Jimma University

Jimma Institute of Technology

School of Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Investigation into Some of the Engineering Properties of Soil: A Case Study in Seka town, Jimma Zone

MSc. Thesis

By:

Sifilet Nigussie Wakjira

January, 2020

Jimma, Oromia, Ethiopia

Jimma University

Jimma Institute of Technology

School of Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Stream

Investigation into Some of the Engineering Properties of Soil: A Case Study in Seka town, Jimma Zone

A Thesis submitted to the School of Graduate Studies of Jimma University in Partial Fulfilment of the Requirements for the Master's Science Degree of Civil Engineering in Geotechnical Engineering

By:

Sifilet Nigussie Wakjira

Main advisors: Prof. Emer T. Quezon, P. Eng

Co-advisors: Ing. Alemineh Sorsa (PHD candidate)

January, 2020

Jimma, Oromia, Ethiopia

Declaration

I, the undersigned, declare that this thesis entitled: "Investigation into Some of the Engineering Properties of Soil: A Case Study in Seka town, Jimma Zone" 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 this thesis have to be duly acknowledged.

Candidate:

Mr. Sifilet Nigussie Wakjira

____________ 25/12/2019

Signature Date

As Master's Research Advisors, I hereby certify that I have read and evaluated this MSc Thesis prepared under my guidance by **Sifilet** entitled: "Investigation into Some of the Engineering Properties of Soil: A Case Study in Seka town, Jimma Zone"

Prof. Emer T. Quezon, P. Eng ______________ ______________

Main Advisor Signature Date

Ing. Alemineh Sorsa (PHD Candidate) ____________ ____________

Co- Advisor Signature Date

Abstract

*Proper understanding of Engineering properties of soils is almost important in any location where all structures are to be founded. Many damages to structures founded on soils are mainly due to the lack of proper investigation of substructure condition. The main aim of this study is to investigate some of the engineering properties of soils found in Seka town in order to know the nature of the soil and also to give information for the design, construction and environmental assessment, so that suitable foundation can be recommended for better design and construction in the town. Laboratory tests were carried out including specific gravity, natural moisture content, Atterberg limits, unconfined compressive strength, consolidation, compaction and permeability tests. Based on the results of this study, the grain size distribution indicates all soil samples have more than 90% fine grained material. Therefore, clayey silty type of soil is dominantly located in the study area. The specific gravity of the soil ranges from 2.65 to 2.77. While the Atterberg limit tests results, the soil is highly plastic clay and highly plastic Silt. The values of Liquidity Index classify the soil under the class of Intermediate strength, which the soil deform like a plastic material. The compaction test result shows that maximum dry density (MDD) ranges from 1.180 g/cm3 to 1.480 g/cm3, and the optimum moisture content ranges 35.9% to 48.00% include both methods of compaction. In addition, the unconfined compressive strength of the Seka soils ranges from 143.52kN/m^2-352.92kN/m^2 and undrained shear strength range from 71.76 kN/m^2–176.46 kN/m^2. Likewise, consistencies of the soils ranges from stiff to very stiff. One-dimensional consolidation tests were done, of which the result showed that the soils have compression index 0.23 and 0.39, swelling index 0.19 and 0.02. The coefficient of permeability (3.75*10^{-5} to 2.75*10^{-4} cm/sec) indicating that the soil investigated is impermeable.*

Keywords; Investigation, soil, engineering properties

Acknowledgement

First of all, I would like to thank the Almighty God for giving me strength and sustain me to complete this thesis.

My deepest gratitude goes to my advisor, Professor Emer T. Quezon and my co-advisor Alemineh Sorsa for all their limitless efforts in guiding me through my work and for providing me in all directions.

I would like to extend special thanks to the Ethiopian Road Authority for giving me sponsorship of further study in Geotechnical engineering, Jimma University, Jimma Institute of Technology also deserves special thanks for accepting and nurturing me as a postgraduate student.

I would like to acknowledge the encouragement given to me by my families, especially my brother Tolesa Nigussie, for his supporting me in all directions through my work. Without him this work is impossible.

My heartfelt gratitude goes to my best friend Dhekebi, and my classmate and best friends Chala and Dinka for their limitless efforts during Laboratory test. And at last but, not least my thank goes to my friends Miss Asnakech Asefa for her support me by ordered my work.

Table of Contents

List of tables

List of Figures

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for testing and Materials
ERA	Ethiopia Road Authority
GI	Group Index
LL/WL	Liquid limit
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OBP	Over Burden Pressure
OMC	Optimum moisture content
OCR	Over Consolidation Ratio
PI/IP	Plasticity Index
PL/PL	Plastic limit
USCS	Unified Soil Classification System
UCST	Unconfined Compression Strength Test
w	Water Content
ρ	Density
γ	Unit weight
γw	Unit weight of water
γd	Dry unit weight
γs	Solid unit weight
σ'o	Effective over burden Pressure
σ'v	Executive Vertical Pressure
ε	Axial Strain
ϕ	Angle of Internal Friction

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

According to many researchers' idea Geotechnical investigation is an essential requirement to the design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, high ways, etc. requires adequate knowledge of sub surface conditions at the sites of the structures. Many damages to buildings, roads and other structures founded on soils are mainly due to the lack of proper investigation of substructure condition. Investigation of the sub-surface conditions at a site is prerequisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies of the proposed project. Public building officials may require soil data together with the recommendations of the geotechnical consultant prior to issuing a building permit, particularly if there is a chance that the project will endanger the public health or safety or degrade the environment [23].

Seka is one of the near Jimma towns in South-western Ethiopia, Oromia National Regional State. It is predominantly covered with red, black and gray soils. The reddish brown to red colours soils are located on higher elevations and good drained condition. In contrast, the gray to reddish brown colours soils are found in the part of the town having flat topography and unfavourable drainage condition. Tropical residual soils such as lateritic soils can have characteristics that are quite distinctly different from those of transported soils. Particles of residual soils often consist of aggregates or crystals of weathered mineral matter that breakdown and become progressively finer if the soil is further manipulated. Depending on soil forming factors such as climate, drainage, topography and parent rocks some red soils of the study area can be lateritic soils. Damage due to soil swelling is very noticeable in ordinary and light weight structures such as buildings, roads, retaining walls and canal and landfill liners. Ethiopia is one of the countries with extensive coverage of weak soil. Therefore, it is important to make localized study for the different regions.

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

This study insight a better understanding of some engineering behavior of the soil in the Town. Identifying the soil characteristic is essential to construct economically different types of civil engineering structures that will serve to the people for various purposes. The results of the study will be of great importance for the ever-growing building construction especially for those yet to be constructed in that area. It can be used as a soil property manual as it will have a customized nature to meet the required soil information of the area with regard to the future development programs in the construction sector.

In this research to achieve the objectives, applying all the requirements procedural starting from literature review, sample collection, conducting relevant laboratory tests and analysis of results obtained from input data is done. Finally, comparison of the results with already available specification and then formulate a recommendation to who it concerns is carried out.

1.2 Statement of the Problem

The construction of civil engineering structures is developing fast in, since the Seka town developing towns in Ethiopia. Furthermore, Seka is known for its production of Arabica Coffee which is the backbone of the country's export economy. Several governmental institutions and private business center are established in the town and because of the Road connect Addis, Jimma, Bonga and Mizan across the town. So, the need for detail geotechnical investigation of the sub-surface condition of these soils has a paramount importance for the safe and economical design and construction activities.

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

1.3 Objectives of the Study

1.3.1 General Objectives of the study

The main objective of this study is to investigate into some of the engineering properties of soils found in Seka town

1.3.2 Specific Objectives

To Investigate the index properties of the soil of study area

To classify the soils according to the standard.

To determine the consolidation and compression characteristic of soils of the area.

To investigate the shear strength of the soils in the Seka town.

1.4 Research question

What are the index properties of the soil?

What are the classes of the soil of the study area?

How can the consolidation and compression characteristic of soils be determined?

What is the shear strength of the Seka soils?

1.5 Scope of the Study

The study covers the investigation of soils' nature, type and classification found in the study area based on the conventional identification procedures and classification schemes. It also covers the exploration to determine different strength characteristics of these soils. To achieve this, from eight tests pits both disturbed and undisturbed samples were collected by stratification from the town sections on representative locations for useful comparison of the differences in the test results.

1.6 Significance of the Study

The research study shall be investigating some of engineering properties of Soils found in Seka Town. In this town there are many expansion projects both building and road construction, beside this there are many problems can be foreseen on the construction which are insufficient geotechnical investigations. So insufficient geotechnical investigations faulty interpretation of results or failure to portray results in a clearly understandable manner may contribute to inappropriate designs; delays in construction schedules, costly construction modifications, and use of substandard material, environmental damage to the site and even failure of a structure. Therefore, to obtain information on type, characteristics and distributions of a soil, geotechnical investigations should be done on soil.

CHAPTER TWO

REVIEW OF RELATED LITERATURE

2.1 Introduction

Engineering properties of soils are investigated by direct methods such as borings and trial pits or through indirect methods such as seismic acoustic, resistivity and ground penetrating radar. Usually it is impossible to define all subsoil characteristics through field investigations and laboratory testing. Site investigation is an important part of civil engineering design whose aim is to reduce uncertainty of ground conditions by various combinations of field and laboratory testing. However, the scope of site investigations is usually dependent on the finances available and time required for carrying out the investigation [8].

The term "soil" can have different meanings, depending upon the field in which it is considered. To a geologist, it is the material in the relative thin zone of the Earth's surface within which roots occur, and which are formed as the products of past surface processes. The rest of the crust is grouped under the term "rock". To a pedologist, it is the substance existing on the surface, which supports plant life. To an engineer, it is a material that can be: Built on: foundations of buildings, bridges; built in: basements, culverts, tunnels; built with: embankments, roads, dams, supported: retaining walls [4].

2.2 Methods of Determining Soil Properties

Geotechnical soil and rock properties of geologic strata are typically determined using one or more of the following methods: In-situ testing data from the field exploration program; Laboratory testing; and back analysis based on site performance data. Laboratory soil testing is used to estimate strength, stress/strain, compressibility, and permeability characteristics. [4]

2.3 Number and Depths of Boreholes

It is practically impossible and economically infeasible to completely explore the whole project site. You have to make judgments on the number, location, and depths of borings to provide sufficient information for design and construction. The number and depths of borings should cover the zone of soil that would be affected by the structural loads. There is no fixed rule to follow. In most cases, the number and depths of borings are governed by experience based on the geological character of the ground, the importance of the structure, the structural loads, and the availability of equipment. Building codes and regulatory bodies provide guidelines on the minimum number and depths of borings. The number of boreholes should be adequate to detect variations of the soils at the site. If the locations of the loads on the footprint of the structure are known (this is often not the case), you should consider drilling at least one borehole at the location of the heaviest load. As a guide, a minimum of three boreholes should be drilled for a building area of about 250 m2 (2500 ft2) and about five for a building area of about 1000 m2 (10,000 ft2). Some general guidance on the depth of boreholes is provided in the following: In compressible soils such as clays, the borings should penetrate to at least between 1 and 3 times the width of the proposed foundation below the depth of embedment or until the stress increment due to the heaviest foundation load is less than 10%, whichever is greater, In very stiff clays and dense, coarse-grained soils, borings should penetrate 5 m to 6 m to prove that the thickness of the stratum is adequate, Borings should penetrate at least 3 m into rock, Borings must penetrate below any fills or very soft deposits below the proposed structure, The minimum depth of boreholes should be 6 m unless bedrock or very dense material is encountered [5].

2.4. Index Properties

The principal soil grain properties are the size and shape of grains and the mineralogical character of the finer fractions (applied to clay soils). The most significant aggregate property of cohesionless soils is the relative density, whereas that of cohesive soils is the consistency. Water content can also be studied as an aggregate property as applied to cohesive soils. The strength and compressibility characteristics of cohesive soils are functions of water content. As such water content is an important factor in understanding the aggregate behavior of cohesive soils. By contrast, water content does not alter the properties of a cohesionless soil significantly except when the mass is submerged, in which case only its unit weight is reduced [6].

The various properties of soils which would be considered as index properties are: 1. the size and shape of particles. 2. The relative density or consistency of soil. The index properties of soils can be studied in a general way under two classes. They are: 1. Soil grain properties, 2. Soil aggregate properties. The principal soil grain properties are the size and shape of grains and the mineralogical character of the finer fractions (applied to clay soils). The most significant aggregate property of cohesion-less soils is the relative density, whereas that of cohesive soils is the consistency. Water content can also be studied as an aggregate property as applied to cohesive soils [6].

2.5 Shear Strength of Soils

The safety of any geotechnical structure is dependent on the strength of the soil. If the soil fails, a structure founded on it can collapse, endangering lives and causing economic damage. The strength of soils is therefore of paramount importance to geotechnical engineers. The word strength is used loosely to mean shear strength, which is internal frictional resistance of a soil to shearing forces. Shear strength is required to make estimate of the load bearing capacity of soils, the stability of geotechnical structure, and in analysing the stress-strain of the characteristic of soils [5].

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 [8].

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 behaviour and development of pore pressures during shear can be obtained [8].

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 [27].

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 Passion’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 is 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 behaviour is well represented by the Mohr-Coulomb failure criterion given as,

$$S = C + \sigma \tan\phi \tag{2.6}$$

Where: S= shear strength, σ=normal stress on the 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 [23].

Table 2.1 Relation between Consistency and unconfined strength of clay soil [1].

Consistency	**Qu(kN/m^2)**
Very Soft	<25
Soft	25-50
Medium	50-100
Stiff	100-200
Very Stiff	200-400
Hard	>400

2.6 Unconfined compressive strength

The unconfined compression test is a special case of the unconsolidated undrained triaxial test. In this case no confining pressure to the specimen is applied. For such conditions, for saturated clays, the pore water pressure in the specimen at the beginning of the test is negative (Capillary pressure). Axial stress on the specimen is gradually increased until the specimen fails [17].

2.7 Soil Mass Structure

The orientation of particles in a mass depends on the size and shape of the grains as well as upon the minerals of which the grains are formed. The structure of soils that is formed by natural deposition can be altered by external forces. The following gives the various types of structures of soil. (a) A single grained structure which is formed by the settlement of coarse grained soils in suspension in water. (b) A flocculent structure formed by the deposition of the fine soil fraction in water. (c) A honeycomb structure which is formed by the disintegration of a flocculent structure under a superimposed load. The particles oriented in a flocculent structure will have edge-to-face contact as shown in (d) whereas in a honeycomb structure, the particles will have face-to-face contact as shown in (e). Natural clay sediments will have more or less flocculated particle orientations. Marine clays generally have a more open structure than fresh water clays. (f) And (g) show the schematic views of salt water and fresh water deposits [6].

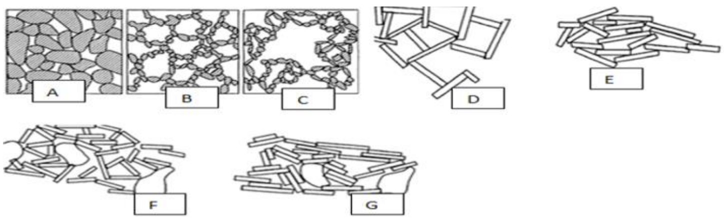


Figure 2.1 Schematic diagrams of various types of structures

2.8 Soil compaction

The compaction of soil is defined as the process of packing soil particles closely together by mechanical manipulation, thus increasing the dry density of soil [17].

2.8.1 Theory of compaction and factors influencing compacted density

The various factors influencing compaction are: water content, amount of compaction, type of soil, method of compaction and admixtures [9].

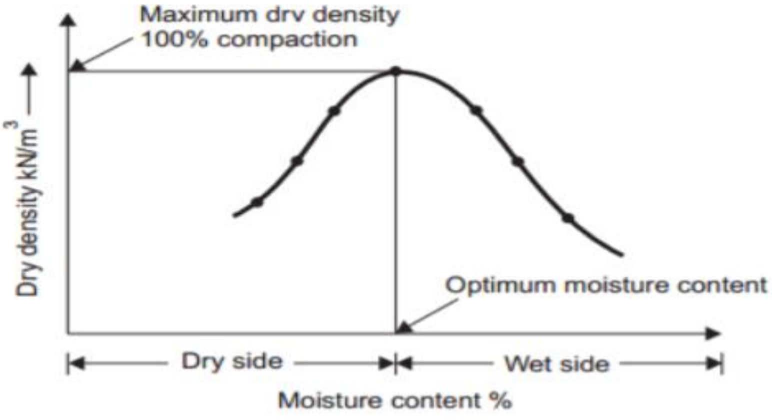


Figure 2-2: Relationship between dry density with moisture content [9]

2.9 Consolidation Characteristics

The amount of settlement induced by the placement of load bearing elements on the ground surface or the construction of earthen embankments will affect the performance of the structure. The amount of settlement is a function of the increase in pore water pressure caused by the loading and the reduction of this pressure over time. The reduction in pore pressure and the rate of the reduction are a function of the permeability of the in-situ soil. All soils undergo elastic compression and primary and secondary consolidation [4].

When a saturated clay water system is subjected to an external pressure, the applied is initially taken by the water in the pores resulting there by an excess pore water pressure. With the advance of time, a portion of applied pressure is transferred to the soil skeleton, which in turn, causes a reduction in the pore water pressure. This process involving a gradual compression occurring simultaneously with a flow of water out of the mass; and with gradual transfer of the applied pressure from the pore water to mineral skeleton [19].

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids or extrusion of the pore air and/or water [17], this process is called consolidation.

2.9.1 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 [28]:

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.

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.

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.

2.9.2. Factors Affecting the Consolidation Characteristics of Clay Soils

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

2.9.2.1 Stress History

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 [29].

2. Overconsolidated.

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 [29].

A soil is called Overconsolidated if the present effective overburden pressure is less than the maximum to which the soil was ever subjected in the past, i.e., present < past maximum. In the natural condition in the field, a soil may be either normally consolidated or Overconsolidated. A soil in the field may become Overconsolidated through several mechanisms, some of which are listed below [23]

Removal of overburden pressure, past structures, Glaciation, Deep pumping, Desiccation due to drying, Desiccation due to plant lift, Change in soil structure due to secondary compression, Change in PH, Change in temperature, Salt concentration, Weathering, Ion exchange, Precipitation of cementing agents.

The preconsolidation pressure from an e versus log plot is generally determined by a graphical procedure suggested by Casagrande (1936), as shown in Figure 2.2. The steps are as follows:

1. Visually determine the point P (on the upper curved portion of the e versus log plot) that has the maximum curvature.
2. Draw a horizontal line PQ.
3. Draw a tangent PR at P.
4. Draw the line PS bisecting the angle QPR.
5. Produce the straight-line portion of the e versus log plot backward to intersect PS at T.
6. The effective pressure corresponding to point T is the preconsolidation pressure c.

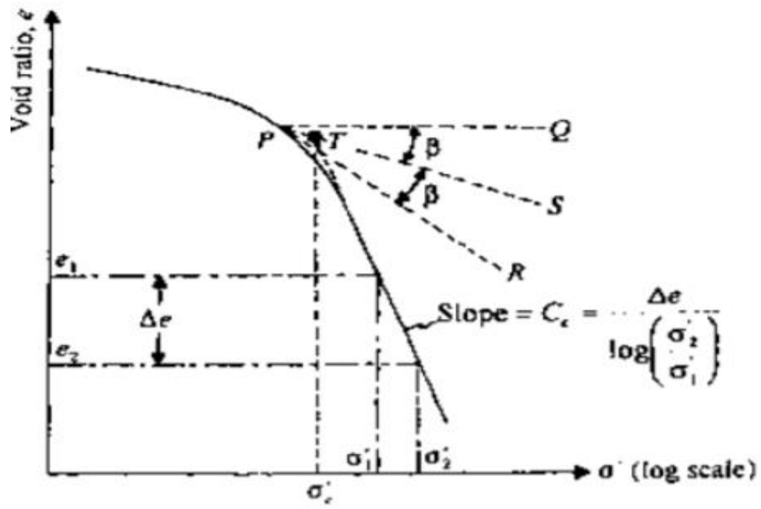


Figure 2.3 Typical e versus log plot showing procedure for determination of Cc and Cs [17]

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 [29]. As to the stress history, the insitu soil can be grouped in to two categories:

2.9.2.2 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). [29]

2.9.3 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] [14].

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 micrometre 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 to be 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, 30,60 minute.......24 hours. 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, Cv, 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].

2.9.3.1. Compression index

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

Thus

$$Cc=\frac{\Delta e}{\log(\frac{Po+\Delta P}{Po})} \quad (2.1)$$

Where: Δe-change in void ratio, Δp -change pressure, Po –initial pressure

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

2.9.3.2. 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

$$Cv=\frac{(1+e).k}{av.\gamma w}=\frac{k}{mv.\gamma w} \quad (2.2)$$

Where: e - void ratio, k – permeability, a_v –coefficient of compressibility, m_v –coefficient of permeability, γ_w – unit weight of water

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

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

$$Cv=\frac{(0.196).H2}{t50} \quad (2.3)$$

Where: H^2 = length of drainage path, t_{50} = time at 50% dual reading

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

$$Cv=\frac{(0.848).H2}{t90} \quad (2.4)$$

Where: H^2 = length of drainage path, t_{90} = time at 90% dual reading

2.9.3.3. 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 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:

$$OCR=\frac{Pc}{Po} \qquad (2.5)$$

Where, P_C = Pre-consolidation, P_O = consolidation

2.10 Soil Classification

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

These groups ideally provide estimates of both the engineering characteristics and performance of soils for design and construction. The descriptions of soils within the groups of a given classification typically are represented by alphabetical or alphanumeric symbols for rapid identification in written material, graphic boring logs, and on engineering drawings. The continued use of a few engineering soil classifications is the result of the provision in each for the needs of the Civil Engineer as well as the adaptability of the classification to the variety of soils encountered in engineering practice. Soils classification can be done in to two main ways. First, Visual classification of soils (field classification method) - during excavation and sampling operations in the field classification has to be carried out quickly and without gradation analyses or Atterberg limits. Second, laboratory classification of soils- this classification system is used after gradation analyses or Atterberg limit test is done in the laboratory. At the present time, two major soil classification systems are available for general engineering use. They are the unified soil classification system (USCS), and the American association of state highway and transportation official (AASHTO) system. Both systems use simple soil properties such as grain-size distribution, liquid limit, and plasticity index of soil.

Similarly, it is possible to use other classification systems depending on the type, size and texture of the soils. Such systems are International Classification System, Massachusetts Institute of Technology (MIT) classification system and Textural Classification System [1]. The soil identified in the field is done by conducting the following simple test. The sample is first spread on a flat surface. If more than 50% of the particle are visible to the naked eye (unaided eye), the soil is coarse-grained; otherwise fine-grained soils. The fine grained particles are smaller than 0.075mm size and are not visible to unaided eye. The fraction of the soil smaller than 0.075mm size, that is the clay and the silt fraction, is referred to as fines [1].

For the fine grained soils, the following tests shall be conducted. These are diletancy (reaction to shaking) test, toughness test and dry strength test as well as consistency test.
A classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. Soil classification is used to specify a certain soil type that is best suitable for a given application. There are several classification schemes available. Each was devised for a specific use. For example, the American Association of State Highway and Transportation Officials (AASHTO) developed one scheme that classifies soils according to their usefulness in roads and highways while the Unified Soil Classification System (USCS) was originally developed for use in airfield construction but was later modified for general use [5].

2.10.1 Unified Soil Classification System (USCS)

The USCS uses symbols for the particle size groups. This symbol sand their representations are: G- gravel, S- sand, M-silt, C-clay. These are combined with other symbols expressing gradation Characteristics-W for well graded and P for poorly graded- and plasticity Characteristics-H for high and L for low, and a symbol. O, indicating the presence of organic- material. A typical classification of CL means a clay soil with low plasticity; while SP means poorly graded sand [5].

Laboratory determination of liquid limit and plasticity indexes for a soil sample permits assignment of fine-grained soils (including the fine fraction of coarse-grained soils) to the proper group by use of the plasticity chart, or A-line diagram, as illustrated by Figure 2.4.

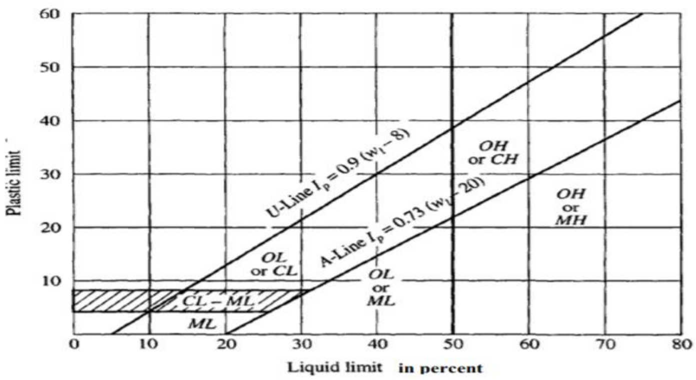


Figure 2. 4 Plasticity chart for classification of fine-grained soils [9].

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

2.10.2 AASHTO Soil Classification System

The AASHTO soil classification system is used to determine the suitability of soils for earthworks, embankments, and road bed materials (sub-grade/natural material below a constructed pavement; sub-base a layer of soil above the sub-grade; and base a layer of soil above the sub-base that offers high stability to distribute loads). According to AASHTO, granular soils are soils in which 35% or less are finer than the No. 200 sieve (0.075 mm). Silt-clay soils are soils in which more than 35% are finer than the No. 200 sieve [26].

Table 2.2 Soil types, average grain size, and description according to AASHTO [26]

No	Item	Size
1	Gravel	75 mm to 2mm (No. 10 sieves)
2	Sand	2 mm (# 10 sieves) to 0.0075 mm (#200 sieves)
3	Silt and Clay	<0.0075 mm (#200 sieves)
		Silty: PI <10 %
		Clayey: PI>11%

The AASHTO system classifies soils into seven major groups, A-1 through A-7. The first three groups, A-1 through A-3, are granular (coarse-grained) soils, while the last four groups, A-4 through A-7 are silt-clay (fine-grained) soils. A group index (GI) value is appended in parentheses to the main group to provide a measure of quality of a soil as highway sub-grade material. The group index is given as [26]

$$\mathbf{GI = (F-35)\,[0.2+0.005(LL-40)] + 0.01(F-15)\,(PI-10)}\ \textbf{[26]} \qquad 2.1$$

where ***F*** is percent passing No. 200 sieve and the other terms have been defined before. The GI index is reported to the nearest whole number (2.4 reported as 2; 2.5 reported as 3), and if GI, 0, it is set to 0. GI for groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 are zero. For groups A-2-6 and A-2-7, the partial group index equation

$$\mathbf{GI = 0.01(F-15)\,(PI-10)}\ \textbf{[26]} \qquad 2.2$$

is used. The higher the group index, the lower the quality of the soil as a sub-grade material. The GI should not exceed 20 for any of groups A-4 through A-7.

CHAPTER THREE

MATERIAL AND RESEARCH METHODOLOGY

3.1 Introduction

Testing of soil samples in the laboratory plays an important role in soil mechanics research and in Civil engineering practice. Some soils tests were only classifying soils into broad groups so that some aspects of a soil's behaviour would be known before more detailed tests are carried out [24].

This chapter deals briefly the material used for the research and the methodology perform for this research.

3.2 Study Area Description

This study considered Seka town which is found in south western Ethiopia, Oromia Region, Jimma Zone, Seka Cokorsa Woreda, and It is about 354 Km from Addis Ababa. It has latitude and longitude of 7^{0}40N and 36^{0}50 E respectively, and also its average elevation 1715 m-1835m above sea level. The topography of this region is predominantly flat. This study will be done on the expansive soil that had been collected under the surface of the earth in Seka town. It is possible to find vehicles for shipment of the collected samples. The town traverses vast flat land that is covered with red clay soil. The red clay soil is underlain by natural sand deposit. It was estimated that the sand deposit is found at depths of 1.0m – 3.0m from the surface of the natural ground (Wakjira, 2019)

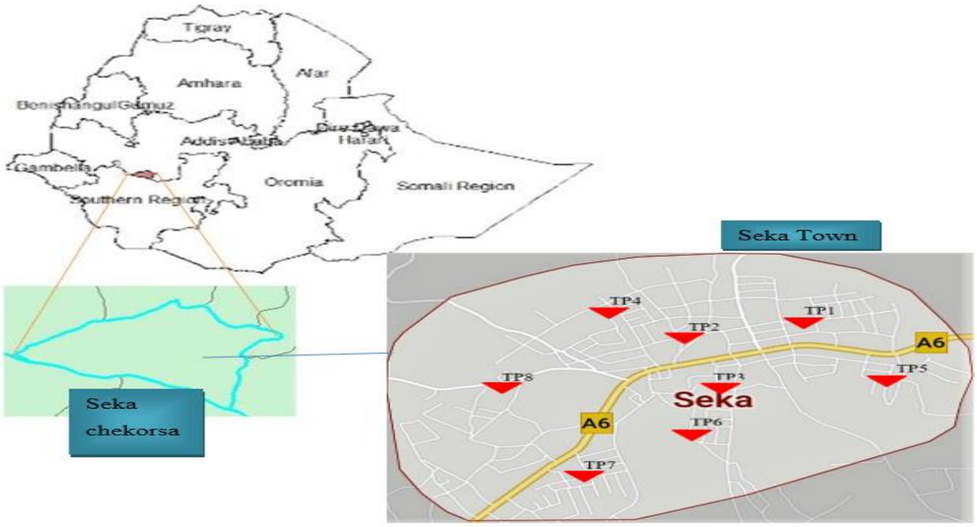


Figure 3.1 Geographic location of study area (Google map, 2019)

3.3 Climate

The climatic classification of Seka Town is classified as "Badda" with a mean temperature of 23^{o} C. It has mean annual rainfall of 1500 mm with maximum rainfalls from June to September. The area has a maximum temperature of 29^{o} C and a minimum temperature of 17^{o} C. The main geologic formation of Jimma town is the Cenozoic tertiary volcanic rock of Nazareth Series and Jimma Volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the Trap series formation. Tuft and Alluvial are found in few amounts at different localities [19].

Tropical Residual fine grained soils, like clays silts, developed mainly on basaltic bedrock represent the soils found in Jimma town. These soils are of two main types. The first type is red clay soil the colour of which is the result of reduction of magnetic minerals. In flat lying location, massive dark silty clay soils (alluvial origin) formation have colour ranging from gray to dark black. Even though most of the town is covered with soils coloured from dark to gray clay soils, there are also red and yellow coloured soils [8].

3.4. Methodology

To investigate the engineering properties of Seka Town, eight sampling areas were selected following to reconnaissance survey of the area, which done by visiting the entire part of the town. Necessary information about the geology, climatic condition and topography of the site are collected and analysed. The location of test pits is selected by stratification (by persona judgement and convenience); so that, it can well represent the soil types (visually) found in the town. Disturbed and undisturbed samples were collected in the field and transported for laboratory testing. Undisturbed samples are used for one dimensional consolidation, unconfined compression test, natural moisture content tests. Disturbed samples are used to conduct index property tests such as specific gravity, Atterberg limit, grain size analysis, compaction and free swell. Using Microsoft Office Excel and Word, grain size distribution curve, liquid limit graph, compaction curve, consolidation and unconfined compression tests are plotted ASTM procedures are followed for all 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 (wet method)
  - ✓ Hydrometer

- compaction test
- ✓ standard
- ✓ compaction
- Unconfined Compression Test
- One-dimensional consolidation test
- Permeability test

All the above tests were done according to American Society for Testing Materials (ASTM) standard.

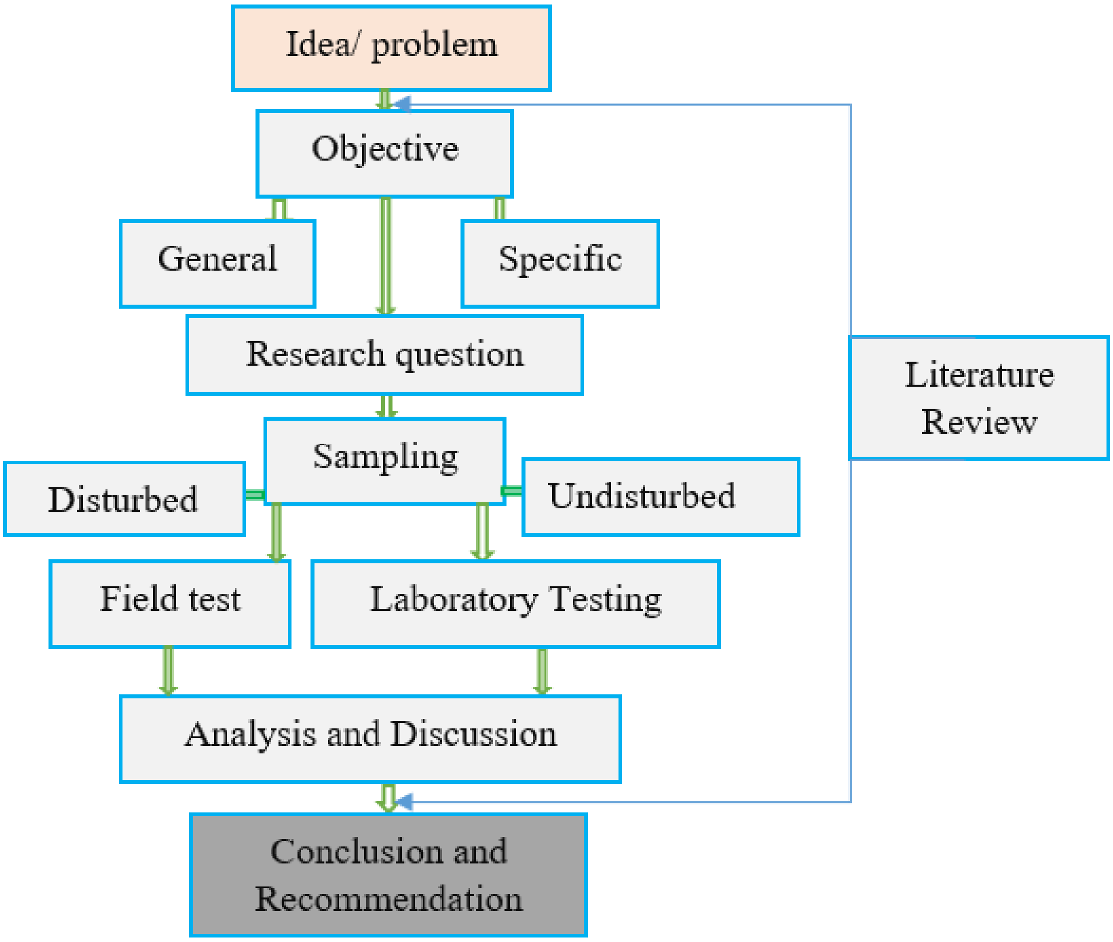


Figure 3.2 Thesis Flow Chart

3.5 Population

3.5.1 Sample size and selection

Study samples include soil which was obtained from various places of Seka town by stratification for good representative of sample in the town.

3.5.2 Sample techniques and procedures

To achieve this thesis, from eight test pits both disturbed and undisturbed samples were collected by stratification for representative locations and for useful comparison of the differences in the test results.

3.6. Study Variables

Dependent Variable: The engineering properties of soils.

Independent Variable:

- Natural Moisture Content
- Specific gravity,
- Grain size analysis
- Atterberg limit
- Compaction
- Shear strength of soil and
- Consolidation.

3.7. In-situ Properties

3.7.1. Identification of soil in the study area

The soil samples for this thesis work are collected from Seka town. Before selecting sampling areas, visual site investigation was made. The soil has the same colour in different places but the topography is varied. Accordingly, eight sampling areas were selected from various locations of the town depending on the topography. Pits were excavated to the maximum depth of three meters. Both disturbed and undisturbed soil samples were collected for this work and taken to the laboratory for testing. Each test listed on objectives was done for all samples taken except for one dimensional consolidation test is done only for two samples at a depth of three meters for TP-2 and TP-4.

3.7.2. In-situ properties Description

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

3.7.2.1. Natural moisture content

For most soils, the water content may be an 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. Since it was difficult to bring undisturbed samples to the laboratory, this test was done by using apparatus like moisture can, balance, core sampler and oven dry. The weight of the moisture can and the weight of can with moist soil was measured. Then the sample put it in to drying oven at a temperature of 105+5°c for 24 hours. Then after, the natural moisture content was determined. The water content of a soil is an important parameter that controls its behaviour. It is quantitative measure of the wetness of a soil mass. The water content of a soil can be determined to a high degree of precision, as it involves only mass which can be determined more accurately than volumes.

3.8. 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 were conducted and the properties of soils are obtained directly and indirectly.

3.9. Data Processing and Analysis

3.9.1 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].

3.10 Index Properties of Soils

3.10.1. General

In nature, soils occur in a large variety. However, soils exhibit similar behaviour can be grouped together to from a particular group. Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail, and precision required with engineering properties tests. These simplified tests provide indirect information about the engineering properties of soils and are, therefore, called index tests. The tests required for the determination engineering properties are elaborate and time consuming. This is possible if index properties are determined.

3.10.2. Specific Gravity Test

The specific gravity of solid particles without void space is called the true or absolute or real specific gravity and is usually denoted by a letter Gs. In this test a known weight of oven-dried soil sample is carefully put in a pycnometer which is then half filled with distilled water. The air entrapped in the soil sample is removed by heating or by means of vacuum pump. The bottle is then topped up with distilled water up to a calibration mark and brought up to a constant temperature.

3.10.3. Grain size analysis

3.10.3.1. General

Particle size analysis is done in two stages: (i) Sieve Analysis, (ii) Hydrometer Analysis. The normal method adopted for separation of particles in a fine grained soil mass is the hydrometer analysis, here sodium hexameta phosphate is used as a dispersing agent and for the coarse grained soils the sieve analysis. Soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods are performed. In this study wet sample preparation in accordance with ASTM D 2217-85 was applied. The test method covers the quantitative determination of the distribution of particle sizes in soils. The data are presented on a semi-log plot of percent finer vs. particle diameters and combined with the data from a sieve analysis of the soil sample retained on the No.200 sieve. The principal value of the hydrometer analysis appears to be to obtain the silt and clay fraction. The combined grain size distribution curve for particles.

3.10.4. Atterberg Limits

Atterberg limits or consistency limits are water contents at which the soil changes from one state to the other. Soil consistency is a term used to describe the degree of firmness of soil and is expressed by such terms as soft, firm or hard. It usually applies to fine grained soils whose condition is affected by changes in moisture content. Consistency limits are very important index properties of fine grained soils. As the consistency of soil changes, its engineering properties also change. Such soil properties as shearing strength and bearing capacity vary significantly with consistency. The Swedish scientist, Atterberg, established the four states of soil consistency Figure below, which are called the liquid, the plastic, the semi-solid, and the solid states. A soil containing high water content is in a liquid state. It offers no shearing resistance and can flow like liquids. As the water content is reduced, the soil becomes stiffer and starts developing resistance to shear deformation. At some particular water content, the soil becomes plastic.

3.10.4.1. Liquid limit

Liquid limit of a soil is generally determined by the Standard Casagrande device. This device consists of a brass cup and a hard rubber base. The brass cup can be dropped onto the base by a cam operated by a crank. To perform the liquid limit test, one must place a soil paste in the cup. By using the crank-operated cam, the cup is lifted and dropped from a height of 10 mm. The moisture content, in percent, required to close after 25 blows is defined as the liquid limit.

3.10.4.2. Plastic limit

The plastic limit is defined as the moist content, in percent, at which the soil crumbles when rolled into threads of 3.2 mm diameter. The plastic limit is the lower limit of the plastic stage of soil. The plastic limit test is simple and is performed by repeated rolling of an ellipsoidal size soil mass by hand on a ground glass plate. The procedure for the plastic limit test is given by ASTM Test Designation D-4318

3.10.4.3. Plasticity index

The range of water content over which the soil remains in the plastic state. It is equal to the difference between Liquid limit and plastic limit. When either liquid limit or plastic limit cannot be determined the soil is non plastic. When the plastic limit greater than liquid limit, the plasticity index is reported as zero not negative.

3.10.4.3. Liquidity index

The relative consistency of a cohesive soil can be defined by a ratio called the liquidity index LI. It is defined as

$$LI = \frac{WN - PL}{PI} \tag{3.1}$$

where W_N is the natural moisture content. It can be seen from Eq. (1.22) that, if $W_N = LL$, then the liquidity index is equal to 1. Again, if $W_N = PL$, the liquidity index is equal to 0. Thus, for a natural soil deposit which is in a plastic state (i.e., $LL \geq W_N \geq PL$), the value of the liquidity index varies between 1 and 0. A natural deposit with $W_N \geq LL$ will have a liquidity index greater than 1. In an undisturbed state, these soils may be stable; however, a sudden shock may transform them into a liquid state. Such soils are called sensitive clays.

Table 3.1 Description of the Strength of Fine-Grained Soils Based on Liquidity Index [5].

Values of LI	Description of soil strength
LI< 0	Semisolid state—high strength, brittle, (sudden) fracture is expected
0 <LI < 1	Plastic state—intermediate strength, soil deforms like a plastic material
LI>1	Liquid state—low strength, soil deforms like a viscous fluid

3.10.4.4. Activity

Since the plastic property of soil is due to the adsorbed water that surrounds the clay particles, we can expect that the type of clay minerals and their proportional amounts in a soil will affect the liquid and plastic limits. Skempton (1953) observed that the plasticity index of a soil linearly increases with the percent of clay-size fraction (percent finer than 2by weight) present in it. The average lines for all the soils pass through the origin. The correlations of PI with the clay-size fractions for different clays plot separate lines. This is due to the type of clay minerals in each soil. On the basis of these results, Skempton defined a quantity called activity that is the slope of the line correlating Plasticity Index, PI and percent finer than 2. This activity A may be expressed as

$$A = \frac{\text{PI}}{\text{percentage of clay fraction}} \quad [17] \qquad 3.2$$

Table 3.2. Degree of Colloidal activity [1].

Activity	Degree of Activity
<0.75	Inactive clay
0.75-1.25	Normal clay
>1.25	Active clay

3.11. Consolidation Test

3.11.1. General

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

3.11.2. Test Procedures and Methods

A one dimensional Consolidation test is conducted using conventional Oedometer apparatus. The type of standard consolidation test used in this case is a controlled stress test (CST) in which a constant load increment is used for each stage of the test. ASTM D2435 is employed to conduct the test. The results are plotted as void ratio e vs. log p [5]. The test is performed on an undisturbed soil sample that is placed in a consolidation ring available in diameters ranging from 45 to 115 mm. The sample height is between 20 and 30 mm; 20 mm is the most commonly used thickness to reduce test time. The larger diameter samples give better parameters, since the amount of disturbance (recovery, trimming, insertion into the test ring, etc.) is less for the larger samples.

The consolidation test proceeds by applying a series of load increments (usually in the ratio of $\Delta p/p=1$ in a pressure range from about 25 to either 1600 kPa) to the sample and recording sample deformation by using either an electronic displacement device or a dial gauge at selected time intervals [8].

For significant comparison of preconsolidation pressure against overburden pressure, the test was conducted on the samples from 3m depth. The other reason is, from design and construction practice, most of the foundation of ordinary building structures placed at this depth. The general outline of Terzaghi-Froehlich's theory of consolidation is employed for the double drainage condition of consolidation test [8]. Furthermore, to calculate the initial void ratio specific gravity (Gs), initial moisture content (NMC), bulk unit weight (γbulk) and dry unit weight (γd) are used.

3.12. 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 Seka 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.

3.13. 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

4.1. Laboratory Test Results and Observations

4.1.1. Field Observations

Soils of Seka town was identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include: The colour of the soil is identified (i.e. black or gray, reddish, brown). The dominant types of soils properties in the test pits are reddish brown in the upper 1.5m and reddish brown to red silty clay below 1.5m. It has high dry strength and low wet strength when the soil is touched with hands. 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

Table 4.1 Summary of natural moisture content

Sr.No.	Test pit	Depth(m)	Moisture content, %	Color of samples
1	TP-01	1.5	48.37	Red
2		3.0	45.40	Red
3	TP-02	1.5	46.91	Red
4		3.0	43.64	Red
5	TP-03	1.5	46.27	Red
6		3.0	53.51	Red
7	TP-04	1.5	46.78	Red
8		3.0	50.51	Red
9	TP-05	1.5	46.09	Gray
10		3.0	43.54	Gray
11	TP-06	1.5	46.17	Red
12		3.0	43.00	Red
13	TP-07	1.5	50.79	Red
14		3.0	47.84	Red
15	TP-08	1.5	52.61	Red Brown
16		3.0	49.35	Red Brown

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. Then, the natural moisture content was determined as table 4.1 above and Refer Appendix-I.

4.1.3 Index Properties

4.1.3.1 Specific gravity

Table 4.2 Summary of specific gravity test result

Sr.No.	Test pit	Depth (m)	Specific gravity		Sr.No.	Test pit	Depth (m)	Specific gravity
1	TP-01	1.5	2.67		9	TP-05	1.5	2.73
2		3.0	2.66		10		3.0	2.62
3	TP-02	1.5	2.68		11	TP-06	1.5	2.65
4		3.0	2.75		12		3.0	2.75
5	TP-03	1.5	2.74		13	TP-07	1.5	2.65
6		3.0	2.77		14		3.0	2.77
7	TP-04	1.5	2.70		15	TP-08	1.5	2.65
8		3.0	2.68		16		3.0	2.68

The specific gravity of soils found in Seka town falls to 2.65-2.77 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 above.

4.1.3.2 Grain Size Analysis

Table 4.3 Percentage of grain size distribution

No.	Test Pit	Location	Depth (m)	Grain size Amount(%)				Percentage finer than #200 sieve
				Gravel	Sand	Silt	Clay	
1	TP-01	Waajjira Qonnaa	1.50	0.00	5.24	39.78	54.98	94.76
2			3.00	0.00	5.42	48.72	45.86	94.58
3	TP-02	Mannahariyaa	1.50	0.00	7.41	47.59	45.00	92.59
4			3.00	0.00	6.92	47.27	45.81	93.08
5	TP-03	Mana Qopheessaa	1.50	0.00	7.84	39.52	52.64	92.16
6			3.00	1.20	6.43	50.33	42.04	92.37
7	TP-04	Bulchiinsa	1.50	0.00	5.38	40.18	54.44	94.62
8			3.00	0.00	9.00	45.33	45.68	91.00
9	TP-05	Seka High school	1.50	0.00	8.56	40.75	50.69	91.44
10			3.00	0.00	8.92	37.56	53.52	91.08
11	TP-06	M/B dhaloota Haaraa	1.50	0.00	7.66	41.59	50.75	92.34
12			3.00	0.00	8.01	41.82	50.17	91.99
13	TP-07	Hospitaalaa	1.50	0.00	8.27	41.38	50.35	91.73
14			3.00	0.00	9.32	40.03	50.65	90.68
15	TP-08	M/K Lidataa	1.50	0.00	9.82	51.74	38.44	90.18
16			3.00	0.00	9.32	47.31	43.37	90.68

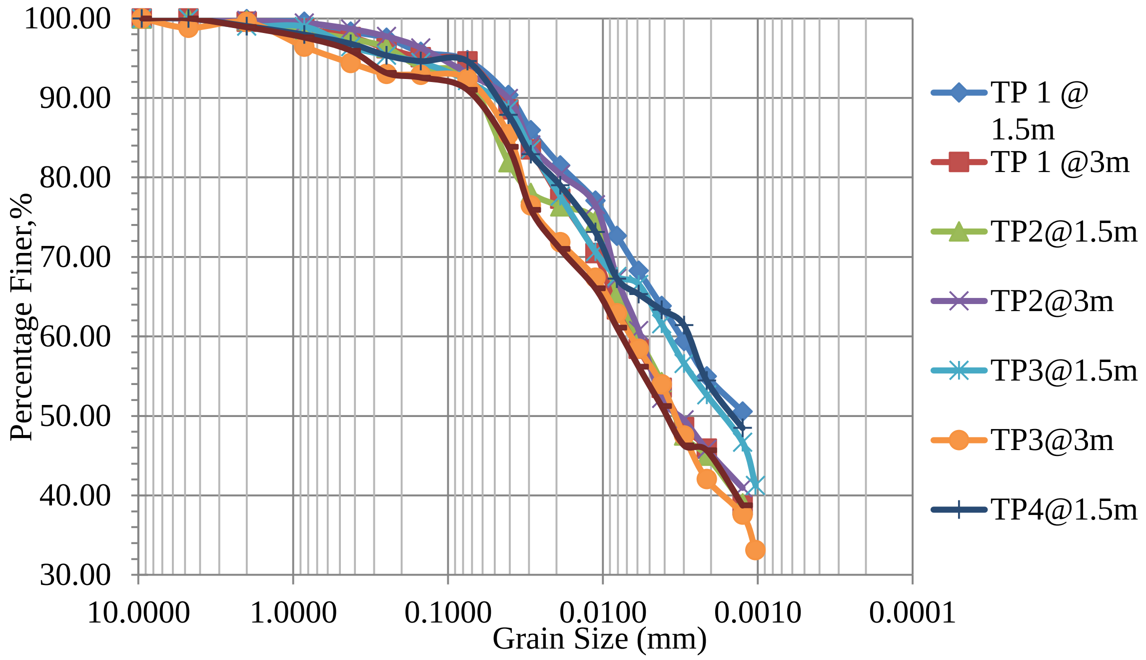


Figure 4.1 Grain size distribution curve of TP (1, 2, 3 and 4) @1.5m and 3m

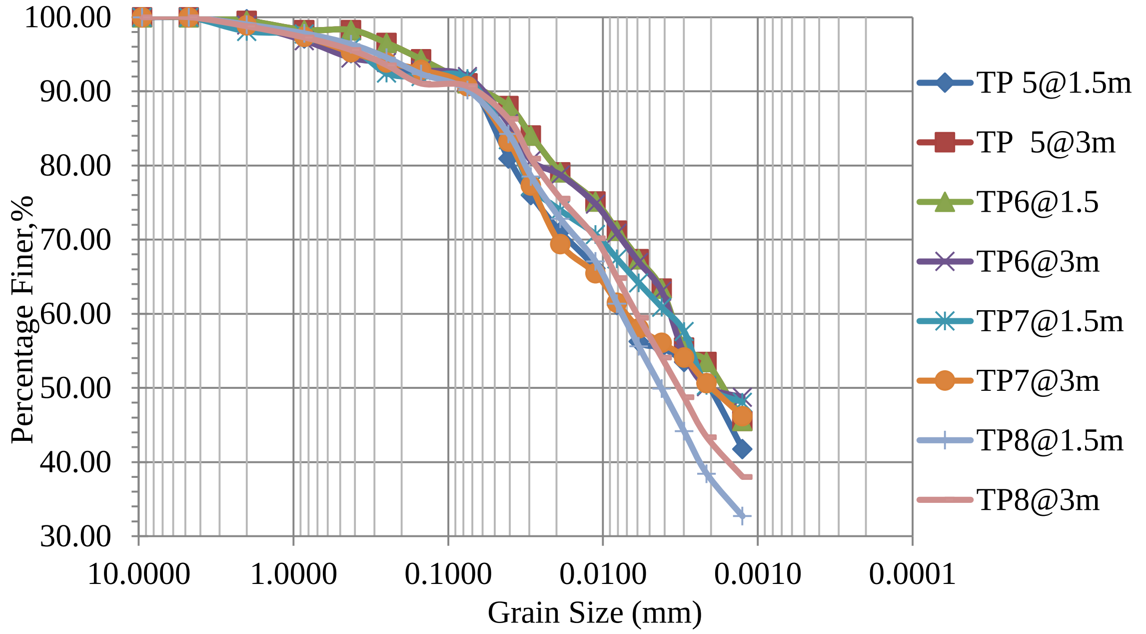


Figure 4.2 Grain size distribution curve of TP (5, 6, 7 and 8) @ 1.5m and 3m.

The soils contain 38.44-54.98% clay, 37.56-51.74% silt, 5.24-9.82% sand and 0% gravel. Summary of the test result and graph of combined analysis is shown above on (Table 4.3), (Fig 4.1) and (Fig 4.2) respectively and detail analysis is attached in Appendix-II

4.1.3.3 Atterberg limits

Table 4.4 Atterberg limit test results

No.	TP	D (m)	W,%	LL, %	PL, %	PI, %	Clay, %	LI	A
1	TP-01	1.50	48.37	74.9	27.69	47.21	39.78	0.44	0.5
2		3.00	45.40	70	24.67	45.33	48.72	0.46	0.54
3	TP-02	1.50	46.91	83.1	48.22	34.88	47.59	-0.04	1.07
4		3.00	43.64	76.9	41.16	35.74	47.27	0.07	0.9
5	TP-03	1.50	46.27	87.1	38.74	48.36	39.52	0.16	0.74
6		3.00	53.51	81.2	42.34	38.86	50.33	0.29	1.01
7	TP-04	1.50	46.78	90	40.96	49.04	40.18	0.12	0.75
8		3.00	50.51	80.5	47.17	33.33	45.33	0.10	1.03
9	TP-05	1.50	46.09	72.3	28.03	44.27	40.75	0.41	0.55
10		3.00	43.54	69.4	26.03	43.37	37.56	0.40	0.49
11	TP-06	1.50	46.17	78.1	42.83	35.27	41.59	0.09	0.84
12		3.00	43.00	74.8	37.21	37.59	41.82	0.15	0.74
13	TP-07	1.50	50.79	84	29.45	54.55	41.38	0.39	0.58
14		3.00	47.84	80.2	27.12	53.08	40.03	0.39	0.54
15	TP-08	1.50	52.61	79.52	48.23	31.29	51.74	0.14	1.25
16		3.00	49.35	75.4	49.47	25.93	47.31	-0.005	1.14

Liquid limit of Seka town falls in the range of 69.4-90% and plastic limit was in the range of 24.67-49.47% (Table 4.4). The plasticity index range of the soils was from 25.93% to 54.55% (Table 4.4). According to Burmister (1947) the plasticity of the soils are 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-II

4.1.4 Classification of Soils

4.1.4.1 Unified Soil Classification System (USCS)

The classification of soils according to USCS scheme shows most of the soil of the study area falls in MH, and CH region. From the plot of plasticity chart and the classification soils, the soils found in Seka town are Silty and Clayey higher plasticity.

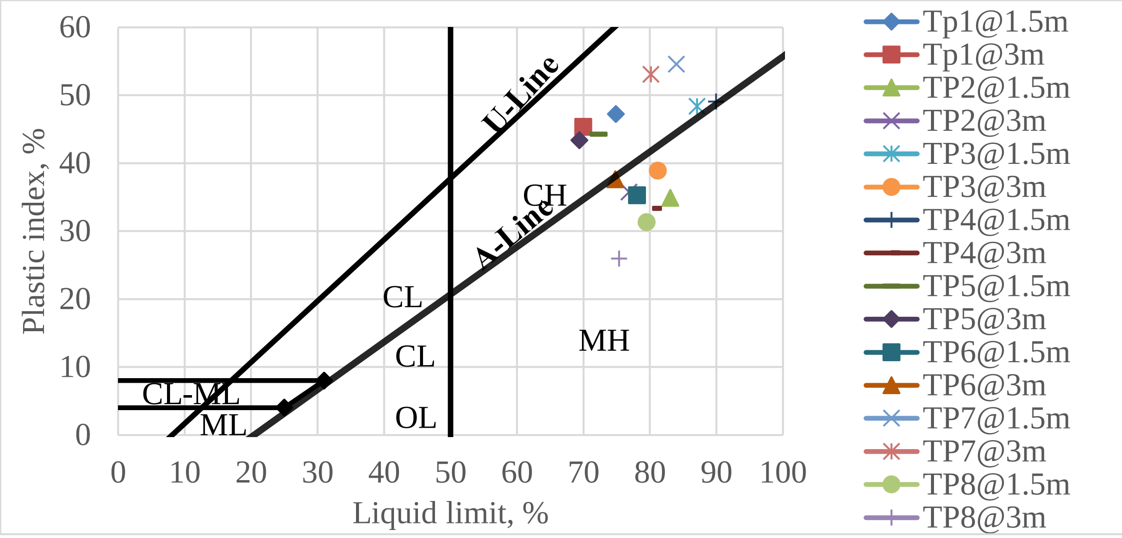


Figure 4. 3 Classification of the fine-grained Seka soils using USCS plasticity chart

Table 4.5 Soil Classification based on USCS

No	TP	D (m)	Grain size Amount(%)				Finer than #200 sieve(%)	LL,%	PI, %	USCS Classification
			Gravel	Sand	Silt	Clay				
1	TP -01	1.50	0.00	5.24	39.78	54.98	94.76	74.9	27.69	CH
2		3.00	0.00	5.42	48.72	45.86	94.58	70	24.67	MH
3	TP -02	1.50	0.00	7.41	47.59	45.00	92.59	83.1	48.22	MH
4		3.00	0.00	6.92	47.27	45.81	93.08	76.9	41.16	MH
5	TP -03	1.50	0.00	7.84	39.52	52.64	92.16	87.1	38.74	CH
6		3.00	1.20	6.43	50.33	42.04	92.37	81.2	42.34	MH
7	TP -04	1.50	0.00	5.38	40.18	54.44	94.62	90	40.96	CH
8		3.00	0.00	9.00	45.33	45.68	91.00	80.5	47.17	MH
9	TP -05	1.50	0.00	8.56	40.75	50.69	91.44	72.3	28.03	CH
10		3.00	0.00	8.92	37.56	53.52	91.08	69.4	26.03	CH
11	TP -06	1.50	0.00	7.66	41.59	50.75	92.34	78.1	42.83	CH
12		3.00	0.00	8.01	41.82	50.17	91.99	74.8	37.21	CH
13	TP -07	1.50	0.00	8.27	41.38	50.35	91.73	84	29.45	CH
14		3.00	0.00	9.32	40.03	50.65	90.68	80.2	27.12	CH
15	TP -08	1.50	0.00	9.82	51.74	38.44	90.18	79.52	48.23	MH
16		3.00	0.00	9.32	47.31	43.37	90.68	75.4	49.47	MH

4.1.4.2 AASHTO Soil Classification System

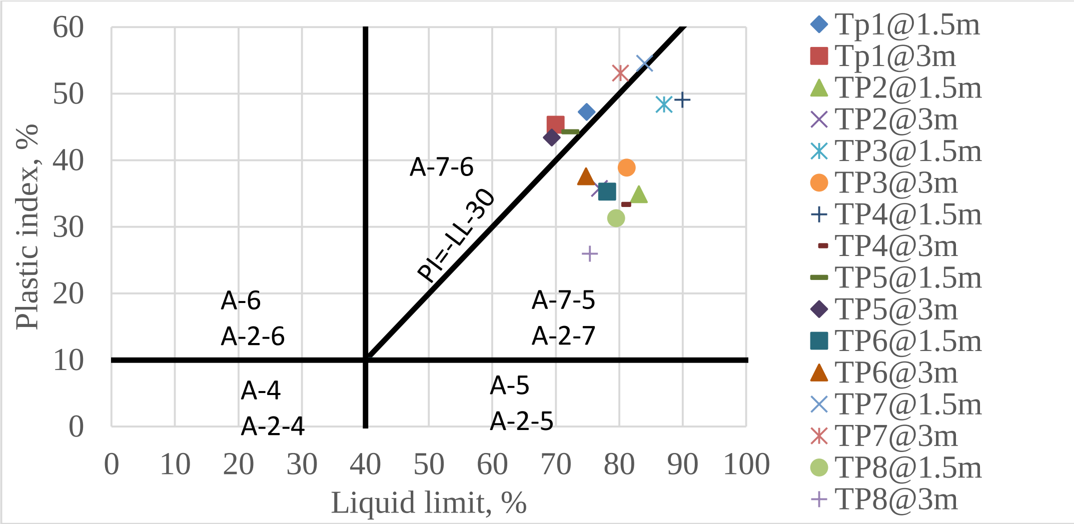


Figure 4. 4 AASHTO plasticity chart classification for the Seka fine-grained soils

Table 4.6 Soil Classification based on AASHTO

No	T P	D (m)	Grain size Amount(%)			LL, %	PI, %	GI	AASHTO Classification	Remark
			#10	#40	#200					
1	TP-01	1.50	99.88	98.34	94.76	74.9	40.21	52	A-7-6	Clayey Soil
2		3.00	99.56	97.65	94.58	70	37.33	49	A-7-6	Clayey Soil
3	TP-02	1.50	99.74	97.44	92.59	83.1	34.88	43	A-7-5	Clayey Soil
4		3.00	99.74	98.67	93.08	76.9	35.74	42	A-7-5	Clayey Soil
5	TP-03	1.50	99.02	96.45	92.16	87.1	48.36	54	A-7-5	Clayey Soil
6		3.00	99.52	94.38	92.37	81.2	38.86	46	A-7-5	Clayey Soil
7	TP-04	1.50	99.03	96.80	94.62	90	49.04	58	A-7-5	Clayey Soil
8		3.00	98.88	95.92	91.00	80.5	33.33	40	A-7-5	Clayey Soil
9	TP-05	1.50	99.71	95.16	91.44	72.3	36.27	47	A-7-6	Clayey Soil
10		3.00	99.51	98.26	91.08	69.4	35.37	45	A-7-6	Clayey Soil
11	TP-06	1.50	99.13	97.86	92.34	78.1	35.27	42	A-7-5	Clayey Soil
12		3.00	98.92	94.51	91.99	74.8	37.59	43	A-7-5	Clayey Soil
13	TP-07	1.50	98.10	95.93	91.73	84	44.55	58	A-7-6	Clayey Soil
14		3.00	98.90	95.28	90.68	80.2	47.08	55	A-7-6	Clayey Soil
15	TP-08	1.50	99.00	96.34	90.18	79.5	31.29	38	A-7-5	clayey Soil
16		3.00	98.75	95.56	90.68	75.4	25.93	33	A-7-5	clayey Soil

Soil classification of the study area based on AASHTO, all soils falls in A-7-5 and A-7-6. So the general rating for all soil is not fair for subgrade material depend on PI.

4.1.5 Compaction Test

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

No	TP	D (m)	Modified Compaction		Standard Compaction	
			OMC, %	MDD, gm/cm^3	OMC, %	MDD, gm/cm^3
1	TP-01	1.50	32.50	1.427	37.00	1.295
2		3.00	35.50	1.460	42.50	1.265
3	TP-02	1.50	29.70	1.452	36.50	1.360
4		3.00	30.90	1.480	40.00	1.315
5	TP-03	1.50	32.00	1.433	39.00	1.352
6		3.00	35.00	1.395	39.00	1.250
7	TP-04	1.50	30.10	1.445	41.00	1.298
8		3.00	31.00	1.445	41.00	1.265
9	TP-05	1.50	32.50	1.427	37.00	1.295
10		3.00	35.50	1.419	48.00	1.185
11	TP-06	1.50	31.80	1.469	36.50	1.280
12		3.00	30.90	1.450	35.90	1.275
13	TP-07	1.50	34.70	1.433	39.00	1.279
14		3.00	35.00	1.359	44.00	1.230
15	TP-08	1.50	33.30	1.418	41.00	1.266
16		3.00	32.00	1.445	45.50	1.245

From the test results the maximum dry density (MDD) of Seka town ranges from1.359 g/cm^3 to 1.480 g/cm^3 for modified compaction and from1.185 g/cm^3 to 1.36 g/cm^3, and the optimum moisture content ranges 29.7% to 35.50% for modified compaction and 35.9% to 48.00% for standard compaction. The summary of the test result is shown in (Table 4.8). Refer Appendix-III.

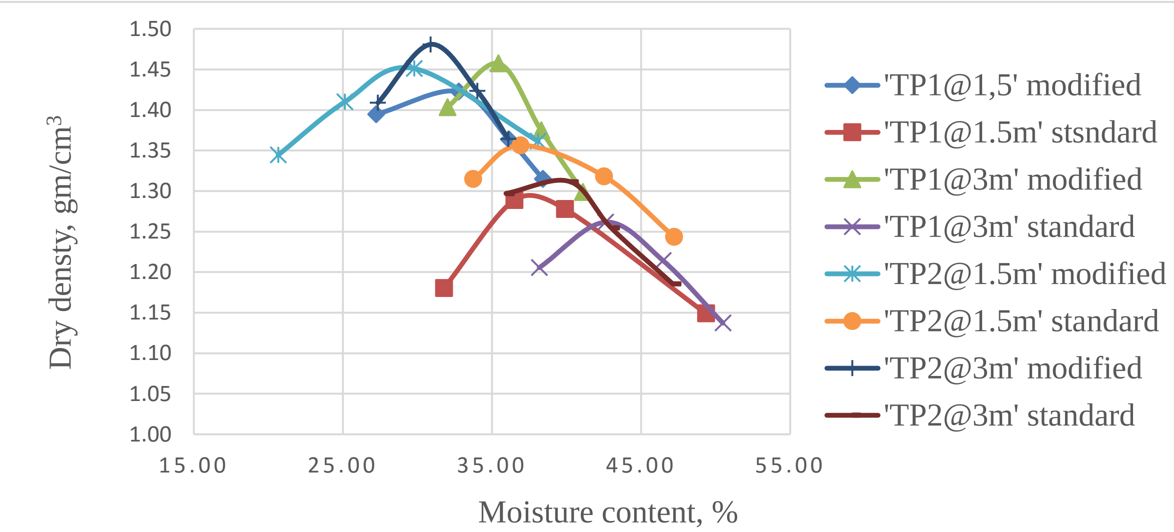


Figure 4.5. Dry Density versus Optimum Moisture Content curve TP1 and TP2

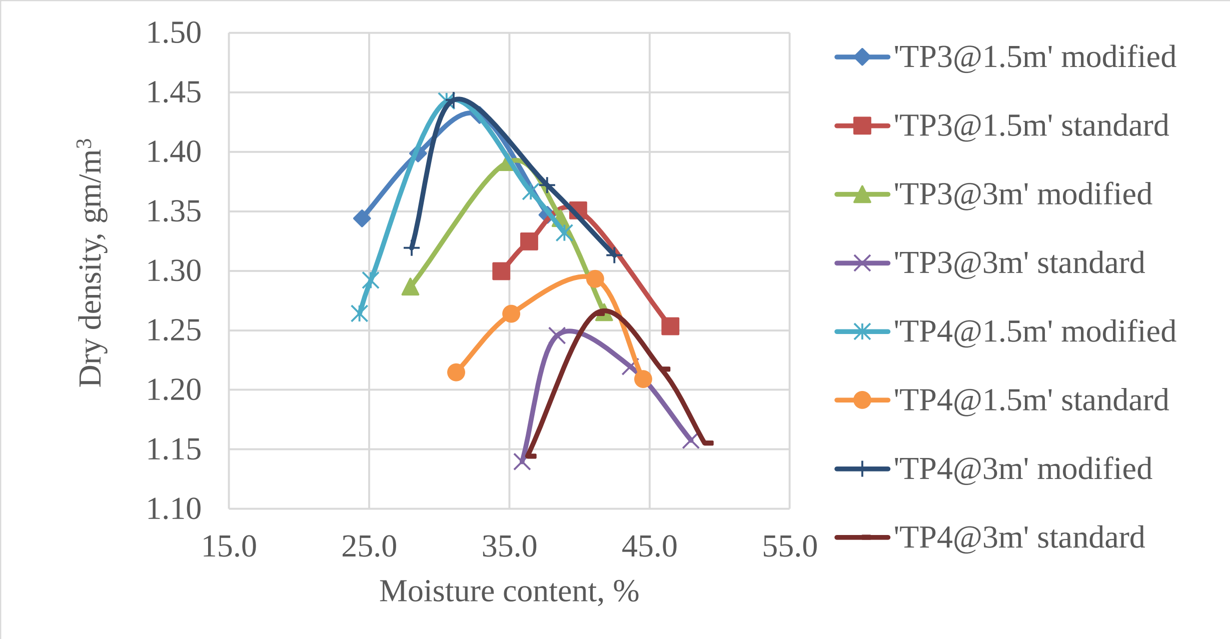


Figure 4.6. Dry Density versus Optimum Moisture Content Curve TP3 and TP4

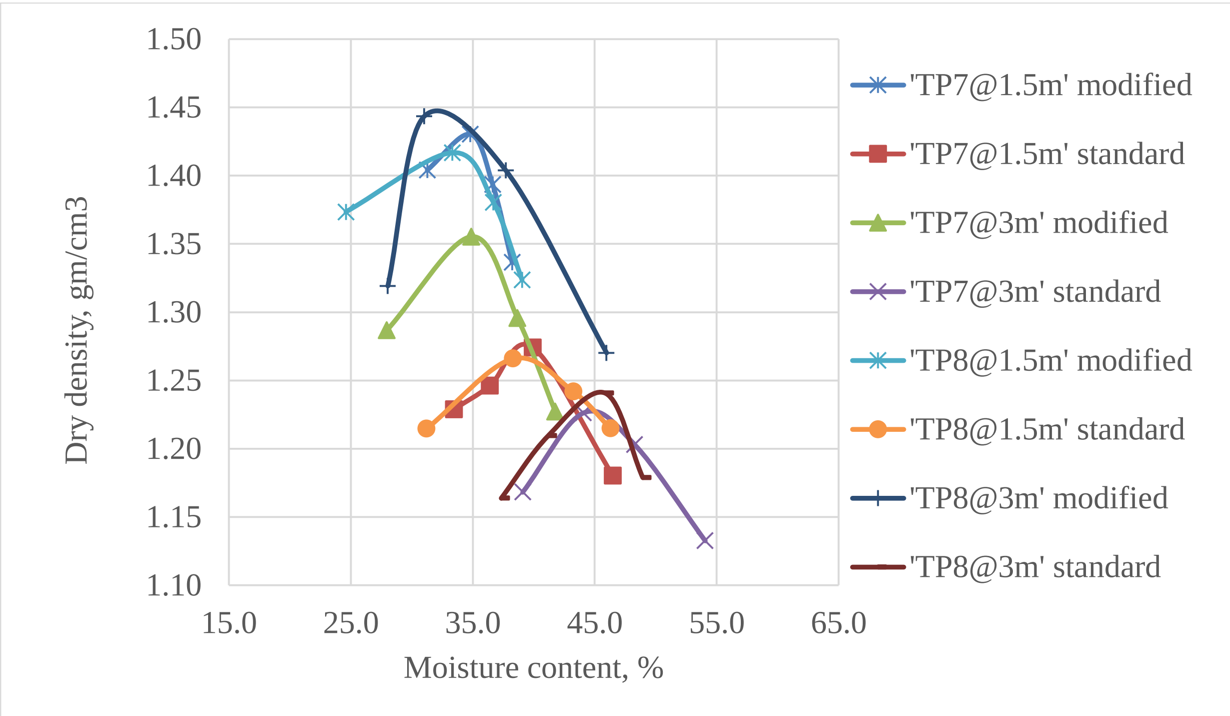


Figure 4.7. Dry Density versus Optimum Moisture Content Curve TP5 and TP6

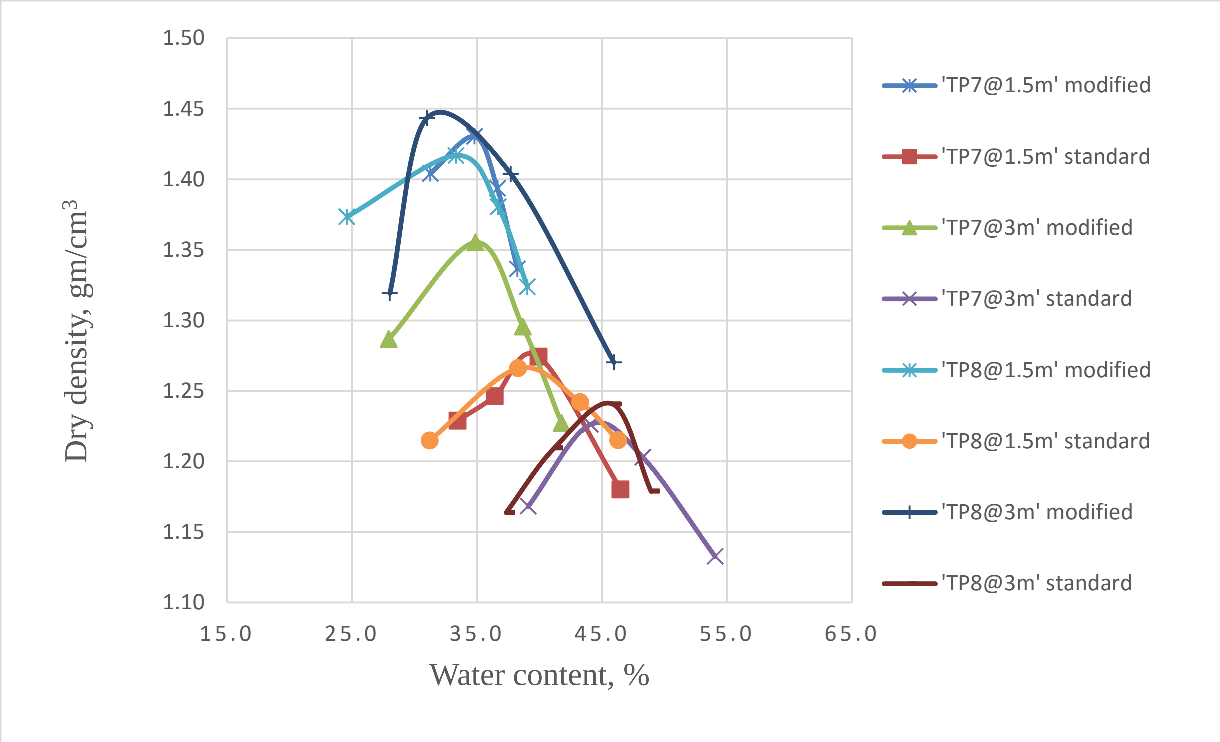


Figure 4.8. Dry Density versus Optimum Moisture Content Curve TP7 and TP8

4.1.6 Unconfined compression strength (UCS) test

Table 4.9. Summary of unconfined compressive strength and cohesion

No.	Test pit	Depth(m)	W, %	qu, (kN/m^2)	Su, (kN/m^2)	Consistency
1	TP-01	3.0	46.71	153.75	76.87	Stiff
2	TP-02	3.0	49.92	352.92	176.46	Very stiff
3	TP-03	1.5	42.02	181.67	90.84	Stiff
4		3.0	43.67	278.57	139.28	Very stiff
5	TP-04	3.0	40.83	256.5	128.25	Very stiff
6	TP-05	3.0	40.37	256.8	128.4	Very stiff
7	TP-06	1.5	40.45	143.52	71.76	Stiff
8		3.0	43.8	193.75	96.87	Stiff
9	TP-08	3.0	49.96	161.16	80.58	Stiff

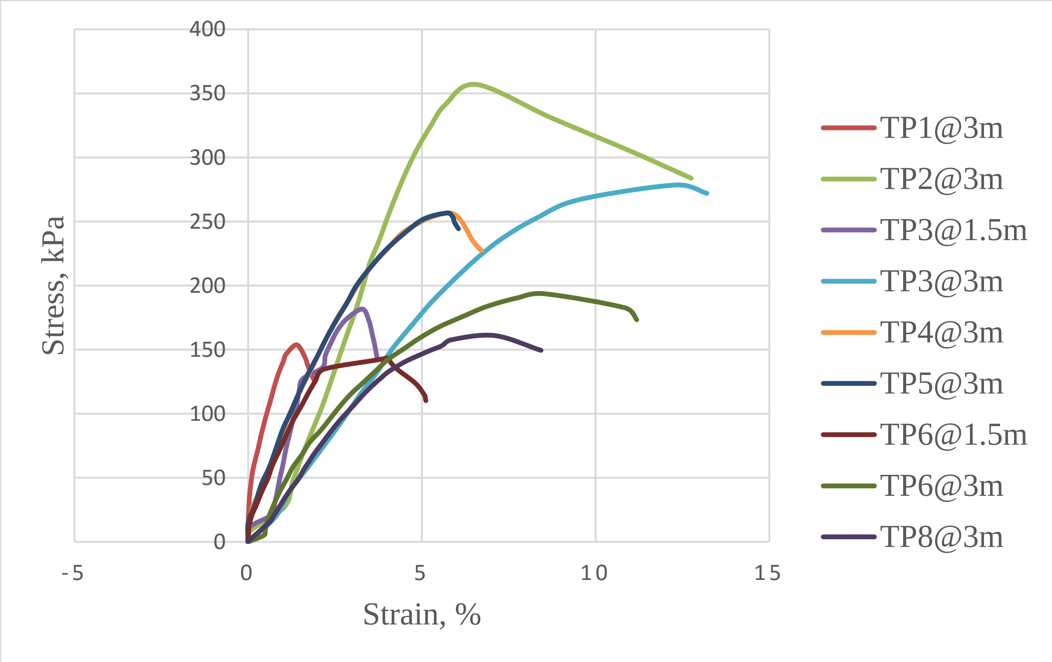


Figure 4.9. Unconfined compressive strength at 3m and some of 1.5m

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 Seka town area. It is observed that the consistency of Seka soil is either stiff or very stiff. Refer Appendix-IV.

4.1.7 Results of Consolidation Test

Pressure – void ratio curve

The pressure-void ratio curve can be obtained if the void ratio of the sample at the end of each increment of load is determined. The basic data used to determine this curve are natural moisture content, Specific gravity, density, cross sectional area and height of the sample, initial void ratio and applied loads. From these curve important parameters such as coefficient of compressibility (av), compression indexes (Cc), Swelling index (Cs) and preconsolidation pressure (pc) are determined. The summary of test results is presented in Table 4.10 and Figure 4.10 below.

Table 4. 10 Summary of applied pressure and void ratio for two samples of Seka soils

TP	TP2@3m	TP4@3m	TP	TP2@3m	TP4@3m
P (KPa)	e	e	P (KPa)	e	E
Loading			Unloading		
50	1.128	1.306	800	0.849	0.973
100	1.104	1.297	400	0.878	1.015
200	1.055	1.253	200	0.885	1.021
400	1.003	1.183	100	0.891	1.026
800	0.929	1.059	50	0.897	1.032
1600	0.841	0.899	800	0.849	0.973

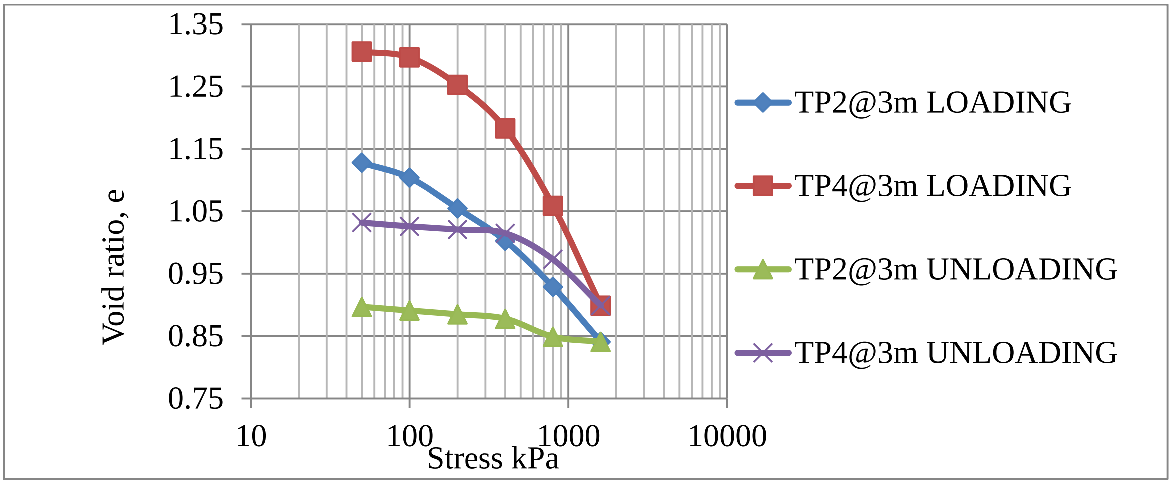


Figure 4.10 summarized void ratio and log pressure curve

Compression Parameters and Preconsolidation pressure

Compression parameters such as coefficient of compressibility (av) is obtained from a plot of void ratio versus pressure and compression index (Cc), swelling index (Cs) and preconsolidation pressure (Pc) are obtained from void ratio versus log pressure curve. There are a few graphical methods for determining the preconsolidation pressure based on laboratory test data. The earliest and the most widely used method was the one proposed by Casagrande [5]. The method involves locating the point of maximum curvature, on the laboratory elog p curve of an undisturbed sample, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the preconsolidation pressure Pc.

Coefficient of Consolidation

The coefficient of consolidation, cv 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: Casagrande Logarithm of Time Fitting Method and Taylor Square Root of Time Fitting Method. However, for the following reasons Taylor Square Root of Time Fitting Method is selected. First the time plot gives a straight line for the initial portion to indicate the point of corrected dial reading, second the square-root time curve is more suitable for soils exhibiting high secondary consolidation and third it is more convenient for a general case as it requires dial gauge readings covering a much smaller shorter period of time comparing with log-time method [6]. The coefficients of consolidation obtained from the test results are variable for different loadings. As a result, Schneider [21] proposed characteristics value which is determined as follows:

$$C_{v,\,ch} = C_{v,\,mean} - 0.5C_{v,\,sd} \quad (4.1)$$

Where: $C_{v,\,ch}$ = characteristic coefficient of consolidation, cv, mean= mean coefficient of consolidation $C_{v,\,sd}$ = standard deviation for coefficients of consolidation Coefficient of Permeability, a_v Average vertical coefficient of permeability, a_v can be estimated using the following formula [5].

$$a_v = C_{v,\,ch} \cdot m_v \cdot \gamma_w \quad (4.2)$$

For coefficient of consolidation and permeability the summary of the results is tabulated as in Table 4.12.

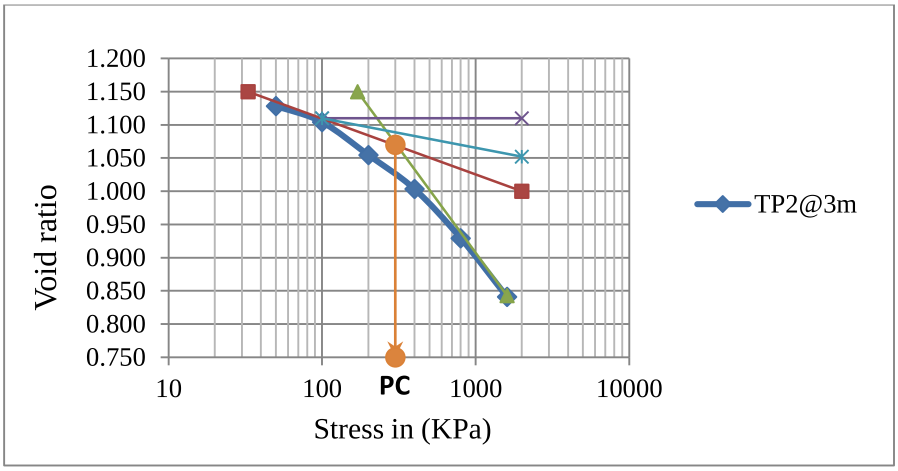


Figure 4.11 summarized void ratio and log pressure curve for TP2.

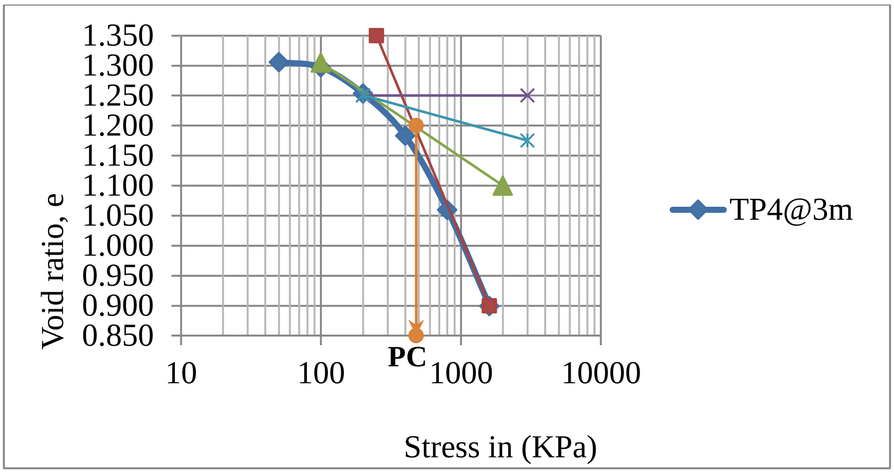


Figure 4.12 summarized void ratio and log pressure curve for TP4

Table 4. 11 Summary for Oedometer test results

No	Location	Test Pit	Natural moisture content	Initial void Ratio	Bulk Density	Compression Index, Cc	swelling Index, Cs	Coefficient of Compression, av	Pre Consolidation, Pc	Over Burden Ratio, OBR	Over Consolidation Ratio, OCR
1	Manna-haariyaa	TP2 @3m	41.3	1.16	17.43	0.23	0.02	1.88E-04	200	43.38	5.3
2	Bulchii-nsa	TP4 @3m	49.12	1.338	17. 21	0.39	0.19	1.56E-04	500	43.63	6.9

Table 4. 12 Characteristic coefficients of consolidation and coefficient of permeability

No	Location	Test pit	Initial void Ratio, e_o	Coefficient of volume change, mv (m^2/kN)	Coefficient of Compression, av (m^2/kN)	Coefficient of consolidation, Cv,ch (cm^2 /min)	Coefficient of permeability, kv, ch (cm/min)
1	Manna-haariyaa	TP2 @3m	1.16	5.23E-05	1.88E-04	6.45E-02	2.67E-07
2	Bulchiinsa	TP4 @3m	1.34	6.15E-05	1.56E-04	1.35E-01	7.34E-07

4.1.8 Permeability

The flow of water through soils depends upon its permeability coefficient I use falling head permeability test since this test is appropriate for fine grained soil. Since my soil is fine grain soil I prefer this test. A representative permeability tests were run on samples of two disturbed samples on two different test pits, where the remaining test pits have the similar result with either of these test pits. In void ratio versus log coefficient of permeability graph all soil samples taken from study area have nearly straight line relationship figure 4.13.

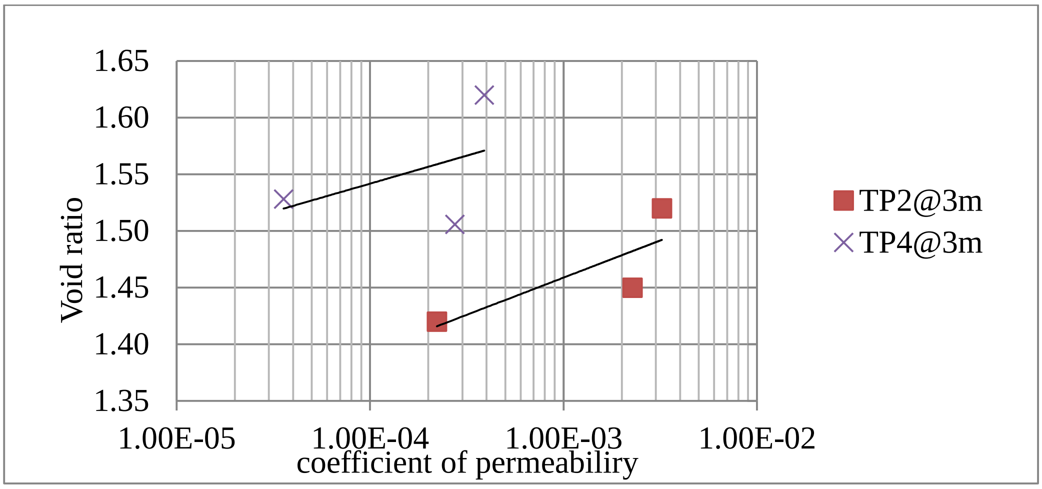


Figure 4.13 Void ratio Vs Log Coefficient of Permeability

Coefficient of permeability for TP-2 is 3.05*10^$^{-4}$, for Tp-4 is 3.11*10^$^{-5}$. The values of coefficient of permeability for the tested soils using falling head test lie between 2.75*10-4 and 3.75*10-5 cm/sec, which indicates that the soils are practically impervious. Refer Appendix-V.

CHAPTER FIVE

DISCUSSION ON TEST RESULTS

5.1 General

Soil classification is the arrangement of soils into different groups such that the soils in a particular group have similar behaviour. They are the American Association of State Highway and Transportation officials (AASHTO) classification system and the Unified Soil Classification System(USCS).

5.2 Discussion

The information provided in the plasticity chart is of great value and is the basis for the classification of fine-grained soils in the Unified Soil Classification System. A high numerical value of plasticity index is an indication of the presence of high percentage of clay in the soil sample. This implies that the plasticity values increase with the responding increase in clay content. Inorganic clay values lie above the A-line, and values for inorganic silts lie below the Aline. According to unified soil classification system as shown figure 4.3 the soils of the study area fall under CH and MH region, which shows that the soils are inorganic high plasticity clay and high plasticity Silt.

AASHTO Classification system results all the samples collected fall under A-7-5 and A-7-6. According to ERA manual, the plasticity Index results indicate that generally the soils of the study area are not Fair for highway subgrade material.

According grain size analysis result the dominant proportion of soil particle in the research area is clayey silty soil. The soils contain 38.44-54.98% clay, 37.56-51.74% silt, 5.24-9.82% sand and 0% gravel. The soils have (clay 48.40 % and Silt 43.81 % averagically) high fraction thus it is better to classify the soil as Clayey Silty. Since, the average amount of clayey soil is greater than Silty soil in percent; we call it clayey silty soil rather than Silty Clayey soil.

The Specific Gravity lies in the range between 2.65 to 2.77, which is within the range according to Arora (2004).

Liquid limit of Seka town falls in the range of 69.4-90% and plastic limit was in the range of 25-93.54.55% (Table 4.4). The plasticity index range of the soils was from 24.67% to

49.47% (Table 4.4). According to Murthy (2007) the plasticity of the soils are high plasticity. Because, the value of the plasticity Index is greater than seventeen.

The compression and recompression index of the soils is calculated from the straight portions of the loading and unloading e-log p curve. This calculation shows that the compression index, Cc, 0.23 and 0.39; swelling index, Cs, 0.02 and 019. The coefficient of permeability of soil under investigation which is calculated from the test results of consolidation test ranges from $3.11*10^{-5}$ and $4.58*10^{-4}$ cm/sec (Table 4.13). The result shows that the soil under investigation is impermeability.

The compaction test result shows that maximum dry density (MDD) ranges from 1.351 g/cm^3 to 1.480 g/cm^3 for modified compaction and from1.180 g/cm^3 to 1.352 g/cm^3 for standard compaction, and the optimum moisture content ranges 29.5% to 35.51% for modified compaction and 35.9% to 48.00% for standard compaction.

Unconfined compressive strength tests conducted on undisturbed representative samples show that unconfined compressive strength of Seka soils ranges from 143.52 – 352.92 kN/m2. According to the result of unconfined compression strength implies the Seka soils consistency is from stiff to very stiff in strength.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1. CONCLUSIONS

No significant variations of engineering properties within the investigated depths as well as in different pits which were found in the research work.

According to USCS the soils of the study area fall under CH and MH region, which shows that the soils are inorganic high plastic clay and high plastic Silt. And AASHTO classified the soil as (A-7-5 and A-7-6).

Grain size analysis result shows the soil under investigation is dominantly clay and silt types. Since, 92.2% average of the soil is fine grained soil; which have higher in plasticity. According to ERA manual not recommended for suitability of soils as sub grade material. Since, Plasticity Index is greater than thirty. The values of specific gravity are within the same ranges standards. The moisture content of the soil is medium.

As determined from the one-dimensional consolidation test conducted on undisturbed soil samples; we can conclude that: Since pit excavation method of exploration is used, the outcomes would be applicable only for light structures which under lie their foundation up to depth of 3m. Compaction tests results shows that, OMC is very high and MDD is very less. Which says that the soil is highly compressive.

From Unconfined Compressive Strength (UCS) result the consistency of soils is Stiff to very Stiff. And, the values of Liquidity Index classify the soil under the class of Intermediate strength, which the soil deform like a plastic material.

6.2. RECOMMENDATIONS

- ✓ In this research, samples of soil were collected only from eight test pits were excavated to the maximum depth of 3m. Eight test pits are not enough to generalize the engineering properties of soils found in Seka town. Therefore, it is recommended that a light weight structure is possible to construct in the study with a depth of the foundation up to 3 meters below the natural grade line.
- ✓ However, by increasing the number of test pits, a more detailed and accurate results can be obtained by deep investigation for further heaver building.
- ✓ To generalize the full description of the engineering properties for future the dynamic properties shall be studied in the town.
- ✓ The research, identify the strength of the soil consistency either the soil is soft, medium, stiff, very stiff or hard). But for deep investigation it is recommended that to find out the shear strength parameters Triaxial test should be carried out for better understanding.

REFERENCES

[1] Arora, K. R. (2004) Soil mechanics and foundation engineering, standard publishers' distributors. New Delhi.

[2] ASTM (2004) Special procedures for testing soil and rock for civil engineering purpose U.S. America.

[3] ASTM 1998, 'American society for testing and materials', Annual Book of ASTM Standards.

[4] Balasubramanian, A. (2017) Soil forming processes. Technical Report, DOI: 10.13140/ RG.2.2.34636.00644

[5] Budhu, M. (2000) Soil mechanics and foundations. John Wiley and Sons: U.S America.

[6] Murthy, V. N. S. (2003). Geotechnical engineering: Principles and practices of soil Mechanics and foundation engineering. Marcel Dekker, Inc. New York, USA.

[7] Blight, G.E., (, 1997), "Mechanics of Residual soils", A.A Balkema, the Netherlands.

[8] Jemal J. (2014) In-depth Investigation into Engineering Characteristics of Jimma Soils, Ethiopia.

[9] Eliud M. and Sixtus K. M., (2011), a study on the engineering behaviour of Nairobi subsoil, University of Nairobi.

[10] Terzaghi, K. R. (1996), Soil mechanics in engineering practice third edition U.S America: John Wiley and Sons.

[11] Yimam M. (2016). Investigations on some of the Engineering properties of soils found in Kemise town MSc thesis, A.A.U. Ethiopia.

[12] Alemayehu B. (2017) Correlation of CBR with Index Properties of Soils in Sululta Town Msc Thesis, AAU Ethiopia.

[13] Desta B. (2018) Investigation on Improving the Geotechnical Properties of Black Cotton Soil by Blending with Pumice, AAU, Ethiopia.

[14] Alemayehu, T. and Mesfin, L. (1999) Soil mechanics. Addis Ababa University Faculty of Technology, Addis Ababa, Ethiopia (Unpublished).

[15] Samuel T. (1989) Investigation into some of the engineering properties of Addis Ababa red clay soils. Addis Ababa, University.

[16] CIRIA, (1995), "Laterite in road pavements", Special publication 47 for Transport Research Laboratory (TRL), Westminster, London.

[17] Das, B. (2008). Advanced Soil Mechanics, third edition. USA: Taylor and Francis.

[18] Murthy, V. N. (1990). Geotechnical Engineering: Principles and Practices of Soil.

[19] Murthy, V. N. (2007). "Principles and Practice of Soil mechanics and Foundation. New York.

[20] Head. K.H. (n.d.). Manual of soil laboratory testing volume 2: Permeability, shear strength and compressibility tests. London: pentech press limited.

[21] Ethiopia Road Authority, (2002). Site Investigation Manual.

[22] Ethiopia Road Authority, R. E. (2013). Geotechnical investigation**.**

[23] Tizita D. (2017). Investigations on some of the Engineering properties of soils found in Sokoru town MSc thesis, JIT. Ethiopia.

[24] Atkinson J.H. The Mechanics of soils, University Series in Civil Engineering, Cardiff

[25] Tadesse A. (2014). Investigation in to some of the Engineering properties of soil in Woldiya town Ethiopia.

[26] Budhu, M. (2011) Soil mechanics and foundations. John Wiley and Sons: U.S America.

[27] Zewdie, A., 2004, Investigation into Shear Strength Characteristics of Expansive Soil of Ethiopia.

[28] Brussels, (2008), European Committee for Standardization, Eurocodes, "Basis of Geotechnical Design (EN 1997-1: 2084)".

[29] Mitchell, J.K., (1976), "Fundamentals of Soil Behaviour", Mc Graw-Hill Companies Inc.

Appendix-I Natural Moisture Content

	TP1@1.5m		**TP1@3m**	
Trial	1	2	1	2
Can No	LC51	9	2	P3
Mass of can, (g)	25.415	32.383	17.659	25.957
Mass of can + wet soil, (g)	132.55	122.644	101.437	136.522
Mass of can +dry soil, (g)	97.56	93.274	75.225	102.064
Mass of water, (g)	34.99	29.37	26.212	34.458
Mass of dry soil, (g)	72.145	60.891	57.566	76.107
Moisture content, (%)	48.50	48.23	45.53	45.28
Av. Moisture content, (%)	48.37		45.40	

	TP2@1.5m		**TP2@3m**	
Trial	1	2	1	2
Can No	ZE	P65	C	G10
Mass of can, (g)	33.075	37.789	36.693	17.123
Mass of can + wet soil, (g)	116.437	141.644	121.809	84.995
Mass of can +dry soil, (g)	89.852	108.44	95.992	64.342
Mass of water, (g)	26.585	33.204	25.817	20.653
Mass of dry soil, (g)	56.777	70.651	59.299	47.219
Moisture content, (%)	46.82	47.00	43.54	43.74
Av. Moisture content, (%)	46.91		43.64	

	TP3@1.5m		**TP3@3m**	
Trial	1	2	1	2
Can No	HC12	23	E	P2
Mass of can, (g)	18.144	17.64	17.159	17.465
Mass of can + wet soil, (g)	91.167	83.164	81.627	85.768
Mass of can +dry soil, (g)	68.204	62.317	59.195	61.915
Mass of water, (g)	22.963	20.847	22.432	23.853
Mass of dry soil, (g)	50.06	44.677	42.036	44.45
Moisture content, (%)	45.87	46.66	53.36	53.66
Av. Moisture content, (%)	46.27		53.51	

	TP4@1.5m		**TP4@3m**	
Trial	1	2	1	2
Can No	0A	F	A2	TC2
Mass of can, (g)	27.808	27.806	14.964	6.341
Mass of can + wet soil, (g)	108.627	106.701	49.826	38.88
Mass of can +dry soil, (g)	82.798	81.627	38.108	27.977
Mass of water, (g)	25.829	25.074	11.718	10.903
Mass of dry soil, (g)	54.99	53.821	23.144	21.636
Moisture content, (%)	46.97	46.59	50.63	50.39
Av. Moisture content, (%)	46.78		50.51	

	TP5@1.5m		**TP5@3m**	
Trial	1	2	1	2
Can No	F	C	E2	D3
Mass of can, (g)	36.423	32.867	16.659	24.957
Mass of can + wet soil, (g)	129.969	140.088	100.437	135.522
Mass of can +dry soil, (g)	100.409	106.31	74.225	103.064
Mass of water, (g)	29.56	33.778	26.212	32.458
Mass of dry soil, (g)	63.986	73.443	57.566	78.107
Moisture content, (%)	46.20	45.99	45.53	41.56
Av. Moisture content, (%)	46.09		43.54	

	TP6@1.5m		**TP6@3m**	
Trial	1	2	1	2
Can No	P15	D	T	I11
Mass of can, (g)	33.538	29.57	35.693	16.123
Mass of can + wet soil, (g)	127.049	130.145	120.809	83.995
Mass of can +dry soil, (g)	97.504	98.387	94.392	64.262
Mass of water, (g)	29.545	31.758	26.417	19.733
Mass of dry soil, (g)	63.966	68.817	58.699	48.139
Moisture content, (%)	46.19	46.15	45.00	40.99
Av. Moisture content, (%)	46.17		43.00	

	TP7@1.5m		**TP7@1.5m**	
Trial	1	2	1	2
Can No	II	C2	P	P3
Mass of can, (g)	18.011	17.563	16.159	16.465
Mass of can + wet soil, (g)	73.44	81.464	80.627	84.768
Mass of can +dry soil, (g)	54.782	59.924	60.495	61.915
Mass of water, (g)	18.658	21.54	20.132	22.853
Mass of dry soil, (g)	36.771	42.361	44.336	45.45
Moisture content, (%)	50.74	50.85	45.41	50.28
Av. Moisture content, (%)	50.79		47.84	

	TP8@1.5m		**TP8@3m**	
Trial	1	2	1	2
Can No	HB	I3	C1	CD14
Mass of can, (g)	9.766	9.545	14.364	6.841
Mass of can + wet soil, (g)	84.299	79.769	49.186	38.28
Mass of can +dry soil, (g)	58.576	55.586	38.788	26.977
Mass of water, (g)	25.723	24.183	10.398	11.303
Mass of dry soil, (g)	48.81	46.041	24.424	20.136
Moisture content, (%)	52.70	52.52	42.57	56.13
Av. Moisture content, (%)	52.61		49.35	

Appendix-II Index Properties

I-Specific Gravity

	TP1@1.5m			TP1@3m		
P. No	1	2	3	7	8	9
Mp (gm)	31.95	30.18	31.36	31.52	30.88	31.98
Mpw (gm) at Ti=22c	126.83	122.99	126.24	126.16	127.97	126.45
Ms (gm)	10.00	10.00	10.00	10.00	10.00	10.39
Mps (gm)	42.17	40.90	41.57	41.50	40.83	42.37
Mpsw (gm)	133.17	129.27	132.26	132.53	134.10	132.81
Temp. Tx in C°	24.00	24.00	24.00	23.00	23.00	23.00
Mpw (gm) at Tx	126.79	122.94	126.20	126.12	127.93	126.41
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.76	2.72	2.54	2.78	2.61	2.61
Gs at 20C°	2.76	2.72	2.54	2.78	2.61	2.60
Average Gs at 20C°	2.67			2.66		

	TP2@1.5m			TP2@3m		
P. No	10	11	12	A	8	F
Mp (gm)	30.82	26.88	30.41	29.54	31.21	30.14
Mpw (gm) at Ti=22 C°	126.21	123.03	126.83	126.88	128.12	125.75
Ms (gm)	10.10	10.01	10.01	10.28	10.00	10.00
Mps (gm)	40.60	36.97	40.45	40.08	40.94	39.74
Mpsw (gm)	132.15	129.24	133.08	133.38	134.14	131.82
Temp. Tx in C°	23.00	23.00	23.00	23.00	23.00	23.00
Mpw in gm at Tx	126.16	122.98	126.79	126.84	128.07	122.16
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.45	2.67	2.70	2.75	2.54	2.51
Gs at 20C°	2.45	2.67	2.69	2.75	2.54	2.51
Average Gs at 20C°	2.68			2.75		

	TP3@1.5m		**TP3@3m**		**TP4@1.5m**		**TP4@3m**	
P. No	1	2	5	6	7	8	11	12
Mp (gm)	31.95	30.18	29.02	30.12	31.52	30.88	26.86	30.41
Mpw (gm) at Ti=22C°	126.83	122.99	124.97	123.34	126.16	127.97	123.03	126.83
Ms (gm)	10.00	10.00	10.10	10.08	10.00	10.00	10.25	10.01
Mps (gm)	42.17	40.90	39.89	40.88	41.50	40.83	37.11	40.42
Mpsw (gm)	133.17	129.27	131.35	129.75	132.53	134.10	129.41	132.85
Temp. Tx in C°	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00
Mpw (gm) at Tx	126.78	122.94	124.92	123.29	126.11	127.92	122.98	126.78
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.77	2.73	2.76	2.79	2.79	2.62	2.69	2.54
Gs at 20C°	2.76	2.72	2.75	2.78	2.78	2.61	2.68	2.54
Average Gs at 20C°	2.74		2.77		2.70		2.68	

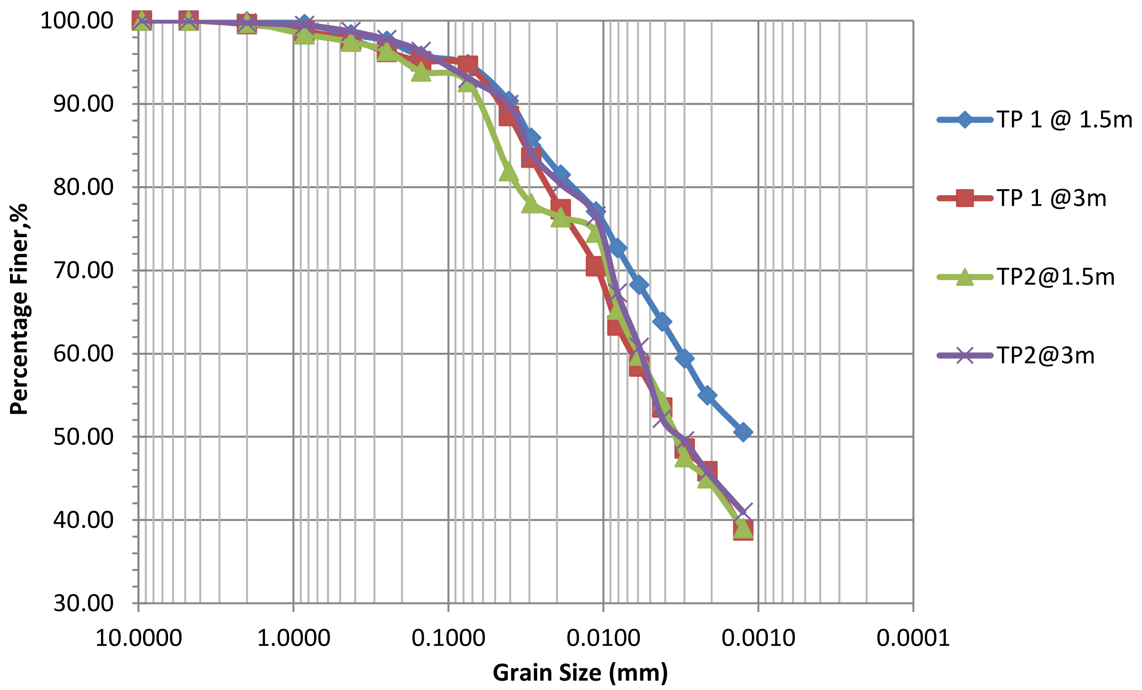
	TP5@1.5m			**TP5@3m**		
P. No	4	5	6	7	8	9
Mp (gm)	30.84	28.79	29.96	31.53	30.91	31.74
Mpw (gm) at Ti=22C°	126.44	124.97	123.34	126.56	128.17	126.45
Ms (gm)	10.00	10.00	10.00	10.28	10.05	10.06
Mps (gm)	38.10	38.30	37.80	41.29	40.90	47.11
Mpsw (gm)	132.87	131.35	129.42	132.67	134.49	132.66
Temp. Tx in C°	24.00	24.00	24.00	23.00	23.00	23.00
Mpw (gm) at Tx	126.40	124.92	123.30	126.52	128.13	126.41
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.84	2.80	2.58	2.65	2.73	2.64
Gs at 20C°	2.83	2.79	2.57	2.65	2.72	2.68
Average Gs at 20C°	2.73			2.69		

	TP6@1.5m			TP6@3m		
P. No	13	14	3H	A	8	F
Mp (gm)	28.53	29.45	28.99	29.54	31.21	30.14
Mpw (gm) at Ti=22C°	124.80	123.77	124.99	126.88	128.12	122.20
Ms (gm)	10.00	10.00	10.00	10.28	10.00	10.00
Mps (gm)	38.55	39.45	38.50	40.08	40.94	3..974
Mpsw (gm)	131.01	129.92	131.08	133.38	134.14	131.82
Temp. Tx in C°	24.00	24.00	24.00	23.00	23.00	23.00
Mpw (gm) at Tx	124.75	123.73	124.95	126.84	128.07	122.16
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.67	2.63	2.59	2.75	2.54	2.56
Gs at 20C°	2.67	2.63	2.59	2.75	2.54	2.56
Average Gs at 20C°	2.65			2.75		

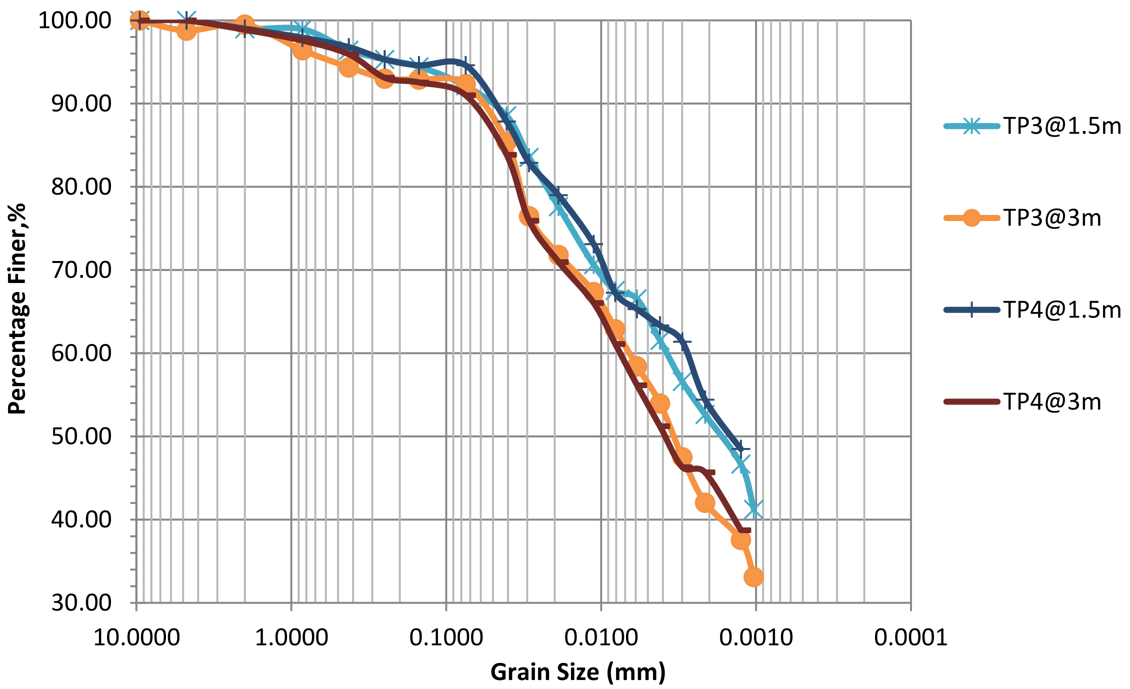
	TP7@1.5m		TP7@1.5m		TP8@1.5m		TP8@1.5m	
P. No	3	4	5	6	9	10	11	12
Mp (gm)	31.36	30.84	29.02	30.12	31.98	30.79	26.86	30.41
Mpw gm (Ti=22C°)	126.24	126.44	124.97	123.34	126.45	126.21	123.03	126.83
Ms (gm)	10.00	10.00	10.10	10.08	10.39	10.19	10.25	10.01
Mps (gm)	41.57	41.05	39.89	40.88	42.37	40.97	37.11	40.42
Mpsw (gm)	132.26	132.57	131.35	129.75	132.81	132.40	129.41	132.85
Temp. Tx in C°	26.00	26.00	26.00	26.00	26.00	26.00	26.00	26.00
Mpw (gm) @ Tx	126.19	126.39	124.92	123.29	126.40	126.16	122.98	126.78
K for Tx	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gs at Tx	2.55	2.62	2.76	2.79	2.67	2.58	2.69	2.54
Gs at 20C°	2.54	2.61	2.75	2.78	2.66	2.68	2.68	2.54
Average Gs at 20C°	2.61		2.77		2.67		2.68	

II-Grain Size Analysis

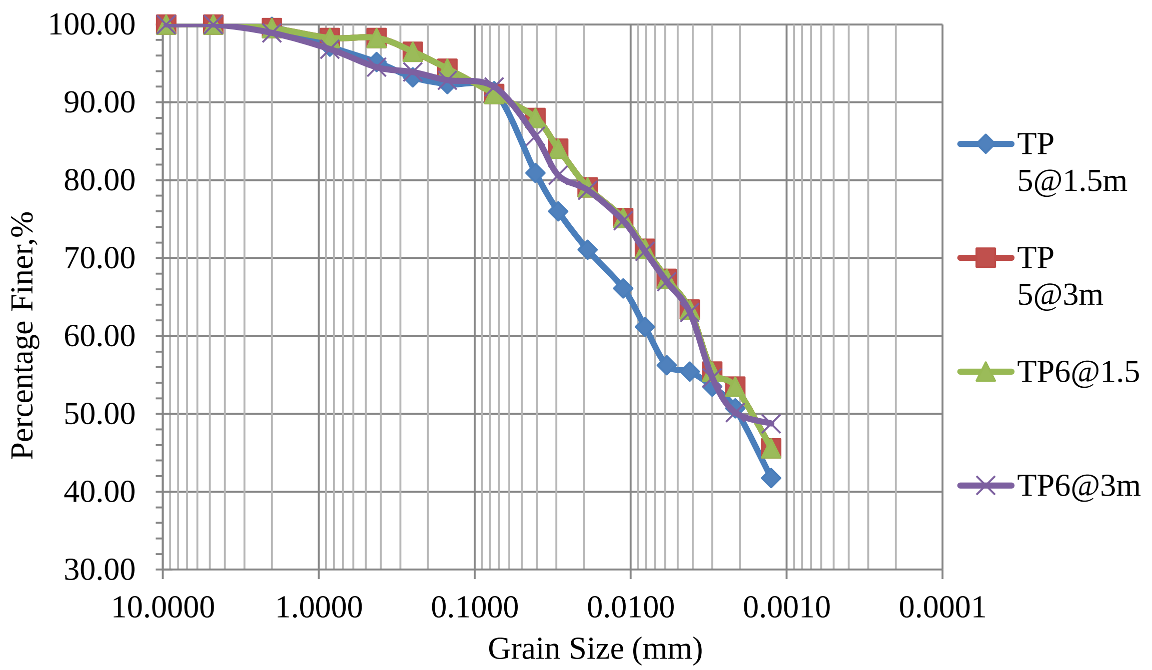
Grain size		TP 1 @ 1.5m		TP 1 @3m		TP2 @1.5m		TP 2@ 3m	
	9.5000		100.00		100.00		100.00		100.00
Gravel	4.7500	0.00	100.00	0.00	100.00	0.00	100.00	0.00	100.00
	2.0000		99.88		99.56		99.74		99.74
	0.8500		99.56		98.71		98.35		99.43
	0.4250		98.34		97.65		97.44		98.67
	0.2500		97.56		96.23		96.33		97.73
	0.1500		95.78		95.12		93.87		96.29
Sand	0.0750	5.24	94.76	5.42	94.58	7.41	92.59	6.92	93.08
	0.0408		90.34		88.54		81.90		89.93
	0.0292		85.92		83.53		78.06		84.08
	0.0189		81.50		77.32		76.37		80.38
	0.0112		77.08		70.47		74.52		76.54
	0.0081		72.66		63.38		65.30		67.29
	0.0059		68.24		58.45		59.77		60.75
	0.0042		63.82		53.52		54.23		52.20
	0.0030		59.40		48.59		47.54		49.50
Silt`	0.0021	39.78	54.98	48.72	45.86	47.59	45.00	47.27	45.81
	0.0013		50.56		28.73		39.01		40.96
Clay	0.0010	54.98		45.86		45.00		45.81	



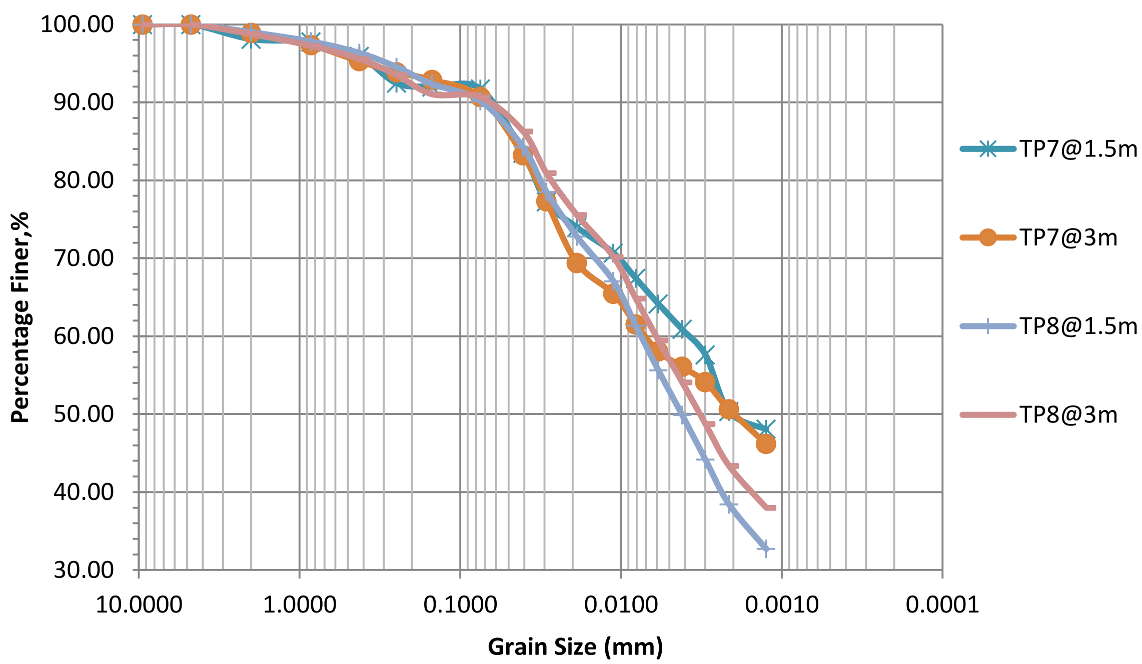
Grain size		TP 3@ 1.5m		TP 3 @ 3m		TP 4 @ 1.5m		TP 4 @3m	
	9.5000		100.00		100.00		100.00		100.00
Gravel	4.7500	0.00	100.00	0.00	100.00	0.00	100.00	0.00	100.00
	2.0000		99.02		99.52		99.03		98.88
	0.8500		98.97		96.43		97.95		97.59
	0.4250		96.45		94.38		96.80		95.92
	0.2500		95.34		94.03		95.34		93.12
	0.1500		94.45		93.89		94.62		92.55
Sand	0.0750	7.84	92.16	6.43	93.57	5.38	94.62	9.00	91.00
	0.0408		88.55		85.42		87.86		83.85
	0.0292		83.54		76.47		82.91		75.91
	0.0189		77.62		71.83		78.99		70.98
	0.0112		70.65		67.37		73.13		66.04
	0.0081		67.54		62.90		67.26		61.10
	0.0059		66.54		58.44		65.31		56.17
	0.0042		61.56		53.97		63.35		51.23
	0.0030		56.60		47.51		61.40		46.30
Silt`	0.0021	39.52	52.64	50.33	42.04	40.18	54.44	45.33	45.68
	0.0013		46.68		37.58		48.49		38.74
Clay	0.0010	52.64	41.23	42.04	33.11	54.44		45.68	



Grain size		TP 5@1.5m		TP 5@3m		TP6 @1.5m		TP 6 @ 3m	
	9.5000		100.00		100.00		100.00		100.00
Gravel	4.7500	0.00	100.00	0.00	100.00	0.00	100.00	0.00	100.00
	2.0000		99.71		99.51		99.13		98.92
	0.8500		97.14		98.26		98.55		96.84
	0.4250		95.16		98.26		97.86		94.51
	0.2500		93.22		96.51		97.65		93.89
	0.1500		92.33		94.32		94.23		92.85
Sand	0.0750	8.56	91.44	8.92	91.08	7.66	92.34	8.01	91.99
	0.0408		80.94		88.00		87.55		85.64
	0.0292		76.00		84.04		80.69		80.64
	0.0189		71.06		79.08		74.49		78.72
	0.0112		66.13		75.13		71.53		74.80
	0.0081		61.19		71.21		68.58		70.88
	0.0059		56.25		67.34		65.62		66.95
	0.0042		55.43		63.39		62.66		63.03
	0.0030		53.51		55.43		59.71		54.60
Silt`	0.0021	40.75	50.69	37.56	53.52	41.59	50.75	41.82	50.17
	0.0013		41.76		45.56		48.79		48.76
Clay	0.0010	50.69		53.52		50.75		50.17	



Grain size		TP 7 @1.5m		TP 7@ 3m		TP 8@1.5m		TP 8@ 3m	
	9.5000		100.00		100.00		100.00		100.00
Gravel	4.7500	0.00	100.00	0.00	100.00	0.00	100.00	0.00	100.00
	2.0000		98.10		98.90		99.00		98.75
	0.8500		97.75		97.35		97.80		97.25
	0.4250		95.93		95.28		96.34		95.56
	0.2500		92.45		93.88		94.54		93.52
	0.1500		91.99		92.88		92.34		91.05
Sand	0.0750	8.27	91.73	9.32	90.68	9.82	90.18	9.32	90.68
	0.0408		83.49		83.23		84.25		86.28
	0.0292		77.22		77.31		78.52		80.91
	0.0189		73.95		69.41		72.80		75.55
	0.0112		70.69		65.46		67.07		70.18
	0.0081		67.42		61.51		61.35		64.82
	0.0059		64.15		58.06		55.62		59.46
	0.0042		60.88		56.08		49.89		54.09
	0.0030		57.61		54.11		44.17		48.73
Silt`	0.0021	41.38	50.35	40.03	50.65	51.74	38.44	47.31	43.37
	0.0013		48.08		46.18		32.72		38.00
Clay	0.0010	50.35		50.65		38.44		43.37	

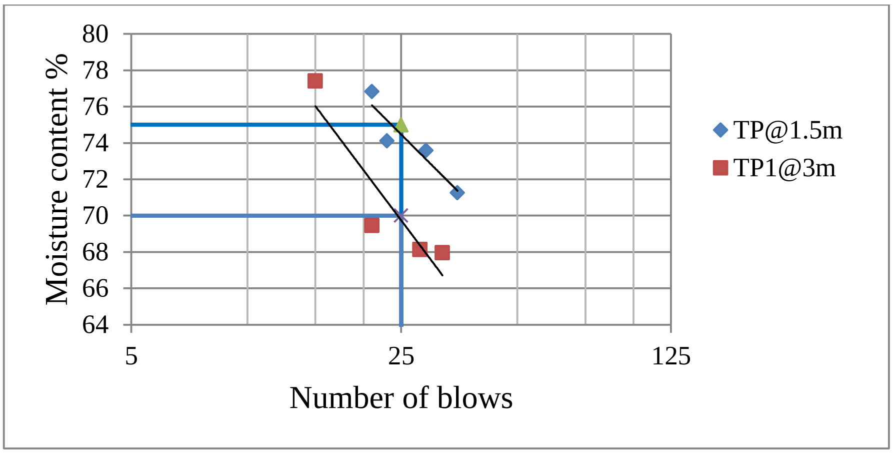


III-Atterberg Limit

Liquid Limit	**TP1@1.5m**				**TP1@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	35	29	23	21	32	28	21	15
Can code	G	G53	DE	NB	A2	30	3	C2
Mass of can, (g)	17.94	17.48	17.39	17.59	5.01	6.895	16.604	5.873
Mass of can + wet soil, (g)	32.92	31.47	28.96	31.11	23.04	21.044	31.324	20.974
Mass of can +dry soil, (g)	26.687	25.539	24.035	25.236	15.744	15.31	25.29	14.385
Mass of water, (g)	6.233	5.931	4.925	5.874	7.296	5.734	6.034	6.589
Mass of dry soil, (g)	8.747	8.059	6.645	7.646	10.734	8.415	8.686	8.512
Moisture content, (%)	71.23	73.59	74.12	76.82	67.97	68.14	69.47	77.41

Plastic Limit	**TP1@1.5m**			**TP1@3m**		
Trial	1	1	2	3	2	3
Can No	3A	G2	205	2	205	2
Mass of can, (g)	17.6	17.93	13.35	15.98	16.63	28.63
Mass of can + wet soil, (g)	24.13	24.01	24.78	24.05	24.15	41.46
Mass of can +dry soil, (g)	22.29	22.605	21.83	21.981	22	39.067
Mass of water, (g)	1.84	1.405	2.95	2.069	2.172	2.393
Mass of dry soil, (g)	4.69	4.675	8.48	6.001	5.348	10.437
Moisture content, (%)	32.23	23.05	27.79	26.48	26.61	20.93
Av. Moisture content, (%)	27.69			24.67		

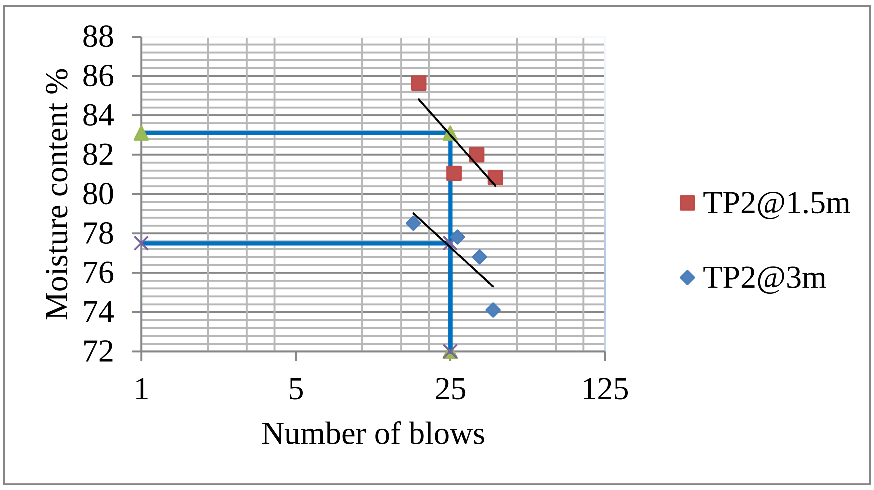
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP1@1.5m	74.9	27.69	47.21
TP1@3m	70	24.67	45.33



Liquid Limit	TP2@1.5m				TP2@3m			
Trial	1	2	3	4	1	2	3	4
No of blows	40	33	26	18	39	34	27	17
Can code	HC12	P2	A3	G3T3	N3	37	12	58
Mass of can, (g)	18.134	17.491	16.987	17.902	5.557	5.978	5.458	6.417
Mass of can + wet soil, (g)	30.917	31.101	30.328	32.372	21.088	21.192	22.255	21.124
Mass of can +dry soil, (g)	25.203	24.969	24.356	25.697	14.478	14.583	14.905	14.656
Mass of water, (g)	5.714	6.132	5.972	6.675	6.612	6.609	7.35	6.468
Mass of dry soil, (g)	7.069	7.479	7.369	7.795	8.921	8.605	9.447	8.239
Moisture content, (%)	80.83	81.989	81.04	85.63	74.09	76.80	77.80	78.50

Plastic Limit	TP2@1.5m			TP2@3m		
Trial	1	2	3	1	2	3
Can No	B3	G53	T1C1	31	49	29
Mass of can, (g)	18.400	18.478	17.92	5.376	5.142	5.694
Mass of can + wet soil, (g)	24.657	25.08	24.717	15.553	11.753	13.358
Mass of can +dry soil, (g)	22.51	22.913	22.655	13.195	9.632	10.94
Mass of water, (g)	2.147	2.167	2.062	2.358	2.121	2.418
Mass of dry soil, (g)	4.11	4.435	4.735	7.819	4.49	5.246
Moisture content, (%)	52.24	48.86	43.55	30.16	47.24	46.09
Av. Moisture content, (%)	48.22			41.16		

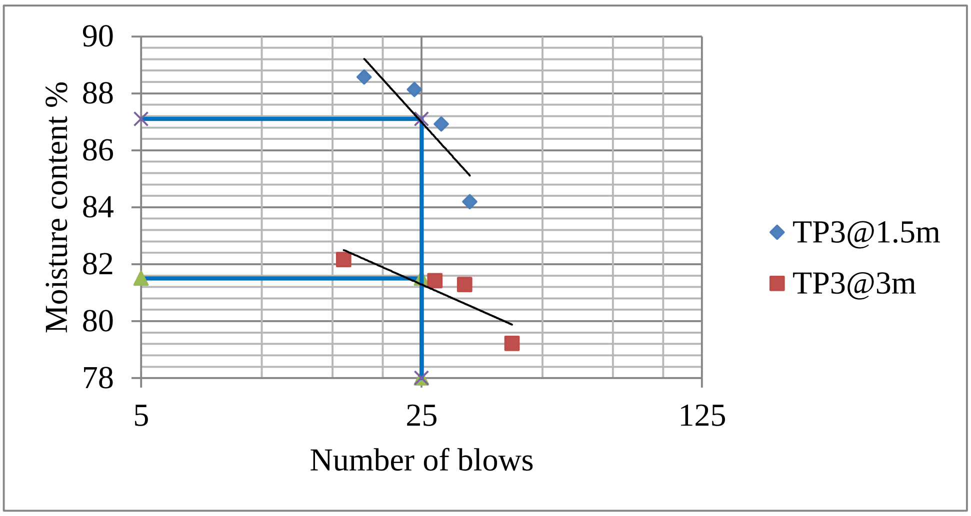
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP2@1.5m	83.1	48.22	34.88
TP2@3m	76.42	41.16	35.26



Liquid Limit	**TP3@1.5m**				**TP3@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	33	28	24	18	42	32	27	16
Can code	31	C4	50	27	2	B1	G7	3
Mass of can, (g)	8.249	17.69	16.98	5.85	17.21	17.876	16.798	16.723
Mass of can + wet soil, (g)	22.415	32.55	31.56	23.5	35.27	35.79	34.686	31.933
Mass of can +dry soil, (g)	15.94	25.64	24.73	15.21	27.287	27.758	26.658	25.073
Mass of water, (g)	6.475	6.91	6.83	8.29	7.983	8.032	8.028	6.86
Mass of dry soil, (g)	7.691	7.95	7.75	9.36	10.077	9.882	9.86	8.35
Moisture content, (%)	84.19	86.92	88.13	88.57	79.22	81.28	81.42	82.16

Plastic Limit	**TP3@1.5m**			**TP3@3m**		
Trial	1	2	3	1	2	3
Can No	3	B3	C12	G3T3	T3	HC11
Mass of can, (g)	16.59	5.468	16.32	17.96	17.69	17.98
Mass of can + wet soil, (g)	22.65	11.68	23.039	24.697	23.65	25.292
Mass of can +dry soil, (g)	20.935	10.02	21.11	23.34	21.726	22.724
Mass of water, (g)	1.715	1.66	1.929	1.357	1.924	2.568
Mass of dry soil, (g)	4.345	4.552	4.79	5.38	4.036	4.744
Moisture content, (%)	39.47	36.47	40.27	25.22	47.67	54.13
Av. Moisture content, (%)	38.74			42.34		

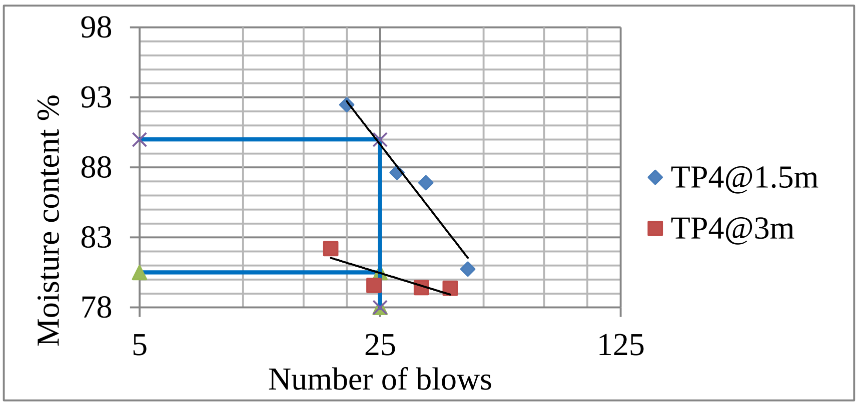
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP3@1.5m	87.1	38.74	48.36
TP3@3m	81.5	42.34	39.16



Liquid Limit	**TP4@1.5m**				**TP4@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	45	34	28	20	40	33	24	18
Can code	30	C2	A2	3	60	A20	38	56
MC, (g)	7.895	6.26	6.05	16.6	6.01	19.17	5.66	5.87
MCMS, (g)	24.93	19.25	18.751	25.727	24.03	35.03	20.75	22.13
MCDS, (g)	17.32	13.21	12.819	21.342	16.056	28.01	14.06	14.79
MW, (g)	7.61	6.04	5.932	4.385	7.974	7.02	6.687	7.34
MDS, (g)	9.425	6.95	6.769	4.742	10.046	8.84	8.403	8.93
M/content, (%)	80.74	86.91	87.63	92.47	79.37	79.44	79.58	82.22

Plastic limit	**TP4@1.5m**			**TP4@3m**		
Trial	1	2	2	1	2	3
Can No	A20	60	38	13	S1	49
Mass of can, (g)	18.27	5.457	6.278	6.397	19.637	5.563
Mass of can + wet soil, (g)	30.965	15.92	15.24	12.071	24.764	10.844
Mass of can +dry soil, (g)	27.48	12.969	12.426	10.305	23.088	9.137
Mass of water, (g)	3.485	2.951	2.814	1.766	1.676	1.707
Mass of dry soil, (g)	9.21	7.512	6.148	3.908	3.451	3.574
Moisture content, (%)	37.84	39.28	45.77	45.19	48.57	47.76
Av. Moisture content, (%)	40.96			47.17		

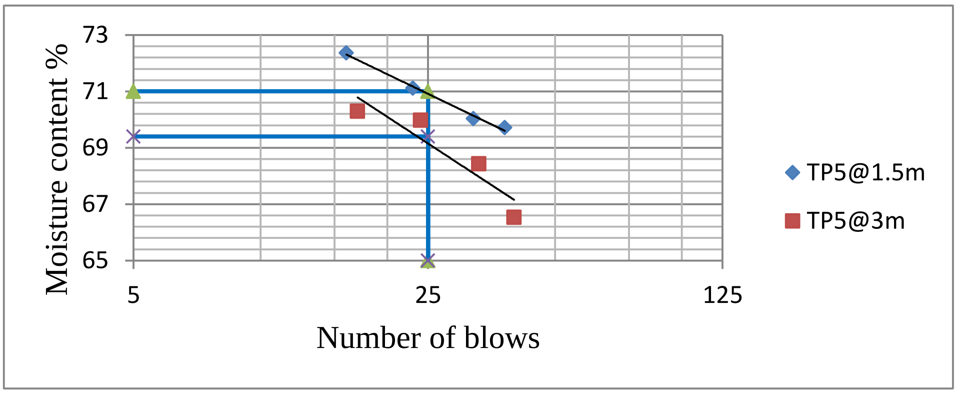
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP4@1.5m	90	40.96	49.04
TP4@3m	80.5	47.17	33.33



Liquid Limit	**TP5@1.5m**				**TP5@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	38	32	23	16	40	33	24	17
Can code	DE	G3T2	P5	T2C1	F	C	T	U
MC, (g)	17.381	17.127	17.164	17.043	16.891	16.876	16.985	16.743
MCMS, (g)	34.807	35.27	35.951	34.653	34.807	35.27	35.951	34.653
MCDS, (g)	27.649	27.797	28.143	27.26	27.649	27.797	28.143	27.26
MW, (g)	7.158	7.473	7.808	7.393	7.158	7.473	7.808	7.393
MDS, (g)	10.268	10.67	10.979	10.217	10.758	10.921	11.158	10.517
M/content, (%)	69.71	70.04	71.12	72.36	66.54	68.43	69.98	70.30

Plastic Limit	**TP5@1.5m**			**TP5@3m**		
Trial	1	2	3	1	2	3
Can No	MK	50	G	FG	SD	AS
Mass of can, (g)	16.44	16.726	16.577	16.293	16.12	16.21
Mass of can + wet soil, (g)	24.033	25.395	25.977	24.033	25.395	25.977
Mass of can +dry soil, (g)	22.126	23.04	23.426	22.126	23.04	23.426
Mass of water, (g)	1.907	2.355	2.551	1.907	2.355	2.551
Mass of dry soil, (g)	5.686	6.314	6.849	5.833	6.92	7.216
Moisture content, (%)	25.54	29.30	29.25	24.69	26.03	27.35
Av. Moisture content, (%)	28.03			26.03		

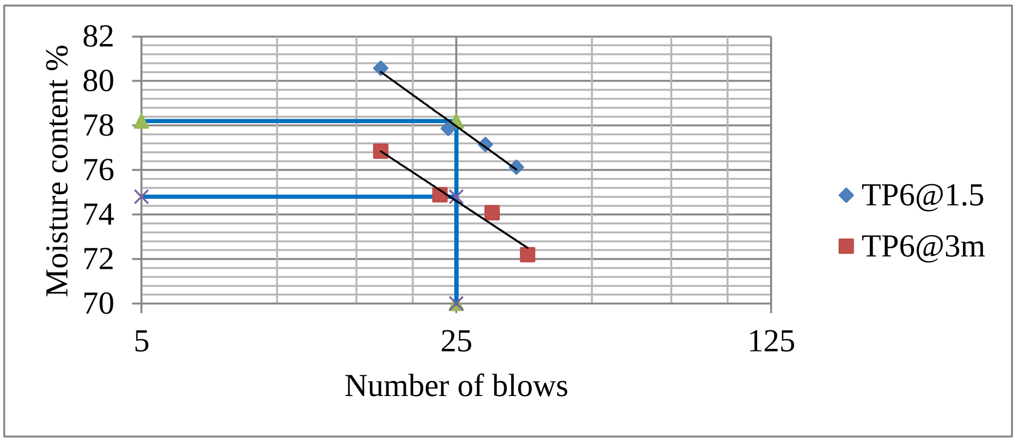
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP5@1.5m	71	28.03	42.97
TP5@3m	69.4	26.03	43.37



Liquid Limit	**TP6@1.5m**				**TP6@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	34	29	24	17	36	30	23	17
Can code	P2	A3	50	B3	P2	A3	50	B3
MC, (g)	17.48	17.93	16.98	17.4	17.32	17.67	16.74	17.01
MCMS, (g)	31.64	29.06	27.71	31.95	31.44	29.06	27.71	31.95
MCDS, (g)	26	24.21	23.01	25.458	26	24.21	23.01	25.45
MW, (g)	6.12	4.84	4.697	6.492	5.92	4.847	4.697	6.492
MDS, (g)	8.04	6.283	6.033	8.058	8.2	6.543	6.273	8.448
M/content, (%)	76.12	77.14	77.86	80.57	72.20	74.08	74.88	76.85

Plastic Limit	**TP6@1.5m**			**TP6@3m**		
Trial	1	2	3	1	2	3
Can No	P1	4	HC12	P2	P1	P15
Mass of can, (g)	17.9	17.56	18.13	16.68	16.524	18.21
Mass of can + wet soil, (g)	24.14	23.95	28.96	24.14	23.95	28.96
Mass of can +dry soil, (g)	22.29	22.01	25.7	22.29	22.017	25.704
Mass of water, (g)	1.85	1.933	3.256	1.85	1.933	3.256
Mass of dry soil, (g)	4.39	4.457	7.574	5.61	5.493	7.494
Moisture content, (%)	42.14	43.37	42.99	32.98	35.19	43.45
Av. Moisture content,(%)	42.83			37.21		

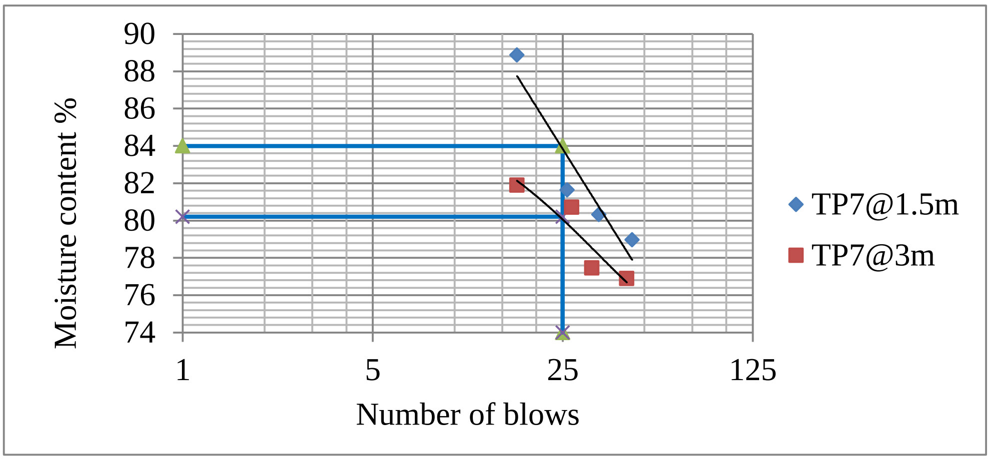
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP6@1.5m	78.2	43.37	34.83
TP6@3m	74.8	37.21	37.59



Liquid Limit	**TP7@1.5m**				**TP7@3m**			
Trial	1	2	3	4	1	2	3	4
No of blows	45	34	26	17	43	32	27	17
Can code	2	MK	3	B1	3	WQ	AS	B1
MC, (g)	17.65	17.637	17.716	18.222	17.45	17.34	17.65	17.56
MCMS, (g)	30.867	32.07	28.222	32.918	30.867	32.07	28.222	32.918
MCDS, (g)	25.035	25.641	23.5	26.003	25.035	25.641	23.5	26.003
MW, (g)	5.832	6.429	4.722	6.915	5.832	6.429	4.722	6.915
MDS, (g)	7.385	8.004	5.784	7.781	7.585	8.301	5.85	8.443
M/content, (%)	78.97	80.32	81.64	88.87	76.89	77.45	80.72	81.90

Plastic Limit	**TP7@1.5m**			**TP7@3m**		
Trial	1	2	3	1	2	3
Can No	6	HC11	G7	7	HC12	G8
Mass of can, (g)	16.329	16.28	17.374	16.03	15.22	15.95
Mass of can + wet soil, (g)	23.246	23.912	23.481	23.246	23.912	23.481
Mass of can +dry soil, (g)	21.308	21.958	21.585	21.308	21.958	21.585
Mass of water, (g)	1.938	1.954	1.896	1.938	1.954	1.896
Mass of dry soil, (g)	4.979	5.678	4.211	5.278	6.738	5.635
Moisture content, (%)	28.92	24.41	35.02	30.72	23.00	27.65
Av. Moisture content,(%)	29.45			27.12		

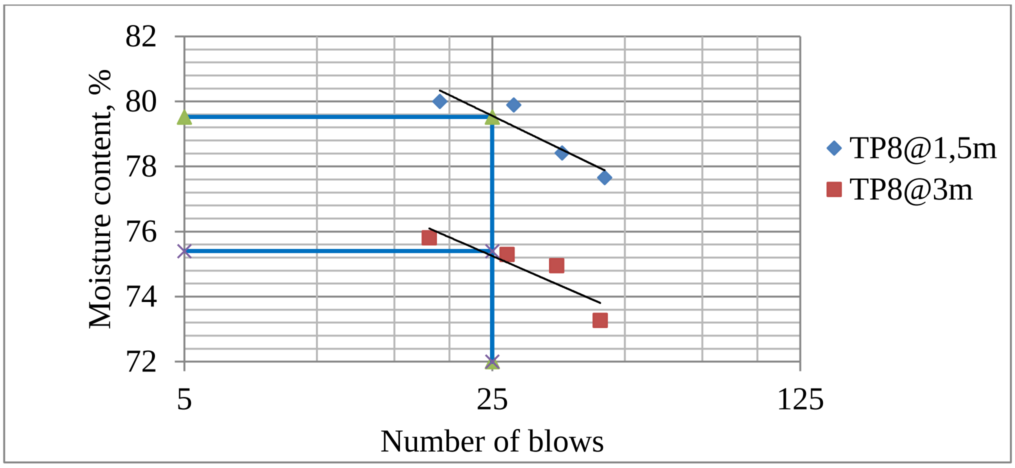
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP7@1.5m	84	29.45	54.55
TP7@3m	80.2	27.12	53.08



Liquid Limit	TP8@1.5m				TP8@3m			
Trial	1	2	3	4	1	2	3	4
No of blows	45	36	28	19	44	35	27	18
Can code	10	C2/R1	A20	13*	11	R2	A10	12
MC, (g)	5.852	6.21	6.35	6.491	5.43	5.87	6.04	6.09
MCMS, (g)	18.368	19.347	19.78	19.552	18.368	19.347	19.78	19.552
MCDS, (g)	12.897	13.573	13.878	13.747	12.897	13.573	13.878	13.747
MW, (g)	5.471	5.774	5.902	5.805	5.471	5.774	5.902	5.805
MDS, (g)	7.045	7.363	7.387	7.256	7.467	7.703	7.838	7.657
M/content, (%)	77.66	78.42	79.90	80.00	73.27	74.96	75.30	75.81

Plastic Limit	TP8@1.5m			TP8@3m		
Trial	1	2	3	1	2	3
Can No	56	7	60	54	6	70
Mass of can, (g)	6.399	6.341	6.459	6.252	6.744	6.544
Mass of can + wet soil, (g)	12.469	13.344	12.872	12.469	13.344	12.872
Mass of can +dry soil, (g)	10.474	11.078	10.795	10.474	11.078	10.795
Mass of water, (g)	1.995	2.266	2.077	1.995	2.266	2.077
Mass of dry soil, (g)	4.075	4.737	4.336	4.222	4.334	4.251
Moisture content, (%)	48.96	47.84	47.90	47.25	52.28	48.86
Av. Moisture content,(%)	48.23			49.47		

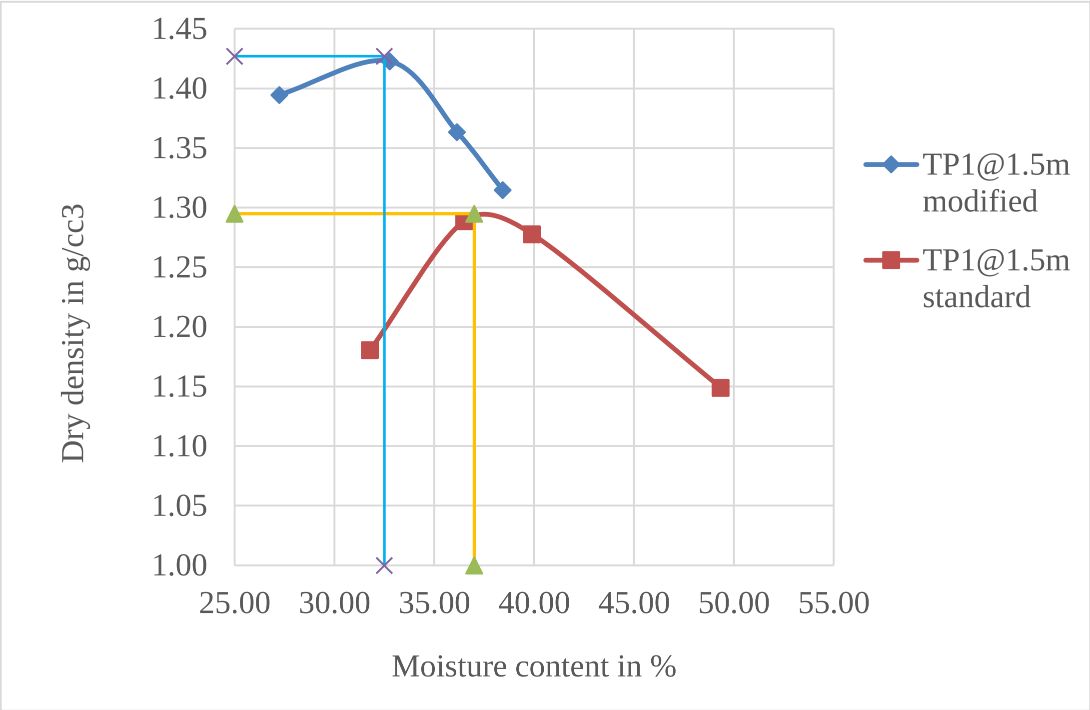
Pit depth	Liquid Limit (LL)	Plastic Limit (PL)	Plastic index (PI) =LL-PL
TP8@1.5m	79.52	48.23	31.29
TP8@3m	75.4	49.47	25.93

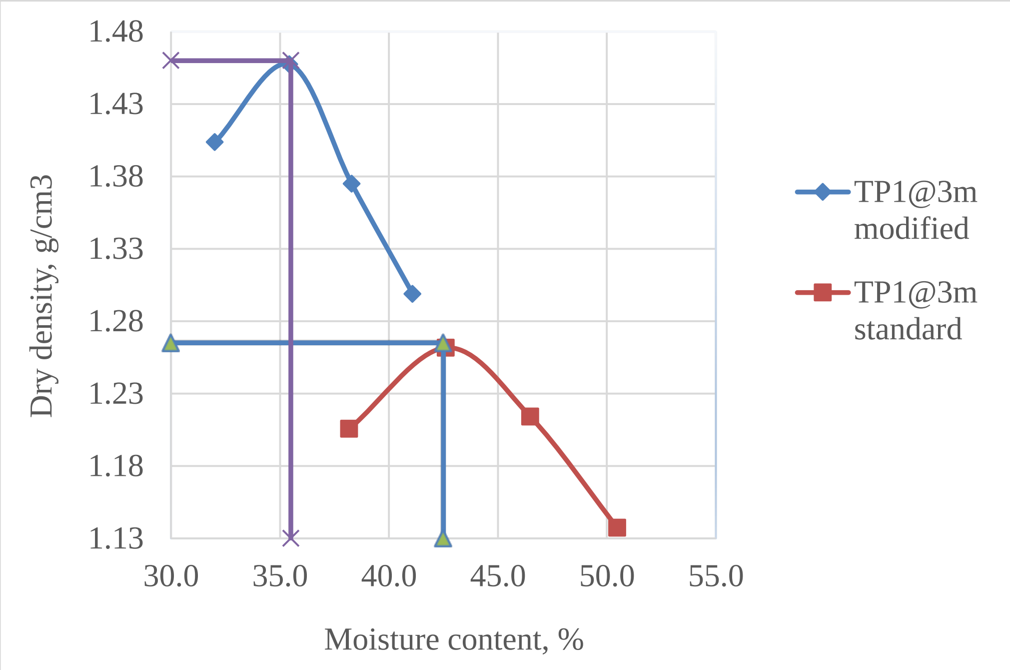

Moisture content, %
82
80
78
76
74
72
5
25
125
Number of blows
TP8@1,5m
TP8@3m

Appendix-III Compaction Test Results

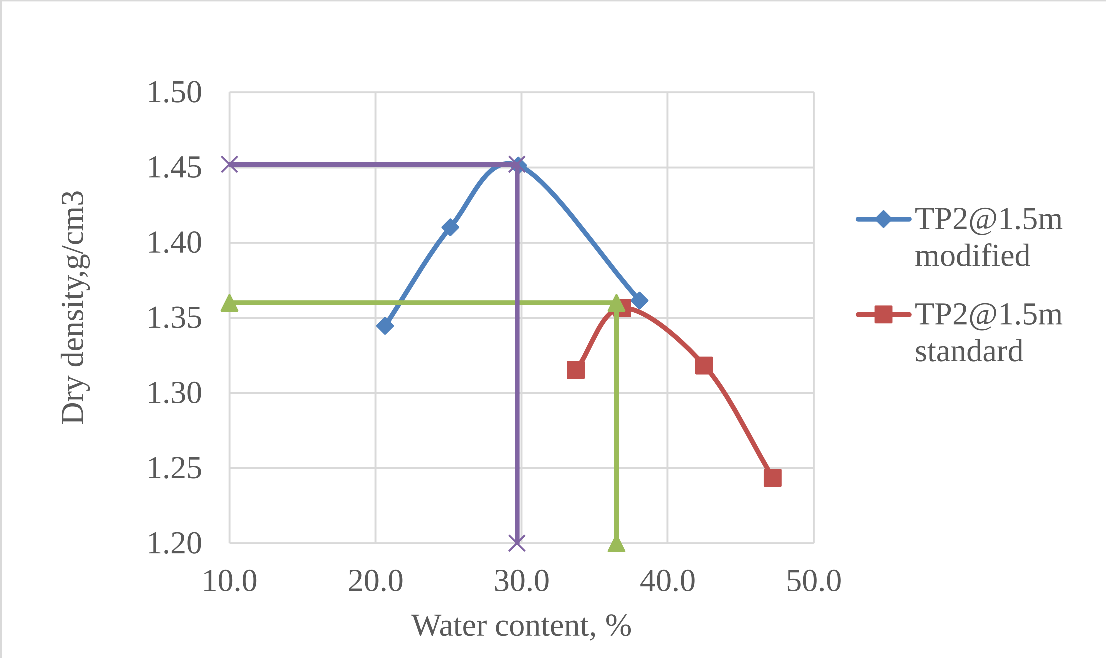
(Both methods)

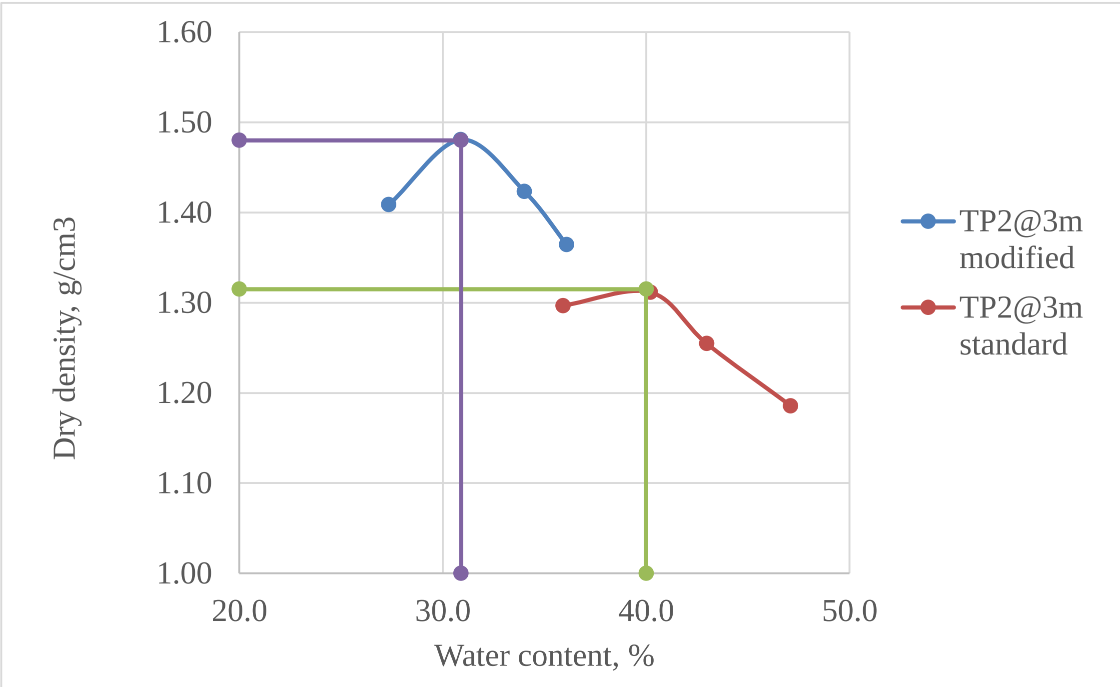
TP	TP1@1.5m					TP1@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10265.8	1.77	27.24	1.39	10350.6	1.85	32.0	1.40
	2	10501.7	1.89	32.76	1.42	10599.6	1.97	35.4	1.46
	3	10434.3	1.86	36.14	1.36	10450.4	1.90	38.3	1.37
	4	10359.6	1.82	38.42	1.31	10308.8	1.83	41.1	1.30
	OMC,%	32.5				35.5			
	MDD, g/cm^3	1.427				1.46			
Standard	1	5508.4	1.56	31.78	1.18	5611.2	1.67	38.2	1.21
	2	5698.6	1.76	36.51	1.29	5735.9	1.80	42.6	1.26
	3	5724.8	1.79	39.89	1.28	5716.5	1.78	46.5	1.21
	4	5658.0	1.72	49.34	1.15	5653.9	1.71	50.5	1.14
	OMC,%	37				42.5			
	MDD, g/cm^3	1.295				1.265			



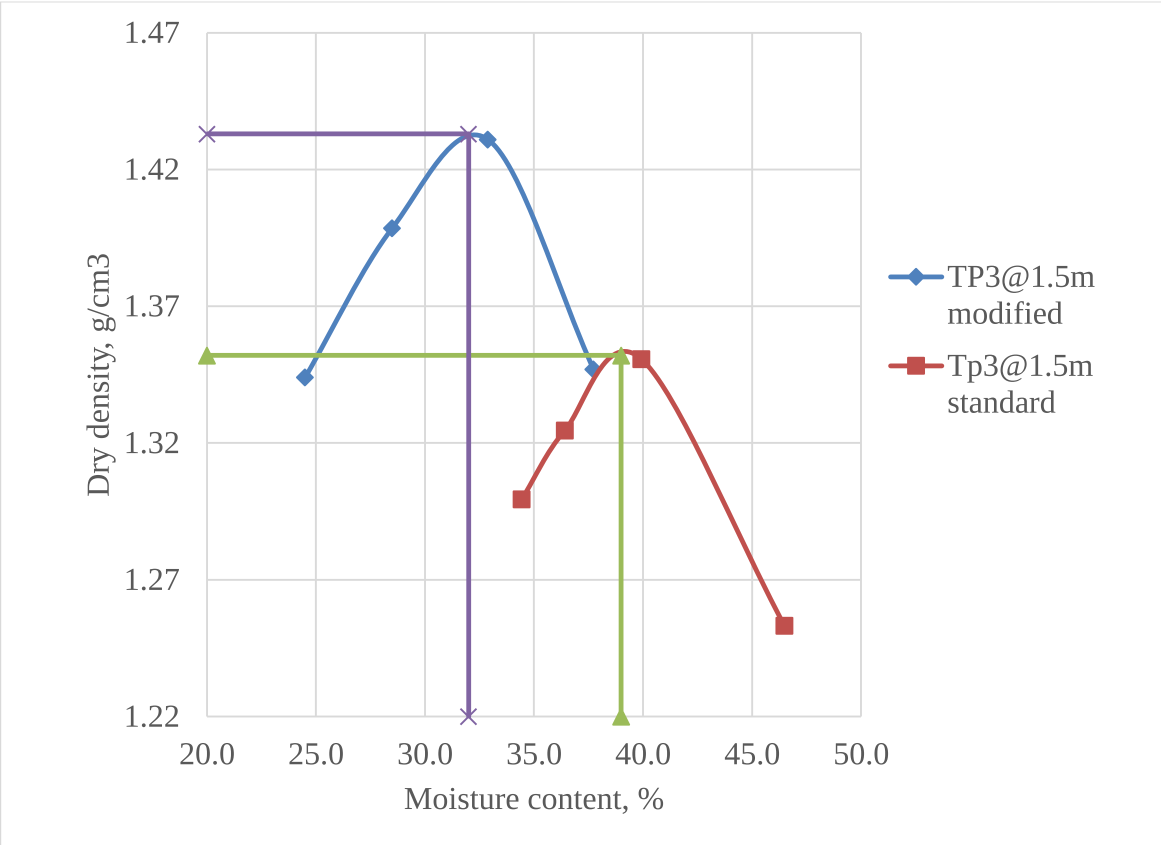


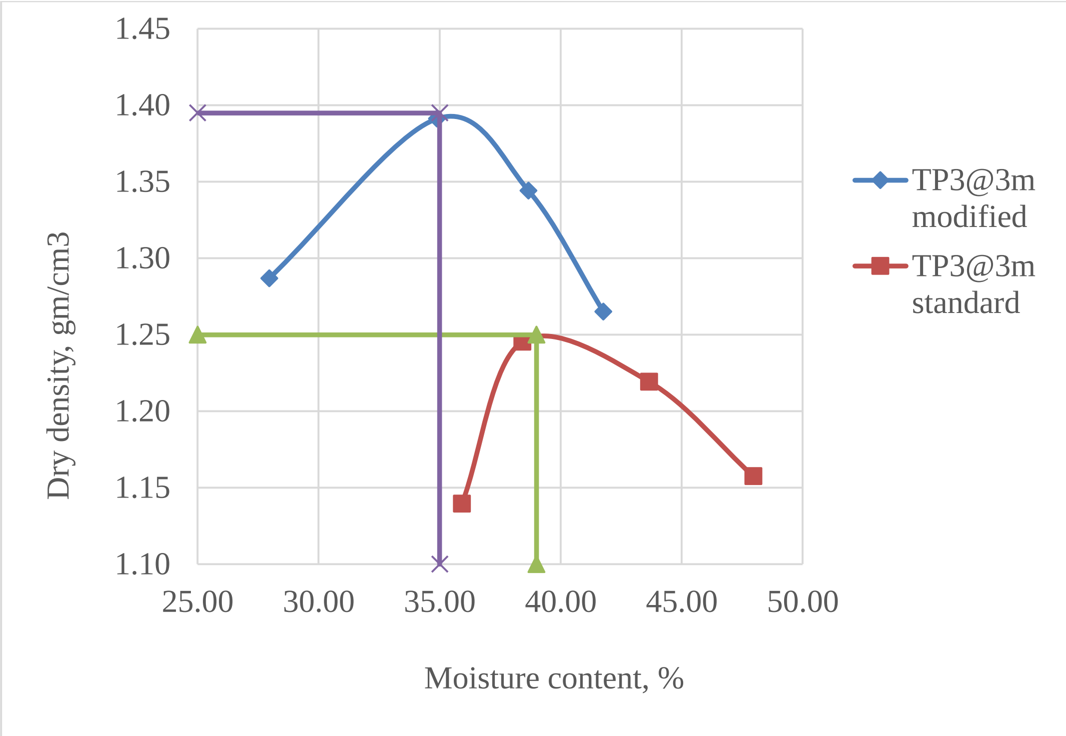
TP	TP2@1.5m					TP2@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10168.7	1.76	20.7	1.34	10229.3	1.79	27.3	1.41
	2	10413.6	1.88	25.1	1.41	10526.4	1.94	30.9	1.48
	3	10406.5	1.88	29.8	1.45	10463.3	1.91	34.0	1.42
	4	10168.7	1.76	38.1	1.36	10358.5	1.86	36.1	1.36
	OMC,%	29.5				30.9			
	MDD, g/cm^3	1.46				1.48			
Standard	1	5698	1.76	33.73	1.32	5701.7	1.76	35.9	1.30
	2	5790	1.86	36.91	1.36	5772.8	1.84	40.2	1.31
	3	5810	1.88	42.52	1.32	5730.8	1.79	43.0	1.25
	4	5765	1.83	47.21	1.24	5684.4	1.74	47.1	1.19
	OMC,%	36.5				40			
	MDD, g/cm^3	1.36				1.315			


1.50
1.45
1.40
1.35
1.30
1.25
1.20
Dry density,g/cm3
10.0
20.0
30.0
40.0
50.0
Water content, %
TP2@1.5m modified
TP2@1.5m standard

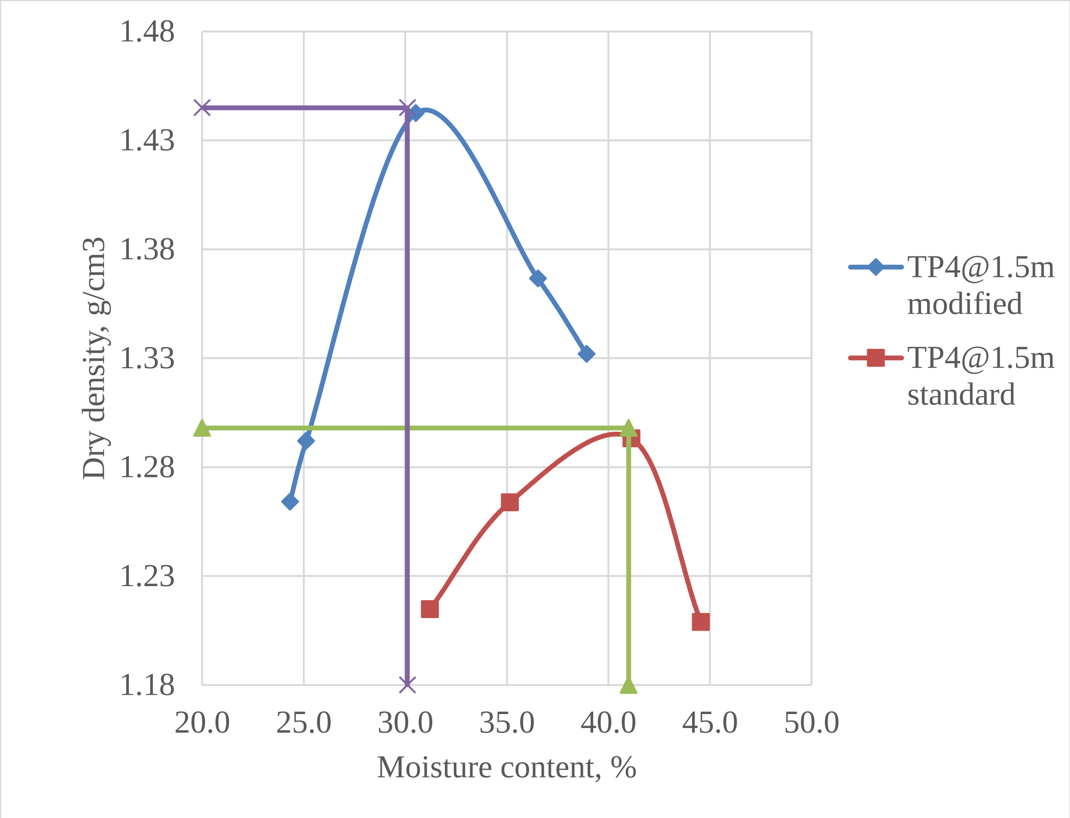

1.60
1.50
1.40
1.30
1.20
1.10
1.00
Dry density, g/cm3
20.0
30.0
40.0
50.0
Water content, %
TP2@3m modified
TP2@3m standard

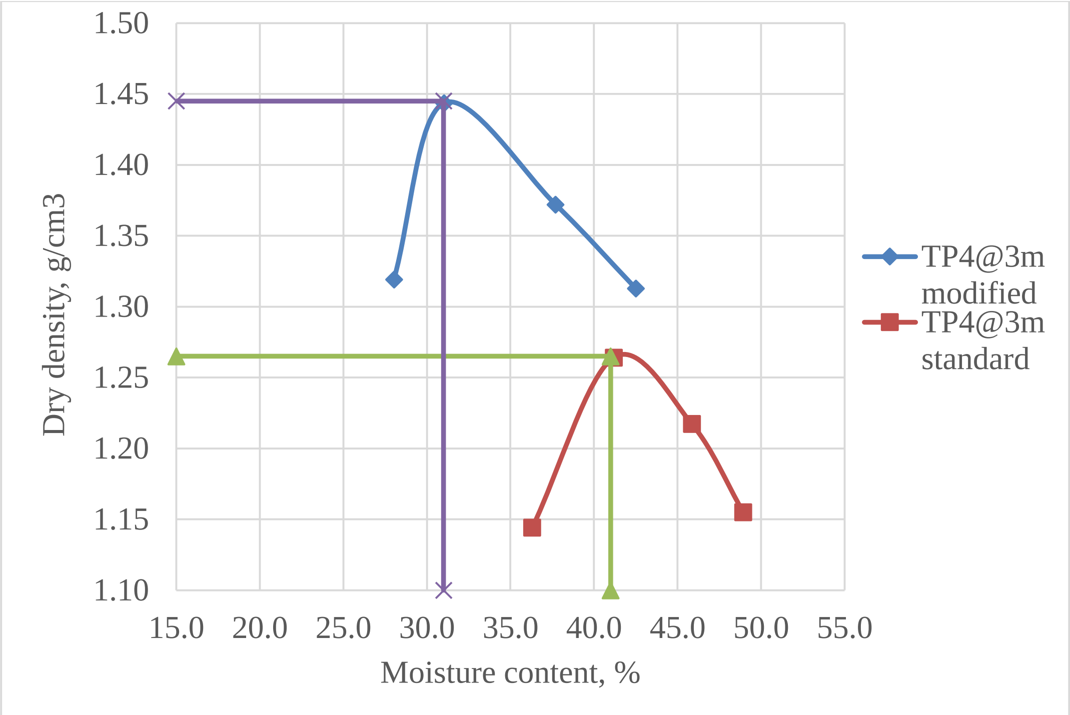
TP	TP3@1.5m					TP3@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	9980.5	1.67	24.5	1.34	10002.1	1.69	27.95	1.29
	2	10235.7	1.80	28.5	1.40	10476.3	1.89	34.89	1.39
	3	10450.6	1.90	32.9	1.43	10450.3	1.89	38.67	1.34
	4	10355.3	1.86	37.7	1.35	10305.1	1.87	41.76	1.27
	OMC,%	32				35			
	MDD, g/cm^3	1.433				1.395			
Standard	1	5687.0	1.75	34.4	1.30	5502.0	1.55	35.92	1.14
	2	5743.0	1.81	36.4	1.32	5665.4	1.72	38.41	1.25
	3	5820.5	1.89	39.9	1.35	5691.3	1.75	43.64	1.22
	4	5770.0	1.84	46.5	1.25	5655.0	1.71	47.96	1.16
	OMC,%	39				39			
	MDD, g/cm^3	1.352				1.25			



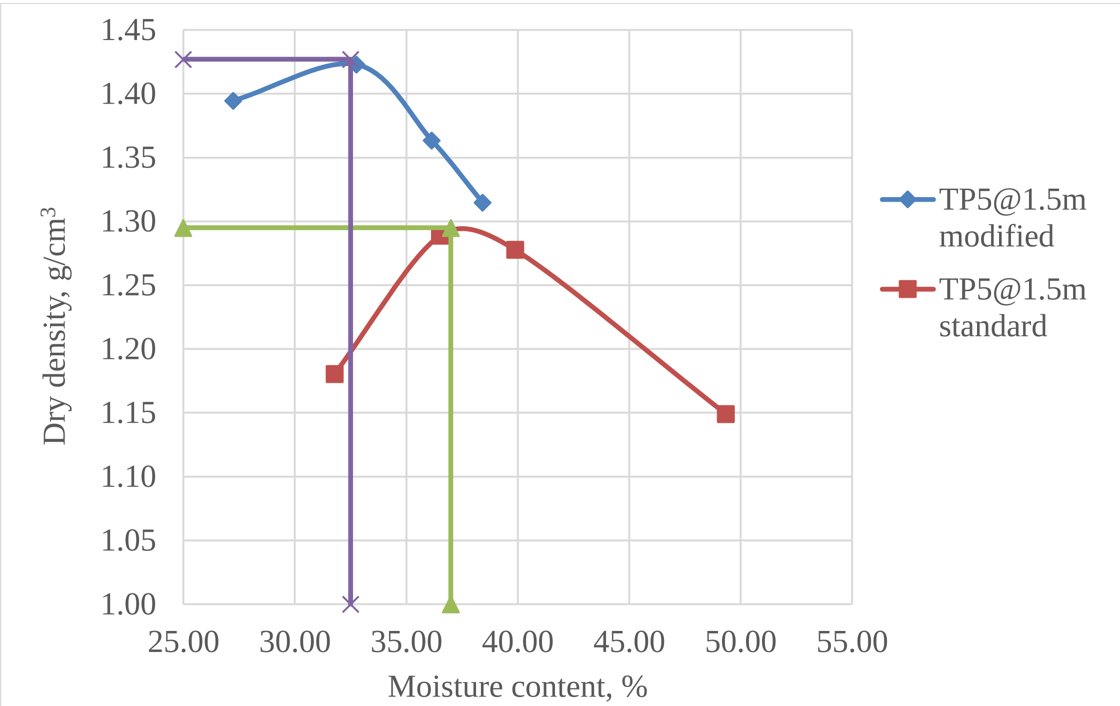


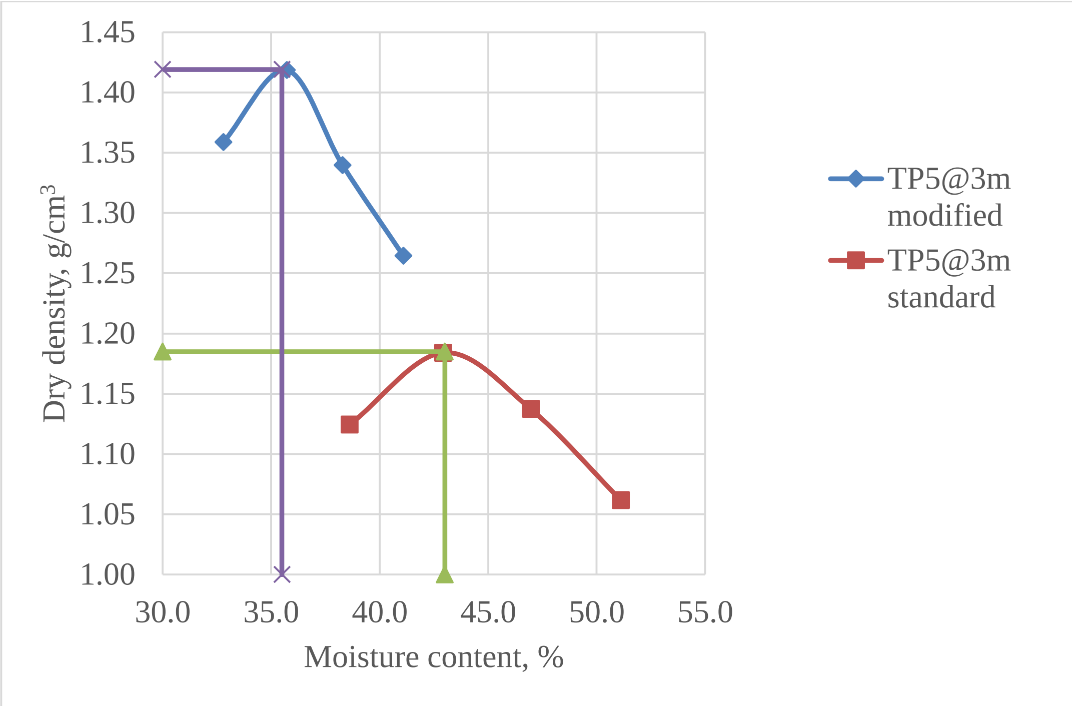
TP	TP4@1.5m					TP4@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	9771.3	1.57	24.3	1.26	10012.6	1.69	28.0	1.32
	2	9863.6	1.62	25.1	1.29	10429.7	1.89	31.0	1.44
	3	10412.3	1.88	30.5	1.44	10425.6	1.89	37.7	1.37
	4	10377.2	1.87	36.5	1.37	10387.7	1.87	42.5	1.31
	OMC,%	30.1				31			
	MDD, g/cm^3	1.445				1.445			
Standard	1	5544.0	1.59	31.21	1.21	5512.00	1.56	36.3	1.14
	2	5650.6	1.71	35.15	1.26	5722.30	1.78	41.2	1.26
	3	5760.0	1.83	41.14	1.29	5713.60	1.78	45.9	1.22
	4	5687.5	1.75	44.55	1.21	5662.20	1.72	48.9	1.16
	OMC,%	41				41			
	MDD, g/cm^3	1.298				1.265			


1.48
1.43
1.38
1.33
1.28
1.23
1.18
Dry density, g/cm3
20.0
25.0
30.0
35.0
40.0
45.0
50.0
Moisture content, %
TP4@1.5m modified
TP4@1.5m standard

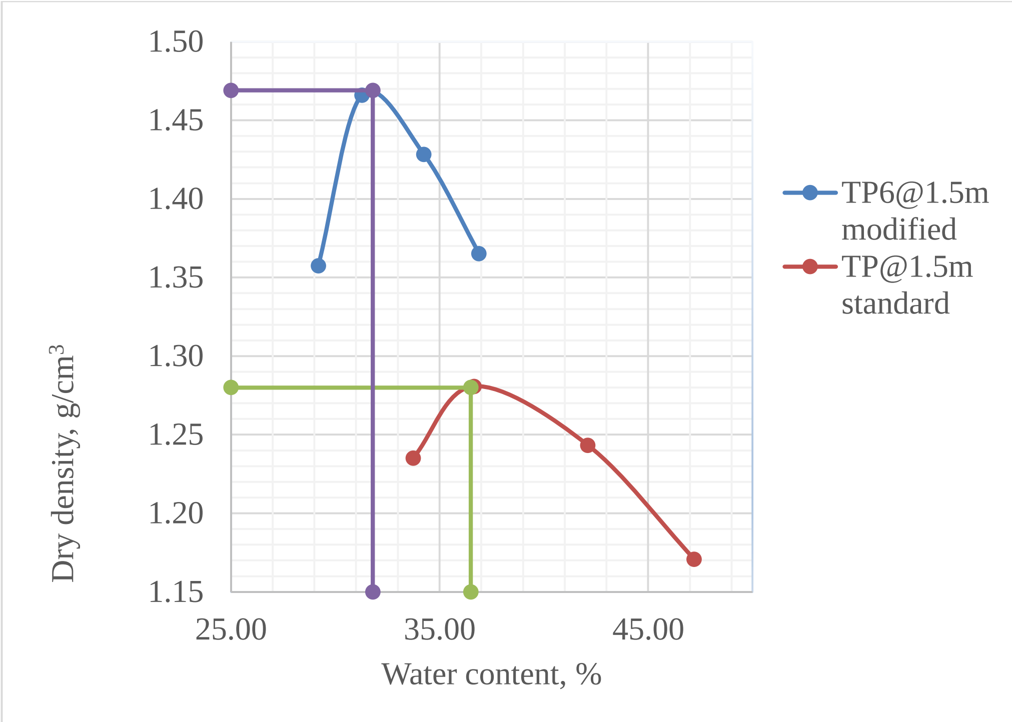

1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
1.10
Dry density, g/cm3
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
Moisture content, %
TP4@3m modified
TP4@3m standard

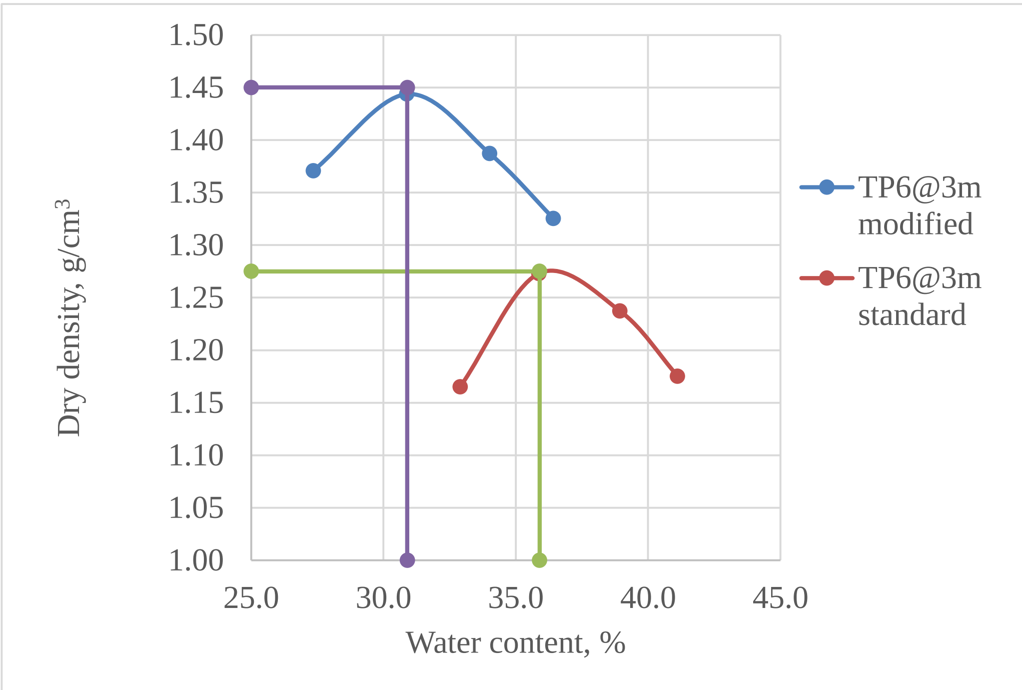
TP	TP5@1.5m					TP5@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10265.8	1.77	27.24	1.39	10250.6	1.80	32.8	1.36
	2	10501.7	1.89	32.76	1.42	10499.6	1.93	35.7	1.42
	3	10434.3	1.86	36.14	1.36	10350.4	1.85	38.3	1.34
	4	10359.6	1.82	38.42	1.31	10208.8	1.78	41.1	1.26
	OMC,%	32.5				35.5			
	MDD, g/cm^3	1.427				1.419			
Standard	1	5508.4	1.56	31.78	1.18	5511.2	1.56	38.6	1.12
	2	5698.6	1.76	36.51	1.29	5635.9	1.69	42.9	1.18
	3	5724.8	1.79	39.89	1.28	5616.5	1.67	47.0	1.14
	4	5658.0	1.72	49.34	1.15	5553.9	1.60	51.1	1.06
	OMC,%	37				43			
	MDD, g/cm^3	1.295				1.185			



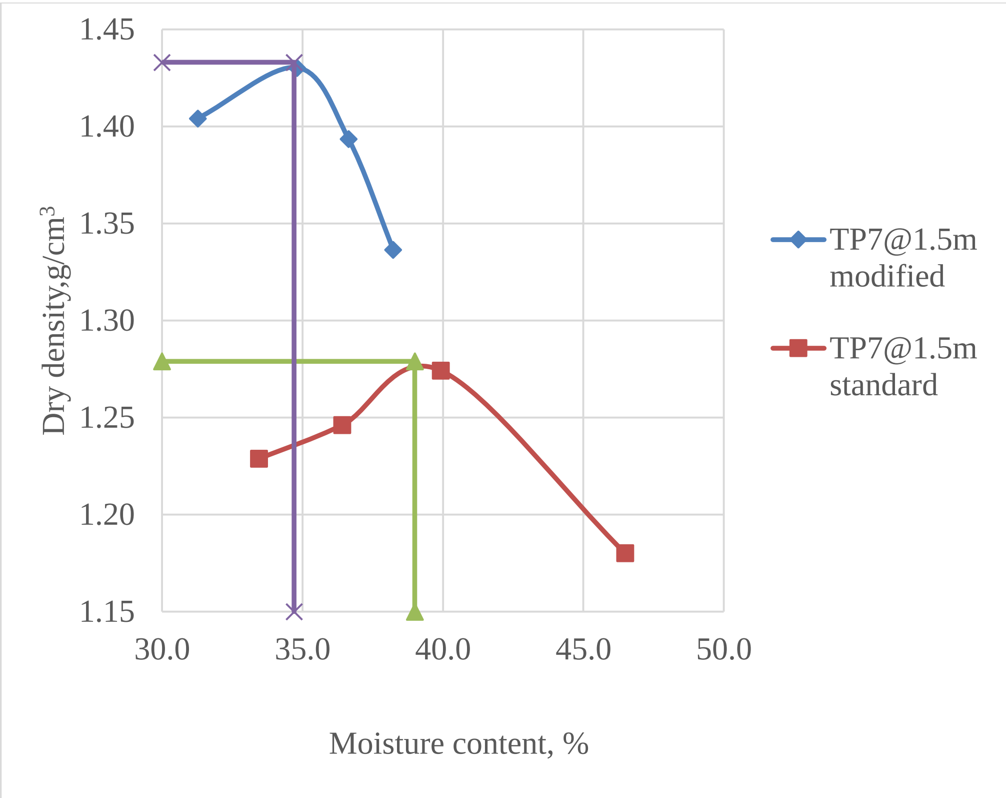


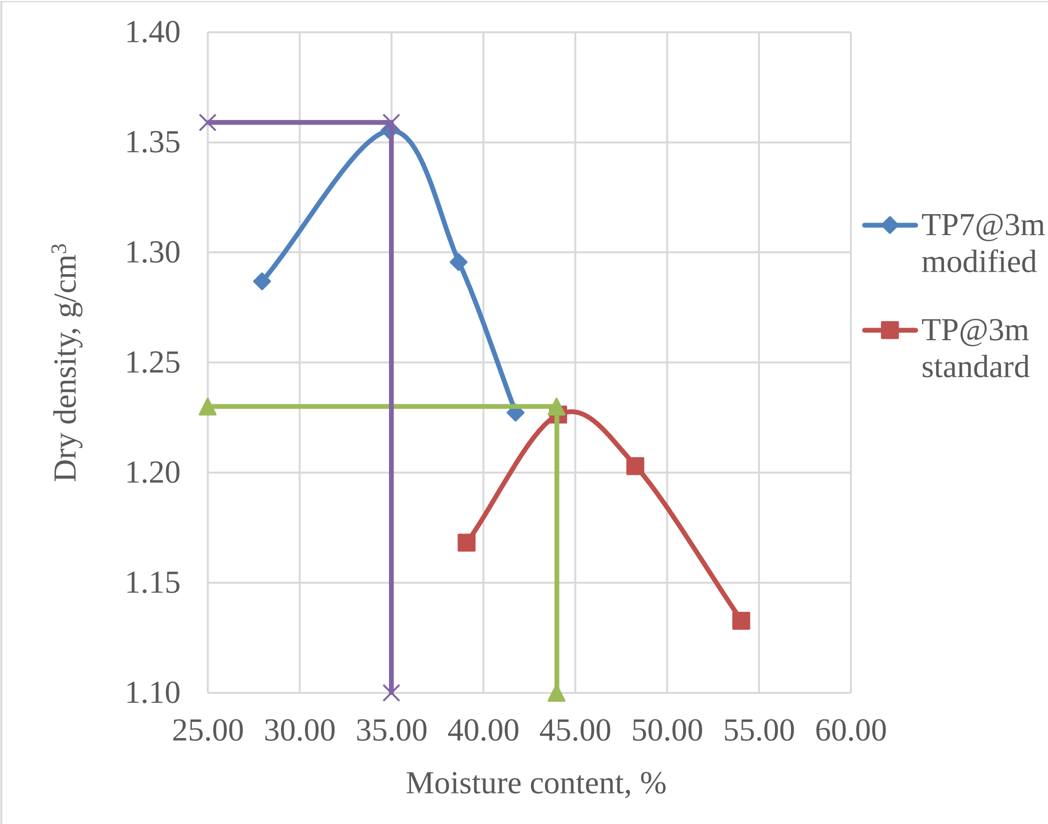
TP	TP6@1.5m					TP6@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10223.1	1.75	29.20	1.36	10129.3	1.75	27.3	1.37
	2	10575	1.92	31.28	1.47	10426.4	1.89	30.9	1.44
	3	10560.5	1.88	34.25	1.43	10363.3	1.86	34.0	1.39
	4	10460.3	1.87	36.89	1.37	10258.5	1.81	36.4	1.33
	OMC,%	31.8				30.9			
	MDD, g/cm^3	1.469				1.45			
Standard	1	5598	1.65	33.73	1.24	5501.7	1.55	32.9	1.17
	2	5690	1.75	36.66	1.28	5670.8	1.73	35.9	1.27
	3	5710	1.77	42.11	1.25	5660.8	1.72	38.9	1.24
	4	5665	1.72	47.21	1.17	5604.4	1.66	41.1	1.18
	OMC,%	36.5				35.9			
	MDD, g/cm^3	1.28				1.275			


1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
Dry density, g/cm3
25.00
35.00
45.00
Water content, %
TP6@1.5m modified
TP@1.5m standard

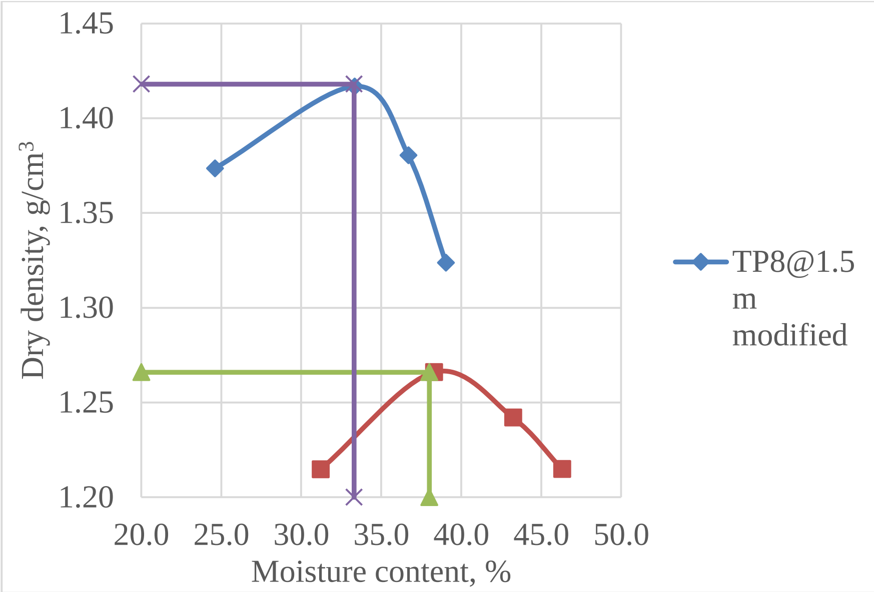

1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
Dry density, g/cm3
25.0
30.0
35.0
40.0
45.0
Water content, %
TP6@3m modified
TP6@3m standard

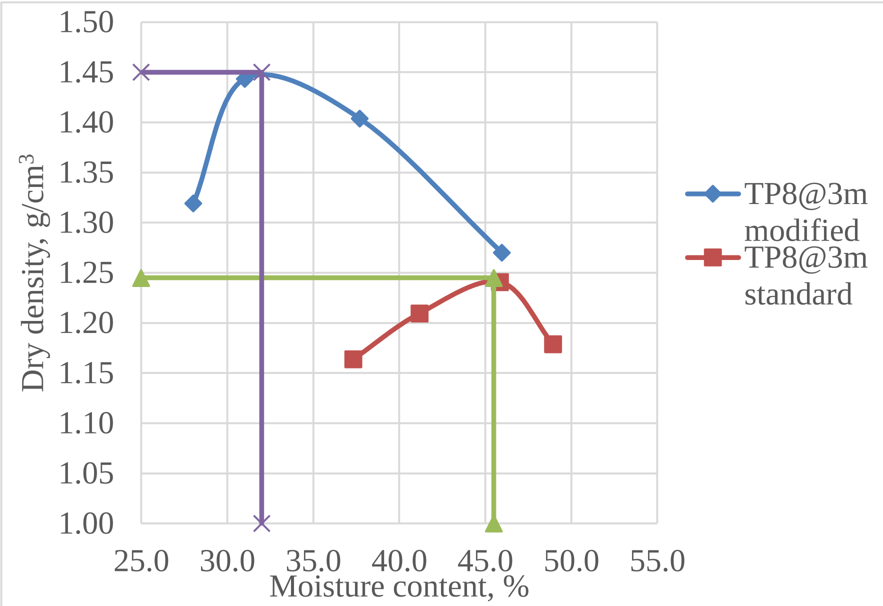
TP	TP7@1.5m					TP7@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10330.4	1.84	31.3	1.40	10002.1	1.65	27.95	1.29
	2	10505.2	1.93	34.8	1.43	10376.3	1.83	34.89	1.36
	3	10455.8	1.90	36.6	1.39	10311.3	1.80	38.67	1.30
	4	10339.1	1.85	38.2	1.34	10194	1.74	41.76	1.23
	OMC,%	34.7				35			
	MDD, g/cm^3	1.433				1.359			
Standard	1	5587.0	1.64	33.5	1.23	5573.0	1.62	39.10	1.17
	2	5643.0	1.70	36.4	1.25	5705.4	1.77	44.07	1.23
	3	5720.5	1.78	39.9	1.27	5721.3	1.78	48.28	1.20
	4	5670.0	1.73	46.5	1.18	5685.0	1.74	54.05	1.13
	OMC,%	39				44			
	MDD, g/cm^3	1.359				1.23			





TP	TP8@1.5m					TP8@3m			
Method	Trial	MCS	BD	W,%	DD	MCS	BD	W,%	DD
Modified	1	10058.8	1.71	24.6	1.37	10012.6	1.69	28.0	1.32
	2	10424.7	1.89	33.3	1.42	10429.7	1.89	31.0	1.44
	3	10420.5	1.89	36.7	1.38	10515.6	1.93	37.7	1.40
	4	10324.9	1.84	39.0	1.32	10352.7	1.85	46.0	1.27
	OMC,%	33.3				32			
	MDD, g/cm^3	1.418				1.445			
Standard	1	5544.0	1.59	31.21	1.21	5548.00	1.60	37.3	1.16
	2	5690.6	1.75	38.29	1.27	5650.30	1.71	41.2	1.21
	3	5717.0	1.78	43.24	1.24	5745.60	1.81	45.9	1.24
	4	5715.5	1.78	46.30	1.21	5695.20	1.76	48.9	1.18
	OMC,%	38				45.5			
	MDD, g/cm^3	1.266				1.245			


1.45
1.40
1.35
1.30
1.25
1.20
Dry density, g/cm^3
20.0 25.0 30.0 35.0 40.0 45.0 50.0
Moisture content, %
TP8@1.5 m modified


1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
Dry density, g/cm^3
25.0 30.0 35.0 40.0 45.0 50.0 55.0
Moisture content, %
TP8@3m modified
TP8@3m standard

Appendix-IV Unconfined Confined Strength

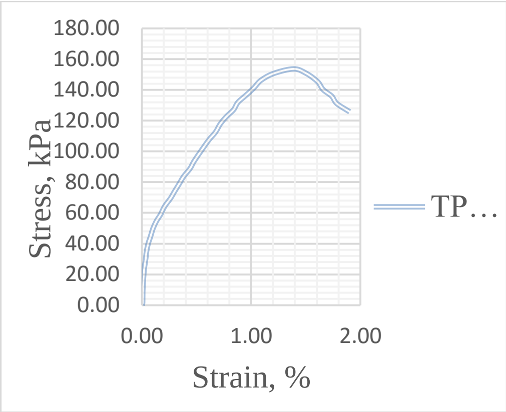
TP1@3m				
Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain %	Stress (kPa)
0.00	0.00	77.50	0.00	0.00
9.20	0.01	77.50	0.01	9.05
17.87	0.01	77.50	0.01	17.56
23.82	0.02	77.50	0.02	23.41
35.19	0.03	77.50	0.04	34.57
45.47	0.06	77.50	0.08	44.66
55.76	0.10	77.50	0.13	54.74
65.51	0.16	77.50	0.21	64.25
75.25	0.23	77.50	0.30	73.75
85.54	0.30	77.50	0.38	83.75
95.28	0.37	77.50	0.47	93.21
100.69	0.41	77.50	0.52	98.45
105.57	0.44	77.50	0.57	103.18
115.31	0.52	77.50	0.67	112.58
125.60	0.60	77.50	0.77	122.51
135.34	0.68	77.50	0.88	131.86
145.08	0.79	77.50	1.02	141.15
150.50	0.84	77.50	1.09	146.32
158.62	**1.08**	**77.50**	**1.39**	**153.75**
150.50	1.25	77.50	1.61	145.56
140.75	1.35	77.50	1.74	135.94
130.47	1.47	77.50	1.90	125.80

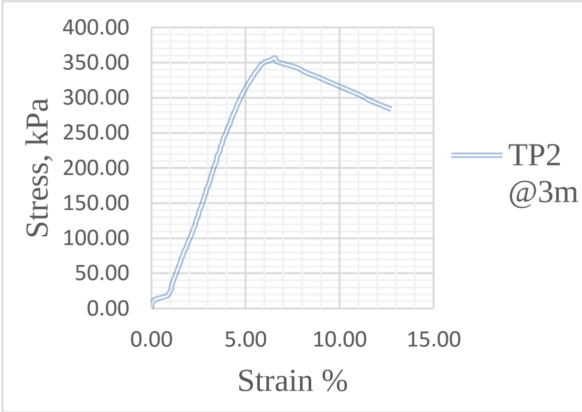
TP2@3m					**TP3@1.5m**				
Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)	Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)
0.00	0.00	77.50	0.00	0.00	0.00	0.00	77.50	0.00	0.00
8.66	0.05	77.50	0.06	8.51	11.91	0.05	77.50	0.06	11.70
28.15	0.82	77.50	1.05	27.38	21.65	0.53	77.50	0.68	21.14
48.72	1.00	77.50	1.29	47.28	30.86	0.60	77.50	0.77	30.10
68.21	1.20	77.50	1.54	66.01	41.14	0.66	77.50	0.85	40.10
88.78	1.42	77.50	1.83	85.67	51.43	0.71	77.50	0.92	50.09
108.81	1.64	77.50	2.12	104.69	60.63	0.78	77.50	1.00	59.00
128.30	1.83	77.50	2.36	123.14	71.46	0.83	77.50	1.07	69.49
149.42	2.02	77.50	2.61	143.04	81.20	0.89	77.50	1.15	78.90
168.90	2.20	77.50	2.84	161.30	91.49	0.96	77.50	1.24	88.81
188.93	2.40	77.50	3.09	179.96	101.78	1.02	77.50	1.32	98.72
208.96	2.56	77.50	3.30	198.62	111.52	1.09	77.50	1.41	108.07
229.00	2.71	77.50	3.50	217.22	121.26	1.14	77.50	1.47	117.44
248.48	2.92	77.50	3.77	235.03	131.55	1.22	77.50	1.58	127.27
267.97	3.11	77.50	4.01	252.84	141.29	1.66	77.50	2.14	135.90
288.00	3.31	77.50	4.27	271.01	151.58	1.73	77.50	2.23	145.67
308.58	3.54	77.50	4.56	289.48	161.33	1.84	77.50	2.38	154.81
328.61	3.78	77.50	4.88	307.25	171.61	1.98	77.50	2.56	164.37
348.64	4.08	77.50	5.26	324.67	181.90	2.20	77.50	2.83	173.73
368.12	4.40	77.50	5.68	341.29	**191.10**	**2.55**	**77.50**	**3.29**	**181.67**
388.70	**5.10**	**77.50**	**6.58**	**356.92**	181.90	2.70	77.50	3.48	172.57
369.00	6.76	77.50	8.72	331.06	170.53	2.77	77.50	3.58	161.62
348.39	8.55	77.50	11.03	304.68	161.87	2.83	77.50	3.65	153.30
330.95	9.88	77.50	12.75	283.83	151.04	2.88	77.50	3.72	142.94

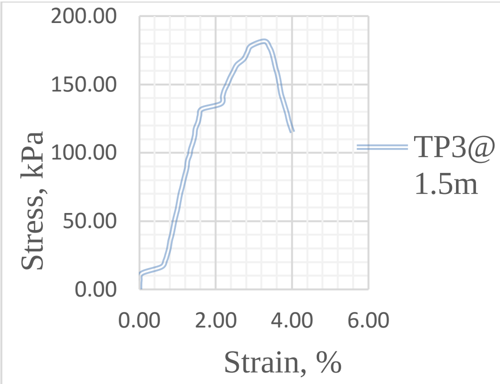
	TP3@3m					TP4@3m			
Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)	Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)
0.00	0.00	77.50	0.00	0.00	1.62	0.00	77.50	0.00	1.59
20.03	0.61	77.50	0.79	19.53	25.44	0.09	77.50	0.11	24.98
40.60	0.92	77.50	1.19	39.43	40.60	0.27	77.50	0.35	39.77
60.63	1.34	77.50	1.73	58.57	60.09	0.48	77.50	0.62	58.70
80.66	1.74	77.50	2.25	77.50	80.12	0.69	77.50	0.89	78.05
100.15	2.12	77.50	2.74	95.75	100.15	0.90	77.50	1.16	97.30
120.72	2.50	77.50	3.22	114.84	120.18	1.14	77.50	1.47	116.39
140.21	2.88	77.50	3.72	132.69	140.21	1.42	77.50	1.83	135.30
160.24	3.23	77.50	4.16	150.95	160.24	1.68	77.50	2.16	154.10
180.81	3.66	77.50	4.72	169.34	180.27	1.96	77.50	2.52	172.72
200.30	4.08	77.50	5.26	186.53	200.30	2.29	77.50	2.95	191.07
220.33	4.57	77.50	5.89	203.81	220.33	2.61	77.50	3.37	209.27
240.91	5.10	77.50	6.58	221.21	240.91	3.07	77.50	3.97	227.41
260.39	5.68	77.50	7.33	237.20	260.94	3.59	77.50	4.63	244.61
280.42	6.43	77.50	8.30	252.76	**277.18**	**4.54**	**77.50**	**5.85**	**256.50**
300.45	7.39	77.50	9.53	267.17	255.48	5.02	77.50	6.47	234.86
323.19	**9.54**	**77.50**	**12.31**	**278.57**	251.14	5.12	77.50	6.60	230.56
318.86	10.23	77.50	13.20	272.06	241.00	5.21	77.50	6.72	227.32

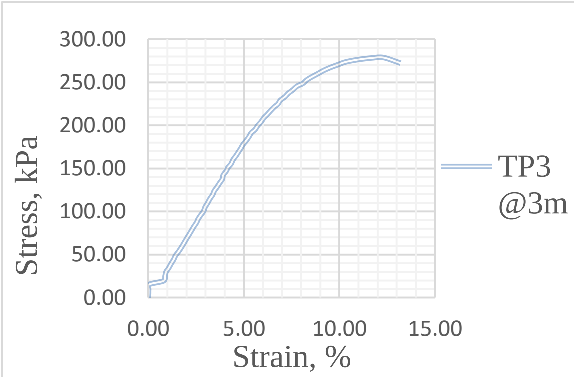
TP5@3m					TP6@1.5m				
Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)	Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain %	Stress (kPa)
0.00	0.00	77.50	0.00	0.00	1.08	0.00	77.50	0.01	1.06
15.70	0.01	77.50	0.01	15.43	10.83	0.02	77.50	0.02	10.64
30.32	0.16	77.50	0.21	29.74	20.57	0.08	77.50	0.10	20.20
45.47	0.30	77.50	0.38	44.53	30.32	0.19	77.50	0.25	29.72
60.09	0.48	77.50	0.62	58.70	40.60	0.31	77.50	0.40	39.75
75.79	0.64	77.50	0.82	73.89	50.89	0.44	77.50	0.57	49.74
90.95	0.79	77.50	1.01	88.49	60.63	0.54	77.50	0.69	59.18
105.57	0.97	77.50	1.25	102.48	70.92	0.67	77.50	0.86	69.11
120.18	1.14	77.50	1.47	116.39	80.12	0.79	77.50	1.01	77.96
135.34	1.33	77.50	1.72	130.74	90.41	0.91	77.50	1.17	87.82
149.96	1.54	77.50	1.99	144.47	100.15	1.04	77.50	1.34	97.12
165.11	1.75	77.50	2.25	158.63	110.44	1.20	77.50	1.55	106.88
180.81	1.98	77.50	2.55	173.20	120.72	1.35	77.50	1.74	116.60
195.43	2.21	77.50	2.85	186.63	130.47	1.50	77.50	1.94	125.75
210.05	2.42	77.50	3.12	200.02	140.21	1.69	77.50	2.18	134.81
225.75	2.73	77.50	3.52	214.09	150.50	2.93	77.50	3.78	142.34
240.91	3.06	77.50	3.95	227.46	**152.12**	**3.11**	**77.50**	**4.01**	**143.52**
255.52	3.45	77.50	4.46	239.97	147.79	3.20	77.50	4.13	139.27
270.14	3.92	77.50	5.06	252.10	142.92	3.34	77.50	4.31	134.42
277.18	**4.45**	**77.50**	**5.74**	**256.80**	132.09	3.74	77.50	4.83	123.57
274.47	4.56	77.50	5.89	253.91	122.35	3.94	77.50	5.09	114.15
269.54	4.61	77.50	5.95	249.17	120.72	3.96	77.50	5.10	112.60
264.75	4.70	77.50	6.06	244.46	115.33	3.97	77.50	5.12	110.32

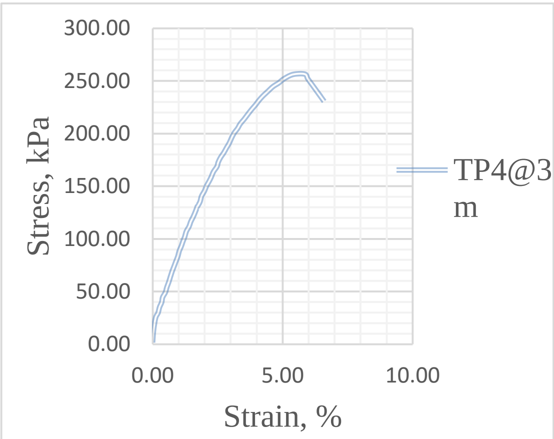
TP6@3m					**TP8@3m**				
Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain, %	Stress (kPa)	Load (N)	Sample Def. ΔL (mm)	ho (mm)	Strain , %	Stress (kPa)
0.00	0.00	77.50	0.00	0.00	0.54	0.00	77.50	0.00	0.53
5.41	0.36	77.50	0.46	5.30	15.70	0.47	77.50	0.61	15.34
9.74	0.39	77.50	0.50	9.53	20.03	0.56	77.50	0.72	19.55
20.03	0.47	77.50	0.60	19.57	30.86	0.75	77.50	0.96	30.04
30.32	0.58	77.50	0.75	29.57	40.06	0.91	77.50	1.17	38.92
40.06	0.70	77.50	0.91	39.02	50.89	1.12	77.50	1.45	49.30
50.35	0.86	77.50	1.11	48.94	60.09	1.27	77.50	1.64	58.10
60.09	1.00	77.50	1.29	58.30	65.51	1.37	77.50	1.77	63.25
70.38	1.20	77.50	1.55	68.10	70.92	1.46	77.50	1.89	68.39
80.66	1.38	77.50	1.78	77.87	80.12	1.65	77.50	2.13	77.08
90.41	1.62	77.50	2.09	87.00	90.95	1.87	77.50	2.41	87.24
100.15	1.83	77.50	2.36	96.12	100.15	2.07	77.50	2.67	95.81
110.44	2.05	77.50	2.64	105.69	110.98	2.34	77.50	3.01	105.80
120.72	2.29	77.50	2.95	115.16	120.18	2.55	77.50	3.29	114.24
130.47	2.56	77.50	3.30	124.01	125.60	2.69	77.50	3.47	119.18
140.21	2.83	77.50	3.65	132.79	130.47	2.82	77.50	3.64	123.57
150.50	3.11	77.50	4.01	142.00	135.34	2.96	77.50	3.82	127.95
160.24	3.46	77.50	4.47	150.47	140.21	3.11	77.50	4.01	132.30
170.53	3.84	77.50	4.95	159.31	145.08	3.29	77.50	4.25	136.55
180.81	4.27	77.50	5.51	167.94	150.50	3.52	77.50	4.55	141.21
190.02	4.75	77.50	6.13	175.33	155.37	3.78	77.50	4.88	145.27
200.84	5.33	77.50	6.88	183.84	160.24	4.05	77.50	5.22	149.28
210.05	6.01	77.50	7.75	190.46	165.11	4.32	77.50	5.58	153.24
215.46	**6.60**	**77.50**	**8.52**	**193.75**	170.53	4.53	77.50	5.85	157.81
208.42	8.43	77.50	10.87	182.59	**176.48**	**5.50**	**77.50**	**7.09**	**161.16**
198.68	8.67	77.50	11.19	173.45	166.20	6.53	77.50	8.43	149.60

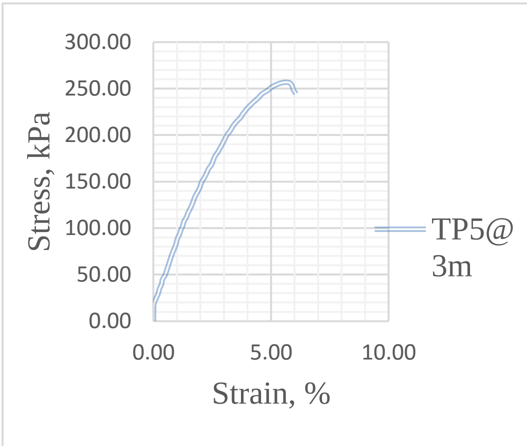

Stress, kPa
Strain, %
TP…

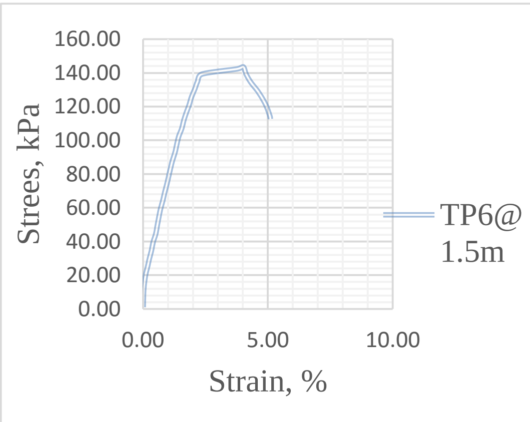

Stress, kPa
Strain %
TP2 @3m

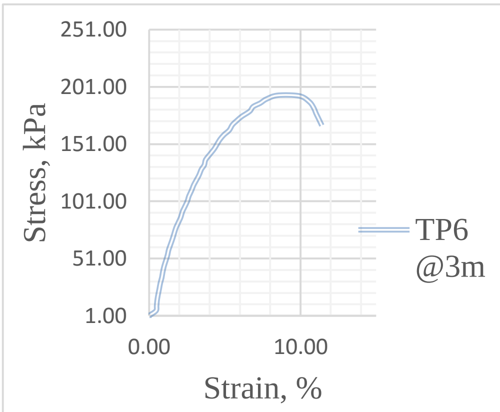

Stress, kPa
Strain, %
TP3@ 1.5m

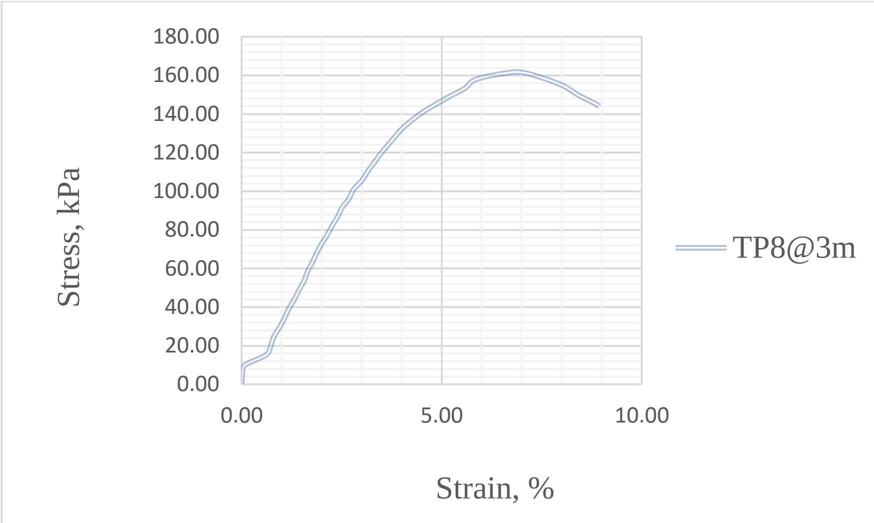

Stress, kPa
Strain, %
TP3 @3m


Stress, kPa
Strain, %
TP4@3 m


Stress, kPa
Strain, %
TP5@ 3m


Strees, kPa
Strain, %
TP6@ 1.5m


Stress, kPa
Strain, %
TP6 @3m


Stress, kPa
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.00
5.00
10.00
Strain, %
TP8@3m

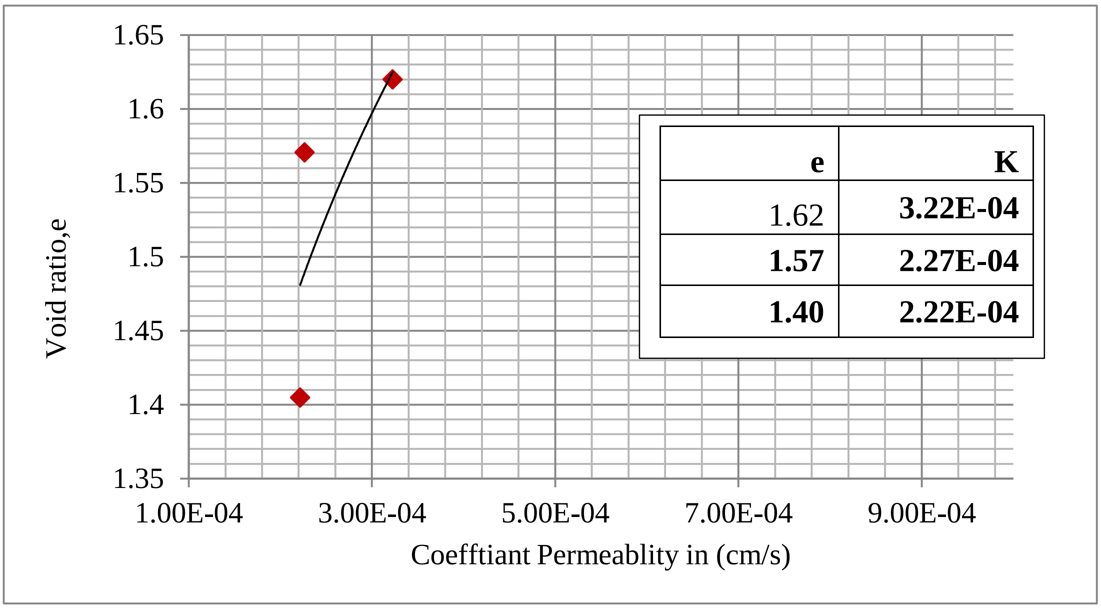

UTEST

Appendix-V Permeability

Pit 2 at 3m depth for 10 No of compaction						
Specimen Data			**Falling head data sheet**			
Specimen Mass (g)	1088		Trial	01	02	03
Specimen Height, L(cm)	11.14		Head, h_o (cm)	99	93.8	94
Specimen diameter, D (cm)	10.16		Head,h_1 (cm)	70	70.3	65.5
Bulk density, g (g/cm^2)	1.21		Time, t (s)	25.56	20.66	28.5
Water Content, w	31.35		Temperature, T (^{o}c)	21	21	21
Dry density, g_{dry} (g/cm2)	**1.00**		Volume, (ml)	54	51	57
Average collected volume (cm3)	54		Height dropped (cm)	29	23.5	28.5
Average of height dropped (cm)	27.00		Permeability at T^{o}c, K_T	3.34E-04	3.44E-04	3.12E-04
Initial void ratio, e	**1.74**		R_t for T	0.9761	0.9761	0.9761
Cross-sectional area of stand pipe, a (cm^2)	2.00		Permeability at 20^{o}C, K_{20}	3.26E-04	3.36E-04	3.05E-04
Cross-sectional area of soil specimen, A (cm^2)	81.03		**Average K_{20} (cm/s)**	**3.22E-04**		

Pit 2 at 3m depth for 20 No of compaction						
Specimen Data			**Falling head data sheet**			
Specimen Mass (g)	1160		Trial	01	02	03
Specimen Height, L(cm)	11.14		Head, h_o (cm)	99.6	85.8	91.6
Specimen diameter, D (cm)	10.16		Head,h_1 (cm)	70.5	70.3	68.7
Bulk density, g (g/cm^2)	1.29		Time, t (s)	47.65	42.23	43.2
Water Content, w	31.35		Temperature, T (^{o}c)	20	20	20
Dry density, g_{dry} (g/cm2)	**1.06**		Volume, (ml)	78.8	57	64
Average collected volume (cm3)	66.6		Height dropped (cm)	29.1	15.5	22.9
Average of height dropped (cm)	22.50		Permeability at T^{o}c, K_T	2.65E-04	1.72E -04	2.43E-04
Initial void ratio, e	**1.57**		R_t for T	1.00	1.00	1.00
Cross-sectional area of stand pipe, a (cm^2)	2.96		Permeability at 20^{o}C, K_{20}	2.65E-04	1.72E -04	2.43E-04
Cross-sectional area of soil specimen, A (cm^2)	81.03		**Average K_{20} (cm/s)**	**2.27E-04**		

Pit 2 at 3m depth for 30 No of compaction						
Specimen Data			**Falling head data sheet**			
Specimen Mass (g)	1240		Trial	01	02	03
Specimen Height, L (cm)	11.14		Head, h_o (cm)	99.6	85.8	91.6
Specimen diameter, D (cm)	10.16		Head, h_1 (cm)	75.5	69.3	76.7
Bulk density, g (g/cm^2)	1.37		Time, t (s)	47.65	42.23	43.2
Water Content, w	31.35		Temperature, T (^{o}c)	20	20	20
Dry density, g_{dry} (g/cm2)	**1.14**		Volume, (ml)	78.8	57	64
Average collected volume (cm3)	66.6		Height dropped (cm)	24.1	16.5	14.9
Average of height dropped (cm)	18.50		Permeability at T^{o}c, K_T	2.58E-04	2.24E-04	1.82E-04
Initial void ratio, e	**1.40**		R_t for T	1.0000	1.0000	1.0000
Cross-sectional area of stand pipe, a(cm^2)	3.60		Permeability at 20^{o}C, K_{20}	2.58E-04	2.24E-04	1.82E-04
Cross-sectional area of soil specimen, A (cm^2)	81.03		**Average K_{20} (cm/s)**	**2.22E-04**		



Pit 4 at 3m depth for 10 No of compaction						
Specimen Data			**Falling head data sheet**			
Specimen Mass (g)	1094		Trial	01	02	03
Specimen Height, L (cm)	11.14		Head, h_o (cm)	93.5	93	90.8
Specimen diameter, D (cm)	10.16		Head, h_1 (cm)	59.5	64.5	66.1
Bulk density, g (g/cm^2)	1.21		Time, t (s)	28.74	23.14	20.33
Water Content, w	31.35		Temperature, T (^{o}c)	22	22	22
Dry density, g_{dry} (g/cm2)	**1.00**		Volume, (ml)	68	62	54
Average collected volume (cm3)	61.33		Height dropped (cm)	34	28.5	24.7
Average of height dropped (cm)	29.07		Permeability at T^{o}c, K_T	4.09E-04	4.11E-04	4.06E-04
Initial void ratio, e	**1.73**		R_t for T	0.9531	0.9531	0.9531
Cross-sectional area of stand pipe, a (cm^2)	2.11		Permeability at 20^{o}C, K_{20}	3.90E-04	3.92E-04	3.87E-04
Cross-sectional area of soil specimen, A (cm^2)	81.03		**Average K_{20} (cm/s)**	**3.90E-04**		

Pit 4 at3m depth for 20 No of compaction						
Specimen Data			**Falling head data sheet**			
Specimen Mass (g)	1179.5		Trial	01	02	03
Specimen Height, L (cm)	11.14		Head, h_o (cm)	90	85	88.1
Specimen diameter, D (cm)	10.16		Head, h_1 (cm)	71	66.2	71
Bulk density, g (g/cm^2)	1.31		Time, t (s)	118.77	144.42	109.67
Water Content, w	31.35		Temperature, T (^{o}c)	23	23	23
Dry density, g_{dry} (g/cm2)	**1.08**		Volume, (ml)	29	34	27
Average collected volume (cm3)	30		Height dropped (cm)	19	18.8	17.1
Average of height dropped (cm)	18.30		Permeability at T^{o}c, K_T	4.03E-05	3.50E-05	3.98E-05
Initial void ratio, e	**1.53**		R_t for T	0.931	0.931	0.9311
Cross-sectional area of stand pipe, a (cm^2)	1.64		Permeability at 20^{o}C, K_{20}	3.76E-05	3.26E-05	3.70E-05
Cross-sectional area of soil specimen, A (cm^2)	81.03		**Average K_{20} (cm/s)**	**3.57E-05**		

Pit 4 at 3m depth for 30 No of compaction					
Specimen Data		**Falling head data sheet**			
Specimen Mass (g)	1190	Trial	01	02	03
Specimen Height, L (cm)	11.14	Head, h_o (cm)	97.5	96.5	96
Specimen diameter, D (cm)	10.16	Head,h_1 (cm)	73.3	75.6	66.3
Bulk density, g (g/cm^2)	1.32	Time, t (s)	32.75	22.89	35.05
Water Content, w	31.35	Temperature, T(^{o}c)	21	21	21
Dry density, g_{dry} (g/cm2)	**1.09**	Volume, (ml)	60	46	65
Average collected volume (cm3)	57	Height dropped (cm)	24.2	20.9	29.7
Average of height dropped (cm)	24.93	Permeability at T^{o}c, K_T	2.45E-04	3.01E-04	2.98E-04
Initial void ratio, e	**1.51**	R_t for T	0.9761	0.9761	0.9761
Cross-sectional area of stand pipe, a(cm^2)	2.29	Permeability at 20^{o}C, K_{20}	2.40E-04	2.93E-04	2.90E-04
Cross-sectional area of soil specimen, A (cm^2)	81.03	**Average K_{20} (cm/s)**	**2.74E-04**