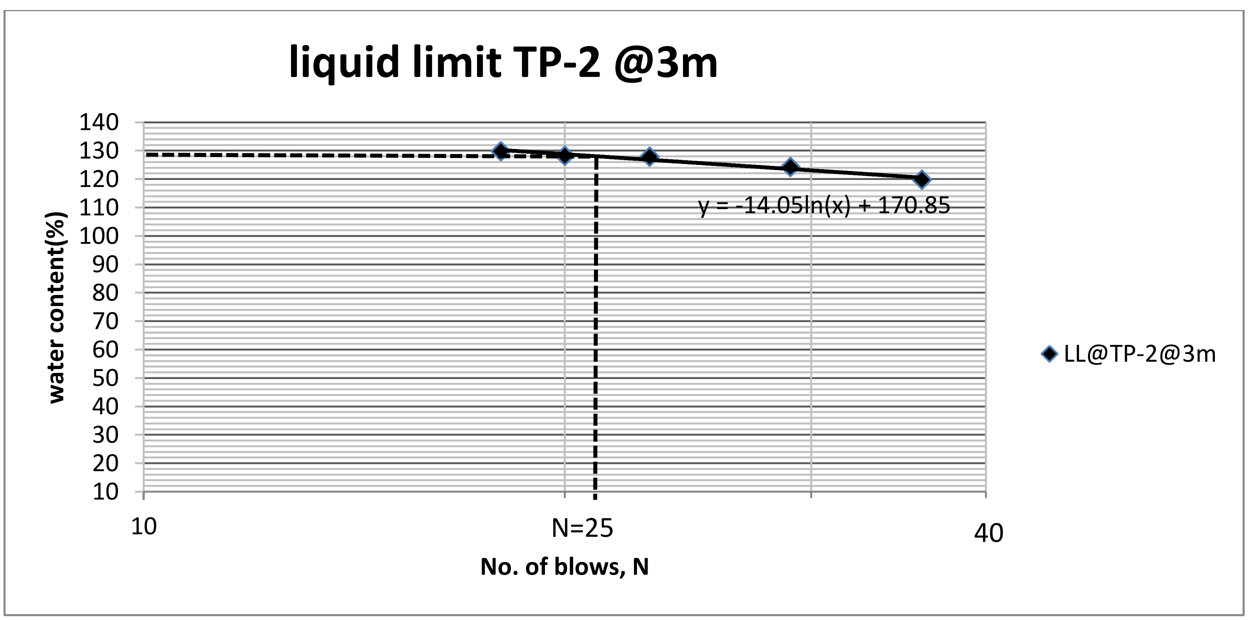
Sample type ……………………………..……disturbed

Sampling depth…………………………….3m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...66.5%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	H10	N2	85	D2	C6	B3	C3	52	H5
Mass of container, g	22.3	22.3	22.2	22.2	22.1	22.2	21.8	20.5	21.7
Mass of container + Wet soil, g	55.7	59.3	56.9	59.4	59.7	30.7	32.5	29.6	36.4
Mass of container + Dry soil, g	37.5	38.8	37.3	38.5	38.6	28.1	29.2	26.8	31.9
Mass of water, g	18.2	20.5	19.6	20.9	21.1	2.6	3.3	2.8	4.5
Mass of dry soil, g	15.2	16.5	15.1	16.3	16.5	5.9	7.4	6.3	10.2
Water content, %	119.74	124.24	129.8	128.22	127.879	44.068	44.5946	44.444	44.118
No of blows	36	29	18	20	23			**PL**	**44.3**



The liquid limit of the sample is **126.1%**

TP-3 @ 1.5m

Data before commencement of test

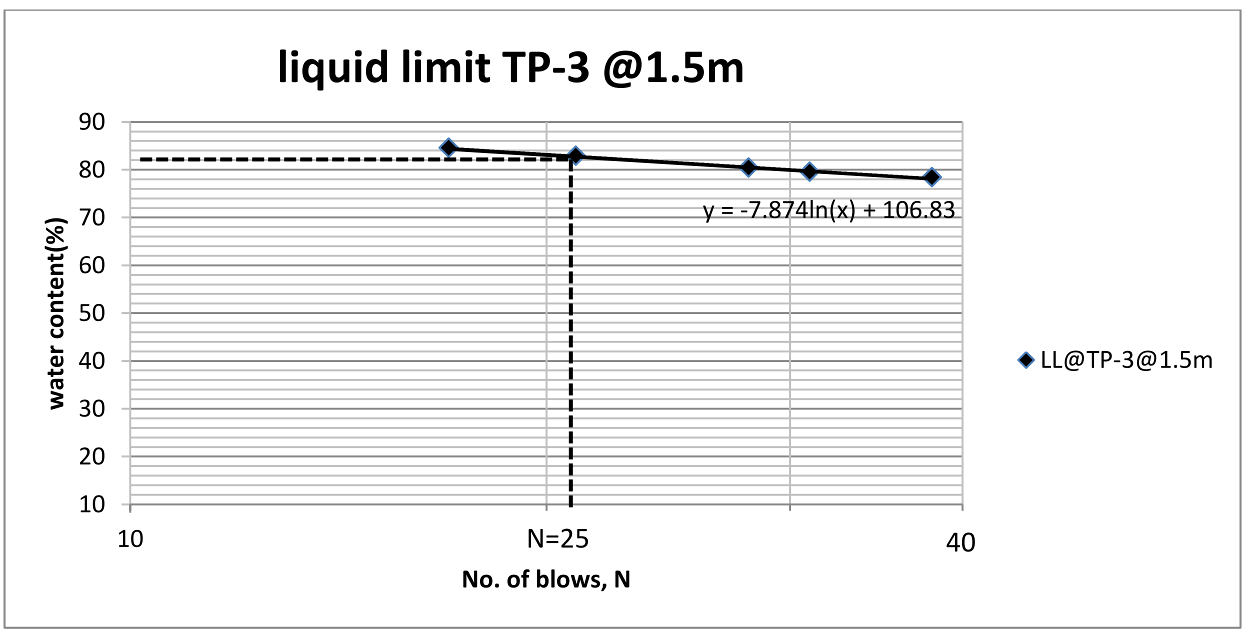
Sample type ……………………………..……disturbed

Sampling depth…………………………….1.5m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...37.6%

	liquid limit					Plastic Limit		
Trial No	1	2	3	4	5	1	2	3
Container No	C3	BA	Z4	H23	H10	T13	H7	B1
Mass of container, g	22.2	22.2	22.1	22.1	22.4	21.6	21.7	22
Mass of container + Wet soil, g	54.5	58.2	54.6	57.1	56.6	30.7	32.1	30.3
Mass of container + Dry soil, g	40.3	41.7	40.2	41.5	41.1	28.3	29.3	28.1
Mass of water, g	14.2	16.5	14.4	15.6	15.5	2.4	2.8	2.2
Mass of dry soil, g	18.1	19.5	18.1	19.4	18.7	6.7	7.6	6.1
Water content, %	78.453	84.62	79.558	80.41	82.89	35.821	36.84	36.07
No of blows	38	17	31	28	21		**PL**	**36.3**



The liquid limit from the above plot is **81.8%**

TP-3 @ 3m

Data before commencement of test

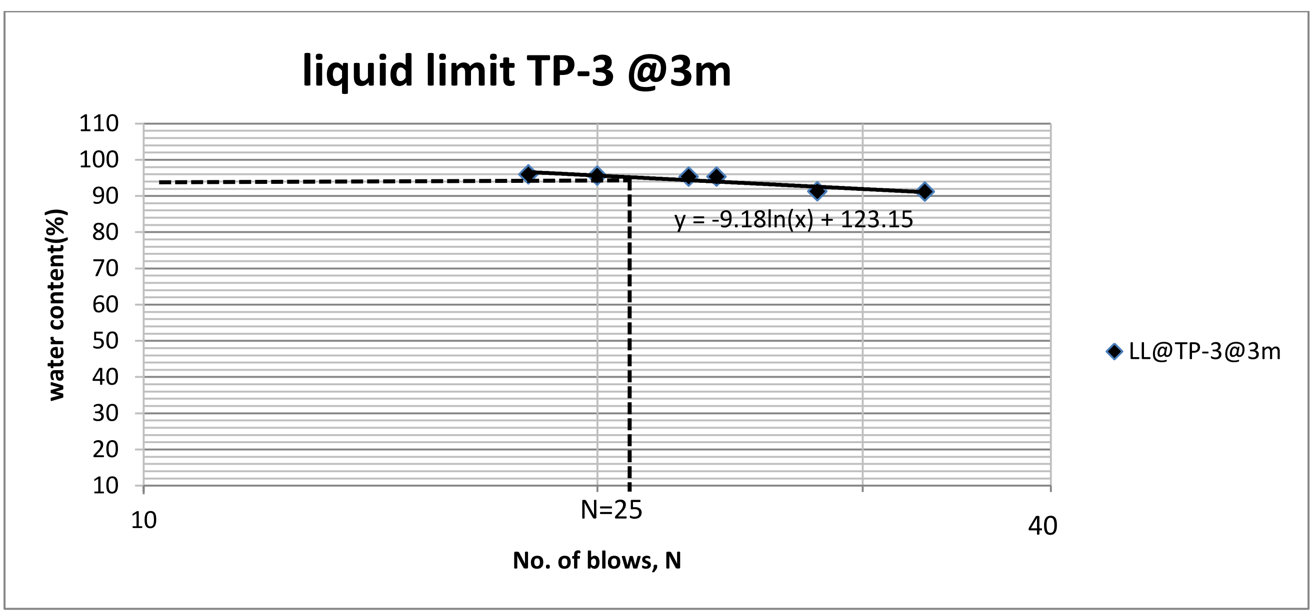
Sample type ……………………………..……disturbed

Sampling depth……………………………3m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...40%

	Liquid Limit						Plastic Limit			
Trial No	1	2	3	4	5	6	1	2	3	4
Container No	Z4	R3	G	BA	H23	C3	A6	E4	B1	H1
Mass of container, g	22	22.4	22.3	22.2	22	22	21.7	22.1	22.1	22.1
Mass of container + Wet soil, g	55	53.1	55.4	54.7	50.9	49.9	30.4	33.5	32.3	33.4
Mass of container + Dry soil, g	38.9	38.1	39.2	39.2	36.8	36.6	27.9	30.3	29.5	30.2
Mass of water, g	16.1	15	16.2	15.5	14.1	13.3	2.5	3.2	2.8	3.2
Mass of dry soil, g	16.9	15.7	16.9	17	14.8	14.6	6.2	8.2	7.4	8.1
Water content, %	95.27	95.541	95.86	91.2	95.27	91.1	40.32	39.02	37.8	39.51
No of blows	24	20	18	28	23	33			**PL**	**39.2**



The liquid limit of the above sample is **93.8%**

TP-4 @ 1.5m

Data before commencement of test

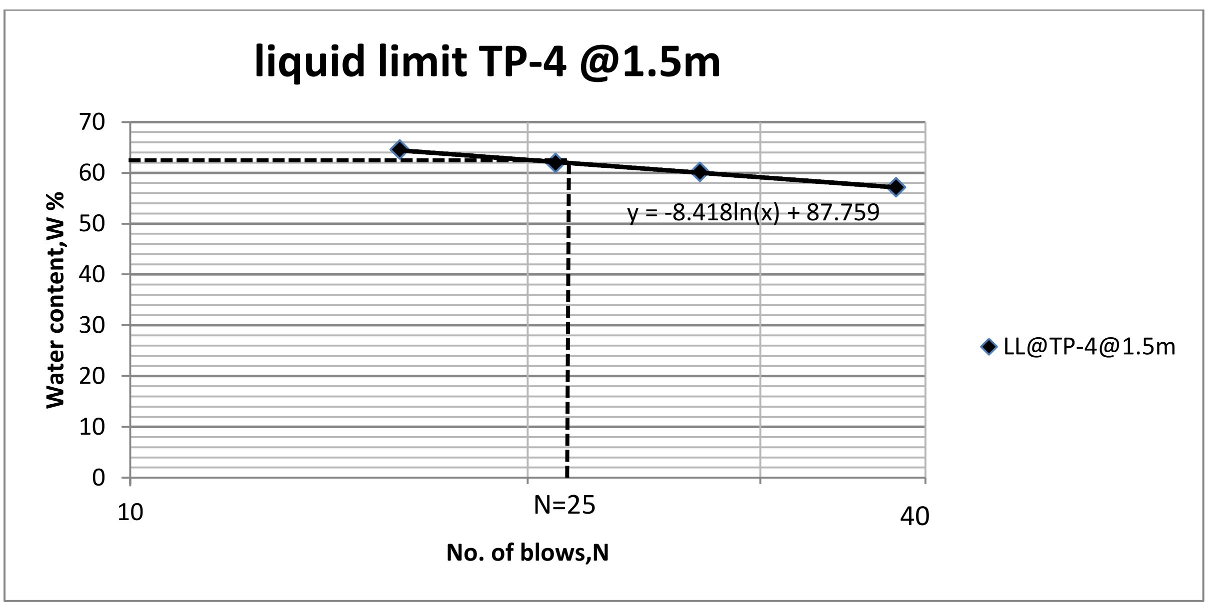
Sample type ……………………………..……disturbed

Sampling depth…………………………….1.5m

Testing condition……………………………air dried

Natural moisture content (ω)…………...27.2%

	Liquid Limit				Plastic Limit			
Trial No	1	2	3	4	1	2	3	4
Container No	A	Z4	MA	G3	Z4	PL3	D4	E4
Mass of container, g	21.9	22.2	22.2	22.3	22.1	22.1	21.8	22
Mass of container + Wet soil, g	78.1	78.3	84.7	86.3	29.4	28.3	29.8	31.4
Mass of container + Dry soil, g	57	57.9	60.8	61.2	27.7	26.8	27.9	29.1
Mass of water, g	21.1	20.4	23.9	25.1	1.7	1.5	1.9	2.3
Mass of dry soil, g	35.1	35.7	38.6	38.9	5.6	4.7	6.1	7.1
Water content, %	60.114	57.1429	61.917	64.524	30.357	31.915	31.148	32.394
No of blows	27	38	21	16			**PL**	**31.4**



The liquid limit from the above plot is **61.1%**

TP-4 @ 2.7m

Data before commencement of test

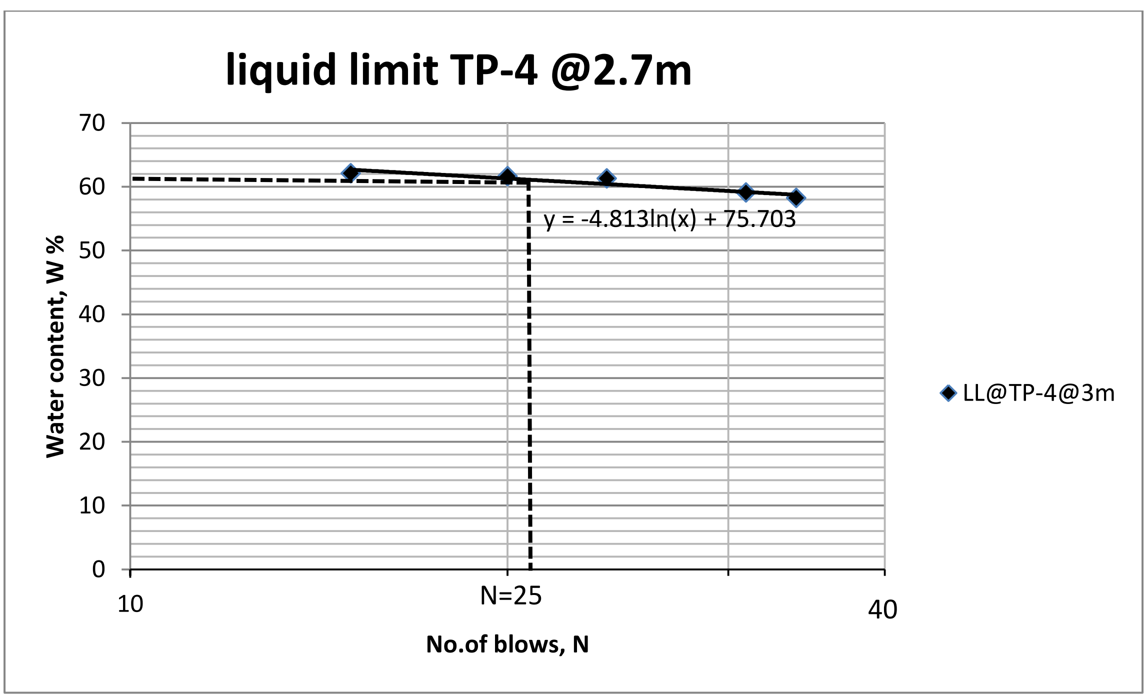
Sample type ……………………………..……disturbed

Sampling depth……………………………2.7m

Testing condition……………………………air dried

Natural moisture content (ω)…………...35.4%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	TP1	TP4	TP5	TP6	C15	C3	G3	R1	E4
Mass of container, g	22	21.7	22.1	22	21.8	21.8	22.1	22.1	21.4
Mass of container + Wet soil, g	71	69	80	74.2	77.4	28.9	33.1	32.4	31.4
Mass of container + Dry soil, g	52.8	51.6	58	54.3	56.1	27.2	30.4	30	29
Mass of water, g	18.2	17.4	22	19.9	21.3	1.7	2.7	2.4	2.4
Mass of dry soil, g	30.8	29.9	35.9	32.3	34.3	5.4	8.3	7.9	7.6
Water content, %	59.09	58.19	61.28	61.61	62.1	31.48	32.53	30.38	31.58
No of blows	31	34	24	20	15			**PL**	**31.5**



The liquid limit from the plot is **60.4%**

TP-5 @ 1.5m

Data before commencement of test

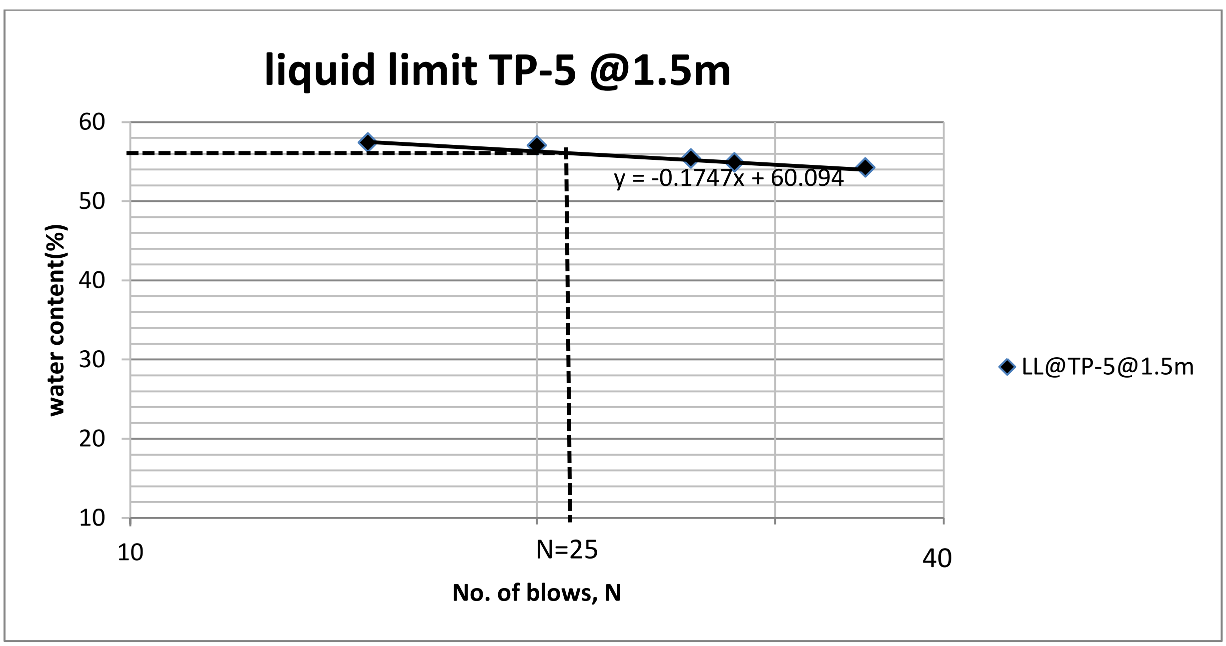
Sample type ……………………………..……disturbed

Sampling depth…………………………….1.5m

Testing condition……………………………air dried

Natural moisture content (ω)…………...36.4%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	C27	A	85	N2	52	R3	N3	L5	E1
Mass of container, g	22	21.9	22.3	22.4	22.4	22.2	22.4	21.9	22.4
Mass of container + Wet soil, g	79.6	73.4	76.5	73.2	77.3	32.4	33.8	33.2	36.6
Mass of container + Dry soil, g	58.6	54.7	57.3	55.1	58	30	31.1	30.6	33.2
Mass of water, g	21	18.7	19.2	18.1	19.3	2.4	2.7	2.6	3.4
Mass of dry soil, g	36.6	32.8	35	32.7	35.6	7.8	8.7	8.7	10.8
Water content, %	57.38	57.01	54.86	55.35	54.21	30.77	31	29.9	31.48
No of blows	15	20	28	26	35			**PL**	**30.8**



The liquid limit of the given sample is **55.7%**

TP-5 @ 2.8m

Data before commencement of test

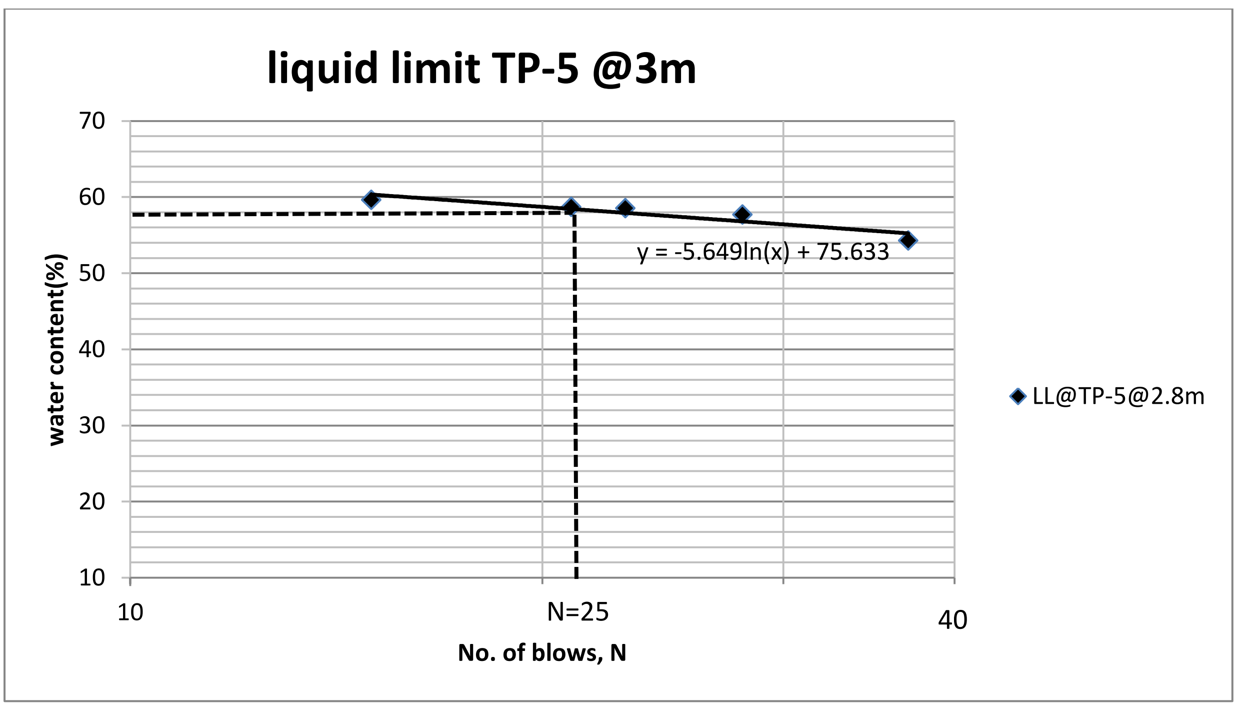
Sample type ………………………….……disturbed

Sampling depth…………………………….2.8m

Testing condition……………………………air dried

Natural moisture content (ω)…………...34%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	E1	E7	2	E4	C1	H5	M	A2	74
Mass of container, g	21.8	22	22.2	22	22.2	22.3	20.7	20.7	22.1
Mass of container + Wet soil, g	73.2	77.8	79.8	78	78.7	31.2	30.5	31.2	31.2
Mass of container + Dry soil, g	54.4	57.2	58.5	58.3	57.6	28.9	28	28.5	28.9
Mass of water, g	18.8	20.6	21.3	19.7	21.1	2.3	2.5	2.7	2.3
Mass of dry soil, g	32.6	35.2	36.3	36.3	35.4	6.6	7.3	7.8	6.8
Water content, %	57.67	58.523	58.68	54.3	59.6	34.8	34.25	34.62	33.8
No of blows	28	23	21	37	15			**PL**	**34.4**



The liquid limit of the sample under consideration is **57.7%**

TP-6 @ 1.5m

Data before commencement of test

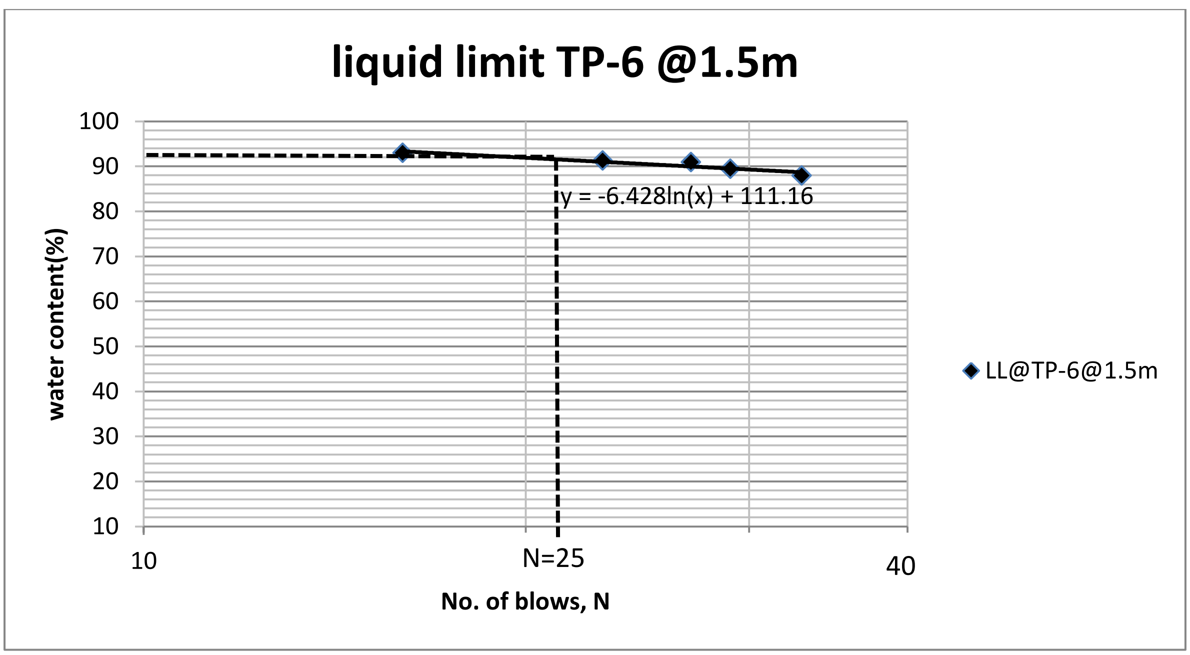
Sample type ………………………..……disturbed

Sampling depth……………………………1.5m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...25.5%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	E1	E2	N3	85	E4	C1	L5	N2	C27
Mass of container, g	22.4	22.2	22.3	22.2	22.1	22.2	21.4	22.3	21.8
Mass of container + Wet soil, g	70.6	72.6	78.5	74.3	82.9	30.8	33	32.4	30.9
Mass of container + Dry soil, g	47.6	48.6	52.2	49.2	54.2	28.5	29.9	29.7	28.4
Mass of water, g	23	24	26.3	25.1	28.7	2.3	3.1	2.7	2.5
Mass of dry soil, g	25.2	26.4	29.9	27	32.1	6.3	8.5	7.4	6.6
Water content, %	91.27	90.91	87.96	92.96	89.41	36.51	36.5	36.5	37.88
No of blows	23	27	33	16	29			**PL**	**36.84**



The liquid limit from the above plot is **90.7%**

TP-6 @ 3m

Data before commencement of test

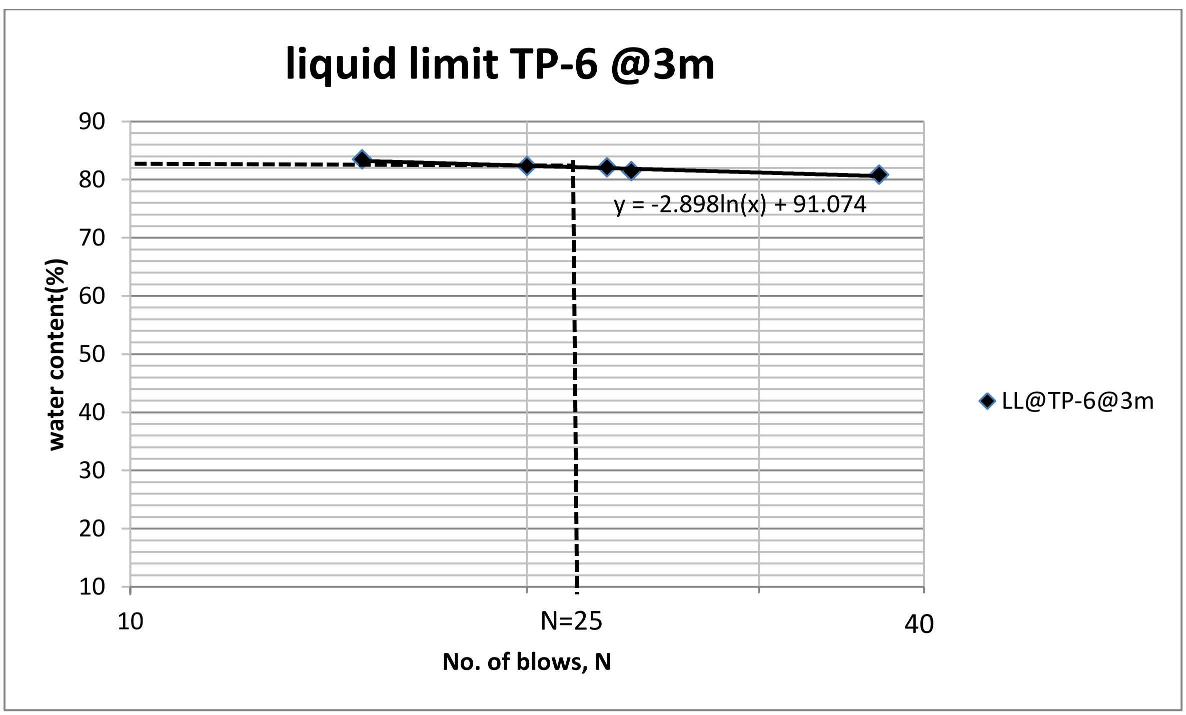
Sample type ……………………………..……disturbed

Sampling depth……………………………3m

Testing condition……………………………oven dried

Natural moisture content (ω)…………...32.9%

	Liquid Limit					Plastic Limit			
Trial No	1	2	3	4	5	1	2	3	4
Container No	H5	E7	D2	52	E3	M	R3	A	A2
Mass of container, g	22.4	22.1	22.1	22.4	22.2	20.4	21.7	21.4	20.8
Mass of container + Wet soil, g	78.9	74.9	78.8	81.4	81.7	29.1	31	31.9	32.2
Mass of container + Dry soil, g	53.2	51.3	53.2	54.8	55	26.9	28.7	29.2	29.1
Mass of water, g	25.7	23.6	25.6	26.6	26.7	2.2	2.3	2.7	3.1
Mass of dry soil, g	30.8	29.2	31.1	32.4	32.8	6.5	7	7.8	8.3
Water content, %	83.44	80.822	82.32	82.1	81.4	33.8	32.86	34.62	37.3
No of blows	15	37	20	23	24			**PL**	**34.7**



The liquid limit of the given sample is **81.9%**

Appendix –C

(Detailed sample calculation for unconfined compression test)

D-1. Test procedure

Unconfined compressive strength tests may be performed on compacted or undisturbed specimens. But for this case undisturbed samples were tested by using standard test method on ASTM D 2166-98a. Undisturbed specimens were carefully trimmed from undisturbed field samples (e.g. Shelby tube samples) using soil trimming tools.

Test specimens satisfied the following criteria: 1) minimum diameter of 1.3 in., 2) maximum particle size less than one-tenth of the specimen diameter (or one-sixth of the diameter for specimens with diameters larger than 2.8 in.), and 3) a height: diameter ratio between 2.0 and 2.5. Moisture loss tried to be minimized between the time the specimen is prepared and when it is tested. Prior to testing, the specimen was weighed and measured.

Test sample #2

Trial #1

Data before commencement of unconfined compression test

Sample type …………………………….……Undisturbed

Sampling depth…………………………….1.5m

Natural moisture content (ω)…………...33.95%

Specific gravity of the soil (Gs)………….2.66

Initial unit weight (γ_b)……………………17.6kN/m^3

Initial height of the specimen (H_0)……….81mm

Diameter of the specimen (D_0)…………...38mm

Deformation dial reading- 1-unit…………0.01mm

Load dial reading-1 unit………………….0.00142kN

Using the above data:

Initial cross-sectional area:

$$A_0 = \pi(\frac{D_0}{2})^2$$

=1133.54mm

Axial strain:

$$\varepsilon_a = \frac{\Delta H}{H_0}$$

Corrected cross-sectional area:

$$A_C = \frac{A_0}{1 - \varepsilon}$$

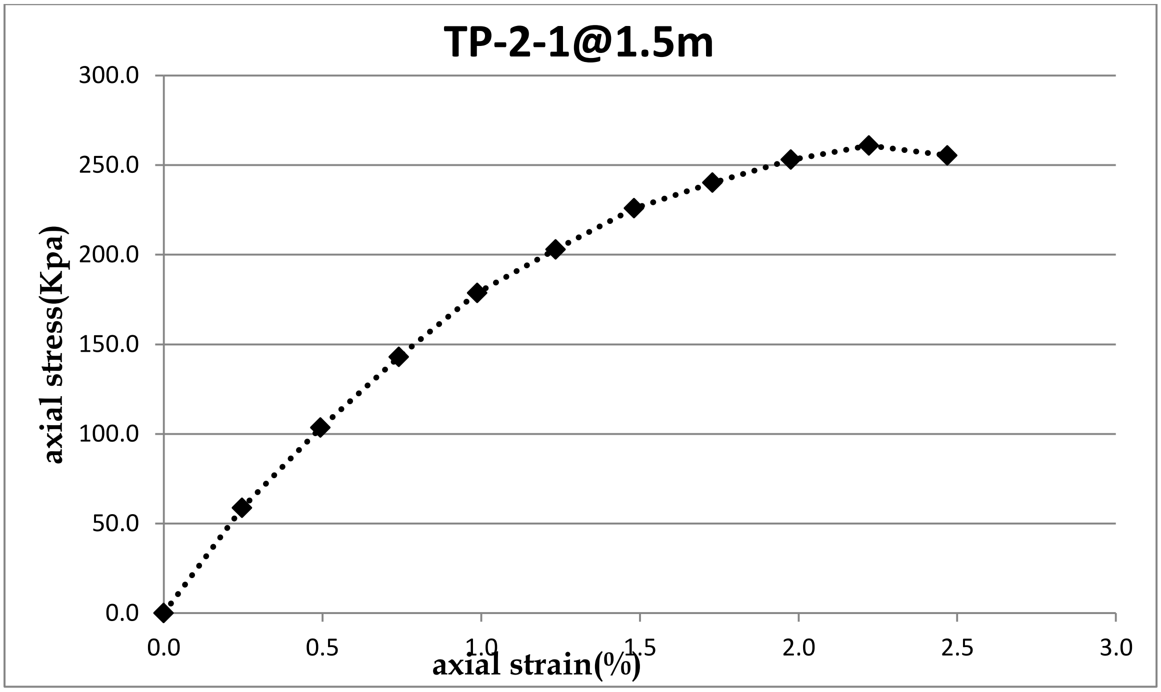
Axial (vertical) stress:

$$\sigma = \frac{N}{A_C}$$

The detailed calculation is presented in table below:

deformation dial reading	load dial reading	sample deformation ΔL(mm)	Strain, ε(mm)	% strain	corrected area,A_C (m^2)	Load, N (kN)	Stress, σ (kpa)
0	0	0	0.0000	0.0000	0.0011	0.0000	0.0000
20	47	0.2	0.0025	0.2469	0.0011	0.0667	58.7321
40	83	0.4	0.0049	0.4938	0.0011	0.1179	103.4617
60	115	0.6	0.0074	0.7407	0.0011	0.1633	142.9948
80	144	0.8	0.0099	0.9877	0.0011	0.2045	178.6090
100	164	1	0.0123	1.2346	0.0011	0.2329	202.9085
120	183	1.2	0.0148	1.4815	0.0012	0.2599	225.8502
140	195	1.4	0.0173	1.7284	0.0012	0.2769	240.0569
160	206	1.6	0.0198	1.9753	0.0012	0.2925	252.9614
180	213	1.8	0.0222	2.2222	0.0012	0.3025	260.8983
200	209	2	0.0247	2.4691	0.0012	0.2968	255.3523

Axial strain versus vertical stress is plotted as shown below:



As obtained previously the consistency of the soil in the study area fall in the range of medium to hard, therefore the unconfined compressive strength (q_u of the specimen is the peak of $\sigma - \varepsilon$ curve.

Therefore from the above curve:

$$q_u = 261kpa$$

Trial #2

Data before commencement of unconfined compression test

Sample type ………………………….……Undisturbed

Sampling depth…………………………….1.5m

Natural moisture content (ω)…………...33.95%

Specific gravity of the soil (Gs)………….2.66

Initial unit weight (γ_b)……………………17.6kN/m3

Initial height of the specimen (H_0)……….83mm

Diameter of the specimen (D_0)…………...38mm

Deformation dial reading- 1-unit…………0.01mm

Load dial reading-1 unit…………………0.00142kN

Using the above data:

Initial cross-sectional area:

$$A_0 = \pi(\frac{D_0}{2})^2$$

=1133.54mm

Axial strain:

$$\varepsilon_a = \frac{\Delta H}{H_0}$$

Corrected cross-sectional area:

$$A_C = \frac{A_0}{1-\varepsilon}$$

Axial (vertical) stress:

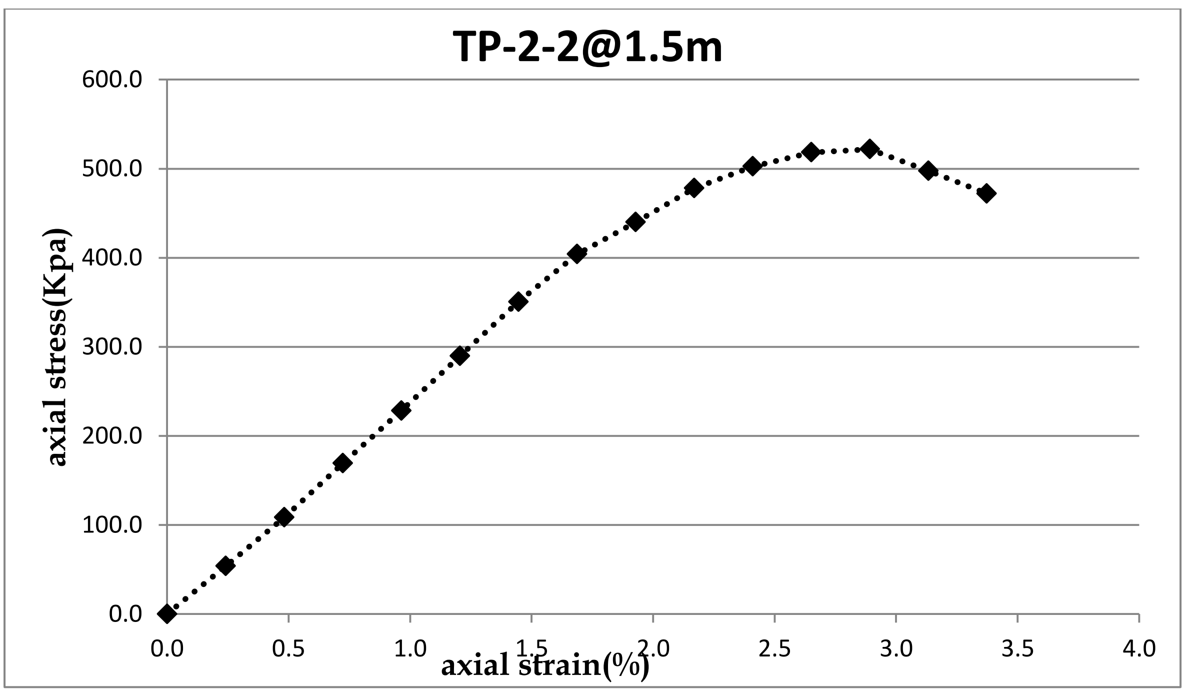
$$\sigma = \frac{N}{A_C}$$

The detailed calculation is presented in table below:

deformation dial reading	load dial reading	sample deformation ΔL(mm)	Strain, ε(mm)	% strain	corrected area, A_C (m²)	Load, N (kN)	Stress, σ (kpa)
0	0	0	0.0000	0.0000	0.0011	0.0000	0.0000
20	43	0.2	0.0024	0.2410	0.0011	0.0611	53.7368
40	87	0.4	0.0048	0.4819	0.0011	0.1235	108.4608
60	136	0.6	0.0072	0.7229	0.0011	0.1931	169.1374
80	184	0.8	0.0096	0.9639	0.0011	0.2613	228.2775
100	234	1	0.0120	1.2048	0.0011	0.3323	289.6030

120	284	1.2	0.0145	1.4458	0.0012	0.4033	350.6267
140	328	1.4	0.0169	1.6867	0.0012	0.4658	403.9591
160	358	1.6	0.0193	1.9277	0.0012	0.5084	439.8259
180	390	1.8	0.0217	2.1687	0.0012	0.5538	477.9627
200	411	2	0.0241	2.4096	0.0012	0.5836	502.4586
220	425	2.2	0.0265	2.6506	0.0012	0.6035	518.2910
240	429	2.4	0.0289	2.8916	0.0012	0.6092	521.8741
260	410	2.6	0.0313	3.1325	0.0012	0.5822	497.5232
280	390	2.8	0.0337	3.3735	0.0012	0.5538	472.0765

Axial strain versus vertical stress is plotted as shown below:



$$q_u = 522kpa$$

<u>Trial #3</u>

Data before commencement of unconfined compression test

Sample type ……………………………Undisturbed

Sampling depth…………………………….1.5m

Natural moisture content (ω)…………...33.95%

Specific gravity of the soil (Gs)…………2.66

Initial unit weight (γ_b)……………………17.6kN/m3

Initial height of the specimen (H_0)………86mm

Diameter of the specimen (D_0)…………...38mm

Deformation dial reading- 1-unit…………0.01mm

Load dial reading-1 unit………………….0.00142kN

Using the above data:

Initial cross-sectional area:

$$A_0 = \pi(\frac{D_0}{2})^2$$

=1133.54mm

Axial strain: $\varepsilon_a = \frac{\Delta H}{H_0}$

Corrected cross-sectional area: $A_C = \frac{A_0}{1-\varepsilon}$

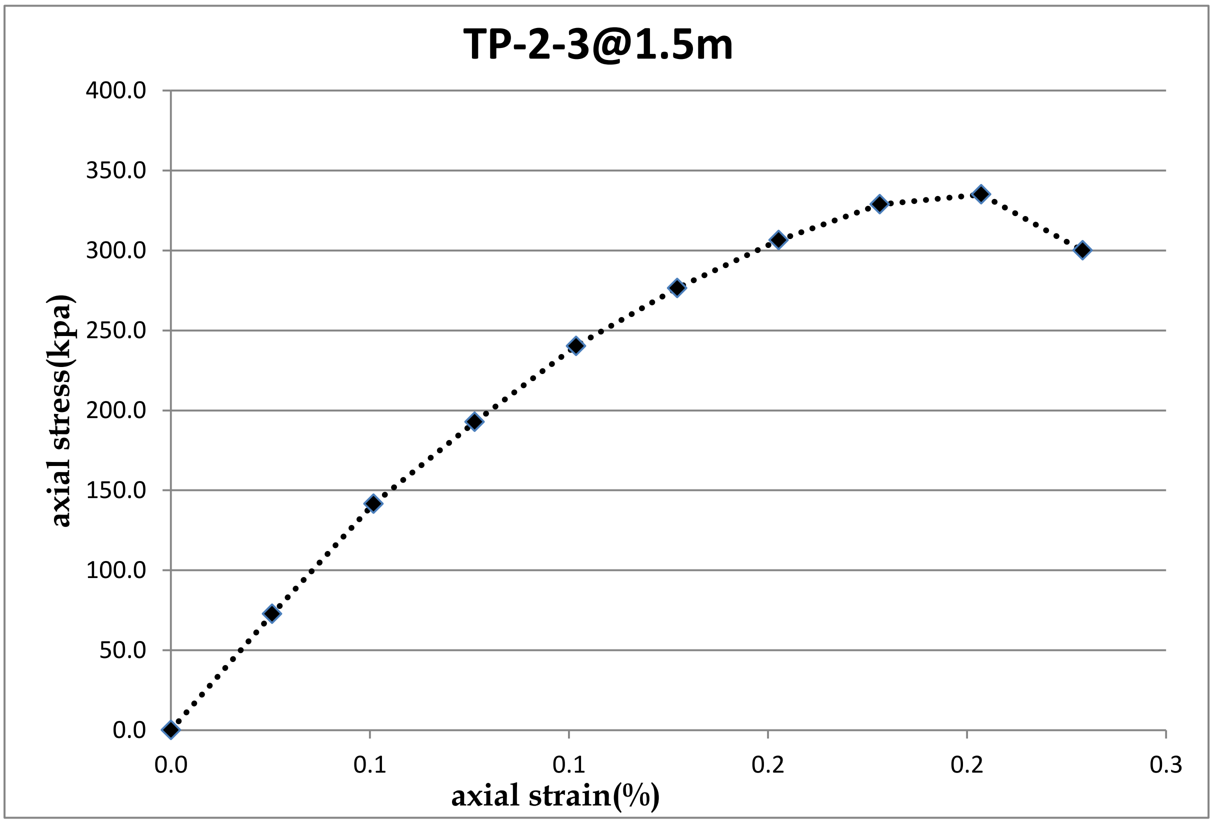
Axial (vertical) stress: $\sigma = \frac{N}{A_C}$

The detailed calculation is presented in table below:

deformation dial reading	load dial reading	sample deformation ΔL(mm)	Strain, ε(mm)	% strain	corrected area, A_C (m²)	Load, N (kN)	Stress, σ (kpa)
0	0	0	0.0000	0.0000	0.0011	0.0000	0.0000
20	58	0.2	0.0003	0.0254	0.0011	0.0824	72.6389
40	113	0.4	0.0005	0.0509	0.0011	0.1605	141.4845
60	154	0.6	0.0008	0.0763	0.0011	0.2187	192.7705

80	192	0.8	0.0010	0.1018	0.0011	0.2726	240.2760
100	221	1	0.0013	0.1272	0.0011	0.3138	276.4973
120	245	1.2	0.0015	0.1527	0.0011	0.3479	306.4460
140	263	1.4	0.0018	0.1781	0.0011	0.3735	328.8766
160	268	1.6	0.0020	0.2036	0.0011	0.3806	335.0436
180	240	1.8	0.0023	0.2290	0.0011	0.3408	299.9625

Axial strain versus vertical stress is plotted as shown below:



$$q_u = 335kpa$$

Finally the unconfined strength of the specimen at the specified depth is taken as the average of the three results:

$$q_u = 370kpa$$

And the undrained shear strength (S_u) of the specimen can be determined by plotting Mohr's Circle, which is the radius of the circle:

$$S_u = \frac{q_u}{2} \qquad =185\text{kpa}$$

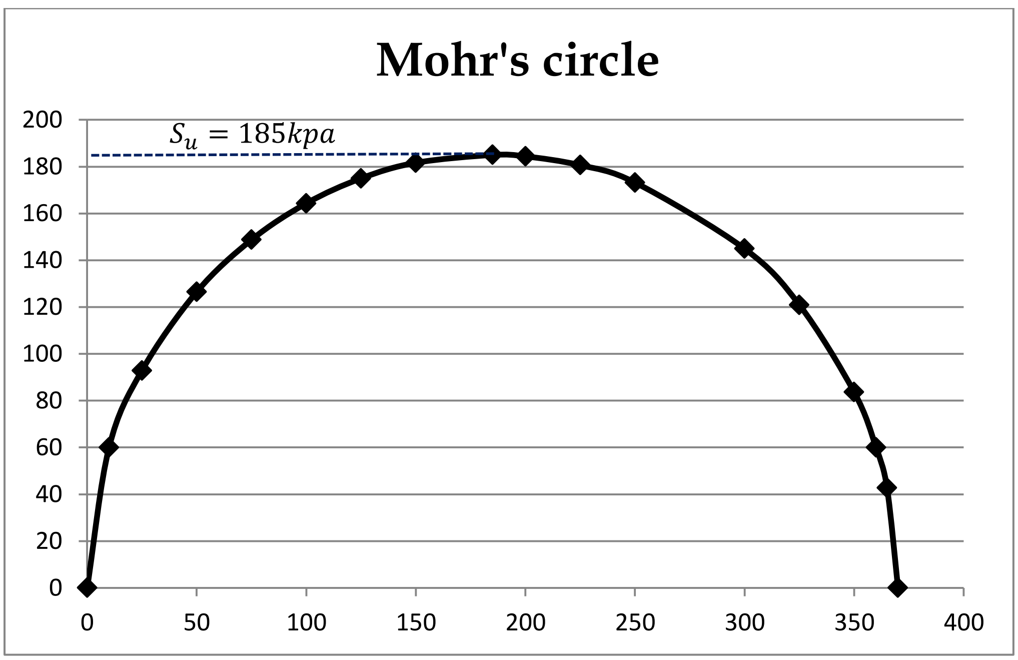
Mohr's circle is drawn using equation:

$$(\sigma - \frac{q_u}{2})^2 - \tau^2 = (\frac{q_u}{2})^2$$

Where $\sigma = axial\ stress$

$\tau = S_u = C_u = undrained\ shear\ strength$

$q_u = unconfined\ compressive\ strength$



σ	τ
0	0
10	60
25	92.87088
50	126.4911
75	148.7447
100	164.3168
125	175
150	181.659
185	185
200	184.3909
225	180.6239
250	173.2051
300	144.9138
325	120.9339
350	83.666
360	60
365	42.72002
370	0

Appendix –D

(Detail calculations of consolidation test results)

D.2. Test procedure

One-dimensional consolidation of a soil is determined by means of the apparatus known as consolidometer (sometimes referred to as Oedometer). The test was performed by placing a cylindrical specimen of undisturbed soil sample in a consolidation cell. With regard to specimen dimensions, ASTM D2435-96 specified that 1) the minimum height and diameter was 0.5 in. and 2.00 in., respectively, 2) the height exceed 10 times the maximum particle size, and 3) the diameter: height ratios exceed 2.5.

An undisturbed specimen is recovered using Shelby tube sampling and carefully trimmed of the soil to remove the outer, more disturbed portion of the soil, and to cut a soil specimen that fits into a consolidation ring. Using this approach, a consolidation ring with a slanting edge was slowly and gently pushed onto the undisturbed soil specimen with a diameter larger than the diameter of the ring. As the ring was incrementally slid down onto the soil specimen, excess soil was carefully trimmed away from the sides of the specimen so that the trimmed soil specimen fits closely into the consolidation ring.

The soil-filled consolidation ring is then placed in the consolidation cell. The soil specimen was sandwiched between two porous stones. The bottom stone was fixed to the consolidation cell, while the top stone was fixed to the loading cap used to transfer load to the soil specimen. The porous stones act as freely draining materials so that drainage in the soil specimen was two-way and the drainage distance was half the height of the specimen. The consolidation cell was filled with water and placed in the load frame.

When the consolidation cell is first placed in the load frame, a seating load of 7kpa was applied until the soil saturated fully. Once the seating load was applied, the deformation indicator was set to zero. Loads were applied in steps such a way that the successive load intensity, p, was twice the preceding one. The load intensities used were 50, 100, 200, 400, 800 and 1600 kpa. Each load was being allowed to stand until compression has practically ceased (for 24 hours). The dial readings were taken at elapsed time of 0, ¼, ½, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, and 1440 minutes from the time the new increment of load was put on the sample. This procedure was followed for all the samples.

C.1. Test Sample # 03

Sample Designation…………………………TP-3

Location…………………………………around Catholic Church

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type ……………………….……Undisturbed

Swelling characteristics …………………. Expansive

Bulk (wet) density (γ_b) …………………..1.76g/cm^3

Dry density (γ_b)……………………………1.51g/cm^3

Natural moisture content (%)………….....40%

Specific gravity of the soil (Gs)…………...2.72

Weight of consolidation ring (gm.)……....37.7

Diameter of specimen (mm)……………….50

Area of the specimen (cm^2)…………………19.63

Volume of sample (cm^3)…………………….39.25

Weight of wet soil (gm.)……………………...73.80

Dry weight of the specimen after

Oven drying (gm.)……………………………52.60

Using the above data detail calculation is presented below:

1. Determination of height of solids, H_S, in the soil specimen:

$$H_S = \frac{M_S}{A * G_S * \rho_W}$$

$$H_S = \frac{52.60}{19.63*2.72*1} * 10 = 9.85mm$$

2. Determination of initial height of voids, H_V:

$$H_V = H - H_S\ ,$$

Where H= initial height of the specimen

$$= 20 - 9.85 \quad = 10.15mm$$

3. Determination of the initial void ratio, e_0, of the specimen:

$$e_0 = \frac{V_V}{V_S} = \frac{H_V}{H_S}\frac{A}{A} = \frac{H_V}{H_S}$$

$$= \frac{10.15}{9.85} = 1.03$$

4. For the first incremental loading σ_1 (total load/unit area of specimen), which causes deformation ΔH_1 , change in void ratio Δe_1:

$$\Delta e_1 = \frac{\Delta H_1}{H_S}$$

$$= \frac{20 - 19.878}{9.85} = 0.012$$

ΔH_1 is obtained from the initial and the final dial readings for the loading. At this time, the effective pressure on the specimen is $\sigma' = \sigma_1 = \sigma_1'$

5. The new void ratio, e_1, after consolidation caused by the pressure increment σ_1 is:

$$e_1 = e_0 - \Delta e_1$$

$$= 1.03 - 0.012 = 1.018$$

For the next loading, σ_2 (*note*: σ_2 equals the cumulative load per unit area of specimen), which causes additional deformation ΔH_2, the void ratio e_2 at the end of consolidation can be calculated as:

$$e_2 = e_1 - \frac{\Delta H_2}{H_S}$$

$$= 1.018 - \frac{19.878 - 19.738}{9.85} = 1.004$$

Note that, at this time, the effective pressure on the specimen is $\sigma' = \sigma_2 = \sigma_2'$.

Proceeding in similar manner, one can obtain the void ratios at the end of the consolidation for all load increments are presented on table C-3 using load dial readings and final specimen height summarized on table C-1 and C-2 for loading unloading respectively.

The effective pressures ($\sigma = \sigma'$) and the corresponding void ratios (e) at the end of consolidation are plotted on semi logarithmic graph as shown on figure C-1 below.

Table C-1 Dial gauge reading and final specimen height for each incremental loading

Load(kpa)	50		100		200		400		800		1600	
Time	Dial reading(mm)	final height, H(mm)	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0se	1.4	20	1.522	19.878	1.662	19.738	1.9485	19.45	2.42	18.98	3.02	18.38
8se.	1.42	19.98	1.56	19.84	1.742	19.658	2.05	19.35	2.47	18.93	3.084	18.316
15sec.	1.46	19.94	1.568	19.832	1.75	19.65	2.06	19.34	2.495	18.905	3.09	18.31
30se.	1.464	19.936	1.572	19.828	1.76	19.64	2.072	19.33	2.51	18.89	3.1	18.3
1min	1.468	19.932	1.578	19.822	1.77	19.63	2.088	19.31	2.52	18.88	3.12	18.28
2min	1.472	19.928	1.584	19.816	1.788	19.612	2.104	19.3	2.532	18.868	3.138	18.262
4min	1.475	19.925	1.5925	19.808	1.799	19.602	2.122	19.28	2.552	18.848	3.162	18.238
8min	1.479	19.921	1.598	19.802	1.818	19.582	2.158	19.24	2.5785	18.822	3.21	18.19
15min	1.482	19.918	1.605	19.795	1.832	19.568	2.185	19.22	2.612	18.788	3.256	18.144
30min	1.489	19.911	1.616	19.784	1.855	19.545	2.215	19.19	2.658	18.742	3.295	18.105
1hr	1.496	19.904	1.626	19.774	1.878	19.522	2.243	19.16	2.7185	18.682	3.348	18.052
2hr	1.502	19.898	1.636	19.764	1.897	19.504	2.281	19.12	2.796	18.604	3.415	17.985
4hr	1.508	19.892	1.646	19.754	1.918	19.482	2.325	19.08	2.897	18.503	3.467	17.933
8hr	1.517	19.883	1.655	19.745	1.938	19.462	2.369	19.03	2.996	18.404	3.548	17.852
12hr	1.521	19.879	1.658	19.742	1.942	19.458	2.399	19	3.012	18.388	3.569	17.831
24hr	1.522	19.878	1.662	19.738	1.949	19.452	2.42	18.98	3.02	18.38	3.585	17.815

Table C-2 Dial gauge readings and final height for each decrement of unloading

Load(kpa)	1600		800		400		200		100		50	
Time	Dial reading (mm)	Final specimen height, H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0se	3.585	17.815	3.426	17.974	3.175	18.225	2.856	18.544	2.573	18.827	2.301	19.099
8se.	3.61	17.79	3.39	18.01	3.12	18.28	2.848	18.552	2.555	18.845	2.294	19.106
15sec.	3.606	17.794	3.388	18.012	3.116	18.284	2.84	18.56	2.553	18.847	2.293	19.107
30se.	3.604	17.796	3.386	18.014	3.114	18.286	2.838	18.562	2.552	18.848	2.292	19.108
1min	3.602	17.798	3.385	18.015	3.113	18.287	2.837	18.563	2.551	18.849	2.29	19.11
2min	3.596	17.804	3.382	18.018	3.11	18.29	2.826	18.574	2.548	18.852	2.288	19.112
4min	3.59	17.81	3.376	18.024	3.108	18.292	2.823	18.577	2.544	18.856	2.282	19.118
8min	3.573	17.827	3.366	18.034	3.094	18.306	2.818	18.582	2.542	18.858	2.276	19.124
15min	3.564	17.836	3.357	18.043	3.09	18.31	2.806	18.594	2.537	18.863	2.268	19.132
30min	3.544	17.856	3.34	18.06	3.078	18.322	2.794	18.606	2.528	18.872	2.25	19.15
1hr	3.516	17.884	3.31	18.09	3.052	18.348	2.78	18.62	2.512	18.888	2.216	19.184
2hr	3.485	17.915	3.275	18.125	3.024	18.376	2.752	18.648	2.493	18.907	2.144	19.256
4hr	3.455	17.945	3.234	18.166	2.972	18.428	2.712	18.688	2.458	18.942	2.09	19.31
8hr	3.449	17.951	3.179	18.221	2.922	18.478	2.659	18.741	2.428	18.972	2.048	19.352
12hr	3.443	17.957	3.177	18.223	2.906	18.494	2.638	18.762	2.403	18.997	2.031	19.369
24hr	3.426	17.974	3.175	18.225	2.856	18.544	2.573	18.827	2.301	19.099	1.98	19.42

Table C-3 summery of void ratios at the end of each incremental loading and unloading

Applied pressure σ' (kpa)	Final Dial Reading (mm)	Change In Specimen Height, ΔH (mm)	Final Specimen Height (mm)	Void Height, H_V (mm)	Void Ratio, e
Loading					
7	1.100	0.000	20.000	10.15	1.030
7	1.700	0.600	20.600	10.75	1.091
50	1.522	0.122	19.878	10.03	1.018
100	1.662	0.140	19.738	9.89	1.004
200	1.949	0.287	19.452	9.60	0.975
400	2.420	0.472	18.980	9.13	0.927
800	3.020	0.600	18.380	8.53	0.866
1600	3.585	0.565	17.815	7.96	0.809
Unloading					
1600	3.585	0.565	17.815	7.965	0.809
800	3.175	-0.410	18.225	8.375	0.850
400	2.856	-0.319	18.544	8.694	0.883
200	2.573	-0.283	18.827	8.977	0.911
100	2.301	-0.272	19.099	9.249	0.939
50	1.980	-0.321	19.420	9.570	0.972

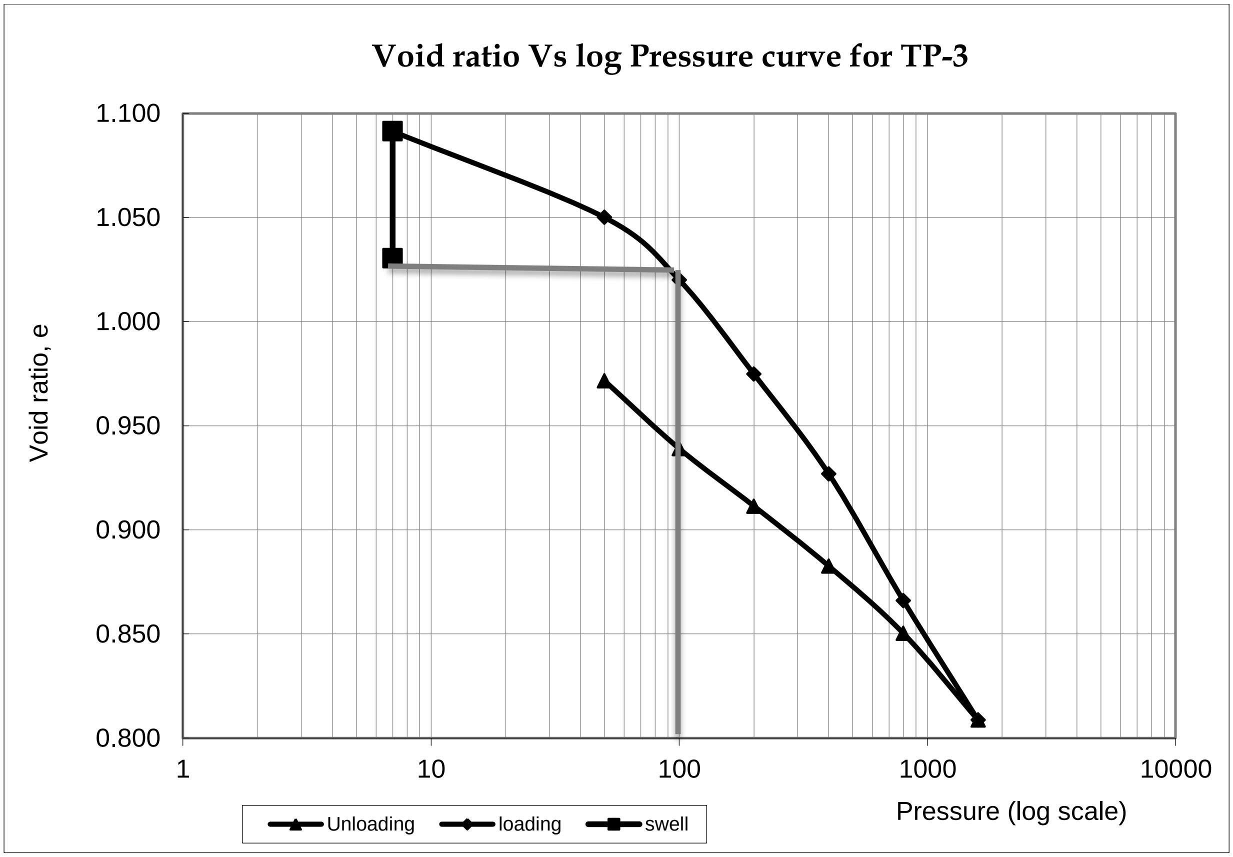


Fig C-1 Effective pressure versus void ratios for each load increment and decrement on semi-log scale

C.1.1 Compression index (C_C)

$$C_C = \frac{e_2 - e_1}{\log \sigma_2' - log\sigma_1'}$$

$$= \frac{1-0.809}{log1600-log195}$$

$$= 0.21$$

C.1.2.Swelling index (C_S)

$$C_S = \frac{e_2 - e_1}{\log \sigma_2' - log\sigma_1'}$$

$$= \frac{0.925 - 0.809}{log1600 - log200} = 0.13$$

C.1.3.Coefficient of consolidation C_V

***i.* Logarithm of time fitting method**

$$C_V = 0.197 \frac{{H_{dr}}^2}{t_{50}}$$

For 50kpa

Using the procedure presented previously in this document sample graphical procedure for the given loading is presented below:

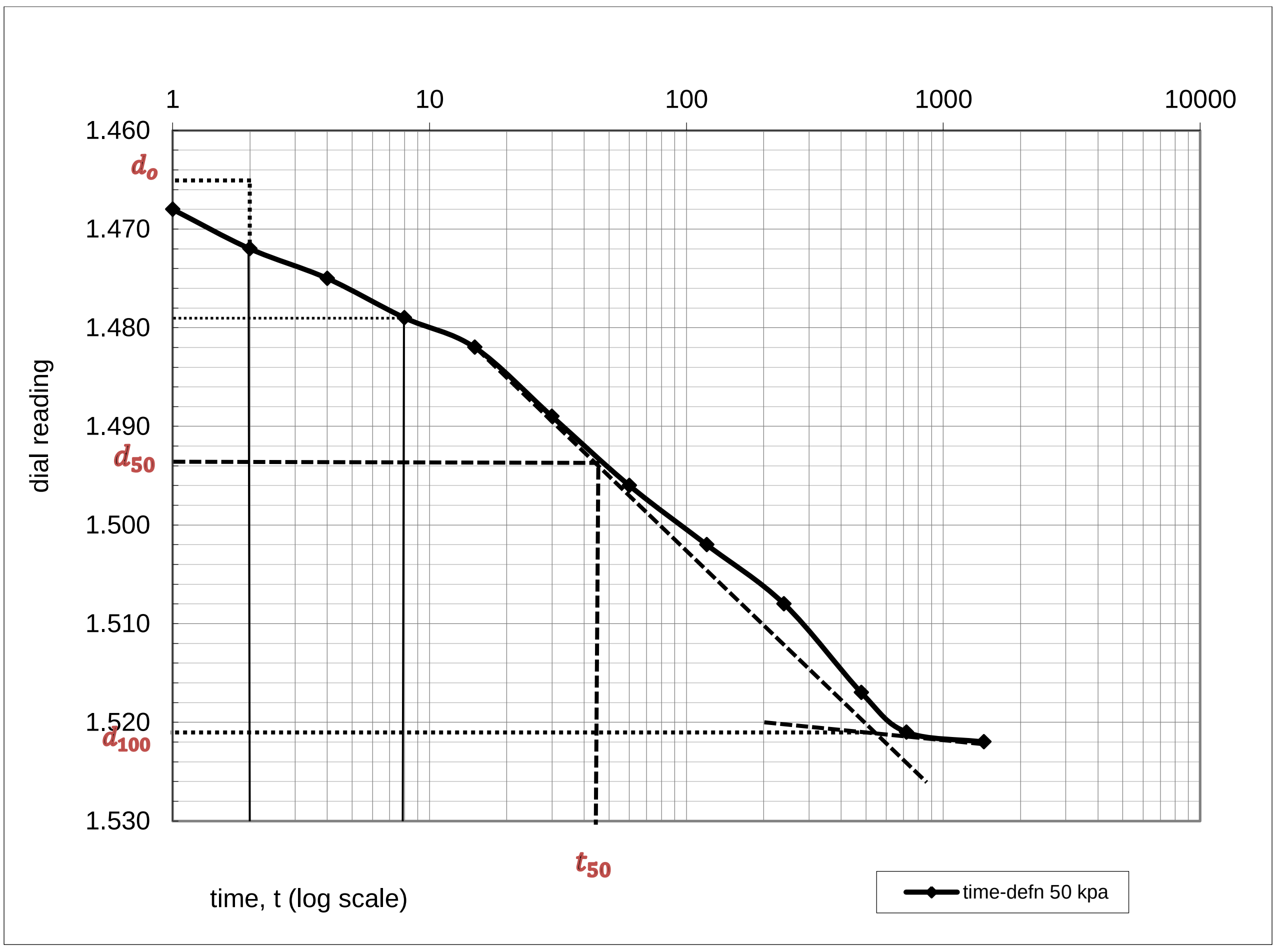


Fig C-2 Typical curve for determining C_V using log-time method

$$d_{50} = \frac{d_{100} + d_0}{2} = \frac{1.465 + 1.521}{2} = 1.493 \text{ mm}$$

From the figure we can read $t_{50} = 45min$

H_{dr} For double drainage is: $$H_{dr} = \frac{H_{AV}}{2} = \frac{H_0+H_f}{4} = \frac{H_0-d_{50}}{2}$$

Where H_{dr} =drainage path

d_{50}= Compression of sample up to 50% consolidation.

H_0= Initial height of the specimen=2cm

$$H_{dr} = \frac{2-0.1493}{2} = 0.925mm$$

$$C_V = \frac{0.197 * 0.925^2}{45} = 3.75 * 10^{-3}\frac{cm^2}{min}$$

In similar manner C_V for each incremental loading can be determined using consolidation data given in table C-4 and the results are summarized in table C-5 below:

Table C-4 summery of dial gauge readings for each incremental loading

Load (kpa)		Dial Gauge Reading, mm						
		5	50	100	200	400	800	1600
Time(min.)	$\sqrt{time}$							
0	0.00	1.40	1.400	1.522	1.662	1.949	2.420	3.020
0.13	0.36		1.420	1.560	1.742	2.050	2.470	3.084
0.25	0.50	-	1.460	1.568	1.750	2.060	2.495	3.090
0.50	0.71	-	1.464	1.572	1.760	2.072	2.510	3.100
1	1.00	-	1.468	1.578	1.770	2.088	2.520	3.120
2	1.41	-	1.472	1.584	1.788	2.104	2.532	3.138
4	2.00	-	1.475	1.593	1.799	2.122	2.552	3.162
8	2.83	-	1.479	1.598	1.818	2.158	2.579	3.210
15	3.87	-	1.482	1.605	1.832	2.185	2.612	3.256
30	5.48	-	1.489	1.616	1.855	2.215	2.658	3.295
60	7.75	-	1.496	1.626	1.878	2.243	2.719	3.348
120	10.95	-	1.502	1.636	1.897	2.281	2.796	3.415
240	15.49	-	1.508	1.646	1.918	2.325	2.897	3.467
480	21.91	-	1.517	1.655	1.938	2.369	2.996	3.548
720	26.83		1.521	1.658	1.942	2.399	3.012	3.569
1440	37.95	1.40	1.522	1.662	1.949	2.420	3.020	3.585

Table C-5 Summary of C_V using log of time method for each incremental loading for TP-3

load	50 kpa	100 kpa	200 kpa	400 kpa	800 kpa	1600 kpa
d_0	1.465	1.57	1.758	2.063	2.485	3.066
d_{100}	1.521	1.65	1.935	2.39	3.01	3.56
d_{50}	1.493	1.61	1.847	2.227	2.748	3.313
t_{50} (min)	45	25	25	50	77	38
H_{dr} (cm)	0.925	0.9195	0.908	0.889	0.863	0.834
$C_V * 10^{-3} \frac{cm^2}{min}$	3.75	6.66	6.5	3.11	1.91	3.61

For the desired loading range C_V can be determined by taking the average value

In similar manner one can determine C_V using log of time method for each incremental loading for all samples summarized below.

Table C-8 Summary of C_V using log of time method for each incremental loading for TP-1

load	50 kpa	100 kpa	200 kpa	400 kpa	800 kpa	1600 kpa
d_0	1.464	1.695	2.025	2.454	2.918	3.417
d_{100}	1.611	1.91	2.308	2.83	3.348	3.98
d_{50}	1.538	1.803	2.167	2.642	3.133	3.699
t_{50} (min)	19	55	38	40	45	30
H_{dr} (cm)	0.9231	0.91	0.89165	0.868	0.843	0.815
$C_V * 10^{-3} \frac{cm^2}{min}$	8.84	2.97	4.12	3.71	3.11	4.36

Table C-9 Summary of C_V using log of time method for each incremental loading

For TP-2

load	50 kpa	100 kpa	200 kpa	400 kpa	800 kpa	1600 kpa
d_0	0.708	0.971	1.324	2.018	2.873	3.82
d_{100}	0.866	1.21	1.83	2.65	3.705	4.88
d_{50}	0.787	1.091	1.577	2.334	3.289	4.35
t_{50} (min)	50	27.5	75	60	110	20
H_{dr} (cm)	0.96	0.945	0.921	0.883	0.836	0.783
$C_V * 10^{-3} \frac{cm^2}{min}$	3.63	6.4	2.23	2.56	1.25	6.03

Table C-10 Summary of C_V using log of time method for each incremental loading

For TP-4

load	50 kpa	100 kpa	200 kpa	400 kpa	800 kpa	1600 kpa
d_0	0.297	0.372	0.536	0.648	0.847	1.093
d_{100}	0.3305	0.5	0.586	0.708	0.931	1.232
d_{50}	0.314	0.436	0.561	0.678	0.889	1.163
t_{50} (min)	38	12	24	25	25	15
H_{dr} (cm)	0.984	0.978	0.972	0.966	0.956	0.942
$C_V * 10^{-3} \frac{cm^2}{min}$	5.02	15.7	7.76	7.35	7.2	11.65

ii. **Square root of time fitting method**

$$C_V = 0.848 \frac{{H_{dr}}^2}{t_{90}}$$

For 50kpa

Graphical procedure for the current loading is presented below:

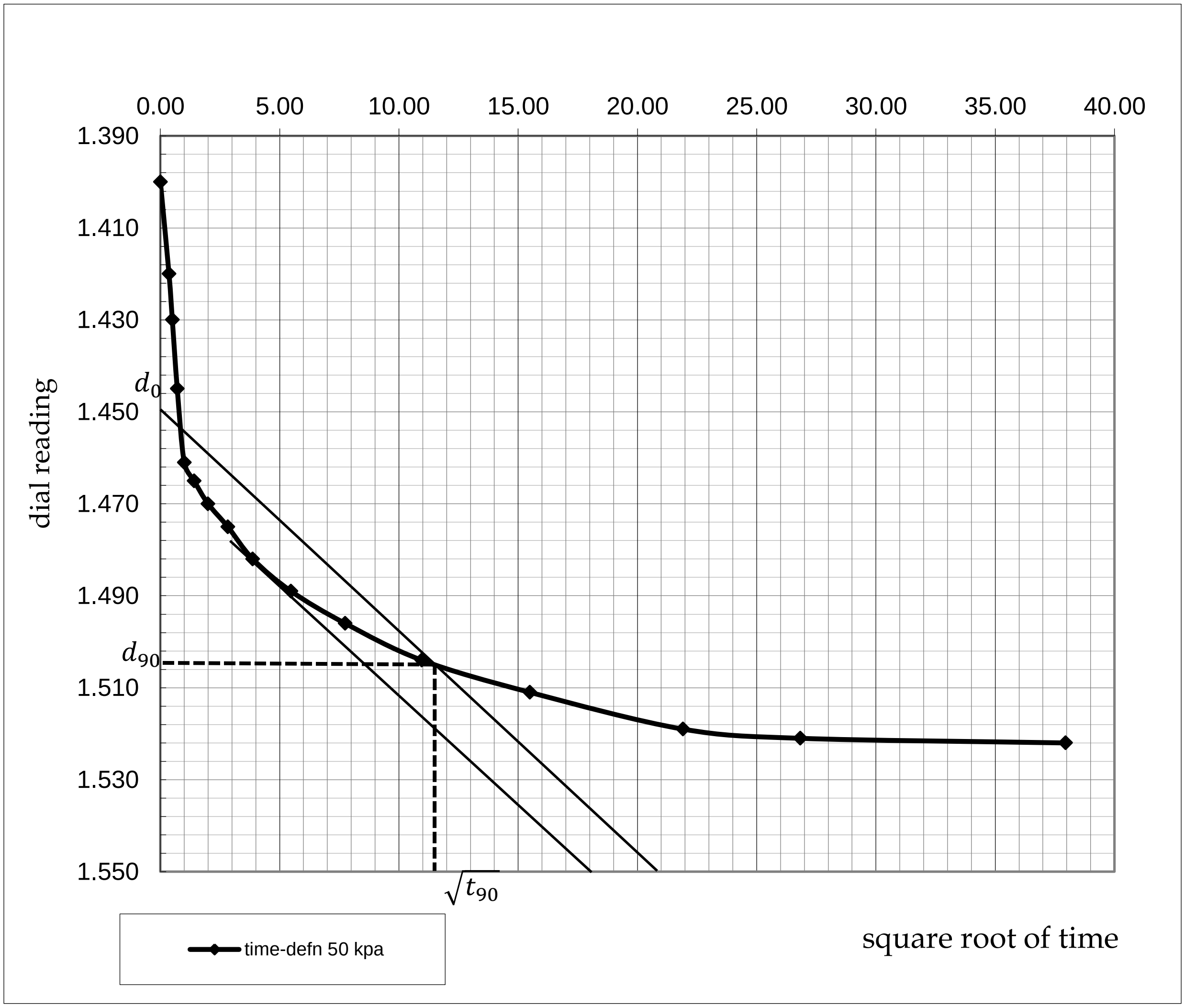


Fig C-3 typical curve for determination of C_V using square root of time method

From the above figure:

$$d_0 = 1.45mm, d_{90} = 1.506mm\ and\ \sqrt{t_{90}} = 13$$

$$d_{100} = \frac{1}{9}(d_{90} - d_0) + d_{90} = d_0 + 1.11(d_{90} - d_0)$$

$$= 1.45 + 1.11(1.506 - 1.45)$$

$$= 1.512mm$$

$$d_{50} = \frac{1.512 + 1.45}{2} = 1.481mm$$

$$H_{dr} = \frac{H_{AV}}{2} = \frac{H_0 + H_f}{4} = \frac{H_0 - d_{50}}{2}$$

$$= \frac{2 - 0.1481}{2} = 0.926mm$$

$$C_V = 0.848 * \frac{0.926^2}{169} = 4.3 * 10^{-3}\frac{cm^2}{min}$$

Table C-6 summary of C_V using square root of time method for each incremental loading for TP-3

Load (kpa)	**50**	**100**	**200**	**400**	**800**	**1600**
$\boldsymbol{d_0}$	1.45	1.568	1.755	2.05	2.43	3.1
$\boldsymbol{d_{90}}$	1.506	1.634	1.89	2.26	2.96	3.49
$\boldsymbol{d_{100}}$	1.512	1.641	1.905	2.283	3.02	3.53
$\boldsymbol{d_{50}}$	1.481	1.605	1.83	2.167	2.725	3.315
$\boldsymbol{H_{dr}\ (cm)}$	0.926	0.92	0.909	0.892	0.864	0.834
$\boldsymbol{\sqrt{t_{90}}}$	13	10	9.5	9	19	17
$\boldsymbol{t_{90}\ (min)}$	169	100	90.25	81	361	289
$\boldsymbol{C_V * 10^{-3}\frac{cm^2}{min}}$	4.30	7.18	7.76	8.33	1.75	2.04

For the desired loading range C_V can be determined by taking the average value.

In similar manner one can determine C_V using square root of time method for each incremental loading for all samples is summarized below.

Table C-11 summary of C_V using square root of time method for each incremental loading for TP-1

Load (kpa)	50	100	200	400	800	1600
d_0	1.44	1.665	1.975	2.43	2.87	3.41
d_{90}	1.55	1.8	2.175	2.72	3.23	3.78
d_{100}	1.56	1.815	2.197	2.752	3.27	3.821
d_{50}	1.5	1.74	2.086	2.591	3.07	3.821
H_{dr} (cm)	0.925	0.913	0.896	0.87	0.847	0.809
$\sqrt{t_{90}}$	6.5	7	6.5	10	10.8	8.8
t_{90} (min)	42.25	49	42.25	100	116.84	77.44
$C_V * 10^{-3} \frac{cm^2}{min}$	17.17	14.46	16.1	6.43	5.21	7.17

Table C-12 summary of C_V using square root of time method for each incremental loading for TP-2

Load (kpa)	50	100	200	400	800	1600
d_0	0.69	0.94	1.26	1.95	2.73	3.75
d_{90}	0.81	1.11	1.78	2.54	3.59	4.79
d_{100}	0.823	1.129	1.838	2.61	3.686	4.906
d_{50}	0.757	1.035	1.549	2.28	3.208	4.328
H_{dr} (cm)	0.962	0.948	0.923	0.886	0.84	0.784
$\sqrt{t_{90}}$	7	7	17	16	16	10.5
t_{90} (min)	49	49	289	256	256	110.25
$C_V * 10^{-3} \frac{cm^2}{min}$	16.02	15.55	2.5	2.6	2.34	4.72

Table C-13 summary of C_V using square root of time method for each incremental loading for TP-4

Load (kpa)	50	100	200	400	800	1600
d_0	1.45	1.568	1.755	2.05	2.43	3.1
d_{90}	1.506	1.634	1.89	2.26	2.96	3.49
d_{100}	1.512	1.641	1.905	2.283	3.02	3.53
d_{50}	1.481	1.605	1.83	2.167	2.725	3.315
H_{dr} (cm)	0.926	0.92	0.909	0.892	0.864	0.834
$\sqrt{t_{90}}$	13	10	9.5	9	19	17
t_{90} (min)	169	100	90.25	81	361	289
$C_V * 10^{-3} \frac{cm^2}{min}$	4.3	7.18	7.76	8.33	1.75	2.04

C.1.4. Coefficient of permeability (k)

$$K = C_V * \gamma_W * \frac{a_v}{1+e_0}$$

Where: C_V=coefficient of consolidation

a_V= coefficient of compressibility= $\frac{\Delta e}{\Delta \sigma_v}$

e=void ratio

γ_w=unit weight of water =9.81kN/m^3

For 50kpa loading

$$a_V = \frac{\Delta e}{\Delta \sigma_V'}$$

From the above given data for 50kpa loading

$$\Delta e = e_0 - e_{50kpa} = 1.03 - 1.018 = 0.012$$

$$\Delta \sigma_V' = 50 - 0 = 50kpa$$

$$a_V = \frac{0.012}{50} = 2.4 * 10^{-4} \frac{m^2}{kN}$$

The average value of C_V determined from logarithm of time and square root of time method for the given loading is:

$$C_V = \frac{4.3 + 3.75}{2} = 4.025 * 10^{-3} \frac{cm^2}{min}$$

$$K = 4.025 * 10^{-3} * 9.81 * 1.67 * 10^{-3} * \frac{2.4*10^{-4}}{1+1.03}$$

$$= 7.79 * 10^{-9} \frac{cm^2}{s}$$

Summary of results are given in table C-7 determined in similar manner as shown by the sample calculation.

Table C-7 summary of *K* for each incremental loading for TP-3

$\sigma_V'(kpa)$	$C_{Vavg} * 10^{-3} cm^2/min$	$a_V * 10^{-4} m^2/kN$	*e*	e_0	*k(cm/s)*	$K * 10^{-8} cm/s$
			1.03			
50	4.025	2.4	1.02	0.012	7.79E-09	0.78
100	6.92	2.8	1.00	0.014	1.56E-08	1.56
200	7.13	2.9	0.97	0.029	1.67E-08	1.67
400	5.72	2.4	0.93	0.048	1.11E-08	1.11
800	1.83	1.5	0.87	0.061	2.21E-09	0.22
1600	2.83	0.71	0.81	0.057	1.62E-09	0.16

B.1. Test Sample # 02

Sample Designation…………………………TP-2

Location……………………………….around prison

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type ……………………………..……Undisturbed

Swelling characteristics …………………. Expansive

Bulk (wet) density (γ_b) …………………..1.7g/cm³

Dry density (γ_b)……………………………1.3g/cm³

Natural moisture content (%)………….....66.5%

Specific gravity of the soil (Gs)…………...2.68

Weight of consolidation ring (gm.)……....37.7

Diameter of specimen (mm)……………….50

Area of the specimen (cm²)…………………19.63

Volume of sample (cm³)…………………….39.25

Weight of wet soil (gm.)……………………...62.4

Dry weight of the specimen after

Oven drying (gm.)……………………………41

Using the above data detail calculation is presented below:

6. Determination of height of solids, H_S, in the soil specimen:

$$H_S = \frac{M_S}{A * G_S * \rho_W}$$

$$H_S = \frac{41}{19.63 * 2.68 * 1} * 10$$

$$H_S = 7.8mm$$

7. Determination of initial height of voids, H_V:

$$H_V = H - H_S \text{ ,}$$

Where H= initial height of the specimen

$$= 20 - 7.8 \quad = 12.2mm$$

8. Determination of the initial void ratio, e_0, of the specimen:

$$e_0 = \frac{V_V}{V_S} = \frac{H_V}{H_S}\frac{A}{A} = \frac{H_V}{H_S}$$

$$= \frac{12.2}{7.8} = 1.56$$

9. For the first incremental loading σ_1 (total load/unit area of specimen), which causes deformation ΔH_1 , change in void ratio Δe_1:

$$\Delta e_1 = \frac{\Delta H_1}{H_S}$$

$$= \frac{20 - 19.724}{7.8} = 0.035$$

ΔH_1 is obtained from the initial and the final dial readings for the loading. At this time, the effective pressure on the specimen is $\sigma' = \sigma_1 = \sigma_1'$

10. The new void ratio, e_1, after consolidation caused by the pressure increment σ_1 is:

$$e_1 = e_0 - \Delta e_1$$

$$= 1.56 - 0.035 = 1.53$$

For the next loading, σ_2 (*note*: σ_2 equals the cumulative load per unit area of specimen), which causes additional deformation ΔH_2, the void ratio e_2 at the end of consolidation can be calculated as:

$$e_2 = e_1 - \frac{\Delta H_2}{H_S}$$

$$= 1.53 - \frac{19.724 - 19.378}{7.8} = 1.48$$

Note that, at this time, the effective pressure on the specimen is $\sigma' = \sigma_2 = \sigma_2'$.

Proceeding in similar manner, one can obtain the void ratios at the end of the consolidation for all load increments are presented on table B-3 using load dial readings and final specimen height summarized on table B-1 and B-2 for loading unloading respectively.

The effective pressures ($\sigma = \sigma'$) and the corresponding void ratios (e) at the end of consolidation are plotted on semi logarithmic graph as shown on figure B-1 below.

Table B-1 Dial gauge reading and final specimen height for each incremental loading

Load(kpa)	50		100		200		400		800		1600	
Time	Dial reading (mm)	final height(mm)	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0se	0.6	20	0.876	19.724	1.222	19.38	1.89	18.71	2.7	17.9	3.715	16.885
8se.	0.672	19.928	0.962	19.638	1.318	19.28	2.034	18.566	2.88	17.72	3.896	16.704
15sec.	0.708	19.892	0.97	19.63	1.326	19.27	2.042	18.558	2.886	17.714	3.912	16.688
30se.	0.716	19.884	0.982	19.618	1.334	19.27	2.052	18.548	2.898	17.702	3.926	16.674
1min	0.726	19.874	0.993	19.607	1.346	19.25	2.066	18.534	2.92	17.68	3.94	16.66
2min	0.736	19.864	1.01	19.59	1.362	19.24	2.084	18.516	2.934	17.666	3.985	16.615
4min	0.75	19.85	1.028	19.572	1.378	19.22	2.108	18.492	2.968	17.632	4.085	16.515
8min	0.764	19.836	1.049	19.551	1.4	19.2	2.15	18.45	2.995	17.605	4.15	16.45
15min	0.778	19.822	1.067	19.533	1.435	19.17	2.184	18.416	3.052	17.548	4.28	16.32
30min	0.796	19.804	1.095	19.505	1.489	19.11	2.247	18.353	3.13	17.47	4.458	16.142
1hr	0.812	19.788	1.118	19.482	1.55	19.05	2.332	18.268	3.25	17.35	4.675	15.925
2hr	0.83	19.77	1.142	19.458	1.647	18.95	2.438	18.162	3.415	17.185	4.815	15.785
4hr	0.852	19.748	1.165	19.435	1.76	18.84	2.529	18.071	3.565	17.035	4.98	15.62
8hr	0.867	19.733	1.199	19.401	1.828	18.77	2.62	17.98	3.665	16.935	5.096	15.504
12hr	0.87	19.73	1.218	19.382	1.8575	18.74	2.67	17.93	3.695	16.905	5.18	15.42
24hr	0.876	19.724	1.222	19.378	1.89	18.71	2.7	17.9	3.715	16.885	5.29	15.31

Table B-2 Dial gauge readings and final height for each decrement of unloading

Load(kpa)	1600		800		400		200		100		50	
Time	Dial reading (mm)	Final height H,(mm)	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0se	5.29	15.31	4.78	15.82	4.45	16.15	3.78	16.82	3.048	17.552	2.603	17.997
8se.	5.154	15.446	4.75	15.85	4.198	16.402	3.634	16.966	3.03	17.57	2.62	17.98
15sec.	5.15	15.45	4.748	15.852	4.196	16.404	3.632	16.968	3.029	17.571	2.616	17.984
30se.	5.148	15.452	4.726	15.874	4.194	16.406	3.631	16.969	3.028	17.572	2.614	17.986
1min	5.142	15.458	4.722	15.878	4.19	16.41	3.63	16.97	3.027	17.573	2.612	17.988
2min	5.134	15.466	4.712	15.888	4.182	16.418	3.628	16.972	3.026	17.574	2.608	17.992
4min	5.122	15.478	4.702	15.898	4.178	16.422	3.624	16.976	3.02	17.58	2.601	17.999
8min	5.103	15.497	4.686	15.914	4.166	16.434	3.616	16.984	3.015	17.585	2.584	18.016
15min	5.079	15.521	4.668	15.932	4.148	16.452	3.606	16.994	3.01	17.59	2.567	18.033
30min	5.042	15.558	4.634	15.966	4.124	16.476	3.583	17.017	2.995	17.605	2.546	18.054
1hr	4.99	15.61	4.58	16.02	4.082	16.518	3.563	17.037	2.974	17.626	2.512	18.088
2hr	4.928	15.672	4.508	16.092	4.026	16.574	3.518	17.082	2.938	17.662	2.43	18.17
4hr	4.844	15.756	4.481	16.119	3.936	16.664	3.47	17.13	2.886	17.714	2.324	18.276
8hr	4.827	15.773	4.462	16.138	3.88	16.72	3.374	17.226	2.789	17.811	2.156	18.444
12hr	4.81	15.79	4.451	16.149	3.823	16.777	3.278	17.322	2.787	17.813	2.077	18.523
24hr	4.78	15.82	4.45	16.15	3.78	16.82	3.048	17.552	2.603	17.997	1.838	18.762

Table B-3 summery of void ratios at the end of each incremental loading and unloading

Applied pressure P (kpa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	0.300	0.00	20.000	12.20	1.56
7	1.200	0.90	20.900	13.10	1.68
50	0.876	0.28	19.724	11.92	1.53
100	1.222	0.35	19.378	11.58	1.48
200	1.890	0.67	18.710	10.91	1.40
400	2.700	0.81	17.900	10.10	1.29
800	3.715	1.02	16.885	9.09	1.16
1600	5.290	1.58	15.310	7.51	0.96
Unloading					
1600	5.290	1.58	15.820	7.51	0.96
800	4.450	0.44	16.150	8.35	1.07
400	3.780	-0.23	16.820	9.02	1.16
200	3.048	-0.96	17.552	9.75	1.23
100	2.603	-1.41	17.997	10.20	1.31
50	2.603	-1.41	18.762	10.96	1.41

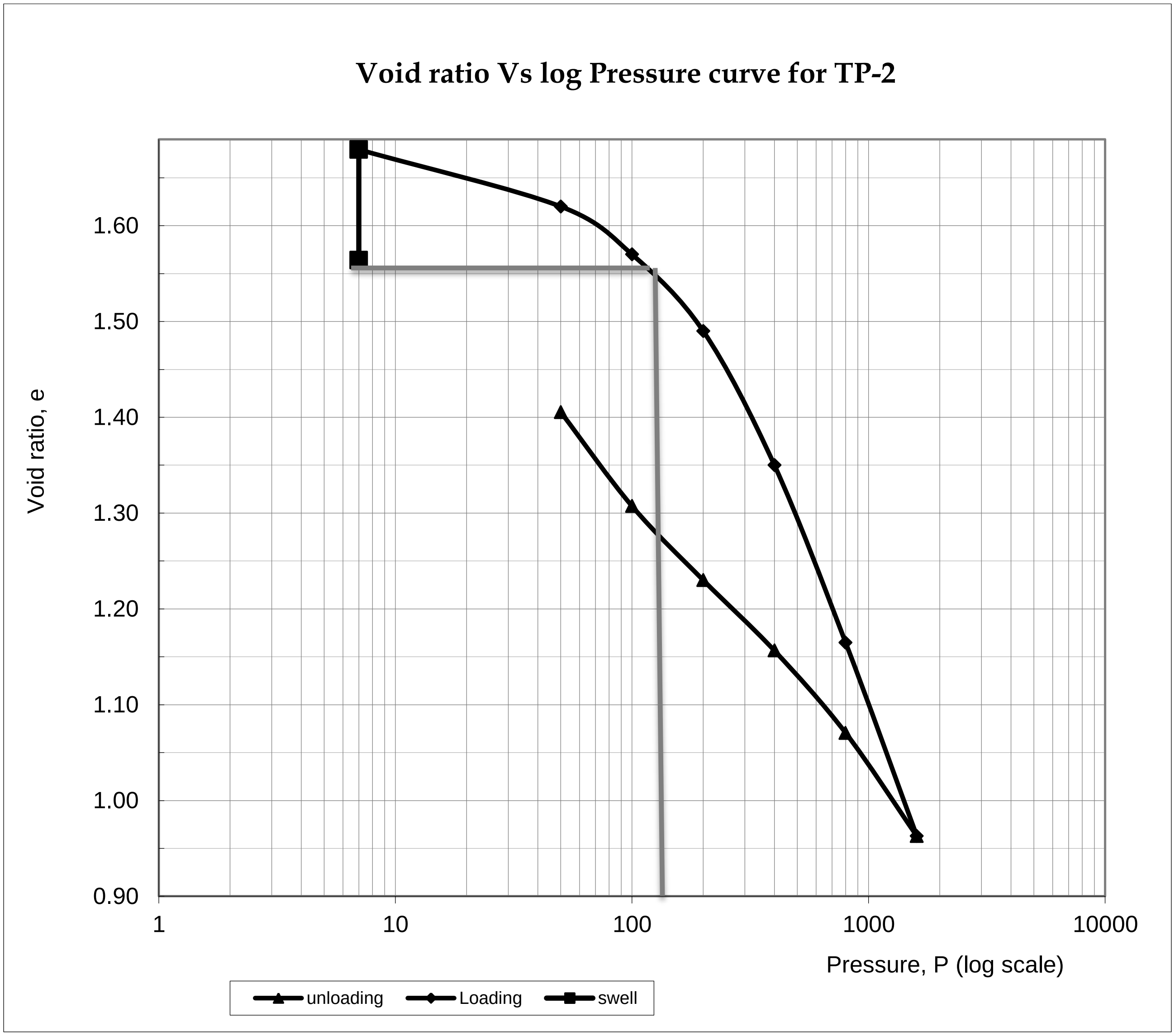


Fig B-1 Effective pressure versus void ratios for each load increment and decrement on semi-log scale

D.1. Test Sample # 01

Sample Designation…………………………TP-01

Location…………………………………around stadium

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type …………………………..……Undisturbed

Swelling characteristics …………………. Expansive

Bulk (wet) density (γ_b) …………………..1.9g/cm^3

Dry density (γ_b)……………………………1.48g/cm^3

Natural moisture content (%)…………....29.4%

Specific gravity of the soil (Gs)…………...2.66

Weight of consolidation ring (gm.)……....37.7

Diameter of specimen (mm)……………….50

Area of the specimen (cm^2)…………………19.63

Volume of sample (cm^3)…………………….39.25

Weight of wet soil (gm.)……………………...79.5

Dry weight of the specimen after

Oven drying (gm.)……………………………50.5

Proceeding in similar manner as in the previous section, one can obtain the void ratios at the end of the consolidation for all load increments are presented on table D-2 using load dial readings and final specimen height summarized on table D-1.

The effective pressures ($\sigma = \sigma'$) and the corresponding void ratios (e) at the end of consolidation are plotted on semi logarithmic graph as shown on figure D-1 below.

Table D-1 Dial gauge reading and final specimen height for each incremental loading

Load(kpa)	50		100		200		400		800		1600	
Time (min)	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0	1.415	20	1.62	19.795	1.926	19.489	2.326	19.089	2.85	18.565	3.368	18.047
0.13	1.435	19.98	1.665	19.75	2.01	19.405	2.42	18.995	2.88	18.535	3.44	17.975
0.25	1.452	19.963	1.692	19.723	2.02	19.395	2.462	18.953	2.922	18.493	3.462	17.953
0.5	1.463	19.952	1.702	19.713	2.03	19.385	2.474	18.941	2.94	18.475	3.476	17.939
1	1.476	19.939	1.716	19.699	2.052	19.363	2.482	18.933	2.95	18.465	3.492	17.923
2	1.49	19.925	1.722	19.693	2.07	19.345	2.504	18.911	2.968	18.447	3.506	17.909
4	1.502	19.913	1.735	19.68	2.08	19.335	2.53	18.885	2.986	18.429	3.543	17.872
8	1.516	19.899	1.749	19.666	2.109	19.306	2.554	18.861	3.018	18.397	3.595	17.82
15	1.532	19.883	1.758	19.657	2.136	19.279	2.584	18.831	3.052	18.363	3.635	17.78
30	1.548	19.867	1.775	19.64	2.158	19.257	2.624	18.791	3.1	18.315	3.701	17.714
60	1.567	19.848	1.806	19.609	2.192	19.223	2.674	18.741	3.162	18.253	3.756	17.659
120	1.586	19.829	1.832	19.583	2.232	19.183	2.732	18.683	3.238	18.177	3.815	17.6
240	1.598	19.817	1.862	19.553	2.2665	19.1485	2.782	18.633	3.296	18.119	3.886	17.529
480	1.61	19.805	1.905	19.51	2.304	19.111	2.826	18.589	3.35	18.065	3.955	17.46
720	1.6155	19.7995	1.917	19.498	2.315	19.1	2.838	18.577	3.359	18.056	3.985	17.43
1440	1.62	19.795	1.926	19.489	2.326	19.089	2.85	18.565	3.368	18.047	4.01	17.405

Table B-2 summery of void ratios at the end of each incremental loading

Applied pressure P (kpa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	0.800	0.00	20.00	10.33	1.07
7	1.415	0.62	20.62	10.95	1.13
50	1.620	0.20	19.80	10.13	1.05
100	1.926	0.31	19.49	9.82	1.02
200	2.326	0.40	19.09	9.42	0.97
400	2.850	0.52	18.57	8.90	0.92
800	3.368	0.52	18.05	8.38	0.87
1600	4.010	0.64	17.41	7.74	0.80

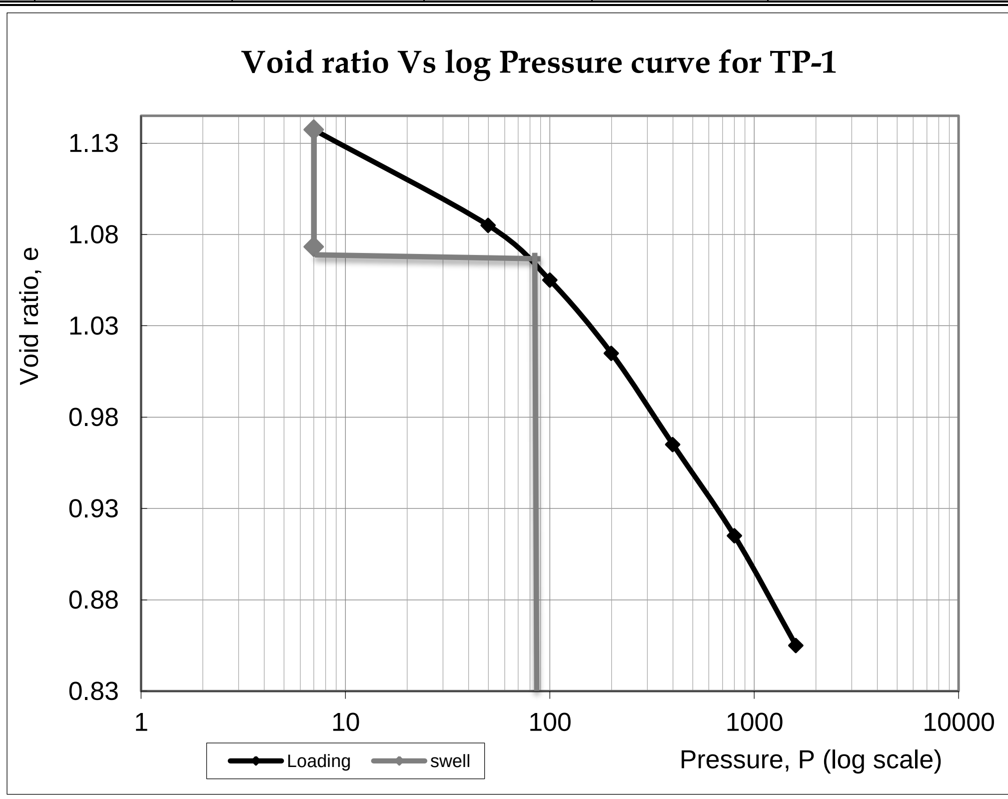


Fig B-1 Effective pressure versus void ratios for each load increment on semi-log scale

A.1. Test Sample # 04

Sample Designation…………………………TP-04

Location…………………………………around Negash Lodge

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type ……………………………..……Undisturbed

Swelling characteristics …………………. Non-Expansive

Bulk (wet) density (γ_b) …………………..1.9g/cm^3

Dry density (γ_b)……………………………1.48g/cm^3

Natural moisture content (%)…………......35.4%

Specific gravity of the soil (Gs)…………...2.76

Weight of consolidation ring (gm.)……....37.7

Diameter of specimen (mm)……………….50

Area of the specimen (cm^2)…………………19.63

Volume of sample (cm^3)…………………….39.25

Weight of wet soil (gm.)……………………...77.7

Dry weight of the specimen after

Oven drying (gm.)……………………………56.3

Proceeding in similar manner as in the previous section, one can obtain the void ratios at the end of the consolidation for all load increments are presented on table A-2 using load dial readings and final specimen height summarized on table A-1.

The effective pressures ($\sigma = \sigma'$) and the corresponding void ratios (e) at the end of consolidation are plotted on semi logarithmic graph as shown on figure A-1 below.

Table A-1 Dial gauge reading and final specimen height for each incremental loading

Load(kpa)	50		100		200		400		800		1600	
Time	Dial reading (mm)	final height, H (mm)	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H	Dial reading	H
0se	0.255	20	0.335	19.92	0.512	19.743	0.59	19.665	0.712	19.543	0.944	19.311
8se.	0.26	19.995	0.386	19.869	0.526	19.729	0.63	19.625	0.82	19.435	1.05	19.205
15sec.	0.28	19.975	0.39	19.865	0.532	19.723	0.64	19.615	0.83	19.425	1.064	19.191
30se.	0.29	19.965	0.394	19.861	0.536	19.719	0.646	19.609	0.844	19.411	1.082	19.173
1min	0.295	19.96	0.396	19.859	0.54	19.715	0.652	19.603	0.858	19.397	1.108	19.147
2min	0.301	19.954	0.401	19.854	0.545	19.71	0.66	19.595	0.861	19.394	1.118	19.137
4min	0.303	19.952	0.409	19.846	0.549	19.706	0.666	19.589	0.868	19.387	1.125	19.13
8min	0.305	19.95	0.43	19.825	0.554	19.701	0.672	19.583	0.875	19.38	1.143	19.112
15min	0.307	19.948	0.46	19.795	0.558	19.697	0.675	19.58	0.881	19.374	1.165	19.09
30min	0.312	19.943	0.48	19.775	0.563	19.692	0.68	19.575	0.895	19.36	1.208	19.047
1hr	0.318	19.937	0.492	19.763	0.567	19.688	0.688	19.567	0.916	19.339	1.234	19.021
2hr	0.325	19.93	0.5014	19.7536	0.572	19.683	0.694	19.561	0.924	19.331	1.242	19.013
4hr	0.329	19.926	0.504	19.751	0.579	19.676	0.699	19.556	0.933	19.322	1.25	19.005
8hr	0.332	19.923	0.506	19.749	0.586	19.669	0.706	19.549	0.939	19.316	1.258	18.997
12hr	0.333	19.922	0.508	19.747	0.588	19.667	0.71	19.545	0.941	19.314	1.262	18.993
24hr	0.335	19.92	0.512	19.743	0.59	19.665	0.712	19.543	0.944	19.311	1.268	18.987

Table A-2 summery of void ratios at the end of each incremental loading

Applied pressure P (kpa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, E
Loading					
7	0.000	0.00	20.00	9.61	0.92
50	0.335	0.08	19.92	9.53	0.92
100	0.512	0.14	19.78	9.39	0.91
200	0.590	0.13	19.65	9.26	0.90
400	0.712	0.11	19.54	9.15	0.88
800	0.944	0.23	19.31	8.92	0.86
1600	1.268	0.32	18.99	8.60	0.83

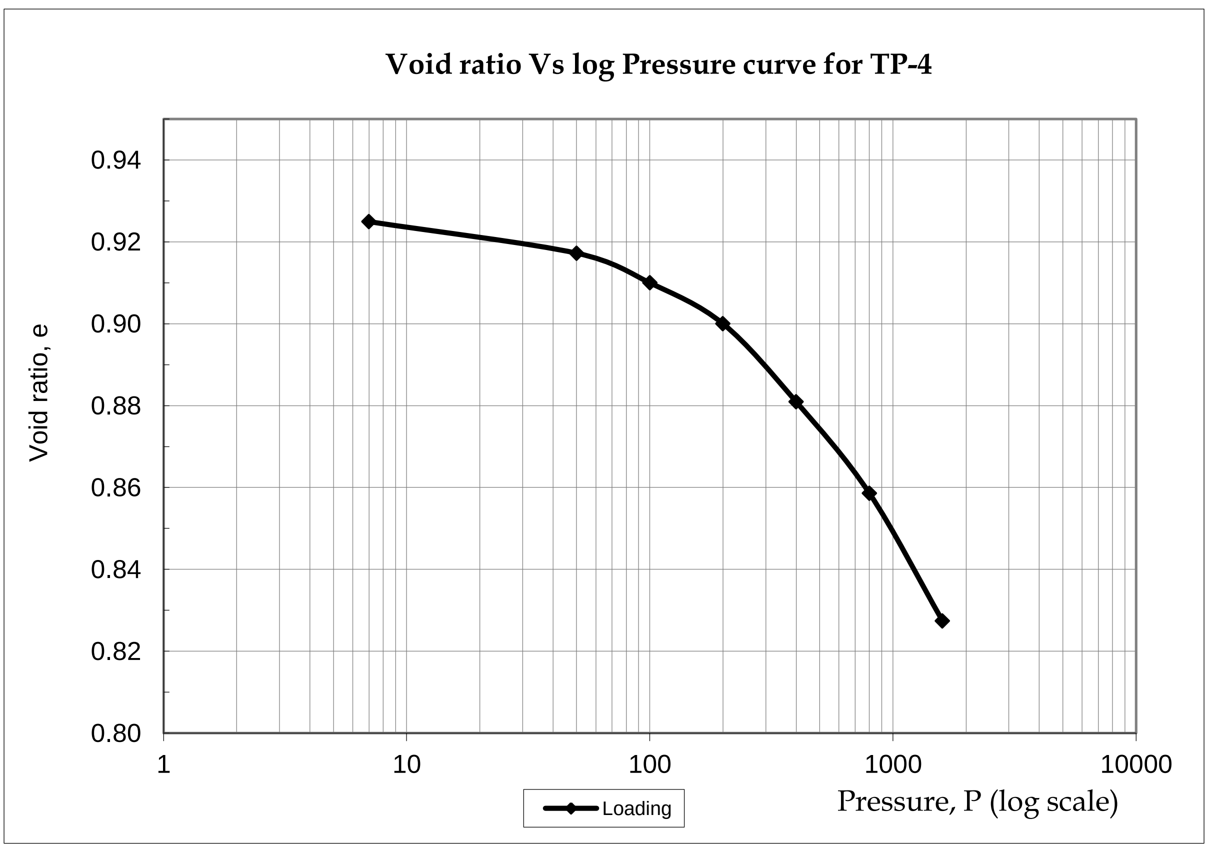


Fig A-1 Effective pressure versus void ratios for each load increment on semi-log scale

Appendix-E

(Meteorological data)

Table D-1 summary of monthly maximum temperature in ^{0}C

Region –Oromia latitude longitude altitude

Station –Woliso (Giyon) 8^{0}33'00'' 37^{0}59'00'' 2058m

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2008	27.2	27.7	29.6	28.2	25.9	23.2	20.9	21.1	23.4	25.1	24.4	25.7
2009	26.1	27.9	29.4	28.4	28.2	26.3	22.3	21.8	23.8	24.9	26.1	25.2
2010	26.9	26.8	27.2	27.1	25.4	23.7	21.3	21.9	23.9	26.7	26.2	25.9
2011	26.6	28.5	27.3	28.9	26.8	23.5	22.2	21.5	22.8	25.9	25.7	25.6
2012	27.4	28.3	29.1	27.1	27.9	23.5	21.2	21.8	23.0	25.9	26.8	26.4

Table D-2 summary of monthly minimum temperature in ^{0}C

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2008	14.1	14.3	15.0	14.4	12.7	13.1	12.2	12.8	11.8	13.0	12.7	13.0
2009	13.2	14.8	15.1	14.0	12.8	12.3	13.2	13.2	12.5	12.9	13.1	14.5
2010	13.7	15.0	15.0	14.9	14.7	13.7	13.1	13.4	12.5	13.1	13.7	13.0
2011	13.8	13.8	14.6	14.7	13.8	13.6	13.1	13.1	12.5	13.0	13.2	12.8
2012	13.6	14.0	15.2	14.4	13.9	13.1	12.8	12.8	12.2	12.6	12.9	12.8

Table D-3 summary of monthly rainfall data (mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2008	0.0	1.1	5.4	43.3	127.3	177.6	326.2	262.8	159.6	50.6	88.3	0.0
2009	16.7	1.6	42.8	37.9	64.4	77.9	234.1	290.4	107.2	83.9	x	x
2010	8.7	48.7	28.1	88.3	x	228.9	306.7	359.1	147.8	0.0	0.0	28.2
2011	x	1.2	35.7	x	132.2	188.9	234.7	283.7	158.8	0.0	30.0	0.0
2012	0.0	0.0	33.3	78.4	60.6	173.7	186.8	195.6	215.6	1.8	3.0	6.2

DECLARATION

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

Name: Biruk Haile

Signature: ______________

Place: Addis Ababa Institute of Technology,

Addis Ababa University,

Addis Ababa.

Date of submission: April, 2015