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GEOTECHNICAL ENGINEERING STREAM

INVESTIGATION ON THE ENGINEERING PROPERTIES OF SOILS IN FICHE TOWN, ETHIOPIA

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (*Geotechnical Engineering)*

BY

HABTAMU GIZACHEW

October, 2016

Jimma, Ethiopia

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HABTAMU GIZACHEW

Advisor: Prof. Emer T.Quezon

Co-advisor: Eng. Jemal Jibril (MSc.)

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DECLARATION

I, the undersigned, declare that this thesis entitled "INVESTIGATION ON THE ENGINEERING PROPERTIES OF SOILS IN FICHE SALALE TOWN, ETHIOPIA", is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for thesis have been dually acknowledged.

Candidate:

Mr. Habtamu Gizachew

Signature____

As Master Research Advisors, we here by certify that we have read and evaluate this MSc. research prepared under our guidance, by Mr. Habtamu Gizachew entitled "INVESTIGATING ON THE ENGINEERING PROPERTIES OF SOILS IN FICHE SALALE TOWN, ETHIOPIA"

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

Prof. Emer T.Quezon	__________________________	______________
(Advisor)	Signature	Date
Eng. Jemal Jibril (MSc.)	___________________	____________________
(Co-advisor)	Signature	Date

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ABSTRACT

Fiche is a town in central Ethiopia, located in North Shewa zone of the Oromia Region. The objective of this research is the investigation of the engineering properties of soils found in Fiche town. Disturbed and undisturbed soil samples were collected from Fiche town from ten test pits at 1.5m and 3m depth in order to classify and determine the necessary engineering properties of soils in area.

Insitu soil properties show that the natural moisture content for the area under study ranges from 20.01% to 58.89 %. Specific gravity is between 2.62 and 2.88. The specific gravity is above 2.62 indicating that inorganic soils are dominant as most of organic soils contain a value of less than 2.40. The grain size analysis of the area under study shows the soil contains gravel 0-25.62%, sand 1.44%-39.73%, silt 26.29%-55.54% and clay in between 8.67% to 67.51%. This shows that the dominant soil types in the area are silt and clay.

The liquid limit of the soils is in the range of 31%-79%. Plastic limit of the soils ranges in between 16% to 42%. Plasticity index for the area under study ranges between 11%-46%. And the free swell result ranges from 35-99%.

Soil classification for the area under study is made by both USCS and AASHTO. Soils classification by USCS shows that the soil contains around 41% CH, 18% CL, 18% MH, 14% ML, 4.5% SM and 4.5% SC and AASHTO classification system shows the soils are classified in either of A-6 and A-7.

From the compaction test results the maximum dry density (MDD) of Fiche town soil ranges from 1.35g/cm^3 to 1.42g/cm^3 and the optimum moisture content ranges 28% to 36%.

The unconfined compressive strength test of soils has a value of q_u *between 77kPa to 234 kPa and the amount of* ***cohesion*** *ranges in between 38 kPa to 117 kPa. Consistency is also determined based on the UCS result showing that the soil is medium, stiff or very stiff.*

Consolidation test was conducted on four different types of soils which are taken based on the classification result. The range of compression index is in between 0.20-0.38 and the pre-consolidation pressure values ranges between 85 kPa to 285 kPa. The amount of over consolidation ratio of the soil found in the area under study is determined and it is found between 1.749 to5.45. Since the over consolidation ratio is greater than one, the soil is over consolidated in its natural state.

The average values of various tests done at different parts of countries, i.e., Sieve analysis, Liquid Limit, Plastic Index and specific gravities showing different properties. Fiche soils show medium plasticity (clay content) as compared to other towns. The data indicate that there is a considerable similarity of Fiche soils within the physical properties of Addis Ababa and Debre Markos town soils.

Some part of the town especially areas at the center of the town i.e around bus station and eastern part of the town is covered dominantly by rock. Thus additional detailed geological study on those areas is highly recommended.

TABLE OF CONTENTS

Contents **Page**

LIST OF TABLES

LIST OF FIGURES

SYMBOLS AND ABBREVIATIONS

AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing Materials standard
Cc	Compression index
CL	Lean clay
Cr	Recompression index
Cv	Coefficient of consolidation
e	Void ratio
Es	Modulus of compressibility
K	Modulus of Permeability
LL	Liquid limit
MDD	Maximum dry density
MH	Inorganic Elastic silt
ML	Inorganic Silt
NMC	Natural moisture content
OMC	Optimum moisture content
OCR	Over-consolidation ratio
Pc	Pre-consolidation pressure
Po	Over burden pressure
PI	Plastic Index
PL	Plastic limit
SM	Silty sand
TP	Test pit
USCS	Unified Soil Classification System
γ_d	Dry unit weight
γ_w	Wet unit weight

CHAPTER ONE
INTRODUCTION

1.1 Background

A geotechnical engineer determines and designs the type of foundation, earthwork, and/or pavement sub grades required for the intended man-made structures to be built. Foundations are designed and constructed for the structures of various sizes such as high-rise buildings, bridges, medium to large commercial buildings, and smaller structures where the soil conditions do not allow code-based design [32].

Investigation of the underground 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 [6].

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 borrow material, environmental damage to the site, post construction remedial work, and even failure of a structure and subsequent litigation [32]. Therefore, to obtained 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.

In a country like Ethiopia which is developing at high growth rate and which needs many construction works in the future, geotechnical investigation on the engineering property of soil is very essential. Because these data are very important for civil engineers in preliminary design and in designing foundation, pavement, retaining structures, etc for future construction projects in the country.

Many researches were done and there are ongoing researches in most big cities of the country like Addis Ababa, Bahir Dar, Mekele, Hawassa, Adama, Ambo etc [32] [11]

Fiche is a town in central Ethiopia, located in North Shewa zone of the Oromia Region, about 100 kilometers north of Addis Ababa on the paved highway to Gojam. It is among the fastest growing town in the country and there is a big volume of construction works due to its location and near distance from Addis Ababa.

Any engineering structure, whether they built above or below the ground surface use soils and rocks as the basic foundation and construction materials. Unlike manmade materials, the properties of these soils and rocks are highly variable and a function of the complex natural processes that occurred in the geologic past. As a consequence, engineers are faced with the challenge of using soils and rocks available near the project site, whose properties are often unknown and of variable quality [11].

Investigation of the underground conditions at a site is prerequisite to the economical design of the sub structured 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 [9].

Fiche town has some industrial facilities like factories (small and large scale), warehouses, resorts and lodges. There is still a great potential for investment expansion in the zone mainly as a result of its proximity to the capital and the main road from Addis Ababa to Gojam.

The soil profile of the town varies from place to place; mainly reddish to white brown; dark to light gray soils and black soils cover different parts of the town. Hence it is necessary to investigate the soils found in the town.

1.2 Statement of Problem

Fiche is the capital of North Shewa zone, which is developing rapidly. Rapid urbanization in the city area has led to an increased interest in the engineering behavior of the soils which are present within the city area. Geotechnical information of the subsoil in an urban area is important for various civil engineering works. Non-availability of the proper geotechnical information of the subsoil makes foundation and engineering works expensive, difficult and sometimes hazardous [22]. Therefore, the aim of this research is to investigate the engineering properties of soils in Fiche Salale town.

1.3 Objective of the Study

1.3.1 General Objective

- The general objective of this research is to investigate the engineering properties of the soils found in Fiche town.

1.3.2 Specific objectives

The specific objectives of this research are:

i. To determine the physical properties and to classify the soils in the study area according to the standard.

ii. To investigate the shear strength of the soils and to determine the consolidation characteristic of soils in the study area.

iii. To compare properties of soils in the study area with other parts of the country's.

1.4. Research Questions

This research study is aimed to formulate and answer the following questions:

1. What are the physical properties of soils in Fiche town and its classification?
2. What are the shear strength and the consolidation characteristic of soils in the study area?
3. What are the properties of the soil in the study area, as compared to the other towns in Ethiopia?

1.5. Significance of the study

The study is significant to obtain information on type, characteristics and distribution of a soil in the study area and to the country. The information which will be generated from the study will be used in the decision – making process for various concerned bodies 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 and for further studies on related works.

1.6. Scope and Limitation of the Study Area

The scope of this study was limited to investigate the index properties, unconfined compression strength and consolidation characteristic. To investigate the engineering properties of Fiche town, ten sampling areas were selected following to reconnaissance survey of the area. Excavations of pits were done at the depth of 1.5m and 3m to take samples. Due to the budget constraint, the depth of investigation in this research was limited to the maximum depth of three meters.

1.7. Structure of the Thesis

The thesis has been divided in to five chapters. The first chapter is the introductory part which includes the general background, statement of the problem, research questions, objective, scope and significance of the study. Chapter two deals with, a brief literature review, including soil formation and deposit, mineralogy of soils, soil particle size and shape, identification and classification of soils. In the third chapter, methodology, description of the sampling area is covered which includes geology, climate, topography and soil characteristics. The fourth chapter includes tests and results of engineering and index properties of the soil, comparison of test results with previous test results conducted by other researchers on different areas. In chapter five conclusions from test results are drawn and recommendation is presented. References and appendices are attached at the end of the thesis.

CHAPTER TWO

LITERATURE REVIEW

2.1. Soil Formation And Soil Deposits

To a civil engineer, the term 'soil' means, the loose unconsolidated inorganic material on the earth's crust produced by the disintegration of rocks, overlying hard rock with or without organic matter [9]. In engineering, soils are considered to include all organic and inorganic earth materials occurring in the zone overlying the rock crust. They are usually non-homogeneous porous material whose engineering behavior is greatly affected by changes in moisture content and density [28]. In order to determine the complexity behavior of soil a detailed geotechnical investigation is required.

A site investigation in one or another is always required for any engineering or building structure. The investigation may range in scope from a simple examination of the surface soils with or without a few shallow trial pits, to a considerable depth below the surface by means of boreholes and in-situ laboratory tests on the materials encountered. The extent of the work depends on the importance and foundation arrangement of the structure, the complexity of soil conditions, and the information which may be available on the behavior of existing foundations on similar soils [30].

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition. The properties of the soil materials depend upon the properties of the rocks from which they are derived [22]

The variety of soil materials encountered in engineering problems is almost limitless, ranging from hard, dense, large pieces of rock through to gravel, sand, silt, and clay to organic deposits of soft compressible peat. To compound the complexity, all of these materials may occur over a range of densities and water contents. At any given site, a number of different soil types may be present, and the composition may vary over intervals of a little as a few inches [18].

It has long been appreciated that the engineering classification of soils is greatly facilitated by taking into account the soil-forming processes by which nature has created the various types of soil conditions. Similar combinations of soil-forming processes in different parts of the world have been found to lead to materials of similar index properties and similar engineering characteristics [27]. The main factors affecting the formations of soil are: Parent materials i.e. geology of the area, topography and drainage, climate and vegetation cover.

2.1.1. Parent materials

There are two main variables in parent materials that affect soils: grain size and composition. Grain size is the main determinant of soil texture. Texture influences the soil structure, consistency, caution exchange capacity, profile drainage, moisture retaining capacity and organic content [22].

2.1.2. Topography and Drainage

Topography has a major influence on drainage characteristics which in turn is known to have major effect on soil mineralogy. Its control over soil properties is particularly strong in tropical environment reflecting the importance of lateral movement of water and soil materials [27].

2.2 Mineralogy of Soils

Mineral particles are inorganic materials derived from rocks and minerals. They are extremely variable in size and composition.

Primary minerals: present in original rock from which soil is formed. These occur predominantly in sand and silt fractions, and are weathering resistant (quartz, feldspars).

Secondary minerals: formed by decomposition of primary minerals, and their subsequent weathering and decomposition into new ones (clay minerals).

Humus or organic matter: means decomposed organic materials.

2.2.1 Clay mineralogy

The soil cover of the area under study is mainly covered by clay and silts. Thus it is essential to review and study the mineralogy of those soils. The coarse-grained oils generally contain the mineral quartz and feldspar. These minerals are strong and electrically inert. The behavior of such soils does not depend upon the nature of the mineral present. The behavior of fine-grained soils, on the other hand, depends to a large extent on the nature and characteristics of the minerals present. The most significant properties of clay depend upon the type of mineral. The crystalline minerals whose surface activity is high are clay minerals. These clay minerals impart cohesion and plasticity [2].

The main groups of clay crystalline materials that make up clays are the minerals kaolinite, illite and montmorillonite.

2.2.1.1 Kaolinite

Kalonite has a structural unit made up of alumina sheets joined to silca sheet and is symbolized as indicated in Fig (2.1a). Kalonite consists of many such layers stacked one on top of the other as shown in fig (2.1b).

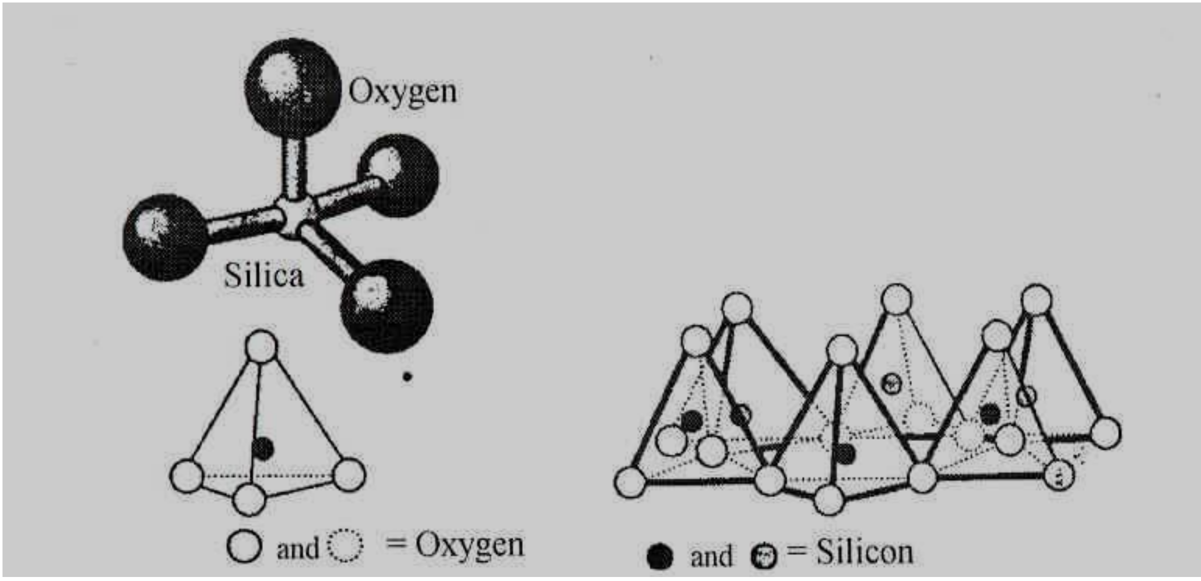
The bond that exists between layers is tight and hence it is difficult to separate the layers. As a result Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently Kaolinite shows little swelling on wetting [28].

2.2.1.2 Illite

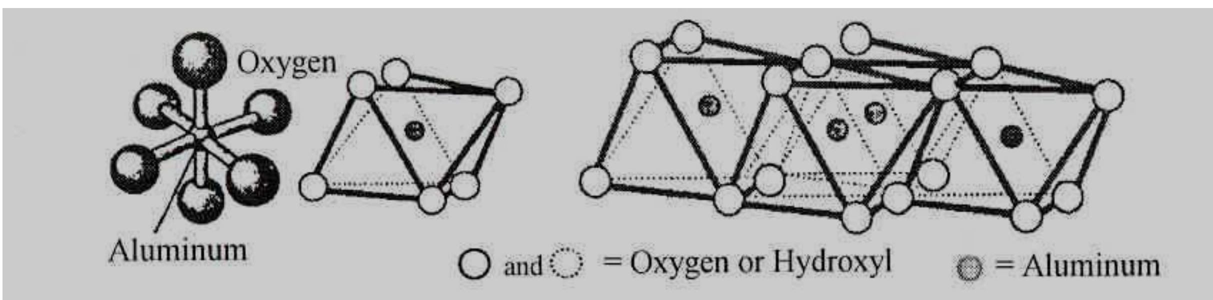
Illite has a basic structure similar to that of montmorillonite. However, the basic illite units are bonded together by potassium ions which are non-exchangeable. Because of this, the illite units are reasonably stable and so that mineral swells much less than montmorillonite.

2.2.1.3 Montmorillonite

These are highly expansive and create major engineering problems. Montmorillonite have a similar structure to illites, however the layers are held together by weak Van der Waals forces and exchangeable ions. Water can easily enter the bond and create swelling. The three groups are all structured in crystal layers. The physical arrangement of the different layers and the method used to bond individual layers of the structural units produces the distinct mineralogy [2].



(a) (b)



(c) **(d)**

Fig. 2.1 (a) A silica tetrahedron, (b) Silica sheets, (c) An Aluminum octahedron, and (d) aluminum sheets.

Source: [2]

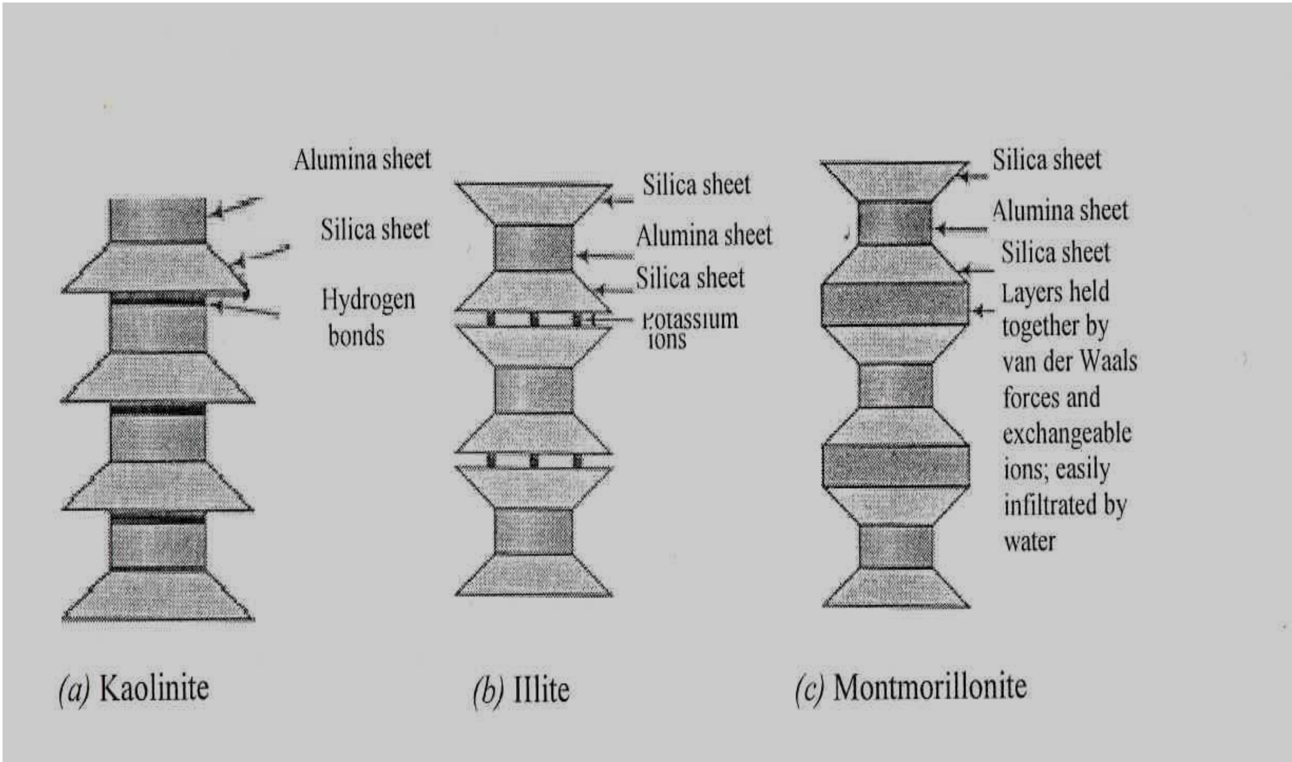


Fig. 2.2 Structure of Kaolinite, illite, and montmorillonite

Source: [2]

2.3 Soil Particle Size and Shape

The size of particles may range from gravel to the finest size possible. Their characteristics vary with the size. Soil particles coarser than 0.075 mm are visible to the naked eye or may be examined by means of a hand lens. They constitute the coarser fractions of the soils. Grains that are finer than 0.075 mm constitute the finer fractions of soils. It is possible to distinguish the grains lying between 0.075 mm and 2μ (1μ = 1 micron = 0.001 mm) under a microscope. Grains having a size between 2μ and 0.1μ can be observed under a microscope but their shapes cannot be made out. The shape of grains smaller than 1μ can be determined by means of an electron microscope. The molecular structure of particles can be investigated by means of X-ray analysis [22].

2.4 Comparison of coarse-grained and fine grained soils for engineering use

Course-grained soils have good load-bearing capacities and good qualities and their strength and volume change characteristics are not significantly affected by change in moisture conditions. They are practically incompressible when dense, but significant volume changes can occur when they are loose. Vibrations accentuate volume changes in loose coarse-grained soils by rearranging the soil fabric into dense configuration [7].

Fine-grained soils have poor load-bearing capacities compared with coarse grained soils. Fine-grained soils are practically impermeable, change volume and strength with variations in moisture conditions, and are frost susceptible. The engineering properties of coarse-grained soils are controlled mainly by the grain size of the particles and their structural arrangement [7].

Thin layers of fine-grained soils, even within thick deposits of coarse-grained soils, have been responsible for many geotechnical failures and therefore you need to pay special attention to fine-grained soils [7].

2.5 Identification and Classifications of Soils

Soil can be described as gravel, sand, silt and clay according to grain size. Most of the natural soils consist of a mixture of organic material in the partly or fully decomposed state. The proportions of the constituents in a mixture vary considerably and there is no generally recognized definition concerning the percentage of, for instance, clay particles that a soil must have to be classified as clay, etc.

Soils in nature rarely exist separately as gravel, sand, silt, clay or organic matter, but are usually found as mixtures with varying proportions of these components. Grouping of soils on the basis of certain definite principles would 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. The classification or grouping of soils is mainly based on one or two index properties of soil which are described in detail in earlier sections. The methods that are used for classifying soils are based on one or the other of the following two broad systems:

1. A textural system which is based only on grain size distribution.
2. The systems that are based on grain size distribution and limits of soil.

The physical properties of fine grained soils are dictated to a great extent by the amounts and types of clay minerals present in them. Hence, for proper interpretation of soil characteristics, the plasticity that is the result of the presence of clay minerals needs to be considered. Many systems are in use that is based on grain size distribution and limits of soil. The systems that are quite popular amongst engineers are the **AASHTO** Soil Classification System and the **Unified Soil Classification System** [22].

2.6. Expansive Soils

Expansive soil is a term generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. Subsequent swelling and shrinkage of this soils due to change in moisture cause damages to civil engineering structures, particularly light buildings and pavements [21].

The origin of expansive soils is related to a complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, will expand. The conditions and processes which determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected [8].

There are two fundamental molecular structures as the basic units of the lattice structure of clay soils. These are the silica tetrahedron and the alumina octahedron. The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions. The alumina octahedron consists of an aluminum atom surrounded octahedrally by six oxygen ions. When each oxygen atom is shared by two tetrahedral, a plate- shaped layer is formed. Similarly, when each aluminum atom is shared by octahedron a sheet is formed [21].

The silica sheets and the aluminum sheets combine to form the basic structural units of the clay particles. Various clay minerals differ in the stacking configuration. The major component of expansive soils, montmorillonite is a three layer clay mineral having a single octahedral sheet sandwiched between two tetrahedral sheets to give a 2 to 1 lattice structure [21].

2.6.1. Mechanics of swelling

Swelling of expansive soils will take place under change in the environment of the soil. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. By far the most important element and of most concern to the practicing engineer is the effect of water on expansive soil. There must be a potential gradient, which can cause water migration and a continuous passage through which water transfer can take place [8]. The potential gradient in expansive soils can be due to seasonal moisture fluctuation or thermal gradient, which can cause vapor and liquid moisture transfer. It is well recognized that the heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soils.

2.6.2. Factors influencing Swelling

The mechanism of swelling in expansive soils is complex and is influenced by a number of factors [21]. Expansion is the result of change in the soil water system that disturbs the internal force equilibrium. The factors influencing the shrink-swell potential of a soil can be considered in three different groups. These are the soil characteristics that influence the basic nature of the internal force field, the environmental factors that influence the changes that may occur in the internal force system, and the state of stress [19].

2.6.2.1. Soil characteristics

Soil characteristics may be considered either as micro scale or macro scale factors. Micro scale factors include the mineralogical and chemical properties of the soil. Macro scale factors include the engineering properties of the soil which intern dictated by the micro scale factors.

i) Micro scale factors (clay mineralogy and soil water chemistry):- clay minerals of different types typically exhibit different swelling potentials because of variation in the electrical field associated with each mineral. The swelling capacity of an entire soil mass depends on the amount and type of clay mineral in the soil, the arrangement and specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles.

Soil water chemistry is important in relation to potential swell magnitude. Salt cations such as sodium, calcium, magnesium, and potassium are dissolved in the soil water and are adsorbed on the clay surfaces as exchangeable cations to balance the negative electrical surface charges. Hydration of these cations and adsorptive forces exerted by the clay crystals themselves can cause the accumulation of a large amount of water between the clay particles. In dry soils, salt cations are held close to the clay crystal surfaces by strong electrostatic forces. As water becomes available, cation hydration energies are sufficiently large to overcome interparticle attraction forces. Thus initially dessicated and densely packed particles are forced apart as adsorbed cations hydrate and become enlarged on the addition of water [19].

ii) Macroscale factors (plasticity and density***):-*** Macroscale soil properties reflect the microscale nature of the soil. Because they are more conveniently measured in engineering work than microscale factors, macroscale characteristics are primary indicators of swelling behavior. Commonly determined properties such as soil plasticity and density can provide a great deal of insight regarding the expansive potential of the soils. Soil consistency, as quantified by the Atterberg limits, is the most widely used indicator of expansive potential. Most expansive soils can exist in a plastic condition over a wide range of moisture contents. This behavior results from the capacity of expansive clay mineral to contain large amount of water between particles and yet retain a coherent structure through the interparticle electrical forces. Soil plasticity, a useful indicator of swell potential, is influenced by the same microscale factors that control the swell potential [19].

2.6.2.2. Environmental conditions

The potential for a soil to absorb or expel water will depend on the water content relative to the water deficiency of the soil. Initial moisture content influences the shrink swell potential relative to possible limits, or ranges, in moisture content. Moisture content alone is not a good indicator or predictor of shrink-swell potential. Instead, the moisture content relative to limiting moisture contents such as the plastic limit and shrinkage limit must be known. Water content changes below the shrinkage limit produce little or no change in volume. There are indications that as a soil imbibes water, little volume change occurs at water content change above the plastic limit [21].

The availability of water to an expansive soil profile is influenced by many environmental and manmade factors. Generally, the upper few meters of the profile are subjected to the widest ranges of potential moisture variation. Also, overburden stress is low and the soil is not restrained against movement at shallow depths. This upper stratum (active zone) of the profile therefore exhibits the major part of the shrinking and swelling. Moisture variation in the active zone of a natural soil profile is affected by climatic cycles. Other obvious and direct causes of moisture variation result from altered drainage conditions or manmade sources of water, such as irrigation or leaky plumbing [21].

2.6.2.3. State of stress

Volume change is directly related to change in the state of stress in the soil. A reduction in the total stress due to excavation of overlying material will result in rebound and heave of the surface. Stress history is another factor which affects the swelling characteristics in that an over-consolidated soil is more expansive than the same soil of the same void ratio but normally consolidated. The thickness and location of potentially expansive layers in the profile also considerably influence potential volume change. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone [21].

2.7. Consolidation

2.7.1.Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. The soil layers are unable to spread laterally as the surrounding soil strata confines them. Hence there must be adjustment to the new pressure by vertical deformation. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. The vertical compression of the soil mass under increased pressures is thus made up of the following components:

i. Deformation of the soil grains

ii. Compression of water and air with in the voids

iii. An escape of water and air from the voids

It is quite reasonable and rational to assume that the solid matter and the pore water relatively are incompressible under the loads encountered. The change in volume of the soil mass under imposed stresses must be only due to the escape of water and air. Generally, the volume change in a soil deposit can be divided in to three stages [21]:

A) Initial consolidation:

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles [21].

B) Primary consolidation:

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as excess pore water, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the permeability of the soil and is, therefore, time dependent. The reduction in volume is called primary consolidation.

In fine grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability. As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles [21].

C) Secondary consolidation

The reduction in volume continues at a very slow rate even after the excess pore water pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in the volume is the called secondary consolidation. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the adsorbed water to the new stress system. In most inorganic soil, it is generally small [21].

2.7.2.Factors Affecting the Consolidation Characteristics of Clay Soils

The consolidation behavior of clay soil in its natural state is highly dependent on stress history and permeability. The effects of these factors are explained below.

2.7.2.1 Stress History

The maximum stress to which the soil is subjected in the past influence the consolidation characteristics of the soil in its insitu condition. In remolded soils, because it has lost its structural characteristics as compared with its structure in its natural condition, it is inferred that a remolded soil is unsuitable for evaluating its stress history [20].

As to the stress history, the insitu soil can be grouped in to two categories:

A) Normally consolidated soils

A normally consolidated soil is one whose present effective overburden pressure on the insitu prototype soil deposit is the maximum pressure to which the soil has ever been subjected at any time in the past history. In other words, the normally consolidated soil is one whose pre-consolidation pressure is equal to its present effective overburden pressure [20].

B) Over-consolidated clay soil

Over-consolidated clay is one which has been completely consolidated under a large overburden pressure in the past that is larger than the present overburden pressure. The response of over-consolidated clays to applied loads is such that at early loading the soil shows relatively small decrease of void ratio with load up to the maximum effective stress to which the soil was subjected in the past. If the effective stress on the soil specimen is increased further, the decrease of void ratio with stress level will be larger [20].

2.7.2.2 Permeability

The expulsion of water from the voids of a saturated clay soil by an externally applied load in the consolidation process and the change in volume associated with such a process are essentially a hydraulic problem. Specifically, it is a problem of permeability of a soil to water. Therefore, the rate of consolidation depends on the permeability of the soil. The permeability of the soil by itself is a function of the soil type, size and shape of the soil particles (rounded, angular, or flaky), and thus, up on the size and geometry of voids. Also, the resistance is a function of the temperature of water (viscosity and surface tension effect). [20]

2.7.3 Theory of one-dimensional consolidation

The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi. The underlying assumptions in the derivation of the mathematical equation are the following:

1. The soil is homogeneous and isotropic
2. The soil is fully saturated
3. The soil particles and the water in the voids are incompressible. The consolidation occurs due to expulsion of water from the voids
4. Darcy's law is valid throughout the consolidation process
5. Soil is laterally confined and the consolidation takes place only in the axial direction.

Drainage of water also occurs only in the vertical direction

The assumptions made by Terzaghi are not fully satisfied in actual field conditions. The results obtained from the use of the theory to practical problem are approximate.

However, considering complexity of the problem, the theory gives reasonably accurate estimate of the time rate of settlement of a structure built on the soil.

The standard one dimensional consolidation test is usually carried out on saturated specimen using an Odometer [9]. In this test a small representative sample of soil s carefully trimmed and fitted into a rigid metal ring. The soil sample is mounted on a porous stone base and a similar stone is placed on top to permit water, which is squeezed out of the sample to escape freely at the top and bottom. Prior to loading, the height of the sample should be accurately measured. Also, a micrometer dial is mounted in such a manner that the vertical strain in the sample can be measured as loads are applied. The consolidation test apparatus is designed to permit the sample to be submerged in water during the test to simulate the position below a water table of the prototype soil sample from which the test sample was taken. Loads are applied in steps in such a way that the successive load intensity, P, is twice the preceding one; the load intensities commonly used being ¼, ½, 1, 2, 4,8,16 kg/cm2. Each load is allowed to stand until primary consolidation is practically ceased. The dial readings are taken at elapsed time of 0, .0.25, 0.50, 1, 2, 4, 8, 15, 30,60minute.......24hours. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitude of plastic or permanent deformation.

The consolidation characteristics (or parameters) of a soil which are the compression index, Cc, and the coefficient of consolidation, C_v, will be determined from the test. The compression index relates to how much consolidation or settlement will take place. The coefficient of consolidation relates to how long it will take for an amount of consolidation to take place. The results of the odometer test are usually presented in the form of an e-P, e-log P, and dial reading- time plots [6].

2.7.3.1. Compression index

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

$$C_c = \frac{\Delta e}{\log(\frac{P_0 + \Delta P}{p_0})} \quad \text{.................. (2.1)}$$

The compression index is useful for the determination of the settlement in the field.

2.7.3.2. Coefficient of consolidation

A factor involved in characterizing the rate of consolidation of a soil is the one called the coefficient of consolidation, Cv, expressed as

$$C_v = \frac{(1+e)k}{a_v.\gamma_w} = \frac{k}{m_v.\gamma_w} \quad \text{.................... (2.2)}$$

Because of the fact that during the process of consolidation k and mv are assumed to be constant, the coefficient of consolidation C_V during the process of consolidation of the clay is constant. [20]

The coefficient of consolidation C_V as determined by Casagrande's semi logarithmic plot method is

$$C_v = \frac{(0.196).H^2}{t_{50}} .....[\frac{cm^2}{s}] \quad \text{...................... (2.3)}$$

The C_V value as determined by Taylor's square root of time fitting method is

$$C_v = \frac{(0.848).H^2}{t_{90}} ..... [\frac{cm^2}{s}] \qquad \text{.....................} (2.4)$$

2.7.3.3. Pre-consolidation pressure

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another. The practical significance of the pre-consolidation load appears in calculating settlements of structures [20].

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

$$OCR = \frac{P_c}{p_o} \qquad \text{..} (2.5)$$

2.8. Shear Strength

The shear strength of soil is one of the most important aspects of geotechnical engineering. The bearing capacity of shallow and deep foundations, slope stability, retaining wall design and pavement design are all influenced by the shear strength of the soil. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads. Thus limiting equilibrium method of analysis is conventionally used for their design, and these methods require determination of the ultimate or limiting shear resistance (shear strength) of the soil [34].

The shear strength can be determined in several different ways. In situ methods such as the vane shear test or penetrometers avoid some of the problems of disturbance associated with the extraction of soil samples from the ground. However, these methods only determine the shear strength indirectly through correlations with laboratory results or back calculated from actual failures. Laboratory tests, on the other hand, yield the shear strength parameters more directly. In addition, valuable information about the stress-strain behavior and development of pore pressures during shear can be obtained [34].

The shear strength of a soil is measured in terms of a limiting resistance to deformation offered by a soil mass or test sample when subjected to loading or unloading. The limiting shearing resistance corresponding to the condition generally referred to as 'failure', can be defined in several different ways. It is the resistance developed from a combination of particle rolling, sliding, and crushing and reduced by any excess pore pressure that develops during particle movement. The shear strength of a test sample is measured in the laboratory by subjecting it to certain defined conditions and carrying out a particular kind of test. Failure can occur in the soil as a whole, or within limited narrow zones referred to as failure planes. There are different criteria of 'failure', from which the shear strength of a soil is determined [34].

Three types of laboratory tests are commonly used to determine shear characteristics of soils. These tests are the direct shear test, the triaxial compression test and the unconfined compression test. The material characteristics that can be determined from these tests are the strength parameters (angle of internal friction, and cohesion). In some triaxial tests properties related to volume change such as modulus of elasticity and Possion's ratio can be obtained. These parameters are used for analysis and design in conventional civil engineering problems relating to slope stability, bearing capacity and any other situations where shear strength controls.

It should be noted, however, that laboratory strength test are meaningful only if the laboratory conditions of loading, drainage etc adequately represent the actual field conditions and also the soil sample being tested is representative of the insitu soil. Out of the three types of tests mentioned above, the unconfined compression test is more versatile and simulates the in situ conditions better. Therefore, it is used for this study.

The shear strength is measured in terms of two soil parameters, cohesion or inter particle attraction, and angle of internal friction, the resistance to inter particle slip. Grain crushing, resistance to rolling, and other factors are implicitly included in these two parameters. This behavior is well represented by the Mohr-Coulomb failure criterion given as,

$$S = c + \sigma \tan \Phi \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.6)$$

Where: S= shear strength

б= normal stress on shear plane

c= Cohesion

The shear parameters are often taken as constant but they depend on drainage condition, previous stress history, and current state (particle packing or density or water content). Therefore, soils seldom exhibit unique strength parameters and obtaining accurate values is not a trivial task [34].

CHAPTER THREE
RESEARCH METHODOLOGY

3.1 Study Area

Fiche is a town in central Ethiopia, Located in the North Shewa Zone of the Oromia Region, about 100 kilometers north of Addis Ababa on the paved highway leading to Gojam. The town has latitude and longitude of 9°48′N38°44′E and an elevation between 2,738 and 2,782 meters above sea level. The town is divided in to 4 kebeles, and it has around 27,493 populations according to (2007 ESA).

Fiche town has some industrial facilities like factories (small and large scale), warehouses, resorts, universities, schools and lodges. There is still a great potential for investment expansion in the area mainly as a result of its location near to the capital and main highway.

3.1.1 Topography and Drainage Conditions

The Fiche town altitude varies from place to place in which the northern part is at higher elevation and the newly constructed places (expansion places) are at relatively lower altitudes which means at central parts of the town.

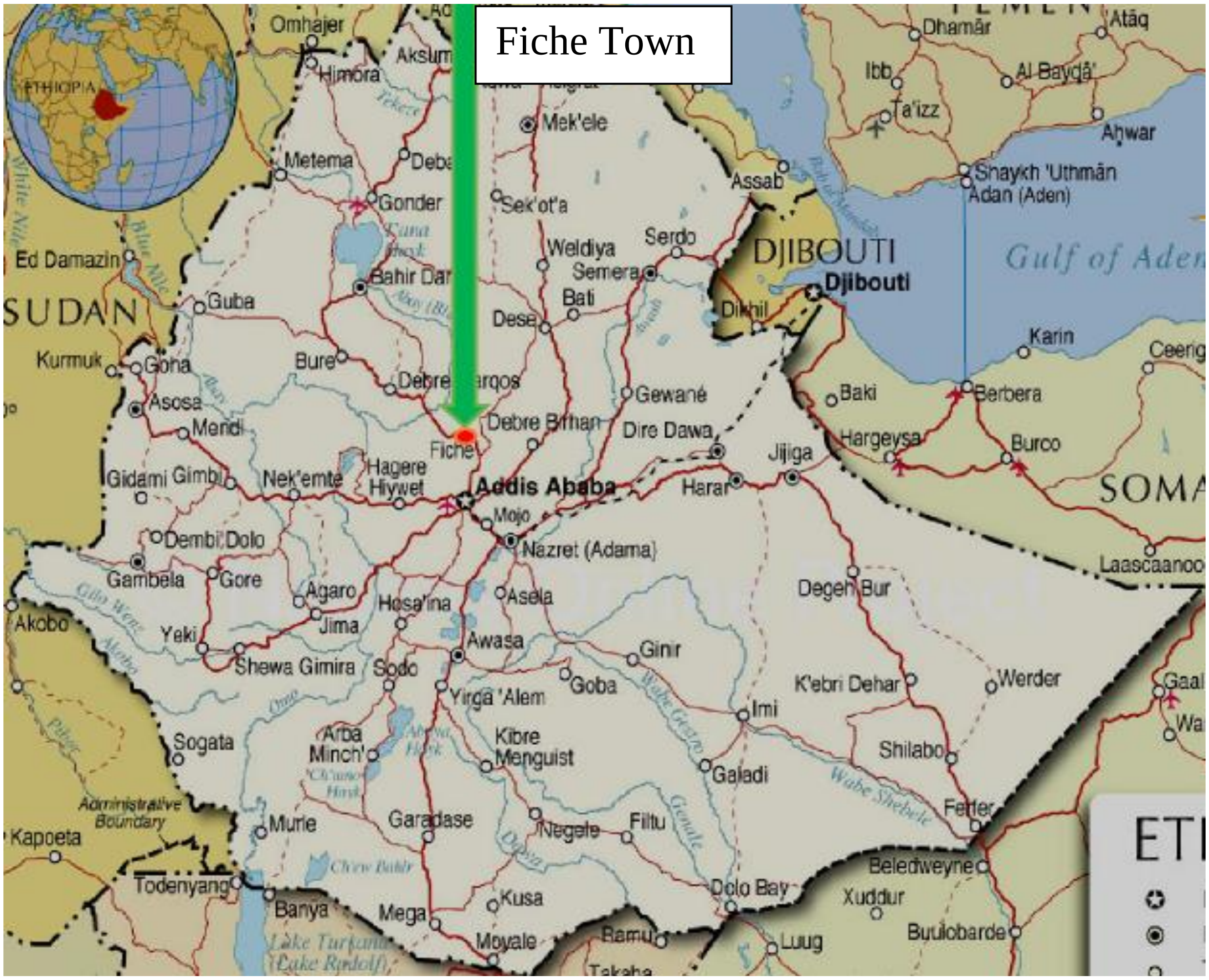


Figure 3.1: Location of study area .

Source: Ethiopian Google map

3.1.2 Geology and Soil Characteristics of Fiche town

The soil profile of the town varies from place to place mainly black, reddish brown and gray soils covers different parts of the town. The soil of Fiche town ranges from red and brown soil which is found in the southern parts of the town to black and gray clay soil which is generally observed in the Northern and other directions of the town. Both soils though have different colors are susceptible to serious soil erosion. As a result, the soil of the town at the central parts of the town and its surrounding areas are seriously degraded and eroded i.e. leaching effects.

Figure 3.2: Color of soil at Fiche town

3.1.3 Climate

The major factors influencing rainfall in Ethiopia are the Inter Tropical Convergence Zone (ITCZ) and winds blowing from the Atlantic and Indian Oceans. The variation in altitude throughout the country also influences climatic conditions. In addition, the micro-climatic changes over small distances are often created by differences in micro-relief. The traditional classification of climatic zones in Ethiopia is based on altitude and temperature. It divides the country into five climatic zones are shown in Figure 3.1 and summarized in Table 3.1 [12].

Table 3.1 Ethiopian Climatic Zones [12]

Climate Zone	Elevation(m)	Average Temperature(^{0} C)	Average Annual Rain fall(mm)
Wurch (Cold)	> 3200	<10	<800
Dega (Cool-cold)	2300-3200	10-16	1000-2000
Weina Dega (warm-cool)	1500-2300	16-20	1200
Kolla (hot-warm)	500-1500	20-28	600 (1000 in places)
Berha (hot)	<500	28-34	<400

3.1.3.1 Rainfall

Rain fall data collected by National Meteorological Service Agency substation on Fiche town located at latitude of 38.7333, longitude 9.7667 and altitude of 2784m for 30 years (1986-2015) shows that the mean annual rain fall is 962 mm. Like other major towns in Ethiopia, Fiche town received highest rain fall from July to August as shown in table 3.2. Since the average altitude of the area under investigation is 2750 m, the climatic zone by traditional classification is Dega (cool-cold).

Table 3.2 Mean annual rainfall of Fiche town (1986 - 2015 G.C.) [13]

Year	Mean annual RF (cm)	Year	Mean annual RF (cm)
1986	91.8	2001	85.7
1987	83.5	2002	88.4
1988	95.5	2003	98.1
1989	95.1	2004	91.9
1990	99.6	2005	81.9
1991	88.4	2006	112.4
1992	94.2	2007	94.9
1993	100.5	2008	87.2
1994	84.9	2009	101.8
1995	96.1	2010	107.0
1996	128.3	2011	103.1
1997	91.6	2012	107.8
1998	94.9	2013	97.2
1999	100.7	2014	93.4
2000	95.8	2015	93.21

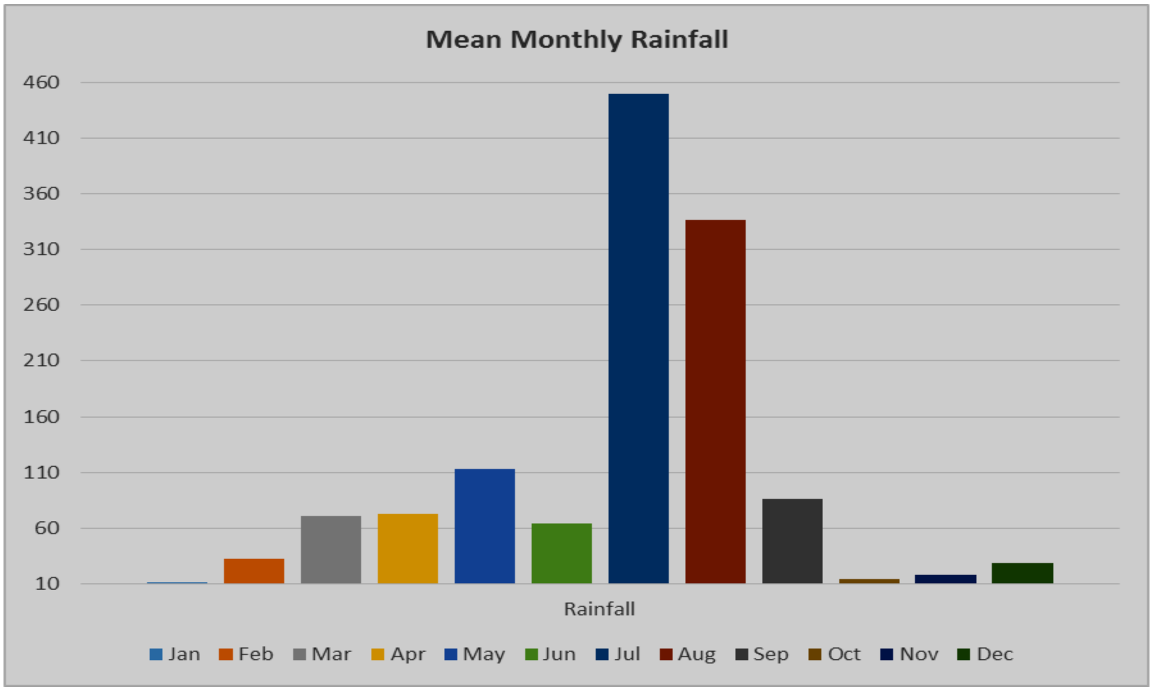


Fig 3.3 Mean monthly rainfall distribution of Fiche town (2014 G.C.) [13]

3.1.3.2 Temperature

In Ethiopia, the mean monthly temperature varies slightly throughout the year, although the difference between the minimum and maximum temperature is high only in the dry season. According to the National Metrological Agency of Ethiopia, the highest mean maximum temperatures in the country, in the range of 40 ^{0}C to 45 ^{0}C, are recorded in the Afar depression. The other hot areas are the north-western lowlands close to the border with Sudan, which experience a mean maximum temperature of 40 ^{0}C in June, and the western and south-eastern lowlands with mean maximum temperatures of 35 ^{0}C during April. Most of the Somali, Dire Dawa and Afar regions are also hot for several months in a year. The lowest mean temperatures in the range of 5 ^{0}C to 15 ^{0}C or even lower are recorded in the morning or at night between October and January in the highland areas, with an elevation of over 2,000 m above sea level. In these areas, the midday warmth diminishes quickly by late afternoon and nights are usually cold [12].

The climate of a given place is the reflection of different factors such as latitude, altitude, pressure difference between high lands and low lands and distance from the Sea as well as the impact of topography and cloud cover. If one looks at the climatic condition of

Fiche town and its surrounding areas, it can be concluded that the climate of Fiche is the reflection of altitude, cloud cover, impacts of winds from high land areas and rugged topography. Because of the impact of altitude, the agro climatic zone of the town is categorized as temperate climate because temperate climate prevails at the altitudinal limit the town is located. In order to analyze the temperature condition of Fiche town the data of 30 years is used.

Data recorded from national metrological service of Ethiopia shows the mean maximum temperature of Fiche town is 22.74 ^{0}C, and mean minimum temperature of 7.7 ^{0}C. From February to June the town receives highest temperature and from October to December lower temperature is dominant. (Table 3.3) shows Mean annual Temperature of Fiche town.

Table 3.3 Mean annual temperature of Fiche town (1986 - 2015 G.C) [13]

Year	Mean annual temp,(^{0} C)	Year	Mean annual temp,(^{0} C)
1986	15.25	2001	14.15
1987	14.18	2002	14.60
1988	14.50	2003	14.50
1989	13.70	2004	14.40
1990	14.12	2005	14.40
1991	14.15	2006	14.30
1992	13.95	2007	14.15
1993	13.85	2008	14.25
1994	14.20	2009	14.60
1995	14.35	2010	14.70
1996	13.85	2011	14.45
1997	14.30	2012	13.65
1998	14.40	2013	15.00
1999	13.95	2014	14.65
2000	13.95	2015	15.20

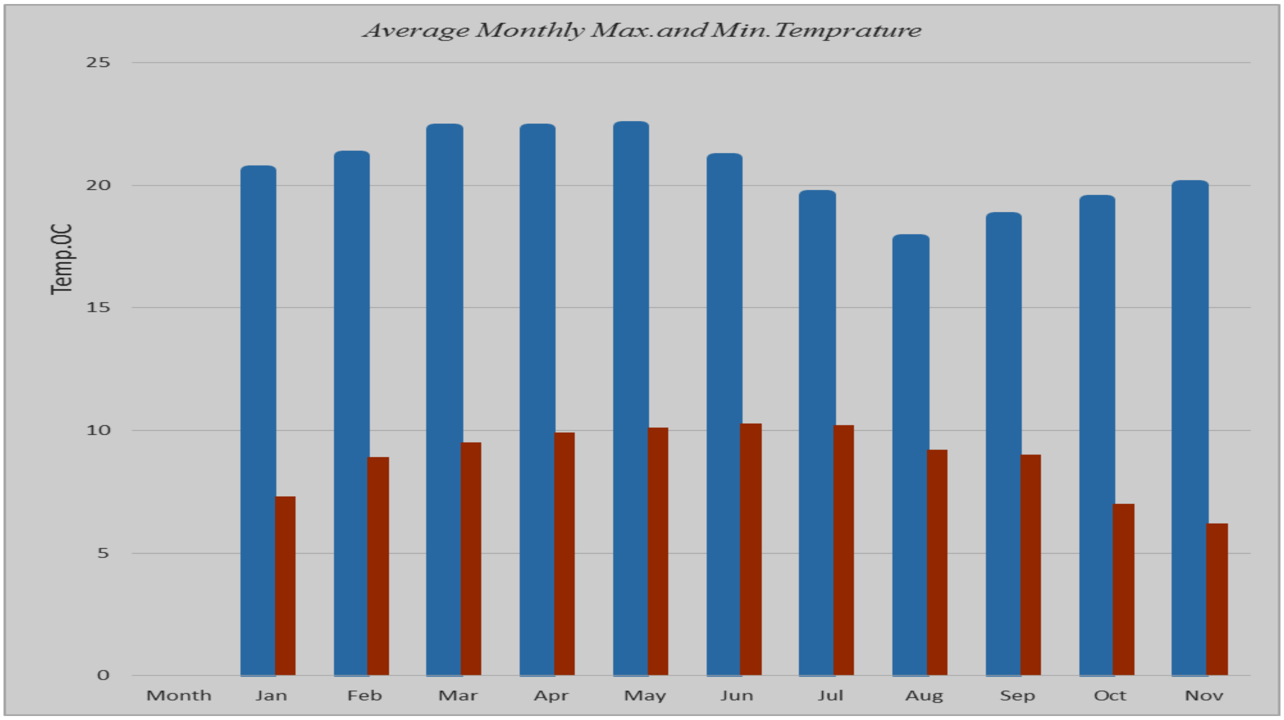


Fig 3.4 Average Monthly Maximum and Minimum temperature distribution of Fiche town (2014G.C) [13]

3.2 Study Design

To investigate the engineering properties of Fiche town, ten sampling areas were selected. The locations of the test pits are selected so that they can well represent soils found in Fiche Town. Excavations of pits were done at the depth of 1.5m and 3m to take samples and to identify the properties of soil in area at different layers. Locations of the sampling test pits were determined by hand held GPS. Disturbed and undisturbed samples were collected in the field and transported for laboratory testing to Addis Ababa Institute of Technology, Geotechnical lab. Disturbed samples are used for grain size analysis, index property, free swell, specific gravity and compaction tests whereas undisturbed samples are used for natural moisture content, consolidation and unconfined compression tests.

From the recovered samples the following laboratory tests were done.

- Natural moisture content
- Specific gravity test

- Atterberg limit tests
- Grain size analysis
  - ✓ Sieve analysis
  - ✓ Hydrometer
- Free swell test
- Unconfined Compression Test
- Standard compaction test,
- One-dimensional consolidation test

All the above tests were done according to American Society for Testing Materials (ASTM) standard. Using Microsoft Office Excel and Word, grain size distribution curve, liquid limit graph, compaction curve, consolidation and unconfined compression tests are plotted.

3.3 Population/Sample Description

A reconnaissance survey and gathering of information was done from different residents of Fiche, in order to understand the general soil formation of the area. Accordingly ten sampling areas were selected so that it can represent approximately the soil types found in the study area. From the primary designated locations twenty samples were recovered from ten test pits. It was impossible to collect samples from more test pits as some of formations are thin layer of soil rested on solid rock and some of them are deposited by residents and construction samples companies. The soil cover of the study area varies from place to place especially in the areas between high and low altitudes.

Both disturbed and undisturbed soil samples were collected from the test pits to determine the engineering and index properties of the soils. During recovery of the sample in the field, visual classification was made and location was recorded as described below in the (Table 3.4).

Table 3.4 Location of test pits

Location	Designation	Latitude	Longitude	Elevation (m)
04 Kebele /Oilibya Area	TP01	9.806	38.739	2784
Agricultural Research Center	TP02	9.800	38.720	2798
Fiche Hospital Area	TP03	9.797	38.737	2744
Nati Bar and Restaurant Area	TP04	9.782	38.724	2797
Biftu Salale Union BDG Area	TP05	9.767	38.743	2753
Kumando /Bas Station	TP06	9.790	38.726	2751
Stadium area	TP07	9.771	38.738	2881
Abdissa Aga School	TP08	9.776	38.748	2819
Medhanialem Church	TP09	9.780	38.746	2743
Infront of Fiche Agricultural Berau	TP10	9.786	38.742	2756

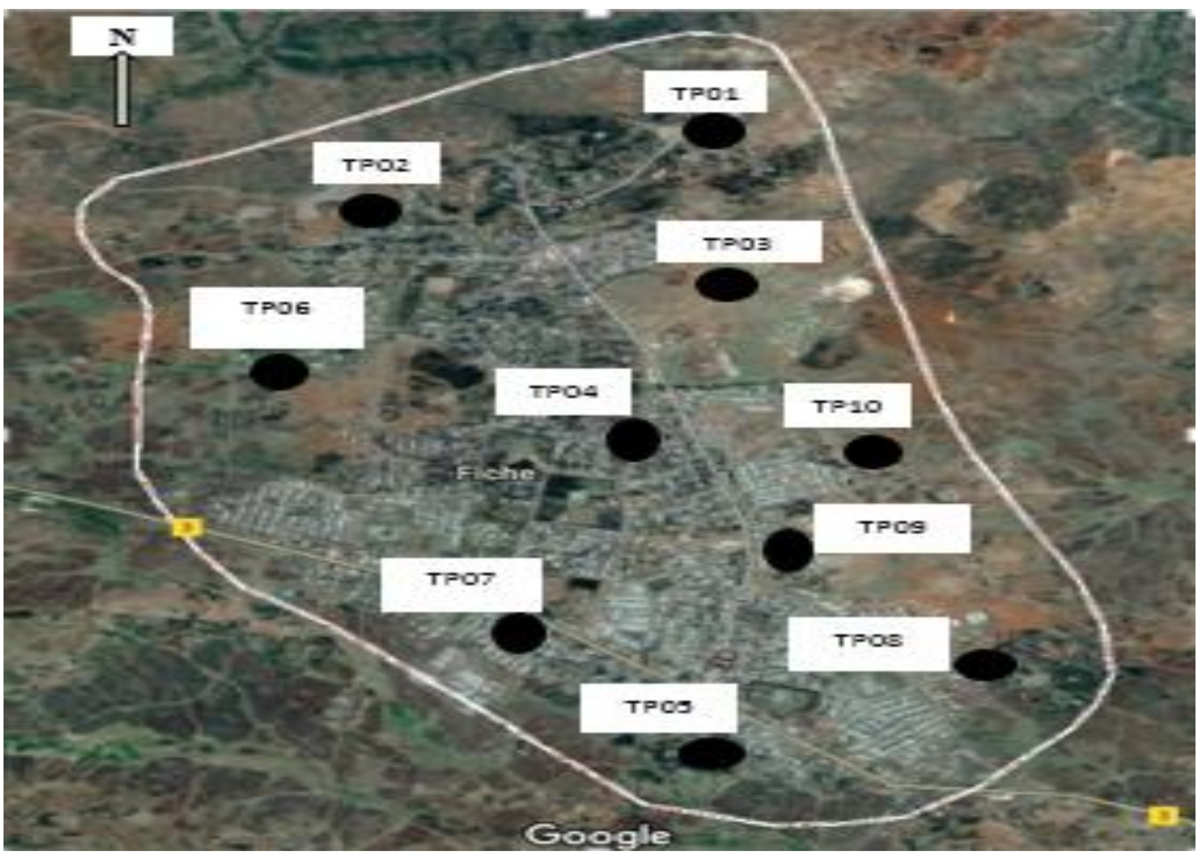


Fig 3.5 Location of test pits on Map (Fiche town)

3.4. Study Variables

Dependent Variable: The engineering properties of soils.

Independent Variable:

- Natural Moisture Content
- Specific gravity,
- Grain size analysis
- Atterberg limit
- Free swell,
- Standard compaction,
- Shear strength of soil and
- Consolidation.

3.5. Data Collection Process

The data was collected through:

- ✓ Collection of disturbed and undisturbed samples of subsurface strata from field
- ✓ Finally, laboratory tests of subsurface material was conducted and the properties of soils are obtained directly and indirectly.

3.6 . Data Processing and Analysis

3.6.1 Natural Moisture content

'Water content' or 'moisture content' of a soil has a direct bearing on its strength and stability. The water content of a soil in its natural state is termed as its 'Natural moisture content', which characterizes its performance under the action of load and temperature. The water content may range from a trace quantity to that sufficient to saturate the soil or fill all the voids in it [9].

For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil [3].

Equipment: Drying oven, Balance, Moisture can, Gloves, Spatula.

Moisture content was determined in the laboratory by collecting samples from all test pits and placed in to moisture can and covered properly by plastic. In the laboratory the weight of the moisture can and the weight of can with moist soil was measured. Then the sample was brought to the laboratory and put it in to drying oven at a temperature of 105+5°c for 24 hours. Then, the natural moisture content was determined.

Data Analysis:

First, the mass of soil solids was determined by using the following formula.

$$M_S = M_{CDS} - M_{SC} \quad \ldots\ldots \quad (3.1)$$

Second, the mass of pore water was determined by the following equation:

$$M_w = M_{CMS} - M_{CDS} \quad \ldots\ldots \quad (3.2)$$

Finally, the water content was determined by using the equation,

$$w = \frac{Mw}{Ms} x100 \quad \ldots\ldots \quad (3.3)$$

Where; M_S……………mass of solid soils

M_{CDS}………mass of the moisture can and lid

M_{SC} ………mass of can and dry soil

M_w…………mass of water

M_{CMS}………mass of can, lid and moist soil

M_{CDS}……….mass of can, lid and dry soil

w……………water content (%)

Results of this test used: It gives an indication the amount of water exists in a given soil mass, for cohesive soils it is used to determine its consistency, and to calculate other soil parameters [3].

3.6.2 Index properties

The index parameters are a measure of the physical properties and behavior of a soil. They are generally governed to a large extent by its geological history, mineralogical composition, and the amount of clay fraction, the structure and distribution of the grains, texture of the grains. Index parameters are mainly used for the purpose of identification, description and classification of soils. Moreover, since their determination in laboratory is relatively simple, and they share the same factors that influence the strength and compression properties, they are usually employed in empirical correlation to predict compression, strength and other parameters. For example, the compression index can be estimated from liquid limit, the undrained shear strength of clay from liquidity index or plasticity index [17].

After collecting samples in the field the index properties of all test pits are determined in the laboratory.

3.6.2.1 Specific gravity

This lab is performed to determine the specific gravity of soil by using a pycnometer. 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 a soil is used in the phase relationship of air, water, and solids in a given volume of the soil [3].

The specific gravity of most minerals found in soils falls within a range of 2.6 to 2.9. the specific gravity of solids of light-colored sand, which is mostly made of quartize, may be estimated to be about 2.65;for clayey and silty, it may vary from 2.6 to 2.9 [10]

Equipment: Pycnometer, Balance, Vacuum pump, Funnel, Spoon.

Data Analysis:

The specific gravity of the soil solids was calculated using the following formula:

$$G_S = \frac{W_0}{W_0 + (W_A - W_B)} \qquad \text{..} (3.4)$$

Where: G_S = Specific gravity of soil

W_O = weight of sample of oven-dry soil, g = $W_{PS} - W_P$

W_A = weight of pycnometer filled with water

W_B = weight of pycnometer filled with water and soil

Results of this test used: to determine the density of soil grain and to calculate other soil parameters [3].

3.6.2.2 Grain Size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles [3].

Soils are usually classified into various types. In many cases these various types also have different mechanical properties. A simple subdivision of soils is on the basis of the grain size of the particles that constitute the soil. The classification according to size divides the soils broadly into two distinctive groups, namely, coarse grained and fine grained. Since the properties of coarse grained soils are, to a considerable extent, based on grain size distribution, classification of coarse grained soils according to size would therefore be helpful. Fine grained soils are so much affected by structure, shape of grain, geological origin, and other factors that their grain size distribution alone tells little about their physical properties. However, one can assess the nature of a mixed soil on the basis of the percentage of fine grained soil present in it. It is, therefore, essential to classify the soil according to grain size [22].

Soil particles which are coarser than 0.075 mm are generally termed as coarse grained and the finer ones as silt, clay and peat (organic soil) are considered fine grained. From an engineering point of view, these two types of soils have distinctive characteristics. In coarse grained soils, gravitational forces determine the engineering characteristics. Interparticle forces are predominant in fine grained soils [22].

The mechanical analysis was done in two stages: 1) sieve analysis, 2) Sedimentation analysis. The first analysis was meant for coarse-grained soils (particle size greater than 75 micron) which can easily pass through a set of sieves. The second analysis was used for fine grained soils (size smaller than 75 microns). As a soil mass may contain the particles of both types of soils, a combined analysis comprising both sieve analysis and sedimentation analysis may be required for such soils [2].

In this research both types of mechanical analysis was implemented to determine the grain size distribution of soils recovered from ten test pits. Sieve analysis was conducted in wet type because the soil samples contain large percentage of clay and silts.

A series of sieves in which the sequence is described below was prepared according to ASTM D422-63 and placed on mechanical shaker. The oven dried sample was placed on series of sieves and shacked for about 10 minutes. The mass of retained soil on each sieve was recorded and the graph was plotted.

Table 3.5 Series of sieves for plotting.

Series No.	Sieve designation	Series No.	Sieve designation
1	3-in. (75-mm))	7	No. 16 (1.18-mm)
2	1 1⁄2-in. (37.5-mm)	8	No. 30 (600-µm)
3	3⁄4-in. (19.0-mm)	9	No. 50 (300-µm)
4	3⁄8-in. (9.5-mm)	10	No. 100 (150-µm)
5	No. 4 (4.75-mm)	11	No. 200 (75-µm)
6	No. 8 (2.36-mm)		

Particle size distribution-Hydrometer analysis

Hydrometer method combined with wet or dry sieving enable a continuous particle size distribution curve of a soil to be plotted from the size of the coarser particles down to clay sizes. This test method of grain size determination was used for soils finer than 75 microns.

Both methods of grain size analysis were conducted for all soils recovered from ten test pits. The above procedures were implemented and a combined grain size distribution is plotted.

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

Equipments:

Balance, Set of sieves, Cleaning brush, Sieve shaker, Mixer (blender), 152H Hydrometer, Sedimentation cylinder, Control cylinder, Thermometer, Beaker, Timing device.

Data Analysis:

Sieve Analysis:

First, the mass of soil retained on each sieve was obtained by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and this mass was recorded as the weight retained on the data sheet. Second, the percent retained on each sieve was calculated by dividing the weight retained on each sieve by the original sample mass.

Thirdly, the percent passing (or percent finer) was calculated by starting with 100 percent and subtracting the percent retained on each sieve as a cumulative procedure. Fourth, a semi logarithmic plot of grain size versus percent finer graph was drowned.

3.6.3 Atterberg limits

This lab was performed to determine the plastic and liquid limits of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling [3].

The Swedish soil scientist Albert Atterberg originally defined seven "limits of consistency" to classify fine-grained soils, but in current engineering practice only two of the limits, the liquid and plastic limits, are commonly used. (A third limit, called the shrinkage limit, is used occasionally.) The Atterberg limits are based on the moisture content of the soil. The plastic limit is the moisture content that defines where the soil changes from a semi-solid to a plastic (flexible) state. The liquid limit is the moisture content that defines where the soil changes from a plastic to a viscous fluid state. The shrinkage limit is the moisture content that defines where the soil volume will not reduce further if the moisture content is reduced. A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system[3].

The presence of water in the voids of a soil can especially affect the engineering behavior of fine-grained soils. Not only is important to know how much water is present in, but also to compare or scale this water content against some standard of engineering behavior. This is what the Atterberg limits do-they are important limits of engineering behavior. Atterberge limits are water contents at certain limiting or critical stages in soil behavior. They, along with the natural water content, are the most important items in the description of fine-grained soils. They are useful in the classification of such soils, and they are useful because they correlate with the engineering properties and engineering behavior of fine-grained soils [9]. Hence depending on moisture content, the behavior of soil can be divided in to four basic states-solid, semisolid, plastic and liquid as shown in Figure 4.6 [10]. Burmister (1947) classified plastic properties of soils according to their plasticity indices as follows:

Table 3.6 Plasticity characteristic according to Burmister (1947)

Plasticity index	**Plasticity**
0	Non-plastic
1 to 5	Slight
5 to 10	Low
10 to 20	Medium
20 to 40	High
>40	Very high

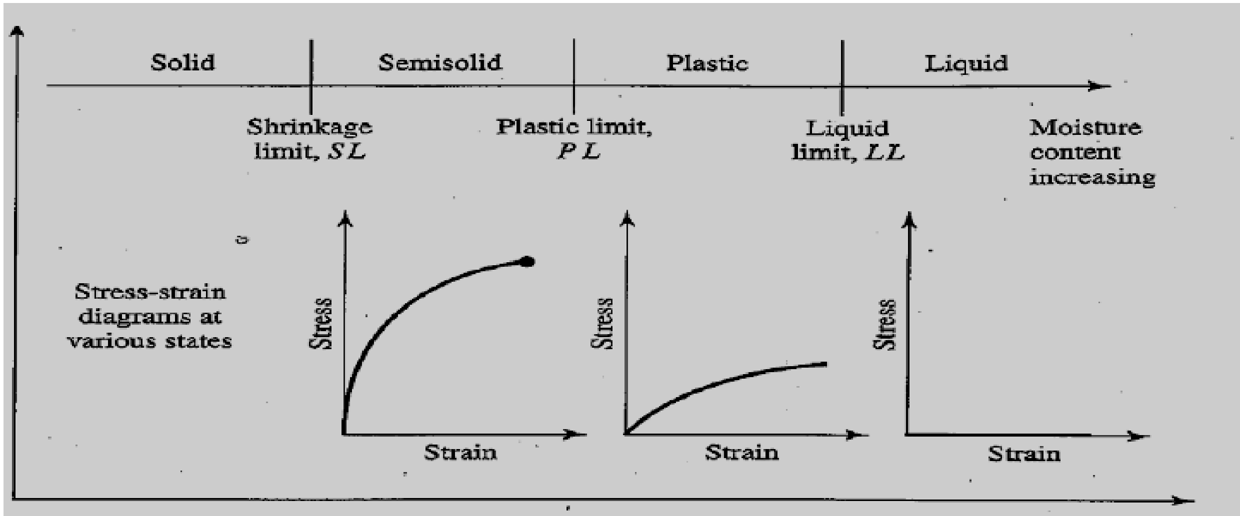


Fig 3.6 Atterberg limits according to Das (2002)

Equipments:

Liquid limit device, Porcelain (evaporating) dish, Flat grooving tool with gage, Moisture cans, Balance, Glass plate, Spatula, Wash bottle filled with distilled water, Drying oven set at 105°C.

Liquid Limit Analysis:

First, the water content of each of the liquid limit moisture cans was calculated after they have been in the oven for at least 24 hours. Second, the graph was plotted using the number of drops, N, (on the log scale) versus the water content (w), and the best-fit straight line was drowned through the plotted points and the liquid limit (LL) was determined as the water content at 25 drops [3]

Plastic Limit Analysis:

First, the water content of each of the plastic limit moisture cans was calculated after they have been in the oven for 24 hours. Second, the average of the water contents was computed to determine the plastic limit, PL. Finally, the plasticity index was calculated using, PI=LL-PL, and the liquid limit, plastic limit, and plasticity index was reported to the nearest whole number, omitting the percent designation.

3.6.4 Classification of Soils

Soil classification systems divide soils and sub groups based on common engineering properties such as the grain size distribution, liquid limit and plastic limit [10]. Soil classification was used to specify a certain soil type that is best suitable for a given application. From engineering point of view, classification may be made based on the suitability of a soil for use as a foundation material or as a construction material. There are several classification schemes available. Each was devised for a specific purpose [7].

3.6.4.1 Unified Soil Classification System (USCS)

This system employs visual inspection; grain-size analysis and Atterberg limit tests in classifying soils. The coarse soils were classified by their grain size and fine grained soils were classified with the aid of plasticity chart [3].

3.6.4.2 AASHTO Soil Classification System

This system was originally proposed in 1928 by the U.S. Bureau of Public Roads for use by highway engineers. A Committee of highway engineers for the Highway Research Board met in 1945 and made an extensive revision of the PRA System. This system is known as the AASHTO (American Association of State Highway and Transportation Officials) System (ASTM D-3242, AASHTO Method M 145). The revised system comprises seven groups of inorganic soils, A-l to A-7 with 12 subgroups in all. The system is based on the following three soil properties:

1. Particle-size distribution
2. Liquid Limit
3. Plasticity Index [22].

This classification uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30.

3.6.5 Free Swell of Soil

This test tries to give a fair approximation of the degree of expansiveness for a given soil sample. The swelling (and shrinking) characteristics of expansive clay vary with the type of clay mineral present in the soil, the percentage of that clay mineral, and the change in water content. The active clay minerals include montmorillonite, mixed-layer combinations of montmorillonite and other clay minerals, and under some conditions chlorites and vermiculites [31].

To study the swelling property of the soils, the simplest test conducted was free swell test. This test was performed by slowly pouring 10 ml of oven dry soil which has passed the No. 40(0.425mm) sieve in to 100 ml graduated cylinder filled with distilled (tap) water. After 24 hours, final volume of the suspension being read. Hence, free swell is calculated as [11]:

$$\text{Free swell} = \frac{\text{Final volume} - \text{Initial volume of the soil}}{\text{Initial Volume}} \times 100\ \% \qquad (3.5)$$

Free swell<50%...............Not expansive

Free swell between 50-100%............Marginal

Free swell > 100%.................Expansive [3]

Holtz and Gibbs suggested that soils having a free-swell value as low as 100 percent can cause considerable damage to lightly loaded structures and soils having a free swell value below 50 percent seldom exhibit appreciable volume change even under light loadings [22].

3.6.6 Compaction Test

This laboratory test was performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass [3].

The term density refers to mass per unit volume. The density of a mass of soil is of interest to the engineer for a variety of reasons including the design of earthworks and foundations and in slope Stability analysis [12].

Soil placed as engineering fill (embankments, foundation pads, road bases) must be compacted to the selected density and water content to ensure the desired performance and engineering properties such as shear strength, compressibility, or permeability. Also, foundation soils are often compacted to improve their engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed in the field, and for controlling construction to assure that the target values are achieved.

The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density [9].

Two types of compaction tests routinely performed are: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. In the Standard Proctor Test, the soil is compacted by a 24.4N hammer falling a distance of 0.305meters into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test, except it employs, a 44.5N hammer falling a distance of 0.457meters, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 0.102meters in diameter and has a volume of about 944 cm3, and the larger type is 0.152meters in diameter and has a volume of about 2123 cm3. If the larger mold is used each soil layer must receive 56 blows instead of 25[9].

Generally course grained soils can be compacted to a higher dry density than fine gained soils for the some compaction effort. When some fines are added to the coarse grained soils to fill the voids, the maximum dry density further increases, but if the amount of fines is too much, more than required to fill the voids, it results in reduction of dry density; well graded soils can attain higher dry density than poorly graded soils. High plasticity clays attain much less dry density than low plasticity clays for the some completive effort [9].

Equipments: Molds, Manual rammer, Extruder, Balance, Drying oven, Mixing pan, Trowel, #4 sieve, Moisture cans, Graduated cylinder, Straight Edge.

Analysis:

First, the moisture content of each compacted soil specimen was calculated by using the average of the two water contents. Second, the wet density in grams per cm3 of the compacted soil sample was computed by dividing the wet mass by the volume of the mold used.

Third, the dry density was computed using the wet density and the water content determined in first step by using the following formula:

$$\rho_d = \frac{\rho}{1 + w} \quad \text{………………………………………………. (3.6)}$$

Where: w = moisture content in percent divided by 100, and ρ = wet density in grams per cm3.

Finally, the dry density values on the y-axis and the moisture contents on the x-axis were plotted by connecting the plotted points.

3.6.7 Unconfined compression strength (UCS) test

The primary purpose of this test was to determine the unconfined compressive strength, which was then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions [3].

The unconfined compression test is a special case of a triaxial compression test in which the all-round pressure $\boldsymbol{\sigma_3}$=0 (minor stress). The tests were carried out only on saturated samples which can stand without any lateral support. The test was, therefore, applicable to cohesive soils only. The test was an undrained test and was based on the assumption that there was no moisture loss during the test. The unconfined compression test was one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils. These tests can also be performed in the field by making use of simple loading equipment [22].

An axial load was rapidly applied to the specimen to cause failure. At failure, the total minor principal stress is zero and the total major principal stress is $\boldsymbol{\sigma_1}$ (Figure 3.7) [10].

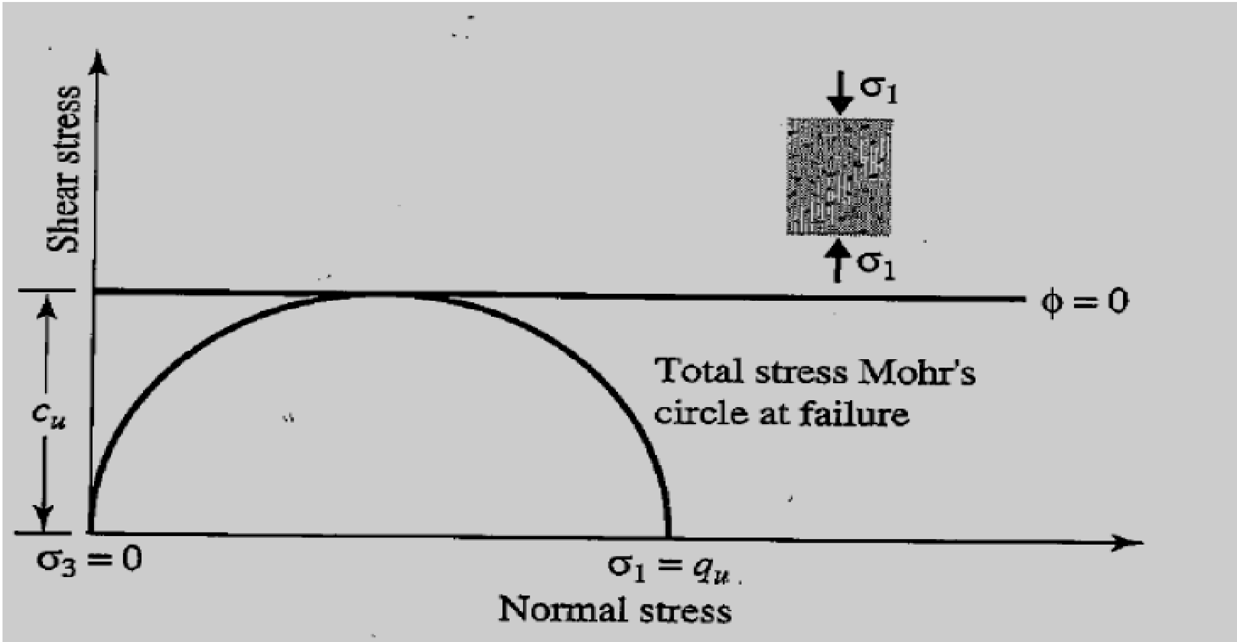


Figure 3.7 Unconfined compression tests [10]

Because undrained shear strength is independent of the confining pressure as long as the soil is fully saturated and fully undrained, we have Eqn 3.2 [10];

$$\tau_f = \frac{\sigma_1}{2} = \frac{q_u}{2} = c_u \quad \text{……………………………… (3.7)}$$

Where, q_u is the unconfined compression strength.

In the unconfined compression test the internal angle of friction is negligible and the soil assumed to have only cohesion.

The general relation between consistency and unconfined compression strength of clays is given in Table 3.7.

Table 3.7. The general Relationship of consistency and Unconfined Compression strength of clay after Das.

Consistency	**q_u, KN/m^2**
Very soft	0-25
Soft	25-50
Medium	50-100
Stiff	100-200
Very Stiff	200-400
Hard	>400

For soils, the undrained shear strength (su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (su) of clays is commonly determined from an unconfined compression test.

Equipments:

Compression device, Load and deformation dial gauges, Sample trimming equipment, Balance, Moisture can.

Analysis:

First, the dial readings converted to the appropriate load and length units, and these values entered on the data sheet in the deformation and total load columns. Second, the sample cross-sectional area was computed by using;

$$A_0 = \frac{\pi}{4} \times (d)^2 \quad \text{……………………………………………. (3.8)}$$

Third, the strain computed by using the equation,

$$\text{Strain } (e) = \frac{\Delta L}{L_o} \quad \text{……………………………………… … (3.9)}$$

Fourthly, the corrected area computed by using,

$$A' = \frac{A_0}{1-e} \quad \text{………………………………………………… ..(3.10)}$$

Fifthly, using A', the specimen stress computed by using the formula,

$$s_c = \frac{P}{A'} \quad \text{…………………………………………………. (3.11)}$$

Then the water content, w% was computed, and the stress versus strain plotted on log scale by showing qu as the peak stress (or at 5% strain) of the test. Finally, the Mohr's circle was drowned using qu from the last step and the undrained shear strength was calculated by using, su = c_u (or cohesion) = qu/2.

3.6.8 Consolidation Test

This test was performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil [3].
One dimensional consolidation test was carried out according to ASTM, D2435-96 test method. Undisturbed sample was recovered from four different test pits at three meters of Fiche town area after classification was done and properly waxed until it was transported to laboratory.
The final void ratios corresponding to each pressure increment have been calculated and a graph was plotted between pressure as abscissa on a log scale and void ratio as ordinate on arithmetic scale.

Compression index, Cc, is numerically equal to the slope of the straight portion of the e-log P curve. Its value is constant beyond the range of the re-compration, since beyond this point the plot of e against log P is straight line. Nothing that,

$$C_c = \frac{e_1 - e_2}{\log \sigma_2' - \log \sigma_1'} = \frac{\Delta e}{\log(\sigma_2'/\sigma_1')}$$
…………………….. (3.12)

There après to be an approximate relationship between the liquid limit of a clay soil and the compression index. Skempton (1944) have demonstrated that this relationship can be expressed by the following formula.

Cc=0.009(LL-10%)………………………………………….... (3.13)

Where LL is liquid limit expressed in percent [28].

Fig 4.8 shows a typical schematic diagram how to determine the compression index from the void ratio versus applied effective pressure.

3.6.8.1 Coefficient of consolidation

The coefficient of consolidation C_v can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical. There are two laboratory methods that are in common use for the determination of ***Cv***. They are;

I. Casagrande Logarithm of Time Fitting Method.

II. Taylor Square Root of Time Fitting Method [22].

I) Taylor Square Root of Time Fitting Method

This method has been devised by D.W. Taylor (1948). The coefficient of consolidation is the soil property that controls the time-rate or speed of consolidation under a load-increment [9]. A straight line was drawn through the points representing the initial readings that exhibit a straight line trend. Then the line was extrapolated back to t=0 and the deformation ordinate representing 0% primary consolidation was obtained.

A second straight line through the 0% ordinate was drawn so that the abscissa of this line was 1.15 times the abscissa of the first straight line through the data. The intersection of this second line with the deformation-square root of time curve was the deformation, d90, and time, t90, corresponding to 90% primary consolidation.

The deformation at 100% consolidation was 1/9 more than the difference in deformation between 0 and 90% consolidation. The time of primary consolidation, t100, may be taken as the intersection of the deformation-square root of time curve and this deformation ordinate. The deformation, d50, corresponding to 50% consolidation was equal to the deformation at 5/9 of the difference b/n 0 and 90% consolidation.

From the measured data and the data obtained from either of the above two methods, the consolidation curve (pressure-void ratio relationship) can be plotted. This data was used in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data 50 obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil.

Because of the fact that during the process of consolidation **k** and m_v are assumed to be constant, and the coefficient of consolidation ***Cv*** by Taylor Square Root of Time Method is given by;

$$c_v = 0.848\,\frac{H_{dr}^2}{t_{90}} \qquad (3.14)$$

Where H_{dr} - drainage path (average)

3.6.8.2 Pre-consolidation pressure

Another very important characteristic of clays is the pre-consolidation pressure σp. It is the vertical effective stress beyond which large strains occur and controls the overall behavior of clays, particularly the sensitive clays. Previously, it was believed that the pre-consolidation pressure estimated with Casagrande method was primarily due to previous loading, usually of geologic nature. However, it has become evident in recent years that the profile of the pre-consolidation stress observed in some deposits is greater than the maximum past pressure that could have existed during its geologic history. This discrepancy was attributed to a number of factors, including desiccation, long term secondary compression, thixotropy, weathering and cementation.

Since the exact origin of the pre-consolidation pressure is difficult to establish, the term has been extended to define the break of the e - log σp curve [17].

From practical point of view engineers are interested in this threshold point beyond which important plastic deformation take place, particularly in sensitive clays where the normally consolidated branch of the compression curve is very steep. The pre-consolidation pressure serves as basis for normalizing the strength and stiffness characteristics of cohesive deposits. For young normally consolidated soft clays, the effective pre-consolidation pressure is equal to the effective overburden pressure, where the soil deposit is not subjected to previous external load such as building loads [17].

There are different types of determining the pre-consolidation pressure in the field as well as in the laboratory. The earliest and the most widely used method was the one proposed by Casagrande in (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log **σ**p curve of an undisturbed sample as shown in Fig. 3.8. 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 ***Pc*** [22].

3.6.8.3 over Consolidation Ratio

We will create a demarcation for soils based on their consolidation history, and we will label a soil whose current vertical effective stress or overburden effective stress, is less than its past maximum vertical effective stress or pre consolidation stress as an over consolidated soil. The degree of over consolidation, called over consolidation ratio, OCR, is defined as;

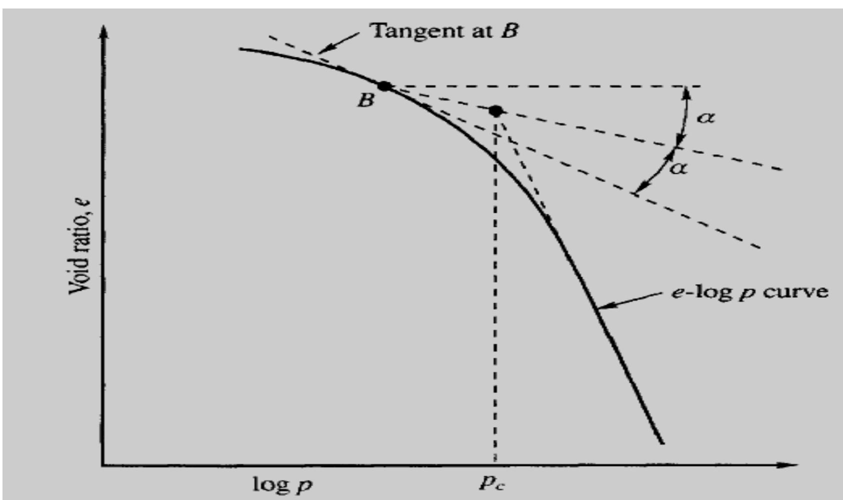


Fig 3.8 Method of determining p_C by Casagrande method [22]

$$\mathrm{OCR} = \frac{P_c}{P_o} \qquad \text{……………………………………………………………….. (3.15)}$$

Where ***P***c=Pre-consolidation pressure ***P***o=current vertical effective stress.

If OCR is less than or equal to one, the soil is normally consolidated soil [7].

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure or an earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance [3].

Equipments: Consolidation device (including ring, porous stones, water reservoir, and load plate), Dial gauge (0.0001 inch = 1.0 on dial), Sample trimming device, glass plate, Metal straight edge, Clock, Moisture can, Filter paper.

Analysis:

First, the initial water content and specific gravity of the soil were calculated. Second, for each pressure increment, a semi log plot of the consolidation dial readings versus the log time (in minutes) were constructed, and D0, D50, D100, and the coefficient of consolidation (cv) were determined using Casagrande's logarithm of time fitting method. Thirdly, the void ratio was calculated at the end of primary consolidation for each pressure increment, the log pressure versus void ratio was plotted. Based on this plot, compression index, recompression index and pre-consolidation pressure were calculated. Finally, the results were summarized and discussed in detail.

3.7 Ethical Considerations

The data was collected after ethical permission was given from ERA and Civil Engineering department of Jimma University. Before continuing the research study, acceptance was given from local authorities of Fiche town. The purpose of the study was clearly described to the organization and to the concerned local communities.

Generally, the following is a rough and general summary of some ethical principles that was considered in this research:

- ✓ Honesty
- ✓ Respect for Intellectual Property
- ✓ Objectivity
- ✓ Integrity
- ✓ Carefulness
- ✓ Openness
- ✓ Confidentiality

- ✓ Respect for colleagues
- ✓ Social Responsibility
- ✓ Non-Discrimination
- ✓ Legality and etc.

3.8 Data Quality Assurance

Data quality was assured by considering the following activities, laboratory test and field work manual were prepared in order to avoid error of data, the training was given for data collectors to handle the data carefully, the reliability and accuracy of data was checked, laboratory instruments are calibrated, and at least three trial experiments were done for one test parameters in order to avoid error of data and results.

CHAPTER FOUR

LABORATORY TEST RESULTS AND DISCUSSIONS

4.1. Laboratory Test Results and Observations

4.1.1 Field Observations

Soils of Fiche town was identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include:

- The color of the soil is identified (i.e. black or gray, reddish, brown).
- The dominant types of soils properties in the test pits are black clay in the upper 1.5m and reddish brown to gray silty clay below 1.5m.
- It has high dry strength and low wet strength when the soil is touched with hands.
- A shiny surface is obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail.
- In some places where there is seasonal moisture variation open or closed fissures (a joint or similar discontinuity), Slickenside (highly polished or glossy fissure surface) and shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils) may observed.

4.1.2 Natural Moisture content

This test is performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of “pore” or “free” water in a given mass of soil to the mass of the dry soil solids. Moisture content was determined in the laboratory by collecting samples from all test pits and placed in to moisture can and covered properly by plastic. In the laboratory the weight of the moisture can and the weight of can with moist soil was measured. Then the sample was brought to the laboratory and put it in to drying oven at a temperature of 105+5°c for 24 hours. Then, the natural moisture content was determined. The test results of all test pits are shown below in Table 4.1.

Table 4.1 Summary of natural moisture content

Sr.No.	Test pit name	Depth of determination	Natural moisture content, %	Color of samples
1	TP-01	1.50m	33.80	Black, brown
		3.00m	58.89	Red, gray
2	TP-02	1.50m	21.91	Black, brown
		3.00m	24.17	Red, gray
3	TP-03	1.50m	29.92	Black, brown
		3.00m	31.04	Red, gray
4	TP-04	1.50m	37.37	Black, brown
		3.00m	23.52	Red, gray
5	TP-05	1.50m	32.75	Black, brown
		3.00m	30.83	Red, gray
6	TP-06	1.50m	42.94	Black, brown
		3.00m	46.86	Red, gray
7	TP-07	1.50m	31.39	Black, brown
		3.00m	43.54	Red, gray
8	TP-08	1.50m	43.54	Black, brown
		3.00m	40.38	Red, gray
9	TP-09	1.50m	25.45	Black, brown
		3.00m	28.37	Red, gray
10	TP-10	1.50m	27.59	Black, brown
		3.00m	20.01	Red, gray

4.1.3 Index Properties

4.1.3.1 Specific gravity

The specific gravity of soils found in Fiche town falls to 2.62-2.88 which was in the range proposed by Bowles and other researchers. The soil is clay soil and its specific gravity varies with the range based on mineral content of the soil.

The laboratory test results of ten test pits were summarized in table 4.2 below.

Table 4.2 Summary of specific gravity test result

Sr.No.	Test pit name	Depth of determination	Specific gravity
1	TP-01	1.50m	2.62
		3.00m	2.68
2	TP-02	1.50m	2.63
		3.00m	2.67
3	TP-03	1.50m	2.71
		3.00m	2.65
4	TP-04	1.50m	2.66
		3.00m	2.69
5	TP-05	1.50m	2.88
		3.00m	2.72
6	TP-06	1.50m	2.77
		3.00m	2.64
7	TP-07	1.50m	2.73
		3.00m	2.70
8	TP-08	1.50m	2.71
		3.00m	2.67
9	TP-09	1.50m	2.68
		3.00m	2.62
10	TP-10	1.50m	2.69
		3.00m	2.71

4.1.4 Grain Size Analysis

The soils of Fiche town contain 8.67-67.51% clay, 26.29-55.54% silt, 1.44-39.73% sand and 0-25.62% gravel. Summary of the test result and graph of combined analysis is shown below on (Table 4.3), (Fig 4.1) and (Fig 4.2) respectively and detail analysis is attached in appendix-A.

Table 4.3 Percentage of grain size distribution

Sr.No.	Test pit name	Depth of determination	Percent amount of particle size			
			Gravel,%	Sand ,%	Silt,%	Clay,%
1	TP-01	1.50m	0.33	2.86	29.30	67.51
		3.00m	0.00	3.38	46.91	49.71
2	TP-02	1.50m	4.03	23.45	47.75	24.77
		3.00m	1.78	34.01	55.54	8.67
3	TP-03	1.50m	0.01	14.13	43.31	42.55
		3.00m	16.89	39.73	26.82	16.56
4	TP-04	1.50m	2.41	15.40	46.09	36.10
		3.00m	0.44	33.22	48.54	17.80
5	TP-05	1.50m	1.41	7.81	41.73	49.05
		3.00m	0.12	8.73	47.05	44.10
6	TP-06	1.50m	0.18	4.83	46.54	48.45
		3.00m	1.46	7.38	47.24	43.92
7	TP-07	1.50m	0.04	8.25	35.25	56.46
		3.00m	3.10	27.37	56.91	12.62
8	TP-08	1.50m	0.01	1.44	46.14	52.41
		3.00m	0.06	18.26	34.07	47.61
9	TP-09	1.50m	2.36	19.80	44.17	33.67
		3.00m	1.81	34.17	31.00	33.02
10	TP-10	1.50m	0.00	7.86	41.81	50.33
		3.00m	25.62	36.03	26.29	12.06

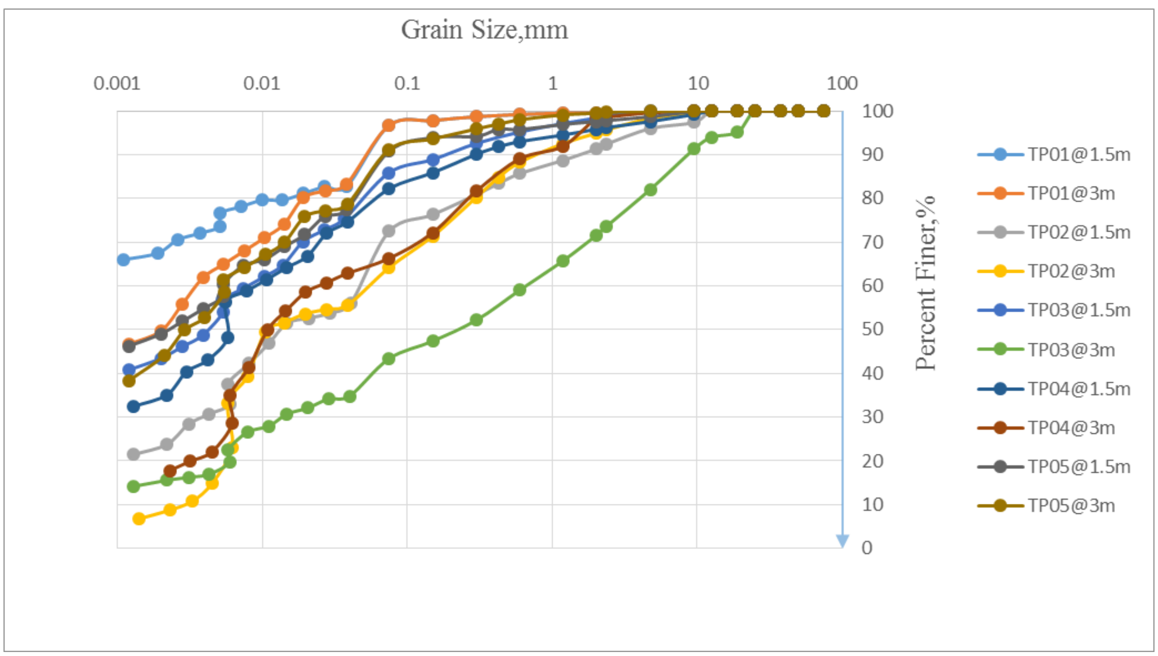


Fig 4.1 Grain size distribution curve of TP (1, 2, 3, 4, 5) @1.5m and 3m

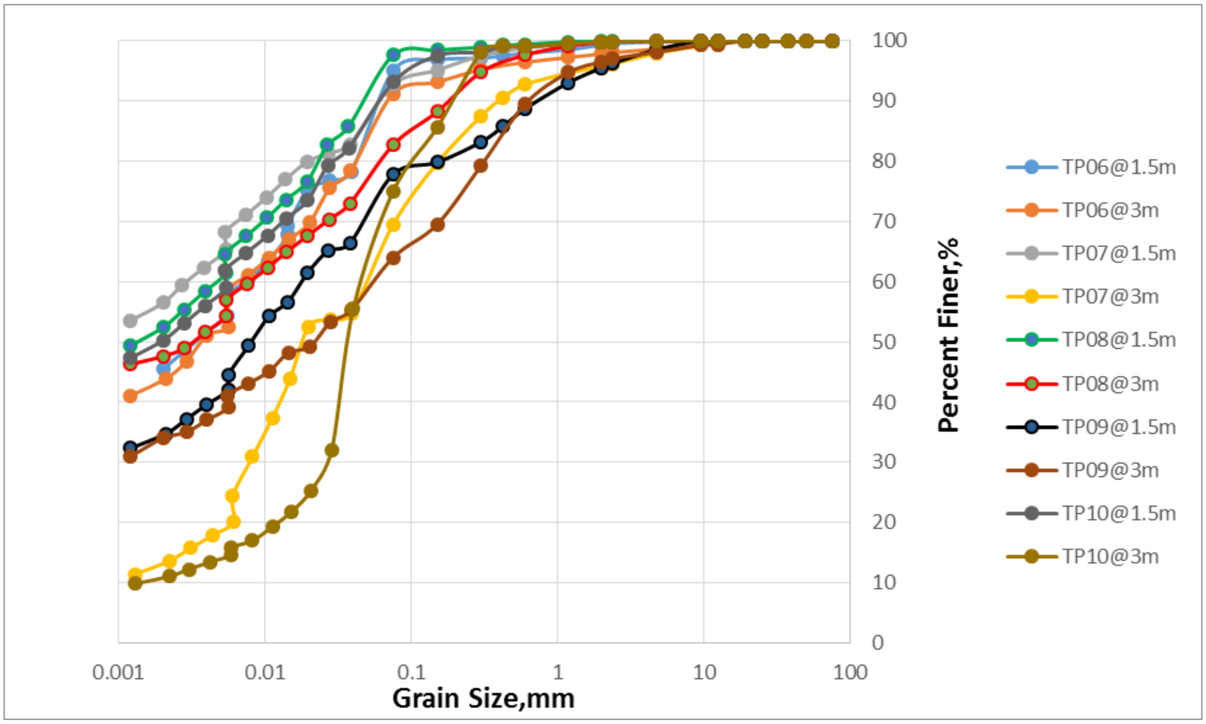


Fig 4.2 Grain size distribution curve of TP (6, 7, 8, 9, and 10) @ 1.5m and 3m.

4.1.5. Atterberg limits

Liquid limit of Fiche town falls in the range of 31-79% and plastic limit was in the range of 16-42% (Table 4.4). The plasticity index range of the soils was from 11% to 46% (Table 4.4). According to Burmister (1947) the plasticity of the soils are Medium to high plasticity. The laboratory test results of both liquid limit and plastic limit is shown below (Table 4.4) and detail analysis of test results is attached in appendix-B.

Table 4.4 Atterberg limit test results

Sr.No.	Test pit name	Depth of determination	Liquid limit,%	Plastic limit,%	Plasticity index,%
1	TP-01	1.50m	73	32	41
		3.00m	79	33	46
2	TP-02	1.50m	48	29	19
		3.00m	43	28	15
3	TP-03	1.50m	65	27	38
		3.00m	31	16	15
4	TP-04	1.50m	55	30	25
		3.00m	44	28	16
5	TP-05	1.50m	76	33	43
		3.00m	64	33	31
6	TP-06	1.50m	73	42	31
		3.00m	60	27	33
7	TP-07	1.50m	69	31	38
		3.00m	56	38	18
8	TP-08	1.50m	71	31	40
		3.00m	48	21	27
9	TP-09	1.50m	48	24	24
		3.00m	45	19	26
10	TP-10	1.50m	61	28	33
		3.00m	40	29	11

4.1.6 Classification of Soils

4.1.6.1 Unified Soil Classification System (USCS)

The classification of soils according to USCS scheme shows most of the soil of the study area falls in ML, MH, CL and CH region. From the plot of plasticity chart and the classification soils, the soils found in Fiche town are Silty and Black clay of low and higher plasticity.

Table 4.5 Soil Classification based on USCS

Sr.No.	Test pit name	Depth of determination	Percent amount of particle size				Liquid limit, %	Plastic Limit,%	Plasticity index ,%	Soil classification USCS
			Gravel	Sand	Silt	Clay				
1	TP-01	1.50m	0.33	2.86	29.30	67.51	73	32	41	CH
		3.00m	0.00	3.38	46.91	49.71	79	33	46	CH
2	TP-02	1.50m	4.03	23.45	47.75	24.77	48	29	19	ML
		3.00m	1.78	34.01	55.54	8.67	43	28	15	ML
3	TP-03	1.50m	0.01	14.13	43.31	42.55	65	27	38	CH
		3.00m	16.89	39.73	26.82	16.56	31	16	15	SC
4	TP-04	1.50m	2.41	15.40	46.09	36.10	55	30	25	MH
		3.00m	0.44	33.22	48.54	17.80	44	28	16	ML
5	TP-05	1.50m	1.41	7.81	41.73	49.05	76	33	43	CH
		3.00m	0.12	8.73	47.05	44.10	64	33	31	MH
6	TP-06	1.50m	0.18	4.83	46.54	48.45	73	42	31	MH
		3.00m	1.46	7.38	47.24	43.92	60	27	33	CH
7	TP-07	1.50m	0.04	8.25	35.25	56.46	69	31	38	CH
		3.00m	3.10	27.37	56.91	12.62	56	38	18	MH
8	TP-08	1.50m	0.01	1.44	46.14	52.41	71	31	40	CH
		3.00m	0.06	18.26	34.07	47.61	48	21	27	CL
9	TP-09	1.50m	2.36	19.80	44.17	33.67	48	24	24	CL
		3.00m	1.81	34.17	31.00	33.02	45	19	26	CL
10	TP-10	1.50m	0.00	7.86	41.81	50.33	61	28	33	CH
		3.00m	25.62	36.03	26.29	12.06	40	29	11	SM

4.1.6.2 AASHTO Soil Classification System

Soil classification of the study area based on AASHTO, all soils falls in between A-6 and A-7. So the general rating as subgrade material for all soil is poor.

Table 4.6 Soil Classification based on AASHTO

Sr.No.	Test pit name	Depth of determination	Percent amount of particle size			Liquid limit, %	Plastic limit, %	Plasticity index, %	Soil classification ASSTHO	Usual types of significant constituent materials
			No.10 (2mm)	No.40 (425µm)	No.200 (75µm)					
1	TP-01	1.50m	99.55	99.05	96.71	73	32	41	A-7-5	Clay Soils
		3.00m	99.94	99.68	97.63	79	33	46	A-7-5	Clay
2	TP-02	1.50m	91.29	83.41	72.52	48	29	19	A-7-5	Clay
		3.00m	94.86	84.76	64.21	43	28	15	A-7-6	Clay
3	TP-03	1.50m	98.42	94.15	85.86	65	27	38	A-7-6	Clay
		3.00m	71.50	56.1	43.38	31	16	15	A-6	Sand
4	TP-04	1.50m	95.81	91.79	82.19	55	30	25	A-7-6	Clay
		3.00m	98.27	86.2	66.34	44	28	16	A-7-6	Clay
5	TP-05	1.50m	97.56	95.75	90.78	76	33	43	A-7-5	Clay
		3.00m	99.52	97.03	91.15	64	33	31	A-7-5	Clay
6	TP-06	1.50m	99.32	97.54	94.99	73	42	31	A-7-5	Clay
		3.00m	97.75	96	96	60	27	33	A-7-6	Clay
7	TP-07	1.50m	99.79	98.3	92.71	69	31	38	A-7-6	Clay
		3.00m	95.69	90.62	69.53	56	38	18	A-7-5	Clay
8	TP-08	1.50m	99.90	99.17	97.55	71	31	40	A-7-5	Clay
		3.00m	99.67	96.3	82.68	48	21	27	A-7-6	Clay
9	TP-09	1.50m	95.46	85.82	77.84	48	24	24	A-7-6	Clay
		3.00m	96.53	85.8	64.02	45	19	26	A-7-6	Clay
10	TP-10	1.50m	99.76	98.95	93.14	61	28	33	A-7-6	Clay
		3.00m	61.13	46.27	38.35	40	29	11	A-7-5	Sand

4.1.7 Free Swell Test

This study shows that, the free swell test of Fiche soil ranges from 35 to 99 (Table 4.7). This shows most of the soil of Fiche town is marginal in swelling potential property. The soils are mostly in between 50% and 99% except for TP02@1.5m, TP04@1.5, 3m, TP07 @3m, TP09@1.5, 3m and TP10@ 3m. Most of the soils have an intermediate expansive nature, which have a little impact on construction of structures. The test conditions of the free swell are oven dried and summary of test results are shown in (table 4.7) below.

Table 4.7 Summary of Free swell test results

Sr. No.	Test pit name	Depth of determination	Free Swell,%
1	TP-01	1.50m	79
		3.00m	99
2	TP-02	1.50m	49
		3.00m	54
3	TP-03	1.50m	69
		3.00m	59
4	TP-04	1.50m	49
		3.00m	49
5	TP-05	1.50m	94
		3.00m	84
6	TP-06	1.50m	89
		3.00m	84
7	TP-07	1.50m	79
		3.00m	49
8	TP-08	1.50m	84
		3.00m	69
9	TP-09	1.50m	44
		3.00m	35
10	TP-10	1.50m	94
		3.00m	49

4.1.8 Compaction Test

From the test results the maximum dry density (MDD) of Fiche town ranges from1.35 g/cm^3 to 1.420 g/cm^3 and the optimum moisture content ranges 28% to 36%. The summary of the test result is shown in (Table 4.8). The detail tests are presented in Appendix-C.

Table 4.8 Summary of Optimum moisture content and the maximum dry density

Serial No	Designation	Depth(m)	OMC (%)	MDD (g/cm3)
1	TP01	1.50m	35	1.39
		3.00m	36	1.37
2	TP02	1.50m	36	1.35
		3.00m	31	1.41
3	TP03	1.50m	28	1.42
		3.00m	34	1.36
4	TP04	1.50m	28	1.40
		3.00m	32	1.36
5	TP05	1.50m	34	1.37
		3.00m	34	1.39
6	TP06	1.50m	29	1.38
		3.00m	30	1.42
7	TP07	1.50m	32	1.37
		3.00m	28	1.39
8	TP08	1.50m	35	1.38
		3.00m	33	1.41
9	TP09	1.50m	34	1.36
		3.00m	30	1.35
10	TP10	1.50m	33	1.37
		3.00m	34	1.40

4.1.9 Unconfined compression strength (UCS) test

Table 4.9 shows the summary of the unconfined compressive strength and cohesion result of soils for the area under study. Figure 4.3 indicate the graph of unconfined compressive strength of Fiche town area. It is observed that the consistency of Fiche soil is either stiff or very stiff. The detail tests are presented in Appendix-D.

Table 4.9.Summary of unconfined compressive strength and cohesion

Sr No.	Test pit name	Depth	Unconfined strength, q_u , (kPa)	Cohesion, $c_u = q_u/2$ (kPa)	Consistency
1	TP01	3.00m	177	87.5	Stiff
2	TP02	3.00m	84	42.5	Medium
3	TP03	3.00m	77	38.0	Medium
4	TP04	3.00m	133	68	Stiff
5	TP05	3.00m	204	102	Very stiff
6	TP06	3.00m	234	117	Very stiff
7	TP07	3.00m	120	60	Stiff
8	TP08	3.00m	115	56	Stiff
9	TP09	3.00m	193	97	Stiff
10	TP10	3.00m	142	72	Stiff

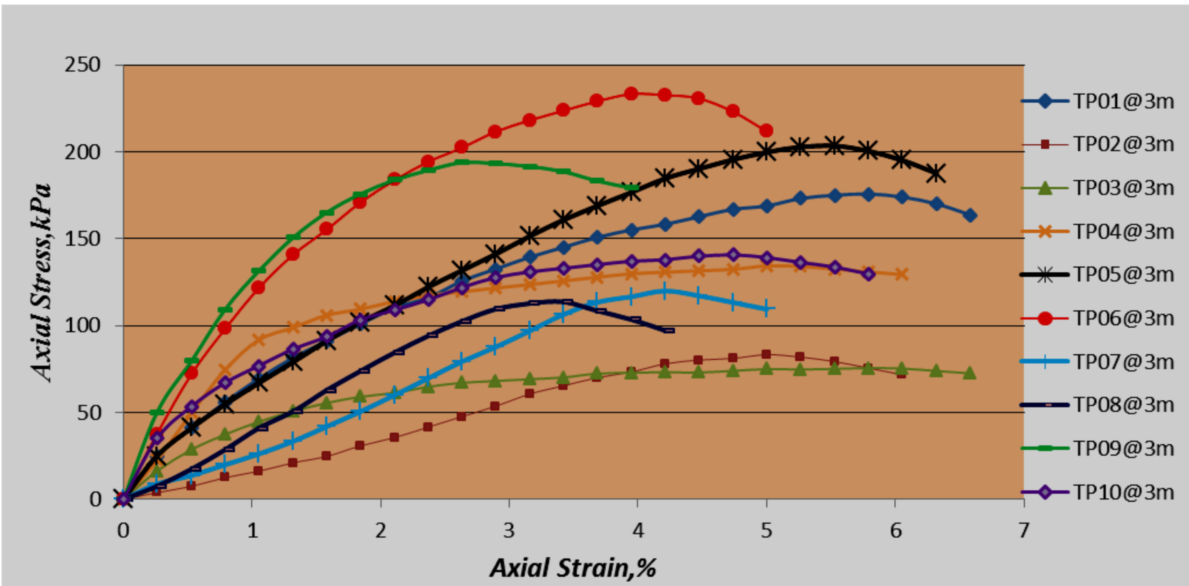


Figure 4.3. Unconfined compressive strength of (TP01, TP02, TP03, TP04, TP05, TP06, TP07, TP08, TP09 and TP10) at 3m

4.1.10 Consolidation Test

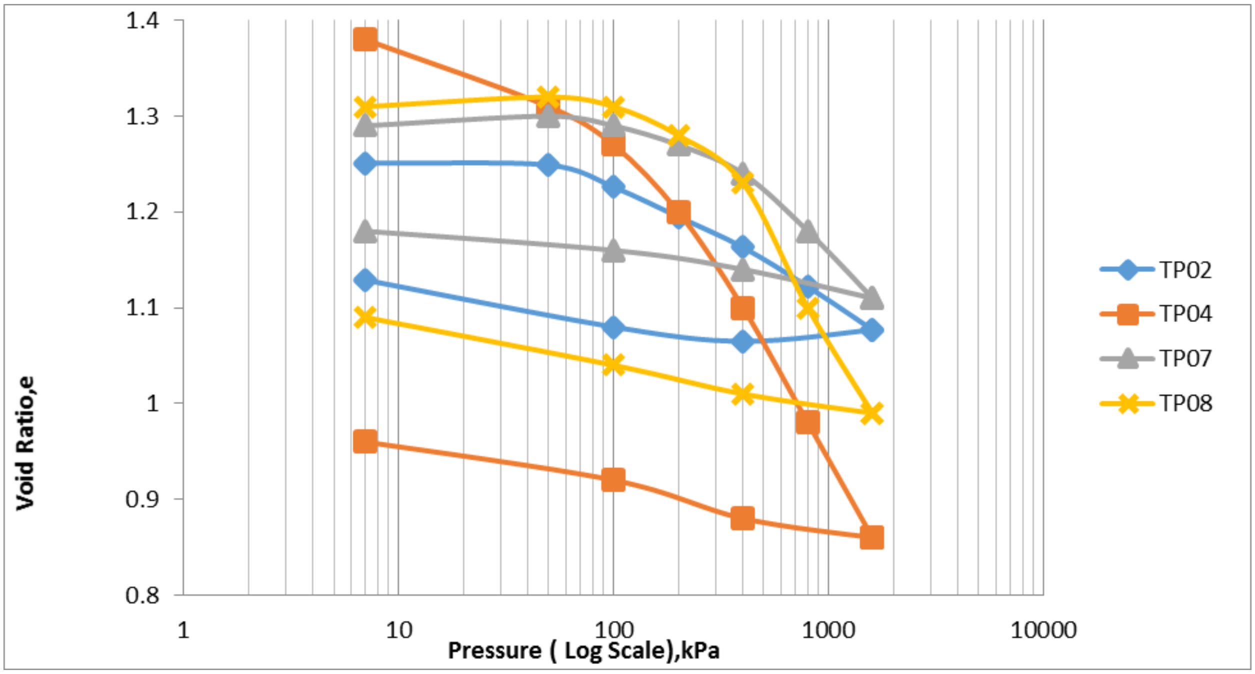


Fig 4.4 Plot of vertical effective stress versus void ratio on semi-log scale

Fig 4.4 shows a typical schematic diagram how to determine the compression index from the void ratio versus applied effective pressure. The compression index of the area under study is summarized in Table 4.10.

From the one dimensional consolidation test result of soil found at Fiche town, the compression index is in the range of 0.20-0.38.

4.1.10.1 Coefficient of consolidation

The square time method is used to determine the coefficient of consolidation and plot is shown below in Fig 4.5.

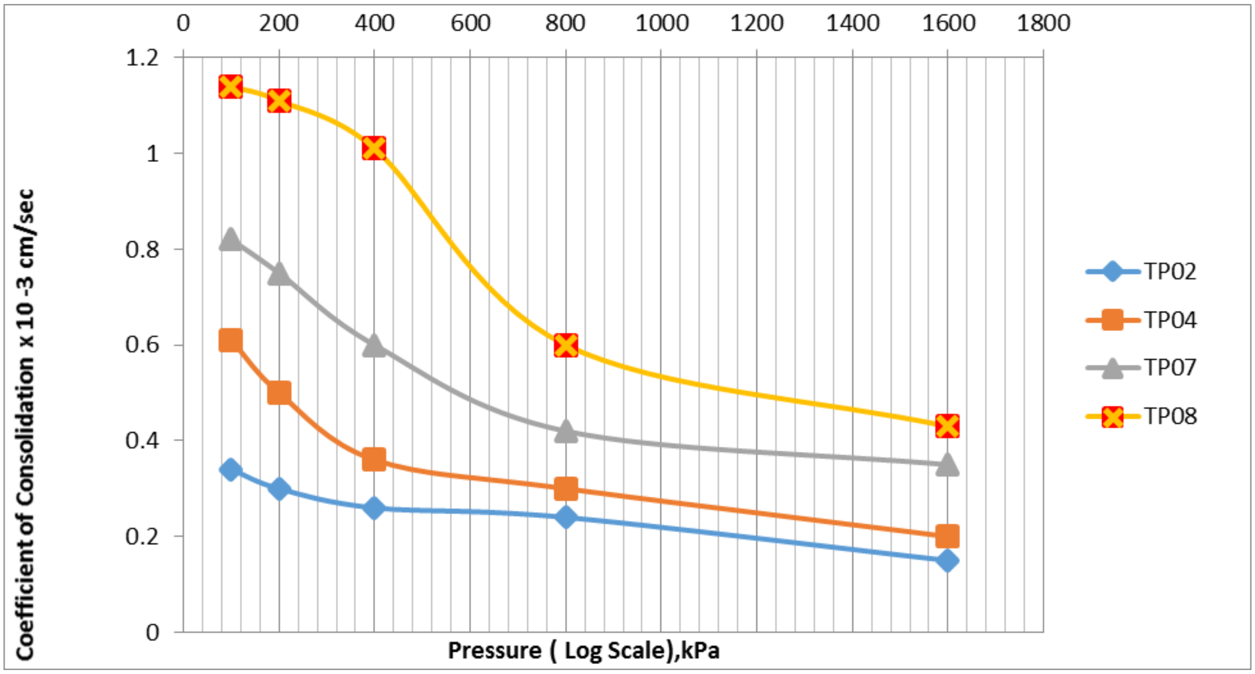


Fig 4.5 Coefficient of Consolidation Vs. Pressure

Table 4.10 Summary of consolidation parameters for TP02, TP04, TP07 and TP08

Sr.No.	Test pit name	Depth (m)	Natural Moisture content (%)	Pressure, P, kPa	Void ratio, e_f	Coefficient of consolidation, Cv,10^-3 cm2/sec	Coefficient of Compression, a_v, 10^-5 m2/sec	Compression index, Cc	Recompression index, Cs
1	TP02	3	49.6	7	1.277			0.223	0.018
				50	1.215	3.533			
				100	1.190	4.743	39.00		
				200	1.151	3.533	28.50		
				400	1.108	2.174	24.00		
				800	1.070	1.570	30.00		
				1600	1.011	1.002	13.25		
2	TP04	3	46.96	7	1.38			0.38	0.069
				50	1.31				
				100	1.27	0.31	95.00		
				200	1.20	0.30	65.00		
				400	1.10	0.28	48.00		
				800	0.98	0.26	31.00		
				1600	0.86	0.14	16.00		
3	TP07	3	43.64	7	1.32			0.20	0.008
				50	1.30				
				100	1.29	1.54	27.00		
				200	1.27	1.42	20.00		
				400	1.24	0.83	15.00		
				800	1.18	0.42	13.00		
				1600	1.11	0.35	8.50		
4	TP08	3	40.48	7	1.33	-	-	0.32	0.062
				50	1.32	-	-		
				100	1.31	1.14	40.00		
				200	1.28	1.11	29.50		
				400	1.23	1.01	25.00		
				800	1.10	0.60	31.50		
				1600	0.99	0.43	14.25		

4.1.10.2. Pre-consolidation pressure

Table 4.11 Summary of pre-consolidation pressure for TP02, TP04, TP07 and TP08

Sr.No.	Test pit name	Depth (m)	Natural Moisture content (%)	Pressure P, kPa	Void ratio, e_f	Overburden Pressure, P_0, kPa	Pre-consolidation pressure, P_c	Over consolidation (OCR)
1	TP02	3	49.6	7	1.277	48.6	85	1.749
				50	1.215			
				100	1.190			
				200	1.151			
				400	1.108			
				800	1.070			
				1600	1.011			
2	TP04	3	46.96	7	1.38	52.17	145	2.78
				50	1.31			
				100	1.27			
				200	1.20			
				400	1.10			
				800	0.98			
				1600	0.86			
3	TP07	3	43.64	7	1.32	52.25	285	5.45
				50	1.30			
				100	1.29			
				200	1.27			
				400	1.24			
				800	1.18			
				1600	1.11			
4	TP08	3	40.48	7	1.33	49.07	260	5.30
				50	1.32			
				100	1.31			
				200	1.28			
				400	1.23			
				800	1.10			
				1600	0.99			

4.2. Comparisons of Laboratory Test Results with Previously Done Researches

4.2.1. Discussions of the Laboratory Test Results of the Study Area

The study area comprises of different geological formation, terrain conditions and climatic conditions. Fiche town is located at higher altitude ranging 2,738 - 2,782 meters above mean sea level. The altitude varies from place to place in which the northern part is at higher elevation and the newly constructed places (expansion places) are at relatively lower altitudes which means at central parts of the town. The soil profile of the town varies from place to place mainly black, reddish brown and gray soils covers different parts of the town.

The climate of a given place is the reflection of different factors such as latitude, altitude, pressure difference between high lands and low lands and distance from the Sea as well as the impact of topography and cloud cover. If one looks at the climatic condition of Fiche town and its surrounding areas, it can be concluded that the climate of Fiche is the reflection of altitude, cloud cover, impacts of winds from high land areas and rugged topography. Because of the impact of altitude, the agro climatic zone of the town is categorized as temperate climate because temperate climate prevails at the altitudinal limit the town is located.

In situ property test result of soil shows the moisture content of the study area is on the range of 20.01%-58.89%. As we see from the (table 4.1), the moisture content increases with depth for some samples, these are due to water holding capacity of soil particles and variability of climate. This is also related to evapo-transpiration, it is decreasing when we go down.

The specific gravity value varies between 2.62 and 2.88. Since the specific gravity is above 2.62 inorganic soils are dominant, as most of organic soils contain a value of less than 2.40. This range of specific gravity is similar to most of silty and clay soils as determined in different literatures.

The grain size analysis of the area under study showed the soil contains gravel 0-25.62%, sand 1.44%-39.73%, silt 26.29%-55.54% and clay in between 8.67% to 67.51%. This analysis shows that the dominant soil types in the area are silt and clay type soils. From the grain size distribution results, it is observed that there is a range of variation of the particle sizes. Clays and silts constitute over 80% of particles. Clay size particles are dominant in each test pits. Each sample contains a higher percentage of silt than the sand fractions. In each test pits the proportion of clay generally decreased with increasing depth for some samples. This implies that the fines content decreases with depth as the degree of weathering decreases. A small variation of silt and sand size fractions at different depths was also observed in each test pits due to the natural variability of the samples.

From the Atterberg limits and Indices, the liquid limit of the soils is in the range of 31%-79%. Higher value of liquid limit for the soil is at test pit No.1 at 3m. From the laboratory test result the Plastic limit of the soils ranges in between 16% to 42%. The difference of liquid limit and plastic limit which is plasticity index (PI) for the area under study ranges between 10%-45% as shown in (table 4.4).

After conducting the grain size analysis and Atterberg limits, soil classification is made, as shown in (table 4.5) by USCS and (table 4.6) by AASHTO classification system. Soils classification by USCS shows the soil contains around 41% CH, 18% CL, 18% MH, 14% ML, 4.5% SM and 4.5% SC. This indicated that a clay soil in the area under study is about 58% and Silty is around 32% both of which are dominants in the area.

AASHTO classification system showed that the soils are classified in either of A-6 and A-7. By this classification system A-6 is around 4.55%, A-7-5 is around 50% and A-7-6 is about 45.45%. It is clearly indicated that the soil is poor to be used for sub grade material as per the AASTHO recommendation for suitability of soils as sub grade material.

Free swell test results are summarized in Table 4.7. From this table it can be observed that the free swell of soils under investigation ranges in between 35% to 99%. This shows that the soil expansiveness property ranges from low to marginal degree of expansiveness.

The UCS test was conducted for undisturbed samples and summary of test results is given on Table 4.9. From UCS test of soils, the value of $\mathbf{q}_u$ is in between 77kPa to 234 kPa and the amount of cohesion lies in between 38 kPa to 117 kPa. Consistency is also determined based on the UCS result and it is found that the soil is medium, stiff or very stiff according to [10].

Consolidation test was conducted on four different types of soils which is taken based on the classification result. The compression index is computed from void ratio versus log of effective stress and it is tabulated on Table 4.10. The range of compression index is in between 0.20-0.38. Higher value of compression index means large settlement may occur under applied effective stress as indicated for soil found at test pit (TP6-3m). Test pit no. 7 shows less value of compression index.

The coefficient of consolidation, *Cv*, is determined from the compression dial reading versus square root of time for each incremental loading and it is plotted as a function of effective stress in Fig 4.5. The value of *Cv* as described in the table is in the range of 0.14x10-3 to 4.743x10-3 cm2/sec. It can be observed that the four curves are almost similar in shape. But for test pit number 7 the value of coefficient of consolidation is smaller than the test pit number 2, 4 and 8. This shows the compressibility of any soil type varies with density, history of previous loading, handling prior to and during compression, and in the magnitude of stress increment relative to the existing loading any point.

Pre consolidation pressure of the soil found in the area under study was calculated from the void ratio versus effective pressure plotted on semi-log scale shown on Fig 4.4. Table 4.11 gives Summary of pre-consolidation pressure and the values of pre-consolidation pressure lies between 85 kPa to 285 kPa. Comparison is made between the current effective pressure and the pre-consolidation pressure and it is found that the over consolidation ratio is greater than one. Hence the soil is over consolidated in its natural state.

4.2.2. Comparison of the Laboratory Test Results with Other Researches

There are numerous researches conducted on expansive and red clay soils found in Ethiopia and worldwide. Thus comparison of test result conducted in this research is made with other researches to visualize the similarity and difference.

The soil in Fiche must be compared with silt and clay soils. Soils with low plasticity inorganic silt and soils with medium to high plasticity inorganic clay were compared with the study area’s soils. Table 4.12 shows the comparison of test result.

Table 4.12 Comparison of test result with other researches

Sr.No	Researcher Name	Morin and Parry	Dagachew Debebe,2011	Haile Mariam,1992	Fasil Abagena, 2003	Adem Ebrahim,2014	Current research
	Location	Ethiopia	Adama	Addis Ababa	Bahir Dar	Debre Markos	Fiche
	Soil type	Black clay	Silt & silt sand	Red Clay	Red clay	Red clay soil	Silt and Clay
1	Specific gravity	2.62-2.94	2.4-2.7	2.61-2.79	2.75-2.83	2.69-2.84	2.62-2.81
2	Free swell		18-50	10.0-40.0	8.0-13.0	30-180	35-100
3	Clay content (%)	13-75	5.4-40.5	48.0-73.0	74-82	50-72	8.77-67.50
4	Liquid limit (%)	37-88	29-73	54.0-81.0	61-68	45-68	31-80
5	Plasticity index (%)	11-48	5.0-34.0	21.0-30.0	24-31	14-40	13-46
6	qu,kN/m2	96.7-267			148-220		75-233
7	Compression index,Cc		0.33-0.40		0.2658-0.4056		0.20-0.38
8	Classification		SM,ML.MH		MH	MH,CH,CL	MH, ML,CH,CL

As we see from table above (table 4.12), the free swell test of Fiche soil ranges from 35 to 99. This shows most of the soil of Fiche town is marginal in swelling potential property. The soils are mostly in between 50% and 100% except for some test pits which are typically black clay soils. Most of the soils have an intermediate expansive nature, which have a little impact on construction of structures.

In the other hand, the free swell test result of Fiche soils show similar properties with that of red clay soils found in Debre Markos town. From comparison table 4.12 the silt and clay soils of Fiche town have similarity in properties with that of Debre Markos soils as it is compared to other clay soils. From their engineering properties and test results, the soils of Fiche town are classified as MH, ML, CH and CL soils while the soils of Debre Markos are classified as MH, CH and CL soils.

Generally, table 4.12 show the average values of various tests done at different parts of countries, i.e., Sieve analysis, Liquid Limit, Plastic Index and specific gravities showing different properties. As indicated in the above table Fiche soils show medium plasticity (clay content) as compared to other towns. The data indicate that there is a considerable similarity of Fiche soils with in the physical properties of Addis Ababa and Debre Markos town soils.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

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

- The moisture content of the soil found in Fiche town range from 20.11%-58%. The specific gravity of the soil is in between 2.62 and 2.88 which is common for most of soil types. Grain size analysis result shows the soil under investigation is dominantly silt and clay types through which about 32% is silt soil and 59% clay soils. Atterberg limit test shows the liquid limit of the soils is in the range of 31%-80%, Plastic limit falls in between 17% to 43% and plasticity index lies between 11%-46%.
- Soils classification by USCS shows the soil contains around 41% CH, 18% CL, 18% MH, 14% ML, 4.5% SM and 4.5% SC. This indicated that the clay content in the area under study is about 59% and silt content is around 32% both of which are dominants in the area. AASHTO classification system shows the soils are classified in either of A-6 and A-7 (A-7-5, A-7-6). The soil is poor to be used for sub grade material as per the AASHTO recommendation for suitability of soils as sub grade material. Free swell test results of soils under investigation ranges in between 35% to 100%. This shows that the soil expansiveness property ranges from low to marginal degree of expansiveness.
- Unconfined Compressive Strength (UCS) test indicates the value of qu is in between 75kPa to 233 kPa and the amount of cohesion lies in between 37.5 kPa to 116.5 kPa. Based on the UCS result the consistency of soils is medium to very stiff.
- From consolidation test the amount of compression index, Cc, is in between 0.20-0.38. And the value of consolidation coefficient, Cv, is 0.14x10-3 to 4.743x10-3 cm2/sec. Coefficient of permeability from consolidation test gives 1.24x10-9 to 22.17x10-9 cm2/sec. This shows the soil is almost impermeable. The amount of pre consolidation pressure of the soil found in the area under study is determined and it

ranges between 85 kPa to 285 kPa. Since the over-consolidation ratio is greater than one, the soil is over consolidated in its natural state.

- The average values of various tests done at different parts of countries, i.e., Sieve analysis, Liquid Limit, Plastic Index and specific gravities showing different properties. Fiche soils show medium plasticity (clay content) as compared to other towns. The data indicate that there is a considerable similarity of Fiche soils with in the physical properties of Addis Ababa and Debre Markos town soils.

5.2 Recommendations

- Fiche town is growing at faster rate. Hence one can take large number of samples to determine the engineering and index property of the soils of the town.

- Some part of the town especially areas at the center of the town i.e. around bus station and eastern part of the town is covered dominantly by rock. Thus additional detailed geological study on those areas is highly recommended.

- The dynamic properties of the soil in Fiche town are not studied. Therefore, the dynamic characteristics of soils with relation to respond of soil to earthquake should be studied in future.

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APPENDIX

Appendix– A

Grain Size Distribution Curves

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m
Sample No: TP01 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3``	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2``	50.0	1190.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1164.6	0.0	0.0	0.0	100.0
No 4	4.75	1262.7	1266.2	3.5	0.4	0.4	99.7
No 8	2.36	990.3	991.0	0.7	0.1	0.4	99.6
No 10	2	944.1	944.4	0.3	0.0	0.5	99.6
No 16	1.18	894.3	895.3	1.0	0.1	0.6	99.5
No 30	0.6	833.8	836.1	2.3	0.2	0.8	99.2
No 50	0.3	750.1	755.3	5.2	0.5	1.3	98.7
No 100	0.15	782.4	791.8	9.4	0.9	2.2	97.8
No 200	0.075	764.4	774.9	10.5	1.1	3.3	96.7
pan	-------	736.0	736.0	0.0	0.0	3.3	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.62 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.01318	0.0381	85.45	82.64
1	1.0300	-0.0027	1.0273	8.36	0.01318	0.0270	85.45	82.64
2	1.0295	-0.0027	1.0268	8.50	0.01318	0.0192	83.88	81.12
4	1.0290	-0.0027	1.0263	8.63	0.01318	0.0137	82.32	79.61
8	1.0290	-0.0027	1.0263	8.63	0.01318	0.0100	82.32	79.61
15	1.0285	-0.0027	1.0258	8.76	0.01318	0.0071	80.75	78.10
30	1.0280	-0.0027	1.0253	8.89	0.01318	0.0051	79.19	76.58
60	1.0270	-0.0027	1.0243	9.16	0.01318	0.0051	76.06	73.56
120	1.0265	-0.0027	1.0238	9.29	0.01318	0.0037	74.49	72.04
240	1.0260	-0.0027	1.0233	9.42	0.01318	0.0026	72.93	70.53
480	1.0250	-0.0027	1.0223	9.69	0.01318	0.0019	69.80	67.50
1440	1.0245	-0.0027	1.0218	9.82	0.01318	0.0011	68.23	65.99

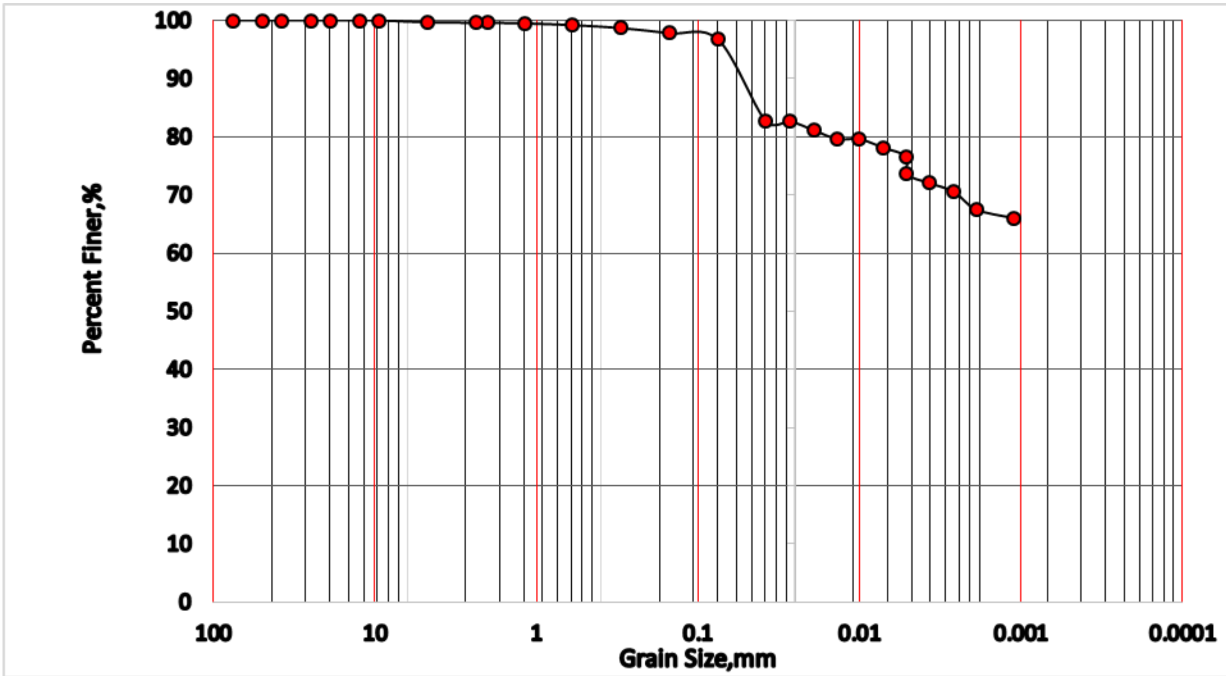


Figure A.1 Combined grain size analysis test result for TP01@1.5m

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

	Gravel	Sand	Silt	Clay	Colloids

75mm 4.75mm 0.075mm 0.005mm 0.001mm

Grain Size Analysis/Sieve Analysis/
Sample No: TP01

Sample Depth: 3.00m
Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2’’	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	427.9	0.0	0.0	0.0	100.0

No 8	2.36	387.7	388.2	0.5	0.1	0.1	100.0
No 10	2	390.2	390.3	0.1	0.0	0.1	99.9
No 16	1.18	372.6	373.2	0.6	0.1	0.1	99.9
No 30	0.6	325.6	326.6	1.0	0.1	0.2	99.8
No40	0.425	291.7	292.7	1.0	0.1	0.3	99.7
No 50	0.3	301.5	304.0	2.5	0.3	0.6	99.4
No 100	0.15	271.4	278.4	7.0	0.7	1.3	98.7
No 200	0.075	273.8	284.8	11.0	1.1	2.4	97.6
pan	-------	736.0	736.0	0.0	0.0	2.4	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.68 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.01314	0.0380	85.27	83.25
1	1.0295	-0.0027	1.0268	8.50	0.01314	0.0271	83.71	81.73
2	1.0290	-0.0027	1.0263	8.63	0.01314	0.0193	82.15	80.20
4	1.0270	-0.0027	1.0243	9.16	0.01314	0.0141	75.90	74.10
8	1.0260	-0.0027	1.0233	9.42	0.01314	0.0104	72.78	71.05
15	1.0250	-0.0027	1.0223	9.69	0.01314	0.0075	69.66	68.01
30	1.0240	-0.0027	1.0213	9.95	0.01314	0.0054	66.53	64.96
60	1.0230	-0.0027	1.0203	10.22	0.01314	0.0054	63.41	61.91
120	1.0210	-0.0027	1.0183	10.75	0.01314	0.0039	57.16	55.81
240	1.0200	-0.0027	1.0173	11.01	0.01314	0.0028	54.04	52.76
480	1.0190	-0.0027	1.0163	11.27	0.01314	0.0020	50.91	49.71
1440	1.0180	-0.0027	1.0153	11.54	0.01314	0.0012	47.79	46.66

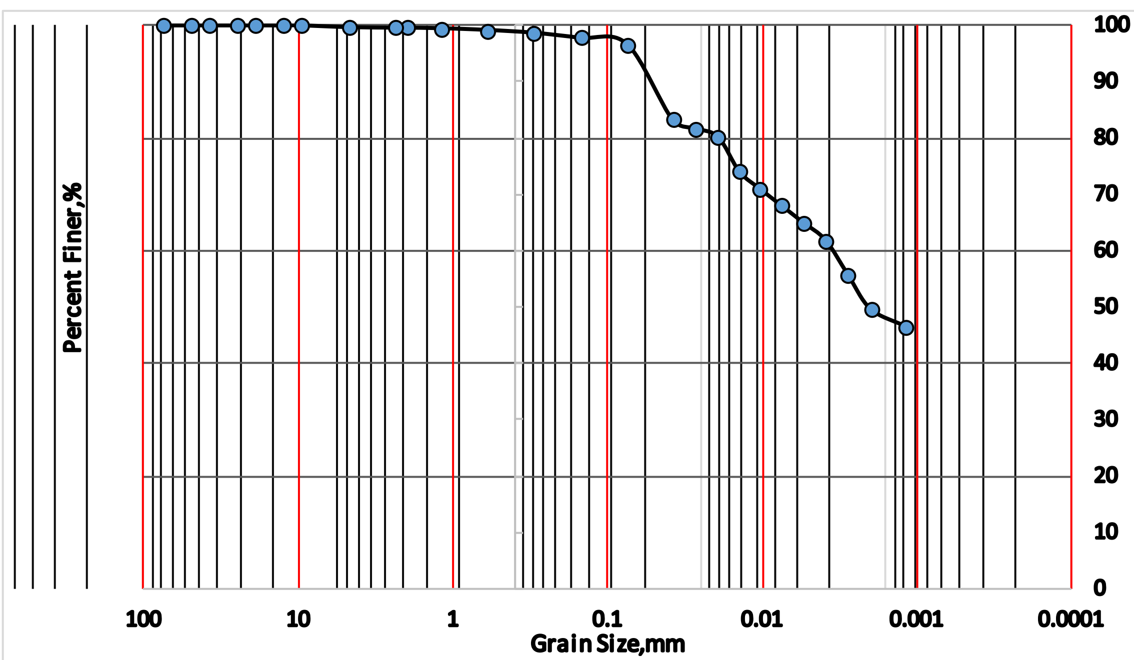


Figure A.2 Combined grain size analysis test result for TP01@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m
Sample No: TP02 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2’’	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	454.8	480.6	25.8	2.6	2.6	97.4
No 4	4.75	427.8	442.4	14.6	1.5	4.0	96.0
No 8	2.36	387.7	423.7	36.0	3.6	7.6	92.4
No 10	2	377.9	388.6	10.7	1.1	8.7	91.3
No 16	1.18	372.6	398.3	25.7	2.6	11.3	88.7
No 30	0.6	323.3	354.0	30.7	3.1	14.4	85.7
No 40	0.425	292.1	314.5	22.4	2.2	16.6	83.4
No 50	0.3	301.6	326.3	24.7	2.5	19.1	80.9
No 100	0.15	276.0	322.6	46.6	4.7	23.7	76.3
No 200	0.075	258.6	296.2	37.6	3.8	27.5	72.5
pan	-------	736.0	736.0	0.0	0.0	27.5	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.63 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0270	-0.0027	1.0243	9.16	0.013482	0.0408	77.36	56.10
1	1.0260	-0.0027	1.0233	9.42	0.013482	0.0293	74.17	53.79
2	1.0255	-0.0027	1.0228	9.55	0.013482	0.0208	72.58	52.64
4	1.0250	-0.0027	1.0223	9.69	0.013482	0.0148	70.99	51.48
8	1.0230	-0.0027	1.0203	10.22	0.013482	0.0111	64.62	46.87
15	1.0210	-0.0027	1.0183	10.75	0.013482	0.0081	58.26	42.25
30	1.0190	-0.0027	1.0163	11.27	0.013482	0.0058	51.89	37.63
60	1.0170	-0.0027	1.0143	11.80	0.013482	0.0060	45.52	33.01
120	1.0160	-0.0027	1.0133	12.07	0.013482	0.0043	42.34	30.70
240	1.0150	-0.0027	1.0123	12.33	0.013482	0.0031	39.16	28.40
480	1.0130	-0.0027	1.0103	12.86	0.013482	0.0022	32.79	23.78
1440	1.0120	-0.0027	1.0093	13.13	0.013482	0.0013	29.61	21.47

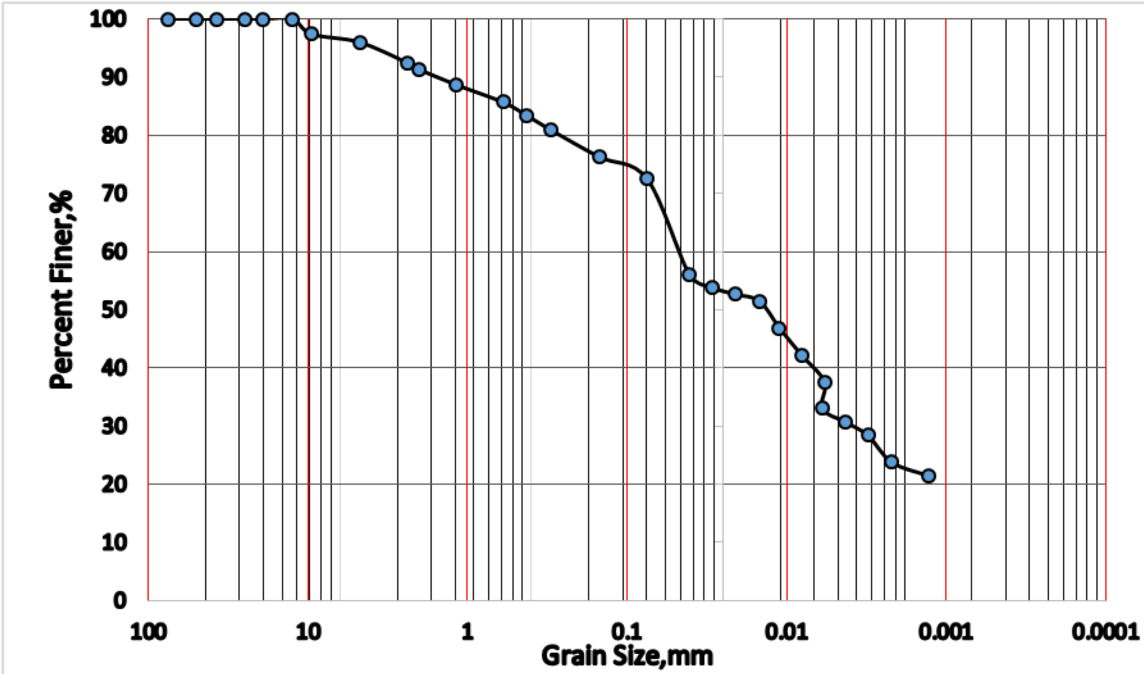


Figure A.3 Combined grain size analysis test result for TP02@1.5m

Grain Size Analysis/Sieve Analysis/
Sample No: TP02

Sample Depth: 3.00m
Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	454.8	458.4	3.6	0.4	0.4	99.6
No 4	4.75	427.8	441.0	13.2	1.3	1.7	98.3
No 8	2.36	387.7	414.3	26.6	2.7	4.3	95.7
No 10	2	377.9	385.9	8.0	0.8	5.1	94.9
No 16	1.18	372.6	396.6	24.0	2.4	7.5	92.5
No 30	0.6	323.3	364.5	41.2	4.1	11.7	88.3
No 40	0.425	292.1	327.9	35.8	3.6	15.2	84.8
No 50	0.3	301.6	346.3	44.7	4.5	19.7	80.3
No 100	0.15	276.0	364.8	88.8	8.9	28.6	71.4
No 200	0.075	258.6	330.6	72.0	7.2	35.8	64.2
pan	-------	736.0	736.0	0.0	0.0	35.8	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.67 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.01344	0.0389	86.72	55.68
1	1.0295	-0.0027	1.0268	8.50	0.01344	0.0277	85.13	54.66
2	1.0290	-0.0027	1.0263	8.63	0.01344	0.0197	83.54	53.64
4	1.0280	-0.0027	1.0253	8.89	0.01344	0.0142	80.36	51.60
8	1.0270	-0.0027	1.0243	9.16	0.01344	0.0105	77.19	49.56
15	1.0220	-0.0027	1.0193	10.48	0.01344	0.0079	61.31	39.36
30	1.0190	-0.0027	1.0163	11.27	0.01344	0.0058	51.78	33.25
60	1.0140	-0.0027	1.0113	12.60	0.01344	0.0062	35.89	23.05
120	1.0100	-0.0027	1.0073	13.65	0.01344	0.0045	23.19	14.89
240	1.0080	-0.0027	1.0053	14.18	0.01344	0.0033	16.84	10.81
480	1.0070	-0.0027	1.0043	14.45	0.01344	0.0023	13.66	8.77
1440	1.0060	-0.0027	1.0033	14.71	0.01344	0.0014	10.48	6.73

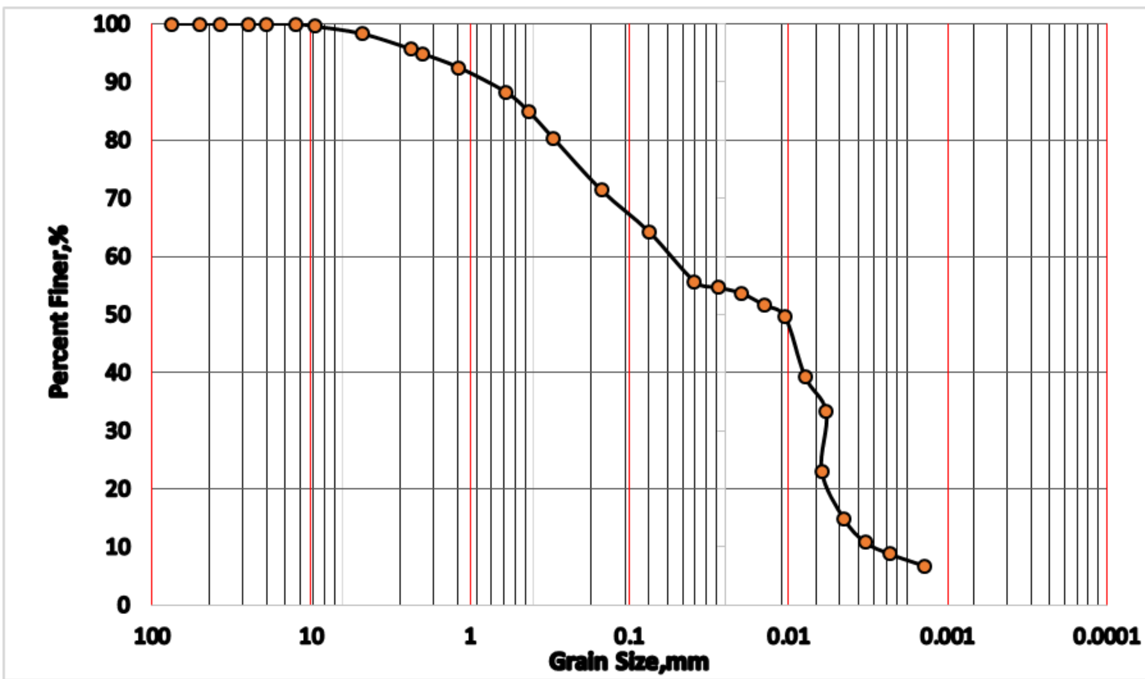


Figure A.4 Combined grain size analysis test result for TP02@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m
Sample No: TP03 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1164.6	0.0	0.0	0.0	100.0
No 4	4.75	1262.7	1263.8	1.1	0.1	0.1	99.9
No 8	2.36	990.2	1000.5	10.3	1.0	1.1	98.9
No 10	2	944.1	948.5	4.4	0.4	1.6	98.4
No 16	1.18	894.5	908.2	13.7	1.4	2.9	97.1
No 30	0.6	833.7	851.8	18.1	1.8	4.8	95.2
No 50	0.3	750.3	776.8	26.5	2.7	7.4	92.6
No 100	0.15	782.7	819.5	36.8	3.7	11.1	88.9
No 200	0.075	765.1	795.6	30.5	3.1	14.1	85.9
pan	-------	736.0	736.0	0.0	0.0	14.1	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.71 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0310	-0.0027	1.0283	8.10	0.013034	0.0371	87.87	75.45
1	1.0300	-0.0027	1.0273	8.36	0.013034	0.0267	84.77	72.78
2	1.0290	-0.0027	1.0263	8.63	0.013034	0.0191	81.66	70.11
4	1.0270	-0.0027	1.0243	9.16	0.013034	0.0139	75.45	64.78
8	1.0260	-0.0027	1.0233	9.42	0.013034	0.0103	72.35	62.12
15	1.0250	-0.0027	1.0223	9.69	0.013034	0.0074	69.24	59.45
30	1.0240	-0.0027	1.0213	9.95	0.013034	0.0053	66.14	56.78
60	1.0230	-0.0027	1.0203	10.22	0.013034	0.0054	63.03	54.12
120	1.0210	-0.0027	1.0183	10.75	0.013034	0.0039	56.82	48.79
240	1.0200	-0.0027	1.0173	11.01	0.013034	0.0028	53.72	46.12
480	1.0190	-0.0027	1.0163	11.27	0.013034	0.0020	50.61	43.45
1440	1.0180	-0.0027	1.0153	11.54	0.013034	0.0012	47.51	40.79

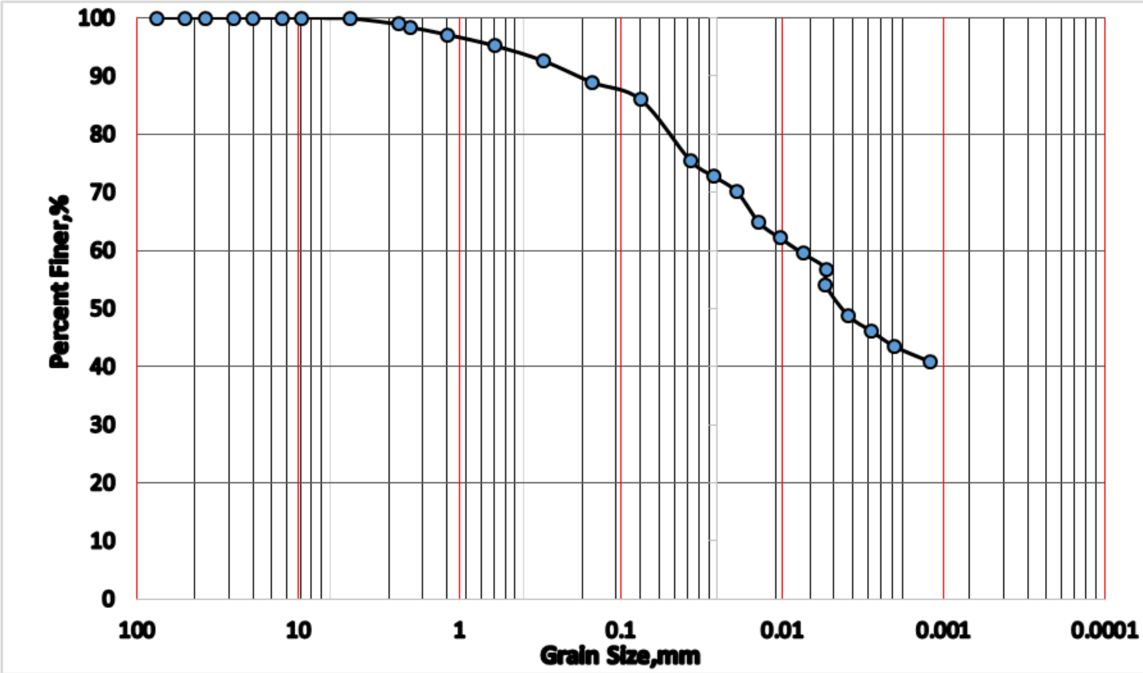


Figure A.5 Combined grain size analysis test result for TP03@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m
Sample No: TP03 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2”	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1226.9	48.5	4.9	4.9	95.2
1/2"	12.5	1216.7	1227.8	11.1	1.1	6.0	94.0
3.8"	9.5	1164.6	1192.5	27.9	2.8	8.8	91.3
No 4	4.75	1262.7	1354.0	91.3	9.1	17.9	82.1
No 8	2.36	990.3	1075.4	85.1	8.5	26.4	73.6
No 10	2	944.1	965.2	21.1	2.1	28.5	71.5
No 16	1.18	894.3	952.3	58.0	5.8	34.3	65.7
No 30	0.6	833.8	900.1	66.3	6.6	40.9	59.1
No 50	0.3	750.1	818.9	68.8	6.9	47.8	52.2
No 100	0.15	782.4	830.6	48.2	4.8	52.6	47.4
No 200	0.075	764.4	804.3	39.9	4.0	56.6	43.4
pan	-------	736.0	736.0	0.0	0.0	56.6	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.65 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0280	-0.0027	1.0253	8.89	0.01344	0.0401	80.36	34.86
1	1.0275	-0.0027	1.0248	9.03	0.01344	0.0286	78.78	34.17
2	1.0260	-0.0027	1.0233	9.42	0.01344	0.0206	74.01	32.11
4	1.0250	-0.0027	1.0223	9.69	0.01344	0.0148	70.84	30.73
8	1.0230	-0.0027	1.0203	10.22	0.01344	0.0111	64.48	27.97
15	1.0220	-0.0027	1.0193	10.48	0.01344	0.0079	61.31	26.59
30	1.0190	-0.0027	1.0163	11.27	0.01344	0.0058	51.78	22.46
60	1.0170	-0.0027	1.0143	11.80	0.01344	0.0060	45.42	19.70
120	1.0150	-0.0027	1.0123	12.33	0.01344	0.0043	39.07	16.95
240	1.0145	-0.0027	1.0118	12.46	0.01344	0.0031	37.48	16.26
480	1.0140	-0.0027	1.0113	12.60	0.01344	0.0022	35.89	15.57
1440	1.0130	-0.0027	1.0103	12.86	0.01344	0.0013	32.72	14.19

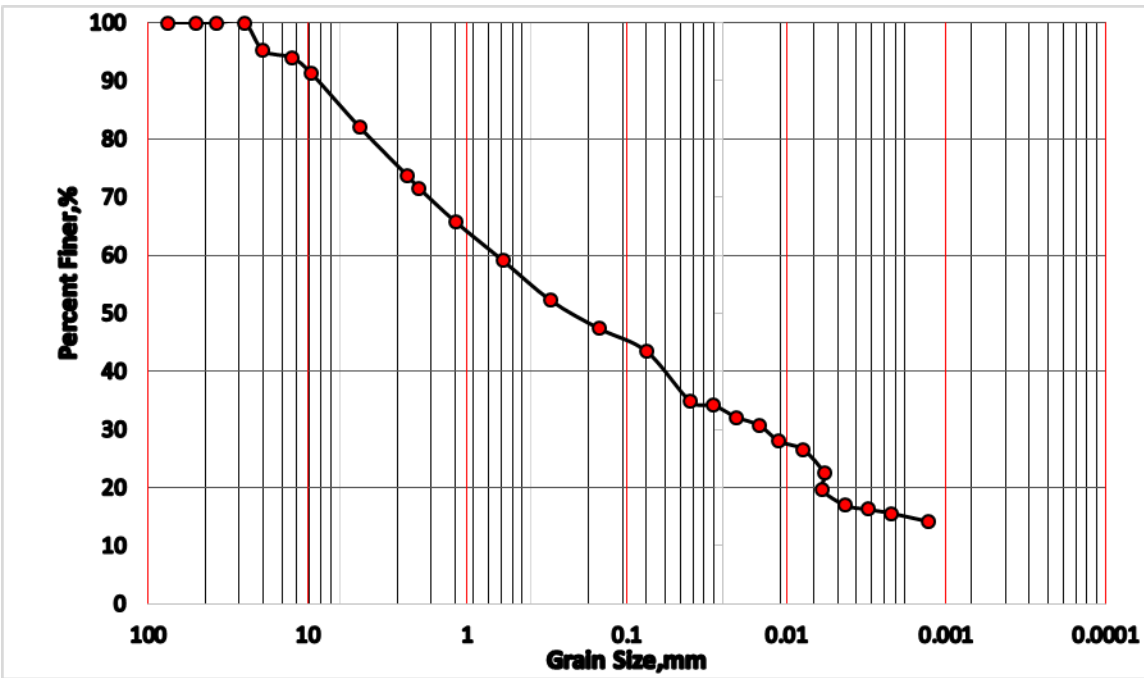


Figure A.6 Combined grain size analysis test result for TP03@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m
Sample No: TP04 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	468.7	8.1	0.8	0.8	99.2
No 4	4.75	427.9	444.0	16.1	1.6	2.4	97.6
No 8	2.36	387.7	401.3	13.6	1.4	3.8	96.2
No 10	2	390.2	394.3	4.1	0.4	4.2	95.8
No 16	1.18	372.6	385.8	13.2	1.3	5.5	94.5
No 30	0.6	325.6	341.0	15.4	1.5	7.1	93.0
No 40	0.425	291.7	303.3	11.6	1.2	8.2	91.8
No 50	0.3	301.5	318.5	17.0	1.7	9.9	90.1
No 100	0.15	271.4	313.1	41.7	4.2	14.1	85.9
No 200	0.075	273.8	311.1	37.3	3.7	17.8	82.2
pan	-------	254.1	254.1	0.0	0.0	17.8	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.66 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0310	-0.0027	1.0283	8.10	0.01365	0.0388	90.90	74.71
1	1.0300	-0.0027	1.0273	8.36	0.01365	0.0279	87.69	72.07
2	1.0280	-0.0027	1.0253	8.89	0.01365	0.0204	81.27	66.79
4	1.0270	-0.0027	1.0243	9.16	0.01365	0.0146	78.05	64.15
8	1.0260	-0.0027	1.0233	9.42	0.01365	0.0108	74.84	61.51
15	1.0250	-0.0027	1.0223	9.69	0.01365	0.0078	71.63	58.87
30	1.0240	-0.0027	1.0213	9.95	0.01365	0.0056	68.42	56.23
60	1.0210	-0.0027	1.0183	10.75	0.01365	0.0058	58.78	48.31
120	1.0190	-0.0027	1.0163	11.27	0.01365	0.0042	52.36	43.03
240	1.0180	-0.0027	1.0153	11.54	0.01365	0.0030	49.15	40.39
480	1.0160	-0.0027	1.0133	12.07	0.01365	0.0022	42.72	35.11
1440	1.0150	-0.0027	1.0123	12.33	0.01365	0.0013	39.51	32.47

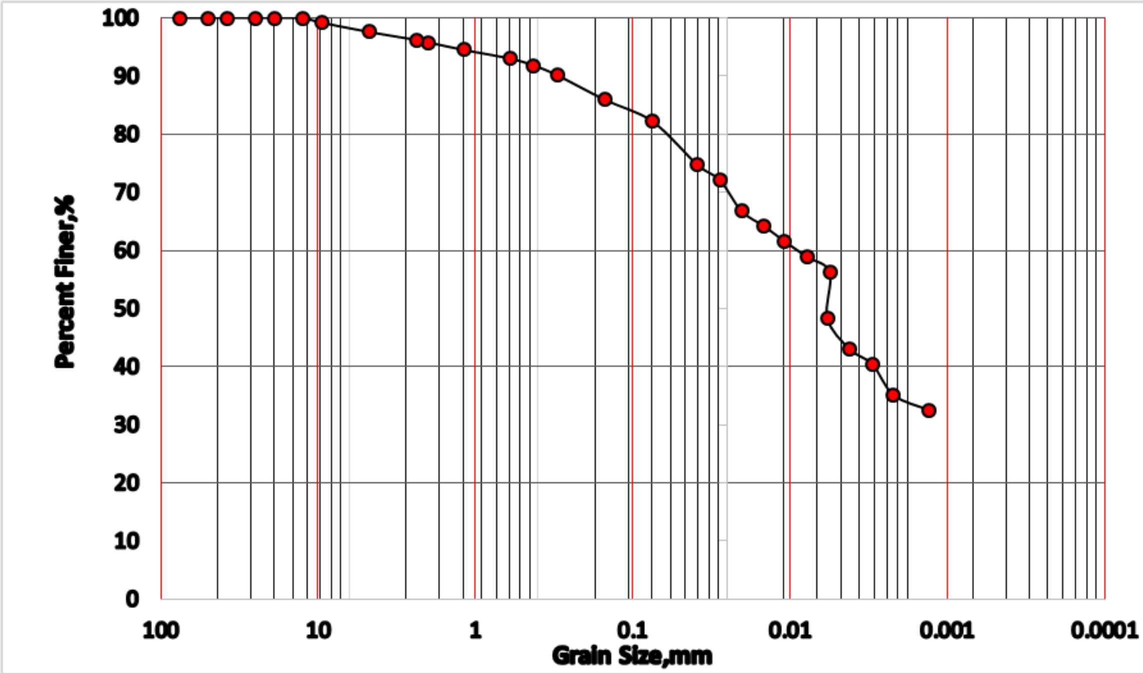


Figure A.7 Combined grain size analysis test result for TP04@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP04 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1166.4	1.8	0.1	0.1	99.9
No 4	4.75	1262.7	1267.3	4.6	0.3	0.4	99.6
No 8	2.36	990.2	1004.8	14.6	1.0	1.4	98.6
No 10	2	944.1	949.0	4.9	0.3	1.7	98.3
No 16	1.18	894.5	990.3	95.8	6.4	8.1	91.9
No 30	0.6	833.7	875.1	41.4	2.8	10.9	89.1
No 50	0.3	750.3	862.3	112.0	7.5	18.3	81.7
No 100	0.15	782.7	924.8	142.1	9.5	27.8	72.2
No 200	0.075	765.1	852.8	87.7	5.8	33.7	66.3
pan	-------	736.0	736.0	0.0	0.0	33.7	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.69 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0320	-0.0027	1.0293	7.84	0.013776	0.0386	94.77	62.87
1	1.0310	-0.0027	1.0283	8.10	0.013776	0.0277	91.54	60.73
2	1.0300	-0.0027	1.0273	8.36	0.013776	0.0199	88.30	58.58
4	1.0280	-0.0027	1.0253	8.89	0.013776	0.0145	81.83	54.29
8	1.0260	-0.0027	1.0233	9.42	0.013776	0.0109	75.37	50.00
15	1.0220	-0.0027	1.0193	10.48	0.013776	0.0081	62.43	41.41
30	1.0190	-0.0027	1.0163	11.27	0.013776	0.0060	52.72	34.98
60	1.0160	-0.0027	1.0133	12.07	0.013776	0.0062	43.02	28.54
120	1.0130	-0.0027	1.0103	12.86	0.013776	0.0045	33.32	22.10
240	1.0120	-0.0027	1.0093	13.13	0.013776	0.0032	30.08	19.96
480	1.0110	-0.0027	1.0083	13.39	0.013776	0.0023	26.85	17.81
1440	1.0100	-0.0027	1.0073	13.65	0.013776	0.0013	23.61	15.66

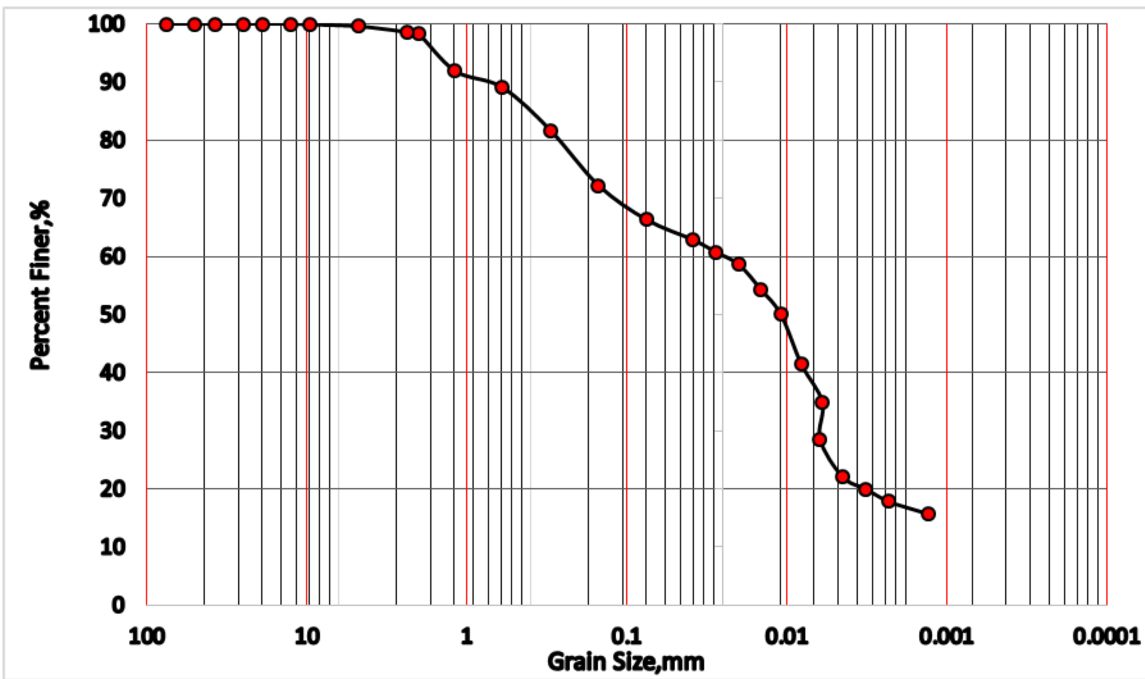


Figure A.8 Combined grain size analysis test result for TP04@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP05 Total mass of sample: 1000gm

Sieve No.	**Sieve Opening (mm)**	**Mass of Sieve(g)**	**Mass of sieve + Retained soil(g)**	**Mass of retained soil (g)**	**Percent Retained (%)**	**Cum. Percentage Retained (%)**	**Percent passing (%)**
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	462.1	1.5	0.2	0.2	99.9
No 4	4.75	427.9	439.5	11.6	1.2	1.3	98.7
No 8	2.36	387.7	396.9	9.2	0.9	2.2	97.8
No 10	2	390.2	392.3	2.1	0.2	2.4	97.6
No 16	1.18	372.6	379.0	6.4	0.6	3.1	96.9
No 30	0.6	325.6	336.4	10.8	1.1	4.2	95.8
No 40	0.425	291.7	292.6	0.9	0.1	4.3	95.8
No 50	0.3	301.5	317.3	15.8	1.6	5.8	94.2
No 100	0.15	271.4	274.0	2.6	0.3	6.1	93.9
No 200	0.075	273.8	305.1	31.3	3.1	9.2	90.8
pan	-------	254.1	254.1	0.0	0.0	9.2	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.88 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013142	0.0380	85.27	77.41
1	1.0295	-0.0027	1.0268	8.50	0.013142	0.0271	83.71	75.99
2	1.0280	-0.0027	1.0253	8.89	0.013142	0.0196	79.03	71.74
4	1.0270	-0.0027	1.0243	9.16	0.013142	0.0141	75.90	68.91
8	1.0260	-0.0027	1.0233	9.42	0.013142	0.0104	72.78	66.07
15	1.0255	-0.0027	1.0228	9.55	0.013142	0.0074	71.22	64.65
30	1.0240	-0.0027	1.0213	9.95	0.013142	0.0054	66.53	60.40
60	1.0230	-0.0027	1.0203	10.22	0.013142	0.0054	63.41	57.56
120	1.0220	-0.0027	1.0193	10.48	0.013142	0.0039	60.29	54.73
240	1.0210	-0.0027	1.0183	10.75	0.013142	0.0028	57.16	51.89
480	1.0200	-0.0027	1.0173	11.01	0.013142	0.0020	54.04	49.06
1440	1.0190	-0.0027	1.0163	11.27	0.013142	0.0012	50.91	46.22

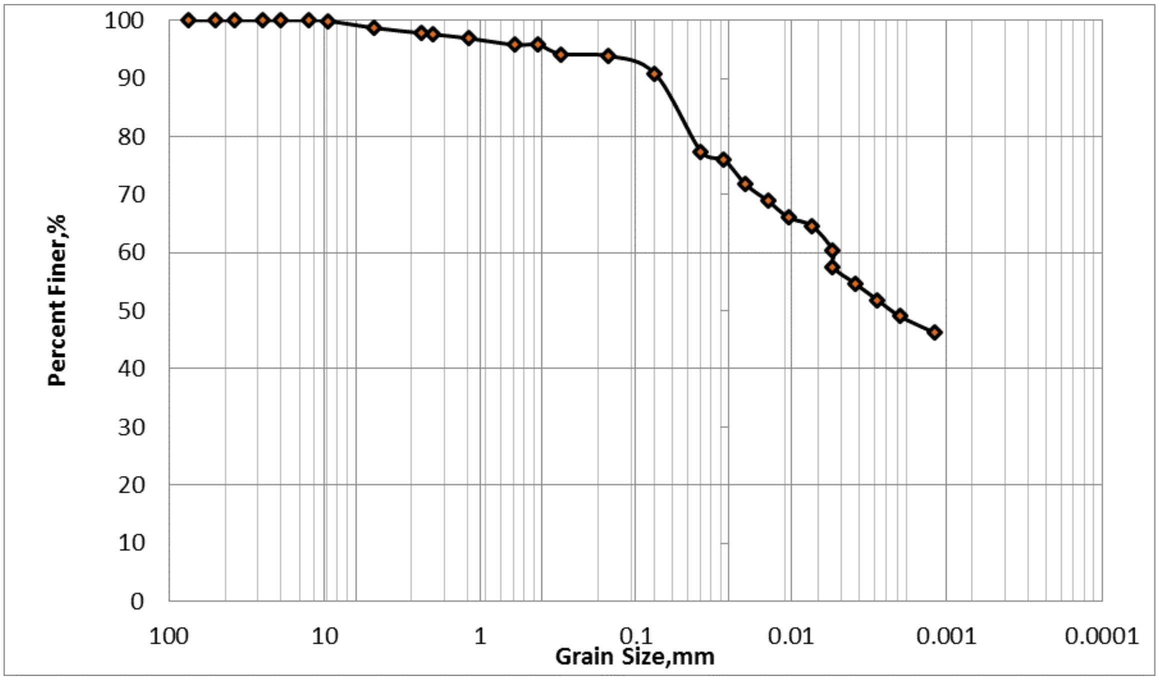


Figure A.9 Combined grain size analysis test result for TP05@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP05 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	429.0	1.1	0.1	0.1	99.9
No 8	2.36	387.7	390.0	2.3	0.2	0.3	99.7
No 10	2	390.2	391.6	1.4	0.1	0.5	99.5
No 16	1.18	372.6	377.0	4.4	0.4	0.9	99.1
No 30	0.6	325.6	336.4	10.8	1.1	2.0	98.0
No 40	0.425	291.7	301.4	9.7	1.0	3.0	97.0
No 50	0.3	301.5	312.9	11.4	1.1	4.1	95.9
No 100	0.15	271.4	294.5	23.1	2.3	6.4	93.6
No 200	0.075	273.8	298.1	24.3	2.4	8.9	91.2
pan	-------	254.1	254.1	0.0	0.0	8.9	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.72 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013364	0.0387	86.34	78.70
1	1.0295	-0.0027	1.0268	8.50	0.013364	0.0275	84.76	77.26
2	1.0290	-0.0027	1.0263	8.63	0.013364	0.0196	83.18	75.82
4	1.0270	-0.0027	1.0243	9.16	0.013364	0.0143	76.86	70.05
8	1.0260	-0.0027	1.0233	9.42	0.013364	0.0106	73.69	67.17
15	1.0250	-0.0027	1.0223	9.69	0.013364	0.0076	70.53	64.29
30	1.0240	-0.0027	1.0213	9.95	0.013364	0.0054	67.37	61.41
60	1.0230	-0.0027	1.0203	10.22	0.013364	0.0055	64.20	58.52
120	1.0210	-0.0027	1.0183	10.75	0.013364	0.0040	57.88	52.76
240	1.0200	-0.0027	1.0173	11.01	0.013364	0.0029	54.72	49.87
480	1.0180	-0.0027	1.0153	11.54	0.013364	0.0021	48.39	44.11
1440	1.0160	-0.0027	1.0133	12.07	0.013364	0.0012	42.07	38.34

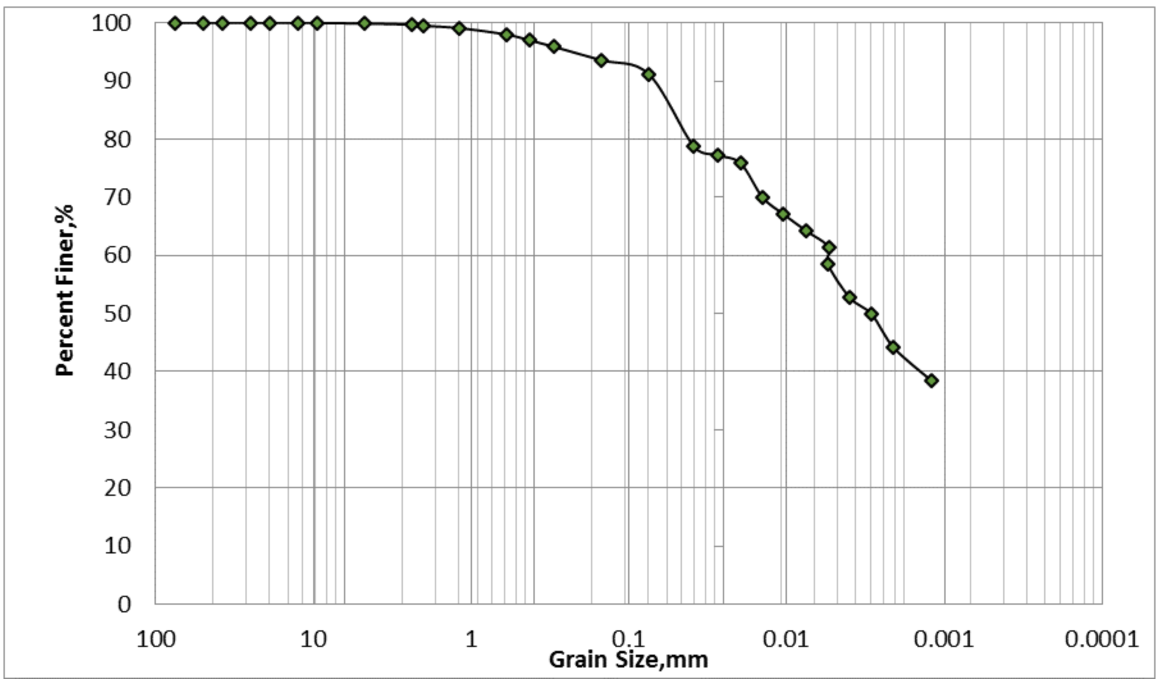


Figure A.10 Combined grain size analysis test result for TP05@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP06 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	429.6	1.7	0.2	0.2	99.8
No 8	2.36	387.7	390.9	3.2	0.3	0.5	99.5
No 10	2	390.2	392.1	1.9	0.2	0.7	99.3
No 16	1.18	372.6	380.7	8.1	0.8	1.5	98.5
No 30	0.6	325.6	331.9	6.3	0.6	2.1	97.9
No 40	0.425	291.7	295.1	3.4	0.3	2.5	97.5
No 50	0.3	301.5	305.8	4.3	0.4	2.9	97.1
No 100	0.15	271.4	274.6	3.2	0.3	3.2	96.8
No 200	0.075	273.8	291.8	18.0	1.8	5.0	95.0
pan	-------	254.1	254.1	0.0	0.0	5.0	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.77 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0290	-0.0027	1.0263	8.63	0.013178	0.0387	82.32	78.19
1	1.0285	-0.0027	1.0258	8.76	0.013178	0.0276	80.75	76.71
2	1.0280	-0.0027	1.0253	8.89	0.013178	0.0196	79.19	75.22
4	1.0260	-0.0027	1.0233	9.42	0.013178	0.0143	72.93	69.27
8	1.0255	-0.0027	1.0228	9.55	0.013178	0.0143	71.36	67.79
15	1.0240	-0.0027	1.0213	9.95	0.013178	0.0105	66.67	63.33
30	1.0230	-0.0027	1.0203	10.22	0.013178	0.0076	63.54	60.35
60	1.0220	-0.0027	1.0193	10.48	0.013178	0.0054	60.41	57.38
120	1.0210	-0.0027	1.0183	10.75	0.013178	0.0055	57.28	54.41
240	1.0200	-0.0027	1.0173	11.01	0.013178	0.0039	54.15	51.44
480	1.0190	-0.0027	1.0163	11.27	0.013178	0.0028	51.02	48.46
1440	1.0180	-0.0027	1.0153	11.54	0.013178	0.0020	47.89	45.49

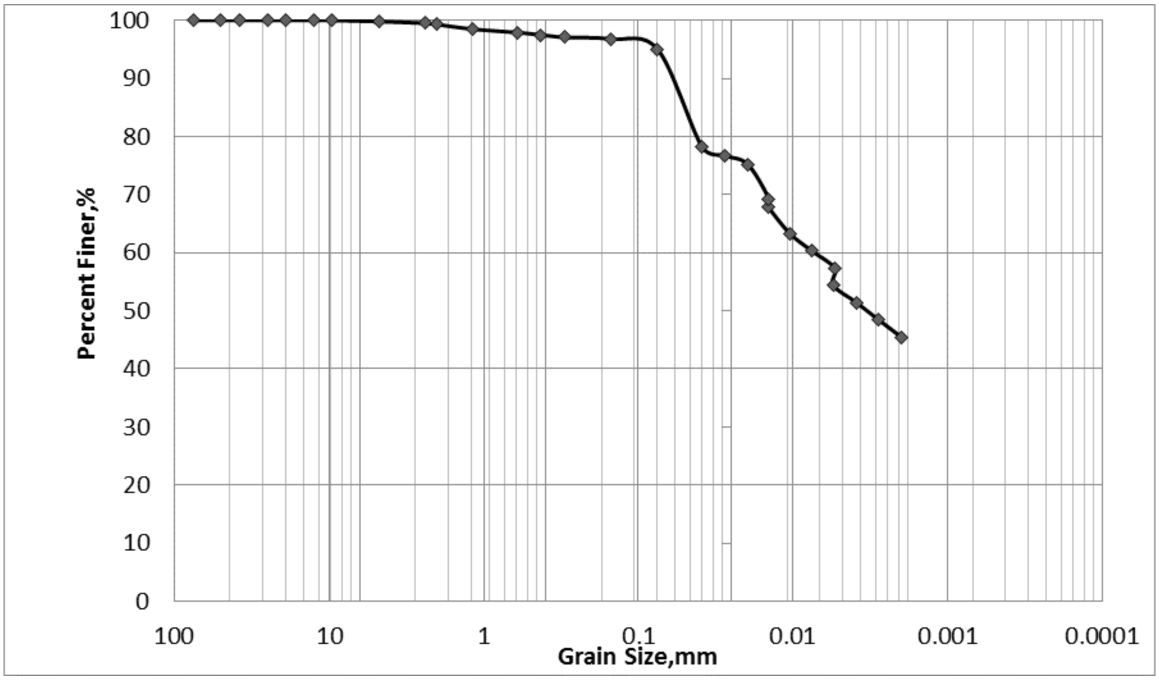


Figure A.11 Combined grain size analysis test result for TP06@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP06 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	1216.8	1222.4	5.6	0.6	0.6	99.4
3.8"	9.5	1164.6	1166.5	1.9	0.2	0.8	99.3
No 4	4.75	1262.7	1269.7	7.0	0.7	1.5	98.6
No 8	2.36	990.2	996.1	5.9	0.6	2.0	98.0
No 10	2	944.1	946.2	2.1	0.2	2.3	97.8
No 16	1.18	894.5	900.1	5.6	0.6	2.8	97.2
No 30	0.6	833.7	841.4	7.7	0.8	3.6	96.4
No 50	0.3	750.3	762.6	12.3	1.2	4.8	95.2
No 100	0.15	782.7	804.0	21.3	2.1	6.9	93.1
No 200	0.075	765.1	784.1	19.0	1.9	8.8	91.2
pan	-------	736.0	736.0	0.0	0.0	8.8	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.64 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013288	0.0384	85.98	78.38
1	1.0290	-0.0027	1.0263	8.63	0.013288	0.0276	82.83	75.51
2	1.0270	-0.0027	1.0243	9.16	0.013288	0.0201	76.53	69.77
4	1.0260	-0.0027	1.0233	9.42	0.013288	0.0144	73.38	66.89
8	1.0250	-0.0027	1.0223	9.69	0.013288	0.0107	70.23	64.02
15	1.0240	-0.0027	1.0213	9.95	0.013288	0.0077	67.08	61.15
30	1.0230	-0.0027	1.0203	10.22	0.013288	0.0055	63.93	58.28
60	1.0210	-0.0027	1.0183	10.75	0.013288	0.0056	57.63	52.54
120	1.0205	-0.0027	1.0178	10.88	0.013288	0.0040	56.06	51.10
240	1.0190	-0.0027	1.0163	11.27	0.013288	0.0029	51.34	46.80
480	1.0180	-0.0027	1.0153	11.54	0.013288	0.0021	48.19	43.93
1440	1.0170	-0.0027	1.0143	11.80	0.013288	0.0012	45.04	41.06

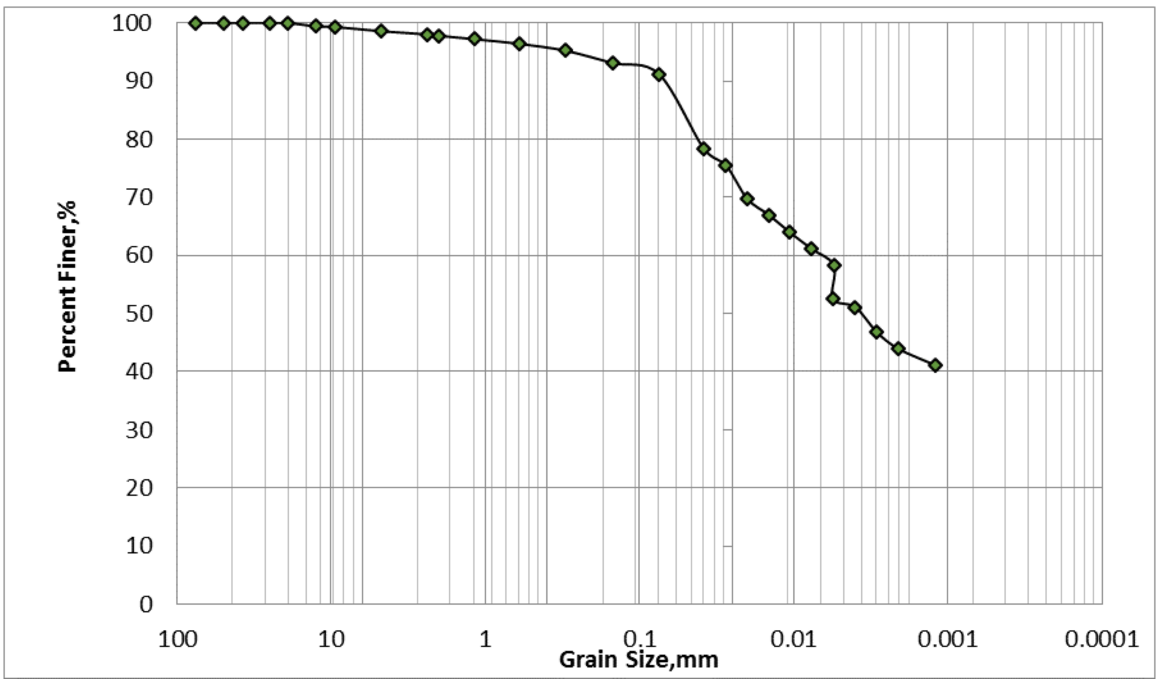


Figure A.12 Combined grain size analysis test result for TP06@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP07 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3''	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2''	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	0.0	0.0	0.0	0.0	0.0	100.0
No 4	4.75	1262.7	1263.1	0.4	0.0	0.0	100.0
No 8	2.36	990.3	991.9	1.6	0.1	0.2	99.8
No 10	2	944.1	944.8	0.7	0.1	0.2	99.8
No 16	1.18	894.3	899.0	4.7	0.4	0.6	99.4
No 30	0.6	833.8	842.3	8.5	0.7	1.2	98.8
No 50	0.3	750.1	767.3	17.2	1.3	2.5	97.5
No 100	0.15	782.4	813.1	30.7	2.4	4.9	95.1
No 200	0.075	764.4	795.4	31.0	2.4	7.3	92.7
pan	-------	736.0	736.0	0.0	0.0	7.3	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.73 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0310	-0.0027	1.0283	8.10	0.013326	0.0379	89.32	82.80
1	1.0305	-0.0027	1.0278	8.23	0.013326	0.0270	87.74	81.34
2	1.0300	-0.0027	1.0273	8.36	0.013326	0.0193	86.16	79.88
4	1.0290	-0.0027	1.0263	8.63	0.013326	0.0138	83.00	76.95
8	1.0280	-0.0027	1.0253	8.89	0.013326	0.0103	79.85	74.03
15	1.0270	-0.0027	1.0243	9.16	0.013326	0.0074	76.69	71.10
30	1.0260	-0.0027	1.0233	9.42	0.013326	0.0053	73.54	68.17
60	1.0250	-0.0027	1.0223	9.69	0.013326	0.0054	70.38	65.25
120	1.0240	-0.0027	1.0213	9.95	0.013326	0.0038	67.22	62.32
240	1.0230	-0.0027	1.0203	10.22	0.013326	0.0027	64.07	59.40
480	1.0220	-0.0027	1.0193	10.48	0.013326	0.0020	60.91	56.47
1440	1.0210	-0.0027	1.0183	10.75	0.013326	0.0012	57.76	53.54

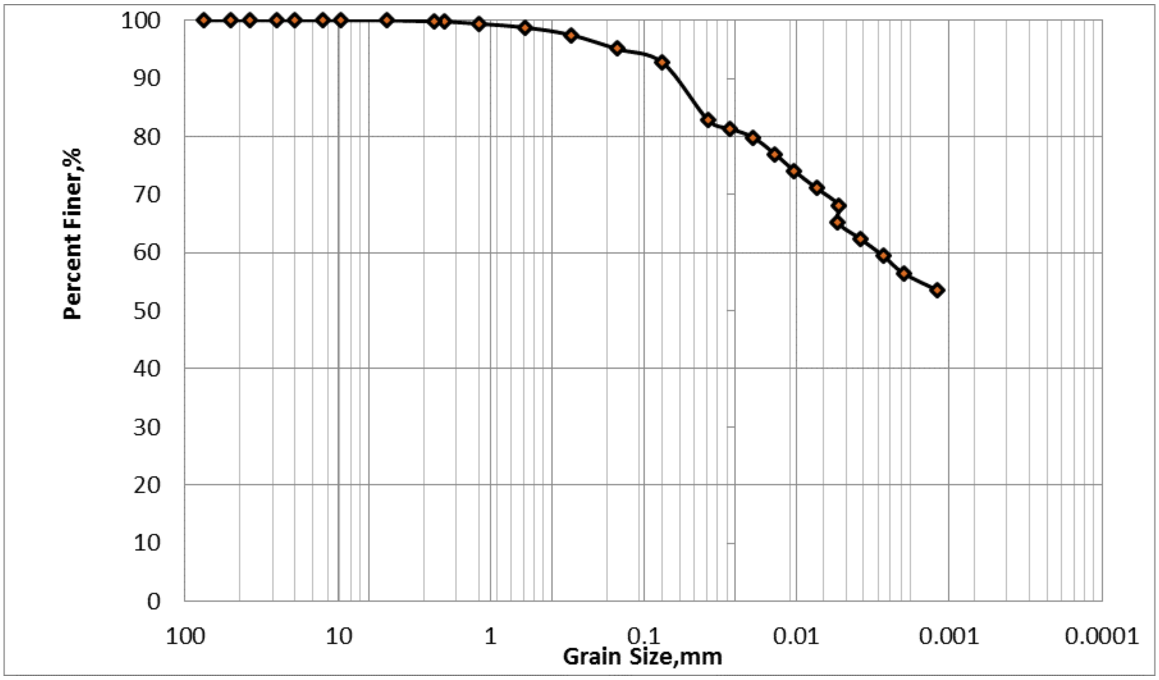


Figure A.13 Combined grain size analysis test result for TP07@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP07 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	454.8	462.9	8.1	0.8	0.8	99.2
No 4	4.75	427.8	440.8	13.0	1.3	2.1	97.9
No 8	2.36	387.7	405.7	18.0	1.8	3.9	96.1
No 10	2	377.9	381.9	4.0	0.4	4.3	95.7
No 16	1.18	372.6	383.2	10.6	1.1	5.4	94.6
No 30	0.6	323.3	342.6	19.3	1.9	7.3	92.7
No 40	0.425	292.1	312.9	20.8	2.1	9.4	90.6
No 50	0.3	301.6	333.0	31.4	3.1	12.5	87.5
No 100	0.15	276.0	353.4	77.4	7.7	20.3	79.7
No 200	0.075	258.6	360.7	102.1	10.2	30.5	69.5
pan	-------	255.6	255.6	0.0	0.0	30.5	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.70 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0280	-0.0027	1.0253	8.89	0.01307	0.0390	78.71	54.73
1	1.0275	-0.0027	1.0248	9.03	0.01307	0.0278	77.16	53.65
2	1.0270	-0.0027	1.0243	9.16	0.01307	0.0198	75.60	52.56
4	1.0230	-0.0027	1.0203	10.22	0.01307	0.0148	63.16	43.91
8	1.0200	-0.0027	1.0173	11.01	0.01307	0.0112	53.82	37.42
15	1.0170	-0.0027	1.0143	11.80	0.01307	0.0082	44.49	30.93
30	1.0140	-0.0027	1.0113	12.60	0.01307	0.0060	35.16	24.44
60	1.0120	-0.0027	1.0093	13.13	0.01307	0.0061	28.93	20.12
120	1.0110	-0.0027	1.0083	13.39	0.01307	0.0044	25.82	17.95
240	1.0100	-0.0027	1.0073	13.65	0.01307	0.0031	22.71	15.79
480	1.0090	-0.0027	1.0063	13.92	0.01307	0.0022	19.60	13.63
1440	1.0080	-0.0027	1.0053	14.18	0.01307	0.0013	16.49	11.46

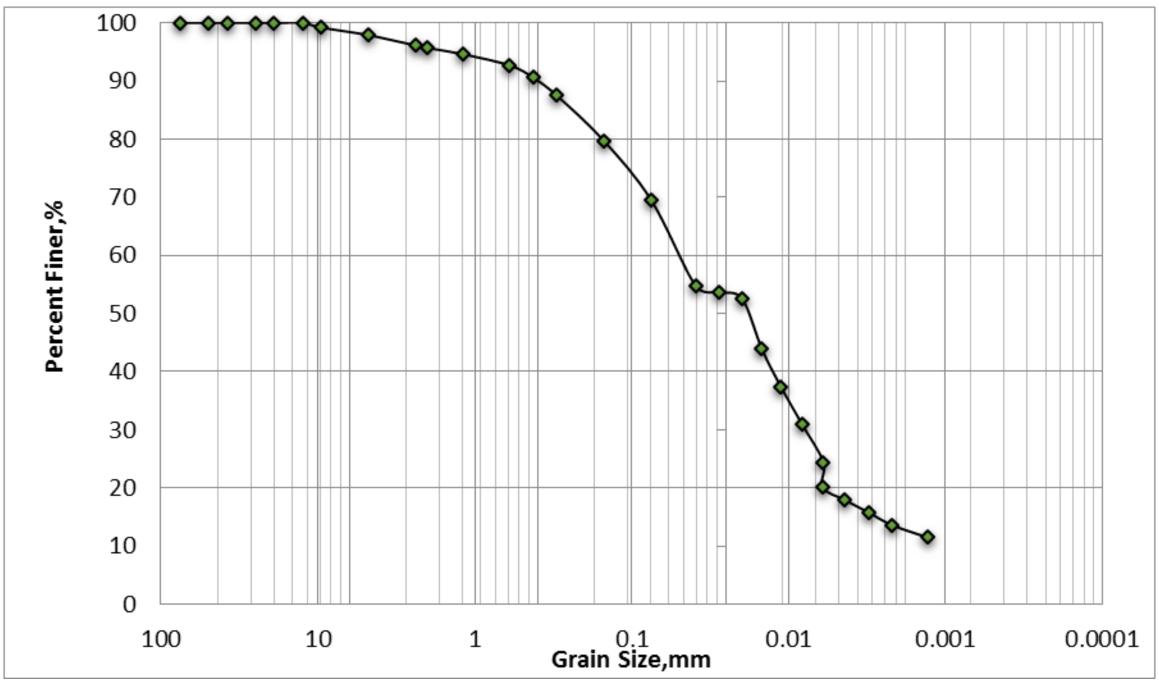


Figure A.14 Combined grain size analysis test result for TP07@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP08 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	428.0	0.1	0.0	0.0	100.0
No 8	2.36	387.7	388.7	1.0	0.1	0.1	99.9
No 10	2	390.2	390.6	0.4	0.0	0.1	99.9
No 16	1.18	372.6	374.6	2.0	0.1	0.2	99.8
No 30	0.6	325.6	330.7	5.1	0.3	0.6	99.4
No 40	0.425	291.7	295.5	3.8	0.3	0.8	99.2
No 50	0.3	301.5	305.8	4.3	0.3	1.1	98.9
No 100	0.15	271.4	278.2	6.8	0.5	1.6	98.4
No 200	0.075	273.8	287.0	13.2	0.9	2.4	97.6
pan	-------	255.6	255.6	0.0	0.0	2.4	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.71 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0310	-0.0027	1.0283	8.10	0.013034	0.0371	87.87	85.72
1	1.0300	-0.0027	1.0273	8.36	0.013034	0.0267	84.77	82.69
2	1.0280	-0.0027	1.0253	8.89	0.013034	0.0194	78.56	76.63
4	1.0270	-0.0027	1.0243	9.16	0.013034	0.0139	75.45	73.60
8	1.0260	-0.0027	1.0233	9.42	0.013034	0.0103	72.35	70.58
15	1.0250	-0.0027	1.0223	9.69	0.013034	0.0074	69.24	67.55
30	1.0240	-0.0027	1.0213	9.95	0.013034	0.0053	66.14	64.52
60	1.0230	-0.0027	1.0203	10.22	0.013034	0.0054	63.03	61.49
120	1.0220	-0.0027	1.0193	10.48	0.013034	0.0039	59.93	58.46
240	1.0210	-0.0027	1.0183	10.75	0.013034	0.0028	56.82	55.43
480	1.0200	-0.0027	1.0173	11.01	0.013034	0.0020	53.72	52.40
1440	1.0190	-0.0027	1.0163	11.27	0.013034	0.0012	50.61	49.37

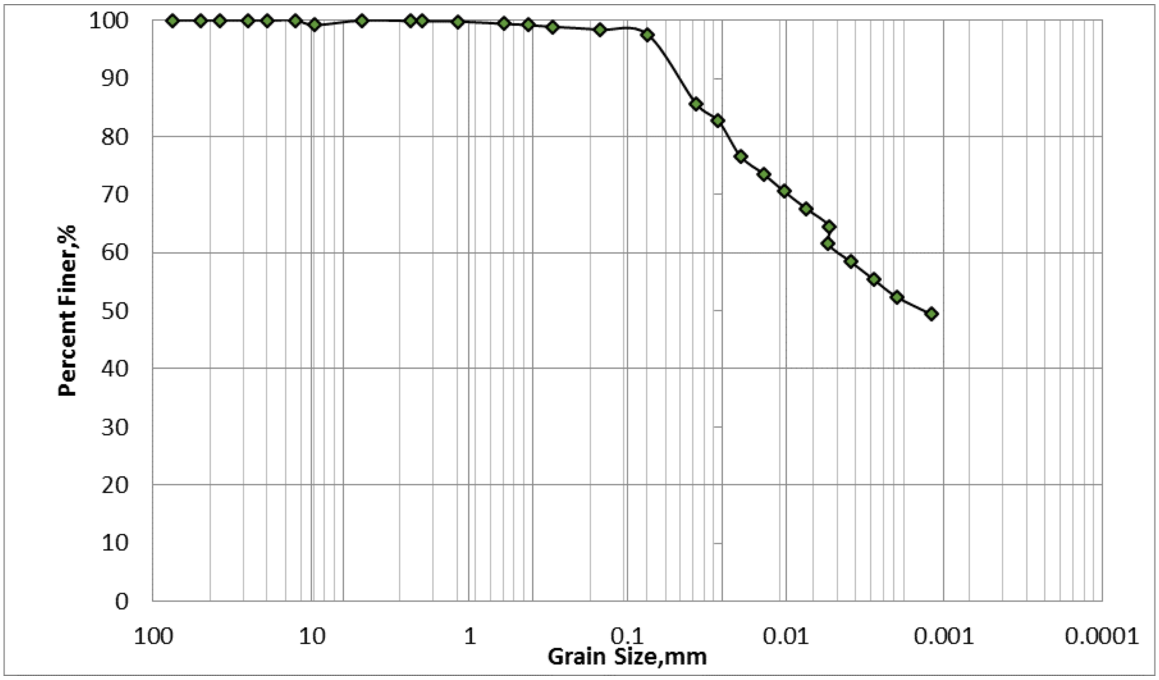


Figure A.15 Combined grain size analysis test result for TP08@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP08 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1187.0	1187.0	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	1216.7	1216.7	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1164.6	0.0	0.0	0.0	100.0
No 4	4.75	1262.7	1263.5	0.8	0.1	0.1	99.9
No 8	2.36	990.2	992.7	2.5	0.2	0.2	99.8
No 10	2	944.1	945.7	1.6	0.1	0.3	99.7
No 16	1.18	894.5	903.0	8.5	0.6	0.9	99.1
No 30	0.6	833.7	856.1	22.4	1.5	2.4	97.6
No 50	0.3	750.3	792.2	41.9	2.8	5.2	94.8
No 100	0.15	782.7	882.2	99.5	6.6	11.8	88.2
No 200	0.075	765.1	847.7	82.6	5.5	17.3	82.7
pan	-------	736.0	736.0	0.0	0.0	17.3	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.67 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013178	0.0381	88.30	73.01
1	1.0290	-0.0027	1.0263	8.63	0.013178	0.0274	85.07	70.34
2	1.0280	-0.0027	1.0253	8.89	0.013178	0.0196	81.83	67.66
4	1.0270	-0.0027	1.0243	9.16	0.013178	0.0141	78.60	64.99
8	1.0260	-0.0027	1.0233	9.42	0.013178	0.0104	75.37	62.31
15	1.0250	-0.0027	1.0223	9.69	0.013178	0.0075	72.13	59.64
30	1.0240	-0.0027	1.0213	9.95	0.013178	0.0054	68.90	56.96
60	1.0230	-0.0027	1.0203	10.22	0.013178	0.0054	65.66	54.29
120	1.0220	-0.0027	1.0193	10.48	0.013178	0.0039	62.43	51.61
240	1.0210	-0.0027	1.0183	10.75	0.013178	0.0028	59.19	48.94
480	1.0205	-0.0027	1.0178	10.88	0.013178	0.0020	57.58	47.60
1440	1.0200	-0.0027	1.0173	11.01	0.013178	0.0012	55.96	46.27

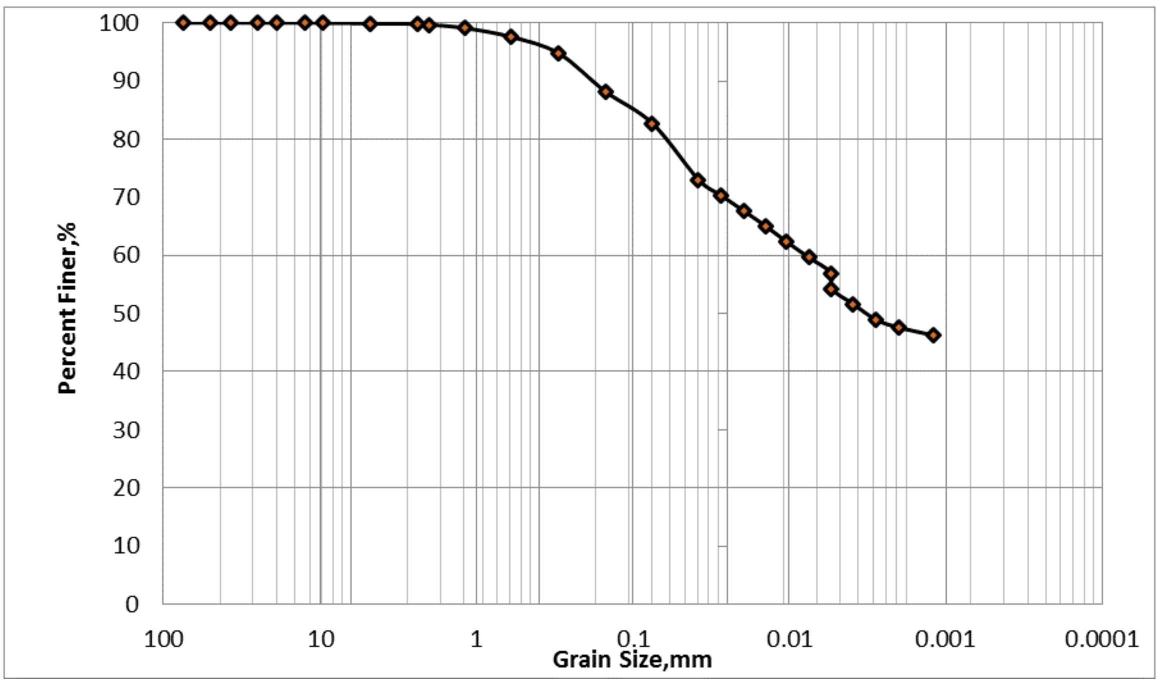


Figure A.16 Combined grain size analysis test result for TP08@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP09 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	449.8	21.9	1.5	1.5	98.5
No 8	2.36	387.7	422.5	34.8	2.3	3.8	96.2
No 10	2	390.2	401.6	11.4	0.8	4.5	95.5
No 16	1.18	372.6	409.9	37.3	2.5	7.0	93.0
No 30	0.6	325.6	389.6	64.0	4.3	11.3	88.7
No 40	0.425	291.7	335.0	43.3	2.9	14.2	85.8
No 50	0.3	301.5	341.5	40.0	2.7	16.8	83.2
No 100	0.15	271.4	322.1	50.7	3.4	20.2	79.8
No 200	0.075	273.8	302.8	29.0	1.9	22.2	77.8
pan	-------	255.6	255.6	0.0	0.0	22.2	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.68 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013142	0.0380	85.27	66.38
1	1.0295	-0.0027	1.0268	8.50	0.013142	0.0271	83.71	65.16
2	1.0280	-0.0027	1.0253	8.89	0.013142	0.0196	79.03	61.51
4	1.0260	-0.0027	1.0233	9.42	0.013142	0.0143	72.78	56.65
8	1.0250	-0.0027	1.0223	9.69	0.013142	0.0106	69.66	54.22
15	1.0230	-0.0027	1.0203	10.22	0.013142	0.0077	63.41	49.36
30	1.0210	-0.0027	1.0183	10.75	0.013142	0.0056	57.16	44.49
60	1.0200	-0.0027	1.0173	11.01	0.013142	0.0056	54.04	42.06
120	1.0190	-0.0027	1.0163	11.27	0.013142	0.0040	50.91	39.63
240	1.0180	-0.0027	1.0153	11.54	0.013142	0.0029	47.79	37.20
480	1.0170	-0.0027	1.0143	11.80	0.013142	0.0021	44.67	34.77
1440	1.0160	-0.0027	1.0133	12.07	0.013142	0.0012	41.54	32.34

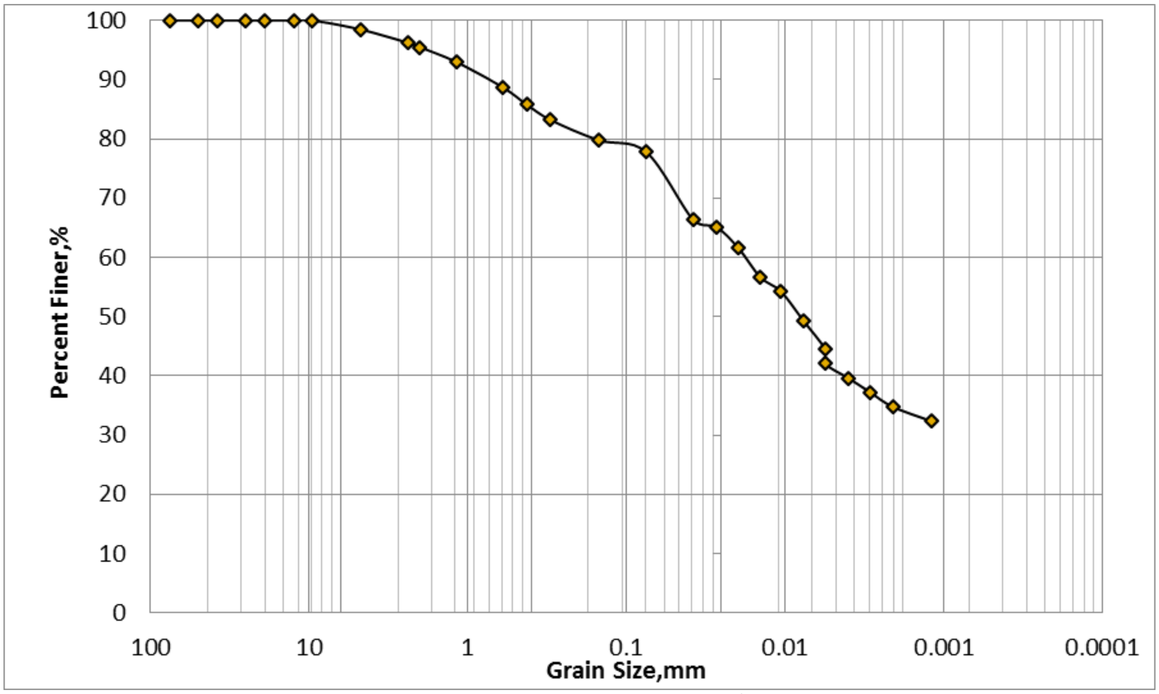


Figure A.17 Combined grain size analysis test result for TP09@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP09 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2”	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	1216.8	1225.8	9.0	0.6	0.6	99.4
3.8"	9.5	1164.6	1167.7	3.1	0.2	0.8	99.2
No 4	4.75	1262.7	1279.2	16.5	1.1	1.9	98.1
No 8	2.36	990.2	1007.1	16.9	1.1	3.0	97.0
No 10	2	944.1	950.7	6.6	0.4	3.5	96.5
No 16	1.18	894.5	921.4	26.9	1.8	5.3	94.7
No 30	0.6	833.7	911.5	77.8	5.2	10.5	89.5
No 50	0.3	750.3	903.3	153.0	10.2	20.7	79.3
No 100	0.15	782.7	930.2	147.5	9.8	30.5	69.5
No 200	0.075	765.1	847.5	82.4	5.5	36.0	64.0
pan	-------	736.0	736.0	0.0	0.0	36.0	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.62 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0300	-0.0027	1.0273	8.36	0.013364	0.0387	86.34	55.28
1	1.0290	-0.0027	1.0263	8.63	0.013364	0.0278	83.18	53.25
2	1.0270	-0.0027	1.0243	9.16	0.013364	0.0202	76.86	49.20
4	1.0265	-0.0027	1.0238	9.29	0.013364	0.0144	75.27	48.19
8	1.0250	-0.0027	1.0223	9.69	0.013364	0.0107	70.53	45.15
15	1.0240	-0.0027	1.0213	9.95	0.013364	0.0077	67.37	43.13
30	1.0230	-0.0027	1.0203	10.22	0.013364	0.0055	64.20	41.10
60	1.0220	-0.0027	1.0193	10.48	0.013364	0.0056	61.04	39.08
120	1.0210	-0.0027	1.0183	10.75	0.013364	0.0040	57.88	37.05
240	1.0200	-0.0027	1.0173	11.01	0.013364	0.0029	54.72	35.03
480	1.0195	-0.0027	1.0168	11.14	0.013364	0.0020	53.13	34.02
1440	1.0180	-0.0027	1.0153	11.54	0.013364	0.0012	48.39	30.98

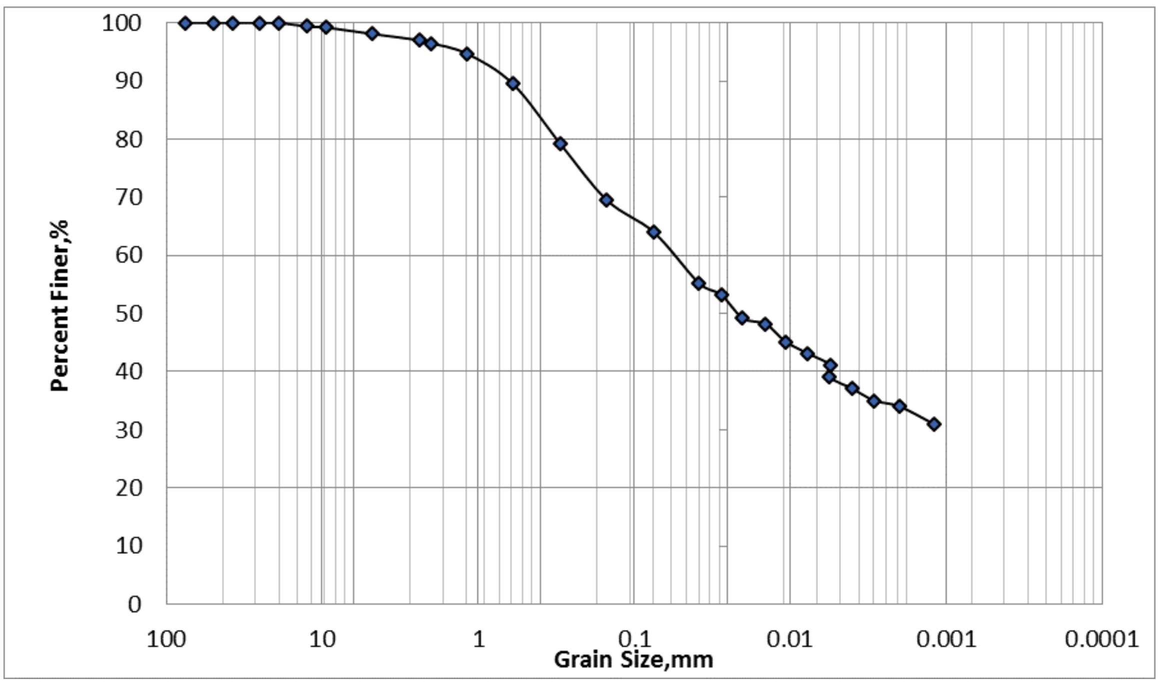


Figure A.18 Combined grain size analysis test result for TP09@3m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 1.5m

Sample No: TP10 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3"	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2"	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	460.6	460.6	0.0	0.0	0.0	100.0
No 4	4.75	427.9	427.9	0.0	0.0	0.0	100.0
No 8	2.36	387.7	389.6	1.9	0.2	0.2	99.8
No 10	2	390.2	390.7	0.5	0.1	0.2	99.8
No 16	1.18	372.6	375.1	2.5	0.3	0.5	99.5
No 30	0.6	325.6	331.0	5.4	0.5	1.0	99.0
No 40	0.425	291.7	291.9	0.2	0.0	1.1	99.0
No 50	0.3	301.5	310.0	8.5	0.9	1.9	98.1
No 100	0.15	271.4	277.0	5.6	0.6	2.5	97.5
No 200	0.075	273.8	317.8	44.0	4.4	6.9	93.1
pan	-------	254.1	254.1	0.0	0.0	6.9	-------

Hydrometer Analysis Sample Depth: 1.50m

Specific Gravity of soil=2.69 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0310	-0.0027	1.0283	8.10	0.013106	0.0373	88.22	82.17
1	1.0300	-0.0027	1.0273	8.36	0.013106	0.0268	85.10	79.26
2	1.0280	-0.0027	1.0253	8.89	0.013106	0.0195	78.87	73.46
4	1.0270	-0.0027	1.0243	9.16	0.013106	0.0140	75.75	70.55
8	1.0260	-0.0027	1.0233	9.42	0.013106	0.0104	72.63	67.65
15	1.0250	-0.0027	1.0223	9.69	0.013106	0.0074	69.52	64.75
30	1.0240	-0.0027	1.0213	9.95	0.013106	0.0053	66.40	61.84
60	1.0230	-0.0027	1.0203	10.22	0.013106	0.0054	63.28	58.94
120	1.0220	-0.0027	1.0193	10.48	0.013106	0.0039	60.16	56.04
240	1.0210	-0.0027	1.0183	10.75	0.013106	0.0028	57.05	53.13
480	1.0200	-0.0027	1.0173	11.01	0.013106	0.0020	53.93	50.23
1440	1.0190	-0.0027	1.0163	11.27	0.013106	0.0012	50.81	47.33

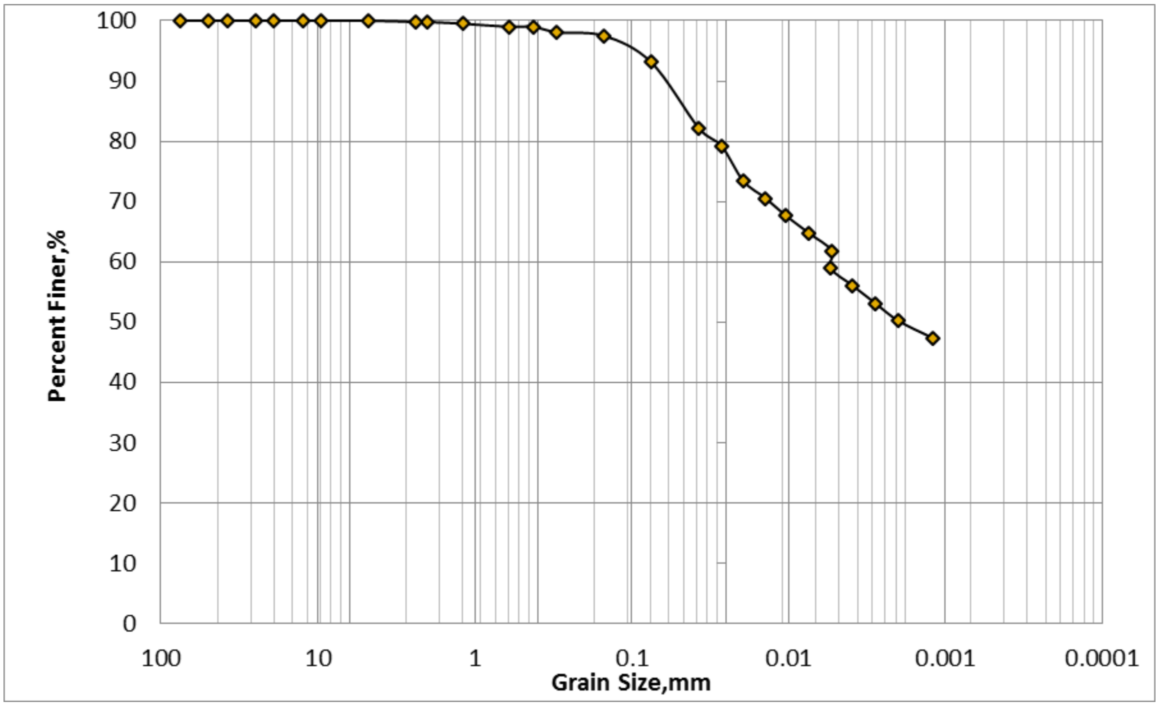


Figure A.19 Combined grain size analysis test result for TP10@1.5m

Grain Size Analysis/Sieve Analysis/ Sample Depth: 3.00m

Sample No: TP10 Total mass of sample: 1000gm

Sieve No.	Sieve Opening (mm)	Mass of Sieve(g)	Mass of sieve + Retained soil(g)	Mass of retained soil (g)	Percent Retained (%)	Cum. Percentage Retained (%)	Percent passing (%)
3’’	75.0	0.0	0.0	0.0	0.0	0.0	100.0
2’’	50.0	0.0	0.0	0.0	0.0	0.0	100.0
1.5"	37.5	0.0	0.0	0.0	0.0	0.0	100.0
1"	25.0	0.0	0.0	0.0	0.0	0.0	100.0
3/4"	19.0	0.0	0.0	0.0	0.0	0.0	100.0
1/2"	12.5	0.0	0.0	0.0	0.0	0.0	100.0
3.8"	9.5	454.8	574.0	119.2	11.9	11.9	88.1
No 4	4.75	427.8	554.9	127.1	12.7	24.6	75.4
No 8	2.36	387.7	504.3	116.6	11.7	36.3	63.7
No 10	2	377.9	403.7	25.8	2.6	38.9	61.1
No 16	1.18	372.6	436.9	64.3	6.4	45.3	54.7
No 30	0.6	323.3	384.3	61.0	6.1	51.4	48.6
No 40	0.425	292.1	315.4	23.3	2.3	53.7	46.3
No 50	0.3	301.6	321.2	19.6	2.0	55.7	44.3
No 100	0.15	276.0	304.0	28.0	2.8	58.5	41.5
No 200	0.075	258.6	290.2	31.6	3.2	61.7	38.4
pan	-------	255.6	254.1	-1.5	-0.2	61.5	-------

Hydrometer Analysis Sample Depth: 3.00m

Specific Gravity of soil=2.71 Test Temperature: 20 ^{0}c

Elapsed Time (min)	Actual Hydro.Rdg	Composite Correction	Corrected Hydro.Rdg	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0260	-0.0027	1.0233	9.42	0.013034	0.0400	72.35	27.74
1	1.0255	-0.0027	1.0228	9.55	0.013034	0.0285	70.79	27.15
2	1.0240	-0.0027	1.0213	9.95	0.013034	0.0206	66.14	25.36
4	1.0210	-0.0027	1.0183	10.75	0.013034	0.0151	56.82	21.79
8	1.0190	-0.0027	1.0163	11.27	0.013034	0.0113	50.61	19.41
15	1.0170	-0.0027	1.0143	11.80	0.013034	0.0082	44.40	17.03
30	1.0160	-0.0027	1.0133	12.07	0.013034	0.0058	41.30	15.84
60	1.0150	-0.0027	1.0123	12.33	0.013034	0.0059	38.19	14.65
120	1.0140	-0.0027	1.0113	12.60	0.013034	0.0042	35.09	13.46
240	1.0130	-0.0027	1.0103	12.86	0.013034	0.0030	31.98	12.26
480	1.0120	-0.0027	1.0093	13.13	0.013034	0.0022	28.88	11.07
1440	1.0110	-0.0027	1.0083	13.39	0.013034	0.0013	25.77	9.88

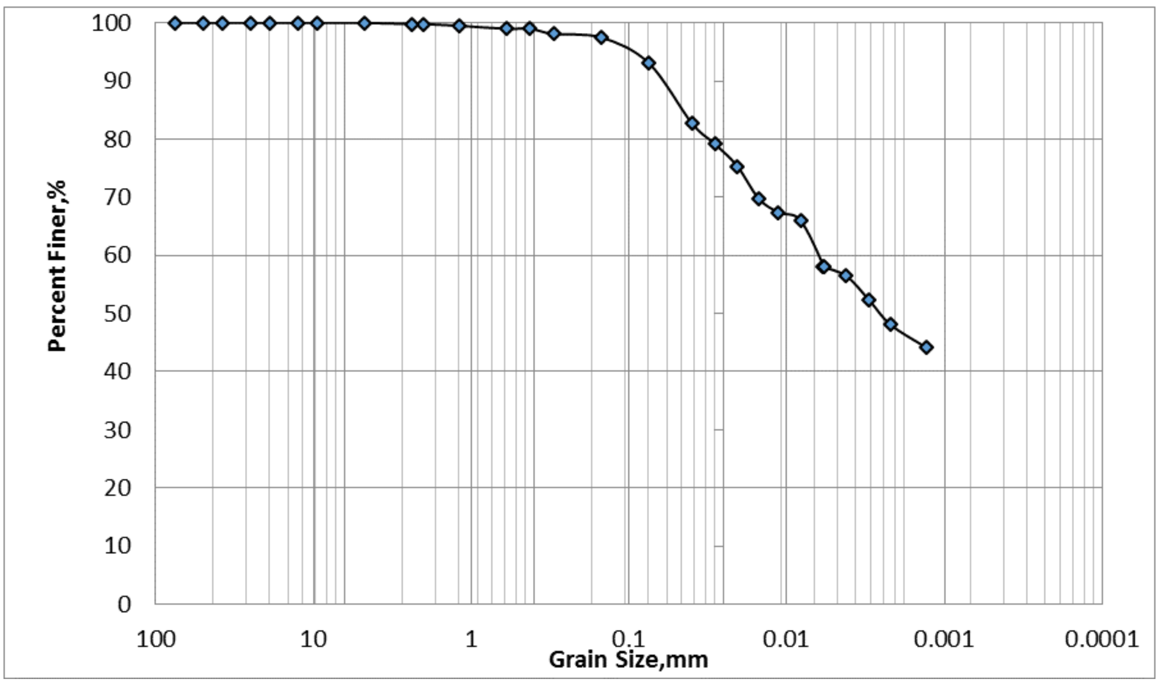


Figure A.20 Combined grain size analysis test result for TP10@3m

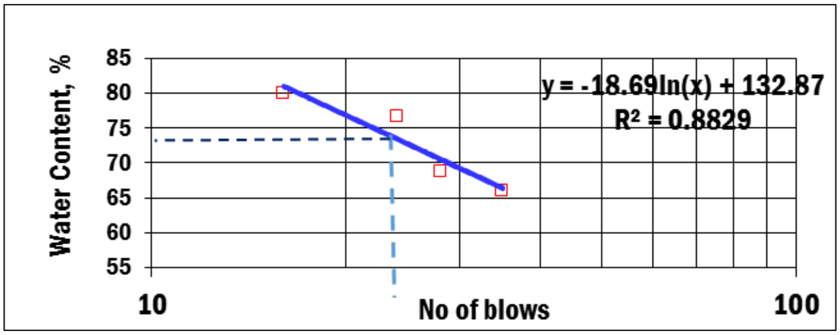
Appendix – B

Atterberg Limit test Results

Sample No: TP 01 **Sample Depth: 1.50m**

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No.	A	B	C	D	E	F
Mass of container, g	15.60	15.50	30.80	15.60	15.80	15.60
Mass of container + Wet soil, g	24.40	23.60	43.70	30.00	23.70	25.90
Mass of container + Dry soil, g	20.90	20.30	38.10	23.60	21.80	23.30
Mass of water, g	3.50	3.30	5.60	6.40	1.90	2.60
Mass of dry soil, g	5.30	4.80	7.30	8.00	6.00	7.70
Water content, %	66.04	68.75	76.71	80.00	31.67	33.77
No of blows (N)	35	28	24	16	-----	------

LL=73% PL=32 % PI=41%



Summary:

For N=25

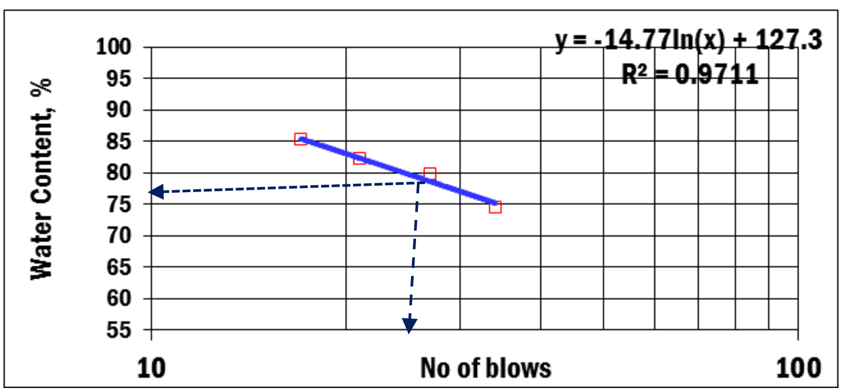
LL=73%

Figure B.1 Flow curve analysis and plastic limits for TP01@1.5m

Sample No: TP 01 **Sample Depth: 3.00m**

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No.	A	B	C	D	E	F
Mass of container, g	15.70	15.90	15.60	15.30	15.70	15.50
Mass of container + Wet soil, g	25.30	29.20	30.90	31.60	22.10	22.50
Mass of container + Dry soil, g	21.20	23.30	24.00	24.10	20.50	20.70
Mass of water, g	4.10	5.90	6.90	7.50	1.60	1.80
Mass of dry soil, g	5.50	7.40	8.40	8.80	4.80	5.20
Water content, %	74.55	79.73	82.14	85.23	33.33	34.62
No of blows (N)	34	27	21	17	------	--------

LL=78 % PL=33 % PI=46%

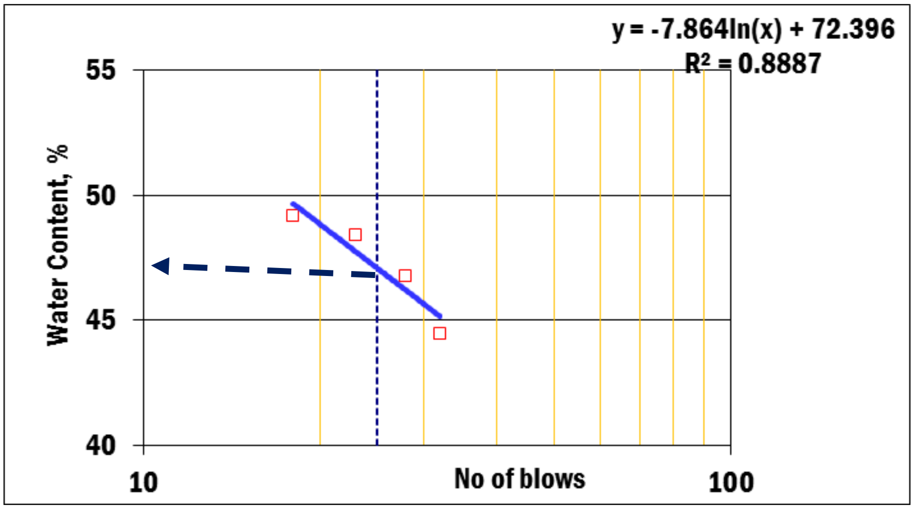


Summary:

For N=25

LL=78%

Figure B.2 Flow curve analysis and plastic limits for TP01@3m

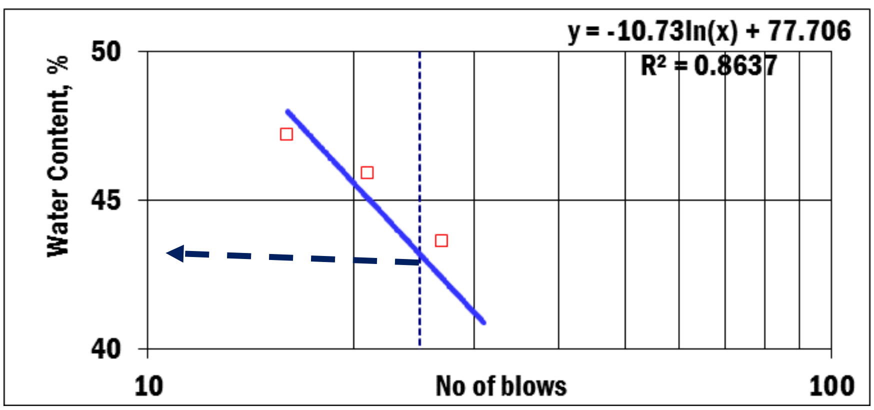


Summary:

For N=25

LL=47%

Figure B.3 Flow curve analysis and plastic limits for TP02@1.5m

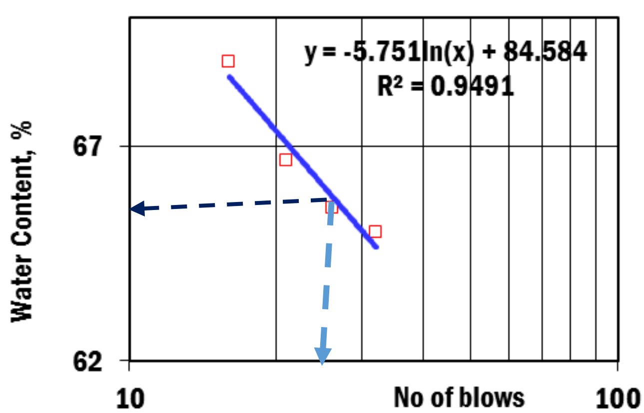


Summary:

For N=25

LL=43%

Figure B.4 Flow curve analysis and plastic limits for TP02@3m

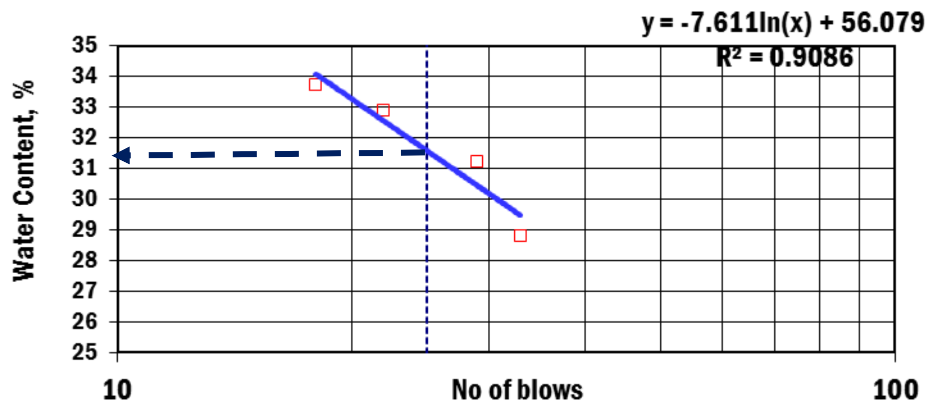


Summary:

For N=25

LL=64%

Figure B.5 Flow curve analysis and plastic limits for TP03@1.5m

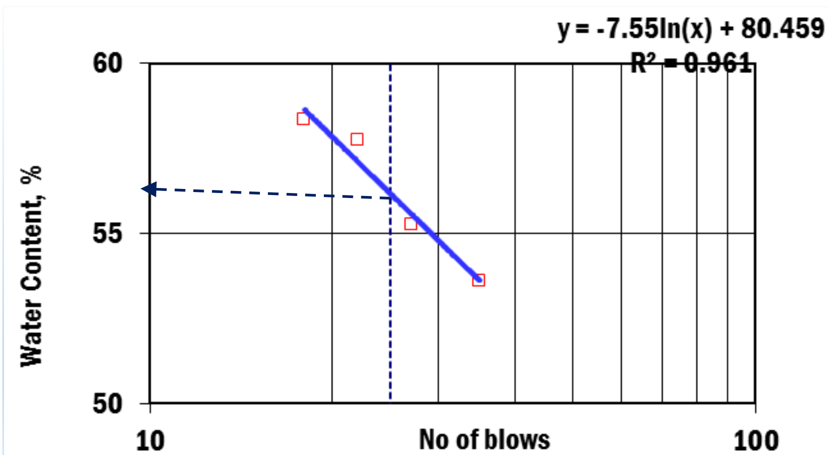


Summary:

For N=25

LL=31.5%

Figure B.6 Flow curve analysis and plastic limits for TP03@3m

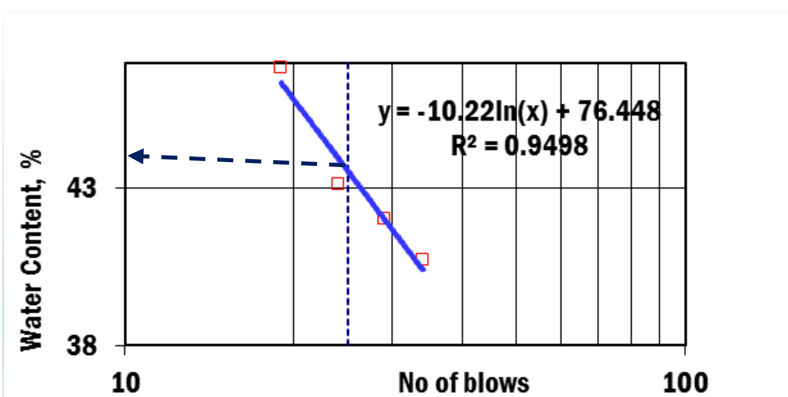


Summary:

For N=25

LL=55.3%

Figure B.7 Flow curve analysis and plastic limits for TP04@1.5m

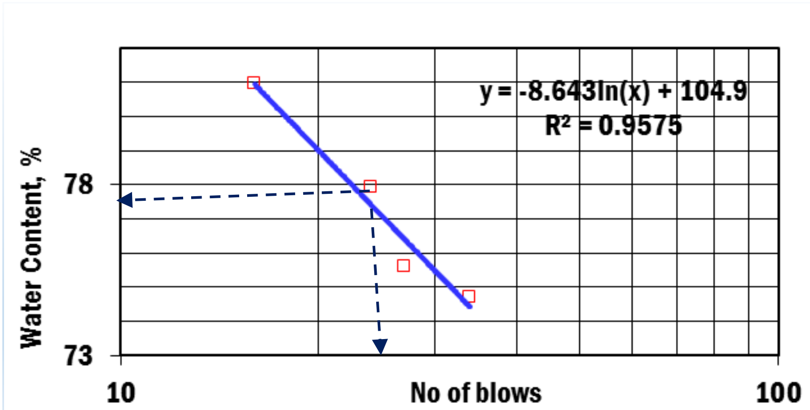


Summary:

For N=25

LL=44%

Figure B.8 Flow curve analysis and plastic limits for TP04@3m

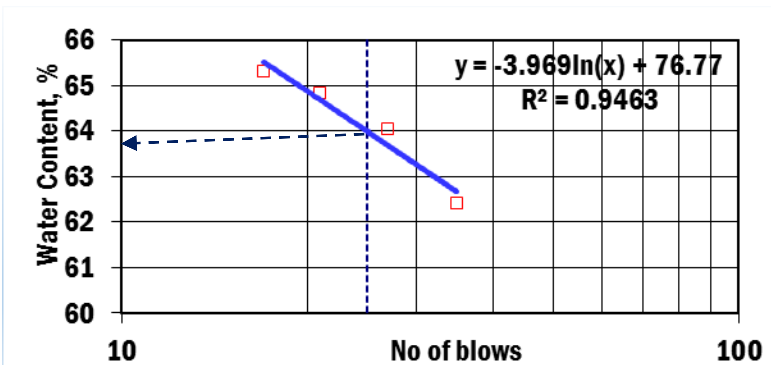


Summary:

For N=25

LL=77%

Figure B.9 Flow curve analysis and plastic limits for TP05@1.5m

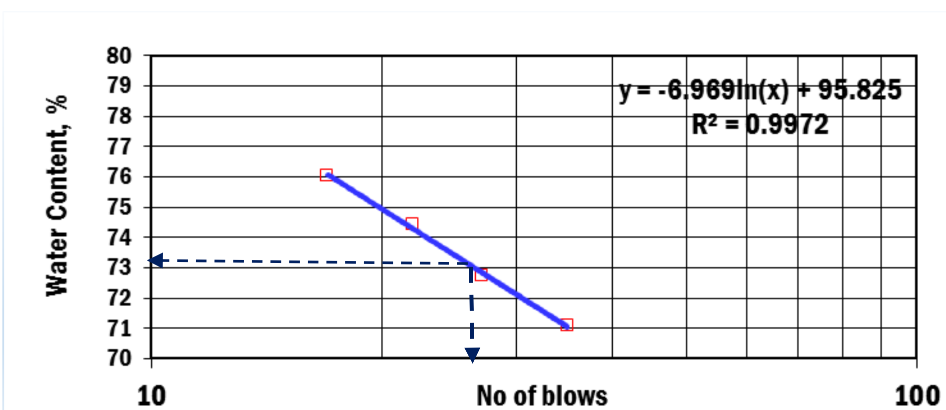


Summary:

For N=25

LL=63.8%

Figure B.10 Flow curve analysis and plastic limits for TP05@3m

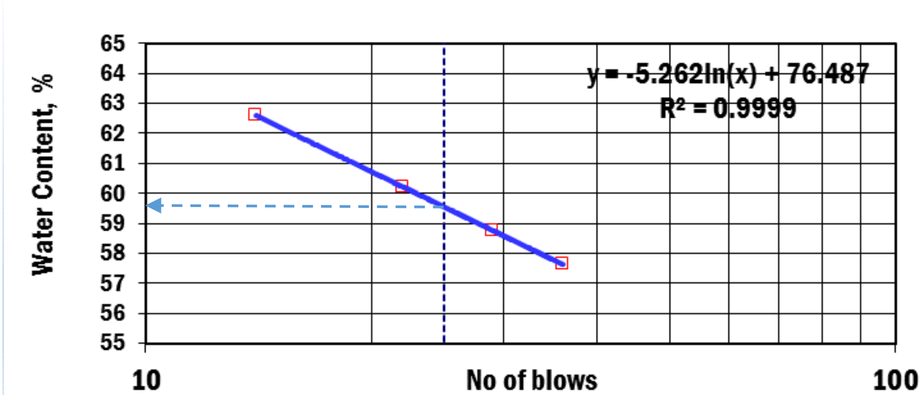


Summary:

For N=25

LL=73.4%

Figure B.11 Flow curve analysis and plastic limits for TP06@1.5m

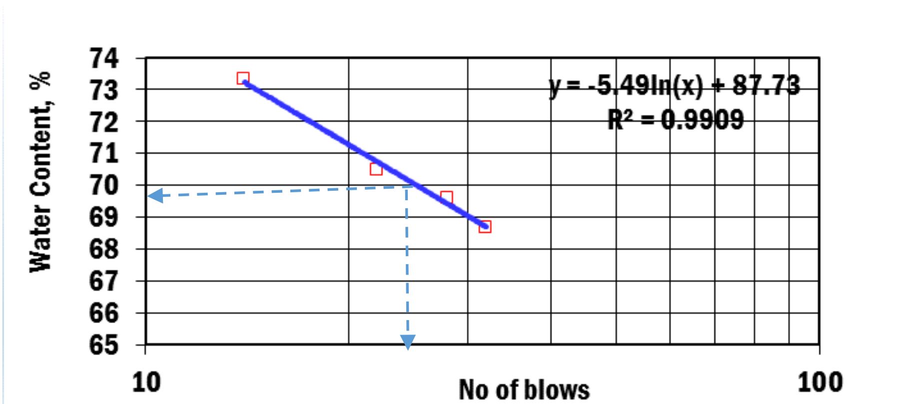


Summary:

For N=25

LL=59.5%

Figure B.12 Flow curve analysis and plastic limits for TP06@3m

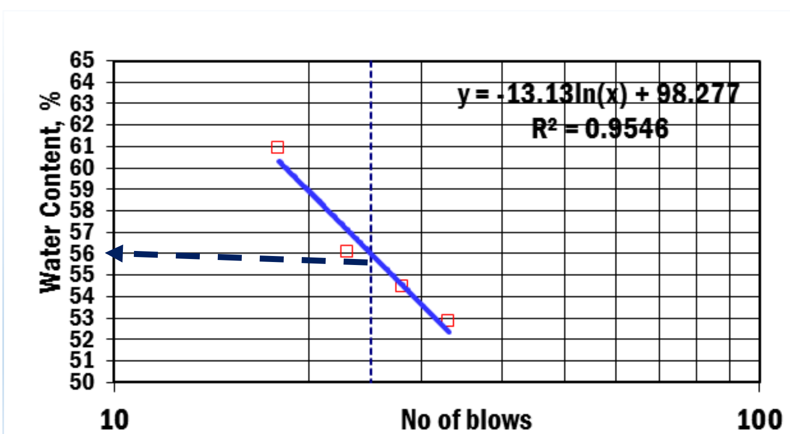


Summary:

For N=25

LL=69.8%

Figure B.13 Flow curve analysis and plastic limits for TP07@1.5m

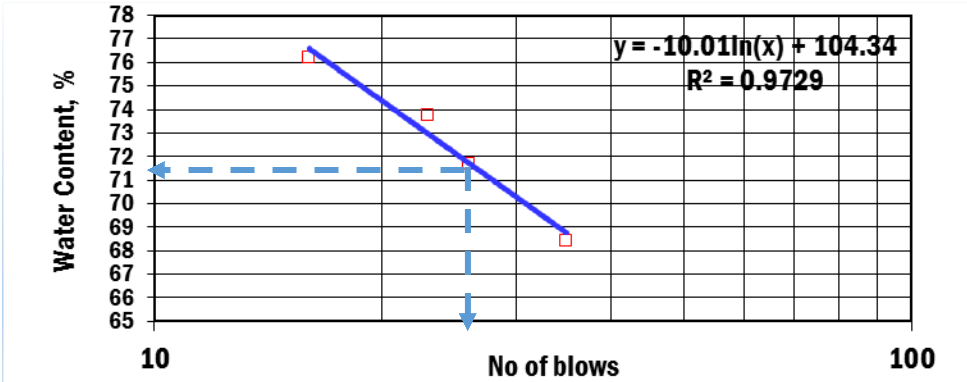


Summary:

For N=25

LL=56%

Figure B.14 Flow curve analysis and plastic limits for TP07@3m

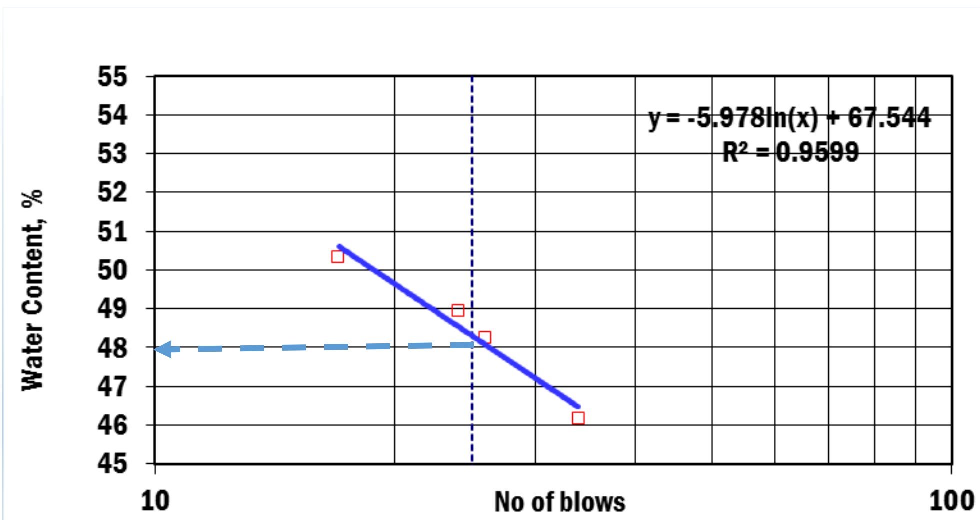


Summary:

For N=25

LL=71.5%

Figure B.15 Flow curve analysis and plastic limits for TP08@1.5m

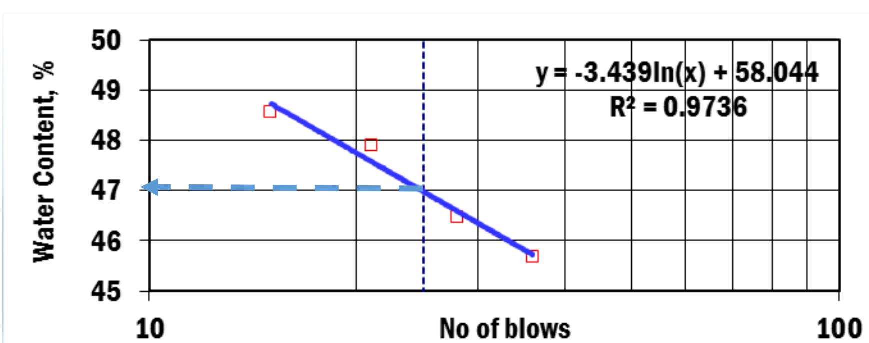


Summary:

For N=25

LL=48%

Figure B.16 Flow curve analysis and plastic limits for TP08@3m

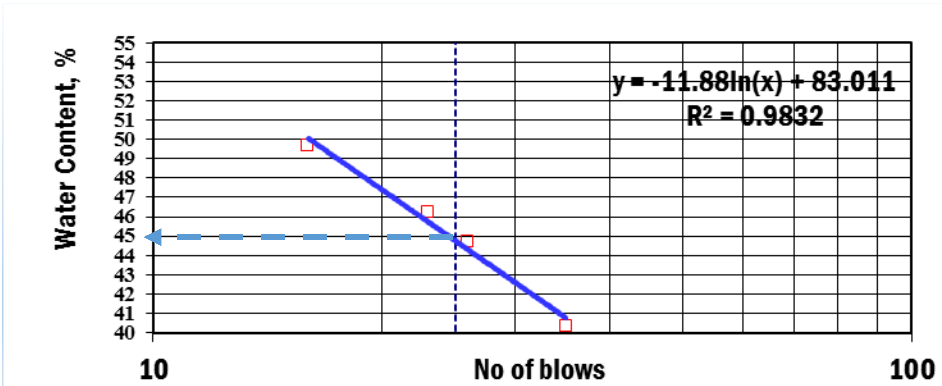


Summary:

For N=25

LL=47%

Figure B.17 Flow curve analysis and plastic limits for TP09@1.5m

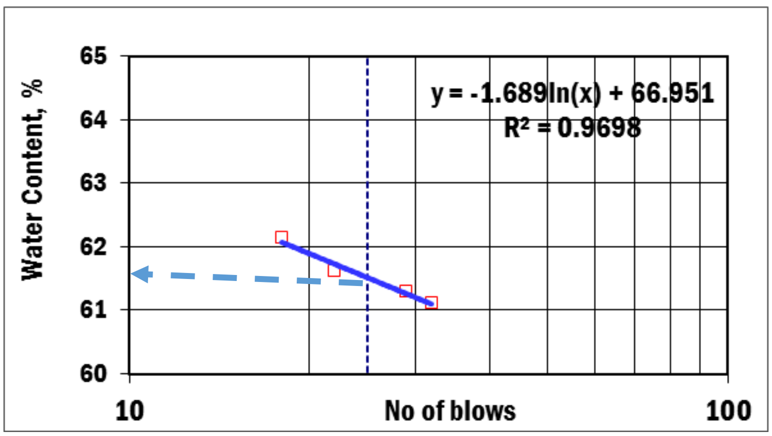


Summary:

For N=25

LL=45%

Figure B.18 Flow curve analysis and plastic limits for TP09@3m

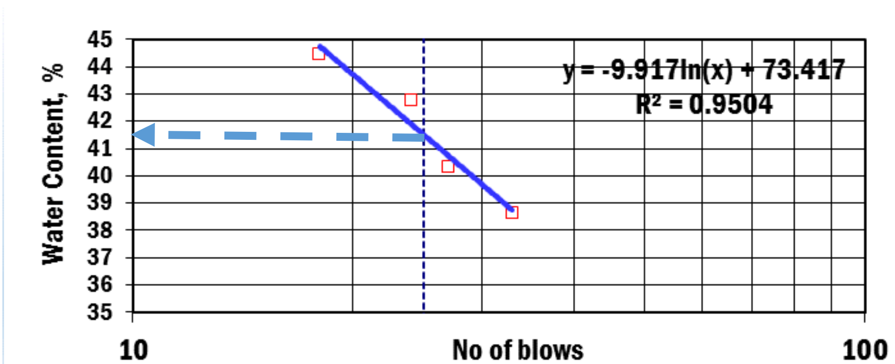


Summary:

For N=25

LL=61.5%

Figure B.19 Flow curve analysis and plastic limits for TP10@1.5m



Summary:

For N=25

LL=41.5%

Figure B.20 Flow curve analysis and plastic limits for TP10@3m

Appendix – C

Compaction Test Results

Sample Number: TP01 **Sample Depth: 1.5 m**

Water Content Determination

Compacted Soil - Sample no.	1	2	3
Moisture can number - Lid number	A	B	C
MC = Mass of empty, clean can + lid (grams)	9.3	9.6	15.9
MCMS = Mass of can, lid, and moist soil (gm)	37.9	28.9	38.2
MCdS = Mass of can, lid, and dry soil (grams)	32.5	24	31.6
MS = Mass of soil solids (grams)	23.2	14.4	15.7
Mw = Mass of pore water (grams)	5.4	4.9	6.6
W = Water content, w%	23.28	34.03	42.04

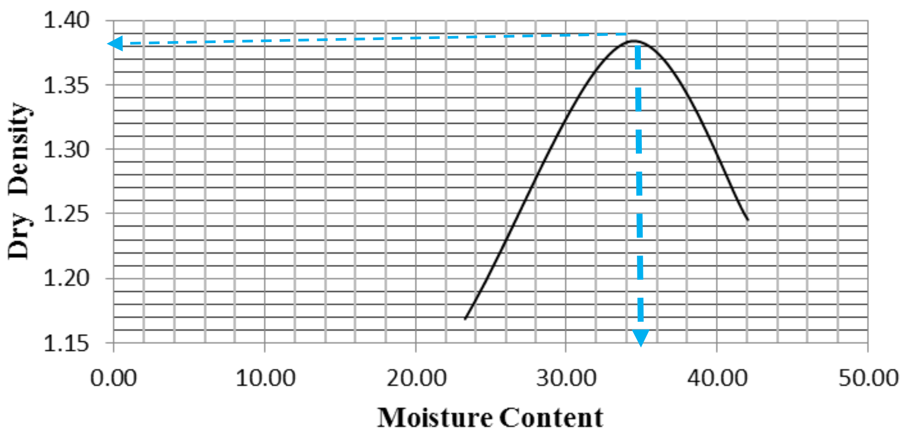
Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3
Mass of compacted soil and mold (grams)	5670	6060	5980
Mass of mold (grams)	4310	4310	4310
Wet mass of soil in mold (grams)	1360	1750	1670
Wet density, ρ , (kg/m)	1.50	1.85	1.77
Dry density, ρd , (kg/m)	1.17	1.38	1.25

OMC	35.00%
MDD	1.39 g/cc

W (%)	23.28	34.03	42.04
DD(g/cc)	1.17	1.38	1.25



Summary:

OMC=35%

MDD=1.39g/cm^3

Figure C.1 Compaction test result for TP01@1.5m

Sample Number: TP01 **Sample Depth: 3m**

Water Content Determination

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	A	B	C	D
MC = Mass of empty, clean can + lid (grams)	15.9	9.5	9.6	15.7
MCMS = Mass of can, lid, and moist soil(gm)	44.7	38.2	34.1	25.2
MCdS = Mass of can, lid, and dry soil (grams)	39.8	31.9	27.5	22.2
MS = Mass of soil solids (grams)	23.9	22.4	17.9	6.5
Mw = Mass of pore water (grams)	4.9	6.3	6.6	3
W = Water content, w%	20.5	28.13	36.87	46.15

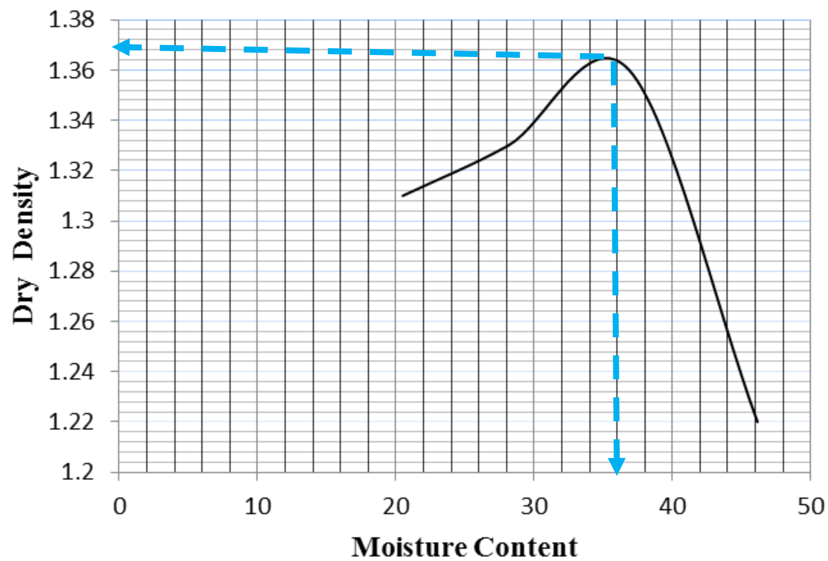
Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold(gm)	5805	5920	6070	5990
Mass of mold (grams)	4310	4310	4310	4310
Wet mass of soil in mold (grams)	1495	1610	1760	1680
Wet density, ρ , (kg/m)	1.58	1.71	1.86	1.78
Dry density, ρd , (kg/m)	1.36	1.38	1.38	1.35

OMC	36.00%
MDD	1.37 g/cc

W (%)	28.50	30.13	38.87	46.15
DD(g/cc)	1.36	1.38	1.38	1.35

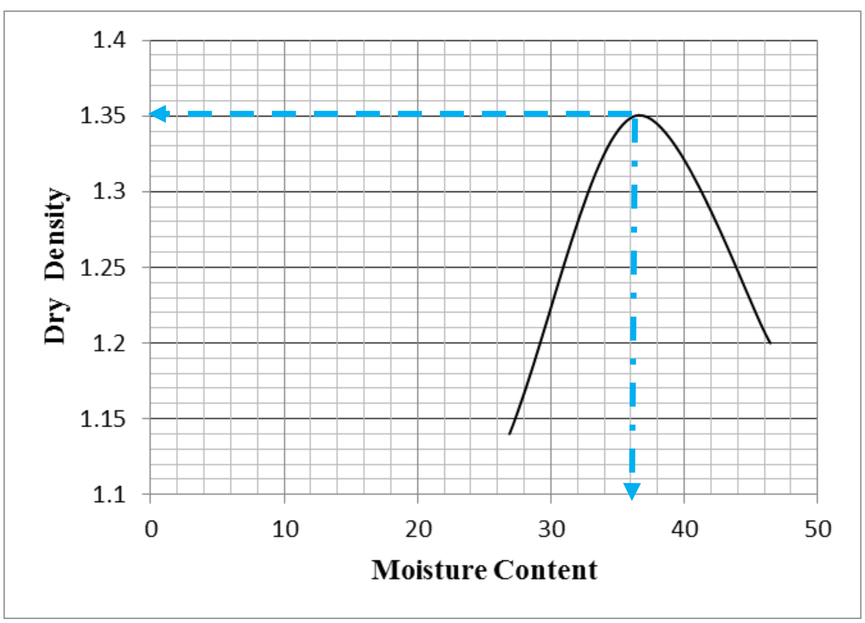


Summary:

OMC=36%

MDD=1.37g/cm^3

Figure C.2 Compaction test result for TP01@3m

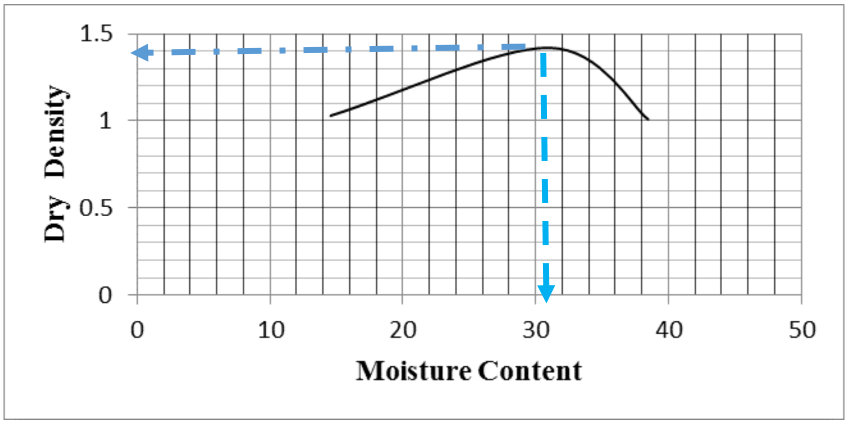


Summary:

OMC=36%

MDD=1.35g/cm^3

Figure C.3 Compaction test result for TP02@1.5m

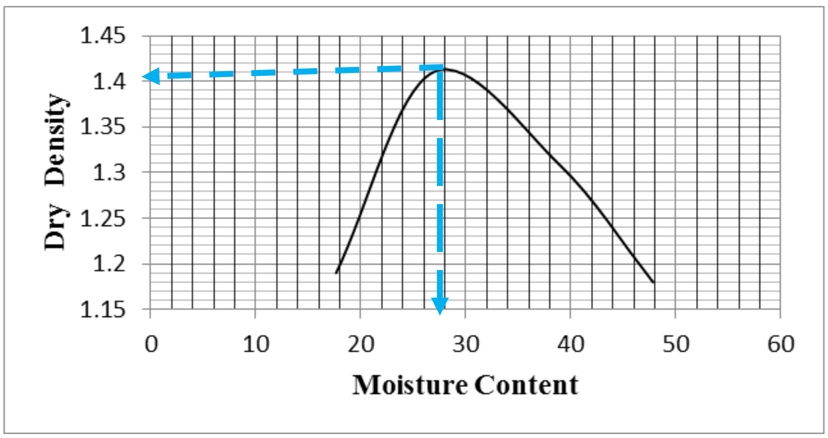


Summary:

OMC=31%

MDD=1.41g/cm^3

Figure C.4 Compaction test result for TP02@3m

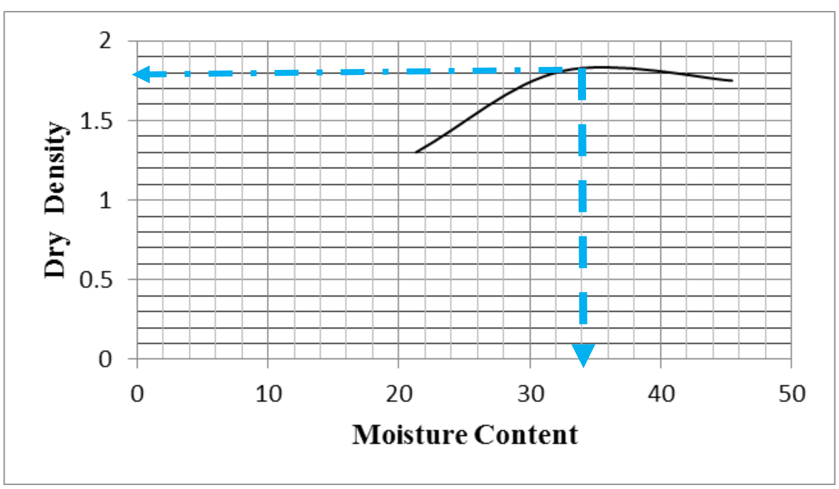


Summary:

OMC=28%

MDD=1.42g/cm^3

Figure C.5 Compaction test result for TP03@1.5m

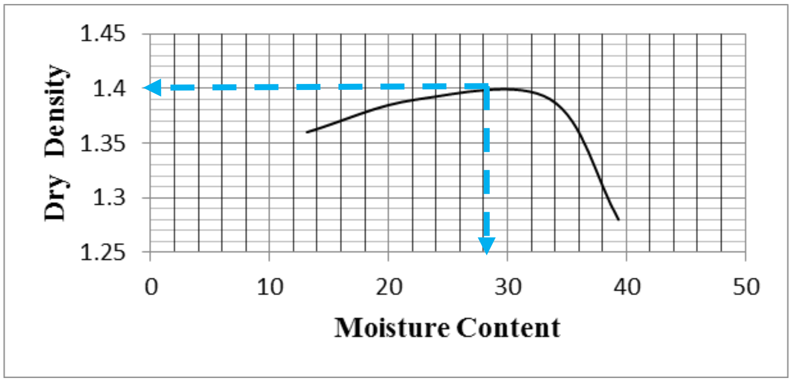


Summary:

OMC=34%

MDD=1.36g/cm^3

Figure C.6 Compaction test result for TP03@3m

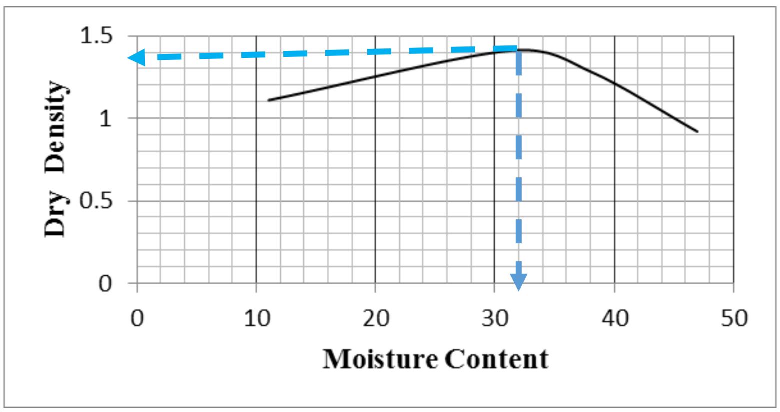


Summary:

OMC=28%

MDD=1.40g/cm^3

Figure C.7 Compaction test result for TP04@1.5m

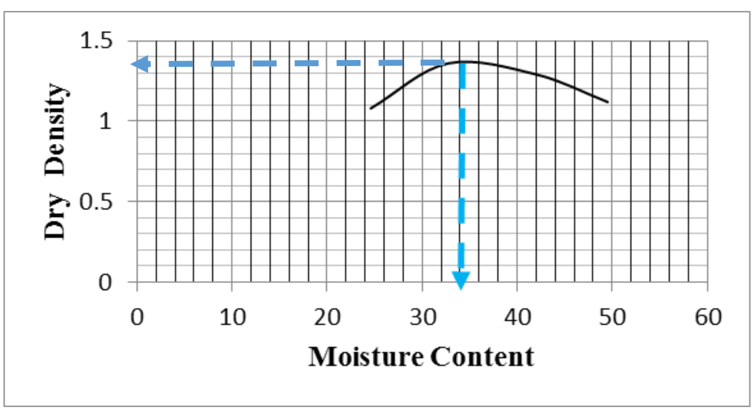


Summary:

OMC=32%

MDD=1.36g/cm^3

Figure C.8 Compaction test result for TP04@3m

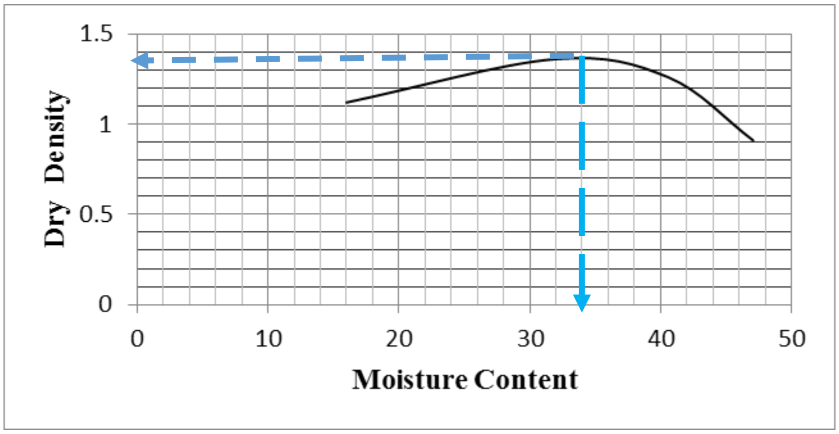


Summary:

OMC=34%

MDD=1.37g/cm^3

Figure C.9 Compaction test result for TP05@1.5m

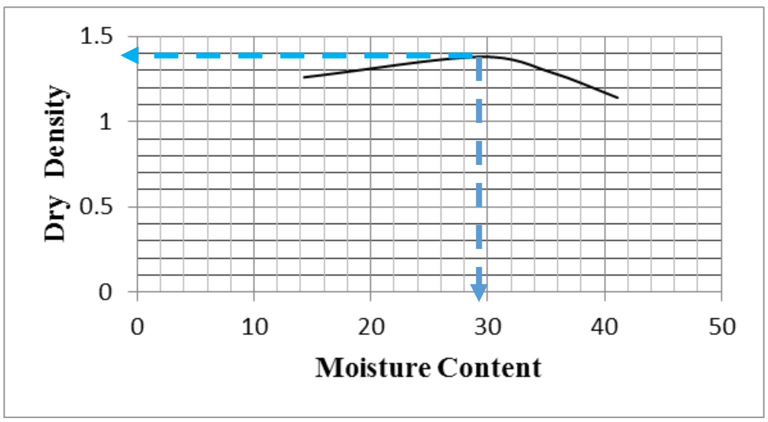


Summary:

OMC=34%

MDD=1.39g/cm^3

Figure C.10 Compaction test result for TP05@3m

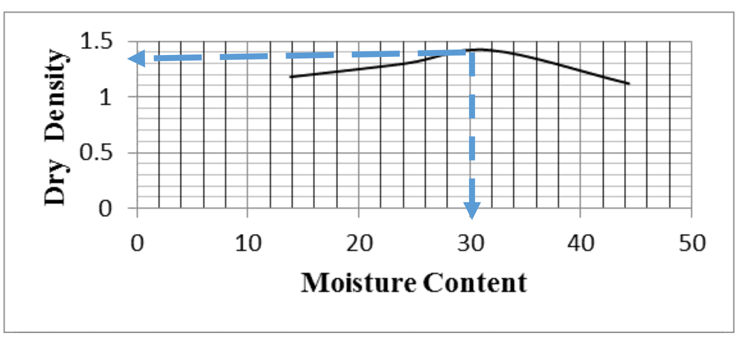


Summary:

OMC=29%

MDD=1.38g/cm^3

Figure C.11 Compaction test result for TP06@1.5m

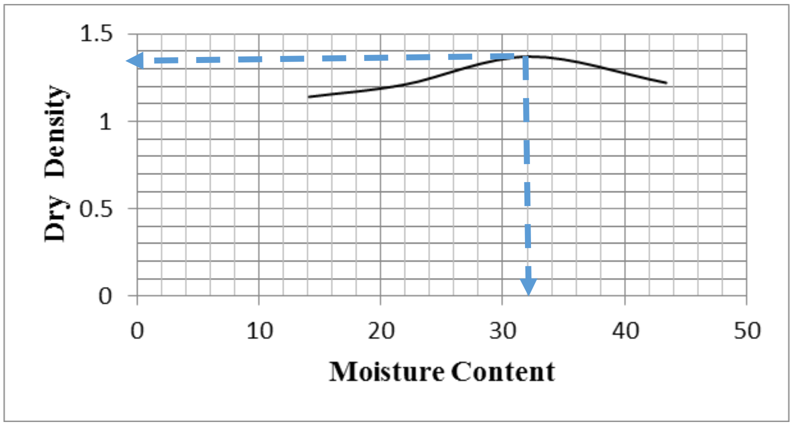


Summary:

OMC=30%

MDD=1.42g/cm^3

Figure C.12 Compaction test result for TP06@3m

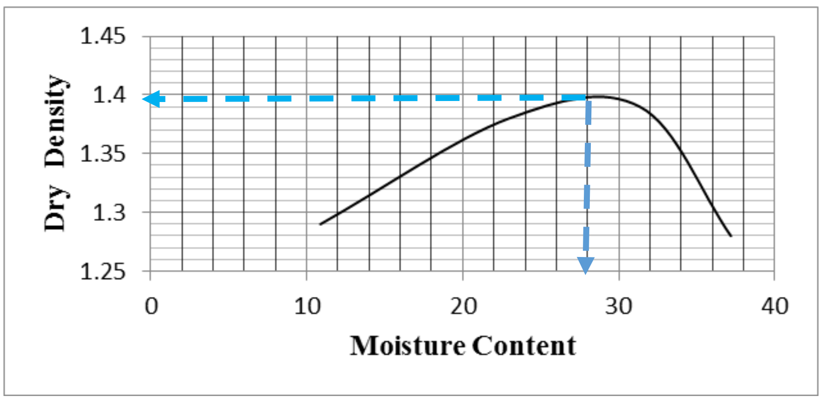


Summary:

OMC=32%

MDD=1.37g/cm^3

Figure C.13 Compaction test result for TP07@1.5m

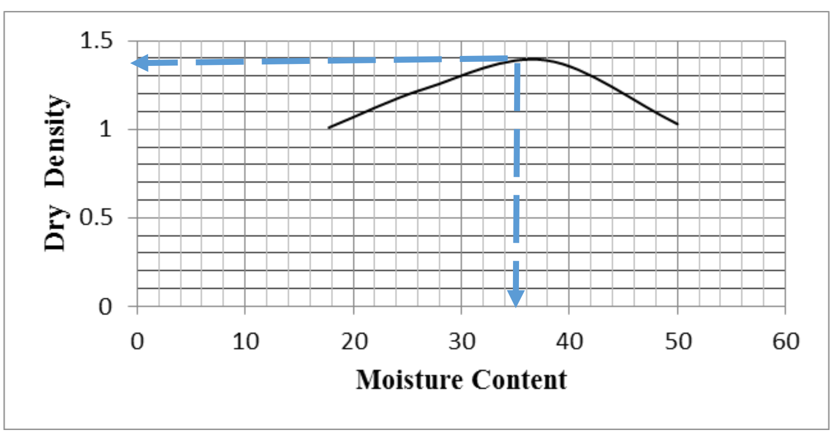


Summary:

OMC=28%

MDD=1.39g/cm^3

Figure C.14 Compaction test result for TP07@3m

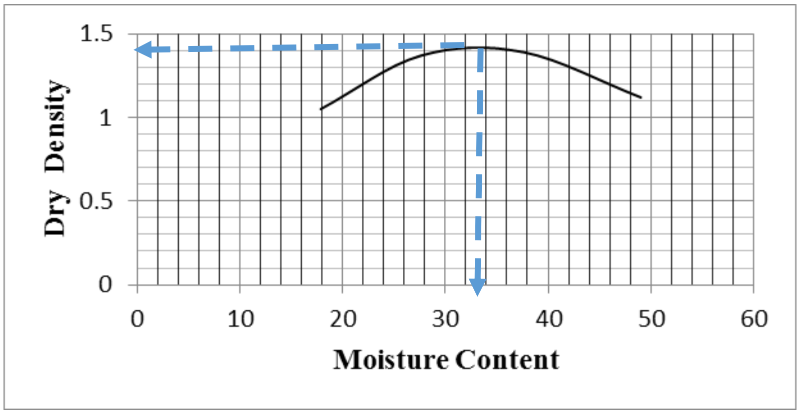


Summary:

OMC=35%

MDD=1.38g/cm^3

Figure C.15 Compaction test result for TP08@1.5m

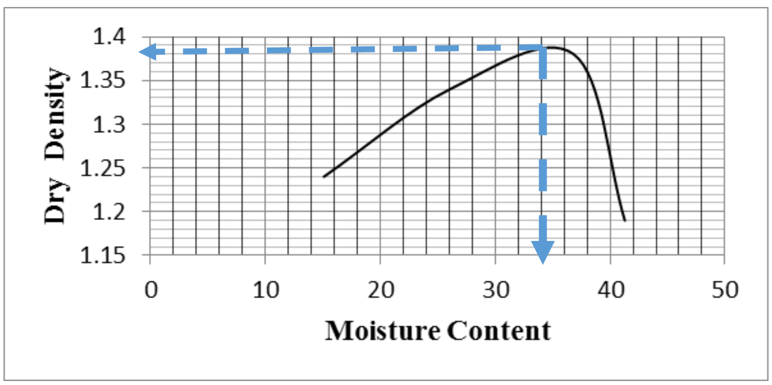


Summary:

OMC=33%

MDD=1.41

Figure C.16 Compaction test result for TP08@3m

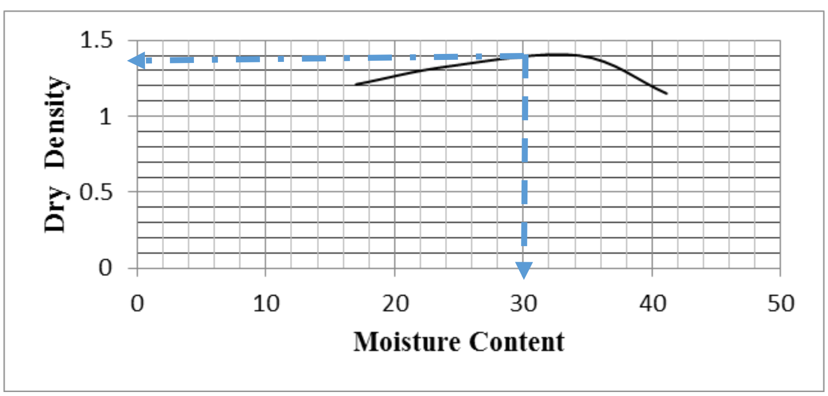


Summary:

OMC=34%

MDD=1.36g/cm^3

Figure C.17 Compaction test result for TP09@1.5m

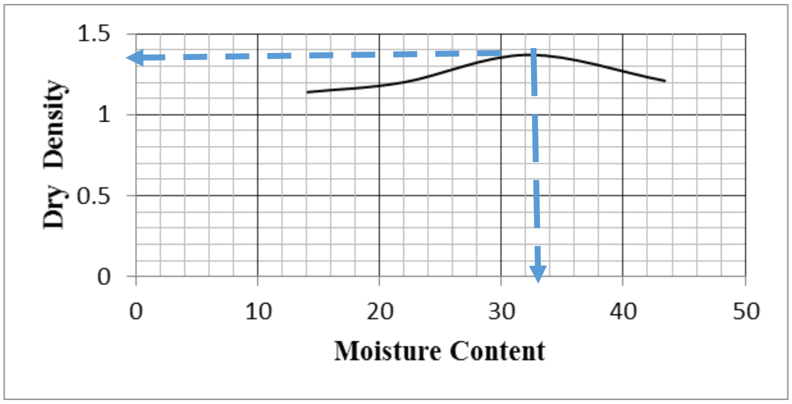


Summary:

OMC=30%

MDD=1.35g/cm^3

Figure C.18 Compaction test result for TP09@3m

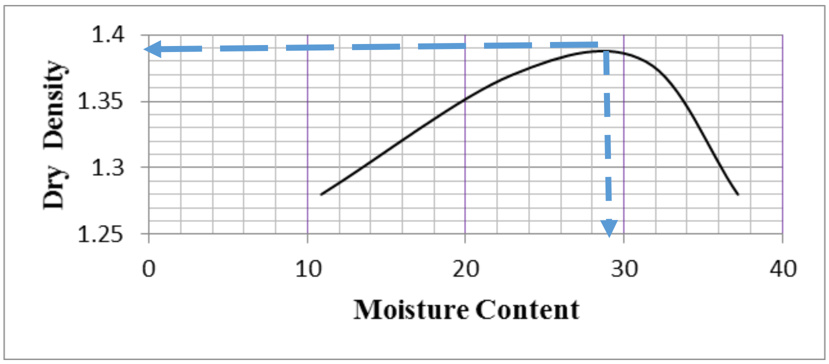


Summary:

OMC=33%

MDD=1.37g/cm^3

Figure C.19 Compaction test result for TP10@1.5m



Summary:

OMC=29%

MDD=1.38g/cm^3

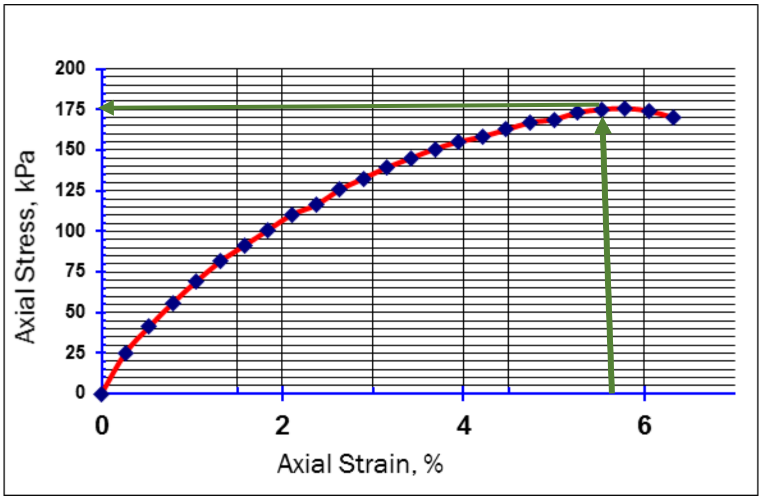
Figure C.20 Compaction test result for TP10@3m

Appendix – D

Unconfined Compression Test Results

Test Pit No: TP01	**Cross- Sectional Area: 0.001134 m^2**
Depth of Sample:3.00m	**Ring Calibration Factor: 0.00142 kN/div**
Sampling type: Undisturbed	**Moisture content: 33.90%**
Diameter of sample:38mm	**Rate of Strain: 1.70 mm/min**
Length of sample: 76mm	

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.26	20	0.0284	0.001137	24.98
0.40	0.53	33	0.0469	0.001140	41.10
0.60	0.79	45	0.0639	0.001143	55.90
0.80	1.05	56	0.0795	0.001146	69.38
1.00	1.32	66	0.0937	0.001149	81.55
1.20	1.58	74	0.1051	0.001152	91.19
1.40	1.84	82	0.1164	0.001155	100.78
1.60	2.11	90	0.1278	0.001159	110.31
1.80	2.37	95	0.1349	0.001162	116.13
2.00	2.63	103	0.1463	0.001165	125.57
2.20	2.89	109	0.1548	0.001168	132.53
2.40	3.16	115	0.1633	0.001171	139.44
2.60	3.42	120	0.1704	0.001174	145.11
2.80	3.68	125	0.1775	0.001177	150.74
3.00	3.95	129	0.1832	0.001181	155.14
3.20	4.21	132	0.1874	0.001184	158.32
3.40	4.47	136	0.1931	0.001187	162.66
3.60	4.74	140	0.1988	0.001191	166.99
3.80	5.00	142	0.2016	0.001194	168.91
4.00	5.26	146	0.2073	0.001197	173.18
4.20	5.53	148	0.2102	0.001200	175.07
4.40	5.79	149	0.2116	0.001204	175.76
4.60	6.05	148	0.2102	0.001207	174.09
4.80	6.32	145	0.2059	0.001211	170.08
5.00	6.58	140	0.1988	0.001214	163.76



Summary:

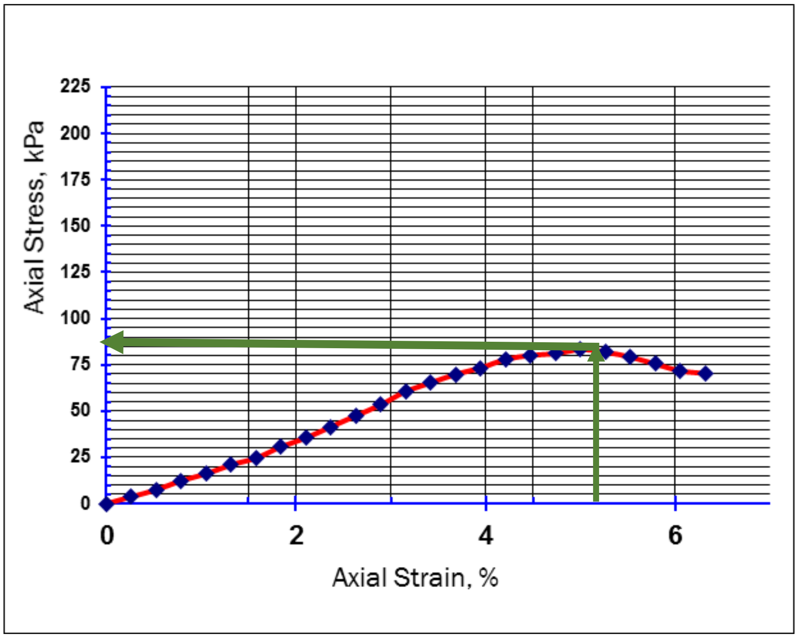
q_u=175kPa

c_u = q_u/2=87.5 kPa

Figure D.1: Unconfined compression test result for TP01@3m

Test Pit No: TP02	**Cross- Sectional Area: 0.001134 m^2**
Depth of Sample:3.00m	**Ring Calibration Factor: 0.001134 kN/div**
Sampling type: Undisturbed	**Moisture content: 58.99%**
Diameter of sample:38mm	**Rate of Strain: 1.70 mm/min**
Length of sample: 76mm	

Axial Deformation [mm]	Axial Strain [%]	Proving Ring Reading [div]	Axial Load [kN]	Corrected Area [m2]	Axial Stress [kPa]
0.00	0.00	0	0.0000	0.001134	0
0.20	0.26	3	0.0043	0.001137	3.75
0.40	0.53	6	0.0085	0.001140	7.47
0.60	0.79	10	0.0142	0.001143	12.42
0.80	1.05	13	0.0185	0.001146	16.11
1.00	1.32	17	0.0241	0.001149	21.01
1.20	1.58	20	0.0284	0.001152	24.65
1.40	1.84	25	0.0355	0.001155	30.73
1.60	2.11	29	0.0412	0.001159	35.55
1.80	2.37	34	0.0483	0.001162	41.56
2.00	2.63	39	0.0554	0.001165	47.55
2.20	2.89	44	0.0625	0.001168	53.50
2.40	3.16	50	0.0710	0.001171	60.63
2.60	3.42	54	0.0767	0.001174	65.30
2.80	3.68	58	0.0824	0.001177	69.95
3.00	3.95	61	0.0866	0.001181	73.36
3.20	4.21	65	0.0923	0.001184	77.96
3.40	4.47	67	0.0951	0.001187	80.14
3.60	4.74	68	0.0966	0.001191	81.11
3.80	5.00	70	0.0994	0.001194	83.26
4.00	5.26	69	0.0980	0.001197	81.85
4.20	5.53	67	0.0951	0.001200	79.25
4.40	5.79	64	0.0909	0.001204	75.49
4.60	6.05	61	0.0866	0.001207	71.75

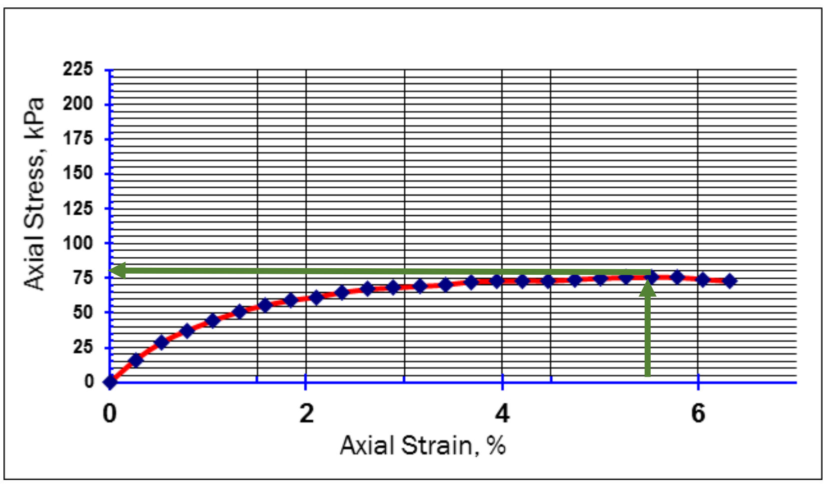


Summary:

q_u=85 kPa

c_u=42.5kPa

Figure D.2: Unconfined compression test result for TP02@3m

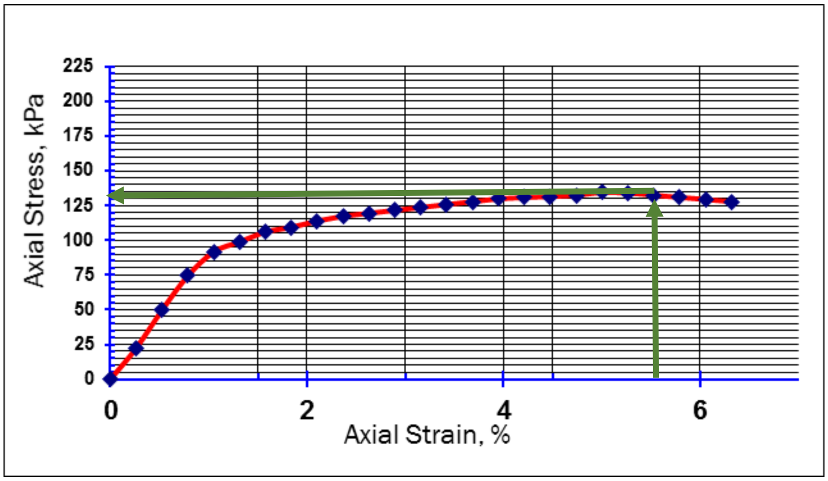


Summary:

q_u=76kPa

c_u=38kPa

Figure D.3: Unconfined compression test result for TP03@3m

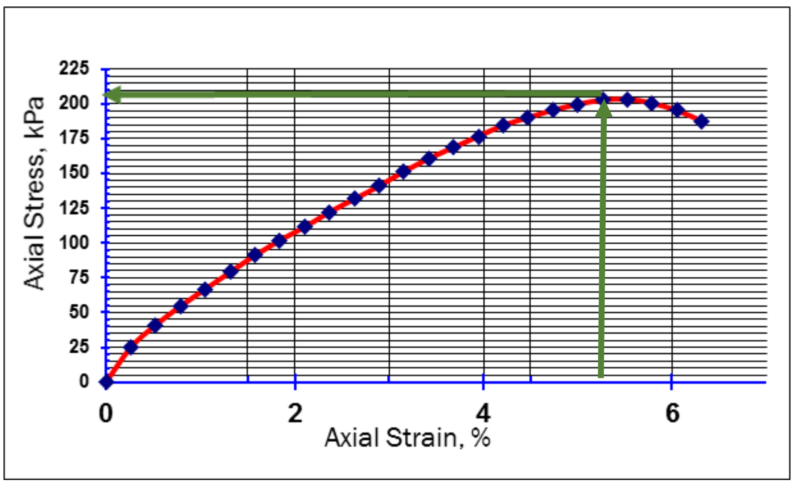


Summary:

q_u=136kpa

c_u=68kPa

Figure D.4: Unconfined compression test result for TP04@3m

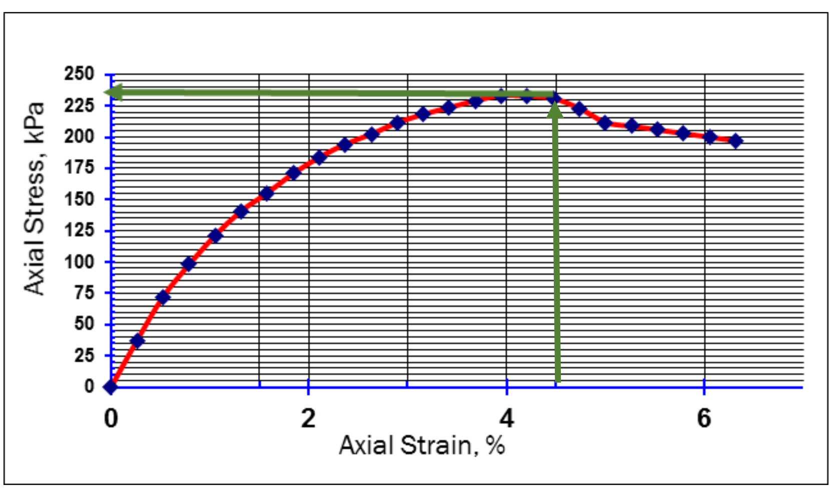


Summary:

q_u=204kpa

c_u=102kPa

Figure D.5: Unconfined compression test result for TP05@3m

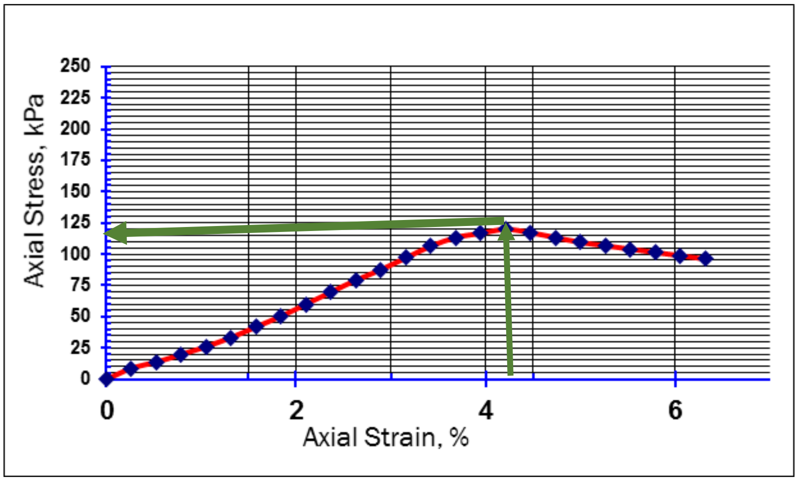


Summary:

q_u=234kPa

c_u=117kPa

Figure D.6: Unconfined compression test result for TP06@3m

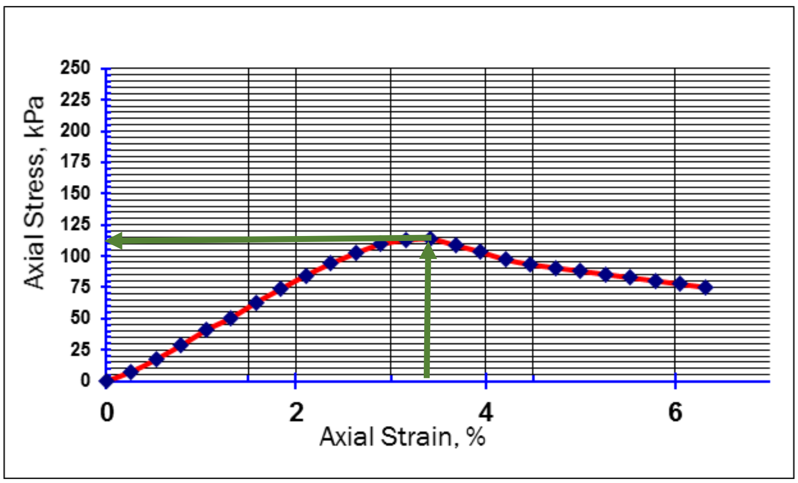


Summary:

q_u=120kPa

c_u=60kPa

Figure D.7: Unconfined compression test result for TP07@3m

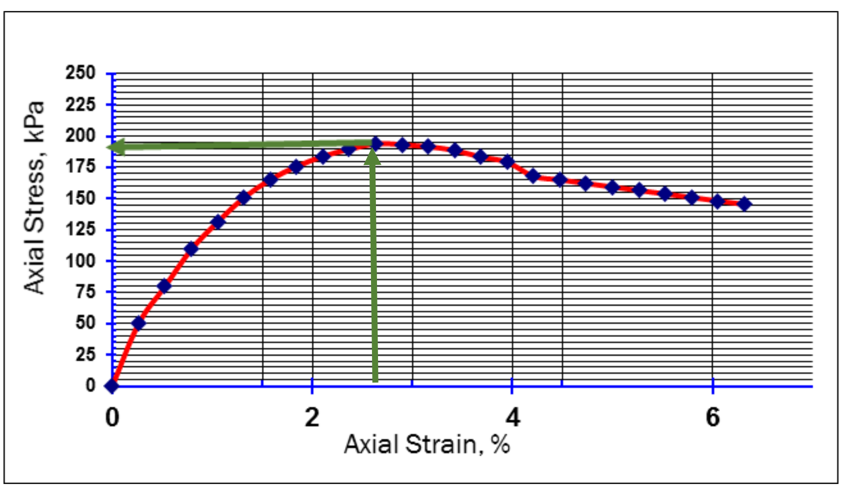


Summary:

q_u=112kPa

c_u=56kPa

Figure D.8: Unconfined compression test result for TP08@3m

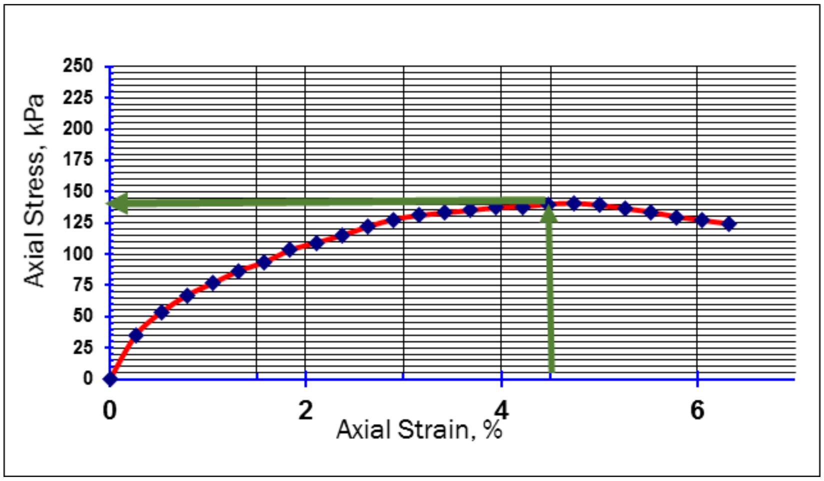


Summary:

q_u=194kPa

c_u=97kPa

Figure D.9: Unconfined compression test result for TP09@3m



Summary:

q_u=144kPa

C_u=72kPa

Figure D.10: Unconfined compression test result for TP10@3m

Appendix – E

One Dimensional Consolidation Results

Sample No: TP02 **Sample Depth: 3.00m**

		Dial Gauge Reading, mm						
Time(min.)	Ö time	7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.042	4.042	4.066	4.268	4.540	4.822	5.192
0.1	0.32	4.042	4.044	4.142	4.318	4.560	4.850	5.370
0.25	0.50	4.042	4.046	4.150	4.320	4.562	4.854	5.374
0.5	0.71	4.042	4.046	4.152	4.322	4.566	4.860	5.378
1	1.00	4.042	4.050	4.154	4.324	4.570	4.866	5.382
2	1.41	4.042	4.050	4.156	4.332	4.576	4.870	5.386
4	2.00	4.042	4.052	4.158	4.336	4.580	4.892	5.388
8	2.83	4.042	4.052	4.160	4.340	4.584	4.904	5.394
15	3.87	4.042	4.054	4.162	4.342	4.590	4.912	5.48
30	5.48	4.042	4.056	4.164	4.344	4.596	4.928	5.526
60	7.75	4.042	4.058	4.168	4.352	4.608	4.946	5.548
120	10.95	4.042	4.060	4.170	4.366	4.612	4.952	5.573
240	15.49	4.042	4.062	4.174	4.372	4.616	4.958	5.576
480	21.91	4.042	4.064	4.176	4.398	4.624	5.164	5.584
960	30.98	4.042	4.064	4.220	4.420	4.628	5.170	5.588
1440	37.95	4.042	4.066	4.268	4.540	4.822	5.192	5.594

Cumulative Dial Gauge Reading at the End of Each Consecutive Unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	7 [kPa]
5.594	5.620	5.701	5.52.	5.56	5.130	5.124

Applied Pressure P [kPa]	Final Dial Reading [mm]	Change in Specimen Height [mm]	Final Specimen Height [mm]	Void Height, H_V	Void Ratio,E
Loading					
7	4.042	0.000	20.00	11.12	1.251
50	4.066	0.024	19.98	11.09	1.249
100	4.268	0.226	19.77	10.89	1.226
200	4.540	0.498	19.50	10.62	1.195
400	4.822	0.780	19.22	10.34	1.163
800	5.192	1.150	18.85	9.97	1.122
1600	5.594	1.552	18.45	9.56	1.077

Unloading					
1600	5.594	1.552	18.45	9.56	1.077
400	5.700	1.658	18.34	9.46	1.065
100	5.56	1.518	18.48	9.60	1.080
7	5.124	1.082	18.92	10.03	1.129

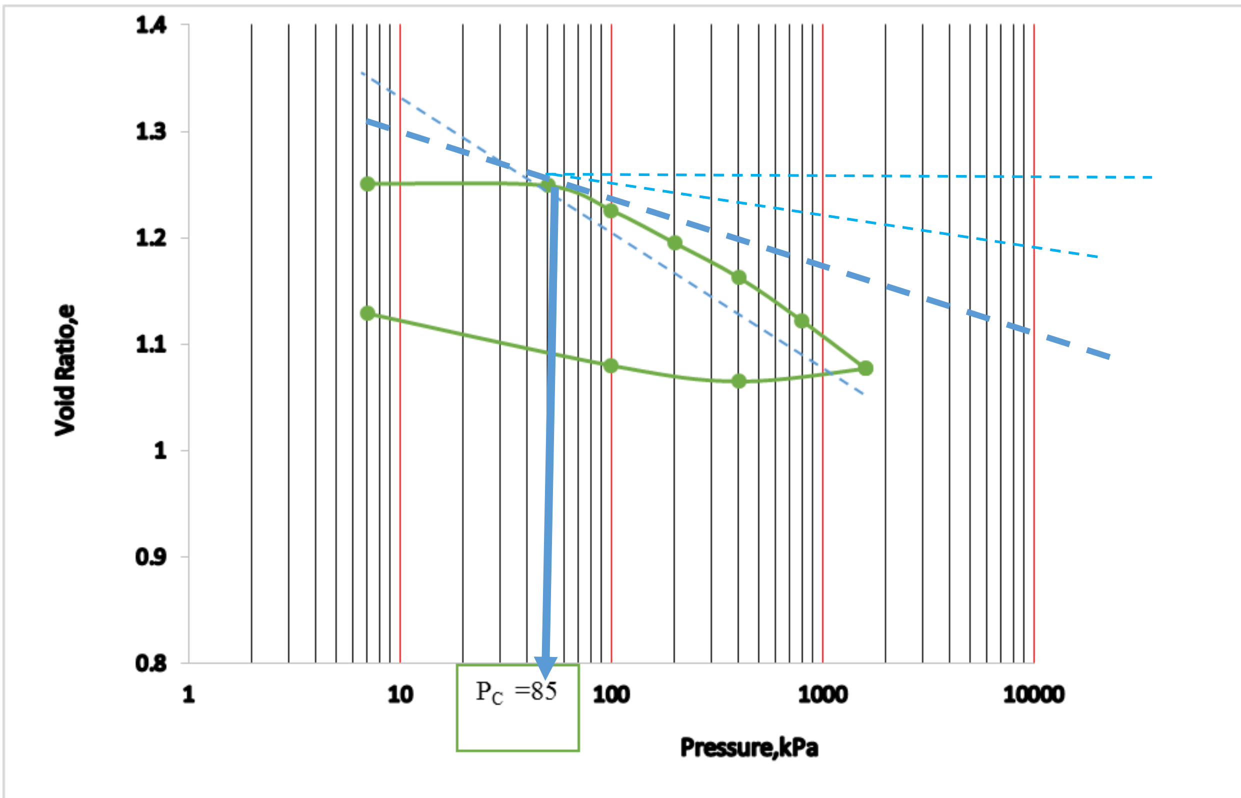


Figure E-1: Void ratio Vs log p curve for TP02 to determine Pc

Sample No: TP04 **Sample Depth: 3.00m**

		Dial Gauge Reading, mm						
Time(min.)	Ö time	7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	7.000	7.026	6.450	6.074	5.502	4.700	3.672
0.1	0.32	-	6.868	6.398	6.002	5.420	4.632	3.612
0.25	0.50	-	6.852	6.390	5.992	5.404	4.618	3.596
0.50	0.71	-	6.840	6.382	5.980	5.392	4.602	3.572
1	1.00	-	6.818	6.372	5.968	5.370	4.576	3.540
2	1.41	-	6.790	6.354	5.938	5.336	4.534	3.488
4	2.00	-	6.746	6.324	5.898	5.280	4.464	3.420
8	2.83	-	6.682	6.284	5.840	5.200	4.456	3.326
15	3.87	-	6.614	6.242	5.770	5.106	4.448	3.212
30	5.48	-	6.552	6.194	5.694	4.992	4.304	3.084
60	7.75	-	6.514	6.160	5.616	4.900	4.178	2.954
120	10.95	-	6.490	6.136	5.596	4.836	4.082	2.826
240	15.49	-	6.472	6.110	5.564	4.790	3.822	2.744
480	21.91	-	6.460	6.096	5.534	4.748	3.772	2.680
1440	37.95	7.026	6.450	6.074	5.502	4.700	3.672	2.616

Cumulative Dial Gauge Reading at the End of Each Consecutive Unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	7 [kPa]
2.616		2.840		3.120		3.520

Applied Pressure P [kPa]	Final Dial Reading [mm]	Change in Specimen Height [mm]	Final Specimen Height [mm]	Void Height, H_V	Void Ratio,E
Loading					
7	7.000	0.00	20.00	11.58	1.38
7	7.026	0.03	20.03	11.61	1.38
50	6.450	-0.55	19.45	11.03	1.31
100	6.074	-0.93	19.07	10.66	1.27
200	5.502	-1.50	18.50	10.09	1.20
400	4.700	-2.30	17.70	9.28	1.10
800	3.672	-3.33	16.67	8.26	0.98
1600	2.616	-4.38	15.62	7.20	0.86
Unloading					
1600	2.616	-4.38	15.62	7.20	0.86
400	2.840	-4.16	15.84	7.42	0.88
100	3.120	-3.88	16.12	7.70	0.92
7	3.520	-3.48	16.52	8.10	0.96

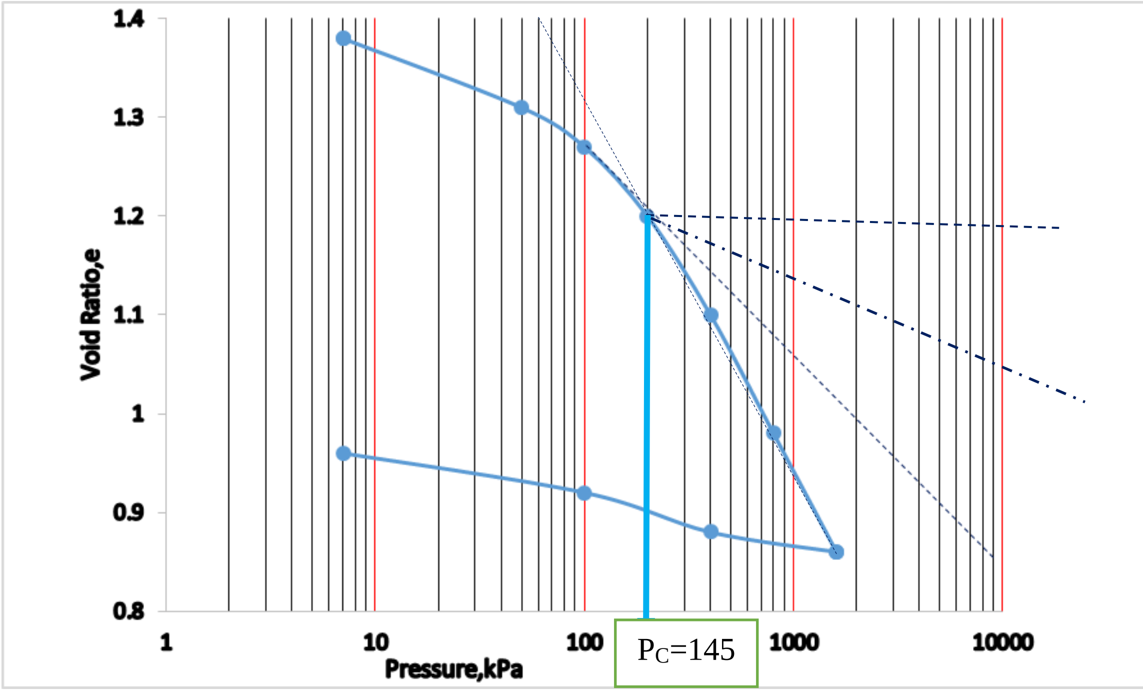


Figure E-2: Void ratio Vs log p curve for TP04 to determine Pc

Sample No: TP07 **Sample Depth: 3.00m**

		Dial Gauge Reading, mm						
Time(min.)	Ö time	7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	8.000	8.282	8.146	8.026	7.838	7.564	7.100
0.1	0.32	-	8.214	8.124	7.950	7.824	7.390	6.916
0.25	0.50	-	8.206	8.108	7.936	7.688	7.384	6.908
0.50	0.71	-	8.202	8.100	7.926	7.682	7.382	6.878
1	1.00	-	8.196	8.094	7.920	7.674	7.334	6.856
2	1.41	-	8.190	8.088	7.914	7.670	7.328	6.830
4	2.00	-	8.186	8.080	7.910	7.652	7.322	6.800
8	2.83	-	8.178	8.072	7.894	7.632	7.276	6.700
15	3.87	-	8.176	8.068	7.872	7.620	7.270	6.658
30	5.48	-	8.172	8.050	7.864	7.612	7.222	6.632
60	7.75	-	8.170	8.060	7.858	7.590	7.194	6.610
120	10.95	-	8.168	8.044	7.852	7.582	7.184	6.600
240	15.49	-	8.164	8.030	7.844	7.574	7.180	6.582
480	21.91	-	8.154	8.030	7.842	7.570	7.164	6.554
1440	37.95	8.282	8.146	8.026	7.838	7.564	7.100	6.504

Cumulative Dial Gauge Reading at the End of Each Consecutive Unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	7 [kPa]
6.504	6.710	6.910	7.110	6.504	6.710	6.910

Applied Pressure P [kPa]	Final Dial Reading [mm]	Change in Specimen Height [mm]	Final Specimen Height [mm]	Void Height, H_V	Void Ratio,E
Loading					
7	8.000	0.00	20.00	11.25	1.29
7	8.282	0.28	20.78	11.53	1.32
50	8.146	0.15	20.15	11.39	1.30
100	8.026	0.03	20.03	11.27	1.29
200	7.838	-0.16	19.84	11.09	1.27
400	7.564	-0.44	19.56	10.81	1.24
800	7.100	-0.90	19.10	10.35	1.18
1600	6.504	-1.50	18.50	9.75	1.11
Unloading					
1600	6.504	-1.50	18.50	9.75	1.11
400	6.710	-1.29	18.71	9.96	1.14
100	6.910	-1.09	18.91	10.16	1.16
7	7.110	-0.89	19.11	10.36	1.18

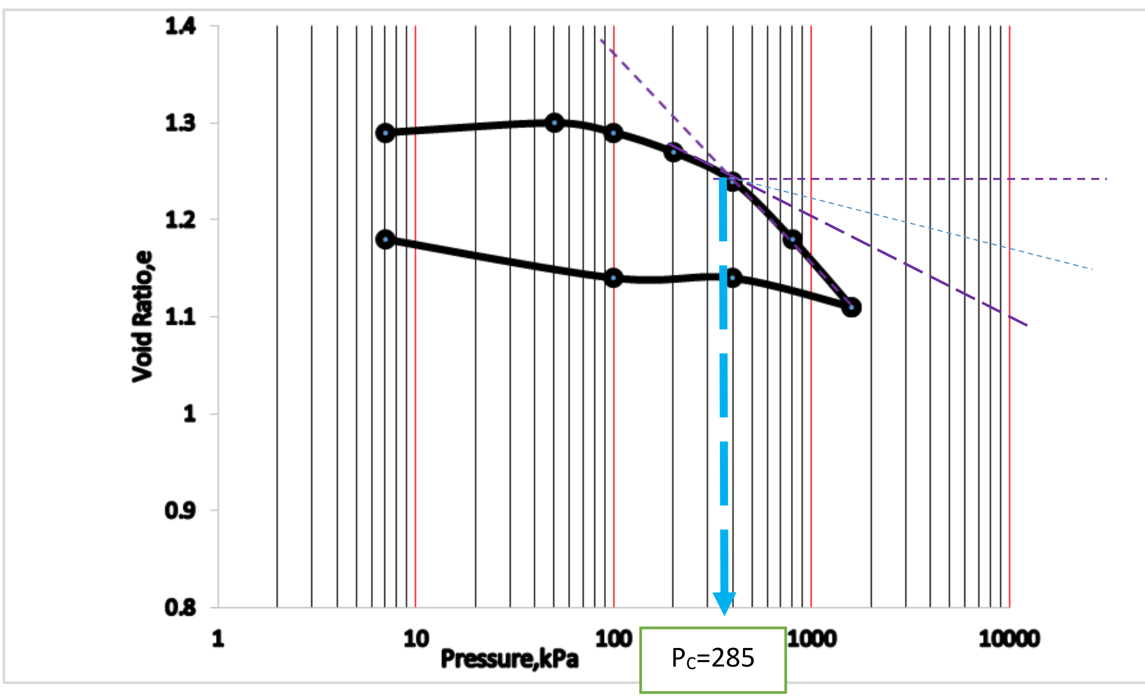


Figure E-3: Void ratio Vs log p curve for TP07 to determine Pc

Sample No: TP08 **Sample Depth: 3.00m**

		Dial Gauge Reading, mm						
Time(min.)	Ö time	7 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	6.000	6.216	6.134	5.996	5.726	5.292	4.200
0.1	0.32	-	6.188	6.052	5.840	5.510	5.200	4.176
0.25	0.50	-	6.178	6.044	5.830	5.490	5.166	4.054
0.50	0.71	-	6.172	6.038	5.820	5.480	5.138	4.036
1	1.00	-	6.166	6.034	5.810	5.460	4.880	3.984
2	1.41	-	6.160	6.028	5.798	5.446	4.814	3.916
4	2.00	-	6.156	6.020	5.788	5.428	4.800	3.824
8	2.83	-	6.152	6.016	5.778	5.408	4.762	3.706
15	3.87	-	6.150	6.010	5.770	5.388	4.656	3.588
30	5.48	-	6.148	6.006	5.762	5.370	4.612	3.472
60	7.75	-	6.134	6.002	5.752	5.354	4.564	3.384
120	10.95	-	6.134	5.996	5.748	5.338	4.480	3.322
240	15.49	-	6.134	5.996	5.742	5.324	4.376	3.280
480	21.91	-	6.134	5.996	5.734	5.312	4.232	3.248
1440	37.95	6.216	6.134	5.996	5.726	5.292	4.200	3.212

Cumulative Dial Gauge Reading at the End of Each Consecutive Unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	7 [kPa]
3.212	3.420	3.710	4.120	3.212	3.420	3.710

Applied Pressure P [kPa]	Final Dial Reading [mm]	Change in Specimen Height [mm]	Final Specimen Height [mm]	Void Height,H_V	Void Ratio,E
Loading					
7	6.000	0.00	20.00	11.33	1.31
7	6.216	0.22	20.22	11.55	1.33
50	6.134	0.13	20.13	11.47	1.32
100	5.996	0.00	20.00	11.33	1.31
200	5.726	-0.27	19.73	11.06	1.28
400	5.292	-0.71	19.29	10.62	1.23
800	4.200	-1.80	18.20	9.53	1.10
1600	3.212	-2.79	17.21	8.54	0.99
Unloading					
1600	3.212	-2.79	17.21	8.54	0.99
400	3.420	-2.58	17.42	8.75	1.01
100	3.710	-2.29	17.71	9.04	1.04
7	4.120	-1.88	18.12	9.45	1.09

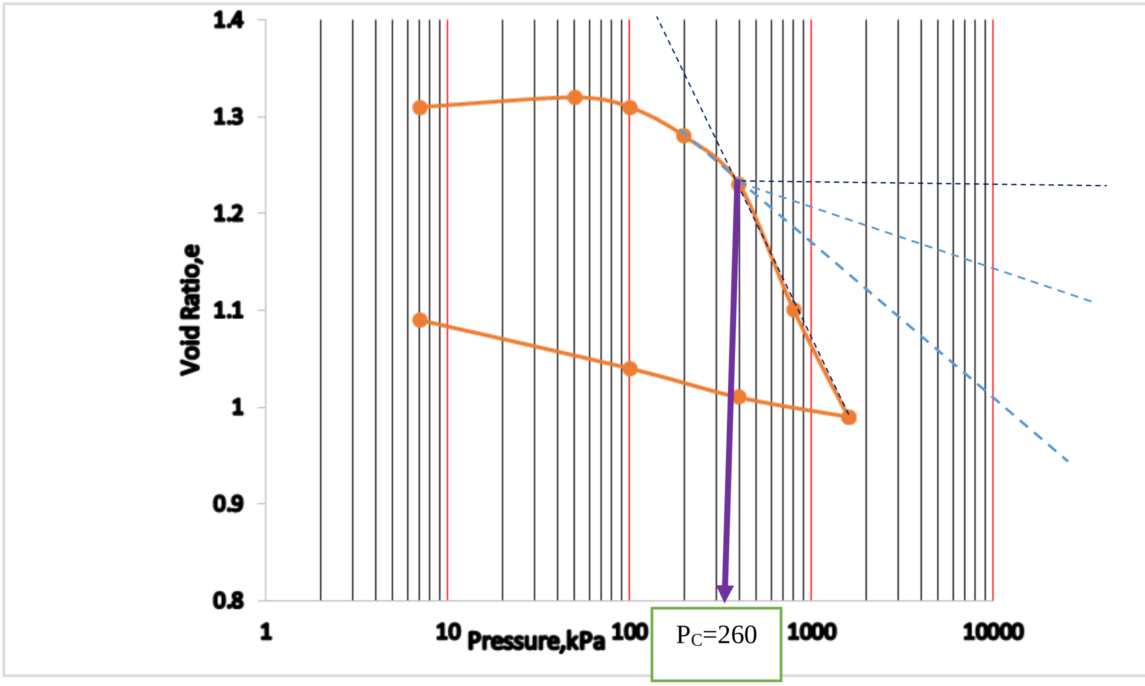


Figure E-4: Void ratio Vs log p curve for TP08 to determine Pc

Appendix – F

Soil Classification and Plasticity Charts

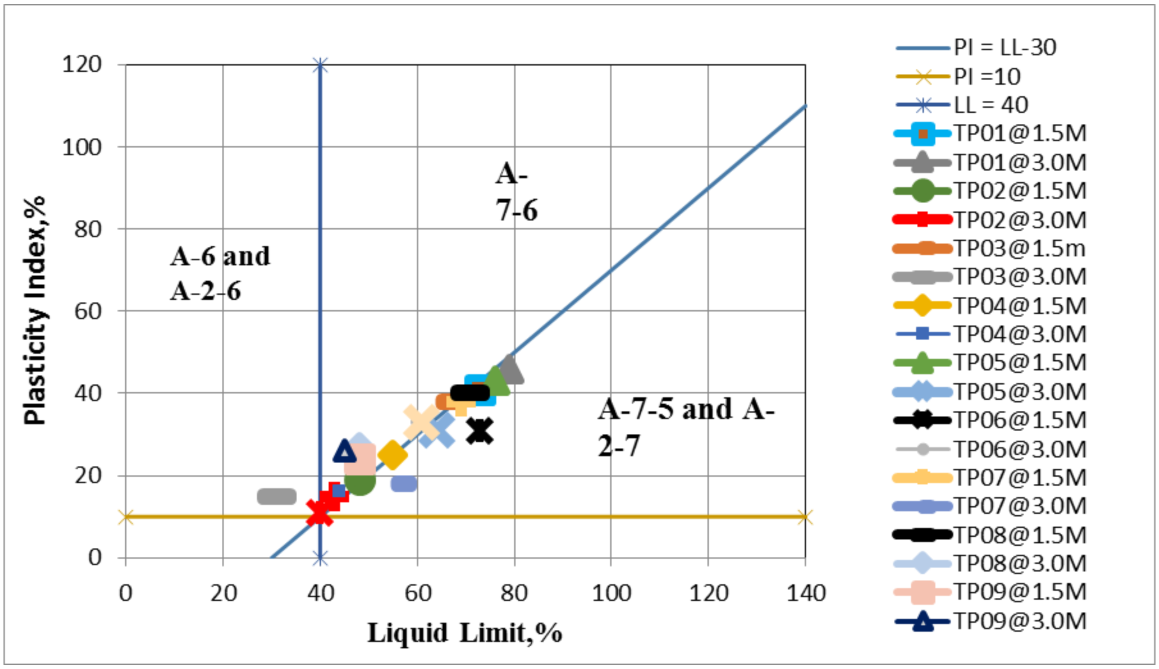


Figure F-1: AASHTO soil classification chart

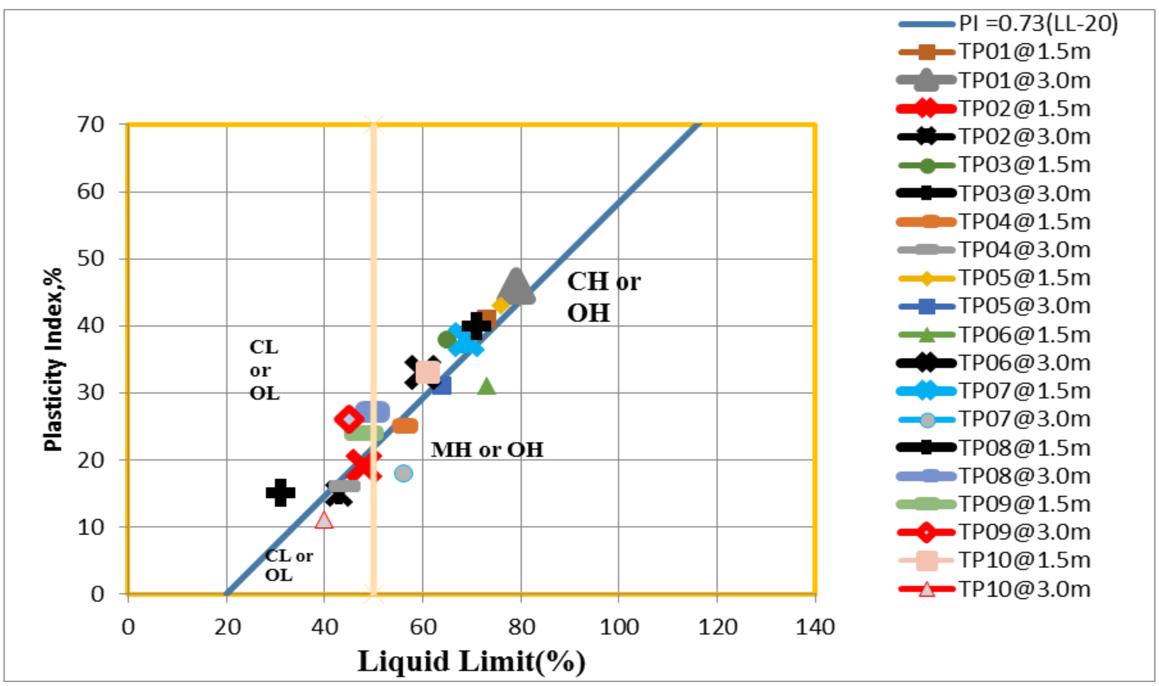


Figure F-2: USC soil classification chart