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

INVESTIGATION ON SOME OF THE ENGINEERING PROPERTIES OF SOILS FOUND IN HAROMAYA TOWN, ETHIOPIA

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“A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in civil engineering”

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APRIL, 2016

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DECLARATION

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

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ACKNOWLEDGEMENTS

First of all, I would like to express my special thanks to my advisor, Dr. Messele Haile, Assistant Professor at the School of Civil and Environmental Engineering, Addis Ababa University for his valuable advice and guidance as well as providing the necessary materials throughout the research work.

I forward my gratitude and appreciation to the laboratory personnel of the School of Civil and Environmental Engineering of the Addis Ababa University for their commitment to spend their time and render assistance in the laboratory works of this thesis.

I would like to forward my appreciation to Haromaya University, Institute of Technology, staff members' for their support and encouraging advice during collecting my sample.

Above all I am very thankful to **Almighty God** who is always with me in each and every step of my life. I thank God for everything He did and made education compatible with the rest of my life. Finally I would like to express my deepest gratitude to my parents, my friends and all who contributed to this research work in one way or another.

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SYMBOLS AND ABBREVIATIONS

AASHTO	-	American Association of Highway and Transportation Officials
AC	-	Activity Number
ASTM	-	American Society for Testing Materials standard
C	-	Cohesion
Cc	-	Compression Index
CD	-	Consolidated Drained
CH	-	Inorganic clay with high plasticity
CL	-	Inorganic clay with low to medium plasticity
CU	-	Consolidated- Undrained
C_V	-	Coefficient of consolidation
DSA	-	Direct Shear Apparatus
e	-	Void ratio
GI	-	Group Index
GIS	-	Geographical Information System
GPS	-	Global Positioning System
G_S	-	Specific Gravity
LL	-	Liquid Limit
MDD	-	Maximum Dry Density
OCR	-	Over -Consolidated Ratio
OD	-	Oven Dry
OMC	-	Optimum Moisture Content
Pc	-	Pre-consolidation pressure
PI	-	Plastic Index

P_o	-	Over- burden pressure
PRA	-	Public Roads Administration
Ps	-	Swelling pressure
SM	-	Silty sand
TP	-	Test Pit
UCS	-	Unconfined Compression Strength
UU	-	Unconsolidated-Undrained
γ	-	Total unit weight
ϕ	-	Angle of internal friction

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Abstract

As construction expands to news frontiers, frequent problems are being faced in the civil engineering structures such as cracking and break-up of pavements, highway,embankments, roadways, building foundations, slab-on-grade members and, channel and reservoir linings, irrigation systems, water lines, sewer lines. The problems are either due to not understanding of the behavior of the soil or lack of information on the engineering properties of the soil. Therefore, to obtained information on type, characteristics and distributions of soil, geotechnical investigations should be done on soil and rock underlying (and sometimes adjacent to) the site of the proposed structures.

The objective of this research is to investigate the engineering properties of soil found in Haromaya town. To achieve this objective ten sampling areas were selected. From the selected sampling areas, pits were excavated to a maximum depth of three meters and both disturbed and undisturbed samples were collected. Laboratory tests were done on the collected samples.

The laboratory test result indicates that the natural moisture content for CH soil ranges from 13.5-33.5%, specific gravity ranges from 2.7-2.84, Liquid Limits ranging from 52-72%, Plastic Limit ranges from 21.3-28.4% and plasticity index ranges from 25.9-45.5%, free swell ranges from 35-75%. The natural moisture content test results showed that for CL soil ranges from 11.5-27.6%, specific gravity ranges from 2.7-2.75, Liquid Limits ranging from 32-44.9%, Plastic Limit ranges from 8.25-22% and plasticity index ranges from 14.7-28%, free swell ranges from 30-75%. The natural moisture content test result indicates that for SM soil ranges from 10.5-25%, specific gravity ranges from 2.65-2.74, free swell ranges from 20-30%. The natural moisture content test results showed that for expansive soil ranges from 29-35.9%, specific gravity ranges from 2.75-2.8, Liquid Limits ranging from 86.4-87.3%, Plastic Limit ranges from 23-23.8% and plasticity index ranges from 63.3-63.4% free swell ranges from 105-113%.

The grain size analysis test results showed that the dominant proportion of soil particles on the research area was clay and sand. For the CH soil a clay content ranging from 36.1-65.75%, silt fraction 18.3-36.7%, sand fraction 6.88- 39.6% and gravel content from 0.0-6.9%. For the CL soil clay content ranging from 49.6-65.4%, silt fraction 19.9-34.1%, sand fraction 7.99-24% and

gravel content from 0.0-8.6%. Clay content for SM soil ranging from 1.6-10.3%, silt fraction 5.2-21.1%, sand fraction 67.7- 84.3% and gravel content from 0.9-9.0%. For the expansive soil clay content ranging from 50.2-50.5%, silt fraction 21.9-32.2%, sand fraction 9.7-10.0% and gravel content from 7.7-18.5%.

From the compaction test results the maximum dry density (MDD) of Haromaya soil ranges from 1.39 to 1.52 g/cm^3 and the optimum moisture content lies between 13 to 23.4 percent.

According to the Unified Soil Classification System, the soil is categorized as CH (high plastic clay soil), CL (low to medium plastic clay soil) and SM (silty sand). The AASHTO Classification System shows that most of soils of study areas categorized on A-7-6 and A-6 therefore those soils have poor quality for use as subgrade.

The unconfined compressive strength test result for CH soil ranges from 132.2kN/m^2 - 273.4kN/m^2 this indicates that consistency of the soil of the study area ranges between stiff to very stiff. The direct shear test result indicates that areas near Lake Haromaya are cohesionless soil material with angle of internal friction, $\phi = 27.3^0$ and cohesion, C = 2.35 (kN/m^2).

One-dimensional consolidation test result shows that the area under investigation is over consolidated in its natural state having compression index ranging from 0.13-0.33, pre-consolidation pressure 90-150kPa, swelling pressure ranging from 14-110kPa and over consolidation ratio ranges from 1.41-2.54.

Therefore it was found out that area Haromaya soil is mostly covered with red clay soil. Those materials are generally considered the poorest performers with regard to roadway construction, but can be utilized as sub-grade material.

1. INTRODUCTION

1.1 Background Information

Investigation of the underground soil conditions at a site is prerequisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies of the proposed project. Public building officials may require soil data together with the recommendations of the geotechnical consultant prior to issuing a building permit, particularly if there is a chance that the project will endanger the public health or safety or degrade the environment [6].

For foundation design and construction work, one must know the actual soil stratification at a given site, the laboratory test results of the soil samples obtained from various depths, and the observations made during the construction of other structures built under similar conditions. For most major structures, adequate subsoil exploration at the construction site must be conducted [3].

Haromaya is a town in East - central Ethiopia located in the Misraq Hararghe Zone of Oromia Regional State and 10km west of Harar,on the road to Dire Dawa. The town has one governmental university (Haramaya University main campus, Gendeje campus), technical college and many commercial buildings. In the town, many buildings are constructed and are under construction without adequate and detailed geotechnical investigation. Therefore, the objective of this research is to investigate some of the engineering properties of the soil, such as; the grain size analysis, Atterberg limits, specific gravity, consolidation, direct shear, unconfined compression strength, standard and modified compaction and free swell tests.

The location of Haromaya, on map of Ethiopia is shown in Fig. 1.1.

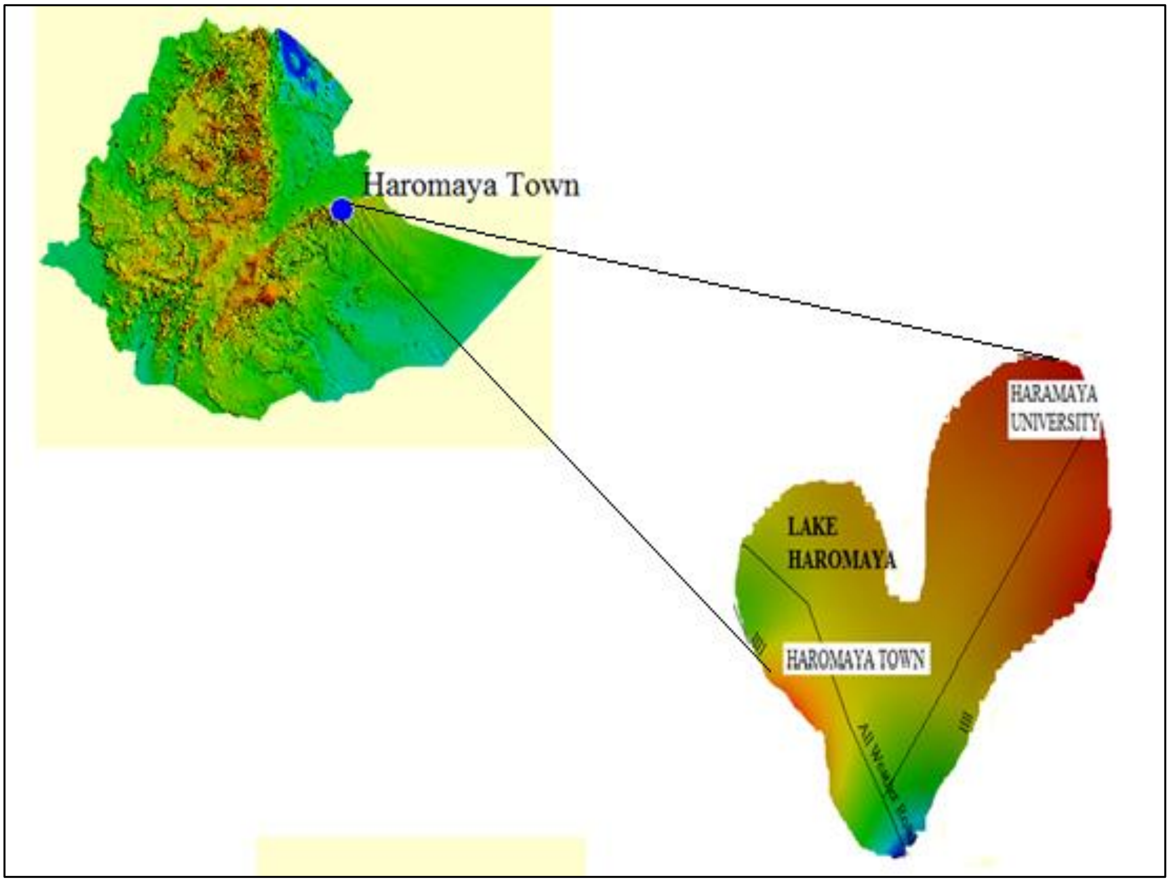

Figure 1.1: The location of Haromaya on Map of Ethiopia [17]

1.2 Statement of the Problem

Investigation of engineering properties of soil is essential for the economical design of sub structural elements as almost everything we build i.e. buildings, bridges, dams and others are in, on or within soils or rocks. It is also necessary to obtain sufficient information for feasibility and economic studies of the proposed project.

In a country like Ethiopia which is developing at high growth rate, many construction works are undergoing at present and will be done in the future. Geotechnical investigation on the engineering properties of soil is very essential, because these data are very important for civil engineers in preliminary design and in the design of foundation, pavement, retaining structures, etc for future construction projects in the country. Many researches were done and there are ongoing researches in most big cities of the country like Addis Ababa, Bahir Dar, Mekele, Hawassa, etc. However, the engineering property of the soil in Haromaya town has not yet been studied. This research is therefore directed to the study of the engineering properties of soils in Haromaya town.

1.3 Objective

1.3.1 General Objective

The major objective, of the research is to investigate engineering properties of soil found in Haromaya Town.

1.3.2 Specific objectives

The specific objectives are:

- to investigate some of the engineering properties of soil in Haromaya town like: natural moisture content, specific gravity, grain size analysis, free swell, shear strength, consolidation and compaction characteristics.
- to determine the range of values of index properties of soil in different parts of the Town.
- to prepare geotechnical map of the town.

1.4 Research Methodology

To achieve the above mentioned objectives ten sampling areas were selected. From the selected sampling areas pits were excavated to a maximum depth of three meters. From the excavated pits both disturbed and undisturbed samples were collected for laboratory testing to evaluate classification indices and estimate relevant engineering properties. Undisturbed soil samples were collected for unique soil type obtained in the field.

In the field GPS reading was taken to locate the ordinate of sampling area. Table 1.1 shows the global coordinates of sampling location, while Figure 1.1 shows the sampling points distribution.

Table 1.1: Global coordinates of sampling areas

Location	Designation	Latitude	Longitude	Elevation
(Kebele- 03) Gendeje Campus	TP-1	09˚24'43.5"	042˚02'26.7"	2130m
(Kebele -03) Bate	TP-2	09˚25'02.6"	042˚02'37.3"	2137m
Kebele-01 (Condominium)	TP-3	09˚23'34.6"	042˚01'18.4"	2089m
Kebele-01 (Markos.Church)	TP-4	09˚23'51.7"	042˚00'37.9"	2093m
Kebele-02 office	TP-5	09˚23'41.8"	042˚00'10.47"	2099m
Kebele-02 (Gende school)	TP-6	09˚23'35.3"	042˚00'23.6"	2094m
Kebele-01 (Near LakeHaromaya)	TP-7	09˚24'5.2"	042˚00'32.6"	2090m
Kebele-02(Gende Meseri)	TP-8	09˚24'2.3"	041˚59'52.0"	2083m
Kebele-02 (Gende kalu)	TP-9	09˚22'50.6	042˚00'35.8"	2069m
Kebele-02 (Gende Beru)	TP-10	09˚23'21.3"	042˚00'21.04"	2101m

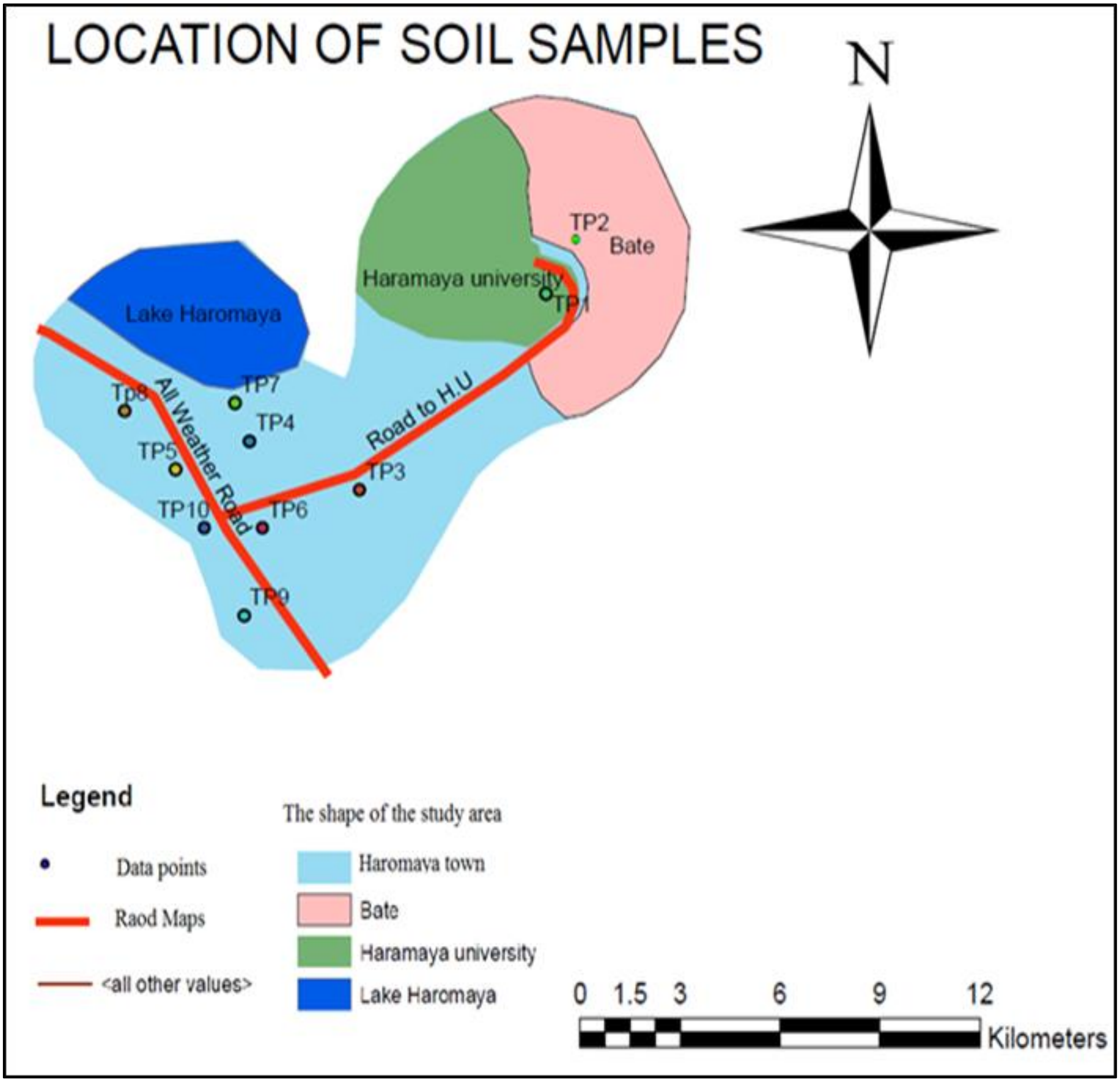

Figure 1.2: Sampling points distribution using GIS software

1.5 Scope of the study

From the samples collected the following laboratory tests were done.

- Natural moisture content
- Specific gravity test
- Atterberg limit tests
  - Liquid limit
  - Plastic limit
- Grain size analysis
  - Sieve analysis(wet method)
  - Hydrometer
- Standard Compaction tests
- Consolidation test
- Direct shear test
- Unconfined Compression Strength test
- Free swell test

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

Twenty samples of soil were collected from ten test pits. The scope of this study is limited to investigating the index properties, shear strength, compaction and consolidation characteristic. The ten test pits were excavated to a maximum depth of three meters.

1.6 Structure of the thesis

This report of the thesis consists of seven Chapters. In the first Chapter background of the problem, objective of the research, methodology and scope of study are presented. Chapter two deals with a brief literature review. Chapter three deals with the description of study area. The fourth chapter deals with laboratory tests results and analysis. The discussion on the laboratory results obtained from this work and comparison with previously done researches and soil map of the study area is covered in chapter five. Chapter six consists of the conclusions and recommendations which are drawn from the research work. Finally, references and appendices are attached at the end of the thesis.

2. LITERATURE REVIEW

2.1 General

From an engineering perspective, soil is any un-cemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks and contains void spaces between particles, which is filled by water, and air [5].

In engineering, soils are considered to include all organic and inorganic earth materials occurring in the zone overlying the rock crust. They are usually non-homogeneous, porous material whose engineering behavior is greatly affected by changes in moisture content and density. The engineering definition of soil is quite different from the agronomist definition of the same. In geology soil has different connotations and simply be stated as a material found in the relatively thin surface zone.

2.2 Soil forming factors

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration and/or chemical decomposition. The properties of the soil materials depend upon the properties of the rocks from which they are derived [14].

The variety of soil materials encountered in engineering problems is almost limitless, ranging from hard, dense, large pieces of rock through to gravel, sand, silt, and clay to organic deposits of soft compressible peat. To compound the complexity, all of these materials may occur over a range of densities and water contents. At any given site, a number of different soil types may be present, and the composition may vary over intervals of a little as a few inches [10].

The main factors affecting the formations of soil are: parent materials i.e. geology of the area, relief (topography) and drainage, climate and organisms (vegetation, fauna and soil biota).

2.2.1 Parent materials

There are two main variables in parent materials that affect soils: grain size and composition. Grain size is the main determinant of soil texture. Texture influences the soil structure, consistency, cation exchange capacity, profile drainage, moisture retaining capacity and organic content [9].

2.2.2 Relief (Topography) and Drainage

Relief is not static; it is a dynamic system (its study is called geomorphology). Relief influences soil formation in several ways**:**

- It influences soil profile thickness i.e. as angle of slope increases so does the erosion hazard.
- It has an effect on climate which is also a soil forming factor
- Gradient affects run-off, percolation and mass movement
- It influences aspect which creates microclimatic conditions

Topography has a major influence on drainage characteristics which in turn is known to have major effect on soil mineralogy. Its control over soil properties is particularly strong in tropical environment reflecting the importance of lateral movement of water and soil materials [9].

The landscapes in most eastern parts of Ethiopia map area forming the highlands are generally flat tablel and plateau surfaces. These lie mostly east of Jijiga and are characterized by alluvial sediments, leached from the limestone bedrock. The highest peaks form the Karamara mountains, continued to Kes Amba and Kundudo Terara. They are mostly formed of the Tertiary basaltic volcanics. Afgugu-Kundudo-Harero and Gures Bereley are elevated to 2000m a.m.s l. and form high flatland with cliffs at their edges. They decrease to gentle slopes with undulating topography [24].

The Lake sediments occur around Haromaya and surrounding areas, and are restricted to Lake processes in that area. As the name implies they are of lacustrine origin and occupy minor areas. The sediments are clays and silts related to the fluctuation of lake levels. The sediments might be mixed with wash sediments of the nearby uplands. Lake sediments cover an area of 11.43 sq.km [24].

2.2.3 Climate

Climate is the principal factor governing the rate and type of soil formation and is also the main determinant of vegetation distribution. Soil climate has two major components; moisture (precipitation) and temperature, influencing evaporation. When precipitation exceeds evaporation, leaching of the soil will occur. Temperature determines the rate of reactions; chemical and biological decay and so has an influence on weathering and humification.

The temperature variable is adequately represented by mean annual temperature, which doesn't differ greatly from the nearly constant temperature in the lower part of the regolith. According to Van's Hoff's principle the velocity of a chemical reaction increases by a factor of 2 or 3 for every 10^0c rise of temperature.

The two main rain fall parameters most widely available are the mean annual total and the length of the dry season. The amount and distribution of precipitation affects the availability of moisture and the relative humidity of the soil atmosphere; it influences the concentration or chemical activities of solutions in the system [8].

2.3 Soil classification

A soil classification is an arrangement of different soils into various groups or subgroups to provide a common language to express briefly the general usage characteristics without detailed descriptions. At the present time, two major soil classification systems that take in to consideration both particle-size distribution and Atterberg limits are available for general engineering use. They are the AASHTO (American association of State Highway and Transport official) classification system and USCS (unified soil classification system).

2.4 General types of soils

According to their Grain Size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials [14].

2.5 Soil particle size and shape

The size of particles may range from gravel to the finest size possible. Their characteristics vary with the size. Soil particles coarser than 0.075 mm are visible to the naked eye or may be examined by means of a hand lens. They constitute the coarser fractions of the soils. The coarser fractions of soils consist of gravel and sand. The individual particles of gravel, which are fragments of rock, are composed of one or more minerals, whereas sand grains contain mostly one mineral which is usually quartz. The individual grains of gravel and sand may be angular, sub angular, sub-rounded, rounded or well-rounded. Gravel may contain grains which may be flat. Some sands contain a fairly high percentage of mica flakes that give them the property of elasticity. Silt and clay constitute the finer fractions of the soil. Any one grain of this fraction generally consists of only one mineral. The particles may be angular, flake-shaped or sometimes needle-like [14].

2.6 Structure of Soils

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 [14].

The structure of soil may be defined as the manner of arrangement and state of aggregation of soil grains. In a broader sense, consideration of mineralogical composition, electrical properties, orientation and shape of soil grains, also may be included in the study of soil structure, which is typical for transported or sediments soils. Structural composition of sediment soils influences many of their important engineering properties such as permeability, compressibility and shear strength [23].

Generally various types of structures of soil

- single grained structure
- flocculent structure
- Honey-comb structure

2.6.1 Single grained structure

A single grained structure which is formed by the settlement of coarse grained soils in suspension in water [14].

Single grained structure is characteristics of coarse grained soils, with a particle greater than 0.02mm. Gravitational force predominate the surface force and hence grain to grain contact results. The deposition may occur in a loose state with large voids or in a dense state with less of voids [23].

2.6.2 Flocculent structure

A flocculent structure formed by the deposition of the fine soil fraction in water. The particles oriented in a flocculent structure will have edge-to-face contact as shown in Fig. 2.1(d) [14].

2.6.3 Honey-comb structure

A honeycomb structure which is formed by the disintegration of a flocculent structure under a superimposed load. The particles oriented in a honey-comb structure will have face-to-face contact as shown in Fig. 2.1(e). Natural clay sediments will have more or less flocculated particle orientations. Marine clays generally have a more open structure than fresh water clays [14].

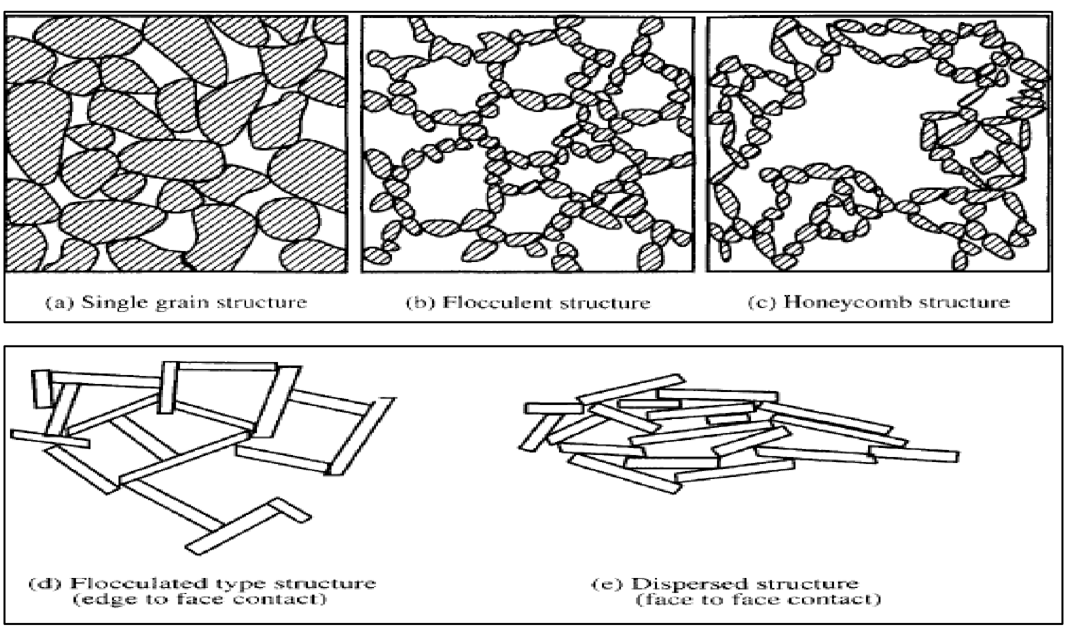

Figure 2.1: Schematic diagrams of various types of structures (Lambe, 1958)

2.7 Previous works on investigation of engineering properties of soils

According to the Unified Soil Classification System, the soil of Adama town is categorized as silt and silty sand. Almost all the samples have free swell value of less than 50%. This shows the soil in the Adama town is non-expansive [7].

The soils of Ambo town are highly plastic. Black and gray soils have higher plasticity index than reddish brown and brown soils. Grain size analysis shows that the predominant proportion of the soils is clay size fraction. According to AASHTO soil classification system soils of Ambo town are grouped in A-7-5. This indicates that they have poor quality and unsuitable for using as a sub grade material. The consistency of Ambo soils in dry season is found to be stiff to hard as the UCS ranges from 112kPa -545kPa [4].

The grain size distribution of Sebeta town indicates majority samples have clay material more than 50%. According to AASHTO soil classification system the soils categorized on A-7 and A-6 therefore those soils have poor quality for use as subgrade [18].

There are both black clay soils and red soils in the town of Adet. The black clay soils are found in the relatively lower elevation and flat lying areas of the town. However, most of the areas in the town are covered with red soils. The free swell values of black clay soils, ranges from 67-100%, indicate that the soils are moderately expansive soils. Most of the red soils have free swell values less than 50 percent whereas the whole red soil's free swell values ranges from 27-67.5%. The consistency of Adet soils is found to be medium stiff to hard as UCS ranges from 84-442kPa at natural moisture content of 28-41% [19].

3. DESCRIPTION OF THE STUDY AREA

3.1 General

Haromaya is a town in East-central Ethiopia located in the Misraq Hararghe Zone of Oromia Regional State. The town has a latitude and longitude of 9°23’40.23’’N, 42°00’33.10”E coordinates and an elevation of 2040m above sea level. Haromaya is 10km west of Harar, on the road to Dire Dawa. It is located near Lake Haromaya, a seasonal freshwater lake which supports a population of flamingo, as well as other birds. Haromaya has a total area of about 24.63square kilometers, which has been subdivided into 3 Kebeles. Based on figures from the central statistical Agency in 2015, Haromaya has an estimated total population of 21,459 of whom 11,500 are men and 9,959 are women [17].

Haromaya town is inhabited mainly by Oromo nationalities and Somalis. Mixed farming is common. Most dominant land use in the study area is agriculture and land is used for crop production. Often land is covered by crops and permanent plants such as sorghum, maize, chat and vegetables. Most of the farmland, which is located in the gentle slope alluvial cover part of the study area, is used for crops and vegetables whereas the slightly hilly side of weathered basement part is covered with chat. Haromaya is home to Haramaya University [17].

3.2 Soil and Geology

According to data obtained from geological survey, the study area can be classified into three major soil types. The eastern and North eastern part mainly constitute well drained red tropical soils with diffuse horizon boundaries, central part with white clay where silicate clay is accumulated and most of western part with gravelly or stony can be found on hard rock .

Mainly the geological set up of the study area consists of the rock types. These are unconsolidated sediments of Mesozoic and Precambrian rocks.

The most dominant rock type is the precambrian rocks which form coarse sand when weathered. The Precambrian basement rocks found exposed in the area are the pink colored, fair to highly weathered and well fractured granite. It is found underlying the area of interest as a whole. This basement lithologic unit is found unconformably overlained by the Mesozoic carbonate rocks. The second dominant rock types are the unconsolidated sediments which range in size from fine clay and silt to gravel and less often cobble. The Mesozoic sediment has not been discerned in the area of interest. It is found exposed further away in the area. It is light in color, slight to fairly weathered and fractured calclusite limestone. North eastern and some of western part of the Haromaya is covered by least abundant Mesozoic rocks [24].

3.3 Topography and drainage conditions

The altitude of Haromaya ranges from 1400 to 2340 meters above sea level; the highest points include Dof and Jeldo. The major river is the Amaresa and major lake is Lake Haromaya [17].

3.4 Climate

3.4.1 Rainfall

Climate of the study area is characterized by semi arid having average annual precipitation of 30 years (1970 – 2008) recorded at Haromaya station is about 721.9mm. Maximum precipitation occurs in April, May, August and September with the highest peak in August (131.9mm) and the minimum in December (10.2mm).

3.4.2 Temperature

In a mountainous tropical country like Ethiopia, altitude is by far the most important factor controlling climate. It affects distribution of both temperature and rainfall. Generally, regions between 1500 - 2300 meters a.m.s.l. (categorized as 'woina dega' or sub-tropical climate) have temperatures that range between 15 - 20^{o}C, areas between 500 – 1500 meters a.m.s.l. (i.e. 'kola' or tropical climate) have 20 - 30^{o}C and areas below 500 meters a.m.s.l. (i.e. 'Bereha' or desert climate) have a temperature of 30^{o}C and above [16].

The town of Haromaya, with an altitude ranging from 1400-2340meters a.m.s.l., has a mean minimum, mean maximum and mean average monthly temperatures of 3.2, 24.9 and 14.05°C respectively. The highest temperatures are during months of August, July, May and June whereas November, December and January have low temperature. From Fig 3.2 the monthly average temperature ranges from 12.61°c to 18.63°c. This shows the temperature variation is almost the same throughout the year.

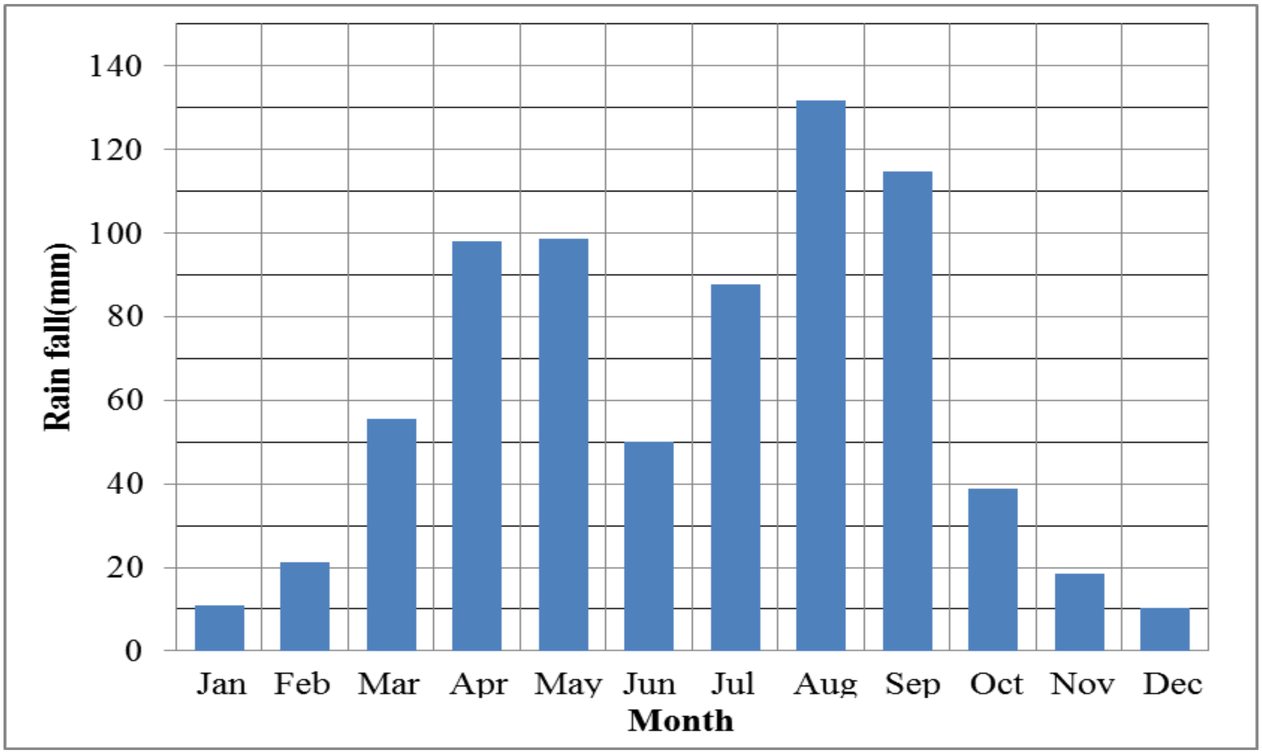

Figure 3.1: Mean monthly rainfall distribution of Haromaya (1970 - 2008)

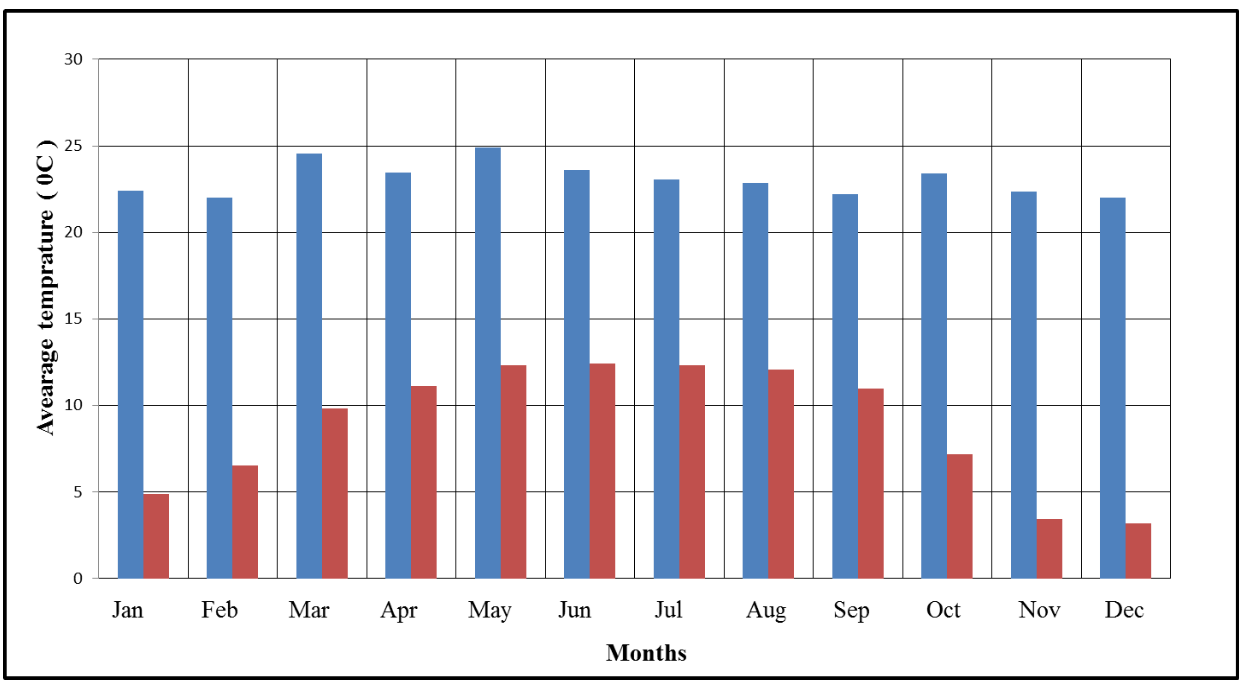

Figure 3.2: Average Monthly Maximum and Minimum temperature distribution of Haromaya, (1970 - 2008)

4. LABORATORY TEST RESULTS AND ANALYSIS

4.1 Index Properties of soil

4.1.1 General

Soils are classified and identified based on index properties. The properties and characteristics of soils vary from point to point. The tests required for determination of engineering properties are generally elaborate and time consuming. Sometimes the geotechnical engineer may need some rough assessment of the engineering properties without elaborate tests. This is possible if index properties are determined. The properties of soils which are not of primary interest to the geotechnical engineer but are indicative of the engineering properties are called index properties [1].

The nature of any soil can be altered by appropriate manipulation. Vibrations; for example, can transform a loose sand into a dense one. Hence, the behavior of a soil in the field depends not only on the significant properties of the individual constituents of the soil mass, but also on those properties that are due to the arrangement of the particles within the mass. Accordingly, it is convenient to divide index properties into two classes: soil grain properties and soil aggregate properties. The principal soil grain properties are the size and shape of the grain and, in clay soils, the mineralogical characteristics of the smallest grains. The most significant aggregate property of cohesionless soils is the relative density, whereas that of cohesive soils is the consistency [22].

The various properties of soils, which could be considered as index properties are:

- Natural moisture content
- Specific gravity
- Atterberg limits
- Particle size distribution
- Free swell

In this thesis work, the index property laboratory tests were carried out in accordance with the ASTM standard testing procedures. Each laboratory tests which were conducted are explained in the following sections.

4.1.2 Natural Moisture content

Moisture content of soils refers to the total amount of water contained in a soil either as free water or capillary water. It is always expressed as a percentage of the weight of solids in the soil [19]. Natural moisture content of a soil is affected by climate, vegetation cover of the area and other artificial factors. Hence the same soil could have different moisture contents in different seasons of a year and in different times. Since such type of moisture content is likely to fluctuate any time it may not indicate the general property of the soil. The soil samples were collected during January.

The natural moisture content of the soil under investigation was determined following ASTM D2216-98. Natural moisture content of soils of the study area ranges from 10.5-35.9%.

Table 4.1 Natural moisture content of soil samples

Test Pit No	Depth (m)	Natural moisture content (%)
TP-1	1.5	26.3
	3	30.5
TP-2	1.5	25
	3	30
TP-3	1.5	29
	3	35.9
TP-4	1.5	24.7
	3	28.4
TP-5	1.5	13.5
	3	15.6
TP-6	1.5	25.3
	3	27.6
TP-7	1.5	11.5
	3	10.5
TP-8	1.5	25.4
	3	27.6
TP-9	1.5	29.4
	3	31.6
TP-10	1.5	30.2
	3	33.5

4.1.3 Specific gravity

The specific gravity of solid matter in a soil particle may be defined as the ratio of the unit weight of solid matter to the unit weight of water. The specific gravity of the solid particles without the void spaces is called the true or absolute or real specific gravity and is usually denoted by a letter Gs [21]. The specific gravity of solid is a measure of and a means of expressing the heaviness of the material.

According to ASTM D 854-98, two procedures for performing specific gravity are provided. These are Method-A, procedures for oven dried specimen and Method-B, procedure for moist specimen. But in this research the specific gravities are determined using Method–A, procedures.

The specific gravity of solid of light-colored sand, which is mostly made of quartz, may be estimated to be about 2.65: for clayey and silty soil, it may vary from 2.6-2.9 [3]. The test results are shown in Table 4.2. The specific gravity for clay lie between 2.7-2.84 and for silty sand ranges from 2.65-2.74.

The specific gravity values showed a variation within a limited range at different depths and at different locations. The small variations may be due to the size range, the type of clay minerals and degree of desiccation or drying [5].

The specific gravity should be lower value due to the presence of the high organic content, whereas the presence of heavy minerals may lead to higher values [5].

Table 4.2: Specific Gravity of the Soil of the Study Area

Serial No	Designation	Depth (m)	Pretreatment	Specific gravity
1	TP1	1.50	OD	2.75
2		3.00	OD	2.74
3	TP2	1.50	OD	2.74
4		3.00	OD	2.71
5	TP3	1.50	OD	2.75
6		3.00	OD	2.8
7	TP4	1.50	OD	2.73
8		3.00	OD	2.84
9	TP5	1.50	OD	2.73
10		3.00	OD	2.71
11	TP6	1.50	OD	2.74
12		3.00	OD	2.7
13	TP7	1.50	OD	2.7
14		3.00	OD	2.65
15	TP8	1.50	OD	2.7
16		3.00	OD	2.75
17	TP9	1.50	OD	2.78
18		3.00	OD	2.73
19	TP10	1.50	OD	2.7
20		3.00	OD	2.76

4.1.4 Atterberg limits

4.1.4.1 General

The Swedish Scientist, Atterberg (1911), developed a method of describing quantitatively the effect of varying water content on the consistency of fine-grained soils. He established the four states of soil consistency, which are called the liquid, the plastic, the semi-solid, and the solid states. He also proposed a series of tests for determining the boundaries known as Atterberg limits between the physical states of soil. Each boundary or limit is defined by the water content that produces a specified consistency. The liquid state is produced when a fine grained soil is mixed with a large quantity of water. In such state the soil behaves like a liquid. That is, it flows freely like a liquid and has no resistance to deformation. If, however, its water content is gradually reduced, it will begin to show a small shearing strength. The limit at which soil suspension passes from no strength to a very small strength is the liquid limit. The limit is defined by moisture content of soil at that point [21].

At moisture content lower than its liquid limit, the soil is in a plastic state. If the sample is subjected to a further decrease in moisture content, it will eventually lose its plasticity. The moisture content at which the sample, when it is rolled into a thread, starts to crumble rather than distort plastically, is called its plastic limit.

After the plastic limit, the soil displays the properties of semi-solid. With a further decrease in moisture content, the soil sample will finally reach a point where it can no longer change in volume. At this point, the soil is said to have reached its shrinkage limit [21].

The different states and consistencies through which the soil sample passes with the decrease in the moisture content are depicted in Fig 4.1.

States	Limit	Consistency	Volume Change
Liquid		Very Soft	↑
………… w_l	Liquid limit…...	Soft	
Plastic		Stiff	decrease in volume
…………. w_p	Plastic limit……	Very Stiff	
Semi Solid			↓
………….. w_s	Shrinkage limit…	Extremely stiff	
Solid		Hard	Constant volume

Figure 4.1: Different states and consistency of soils with Atterberg limits [9]

4.1.4.2 Test procedure and results

Atterberg Limits were determined for air-dried samples. It was done based on the Standard Reference: ASTM D 4318-98 –Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The air-dried soil sample were prepared by spreading the material out in trays the laboratory and leaving it open to the air for at least 15days or equivalent put inside oven at a temperature of 50^0C for at least 5 days until they were dried thoroughly. The room temperature was about 20^oC. The portions of the samples passing the No. 40 (0.425mm) sieve were used for the preparation of the sample for this test.

The Atterberg limits test results for soils of study area are summarized in Table 4.3. From this we can observe that liquid limit ranges from 32-87.3%, plastic limit ranges from 8.25-28.4% and plastic index from 14.7-63.4%.

The test results show that soils of the study area are highly plastic with high plasticity index values. However soils from TP2 at 1.5m depth and TP7 at 3m depth are non-plastic material.

There are some fine grained soils that appear similar to clays but they cannot be rolled into threads so easily. Such materials are not really plastic. They may be just at the border line between plastic and non-plastic soils. In such soils, one finds the liquid limit practically identical with the plastic limit and $I_p = 0$ [14].

Table 4.3: Atterberg limits of soils of the study area

Serial No.	Designation	Depth (m)	Liquid Limit (%)	Pastic Limit (%)	Plastic Index (%)	Plasticity
1	TP1	1.5	36.0	17.6	18.2	High Plastic
2		3	72.0	26.5	45.5	High Plastic
3	TP2	1.5				Non Plastic material
4		3	65.6	23.8	41.8	High Plastic
5	TP3	1.5	86.4	23.0	63.4	High Plastic
6		3	87.3	24.0	63.3	High Plastic
7	TP4	1.5	60.0	25.0	35.0	High Plastic
8		3	66.0	27.6	38.4	High Plastic
9	TP5	1.5	61.1	23.9	37.2	High Plastic
10		3	54.0	28.4	25.9	High Plastic
11	TP6	1.5	36.4	17.7	18.7	High Plastic
12		3	32.0	17.4	14.7	Medium
13	TP7	1.5	39.5	22.0	17.5	High Plastic
14		3				Non Plastic material
15	TP8	1.5	44.9	20.7	24.2	High Plastic
16		3	39.0	8.25	28	High Plastic
17	TP9	1.5	57.2	24.0	33.3	High Plastic
18		3	55.0	23.0	31.97	High Plastic
19	TP10	1.5	55.1	21.3	33.8	High Plastic
20		3	52.0	24.5	27.8	High Plastic

4.1.5 Particle size distribution of soil

4.1.5.1 General

The size of soil particle and their distribution throughout the soil mass are important factors which influence soil properties and performance. Particle size is expressed in term of single diameter. For the larger particles this is taken as being equal to the size of the smallest square hole of a sieve through which the particle will pass [21].

4.1.5.2 Test procedure and results

The test was conducted according to ASTM D422-63, the distribution of particles finer than 75μm can be done by hydrometer test and courser than 75μm by mechanical sieve. Therefore, the samples collected from the site were air dried first and representative sample was taken by quartering. The weight of the sample was measured and then after it was washed on sieve No. 200. Mechanical sieve was done on samples of soil retained on sieve No. 200, after oven drying it for 24 hours.

In the hydrometer test 50grams of soil was taken and soaked for 24 hours by adding dispersing agent. At the end of soaking, the sample was dispersed further using stirring apparatus. Then it's poured into 1000ml cylinder and stirred again for a period of 1 min by covering it with the palm.

The hydrometer reading and test temperature was taken for 0.75, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, 1440 minutes.

The procedures mentioned above were followed for grain size analysis for all the twenty samples. The combined grain size distribution curve for particles retained on No.200 sieve and passing No.200 sieve is shown in Fig 4.2 and 4.3 respectively. The gradation of soils in the study area varies considerably (Table 4.4).

From the particle size distribution results, it is observed that there is a range of variation of the particle sizes. Clay and sand size particles are dominant. The Grain Size Analysis result clay content ranging from 1.6-65.75%, silt fraction 5.2-36.7%, sand fraction 6.88-84.3% and gravel content from 0.0 – 18.5%.

Table 4.4: Summary of grain size analysis result

Serial. No	Designation	Depth (m)	Percentage amount of particle size			
			Gravel %	Sand %	Silt %	Clay%
1	TP1	1.5	0	7.99	26.55	65.35
2		3	0	6.88	32.37	65.75
3	TP2	1.5	0.9	67.7	21.1	10.3
4		3	0.4	16.7	36.7	46.3
5	TP3	1.5	7.7	10.0	32.2	50.2
6		3	18.5	9.7	21.9	50.5
7	TP4	1.5	6.9	32.6	19.8	40.7
8		3	0.7	13.7	20.1	65.5
9	TP5	1.5	4.2	35.0	24.7	36.1
10		3	1.1	39.6	18.3	41.0
11	TP6	1.5	1.1	12.0	34.1	52.8
12		3	0.8	20.8	22.7	55.7
13	TP7	1.5	8.6	23.0	19.9	49.6
14		3	9.0	84.3	5.2	1.6
15	TP8	1.5	1.3	24.0	21.0	53.7
16		3	1.3	21.4	20.4	57.0
17	TP9	1.5	0.2	19.6	27.6	52.6
18		3	5.9	11.0	30.0	53.1
19	TP10	1.5	3.1	19.5	26.6	50.8
20		3	1.8	28.7	33.3	36.2

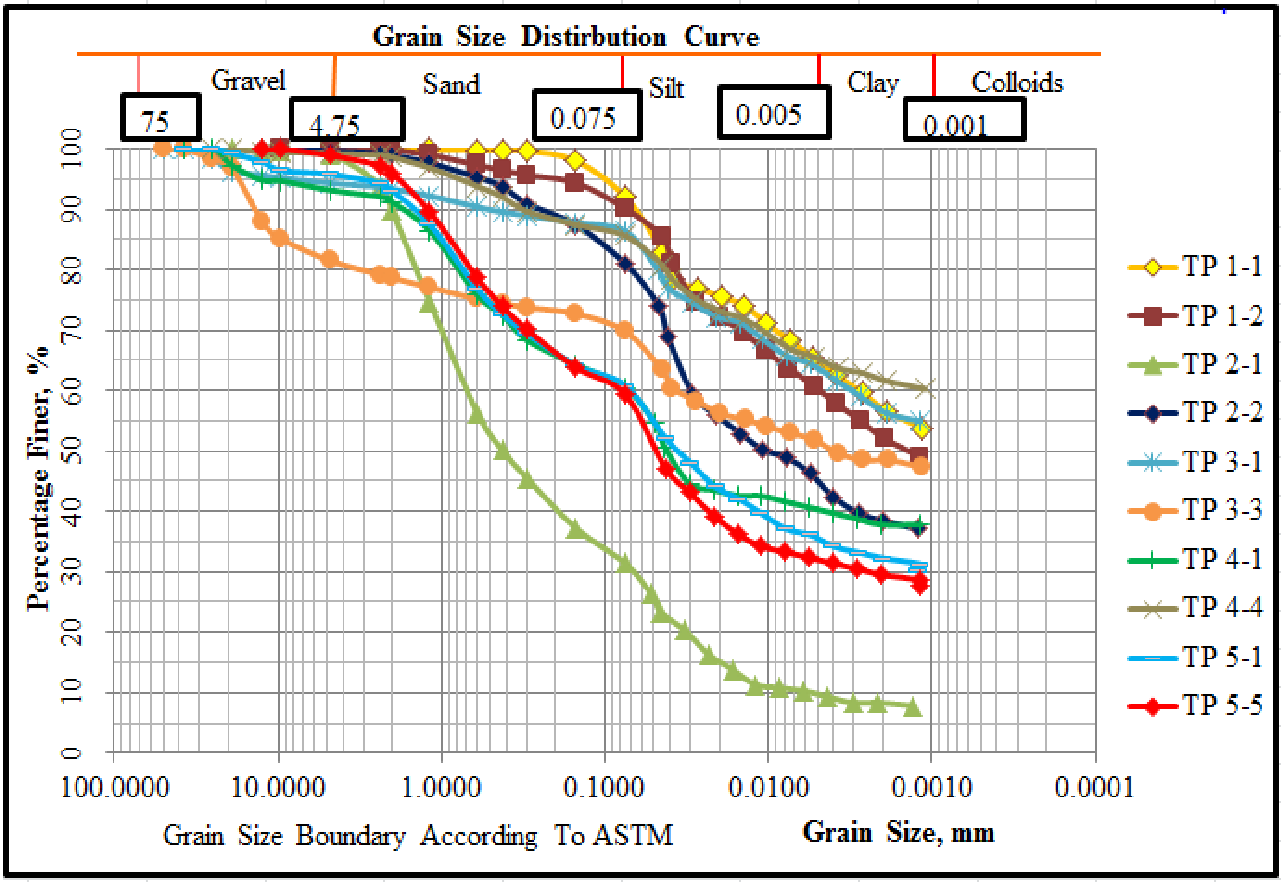

Figure 4.2: Grain size distribution curve for samples from test pits 1 to 5

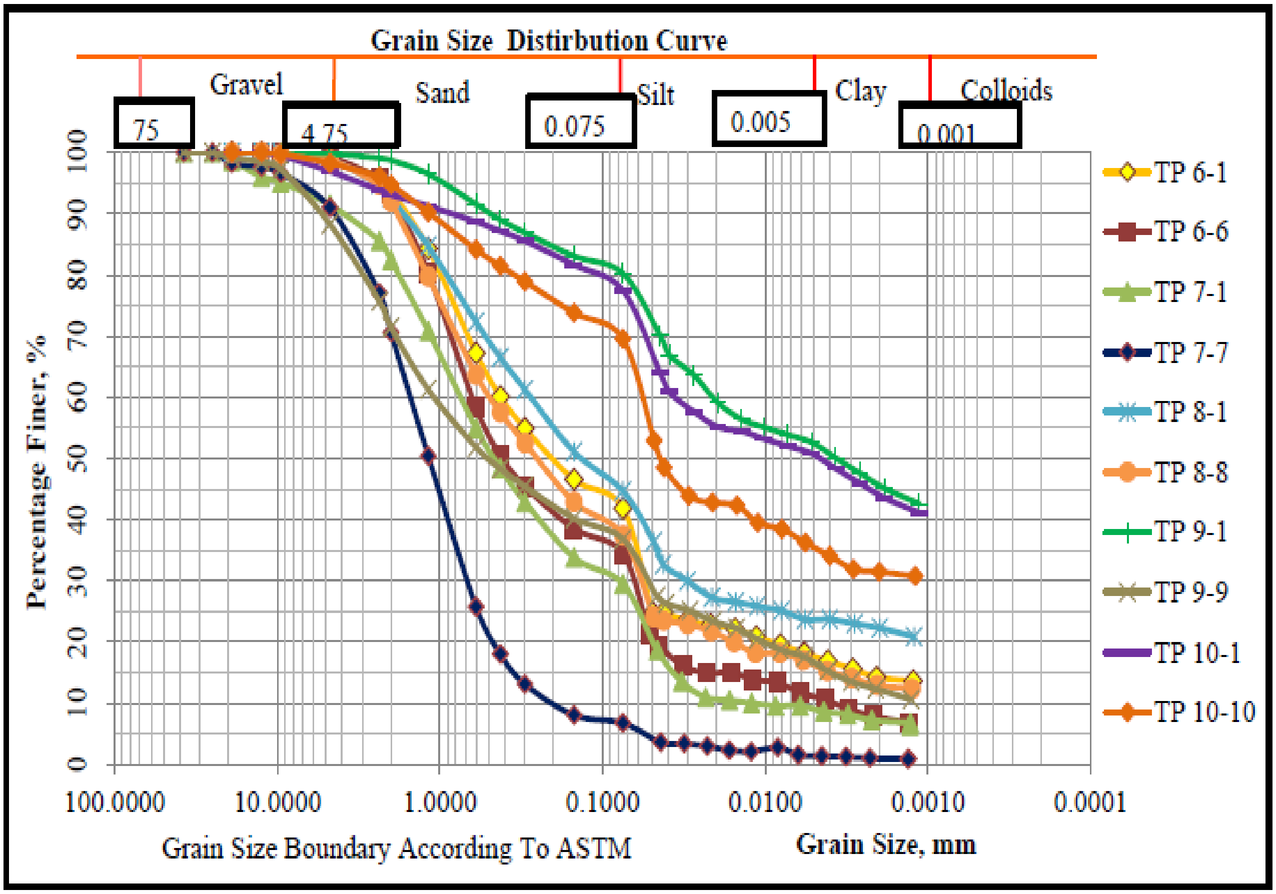

Figure 4.3: Grain size distribution curve for samples from test pits 6 to 10

4.1.6 Free swell

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

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

This test tries to give a fair approximation of the degree of expansiveness of a give soil sample. Soils with free swell less than 50% are not likely to show expansive property, while soils with free swells in excess of 50% could present swell problems. Values of 100% or more are associated with clay which could swell considerably.

Free swell test results for oven dried samples at a temperature of 105±5^{o}c are summarized in Table 4.5. From the test result one can see that the free swell of the soil under investigation ranges from 20% to 113%. The soils are mostly in between 50% and 100% except for TP3 at 1.5m and 3.0m which are typically black cotton soils. Most of the soils have moderate expansive nature, which have a little impact on construction of structures.

Table 4.5: Free swell test results of the stud area

Designation	Depth (m)	Test condition	Free Swall (%)
TP1	1.5	OD	45
	3	OD	68
TP2	1.5	OD	35
	3	OD	55
TP3	1.5	OD	113
	3	OD	105
TP4	1.5	OD	60
	3	OD	55
TP5	1.5	OD	68
	3	OD	75
TP6	1.5	OD	30
	3	OD	40
TP7	1.5	OD	35
	3	OD	20
TP8	1.5	OD	68
	3	OD	75
TP9	1.5	OD	35
	3	OD	75
TP10	1.5	OD	55
	3	OD	75

4.2 Compaction Test

Compaction means pressing the soil particles close to each other by mechanical methods. Air is expelled from the void space in the soil mass during compaction and the mass density is increased. Compaction of a soil is done to improve its engineering properties of the soil. Compaction generally increases the shear strength of the soil, and hence the stability and bearing capacity. It is also useful in reducing the compressibility and permeability of the soil [1].

The optimum water content is the water content that results in the greatest density for a specified compaction effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density[12].

There are two types compaction tests are routinely performed;

- Standard proctor compaction test
- Modified proctor (or modified AASHTO) compaction test

In the Standard Proctor Test, the soil is compacted by a 24.4N hammer falling a distance of 0.305meters into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test, except it employs, a 44.5N hammer falling a distance of 0.457meters, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 0.102meters in diameter and has a volume of about 944 cm^3, and the larger type is 0.152meters in diameter and has a volume of about 2123 cm^3. If the larger mold is used each soil layer must receive 56 blows instead of 25[12].

From the test results the maximum dry density (MDD) of Haromaya ranges from 1.39 to 1.52 g/cm^3 and the optimum moisture content ranges 13 to 23.4 percent. The summary of the test result is shown in Table 4.6.

Table 4.6: Optimum moisture content and the maximum dry density

Designation	Depth(m)	MDD (g/cm^3)	OMC (%)
TP-3	3	1.39	23.4
TP-6	3	1.41	13.5
TP-7	3	1.44	13
TP-8	3	1.52	15

4.3 Classification of the Soils

4.3.1 General

A soil classification system is an arrangement of different soils into groups having similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known and to provide the engineer with accurate method of soils description [21].

There are several methods of classifying soils. The most widely used classification systems by engineers are described here. The soils under investigation have been classified according to AASHTO M-145 and USCS .The classification results are shown in Table 4.7 and 4.8

4.3.2 Unified soil classification system (USCS)

The unified classification system is based on recognition of the type and predominance of the constituents considering grain size, gradation and plasticity. It divides soil into three major divisions; coarse-grained soils, fine-grained soils, and highly organic (peat) soils. In the laboratory, the Grain-Size curve and the Atterberg limits can be used. The peat soils are readily identified by color, odor, spongy feel and fibrous texture [14].

The name and symbols used to distinguish between the typical and boundary soil groups are GW, GP, GM, GC, SW, SP, SM and SC. The symbols that started with a prefix G stands for gravel or gravelly soil and symbol that started with S are sand or sandy soil.

These symbols and their representations are: G-gravel, S-sand, M-silt and 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 symbol ''O'' indicating the presence of organic material.

Table 4.7: Classifications of soils based on USCS Classification system

Designation	Depth (m)	LL (%)	PI (%)	Percentage amount of particle size				Classification according to USCS
				Gravel (%)	Sand (%)	Silt (%)	Clay (%)	
TP1	1.5	36	18.2	0	7.99	26.55	65.35	CL
	3	72	45.5	0	6.88	32.37	65.75	CH
TP2	1.5	-	-	0.9	67.7	21.1	10.3	SM
	3	65.6	41.8	0.4	16.7	36.7	46.3	CH
TP3	1.5	86.4	63.4	7.7	10.0	32.2	50.2	CH
	3	87.3	63.3	18.5	9.7	21.9	50.5	CH
TP4	1.5	60	35	6.9	32.6	19.8	40.7	CH
	3	66	38.4	0.7	13.7	20.1	65.5	CH
TP5	1.5	61.1	37.2	4.2	35.0	24.7	36.1	CH
	3	54	25.9	1.1	39.6	18.3	41.0	CH
TP6	1.5	36.4	18.7	1.1	12.0	34.1	52.8	CL
	3	32	14.7	0.8	20.8	22.7	55.7	CL
TP7	1.5	39.50	17.5	8.6	23.0	19.9	49.6	CL
	3	-	-	9.0	84.3	5.2	1.6	SM
TP8	1.5	44.9	24.2	1.3	24.0	21.0	53.7	CL
	3	39	28	1.3	21.4	20.4	57.0	CL
TP9	1.5	57.2	33.3	0.2	19.6	27.6	52.6	CH
	3	55.0	32.0	5.9	11.0	30.0	53.1	CH
TP10	1.5	55.1	33.8	3.1	19.5	26.6	50.8	CH
	3	52.3	27.8	1.8	28.7	33.3	36.2	CH

According to USCS classification scheme most of the soil of the study area falls in CH or CL region. From the plot of plasticity chart in Figure 4.4 and the classification soils on Table 4.7 the soils found in Haromaya town is highly plastic clay, Low to medium plastic clay and silty sand.

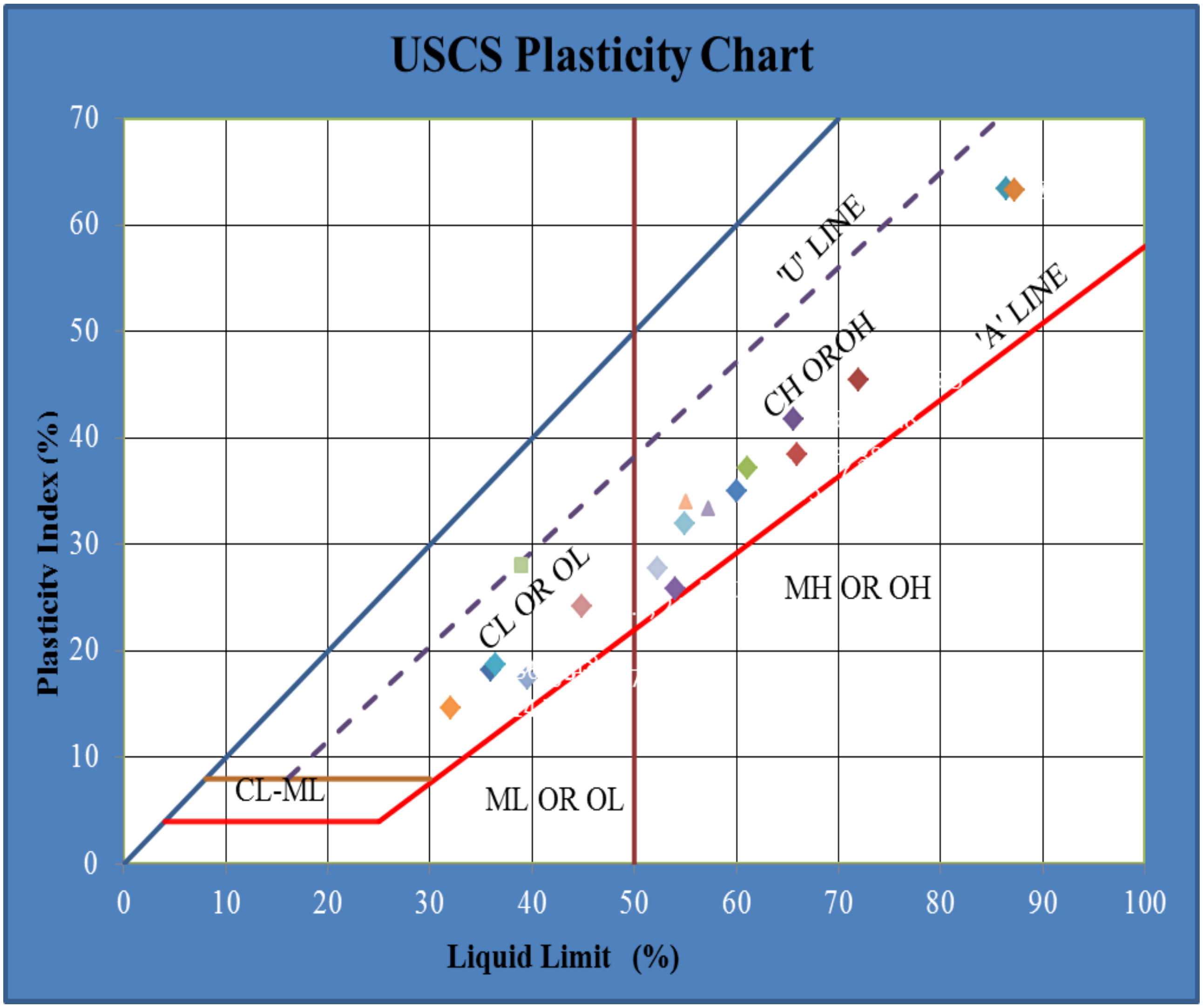

Figure 4.4: Plasticity chart of the study area according to Unified Soil Classification System

4.3.3 AASHTO Classification System

The AASHTO classification system, also called public roads administration (PRA) classification, is based on Grain Size Distribution, Liquid Limit and Plasticity Index. This system is generally used by highway engineers for classification of sub-grade soils for the highway pavement. It classifies a soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2 and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6 and A-7.

According to AASHTO classification in (Table 4.8 and Fig 4.5) scheme most of the soil of the study area falls in A-7-6 and A-6 groups except samples from TP-2 at 1.5m and TP-7 at 3m which is Silty or clayey gravel and sand. A-7-6 and A-6 materials are generally considered the poorest performers with regard to roadway construction, but can be utilized in sub-grade material. Further, more group index values of the samples is calculated and indicate that 50% of the samples are below 20, which are good source of sub-grade material, other samples may need some stabilization techniques to use as source of material. The value of GI is greater 20 for the rest of 50% samples this indicates that percentage amount of fine particles (#200) is high; this indicates the soils are highly plastic nature.

Table 4.8: Classifications of soils based on AASHTO Classification system

Serial No.	Designation	Depth In (m)	Prcent Passing on Sieve			LL (%)	PI (%)	Group Index	Group classification	Usual types of significant Constituent material	General Rating as sub Grade material
			No.10	No.40	No.200						
1	TP1	1.5	99.96	99.78	92.01	36	18.2	17.0	A-6	clay soils	Fair to poor
2		3	99.81	96.45	93.12	72	45.5	49.0	A-7-6	clay soils	Fair to poor
3	TP2	1.5	89.82	50.15	31.38	24.7	0	0.0	A-2-4	Silty or clayey gravel and sand	Excellent to good
4		3	98.96	93.8	82.98	65.6	41.8	37.0	A-7-6	clay soils	Fair to poor
5	TP3	1.5	93.37	89.55	86.37	86.4	63.4	60.0	A-7-6	clay soils	Fair to poor
6		3	78.63	74.41	71.79	87.3	63.3	46.0	A-7-6	clay soils	Fair to poor
7	TP4	1.5	91.20	71.53	60.51	60	35	19.0	A-7-6	clay soils	Fair to poor
8		3	98.63	91.03	85.61	66	38.4	37.0	A-7-6	clay soils	Fair to poor
9	TP5	1.5	93.09	72.67	60.82	61.1	37.2	20.0	A-7-6	clay soils	Fair to poor
10		3	96.01	73.96	59.34	54	25.9	14.0	A-7-6	clay soils	Fair to poor
11	TP6	1.5	93.78	60.08	66.86	36.4	18.7	4.0	A-6	clay soils	Fair to poor
12		3	93.23	50.43	78.40	32	14.7	1.0	A-6	clay soils	Fair to poor
13	TP7	1.5	82.4	48.4	53.3	39.5	17.5	2.0	A-6	clay soils	Fair to poor
14		3	70.6	18	6.73	32	0	0.0	A-2-4	Silty or clayey gravel and sand	Excellent to good
15	TP8	1.5	92.49	66.48	59.43	44.9	24.2	6.0	A-7-6	clay soils	Fair to poor
16		3	91.98	57.50	64.46	39	30.4	5.0	A-6	clay soils	Fair to poor
17	TP9	1.5	98.7	89	80.2	57.2	33.3	28.0	A-7-6	clay soils	Fair to poor
18		3	71.4	48.3	63.13	55	32	8.0	A-7-6	clay soils	Fair to poor
19	TP10	1.5	93.1	87.1	77.4	55.1	33.8	27.0	A-7-6	clay soils	Fair to poor
20		3	94.8	81.5	69.5	52	27.8	19.0	A-7-6	clay soils	Fair to poor

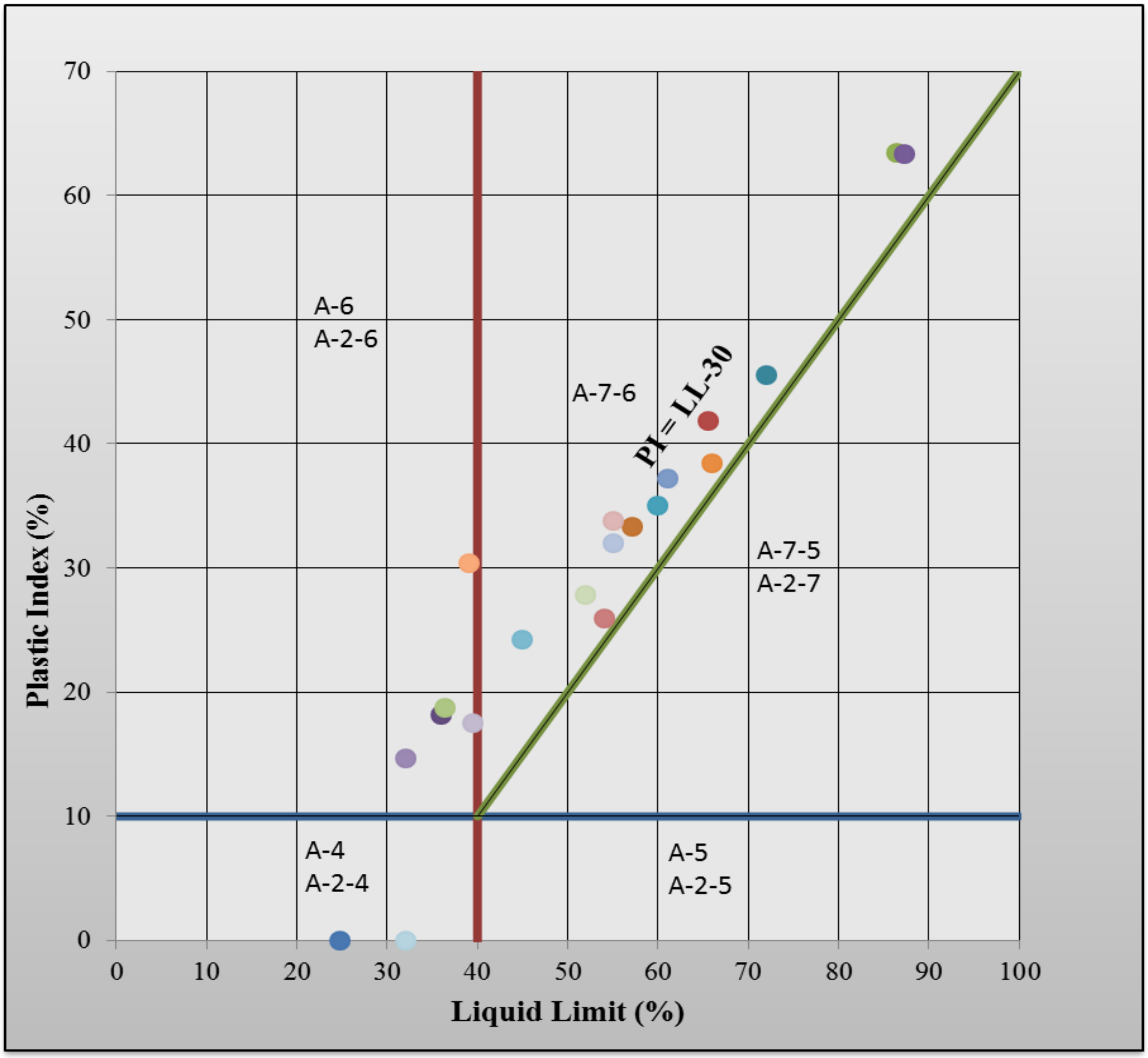

Figure 4.5: Plasticity chart of soil in the study area according to AASHTO system of classification

4.3.4 Classification based on activity

Skempton (1953) considers that the significant change in the volume of a clay soil during shrinking or swelling is a function of plasticity index and the quantity of colloidal clay particles present in soil. The clay soil can be classified inactive, normal or active [14].

The activity of clay is expressed as:

$$\text{Activity (A)} = \frac{Plastic\ Index}{Clay\ fraction(\%)} \tag{4.2}$$

Table 4.9: Classification of soils based on activity

Serial No.	Activity	Soil type
1	< 0.75	In active
2	0.75-1.25	Normal
3	> 1.25	Active

Activities of soils of the study area are computed based on results obtained from hydrometer analysis (% of clay fraction) and Atterberg's limit (PI). Accordingly soils of the study area ranges from in active soil to active soil.

Table 4.10: Classification of soil based on Activity system

Designation	Depth(m)	PI	Clay Fraction (%)	AC	Remark
TP1	1.5	18.2	65.35	0.28	In active
	3	45.5	51.91	0.88	Normal
TP2	1.5	0	10.32	0.00	In active
	3	41.8	46.29	0.90	Normal
TP3	1.5	63.5	50.2	1.26	Active
	3	63.3	50.47	1.25	Active
TP4	1.5	35	40.68	0.86	Normal
	3	38.4	65.54	0.76	Normal
TP5	1.5	37.2	36.13	1.03	Normal
	3	25.9	41	0.63	In active
TP6	1.5	18.7	52.8	0.35	In active
	3	14.7	55.7	0.26	In active
TP7	1.5	17.5	49.6	0.35	In active
	3	0	1.56	0.00	In active
TP8	1.5	24.2	53.7	0.45	In active
	3	28	57	0.49	In active
TP9	1.5	33.3	52.61	0.63	In active
	3	32.0	53.1	0.60	In active
TP10	1.5	33.8	50.79	0.67	In active
	3	27.8	36.23	0.77	Normal

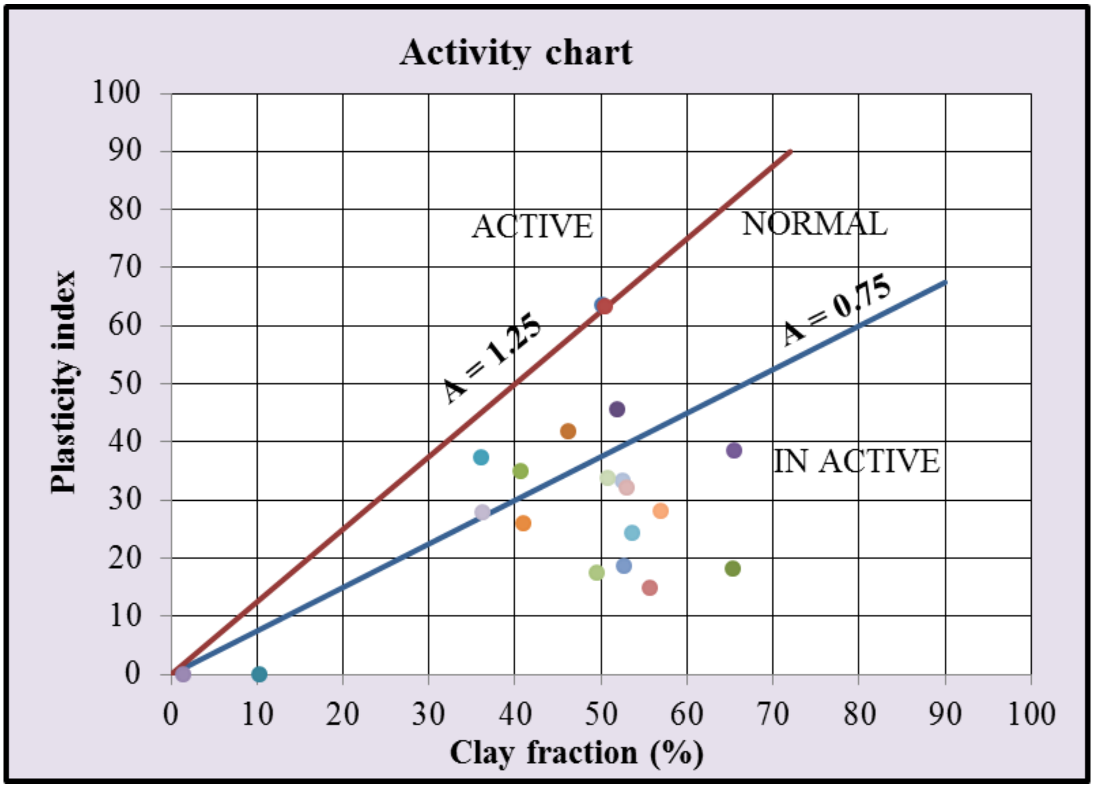

Figure 4.6: Activity chart of soils of the study area

4.4 Consolidation Test

4.4.1 General

Structures are built on soils. They transfer loads to the subsoil through the foundations. The effect of the loads is felt by the soil normally up to a depth of about two to three times the width of the foundation. The soil within this depth gets compressed due to the imposed stresses. The compression of the soil mass leads to the decrease in the volume of the mass which results in the settlement of the structure [14].

The displacements that develop at any given boundary of the soil mass can be determined on a rational basis by summing up the displacements of small elements of the mass resulting from the strains produced by a change in the stress system. The compression of the soil mass due to the imposed stresses may be almost immediate or time dependent according to the permeability characteristics of the soil. Cohesionless soils which are highly permeable are compressed in a relatively short period of time as compared to cohesive soils which are less permeable. The compressibility characteristics of a soil mass might be due to any or a combination of the following factors [14].

1) Compression of the solid mater

2) Compression of water and air within the voids

3) Escape of water and air from the voids

It is quite reasonable and rational to assume that the solid matter and the pore water are relatively incompressible under the loads usually encountered in soil masses. The change in volume of a mass under imposed stresses must be due to the escape of water if the soil is saturated. But if the soil is partially saturated, the change in volume of the mass is partly due to the compression and escape of air from the voids and partly due to the dissolution of air in the pore water.

Generally, the volume change in a soil deposit can be divided in to three stages: [1]

- Initial consolidation
- Primary consolidation
- Secondary consolidation

4.4.2 Test procedure and results

This test was done according to the procedure called standard test method for one dimensional consolidation properties of soils on the ASTM standard, Designation D2435-96.

A small representative sample of undisturbed soil is carefully trimmed and fitted into the 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. A sitting load of 7kPa was applied. The loads were applied through the lever arm and the dial gauge readings were taken at a time interval of 0.1,0.25,0.5,1,2,4,8,15,30,60,120,240,480,1440 minutes. The loads were doubled every 24hrs starting from 50kPa to 1600kPa. This procedure was followed for all the samples. The plot of void ratio versus logarithm of pressure and pressure for all the samples is shown in Fig 4.8 and 4.9 respectively.

Three test pits are selected based on the soil classification. Almost all soils are fall under CH and CL so for this soil groups a representative consolidation test is conducted.

4.4.2.1 Pre-consolidation pressure

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another. The practical significance of the pre-consolidation load appears in calculating settlements of structures [11].

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). Pre-consolidation pressures of soils of study area were done for representative samples by using graphical method (void ratio vs log p curve) as shown in Fig 4.7.

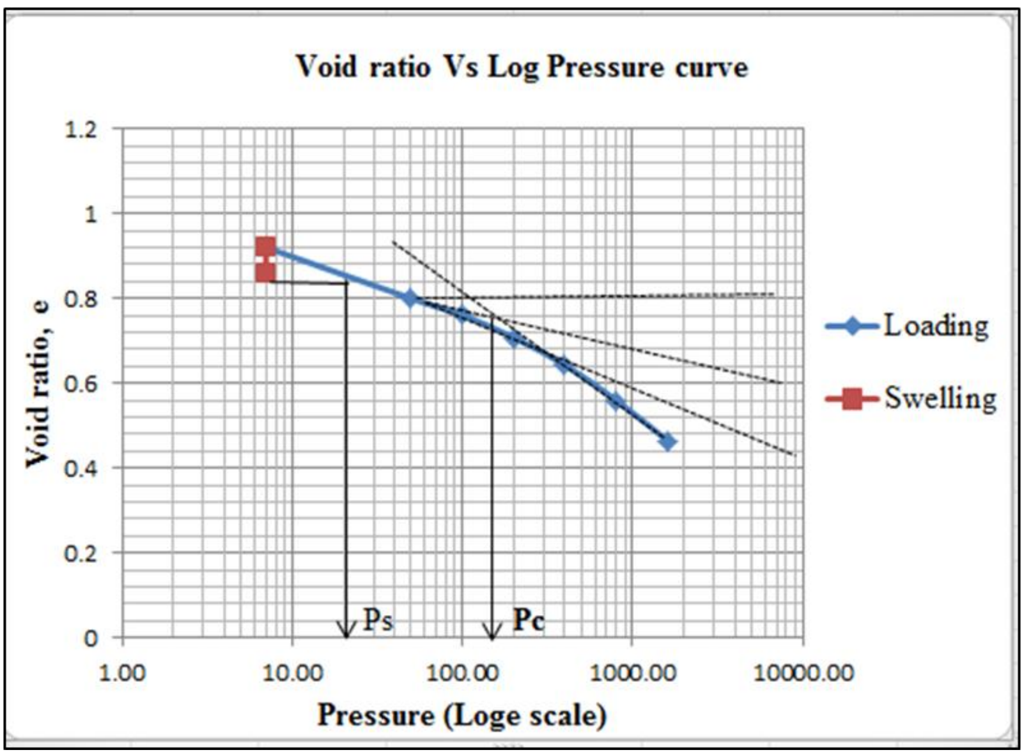

Figure 4.7: Typical void ratio Vs pressure curve to determine P_s and P_c

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

$$OCR = \frac{P_C}{P_O} \tag{4.3}$$

The pre-consolidation pressure for the three samples is determined as depicted in Fig 4.8. The results are shown in Table 4.11. From Fig 4.8 and 4.9 all the samples has almost similar pre-consolidation pressure.

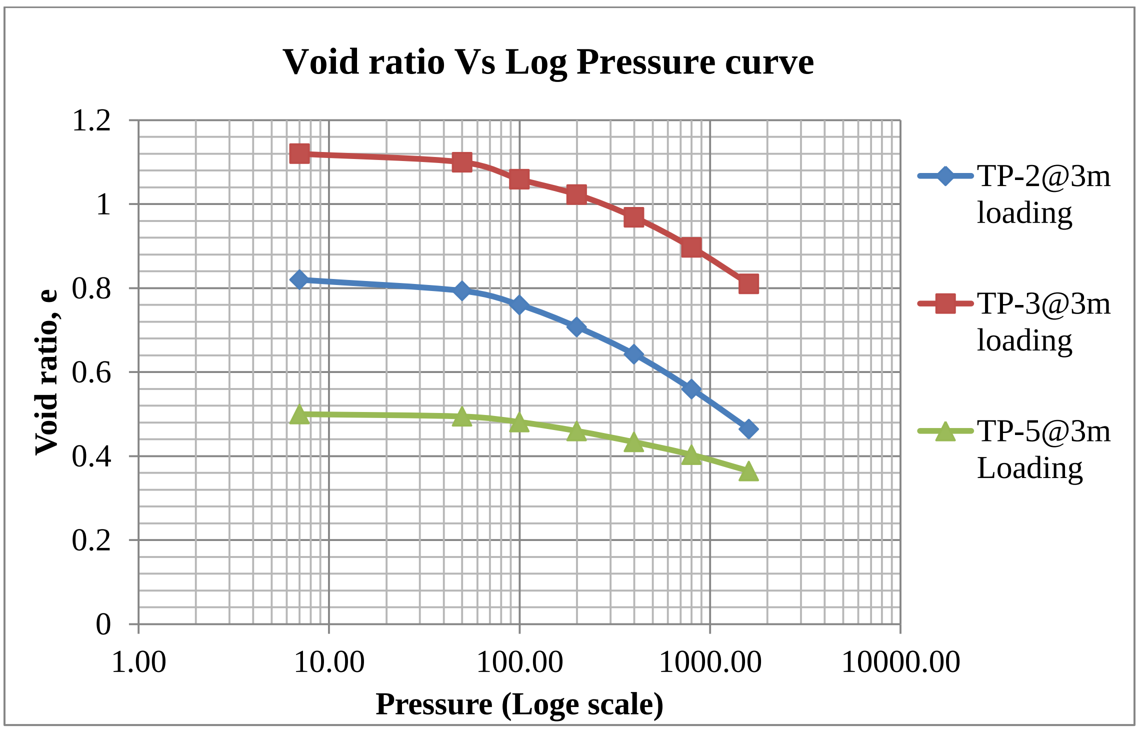

Figure 4.8: Plot of Pressure (kPa) Vs void ratio on semi-log scale

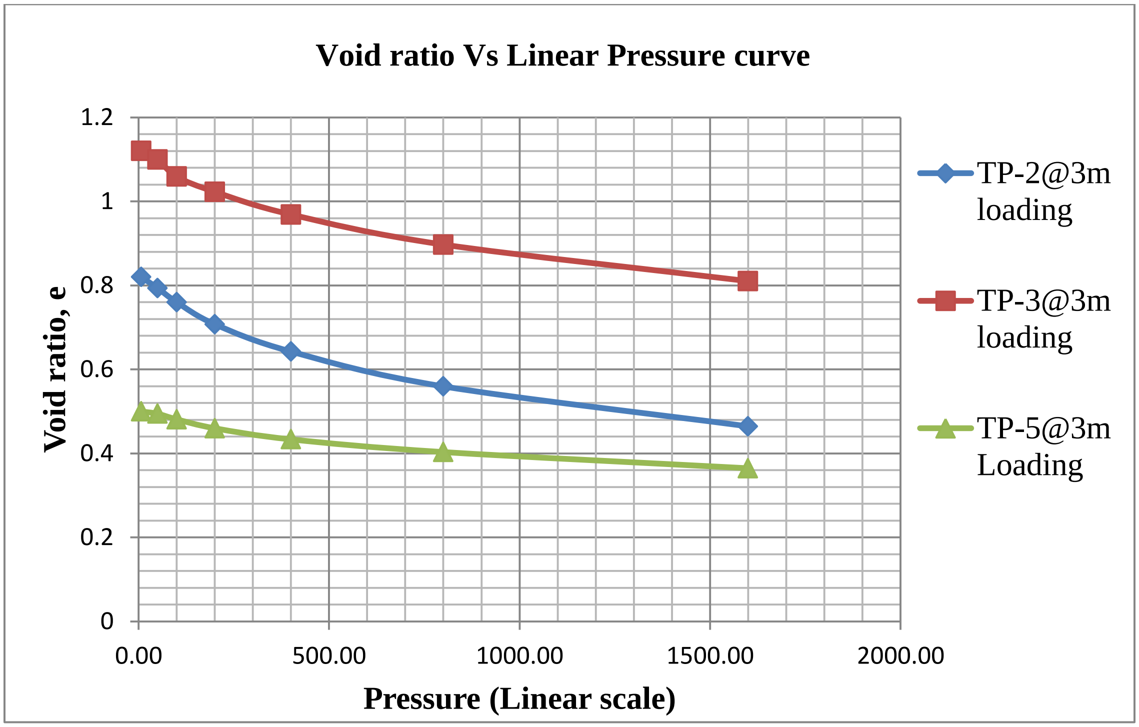

Figure 4.9: Plot of Pressure (kPa) Vs void ratio on linear scale

4.4.2.2 Compression index

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

$$C_c = \frac{e_1 - e_2}{\log p2 - \log p1} \quad (4.4)$$

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

4.4.2.3 Coefficient of consolidation

For a given load increment, the coefficient of consolidation C_v can be determined from the laboratory observation of time versus dial reading. There are two graphical methods commonly used for determining C_v from laboratory one dimensional consolidation tests.

i. Logarithm-of-time-fitting method

The logarithm- of time method was originally proposed by casagrande and fadum (1940). The coefficient of consolidation as determined by Casagrande's semi logarithmic plot method is given by:

$$C_v = \frac{(0.197)*(H^2{}_{dr})}{t_{50}} \quad (4.5)$$

ii. Square-root-of-time fitting method

Square root of time fitting method was devised by Taylor. The fitting method consists of first drawing the straight line which best fits the early portion of the laboratory curve. Next a straight line is drawn which at all points has abscissa 1.15 times as great as those of the first line. The intersection of this second line with the deformation-square root of time curve is the deformation, d_{90}, and time, t_{90}, corresponding to 90% primary consolidation.

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

$$C_v = \frac{(0.484)*H^2dr}{t_{90}} \quad (4.6)$$

From the measured data and the data obtained from either of the above two methods, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the Soil [7].

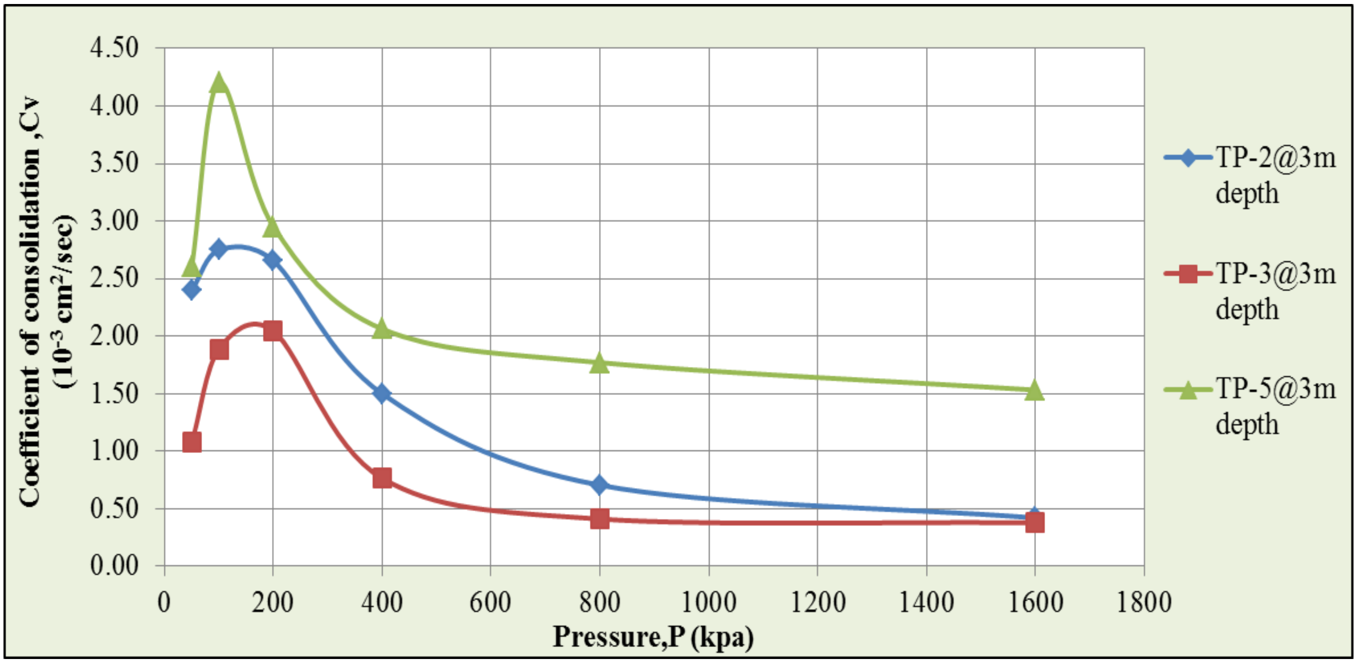

Figure 4.10: Plot of Coefficient of Consolidation (10^{-3} cm^2/sec) Vs Pressure (kPa)

From the Coefficient of Consolidation verses pressure graph it can be observed that at the initial stage as Coefficient of Consolidation value increase the pressure also increase this shows the pre-consolidation pressure have a large value as compared with over burden pressure.

Table 4.11: Summery of the consolidation test results

Designation	Depth (m)	Natural Moisture Content (%)	Total unit weight in (γ) kPa	Pressure (kPa)	Void ratio (e_f)	Coefficient of consolidation, Cv (10^{-3} cm^2/sec)	Compression Index(Cc)	Over- burden pressure,Po (kPa)	Pre-consolidation pressure ,Pc (kPa)	Swelling pressure (kPa)	over-Consolidation ratio (OCR)
TP-2	3	30	19.7	50	0.79	2.40	0.33	59.1	150	20	2.54
				100	0.76	2.75					
				200	0.71	2.65					
				400	0.64	1.50					
				800	0.56	0.71					
				1600	0.46	0.42					
TP-3	3	35.9	17.7	50	1.10	1.08	0.3	53.1	100	110	1.9
				100	1.06	1.89					
				200	1.02	2.04					
				400	0.97	0.77					
				800	0.90	0.41					
				1600	0.81	0.38					
TP-5	3	15.6	21.3	50	0.49	2.60	0.13	63.9	90	14	1.41
				100	0.48	4.20					
				200	0.47	2.95					
				400	0.43	2.06					
				800	0.40	1.77					
				1600	0.36	1.53					

4.5 Shear strength test

4.5.1 General

The shear strength of soils is an important aspect in many foundation-engineering problems related to stability such as the bearing capacity of shallow foundations and piles, the stability of slopes of dams and embankments, and lateral earth pressure on retaining walls. The purpose of shear strength testing is to establish representative values for the shear strength parameters. The drainage conditions during the test influence the measured values considerably. The most common laboratory methods employed to obtain shear strength parameters are direct shear test, triaxial compression test and unconfined compression test. For this thesis direct shear test and UCS is conducted**.**

4.5.2 Unconfined Compression Test

There are three limiting conditions of consolidation (happens before shear) and drainage (happens during shear) that model real field situations: consolidated drained (CD), consolidated-undrained (CU), and unconsolidated-undrained (UU).Unconsolidated-drained is not a meaningful condition because drainage would occur during shear and the effects of confining pressure and shear could not be separated. A special case of the UU test is the unconfined compression, where the confining pressure equals zero (atmospheric pressure). This is by far the most common laboratory strength test used in geotechnical engineering today [4].

The consistency of clay soils and other cohesive soils is usually described as soft, medium, stiff or hard. The most direct quantitative measure of consistency is the load per unit area at which unconfined cylindrical samples of the soil fails in compression test. This quantity is known as the unconfined compressive strength of the soil [22].

Table 4.12 shows the general relation between Consistency and unconfined strength of clay soil [22].

Table 4.12: Consistency and unconfined strength of clay soil

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

4.5.3 Test procedure and results

According to ASTM standard (D 2166) Place the specimen in the loading device so that it is centered on the bottom platen. Adjust the loading device carefully so that the upper platen just makes contact with the specimen. Zero the deformation indicator. Apply the load so as to produce an axial strain at a rate of ½ to 2 %/min. Record load, deformation, and time values at sufficient intervals to define the shape of the stress-strain curve (usually 10 to 15 points are sufficient). The rate of strain should be chosen so that the time to failure does not exceed about 15 min. Continue loading until the load values decrease with increasing strain, or until 15 % strain is reached. The rate of strain used for testing sealed specimens may be decreased if deemed desirable for better test results.

Unconfined compressive strength of soils of the study area is conducted for representative undisturbed samples taken at a depth of 3m. All samples were taken during dry season. The obtained results are listed in Table 4.13

Compressive strength of soils of the study area ranges from 132.2kN/m^2 -273.4kN/m^2. This shows that consistency of the soil is ranging from stiff to very stiff. Results indicated that undisturbed samples, the soil particles were very close together due to the slow rate of sedimentation and compaction. The compacted structure of undisturbed soil resulted in higher Unconfined Compressive Strength value. Analysis of the test results are presented in Appendix B.

Table 4.13: Unconfined compressive strength of soils of the study area

Designation	Depth (m)	UCS (kPa)	Moisture Content (%)	Consistency
TP-2	3	132.2	21.35	Stiff
TP-3	3	176.76	35.9	Stiff
TP-5	3	273.4	16.91	Very stiff

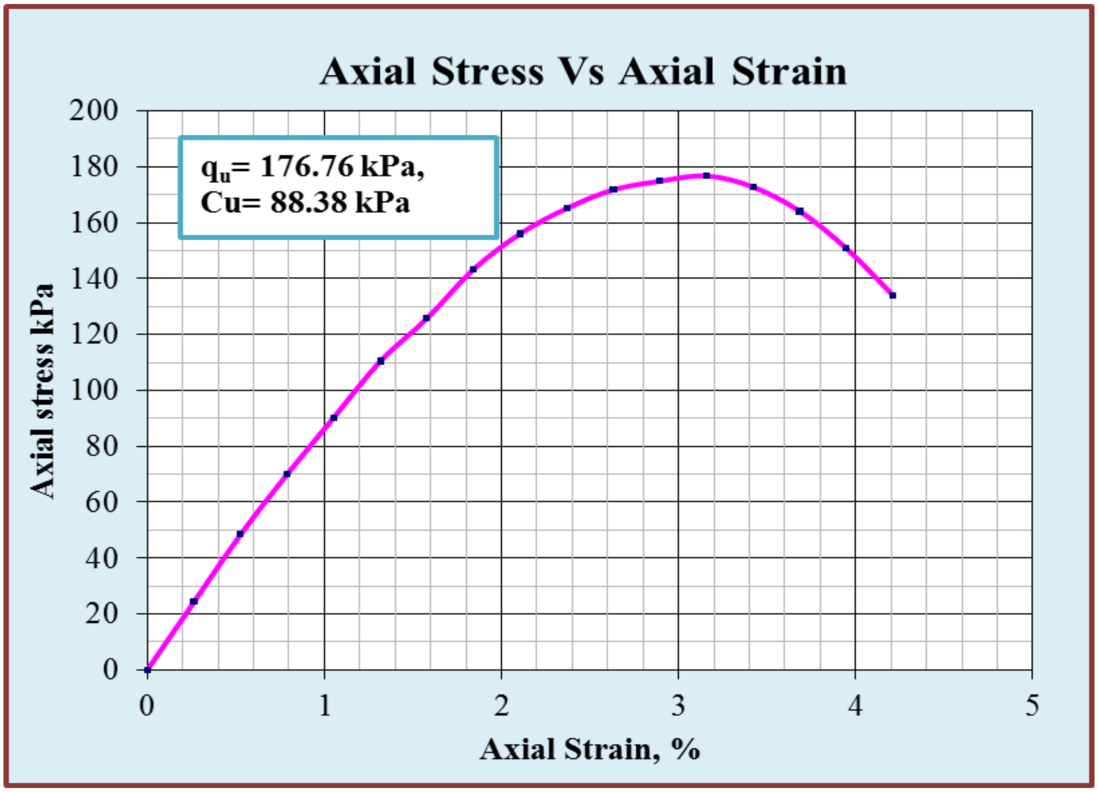

Figure 4.11: Plot of Axial stress vs Axial Strain

4.5.4 Direct Shear test

The direct shear apparatus (DSA) is one of the most widely used geotechnical testing devices. It has played an important role in the history of geotechnical engineering and in our understanding of soil mechanics. Its use dates back about 150 years when Alexandre Collin conducted direct shear tests for slope stability analyses (Skempton 1949).

Major criticisms of the direct shear box test include non-uniform stress and strain applied to the sample (Terzaghi and Peck 1948; Hvorslev 1960; Saada and Townsend 1981) and ambiguity in interpreting shear strength parameters at failure (Morgenstren and Tchalenko 1967). For these reasons, the direct shear test fell out of favor in the geotechnical community.

4.5.5 Test procedure and results

According to ASTM D 3080 - Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions a soil sample is placed in a shear box and a known load is applied in the direction normal to the shear plane. Then a shearing force is applied parallel to the shear plane. The normal load is held constant while the shearing force is increased gradually. The first increment of shearing forces cause only slight movements or shearing strain but as the test progresses a point is reached where continuous displacement at virtually constant shearing force takes place. This is an indication of failure of the sample along shear plane. The shearing force required to produce continuous movement is a measure of shearing strength of the specimen [21].

The results of direct shear test are presented in a stress- strain diagram as shown in Fig. 4.12. From a series of direct shear tests under different normal loads, a relation between normal stress and shear stress at failure as illustrated in Fig.4.13, However the angle of internal friction and cohesion of soils of study area are obtained $\phi = 27.3^0$, $C = 2.35$ (kN/m^2).This shows that the soil material near Lake Haromaya is cohesion less.

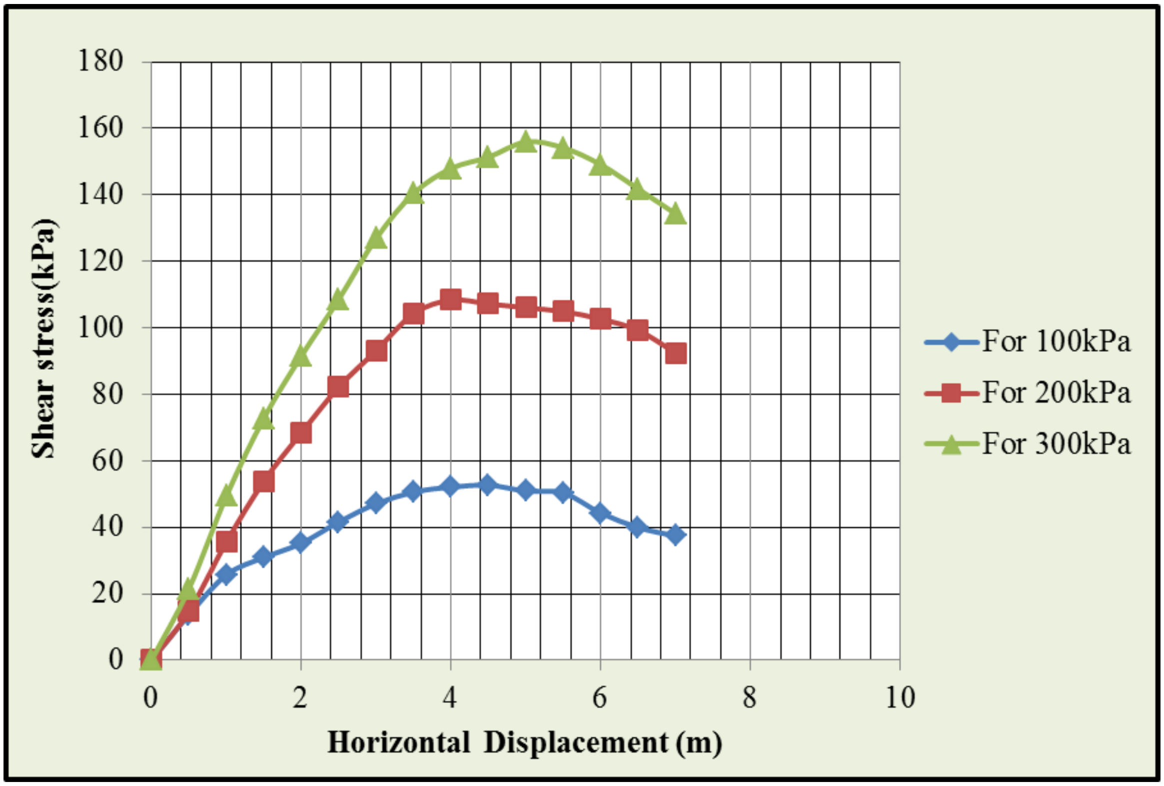

Figure 4.12: Plot of shear stress against Horizontal displacement

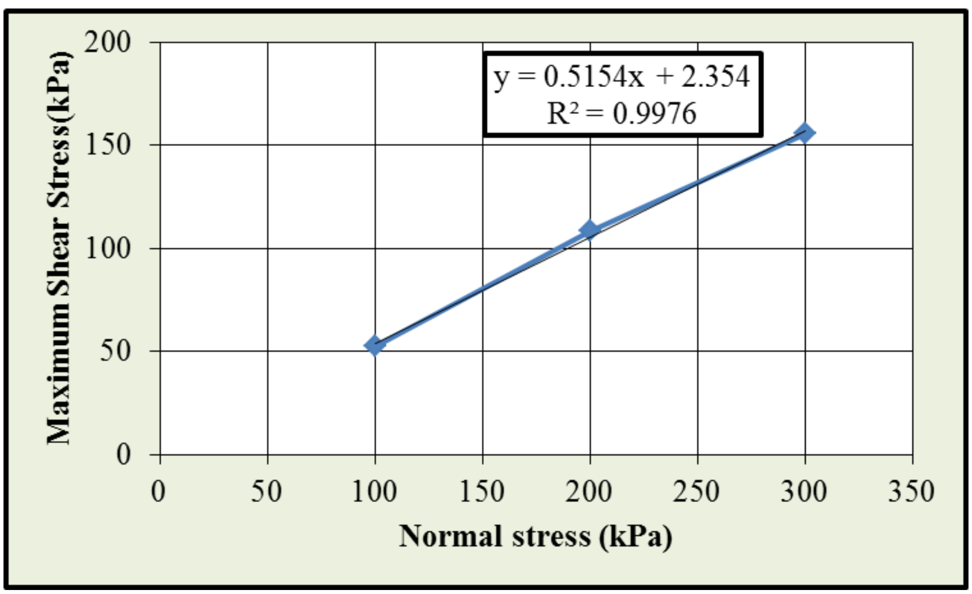

Figure 4.13: Plot of maximum shear stress against normal stress

5. DISCUSSIONS OF THE LABORATORY TEST RESULTS AND COMPARISONS WITH PREVIOUSLY DONE RESEARCHES

5.1 Discussions of the laboratory test results

The results of specific gravity test indicate that the specific gravity of soils of the study area, which is in the range of typical specific gravity values of inorganic soils [5].

The test results of Atterberg's limits indicate that some of soils of the study area are highly plastic soil with high plasticity index values. However soils from TP2 at 1.5m depth and TP7 at 3m depth are non-plastic material.

The Grain Size Analysis result is shown in Fig 4.2 and 4.3 and the summary of Grain Size Analysis result is shown on Table 4.4. The results obtained from the Grain Size Analyses indicate that the dominant proportion of soil particle in the study area is clay and sand.

The free swell test results of soils of study area are mostly in between 50% and 100% except soils from TP-3 at 1.5m, 3m depth which are typically expansive soil. Most of the soils have moderate expansive nature, which have a little impact on construction of structures.

Optimum moisture content and the maximum dry density of the study area are summarized in Table 4.6. From the test results the maximum dry density (MDD) of Haromaya from 1.39 to 1.52 g/cm^3 and the optimum moisture content ranges 13 to 23.4 percent.

Fig 4.4 shows plasticity chart of the study area according to Unified Soil Classification System. This chart shows that the soil under investigation lies above the A-line in the region of inorganic clay. That means inorganic clay with high plastic (CH) and Low to medium plastic (CL).

Classifications of soils in the study area based on AASHTO Classification system is shown in Table 4.8 and also Fig 4.5 shows plasticity chart of soil in the study area according to AASHTO system of classification. From this table and chart it can be observed that soil in the study area is classified in group A-2-4, A-6, and A-7-6.

According to AASHTO classification in Table 4.8 and Fig 4.5 shows that most of the soil of the study area falls in A-6 and A-7-6 groups except samples from TP-2 at 1.5m and TP-7 at 3m which is silty or clayey gravel and sand. A-6 and A-7-6 materials are generally considered the poorest performers with regard to roadway construction, but can be utilized in sub-grade material. Further, more group index values of the samples is calculated and indicate that 50% of the samples are below 20, which are good source of sub-grade material, other samples may need some stabilization techniques to use as source of material. The value of GI is greater than 20 for the rest of 50% samples this indicates that percentage amount of fine particles (#200) is high; this indicates the soils are highly plastic nature.

Classification of soil in the study area based on activity is shown in Table 4.10 and Fig 4.6. From this table and activity chart observed that soil of study area ranges from in active to active soil.

Figure 4.8 and 4.9 shows the plot of vertical pressure versus void ratio on semi-log and linear scale. Except their variation in initial void ratio the plot shows similar curvature for all the samples. The soil has a pre-consolidation pressure of 90-150kPa. Over-consolidation ratios of the soils are more than one, so the soil in the study area is over consolidated in its natural state.

The coefficient of consolidation, Cv, which was calculated from curve of compression dial reading versus logarithm-of-time fitting method for each incremental loading is plotted as a function of effective stress in Fig 4.10. From this figure it can be observed that, the shapes of the curves for three samples are almost similar and the same is true for value of coefficient of consolidation for each incremental loading. It was observed that the Cv value increased with increasing void ratio. The samples also broadly showed a decrease of coefficient of consolidation with increasing effective pressure.

The compression index, Cc of the soil is calculated from the slope of the straight portion of the void ratio versus log pressure plot (Figure 4.8). This calculation shows that the compression index, Cc, ranges from 0.13-0.33.

Unconfined Compressive Strength of soils of the study area is conducted for representative undisturbed samples taken at a depth of 3m. Compressive strength of soils of the study area ranges from 132.2kN/m^2-273.4kN/m^2. This shows that consistency of the soil ranges from stiff to very stiff. Results indicated that undisturbed samples, the soil particles were very close together due to the slow rate of sedimentation and compaction. The compacted structure of undisturbed soil resulted in higher Unconfined Compressive Strength.

The direct shear test results are presented in a stress- strain diagram (Figure 4.12). From a series of direct shear tests under different normal loads, a relation between normal stress and shear stress at failure as illustrated in Figure 4.13, However the angle of internal friction and cohesion of soils of study area are obtained $\phi = 27.3^0$, $C = 2.35$ (kN/m^2). This result shows that the soils of near Lake Haromaya area are cohesionless material.

5.2 Comparison of index property test results in different parts of the country

Index property test results of the present study are summarized together with ranges of values of soils in different part of country (Table 5.1).

Table 5.1: Index property test results in different parts of the country

Name of Previous Researchers	Dagnachew Debebe (2011)	Behaylu Hunde (2014)	Haile Mariam, (1992)	Fasil Abagena (2003)	Samuel (1989)			Current Research
Soil type	Silt&Silt sand	Clay&silt	Red clay	Red clay	Red clay			Clay & Silt sand
Location	Adama	Ambo	Addis Ababa	Bahirdar	kolfe	Semen Gebeya	Rufael	Haromaya
Clay Content (%)	5.4-40.5	28-67.6	48-73	74-82	58-70	53-68	50-70	1.6-65.75
Liquid Limit (%)	29-73	62-114	54-81	61-68	61-75	57-76	56-75	32-87.3
Plasticity Index (%)	5.0-34.0	30-83	21-30	24-31	30-43	33-47	29-41	14.7-63.4
Specific gravity	2.4-2.7	2.51-2.78	2.61-2.79	2.75-2.83	2.66-2.73	2.70-2.77	2.66-2.74	2.65-2.84
Free swell (%)	18-50	35-155	10.0-40.0		15-45	15-50	30-40	20-113
Activity		0.62-1.29		0.56				0-1.26
From plasticity chart	SM,ML, MH	CH,MH			CH	CH	CH	CH,CL, SM

Therefore from Table 5.1 Comparison Index Property Test Result of different part of the country Haromaya soil is dominate clay proportion with highly plasticity and most of the soils have moderate expansive nature.

5.3 Soil map of Haromaya town

5.3.1 General

Soil map of the town was prepared to give an over view of the stratification, type and location of soil and rock found in the town. Moreover, remote sensing and geographical information systems (GIS) have added different concepts and enforcements to soil classification [13].

Mapping of soils has been one of the challenging and thought provoking aspects of the soil science discipline. The process of developing a soil map forces one to understand the fundamentals of soils, how they were formed, occur across the landscape or the globe, and how they might respond to use and management. Soil mapping also aims to unravel deficiencies in our understanding of soil properties and processes both in time and space [15].

5.3.2 Soil map of the study area

Soil map of Haromaya town was prepared based on information collected from GPS data, field visual observation of different areas and laboratory tests results.

Visual observation which is made during reconnaissance survey for test pit location shows that most parts of Haromaya town are covered with red clay soil. Areas such as: Haramaya university, near 02-Keble, Gende School, Gende Mesere, around Haromaya hospital is covered with red clay soil. Area around Markos Church is covered with reddish brown clay soil. Black expansive soils are dominant around Condominium of Haromaya town. Dark gray soils are abundant around Gende Qallu and Gende Beru areas. Area near Lake Haromaya is covered with light yellowish white Quartz soil. Areas around Bate are covered with red clay and light yellowish white Silty Sand.

The method used during delineation between each types of soil in Haromaya town is physical observation (walking) between neighboring test pits and supplemented by researchers from neighboring test pits.

Test pits are excavated to a maximum depth of 3m and the vertical soil profiles of the ten test pits shows that the depth of red clay soils ranges from 0.5m-3m. Gende School, Haramaya University and Gende Mesere areas are covered with soil to a depth of 3m. For areas such as

Gendeje, near 02-Keble, Markos Church and Haromaya hospital only the top layer is covered with red clay soil.

According to soil classification the test results of study area is summarized as shown in Table 5.2 due to subdivision of soil map.

Table 5.2: Summery of test results of study area

Soil Type	Specific Gravity	Free Swell (%)	Liquid Limit(%)	Plasticity Index (%)	UCS, qu (kPa)	Pre-consolidation Pressure, (kPa)	Compression Index	Swelling Pressure(,kPa)
CH	2.7-2.84	35-75	52-72	25.9-45.5	132.2-273.4	90-150	0.13-0.33	14-20
CL	2.7-2.75	30-75	32-44.9	14.7-28				
SM	2.65-2.74	20-35						
Expansive	2.75-2.8	105-113	86.4-87.3	63.3-63.4	176.76	100	0.3	110

Therefore based on results of soil classification, this study declares the soil map of Haromaya town as shown in Fig 5.1

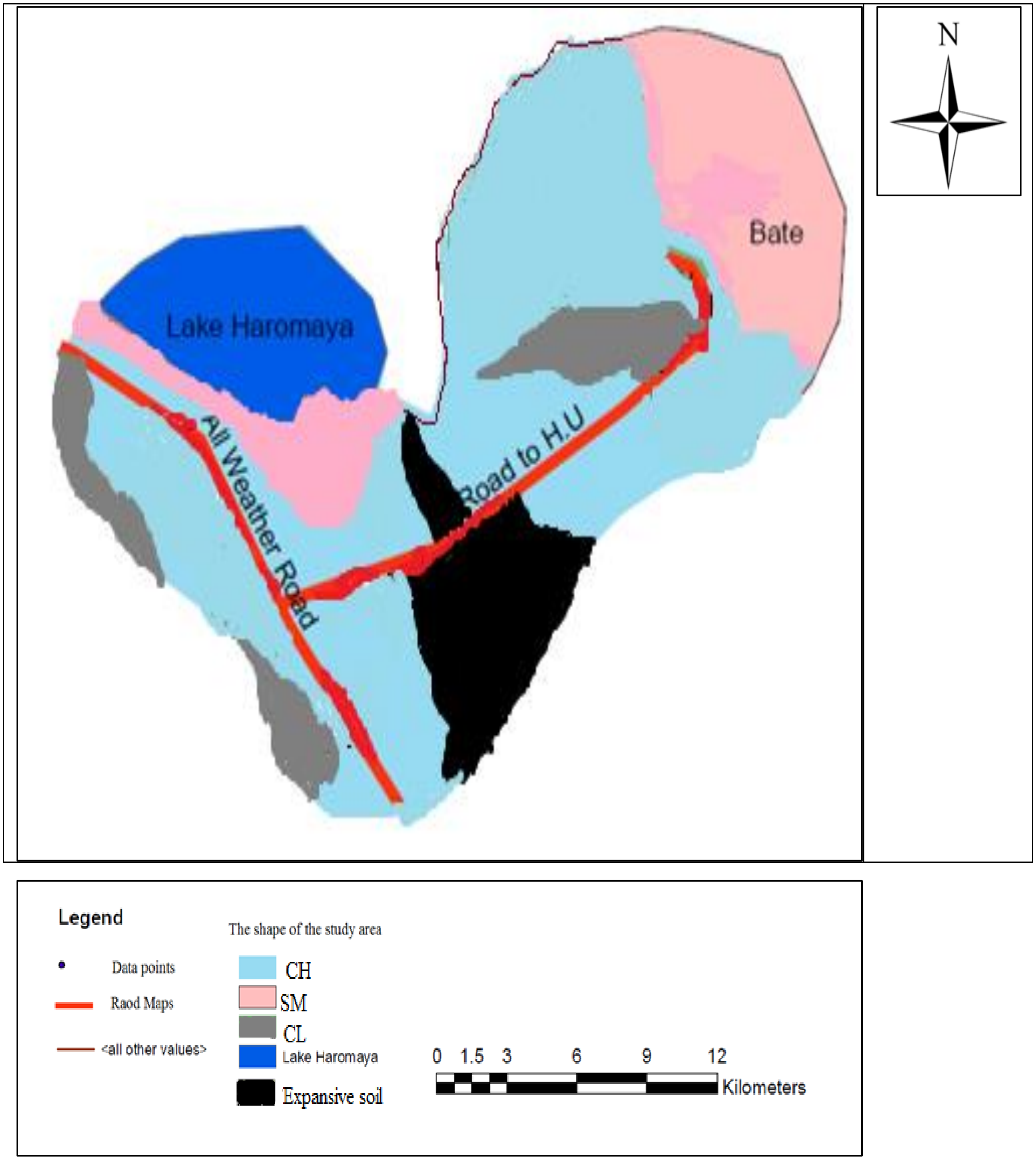

Figure 5.1: Proposed soil map of Haromaya town

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. The specific gravity of the mostly Haromaya soil is ranging from 2.65-2.84.
2. The Atterberg limit tests results showed that, except for soils around Gende School and Gende Mesere are highly plastic. However areas around Bate and near Lake Haromaya are non-plastic material.
3. The Grain Size Distribution indicates majority samples have clay material except soils around Bate and near Lake Haromaya. Therefore, clay type of soil is dominantly located in the study area.
4. Free swell test result indicates that most of the study areas have moderate expansive nature except areas around Haromaya Condominium which are typically expansive soil.
5. According to AASHTO soil classification system most of soils of study areas categorized on A-7-6 and A-6 therefore those soils have poor quality with regard to roadway construction, but can be utilized as sub-grade material.
6. The unconfined compressive strength test result ranges from 132.2kN/m^2 -273.4kN/m^2 indicates the soil consistency of the study area is range from stiff to very stiff.
7. The direct shear test result indicates that areas near Lake Haromaya are cohesionless soil material with angle of internal friction, $\phi = 27.3^0$ and cohesion, C = 2.35 (kN/m^2).
8. The one dimensional consolidation test result shows that the soil exists naturally in a condition of over-consolidated, which has over consolidation ratio is grater than one.

6.2 Recommendations

1. The method of exploration used during collecting soil samples are pit excavation, the outcomes would be applicable only for light structures which under lie their foundation up to depth of 3m.

2. It is recommended that the mineralogical composition of fine grained soil is known, through mineralogical test and its effect on the engineering properties of soils can be clearly identified.

3. In this research samples of soil were collected only from ten test pits, by increasing the number of sampling area in-depth investigation should be done in future.

7. References

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APPENDIX A Consolidation and swelling consolidation test result

A1 Consolidation and Swelling consolidation test result of TP2 at D=3m

Sample type: **Undisturbed** **Initial void ratio:** **0.6**

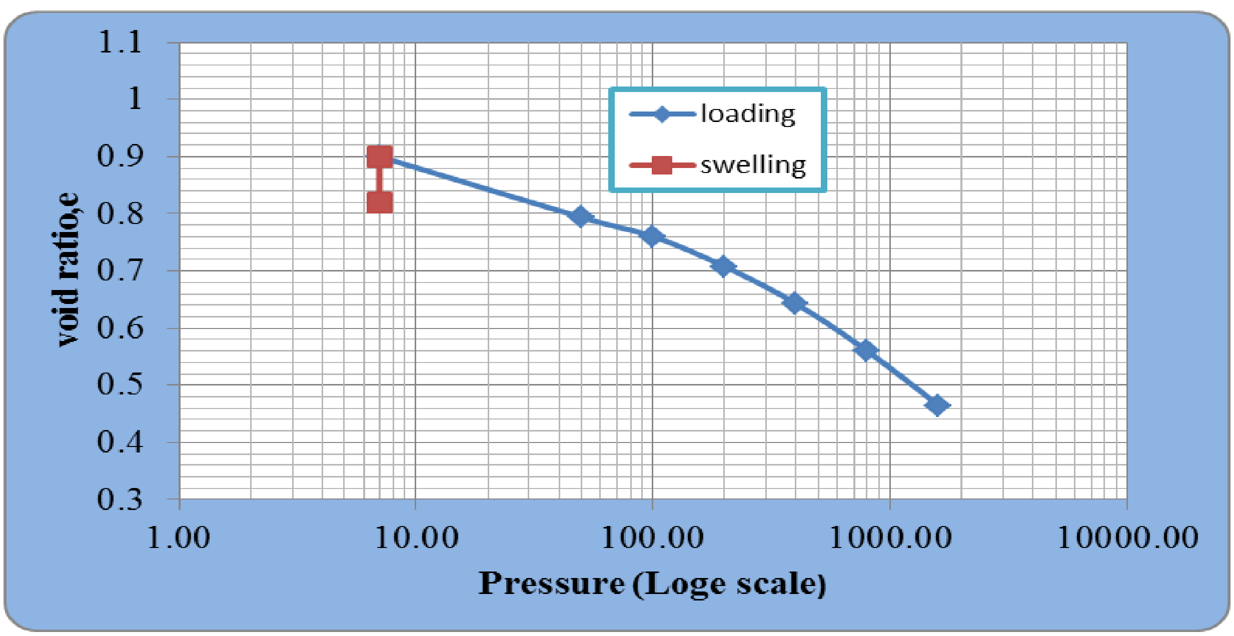
Sample height: **2cm** **Dray density, g/cm^3** **1.69**

Sample area: **19.635cm^2** **Seating Load:** **7kPa**

Initial moisture **19.0%**

Applied Pressure P (kPa)	Final dial reading (mm)	Change in Spacemen Height (ΔH)	Final Spacemen Height (mm)	Void height (mm)	Void Ratio (e)
Loading					
7.00	8.2	0	20.00	7.53	0.82
7.00	8.37	0.17	20.17	7.70	0.90
50.00	8.38	0.01	19.99	7.52	0.79
100.00	8.70	0.32	19.67	7.20	0.76
200.00	9.19	0.50	19.17	6.70	0.71
400.00	9.81	0.62	18.55	6.08	0.64
800.00	10.60	0.79	17.76	5.29	0.56
1600.00	11.50	0.90	16.86	4.39	0.46

A2 Void ratio Vs pressure (log scale) curve for TP2 at D=3m



A3 Consolidation and Swelling consolidation test result of TP3 at D=3m

Sample type: **Undisturbed** **Initial void ratio:** **1.52**

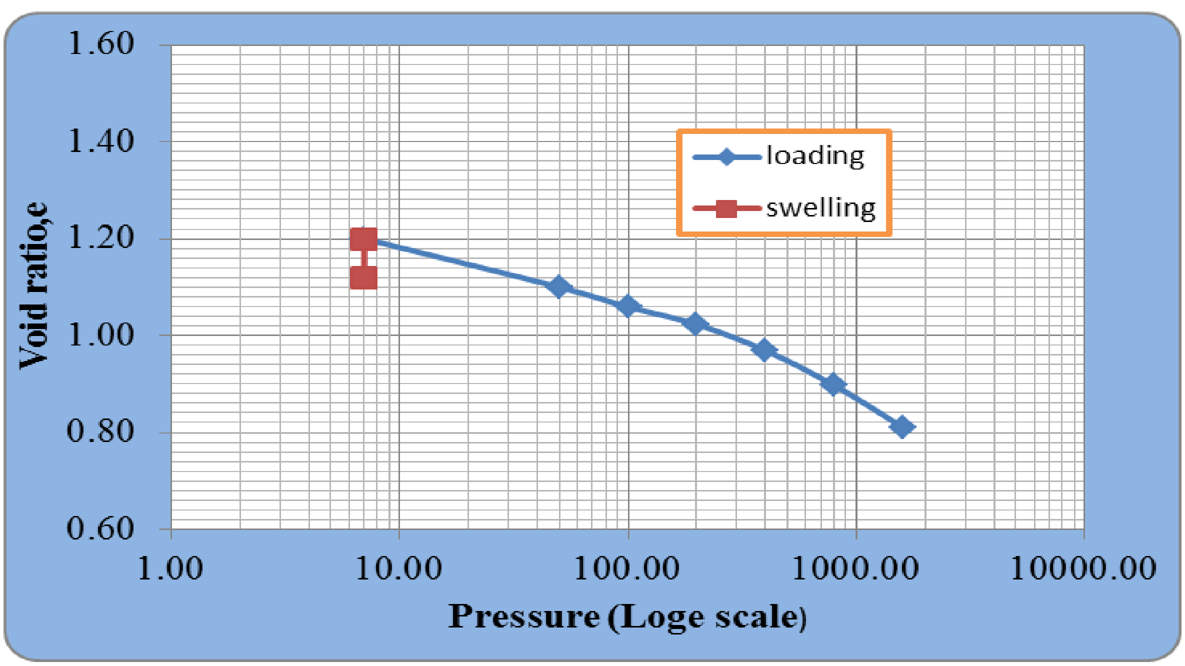
Sample height: **2cm** **Dray density, g/cm^3** **1.33**

Sample area: **19.635cm^2** **Seating Load:** **7kPa**

Initial moisture **35.9%**

Applied Pressure P (kPa)	Final dial reading (mm)	Change in Spacemen Height (ΔH)	Final Spacemen Height (mm)	Void height (mm)	Void Ratio (e)
Loading					
7.00	2.04	0	20.00	10.53	1.12
7.00	1.43	0.61	20.61	11.14	1.20
50.00	1.76	0.33	19.67	10.20	1.08
100.00	1.93	0.17	19.50	10.03	1.06
200.00	2.27	0.34	19.16	9.69	1.02
400.00	2.78	0.51	18.65	9.18	0.97
800.00	3.46	0.68	17.97	8.50	0.90
1600.00	4.29	0.82	17.15	7.68	0.81

A4 Void ratio Vs Pressure (Log scale) curve for TP3 at D =3m



A5 Consolidation and Swelling consolidation test result of TP5 at D=3m

Sample type: **Undisturbed** **Initial void ratio:** **0.5**

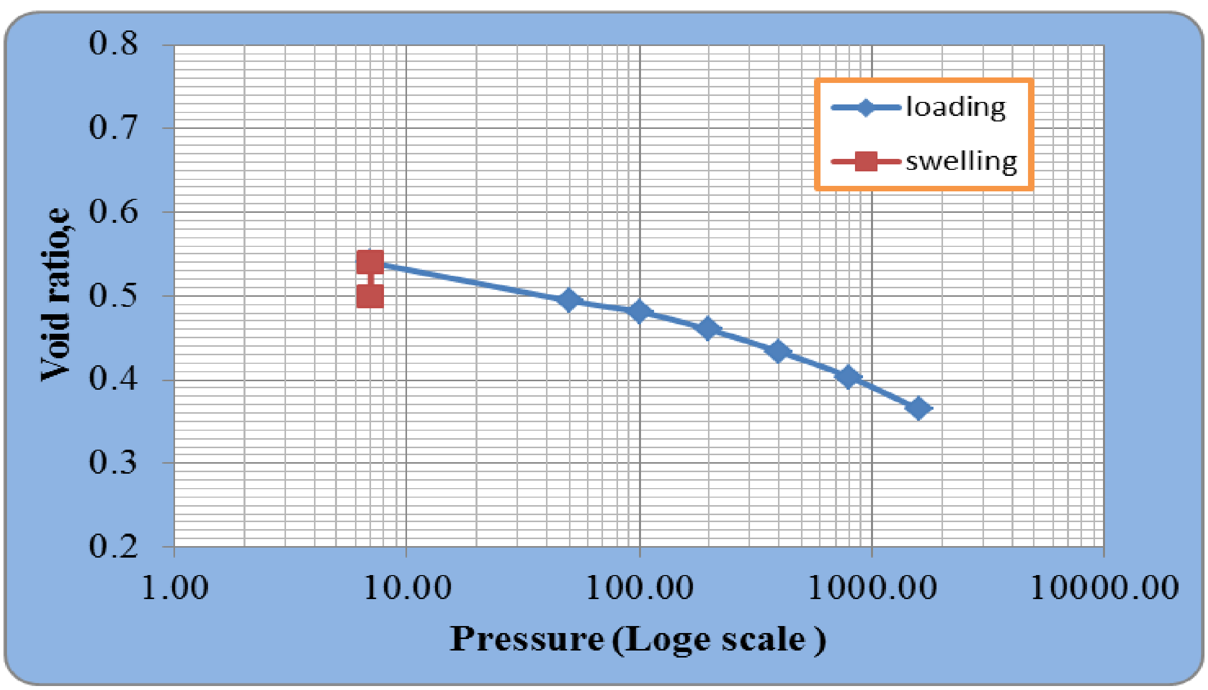
Sample height: **2cm** **Dray density, g/cm^3** **1.81**

Sample area: **19.635cm^2** **Seating Load:** **7kPa**

Initial moisture **15.89%**

Applied Pressure P (kPa)	Final dial reading (mm)	Change in Spacemen Height (ΔH)	Final Spacemen Height (mm)	Void Height (mm)	Void Ratio (e)
Loading					
7.00	6.4	0	20.00	6.63	0.50
7.00	6.52	0.12	20.12	6.75	0.54
50.00	6.60	0.08	19.92	6.55	0.49
100.00	6.72	0.12	19.80	6.43	0.48
200.00	6.85	0.14	19.66	6.29	0.47
400.00	7.37	0.50	19.16	5.79	0.43
800.00	7.78	0.40	18.76	5.39	0.40
1600.00	8.29	0.52	18.24	4.87	0.36

A6 Void ratio Vs Pressure (Log scale) curve for TP5 at D =3m



APPENDIX B Unconfined Compression test results

B1 Unconfined Compression test results of TP2 at D=3m

Test Pit No	**2**		
Test Pit Depth, m	**3.00**	**Ring Calibration Factor, kN/div**	**0.00138**
Diameter of sample, mm	**38**	**Moisture Content, %**	**21.35**
Length of sample, mm	**76**	**Wet unit weight, kN/m^3**	**18.86**

Deformation Dial reading (Division)	Sample deformation (mm)	Strain ε= (ΔL/Lo)	Axial Strain (%)	Corrected Area Ac =Ao/(1-ε) [m2]	Proving Ring Reading (division)	Axial Load [kN]	Axial Stress [kPa]
0	0	0	0	0.0011	0	0	0
20	0.2	0.003	0.263	0.0011	15	0.021	18.20
40	0.4	0.005	0.526	0.0011	25	0.035	30.26
60	0.6	0.008	0.789	0.0011	30	0.041	36.22
80	0.8	0.011	1.053	0.0011	35	0.048	42.14
100	1.0	0.013	1.316	0.0011	40	0.055	48.03
120	1.2	0.016	1.579	0.0012	44	0.061	52.69
140	1.4	0.018	1.842	0.0012	48	0.066	57.33
160	1.6	0.021	2.105	0.0012	50	0.069	59.56
180	1.8	0.024	2.368	0.0012	54	0.075	64.15
200	2.0	0.026	2.632	0.0012	58	0.080	68.72
220	2.2	0.029	2.895	0.0012	60	0.083	70.90
240	2.4	0.032	3.158	0.0012	64	0.088	75.42
260	2.6	0.034	3.421	0.0012	66	0.091	77.56
280	2.8	0.037	3.684	0.0012	70	0.097	82.04
300	3.0	0.039	3.947	0.0012	72	0.099	84.15
320	3.2	0.042	4.211	0.0012	75	0.104	87.42
340	3.4	0.045	4.474	0.0012	78	0.108	90.66
360	3.6	0.047	4.737	0.0012	80	0.110	92.73
380	3.8	0.050	5.000	0.0012	82	0.113	94.79
400	4.0	0.053	5.263	0.0012	85	0.117	97.99
420	4.2	0.055	5.526	0.0012	87	0.120	100.01
440	4.4	0.058	5.789	0.0012	90	0.124	103.17
460	4.6	0.061	6.053	0.0012	91	0.126	104.03
480	4.8	0.063	6.316	0.0012	94	0.130	107.16

B1 Unconfined Compression test results of TP2 at D=3m

Deformation Dial reading (Division)	Sample deformation (mm)	Strain ε= (ΔL/Lo)	Axial Strain (%)	Corrected Area Ac =Ao/(1-ε) [m2]	Proving Ring Reading (division)	Axial Load [kN]	Axial Stress [kPa]
500	5	0.066	6.579	0.0012	96	0.132	109.13
520	5.2	0.068	6.842	0.0012	98	0.135	111.09
540	5.4	0.071	7.105	0.0012	100	0.138	113.04
560	5.6	0.074	7.368	0.0012	102	0.141	114.97
580	5.8	0.076	7.632	0.0012	104	0.144	116.89
600	6	0.079	7.895	0.0012	106	0.146	118.80
620	6.2	0.082	8.158	0.0012	108	0.149	120.69
640	6.4	0.084	8.421	0.0012	110	0.152	122.58
660	6.6	0.087	8.684	0.0012	114	0.157	126.67
680	6.8	0.089	8.947	0.0012	116	0.160	128.52
700	7	0.092	9.211	0.0012	119	0.164	131.46
720	7.2	0.095	9.474	0.0013	120	0.166	132.18
740	7.4	0.097	9.737	0.0013	120	0.166	131.80
760	7.6	0.100	10.000	0.0013	120	0.166	131.42
780	7.8	0.103	10.263	0.0013	119	0.164	129.94
800	8	0.105	10.526	0.0013	118	0.163	128.47
820	8.2	0.108	10.789	0.0013	116	0.160	125.92
840	8.4	0.111	11.053	0.0013	114	0.157	123.38
860	8.6	0.113	11.316	0.0013	112	0.155	120.86

B2 Unconfined Compression test results of TP3 at D=3m

Test Pit No	**3**		
Test Pit Depth, m	**3.00**	**Ring Calibration Factor, kN/div**	**0.00138**
Diameter of sample, mm	**38**	**Moisture Content, %**	**19.50**
Length of sample, mm	**76**	**Wet unit weight, kN/m^3**	**17.72**

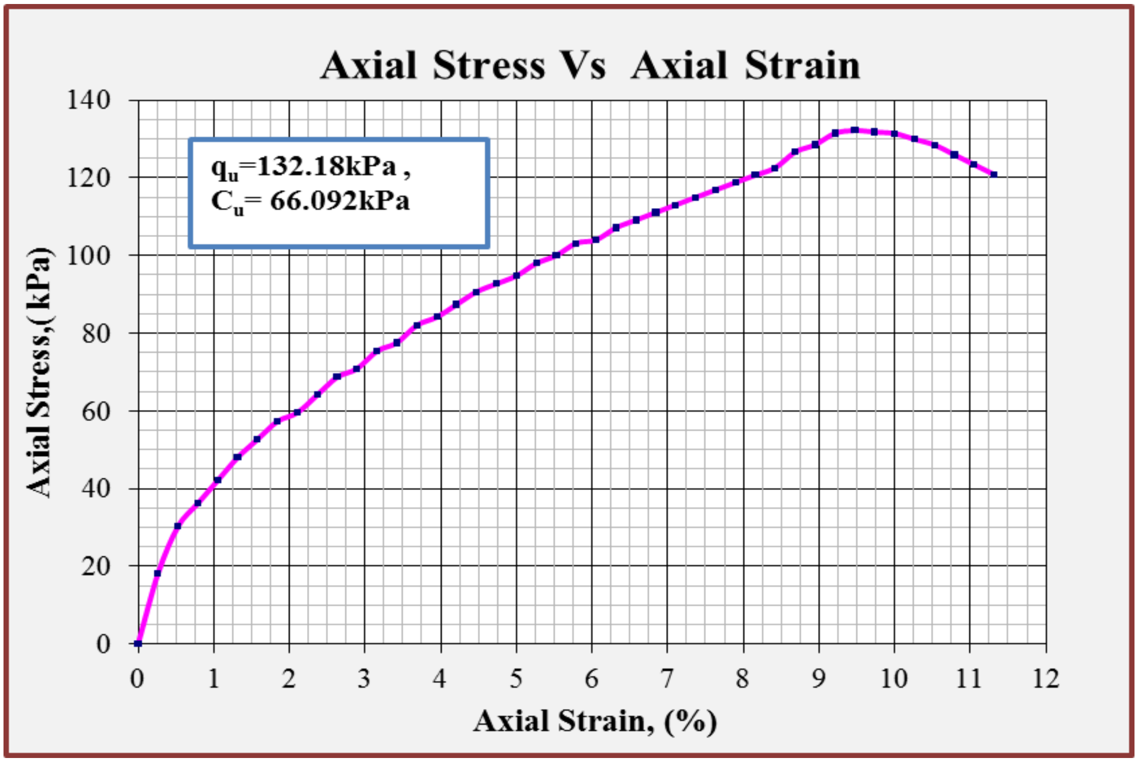
Deformation Dial reading (Division)	Sample Deformation (mm)	Strain ε= (ΔL/Lo)	Axial Strain (%)	Corrected Area Ac =Ao/(1-ε) [m2]	Proving Ring Reading (Division)	Axial Load [kN]	Axial Stress [kPa]
0	0	0	0	0.001134	0	0	0
20	0.2	0.00263	0.26316	0.001137	20	0.0276	24.27
40	0.4	0.00526	0.52632	0.001140	40	0.0552	48.42
60	0.6	0.00789	0.78947	0.001143	58	0.0800	70.02
80	0.8	0.01053	1.05263	0.001146	75	0.1035	90.30
100	1	0.01316	1.31579	0.001149	92	0.1270	110.47
120	1.2	0.01579	1.57895	0.001152	105	0.1449	125.75
140	1.4	0.01842	1.84211	0.001155	120	0.1656	143.33
160	1.6	0.02105	2.10526	0.001159	131	0.1808	156.05
180	1.8	0.02368	2.36842	0.001162	139	0.1918	165.13
200	2	0.02632	2.63158	0.001165	145	0.2001	171.79
220	2.2	0.02895	2.89474	0.001168	148	0.2042	174.87
240	2.4	0.03158	3.15789	0.001171	150	0.2070	176.76
260	2.6	0.03421	3.42105	0.001174	147	0.2029	172.75
280	2.8	0.03684	3.68421	0.001177	140	0.1932	164.08
300	3	0.03947	3.94737	0.001181	129	0.1780	150.77
320	3.2	0.04211	4.21053	0.001184	115	0.1587	134.04

B3 Unconfined Compression test results of TP5 at D=3m

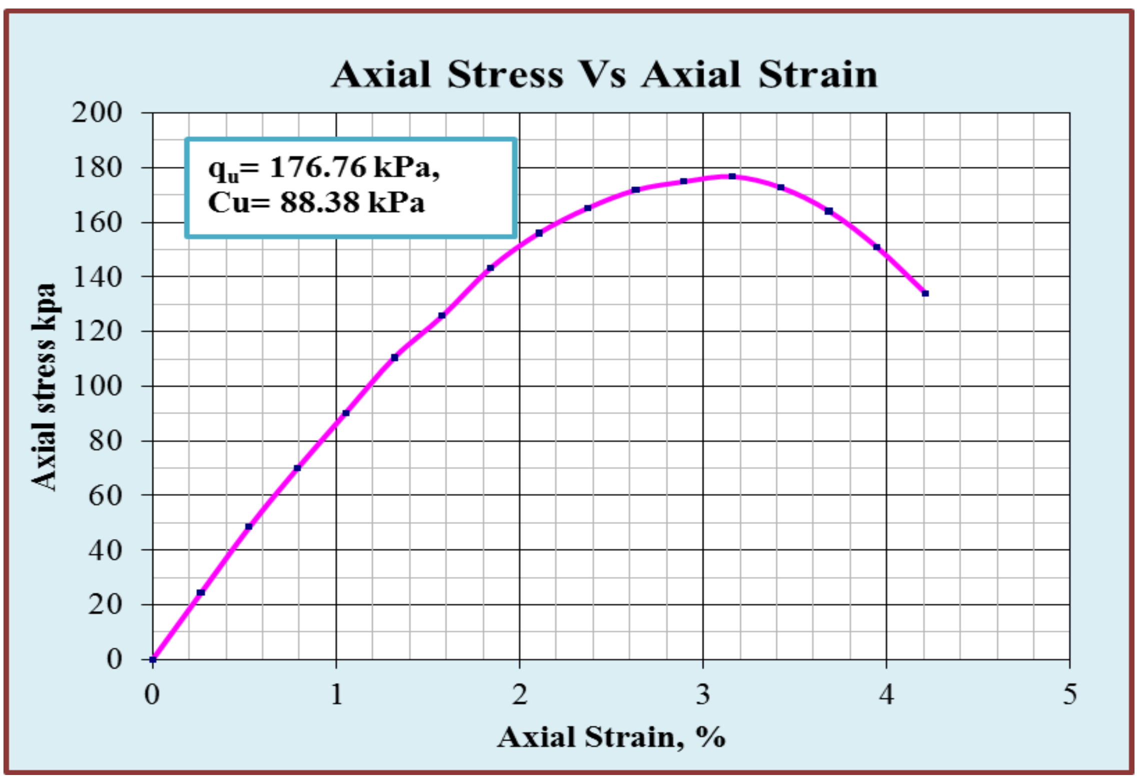
Test Pit No	**5**		
Test Pit Depth, m	**3.00**	**Ring Calibration Factor, kN/div**	**0.00138**
Diameter of sample, mm	**38**	**Moisture Content, %**	**16.91**
Length of sample, mm	**76**	**Wet unit weight, kN/m3**	**16**

Deformation Dial reading (Division)	Sample Deformation (mm)	Strain ε = (ΔL/L)	Axial Strain (%)	Corrected Area Ac =Ao/(1-ε) [m^2]	Proving Ring Reading (division)	Axial Load [kN]	Axial Stress [kPa]
0	0	0	0	0.00113	0	0	0
20	0.2	0.003	0.263	0.00114	35	0.048	42.48
40	0.4	0.005	0.526	0.00114	50	0.069	60.52
60	0.6	0.008	0.789	0.00114	60	0.083	72.43
80	0.8	0.011	1.053	0.00115	72	0.099	86.69
100	1	0.013	1.316	0.00115	82	0.113	98.47
120	1.2	0.016	1.579	0.00115	95	0.131	113.77
140	1.4	0.018	1.842	0.00116	110	0.152	131.38
160	1.6	0.021	2.105	0.00116	115	0.159	136.99
180	1.8	0.024	2.368	0.00116	120	0.166	142.56
200	2	0.026	2.632	0.00116	130	0.179	154.02
220	2.2	0.029	2.895	0.00117	145	0.200	171.33
240	2.4	0.032	3.158	0.00117	155	0.214	182.65
260	2.6	0.034	3.421	0.00117	160	0.221	188.03
280	2.8	0.037	3.684	0.00118	170	0.235	199.24
300	3	0.039	3.947	0.00118	180	0.248	210.38
320	3.2	0.042	4.211	0.00118	190	0.262	221.46
340	3.4	0.045	4.474	0.00119	195	0.269	226.66
360	3.6	0.047	4.737	0.00119	205	0.283	237.63
380	3.8	0.050	5.000	0.00119	210	0.290	242.75
400	4	0.053	5.263	0.00120	220	0.304	253.61
420	4.2	0.055	5.526	0.00120	225	0.311	258.65
440	4.4	0.058	5.789	0.00120	230	0.317	263.66
460	4.6	0.061	6.053	0.00121	235	0.324	268.64
480	4.8	0.063	6.316	0.00121	240	0.331	273.59
500	5	0.066	6.579	0.00121	240	0.331	272.82
520	5.2	0.068	6.842	0.00122	235	0.324	266.38
540	5.4	0.071	7.105	0.00122	230	0.317	259.98
560	5.6	0.074	7.368	0.00122	225	0.311	253.61
580	5.8	0.076	7.632	0.00123	220	0.304	247.27

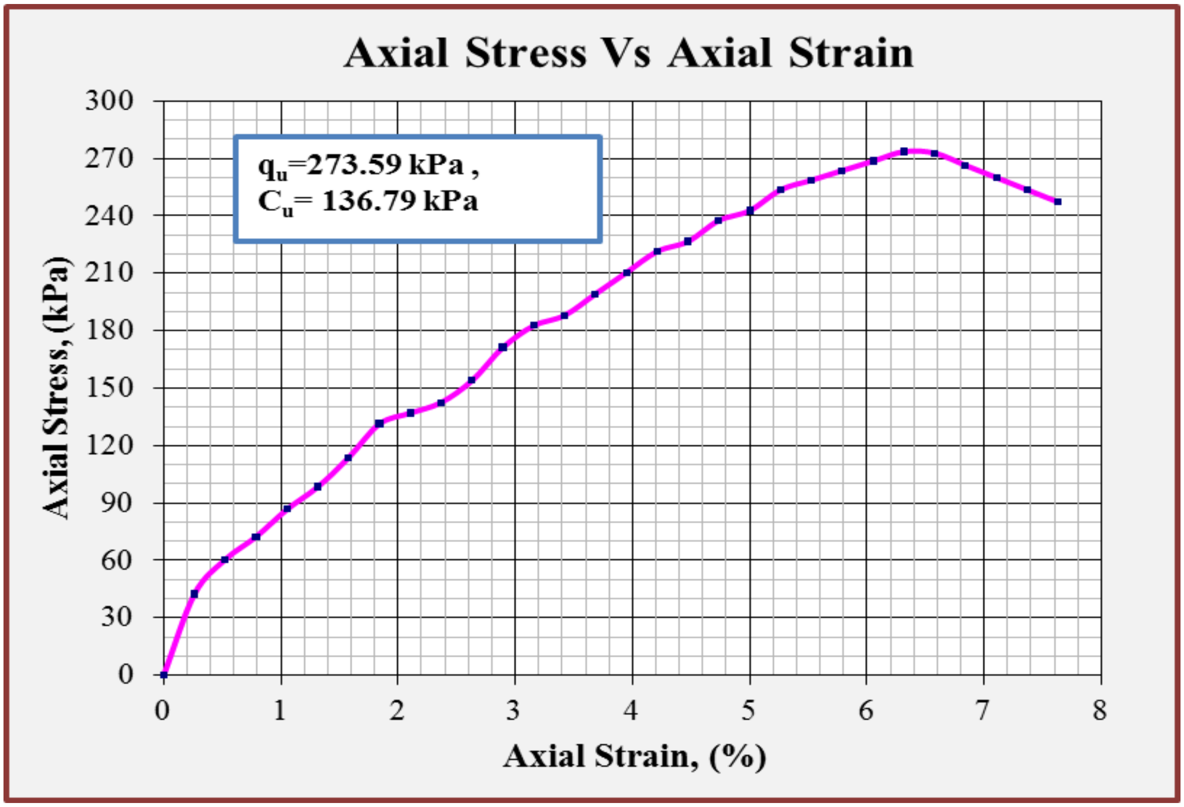
B4 Axial Strain Vs Axial Stress Curve for TP2 at D =3m



B5 Axial Strain Vs Axial Stress Curve for TP3 at D =3m



B6 Axial Strain Vs Axial Stress Curve for TP5 at D =3m



Appendix C Grain size analysis results

C1Grian size analysis results of TP4 at D = 1.5m

Sieve Analysis **Total mass of sample, g 1200**

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained Soil (g)	Mass of Retained Soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1217.3	1217.3	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1211.4	33.0	2.8	2.8	97.3
1/2"	12.5	459.4	489.0	29.6	2.5	5.2	94.8
3.8"	9.5	1164.6	1166.0	1.4	0.1	5.3	94.7
No 4	4.75	1265.9	1284.2	18.3	1.5	6.9	93.1
No 8	2.36	989.8	1002.8	13.0	1.1	7.9	92.1
No 10	2	955.8	966.1	10.3	0.9	8.8	91.2
No 16	1.18	894.4	951.6	57.2	4.8	13.6	86.4
No 30	0.6	830.9	959.0	128.1	10.7	24.2	75.8
N0 40	0.425	786.3	837.0	50.7	4.2	28.5	71.5
No 50	0.3	750.0	788.1	38.1	3.2	31.6	68.4
No 100	0.15	777.7	838.5	60.8	5.1	36.7	63.3
No 200	0.075	763.1	796.5	33.4	2.8	39.5	60.5
Pan	-----	735.0	735.9	0.9	0.1	39.6	-------

Hydrometer Analysis **Specific Gravity of soil 2.73**

Elapsed Time (mm)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
0.75	1.0210	-0.0023	1.0233	10.75	0.01301	0.0492	73.54	51.50
1	1.0205	-0.0023	1.0228	10.88	0.01301	0.0429	72.20	48.54
2	1.0205	-0.0023	1.0228	10.88	0.01301	0.0303	71.96	44.54
4	1.0205	-0.0023	1.0228	10.88	0.01301	0.0215	71.90	43.51
8	1.0200	-0.0023	1.0223	11.01	0.01301	0.0153	70.38	42.59
15	1.0200	-0.0023	1.0223	11.01	0.01301	0.0111	70.38	42.59
30	1.0195	-0.0023	1.0218	11.14	0.01301	0.0079	68.80	41.63
60	1.0190	-0.0023	1.0213	11.27	0.01301	0.0056	67.22	40.68
120	1.0185	-0.0023	1.0208	11.41	0.01301	0.0040	65.65	39.72
240	1.0180	-0.0023	1.0203	11.54	0.01301	0.0029	64.07	38.77
480	1.0175	-0.0023	1.0198	11.67	0.01301	0.0020	62.49	37.81
1440	1.0175	-0.0023	1.0198	11.67	0.01301	0.0012	62.49	37.81

C2 Grain size analysis results of TP4 at D = 3.0m

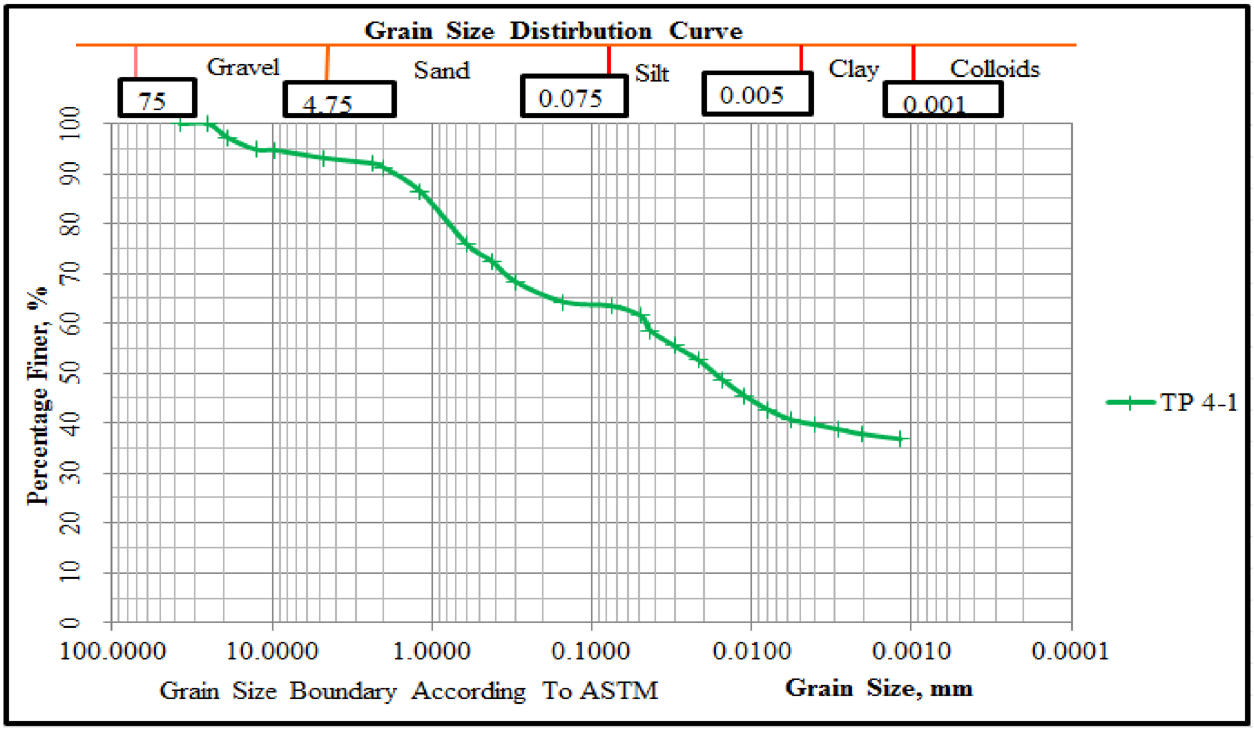
Sieve Analysis **Total mass of sample, g 1200**

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained Soil (g)	Mass of Retained Soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1217.3	1217.3	0.0	0.0	0.0	100.0
3/4"	19.0	1178.8	1178.8	0.0	0.0	0.0	100.0
1/2"	12.5	467.0	473.8	6.8	0.6	0.6	99.4
3.8"	9.5	1164.6	1164.6	0.0	0.0	0.6	99.4
No 4	4.75	1265.7	1267.8	2.1	0.2	0.7	99.3
No 8	2.36	989.8	993.4	3.6	0.3	1.0	99.0
No 10	2	955.8	959.8	4.0	0.3	1.4	98.6
No 16	1.18	894.4	915.6	21.2	1.8	3.1	96.9
No 30	0.6	830.9	879.5	48.6	4.1	7.2	92.8
No 40	0.425	786.3	807.7	21.4	1.8	9.0	91.0
No 50	0.3	750.0	766.1	16.1	1.3	10.3	89.7
No 100	0.15	777.7	804.6	26.9	2.2	12.6	87.4
No 200	0.075	763.1	785.1	22.0	1.8	14.4	85.61
Pan	------	735.5	736.6	1.1	0.1	14.5	-------

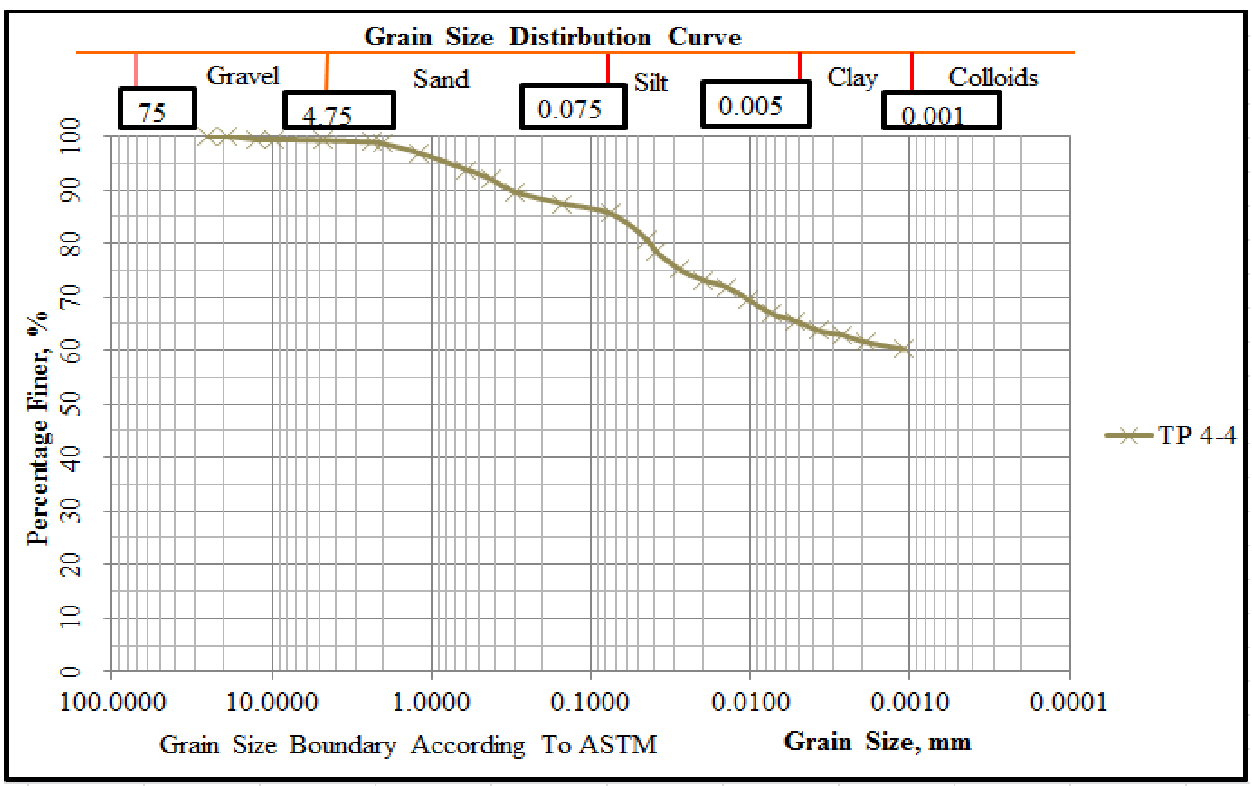
Hydrometer Analysis **Specific Gravity of soil 2.73**

Elapsed Time (mm)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
0.75	1.0260	-0.0023	1.0283	9.42	0.012612	0.0447	87.36	74.79
1	1.0255	-0.0023	1.0278	9.55	0.012612	0.0390	85.82	73.47
2	1.0250	-0.0023	1.0273	9.69	0.012612	0.0278	84.27	72.15
4	1.0250	-0.0023	1.0273	9.69	0.012612	0.0196	84.27	72.15
8	1.0245	-0.0023	1.0268	9.82	0.012612	0.0140	82.73	70.82
15	1.0240	-0.0023	1.0263	9.95	0.012612	0.0103	81.19	69.50
30	1.0230	-0.0023	1.0253	10.22	0.012612	0.0074	78.10	66.86
60	1.0225	-0.0023	1.0248	10.35	0.012612	0.0052	76.56	65.54
120	1.0218	-0.0023	1.0241	10.53	0.012612	0.0037	74.40	63.69
240	1.0215	-0.0023	1.0238	10.61	0.012612	0.0027	73.47	62.90
480	1.0210	-0.0023	1.0233	10.75	0.012612	0.0019	71.93	61.57
1440	1.0205	-0.0023	1.0228	10.88	0.012612	0.0011	70.38	60.25

C3 Grain size distribution curve for TP4 at D =1.5m



C4 Grain size distribution curve for TP4 at D =3.0m



C5 Grain size analysis results of TP8 at D = 1.5m

Sieve Analysis **Total mass of sample, g 1200**

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained Soil (g)	Mass of Retained Soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1217.3	1217.3	0.0	0.0	0.0	100.0
3/4"	19.0	1178.4	1178.4	0.0	0.0	0.0	100.0
1/2"	12.5	466.8	466.8	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1165.2	0.6	0.1	0.1	100.0
No 4	4.75	1265.9	1280.8	14.9	1.2	1.3	98.7
No 8	2.36	989.8	1037.5	47.7	4.0	5.3	94.7
No 10	2	955.8	982.7	26.9	2.2	7.5	92.5
No 16	1.18	894.4	988.7	94.3	7.9	15.4	84.6
No 30	0.6	830.9	979.2	148.3	12.4	27.7	72.3
N0 40	0.425	786.3	855.9	69.6	5.8	33.5	66.5
No 50	0.3	750.0	813.2	63.2	5.3	38.8	61.2
No 100	0.15	777.7	899.2	121.5	10.1	48.9	51.1
No 200	0.075	763.1	840.0	76.9	6.4	55.3	44.7
Pan	------	735.5	738.9	3.4	0.3	55.6	-------

Hydrometer Analysis **Specific Gravity of soil 2.7**

Elapsed Time (mm)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
0.75	1.0230	-0.0023	1.0207	10.22	0.01317	0.0486	65.75	36.38
1	1.0225	-0.0023	1.0202	10.35	0.01318	0.0424	64.16	32.67
2	1.0220	-0.0023	1.0197	10.48	0.01312	0.0300	62.58	27.96
4	1.0215	-0.0023	1.0192	10.61	0.01308	0.0213	60.99	27.25
8	1.0210	-0.0023	1.0187	10.75	0.01309	0.0152	59.40	26.54
15	1.0205	-0.0023	1.0182	10.88	0.01315	0.0112	57.81	25.83
30	1.0200	-0.0023	1.0177	11.01	0.01314	0.0080	56.22	25.12
60	1.0190	-0.0023	1.0167	11.27	0.01313	0.0057	53.05	23.70
120	1.0190	-0.0023	1.0167	11.27	0.01312	0.0040	53.05	23.70
240	1.0185	-0.0023	1.0162	11.41	0.01294	0.0028	51.46	22.99
480	1.0180	-0.0023	1.0157	11.54	0.01273	0.0020	49.87	22.28
1440	1.0170	-0.0023	1.0147	11.80	0.01342	0.0012	46.69	20.86

C6 Grain size analysis results of TP8 at D = 3.0m

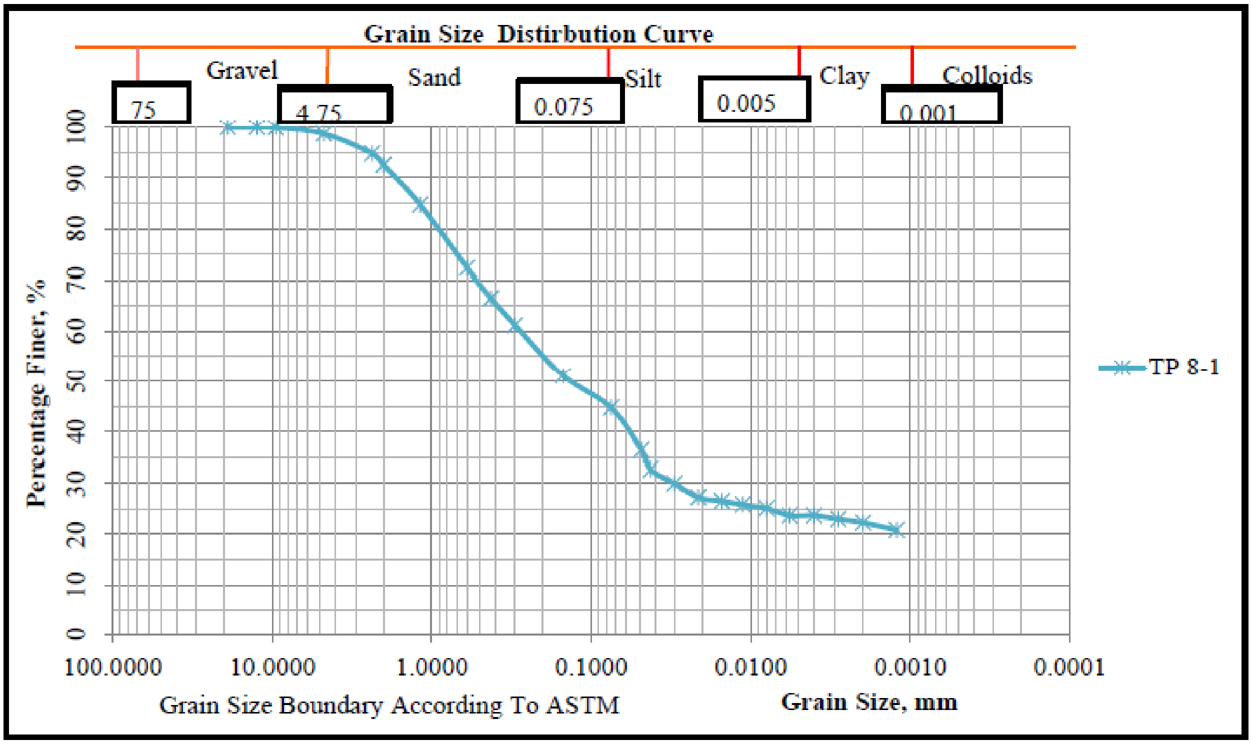
Sieve Analysis **Total mass of sample, g 1200**

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained Soil (g)	Mass of Retained Soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1217.3	1217.3	0.0	0.0	0.0	100.0
3/4"	19.0	1178.8	1178.8	0.0	0.0	0.0	100.0
1/2"	12.5	467.0	467.0	0.0	0.0	0.0	100.0
3.8"	9.5	1164.6	1168.7	4.1	0.3	0.3	99.7
No 4	4.75	1262.9	1274.0	11.1	0.9	1.3	98.7
No 8	2.36	989.8	1038.5	48.7	4.1	5.3	94.7
No 10	2	955.8	988.2	32.4	2.7	8.0	92.0
No 16	1.18	894.4	1042.0	147.6	12.3	20.3	79.7
No 30	0.6	830.9	1023.2	192.3	16.0	36.4	63.7
No 40	0.425	786.3	860.1	73.8	6.2	42.5	57.5
No 50	0.3	750.0	810.4	60.4	5.0	47.5	52.5
No 100	0.15	777.7	892.8	115.1	9.6	57.1	42.9
No 200	0.075	763.1	829.1	66.0	5.5	62.6	37.38
Pan	-------	735.0	742.3	7.3	0.6	63.2	-------

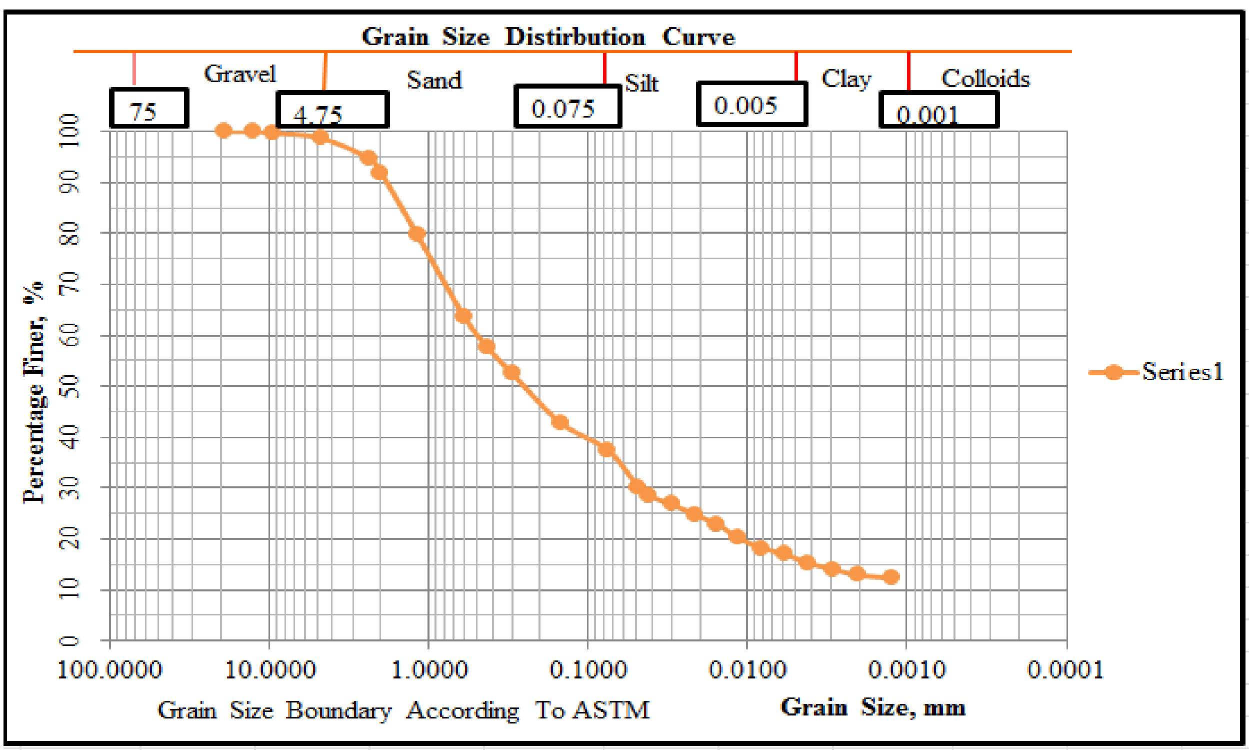
Hydrometer Analysis **Specific Gravity of soil 2.75**

Elapsed Time (mm)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
0.75	1.0230	-0.0025	1.0205	10.22	0.01314	0.0485	64.43	24.08
1	1.0225	-0.0025	1.0200	10.35	0.01312	0.0422	62.86	23.49
2	1.0220	-0.0025	1.0195	10.48	0.01312	0.0300	61.29	22.91
4	1.0210	-0.0025	1.0185	10.75	0.01315	0.0216	58.14	21.73
8	1.0195	-0.0025	1.0170	11.14	0.01317	0.0155	53.43	19.97
15	1.0180	-0.0025	1.0155	11.54	0.01312	0.0115	48.71	18.21
30	1.0180	-0.0025	1.0155	11.54	0.01311	0.0081	48.71	18.21
60	1.0170	-0.0025	1.0145	11.80	0.01317	0.0058	45.57	17.03
120	1.0155	-0.0025	1.0130	12.20	0.01306	0.0042	40.86	15.27
240	1.0145	-0.0025	1.0120	12.46	0.01293	0.0029	37.71	14.10
480	1.0135	-0.0025	1.0110	12.73	0.01266	0.0021	34.57	12.92
1440	1.0130	-0.0025	1.0105	12.86	0.01327	0.0013	33.00	12.33

C7 Grain size distribution curve for TP8 at D =1.5m



C8 Grain size distribution curve for TP8 at D =3.0m



C9 Grain size analysis results of TP10 at D = 3.0m

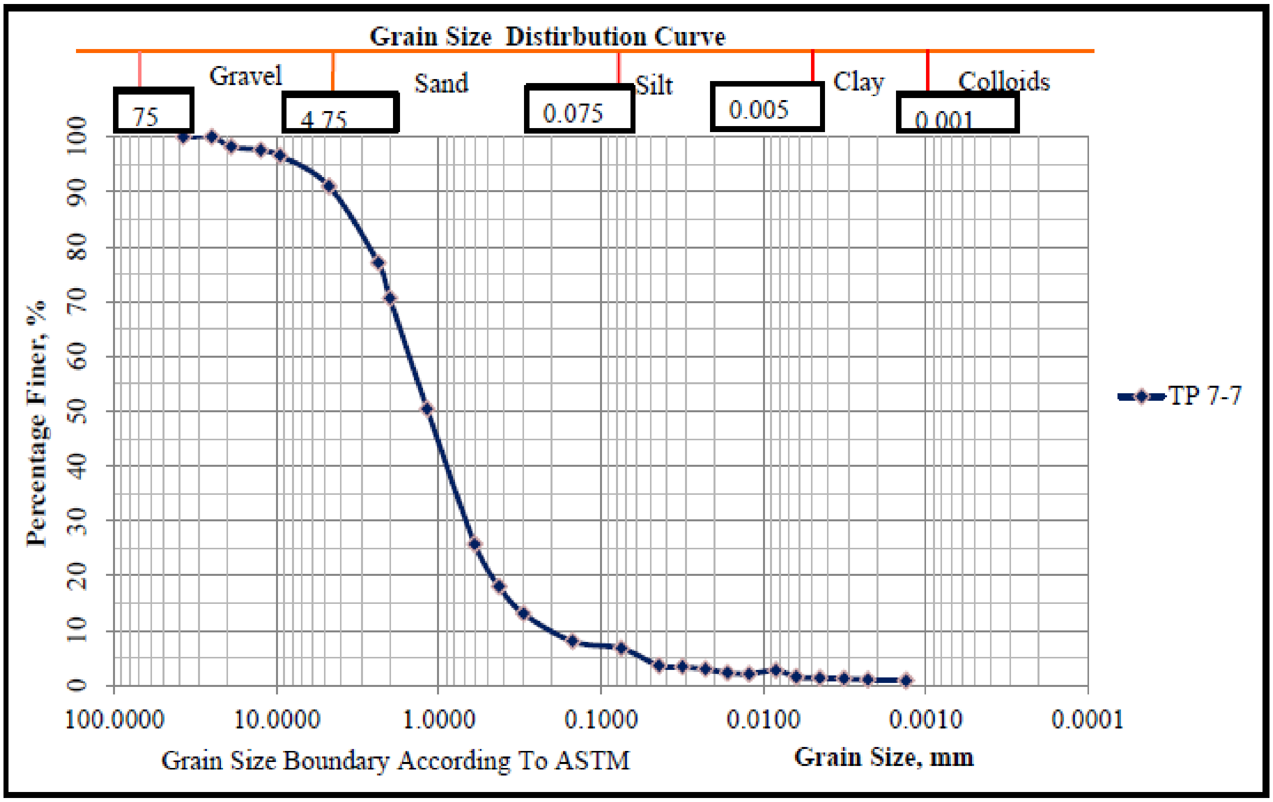
Sieve Analysis **Total mass of sample, g 1200**

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3"	75.0	1057.0	1057.0	0.0	0.0	0.0	100.0
2"	50.0	1199.0	1199.0	0.0	0.0	0.0	100.0
1.5"	37.5	1084.0	1084.0	0.0	0.0	0.0	100.0
1"	25.0	1217.3	1217.3	0.0	0.0	0.0	100.0
3/4"	19.0	1194.6	1216.0	21.4	1.8	1.8	98.2
1/2"	12.5	1163.9	1171.3	7.4	0.6	2.4	97.6
3.8"	9.5	1170.7	1183.0	12.3	1.0	3.4	96.6
No 4	4.75	1263.6	1330.8	67.2	5.6	9.0	91.0
No 8	2.36	383.3	550.9	167.6	14.0	23.0	77.0
No 10	2	955.4	1032.8	77.4	6.5	29.4	70.6
No 16	1.18	895.6	1137.8	242.2	20.2	49.6	50.4
No 30	0.6	831.8	1127.4	295.6	24.6	74.3	25.7
No 40	0.425	788.7	881.6	92.9	7.7	82.0	18.0
No 50	0.3	751.2	810.2	59.0	4.9	86.9	13.1
No 100	0.15	778.4	838.9	60.5	5.0	92.0	8.0
No 200	0.075	815.1	830.8	15.7	1.3	93.3	6.73
Pan	-------	735.6	748.6	13.0	1.1	94.4	-------

Hydrometer Analysis **Specific Gravity of soil 2.65**

Elapsed Time (mm)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
¾	1.0190	-0.0023	1.0167	11.27	0.0131	0.0440	53.64	3.61
1	1.0180	-0.0023	1.0157	11.54	0.0131	0.0315	50.43	3.40
2	1.0160	-0.0023	1.0137	12.07	0.0131	0.0228	44.01	2.96
4	1.0130	-0.0023	1.0107	12.86	0.0131	0.0166	34.37	2.31
8	1.0120	-0.0023	1.0097	13.13	0.0131	0.0123	31.16	2.10
15	1.0150	-0.0023	1.0127	12.33	0.0131	0.0084	40.79	2.75
30	1.0095	-0.0023	1.0072	13.79	0.0131	0.0063	23.13	1.56
60	1.0085	-0.0023	1.0062	14.05	0.0131	0.0045	19.92	1.34
120	1.0080	-0.0023	1.0057	14.18	0.0131	0.0032	18.31	1.23
240	1.0070	-0.0023	1.0047	14.45	0.0131	0.0023	15.10	1.02
480	1.0065	-0.0023	1.0042	14.58	0.0131	0.0013	13.49	0.91
1440	1.0060	-0.0023	1.0037	14.71	0.0131	0.0013	11.88	0.80

C10 Grain size distribution curve for TP7 at D =3.0m

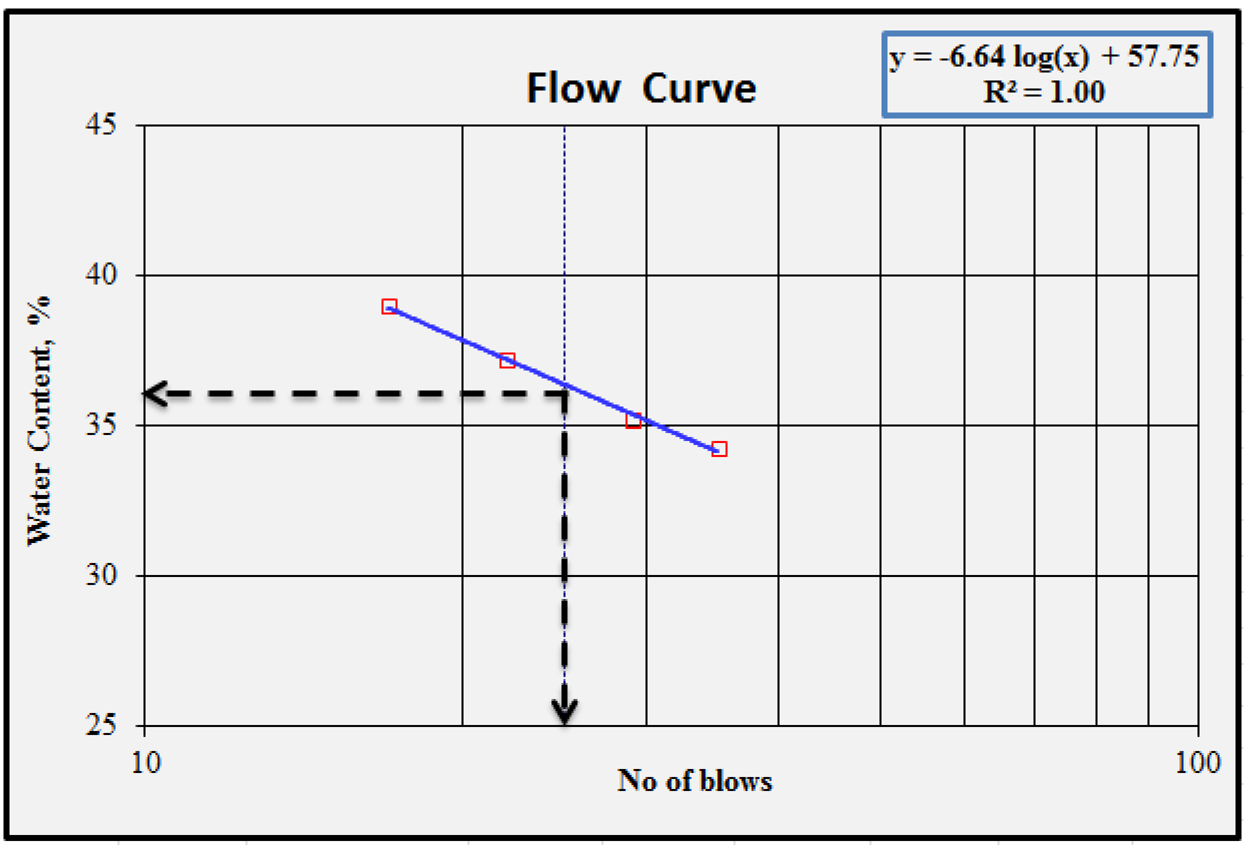


Appendix D Liquid Limit and Plastic Limit test results

D1 Liquid Limit and Plastic Limit test result of TP 6 at D = 1.5m

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A1	A2	A3	A4	A11	A12
Mass of container, g	15.50	15.60	15.90	15.80	15.70	15.70
Mass of container + Wet soil, g	38.60	37.10	37.30	38.60	22.50	22.20
Mass of container + Dry soil, g	32.70	31.50	31.50	32.20	21.40	21.30
Mass of water, g	5.90	5.60	5.80	6.40	1.10	0.90
Mass of dry soil, g	17.20	15.90	15.60	16.40	5.70	5.60
Water content, %	34.3	35.2	37.2	39.0	19.30	16.07
No of blows	35	29	22	17	------	------

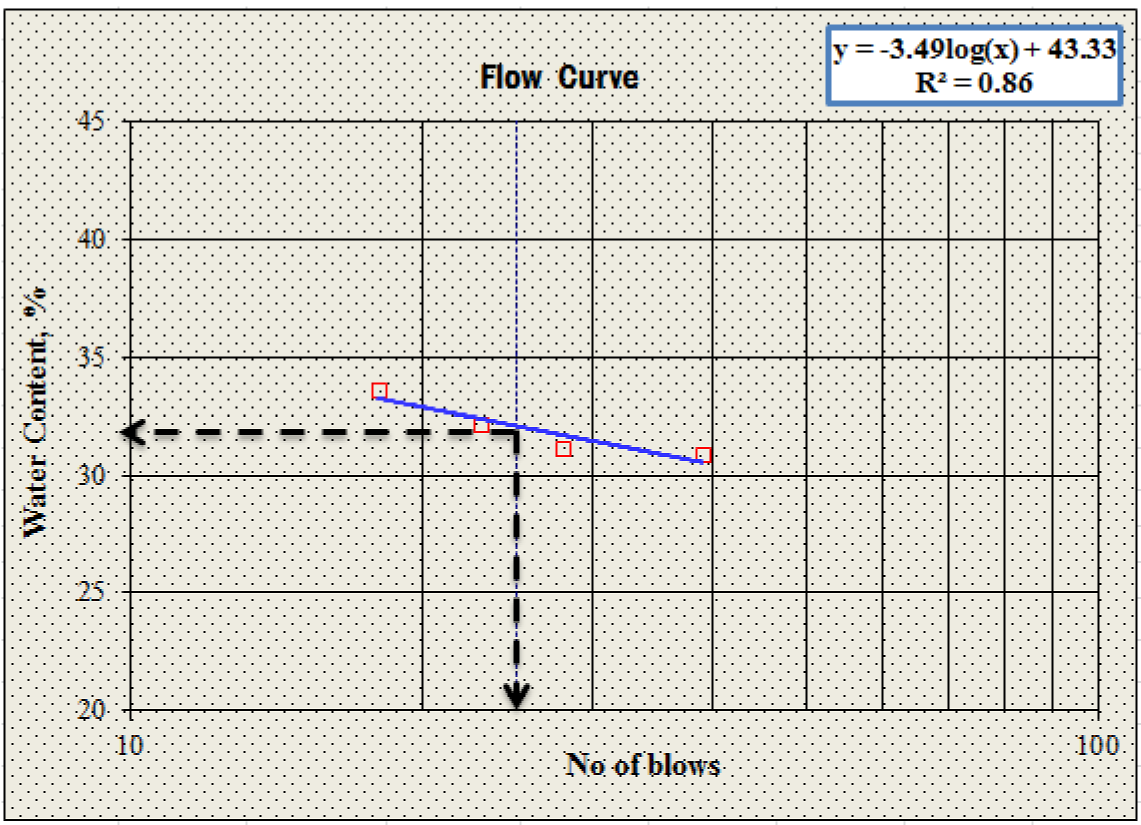
D2 Water Content Vs No of Blows for TP6 at D = 1.5m



D3 Liquid Limit and Plastic Limit test result of TP 6 at D = 3.0m

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	K11	K12	K13	K14	K15	K16
Mass of container, g	15.80	15.60	15.50	15.80	15.50	15.70
Mass of container + Wet soil, g	35.70	35.80	30.70	31.30	24.50	23.60
Mass of container + Dry soil, g	31.00	31.00	27.00	27.40	23.20	22.40
Mass of water, g	4.70	4.80	3.70	3.90	1.30	1.20
Mass of dry soil, g	15.20	15.40	11.50	11.60	7.70	6.70
Water content, %	30.9	31.2	32.2	33.6	16.88	17.91
No of blows	39	28	23	18	------	------

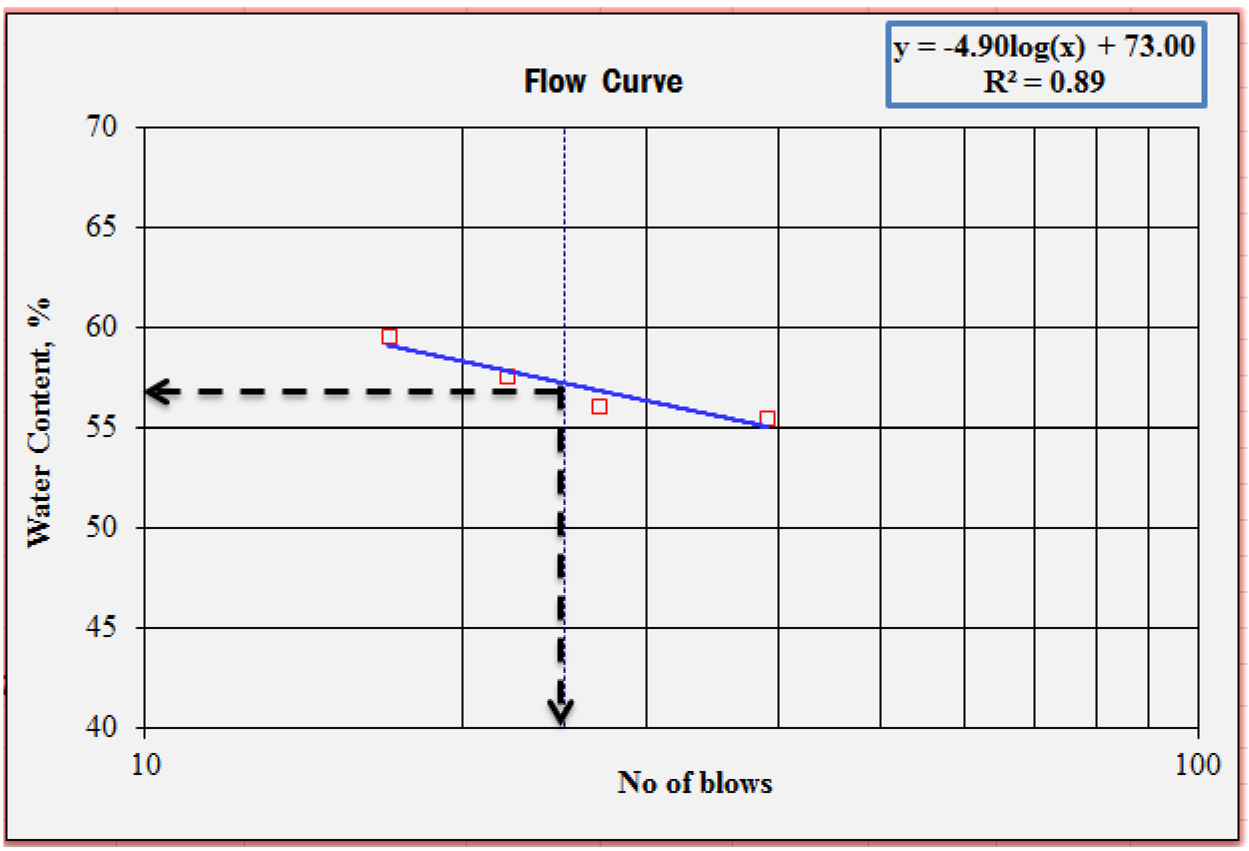
D4 Liquid Limit and Plastic Limit test result of TP 6 at D = 3.0m



D5 Liquid Limit and Plastic Limit test result of TP 9 at D = 1.5m

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	B1	B2	B3	B4	B11	B12
Mass of container, g	15.60	15.60	15.80	15.60	15.50	15.70
Mass of container + Wet soil, g	31.00	30.90	30.30	33.00	21.90	22.80
Mass of container + Dry soil, g	25.50	25.40	25.00	26.50	20.60	21.50
Mass of water, g	5.50	5.50	5.30	6.50	1.30	1.30
Mass of dry soil, g	9.90	9.80	9.20	10.90	5.10	5.80
Water content,%	55.6	56.1	57.6	59.6	25.49	22.41
No of blows	39	27	22	17	------	------

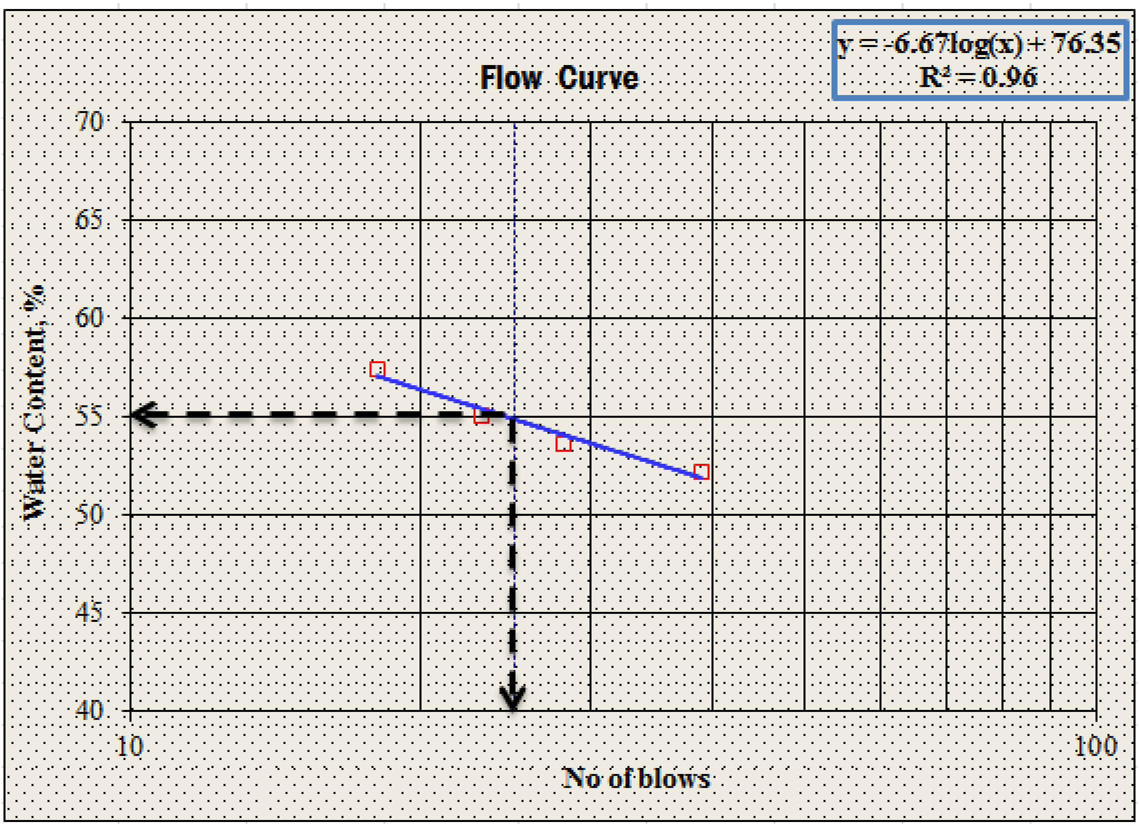
D6 Water Content Vs No of Blows for TP9 at D = 1.5m



D7 Liquid Limit and Plastic Limit test result of TP 9 at D = 3.0m

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C11	C12	C13	C14	C15	C16
Mass of container, g	15.90	15.60	15.60	15.80	15.60	15.60
Mass of container + Wet soil, g	29.30	28.20	32.20	29.50	23.00	22.00
Mass of container + Dry soil, g	24.70	23.80	26.30	24.50	22.70	21.30
Mass of water, g	4.60	4.40	5.90	5.00	0.30	0.70
Mass of dry soil, g	8.80	8.20	10.70	8.70	7.10	5.70
Water content, %	52.3	53.7	55.1	57.5	4.23	12.28
No of blows	39	28	23	18	------	------

D8 Water Content Vs No of Blows for TP9 at D = 3.0m



Appendix E Free swells test results

E1 Free swell test result of TP1 at D = 1.5m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	15.0	14.0	14.5	45

E2 Free swell test result of TP1 at D = 3.0m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	17.0	16.5	16.8	68

E3 Free swell test result of TP3 at D = 1.5m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	21.5	21.0	21.3	113

E4 Free swell test result of TP3 at D = 3.0m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	20.0	21.0	20.5	105

E5 Free swell test result of TP7 at D = 1.5m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	14.0	13.0	13.5	35

E6 Free swell test result of TP7 at D = 3.0m

Initial Volume	Final Volume		Average Final Volume	Free Swell
	Sample No.1	Sample No.2		
(cc)	(cc)	(cc)	(cc)	(%)
10.0	12.0	12.0	12.0	20

Appendix F Compaction test

Test pit No. TP7

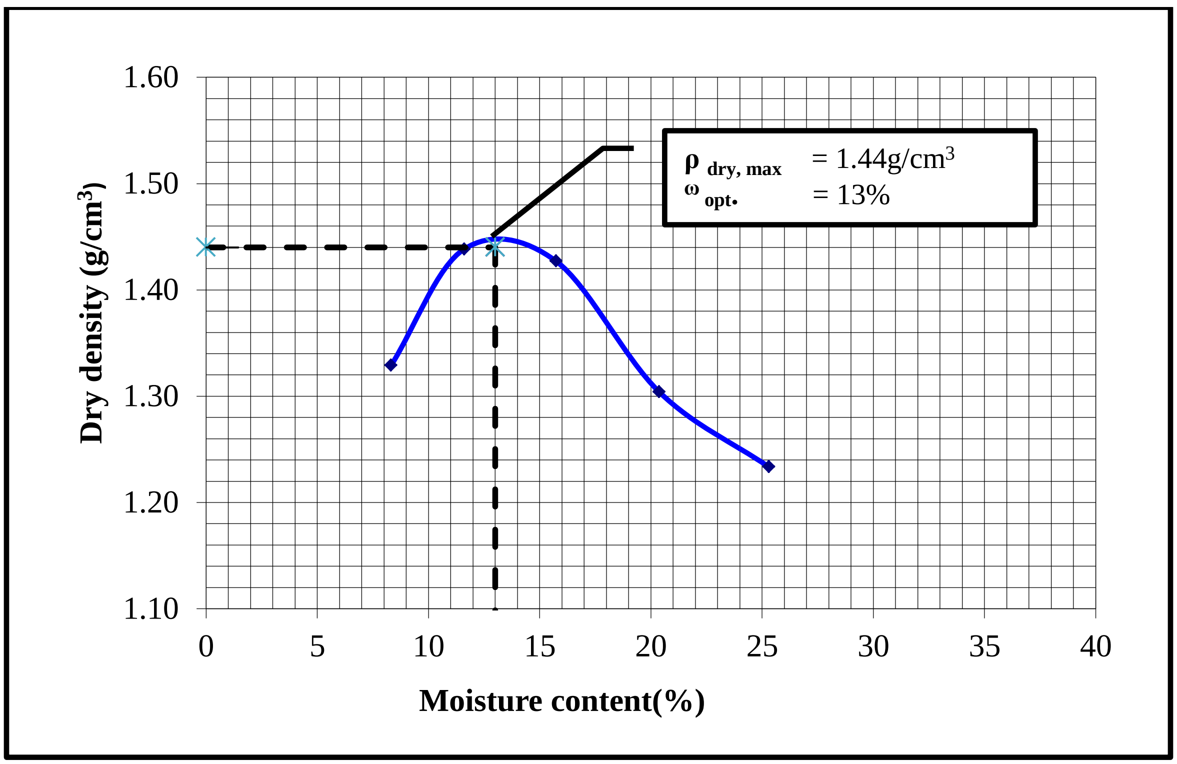
No. of blows: 25 **Weight of hammer, kg: 2.5**

No. of layers: 3 **Volume of mold, cm^3: 944**

F1 Moisture content versus dry density test result of TP7 at D = 3m

Determination No.	1	2	3	4	5
Mass of Mold, g	3141.1	3141.1	3141.1	3141.1	3141.1
Mass of mold + Compacted Soil, g	4500.0	4656.5	4700.2	4622.9	4600.1
Mass of Compacted soil, g	1358.9	1515.4	1559.1	1481.8	1459.0
Volume of Mold, cm3	944.0	944.0	944.0	944.0	944.0
Bulk density, g/cm3	1.44	1.61	1.65	1.57	1.55
Water Contenet, %	8.32	11.60	15.73	20.37	25.30
Dry density, g/cm3	1.33	1.44	1.43	1.30	1.23

F2 Moisture content versus Dry density for TP7 at D=3m



Appendix G Direct shear test result for TP-7 at D = 3m

Test Pit Depth, m **3.00**
Thickness of sample, mm **25**
Length of sample, mm **60**
Width of sample, mm **60**
Ring Calibration Factor, kN/div **0.7**
Rate of strain, mm/min **1.6**
Moisture Content, % **13**

				Trail 1 (100kPa)			Trail 2 (200kPa)			Trail 3 (300kPa)		
Horizontal Dial reading	Deflection (mm)	Strain (ε)	Corrected Area (mm^2)	Load Dial	Load	Shear Stress(kPa)	Load Dial	Load	Shear Stress(kPa)	Load Dial	Load	Shear Stress(kPa)
0	0	0	3600	0	0	0	0	0	0	0	0	0
50	0.5	0.0083	3570	70	49	13.73	75	52.5	14.71	110	77	21.57
100	1	0.0167	3540	130	91	25.71	180	126	35.59	250	175	49.44
150	1.5	0.0250	3510	155	109	30.91	270	189	53.85	365	256	72.79
200	2	0.0333	3480	175	123	35.20	340	238	68.39	455	319	91.52
250	2.5	0.0417	3450	205	144	41.59	405	284	82.17	535	375	108.55
300	3	0.0500	3420	230	161	47.08	455	319	93.13	620	434	126.90
350	3.5	0.0583	3390	245	172	50.59	505	354	104.28	680	476	140.41
400	4	0.0667	3360	250	175	52.08	520	364	108.33	710	497	147.92
450	4.5	0.0750	3330	251	175.4	52.66	510	357	107.21	720	504	151.35
500	5	0.0833	3300	240	168	50.91	500	350	106.06	735	515	155.91
550	5.5	0.0917	3270	235	164.5	50.31	490	343	104.89	720	504	154.13
600	6	0.1000	3240	205	143.5	44.29	475	332.5	102.62	690	483	149.07
650	6.5	0.1083	3210	183	128.1	39.91	455	318.5	99.22	650	455	141.74
700	7	0.1167	3180	170	119	37.42	420	294	92.45	610	427	134.28