ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

INVESTIGATION ON SOME OF THE ENGINEERING CHARACTERISTICS OF SOILS IN ADAMA TOWN, ETHIOPIA

BY:

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

ADVISOR:

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TABLE OF CONTENTS

Contents | Page

SYMBOLS AND ABBREVIATIONS

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

LIST OF TABLES

Page

LIST OF FIGURES

Page

ABSTRACT

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

The objective of this research is to investigate the engineering property of soil found in Adama town. To achieve its objective samples from different parts of the city were collected and laboratory tests were done on the collected samples.

The grain size analysis test result showed that the dominant proportion of soil particle in the research area is silt, which have clay content ranging from 5.4 – 40 5%, silt faction 17.6 - 60.7%, sand fraction 14.5 - 54.6% and gravel content from 0.0 – 24.8%.

The result of Atterberg Limit test on the soil in the research area showed a liquid limit ranging from 29 – 73%, plastic limit ranging from 21 – 35% and plastic index from 13 – 34%.

The Specific Gravity ranges from 2.40 to 2.70. Free swell test conducted on the samples collected shows range from 18 – 50%.

From the compaction test results the maximum dry density (MDD) of Adama soil ranges from1.20 to 1.62 g/cm^3 and the optimum moisture content ranges 17.5 to 36.5 percent.

According to the Unified Soil Classification System, the soil is categorized as silt and silty sand. The AASHTO Classification System shows that the usual types of significant constituent materials of the Adama soil are silt soil and clayey soil.

Finally one-dimensional consolidation test was done and it shows that the area under investigation is over consolidated in its natural state, have compression index ranging

from 0.33-0.40, recompression index ranging from 0.017-0.066, coefficient of permeability ranging from 10^{-7} to 10^{-9} cm/sec and modulus of compressibility ranging from 3600-7500 kPa.

1. Introduction

1.1. Back ground of the problem

A geotechnical engineer determines and designs the type of foundation, earthwork, and/or pavement sub grades required for the intended man-made structures to be built. Foundations are designed and constructed for the structures of various sizes such as high-rise buildings, bridges, medium to large commercial buildings, and smaller structures where the soil conditions do not allow code-based design [25].

Investigation of the underground conditions at a site is prerequisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies of the proposed project. Public building officials may require soil data together with the recommendations of the geotechnical consultant prior to issuing a building permit, particularly if there is a chance that the project will endanger the public health or safety or degrade the environment [4].

Insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to inappropriate designs; delays in construction schedules, costly construction modifications, and use of substandard borrow material, environmental damage to the site, post construction remedial work, and even failure of a structure and subsequent litigation [26]. Therefore, to obtained information on type, characteristics and distributions of a soil, geotechnical investigations should be done on soil and rock underlying (and sometimes adjacent to) a site of proposed structures

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

Many researches were done and there are ongoing researches in most big cities of the country like Addis Ababa, Bahir Dar, Mekele, Hawassa, etc. To mention some of the researches undertaken are: 'Investigation into some of the engineering properties of red clay soils in Bahir Dar (Fasil, A., 2003), 'Investigation into some of the engineering properties of red clay soils in Addis Ababa' (Samuel, T., 1989), 'Investigation into engineering properties of Mekelle soils with emphasis on expansive soils'(Kibrom, G., 2005), etc.

Adama is one of the fastest growing cities in the country and there is a big volume of construction works. Since it is the transit way on the road from Addis Ababa to Djibouti, the growth of trade and commerce is very high in the city. Due to its location and near distance which is around 100km from Addis Ababa investors are attracted to construct in Adama town and the nearby areas. However, the engineering property of the soil in the city is not studied. This research is therefore directed to the study of the physical and mechanical property of soils i.e. investigating the index property and consolidation characteristic, identifying the characteristics of the soil and preparing soil map of the city.

1.2. Objectives of the Study

The objectives of this thesis work are the following:

- To investigate some the engineering and index properties of the Adama soil like: Natural moisture content, Field density, Specific gravity, Consistency limits, Grain size analysis, Swelling potential, Consolidation and Compaction characteristics etc.
- To determine the range of values of index property of soil in different parts of the city.
- To determine the consolidation characteristic of soils in the city.
- To prepare geotechnical map of the city.

1.3. Methodology

To achieve the above mentioned objectives ten sampling areas were selected. From the selected sampling areas pit was excavated to a depth of around three meters. Disturbed samples of soils were collected for laboratory testing.

In the field GPS reading was taken to locate the ordinate of sampling area. Visual classification, field density and natural moisture content tests were done in the field. The natural moisture content test was done by taking balance and moisture can to the field. After measuring the weight of empty can and can with moist soil the sample is brought to laboratory and put into drying oven.

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

- Specific gravity test
- Atterberg limit tests
  - Liquid limit
  - Plastic limit
- Grain size analysis
  - Sieve analysis(wet method)
  - Hydrometer
- Free swell test
- Standard compaction test
- One-dimensional consolidation test

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

1.4. Scope of the Study

Twenty samples of soil from ten pits were collected. The scope of this study is limited to investigating the index properties, and compaction and consolidation characteristic. Due to the budget constraint, the depth of investigation in this research is limited to the maximum depth of three meters. The permeability coefficient of the soil in the research area is calculated from consolidation test results.

1.5. Structure of the Thesis

This thesis work is divided in to six Chapters, each covering a specific topic of the research work. In this introductory Chapter the background of the problem, objective, methodology and scope of the thesis work and structure of the thesis are presented. Chapter two deals with a brief literature review. Chapter three deals with the description of area in which this research is done. The fourth chapter deals with in-situ properties with sample description and the types of laboratory tests conducted and results obtained. The discussion on the laboratory results obtained from this work and comparison with previously done researches is covered in chapter five. Chapter six is the conclusions and recommendations drawn from the research. Finally, detail calculation of the consolidation test results, meteorological data and soil profiles for each test pits are included in appendix.

2. Literature review

2.1. Soil formation and soil deposits

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 [15].

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 [11].

It has long been appreciated that the engineering classification of soils is greatly facilitated by taking into account the soil-forming processes by which nature has created the various types of soil conditions. Similar combinations of soil-forming processes in different parts of the world have been found to lead to materials of similar index properties and similar engineering characteristics [21]. The main factors affecting the formations of soil are: Parent materials i.e. geology of the area, topography and drainage, climate and vegetation cover.

2.1.1. Parent materials

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

2.1.2. Topography and Drainage

Topography has a major influence on drainage characteristics which in turn is known to have major effect on soil mineralogy. Its control over soil properties is particularly strong in tropical environment reflecting the importance of lateral movement of water and soil materials [21].

2.1.3. Climate

Climate is the principal factor governing the rate and type of soil formation. The two important components of climate are the amount and distribution of precipitation, and temperature. 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 °c rise of temperature [25].

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].

2.2. 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 [15].

2.2. 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 [15].

2.3. Soil mineralogical composition [26]

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

- Primary minerals**:** present in original rock from which soil is formed. These occur predominantly in sand and silt fractions, and are weathering resistant (quartz, feldspars).
- Secondary minerals**:** formed by decomposition of primary minerals, and their subsequent weathering and recomposition into new ones (clay minerals).
- Humus or organic matter (decomposed organic materials).

3. Description of the study area

3.1 General

Adama is located in eastern Showa in the Oromiya Region [10]. It is one of the largest and most populated towns in Oromiya National Regional State. It is located at $8^{\circ}33'35''$N - $8^{\circ}3'46''$ N latitude and $39^{\circ}11'57''$ E – $39^{\circ}21'15''$ E longitude. It is about 100 kilometers away from Addis Ababa in southeast direction. Adama has a total area of about 13,000 hectares, which has been subdivided into 14 urban kebele (least administrative structure) administrations.

Available documents evidenced that Adama had been known as Nazreth (the name given to it by Emperor Haile Sellasie I) for most of the 20th century up until it was officially reverted to its original Oromo language name, Adama, in 2000. Adama had been serving as the capital city of Oromiya National Regional State during 2000-2005. On the 10th of June 2005 that the Oromiya National Regional Government announced the move of the regional capital back to Addis Ababa [14]. The location of the research area, i.e. Adama, on the map of Ethiopia is shown in figure 3.1.

Fig 3.1 Location of the research area on the map of Ethiopia

(The original map was taken from [1])

3.2. Soil and Geology

The Ethiopian rift system which is part of the East African Rift System may be sub-divided into three main sections. There are: the south western rift zone, the main Ethiopian rift, and Afar. Adama is part of the central Rift Valley and dominated by flatlands stretching between the escarpments bordering the eastern and western sides of the Rift Valley [10].

Adama is found within the Wonji Fault Belt, which is one of the main structural systems in the Ethiopian Rift Valley. Its physiographic condition is, therefore, mainly the result of volcano-tectonic activities that occurred in the past and also partly the result of the deposition of sediments, which are considered largely of fluvial and lacustrine origin. Adama is regarded as seismically active area concerning earthquake hazards with the probability occurrence of 0.99 in every 100 years [After Messay, 2010].

Adama has three types of soil: andosols, fluvisols and lithosols. The dominant type is mollic andosols (Anm), with a water-holding capacity of 144mm for a 1m depth [10].

3.3. Topography and drainage conditions

The altitude of Adama varies from about 1500m to 1670m above mean sea level. The only perennial river in the vicinity of Adama is awash into which all the streams in the town join [After Messay, 2010].

3.4 Climate

3.4.1. Rainfall

The records of National Meteorological Service Agency from Adama observatory substation show that the mean annual rainfall for thirty three years i.e. from 1977 to 2009 at an altitude of 1622m above mean sea level, latitude of $39^{\circ}17'$ and longitude of $8^{\circ}33'$ is 881.9mm. As shown in Fig 3.1, the rainy season is only in months of July and August followed by rainfall of almost equal to or below evaporation rate of the city. Around 50% of the rainfall arrives in these two months.

3.4.2. Temperature

In a mountainous tropical country like Ethiopia altitude is by far the most important factor in 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°C, areas between 500 – 1500 meters a.m.s.l. (i.e. 'kola' or tropical climate) have 20 -30°C and areas below 500 meters a.m.s.l. (i.e. 'bereha' or desert climate) have a temperature of 30°C and above[16].

The town of Adama, with an altitude ranging from 1500-1670meters a.m.s.l., has a mean minimum, mean maximum and mean average monthly temperatures of 14.3, 28.1 and 21.2°C respectively. The highest temperatures are during months of March, April, May and June where as November, December and January have low temperature. From Fig3.3 the Mean monthly average temperature ranges from 19.33°c to 23.54°c. This shows the temperature variation is almost the same throughout the year

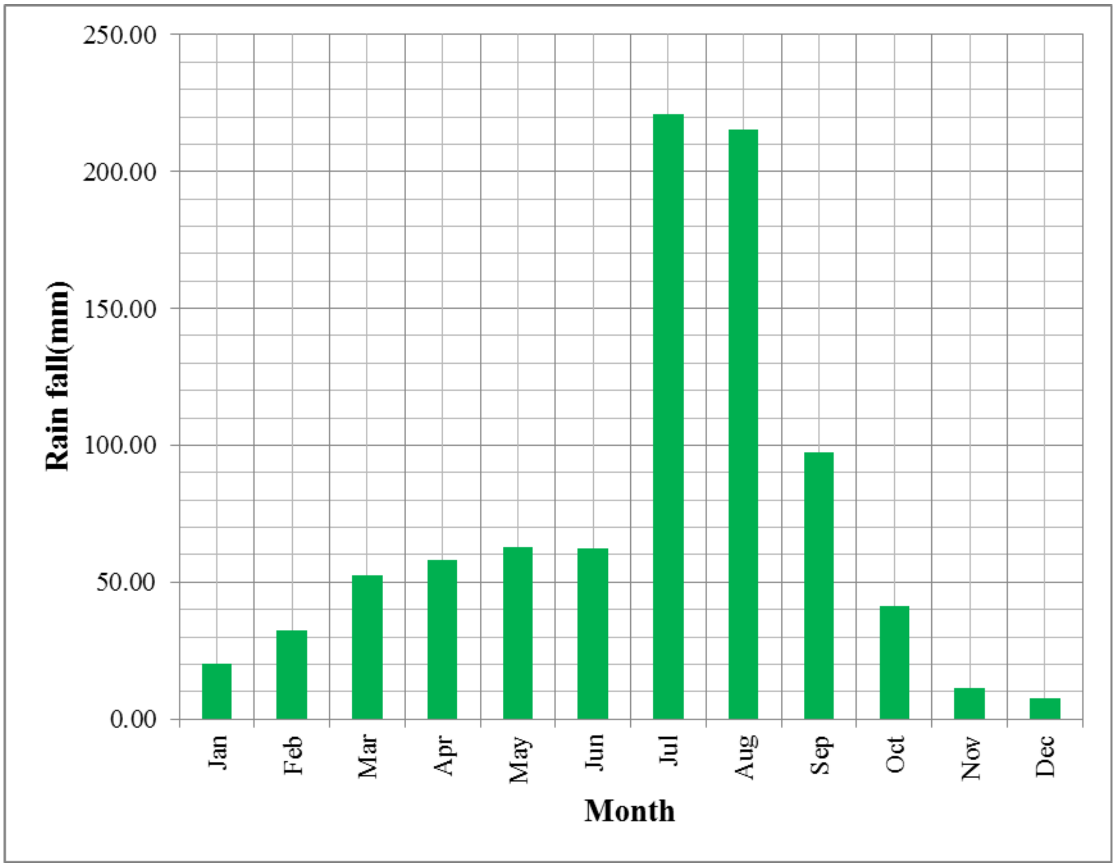

Fig 3.1 Mean monthly rainfall distribution of Adama (1977 - 2009 G.C.)

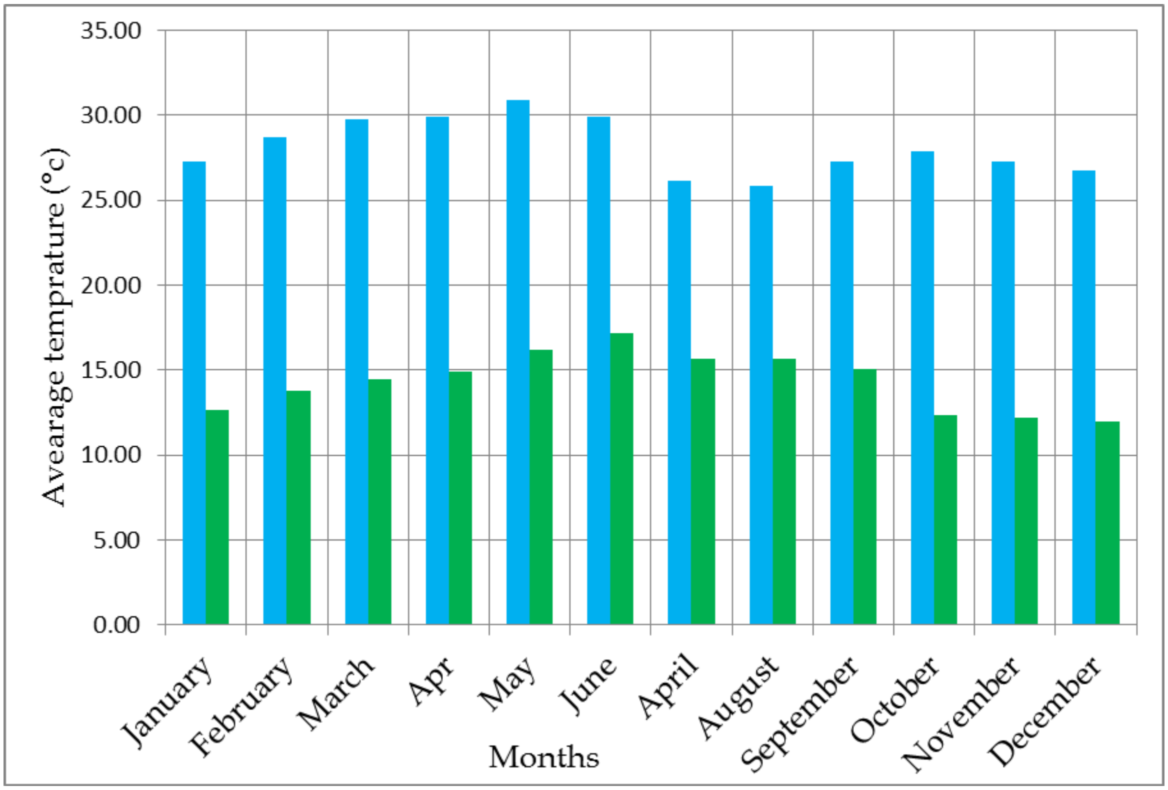

Fig 3.2 Average Monthly Maximum and Minimum temperature distribution of Adama, (1977 - 2009 G.C.)

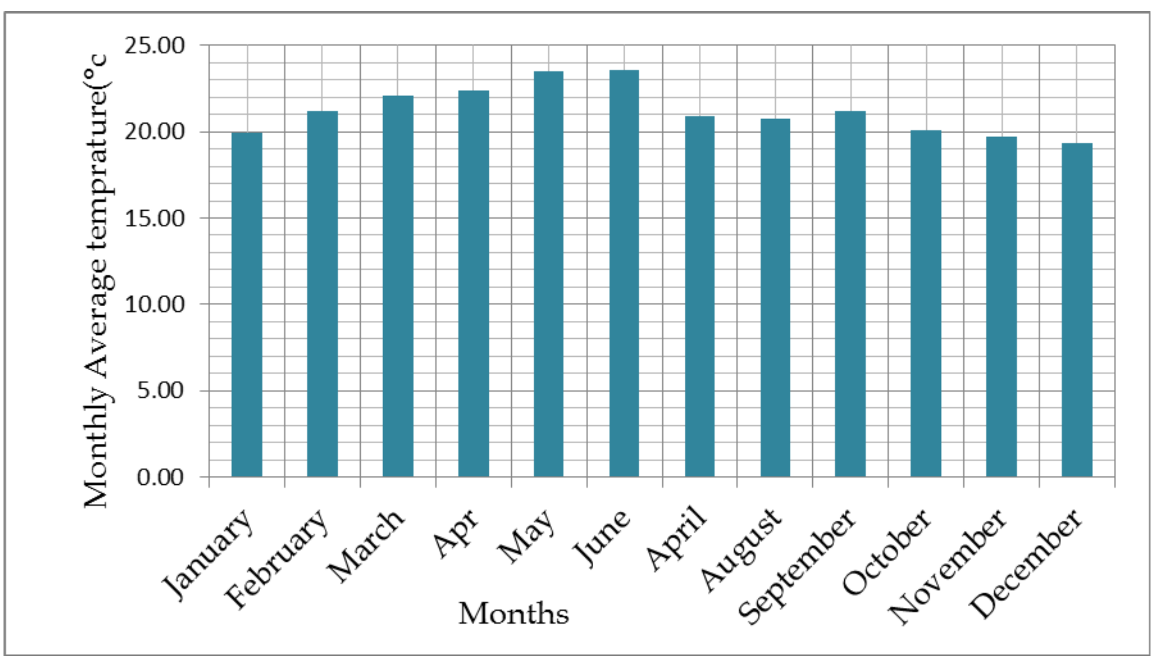

Fig 3.3 Monthly Average Temperature distribution of Adama (1977 - 2009 G.C)

4. In-situ Properties and Laboratory tests results

4.1. In-situ properties

4.1.1. Identification of Soil in the Study Area

Before selecting sampling areas, visual site investigation and information from resident, and construction firms were collected to consider the different soil types and to take sample evenly in the whole town. Accordingly, ten sampling areas were selected from different locations of the town. Pits were excavated to the maximum depth of three meters, but in some areas boulders were encountered making the digging difficult. Only disturbed samples were taken because of the silt nature of the soil which makes recovering of undisturbed sample difficult. In the field visual soil description was made and sample for laboratory testing were collected. The global coordinates of sampling location i.e. northing, easting and elevations are shown in Table 4.1.

Table 4.1 Global coordinates of sampling areas

Test Pit	Location	Northing	Easting	Elevation (m)
TP-1	Around Adama University (Gichi)	531856.00	945678.00	1653
TP-2	Kebele-12(Municipality)	529976.00	944650.00	1615
TP-3	Kebele-09 (Ketara)	528655.00	943297.00	1612
TP-4	Kebele-01 (Kechema)	529233.00	945691.00	1641
TP-5	Kebele-03 (Boku)	530258.00	941824.00	1598
TP-6	Kebele-05 (Cheffe)	527055.00	943485.00	1643
TP-7	Kebele-14 (Dibibisa)	531379.00	944277.00	1604
TP-8	Kebele-02 (Migira)	530640.00	942118.00	1603
TP-9	Kebele-04 (Sole)	530683.00	946375.00	1634
TP-10	Kebele-03 (Dabe)	531303.00	939312.00	1633

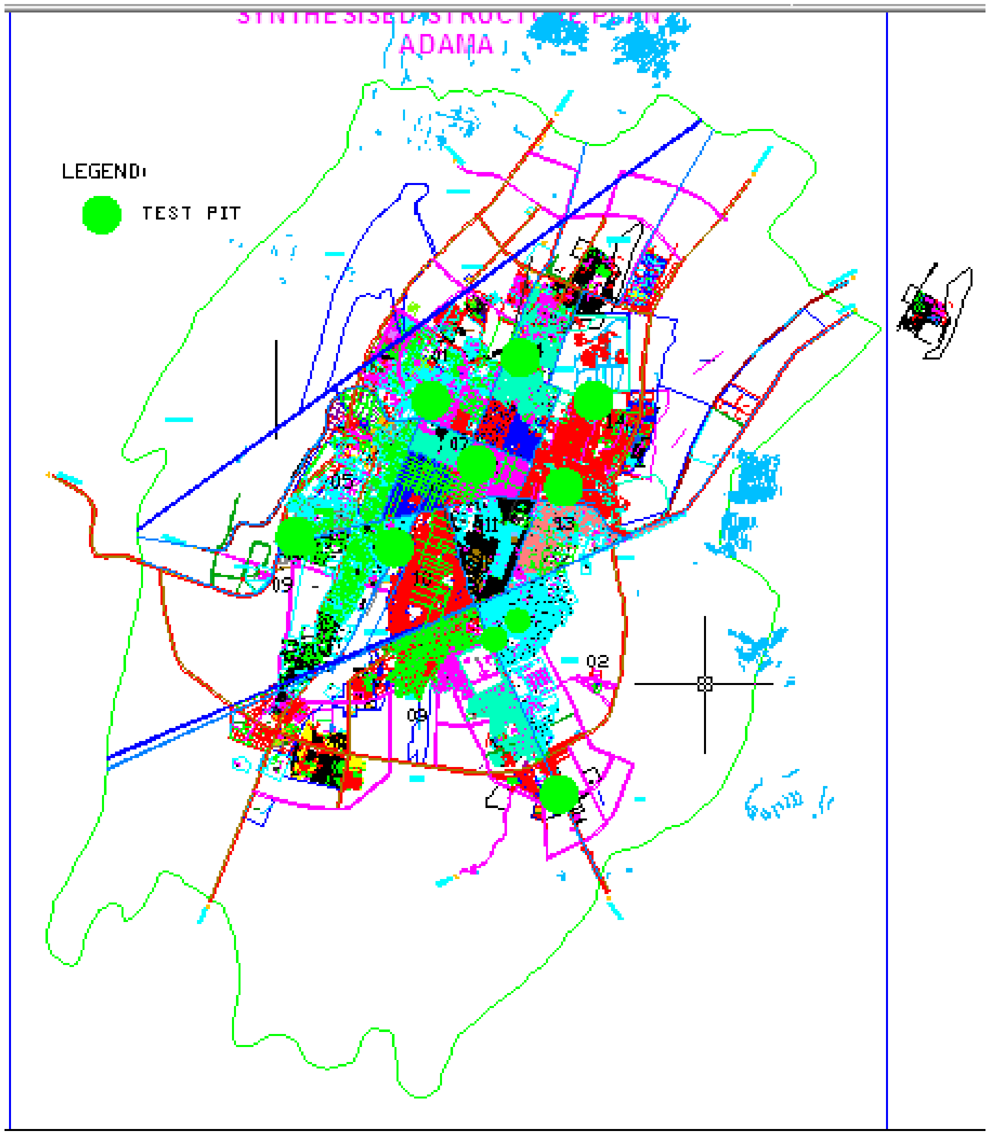

Fig. 4.1 Location of sampling areas shown on map of Adama town

4.1.2. In-situ properties Description

From the entire test pits two basic properties were identified: in the top layer brown clayey silt was identified and at deeper level reddish white clay silt with very small amount of sand was found (see App A. for soil profiles of each test pits). In the field part of the natural moisture content and field density tests were done.

4.1.2.1. Natural moisture content

For most soils, the water content may be an important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The water content is also used in expressing the phase relationships of air, water, and solids in a given volume of soil [13].

Since it was difficult to bring undisturbed samples to the laboratory, this test was done by taking moisture can and balance to the field. In the site the weight of the moisture can and the weight of can with moist soil was measured. Then the sample was brought to the laboratory and put it in to drying oven at a temperature of 105$\pm$5°c for 24 hours. Then after, the natural moisture content was determined. The natural moisture content of the ten pits for each profile is shown in Table 4.2.

4.1.2.2. In situ density

The bulk density is the ratio of mass of moist soil to the volume of the soil sample, and the dry density is the ratio of the mass of the dry soil to the volume the soil sample. The in-place density of soils is used to determine density of compacted soils used in the construction of structural fills, highway embankments, or earth dams.

This test was done according to the Standard Reference: ASTM D 2937-00 – Standard Test for Density of Soil in Place by the Drive-Cylinder Method. The in situ density of the samples around three meter is shown in Table 4.2. Field densities were taken only around three meters because these values were required for remolding sample in consolidation test.

Table 4.2The In-situ density and natural moisture contents of soil samples

Test Pit No	Depth(m)	Natural moisture content (%)	In-situ density(g/cm3)	Dry density(g/cm3)
TP-01	0.00-0.80	12.08	-	-
	0.80-1.40	9.60	-	-
	1.40-3.00	19.30	1.36	1.14
TP-02	0.00-0.80	fill	-	-
	0.80-1.50	16.60	-	-
	1.50-3.0	27.62	1.34	1.05
TP-03	0.00-1.00	27.16	-	-
	1.0-3.0	37.52	1.47	1.07
TP-04	0.00-0.30	fill	-	-
	0.30-2.0	23.65	-	-
	2.0-3.0	21.16	1.38	1.14
TP-05	0.00-0.80	15.45	-	-
	0.80-3.0	31.86	1.34	1.02
TP-06	0.00-0.30	fill	-	-
	0.3-1.10	21.17	-	-
	1.10-3.0	14.06	1.3	1.14
TP-07	0.00-1,10	15.81	-	-
	1.10-2.60	12.46	1.12	1.00
TP-08	0.00-0.60	12.50	-	-
	0.60-1.45	16.97	-	-
	1.45-2.30	15.21	1.23	1.07
TP-09	0.00-0.60	fill	-	-
	0.60-1.85	35.30	-	-
	1.85-3.0	27.01	1.40	1.10
TP-10	0.00-2.0	25.01	-	-
	2.00-3.00	25.44	1.25	1.00

4.2. Index properties

4.2.1. General

A bulk soil, as it exists in nature, is more or less randomly assembled of soil particles, water and air. The properties of soils are complex and variable. Every civil engineering work involves the determination of soil type and its associated engineering application; certain properties are more significant than others. The common problems faced by civil engineers are related to bearing capacity and compressibility of soil and seepage through the soil. The possible solution to these problems is arrived at based on the study of the physical and index properties of the soil [2].

Soil is a heterogeneous material. 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 is interested to have some rough assessment of the engineering properties without conducting 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 which are indicative of the engineering properties are called index properties [2]. According to Dr. B.C. Punimia, ***et al*** the index properties of soils are water content, specific gravity, particle size distribution, consistency limits, in-situ density, free- swell and density index.

4.2.2. Specific gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of a soil is used in the phase relationship of air, water, and solids in a given volume of the soil [13].

The specific gravity of the minerals affects the specific gravity of soils derived from them. The specific gravity of most rock and soil forming minerals varies from 2.50 (some

Feldspars) and 2.65 (Quartz) to 3.5 (Augite or Olivine). Gypsum has a smaller value of 2.3 and salt (NaCl) has 2.1. Some iron minerals may have higher values, for instance, magnetite has 5.2[15].

According to ASTM D 854-98, two procedures for performing specific gravity are provided. These are Method-A, procedures for oven dried specimen and Method-B, procedure for moist specimen. For specimens of organic soils and highly plastic, fine-grained soils, Procedure B shall be the preferred method.

But in this research the specific gravities are determined using both procedures. The test results are shown in Table 4.3. From this table we observe that Method-B give more reasonable value than that of Method-A, and values determined on Method-B are used in other calculations like hydrometer analysis. Within the depth of exploration, the top layer which is brown clayey silt has a specific gravity as low as 2.40 but as excavated deeper it becomes between 2.60 to 2.70. Therefore, the specific gravity of Adama town ranges from 2.40 to 2.70.

Table 4.3 Specific Gravity of the Soil of the Study Area

Serial No	Designation	Depth(m)	Specific Gravity Using		Water used for testing
			Method-A	Method-B	
1	TP-1-1	0.80-1.40	2.56	2.55	Tap water
2	TP-1-2	1.40-3.00	2.62	2.65	〃
3	TP-2-1	0.80-1.50	2.52	2.52	〃
4	TP-2-2	1.50-3.0	2.57	2.66	〃
5	TP-3-1	0.00-1.00	2.53	2.47	〃
6	TP-3-2	1.0-3.0	2.61	2.66	〃
7	TP-4-1	0.30-2.0	2.52	2.45	〃
8	TP-4-2	2.0-3.0	2.55	2.70	〃
9	TP-5-1	0.00-0.80	2.46	2.48	〃
10	TP-5-2	0.80-3.0	2.57	2.60	〃
11	TP-6-1	0.3-1.10	2.59	2.61	〃
12	TP-6-2	1.10-3.0	2.62	2.69	〃
13	TP-7-1	0.00-1,10	2.48	2.47	〃
14	TP-7-2	1.10-2.60	2.62	2.67	〃
15	TP-8-1	0.60-1.45	2.46	2.53	〃
16	TP-8-2	1.45-2.30	2.63	2.69	〃
17	TP-9-1	0.60-1.85	2.46	2.40	〃
18	TP-9-2	1.85-3.0	2.56	2.56	〃
19	TP-10-1	0.00-2.0	2.57	2.64	〃
20	TP-10-2	2.00-3.00	2.57	2.53	〃

4.2.3. Grain-size distribution of soil

4.2.3.1. General

For a basic understanding of the nature of soil, the distribution of the grain size present in a given soil mass must be known. The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution required in classifying the soil. Grain size Analysis test is used to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles. The test method covers the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 75μm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75μm is determined by a sedimentation process, using a hydrometer to secure the necessary data[3, 13].

4.2.3.2. Test procedure and results

The procedure followed to run this test is according to ASTM standard with designations D422-63 and D1140-97.

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 existing moisture content of the air dried sample was measured which was used for hygroscopic correction. 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. The sample of soil passing No. 200 was transferred to large dish and soaked until the water becomes clean, then the clean water was decanted. After the sample has dried in room temperature, it's pulverized and 50 grams of soil was taken for hydrometer test.

The following series of sieves, of square-mesh woven-wire cloth, was used for sieve analysis based on the maximum particle size.

3-in. (75-mm)	No. 10 (2.00-mm)
2-in. (50-mm)	No. 20 (850-μm)
11⁄2-in. (37.5-mm)	No. 40 (425-μm)
1-in. (25.0-mm)	No. 60 (250-μm)

3⁄4-in. (19.0-mm) No. 140 (106-μm)
3⁄8-in. (9.5-mm) No. 200 (75-μm)
No. 4 (4.75-mm)

But for drawing the grain size distribution curves the sets of sieves shown below are used to get uniform spacing of points for the graph.

3-in. (75-mm) No. 16 (1.18-mm)
11⁄2-in. (37.5-mm) No. 30 (600-μm)
3⁄4-in. (19.0-mm) No. 50 (300-μm)
3⁄8-in. (9.5-mm) No. 100 (150-μm)
No. 4 (4.75-mm) No. 200 (75-μm)
No. 8 (2.36-mm)

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 actual hydrometer reading and test temperature was taken for 0.1, 0.5, 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 grain size analysis result clay content ranging from 5.4 – 40 5%, silt faction 17.6 - 60.7%, sand fraction 14.5 - 54.6% and gravel content from 0.0 – 24.8%.

Table 4.4 Summary of grain size analysis result

Serial No	Designation	Depth(m)	Percent amount of particle size			
			Gravel	sand	silt	clay
1	TP-1-1	0.80-1.40	24.78	52.23	17.62	5.37
2	TP-1-2	1.40-3.00	6.08	44.85	34.18	15.49
3	TP-2-1	0.80-1.50	0.00	27.23	49.76	23.02
4	TP-2-2	1.50-3.0	0.64	14.48	54.31	30.57
5	TP-3-1	0.00-1.00	9.54	33.59	28.70	28.17
6	TP-3-2	1.0-3.0	0.00	30.29	39.18	30.54
7	TP-4-1	0.30-2.0	0.00	40.84	47.25	11.91
8	TP-4-2	2.0-3.0	4.32	26.93	47.92	20.83
9	TP-5-1	0.00-0.80	2.03	20.91	60.72	16.33
10	TP-5-2	0.80-3.0	0.00	23.34	59.47	17.18
11	TP-6-1	0.3-1.10	0.15	29.97	52.54	17.34
12	TP-6-2	1.10-3.0	13.8	47.14	31.19	5.87
13	TP-7-1	0.00-1,10	0.00	33.00	52.01	14.99
14	TP-7-2	1.10-2.60	1.22	36.80	47.18	14.8
15	TP-8-1	0.60-1.45	0.80	24.00	60.78	14.42
16	TP-8-2	1.45-2.30	0.00	36.34	49.90	13.76
17	TP-9-1	0.60-1.85	0.61	31.52	36.25	31.62
18	TP-9-2	1.85-3.0	0.00	17.98	62.14	19.88
19	TP-10-1	0.00-2.0	4.86	33.47	44.11	17.56
20	TP-10-2	2.00-3.00	1.22	54.60	33.15	11.02

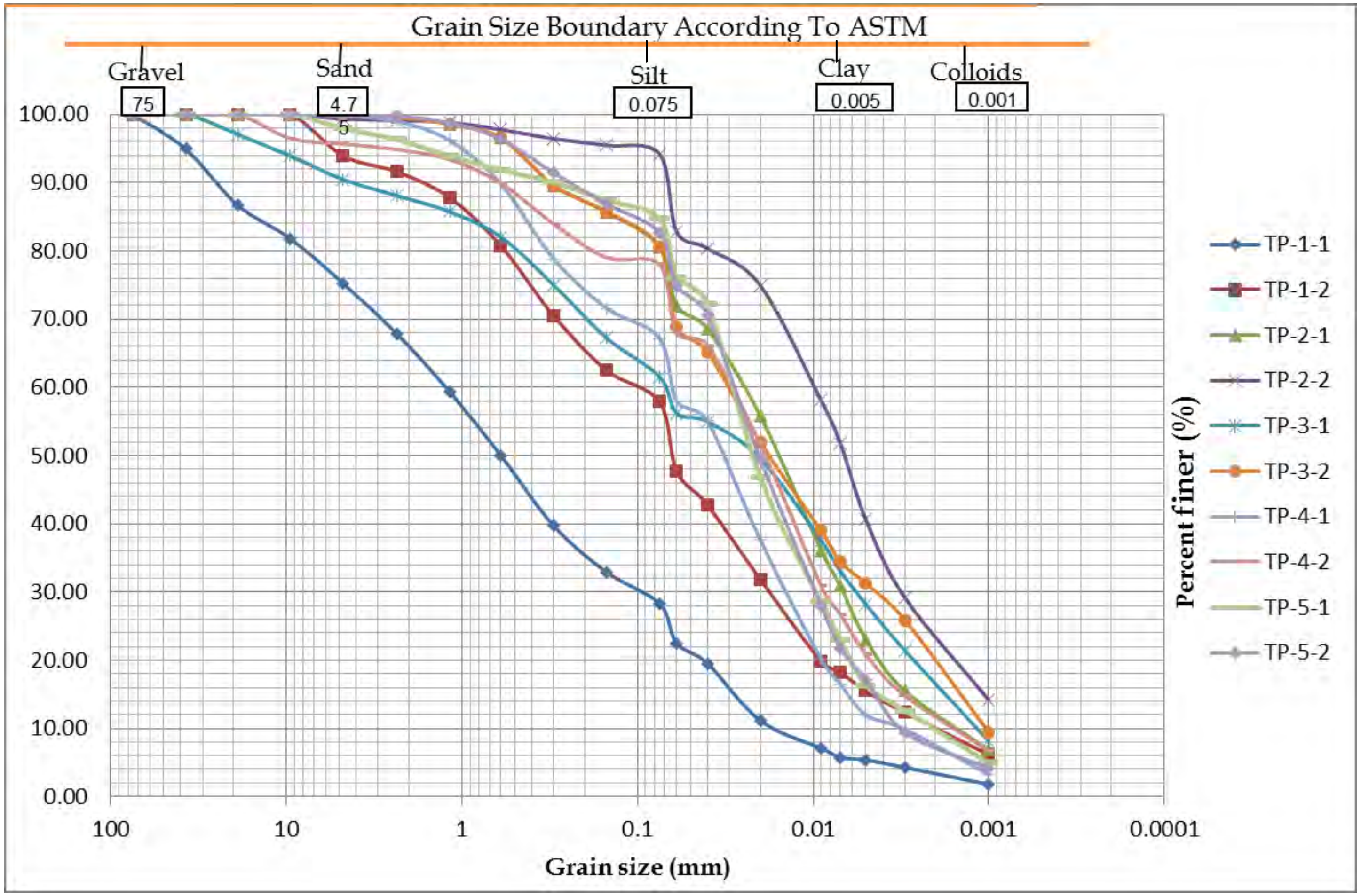

Fig 4.2 Grain size distribution cure for samples from test pits 1 to 5

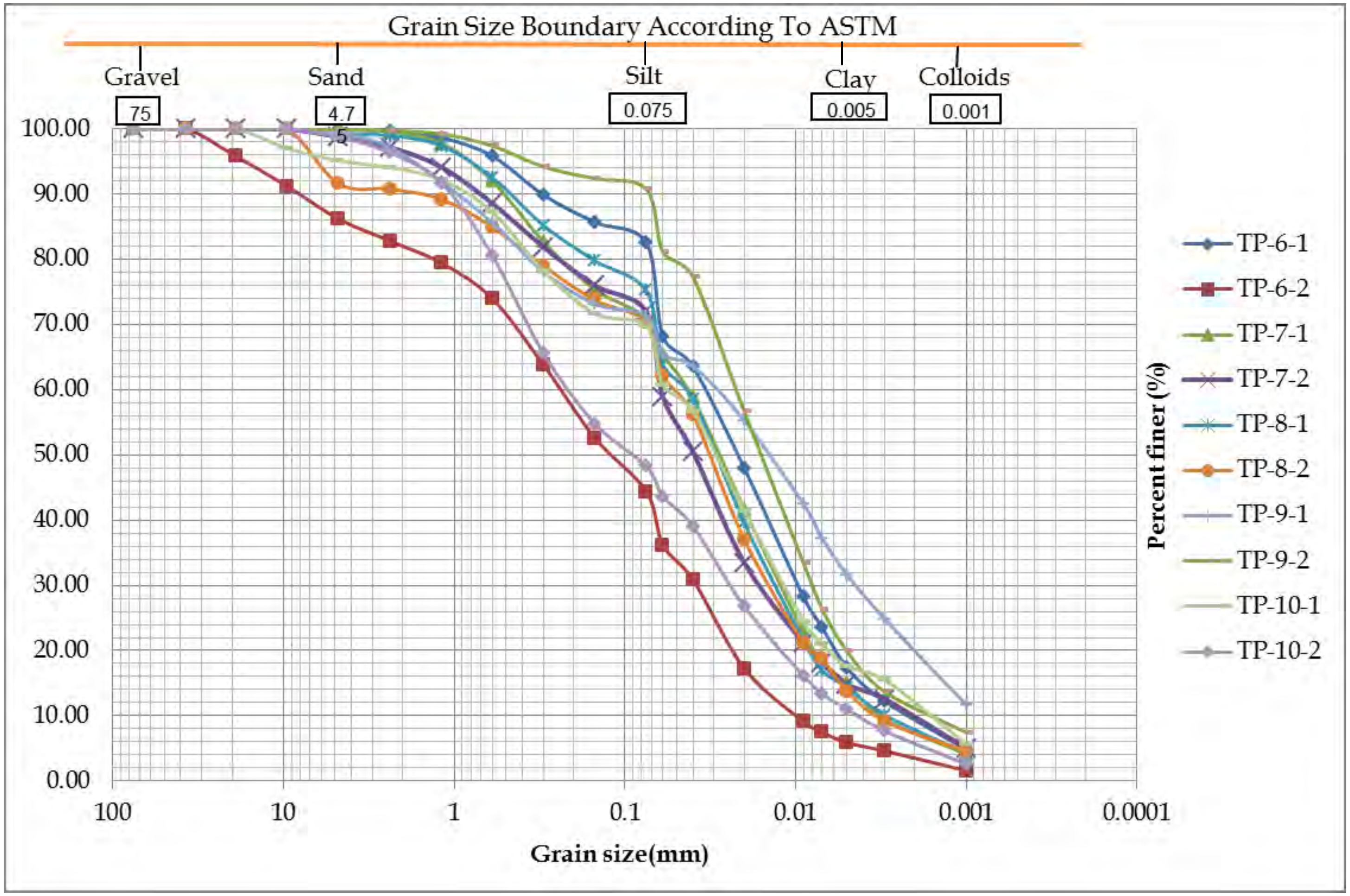

Fig 4.3 Grain size distribution cure for samples from test pits 6 to 10

4.2.4. Atterberg limits

4.2.4.1. General

Atterberg Limits are defined as water contents at certain limiting or critical stages in soil behavior. They, along with the natural water content, are the most important items in the description of fine grained soils. They are used in classification of fine grained soils, and they are useful because they correlate with the engineering properties and engineering behavior of fine-grained soils [18]. The liquid limit and plastic limit of soils (along with the shrinkage limit) are collectively referred to as the Atterberg Limits. The liquid limit is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm diameter threads without crumbling. The term shrinkage limit, expressed as moisture content in percent, represents the amount of water required just to fill all of the voids of a given cohesive soil at its minimum void ratio obtained by oven drying. The shrinkage limit can be used to evaluate the shrinkage potential, crack development potential, and swell potential of earthwork involving cohesive soils [3].

The different states and consistencies through which the soil sample passes with the decrease in the moisture content are depicted in Fig 4.4.

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

Fig 4.4 Different states and consistency of soils with Atterberg limits [15]

4.2.4.2. 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 samples were prepared by spreading the specimen in the air until it dried. The room temperature was about 18-23°c. The portions of the samples passing the No. 40 (0.425mm) sieve were used for the preparation of the sample for this test.

The liquid limit test on oven dry sample was done only on five representative samples. These values are required, to know the ratio of liquid limit on oven dried and air dried sample, which will intern used to classify the soil as organic and inorganic. The oven drying samples were prepared by putting the sample in an oven for 24 hours at a temperature of 110 °c ± 5°.

The Atterberg Limits for soil in Adama town are summarized in Table 4.5 and the liquid limits for oven dried samples are given in Table 4.6. From this we can observe that liquid limit ranges from 29 – 73%, plastic limit ranges from 21 – 35% and plastic index from 0 – 34%

Table 4.5 Atterberg Limit Result Summary for air dried soil samples

Serial No	Designation	Liquid Limit	Plastic Limit	Plasticity Index
1	TP-1-1	29	22	7
2	TP-1-2	38	24	14
3	TP-2-1	48	31	17
4	TP-2-2	48	35	13
5	TP-3-1	49	24	25
6	TP-3-2	73	39	34
7	TP-4-1	42	28	14
8	TP-4-2	39	30	9
9	TP-5-1	47	32	15
10	TP-5-2	47	33	14
11	TP-6-1	48	31	17
12	TP-6-2	29	21	8
13	TP-7-1	41	30	11
14	TP-7-2	35	28	7
15	TP-8-1	42	31	11
16	TP-8-2	31	24	7
17	TP-9-1	52	35	17
18	TP-9-2	57	29	28
19	TP-10-1	41	30	11
20	TP-10-2	30	25	5

Table 4.6The ratio of liquid limits on oven dried and air dried representative samples

Test pit	Depth	Liquid limit (%)		Ratio
		Oven dried	Air dried	
TP-03-2	1.0-2.8@2.60	60	73	82
TP-07-2	1.10-2.60	33	35	94
TP-08-2	1.45-2.30	31	31	100
TP-09-1	0.6-1.85	47	52	90
TP-09-2	1.85-3.0	56	57	98

4.2.5. Free swell

Both the amount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals Montimorillonite influence the magnitude of swelling maximally as compared to Illites and Kaolinites [24].

To study the swelling property of the soils, the simplest test conducted is free swell test. This test is performed by slowly 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:

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

Free swell test results for oven dried samples at a temperature of105 ± 5 ^{o}c are summarized in Table 4.7. From the test result one can see that the free swell of the soil under investigation ranges from 18% to 50%. Those soils having a free swell less than 50% are considered as low in degree of expansion [23]. Hence all soil samples under investigation are non expansive soils.

Table 4.7 Free swell test results of the stud area

Serial No	Designation	Depth(m)	Test condition	Free swell (%)	Water used for testing
1	TP-1-1	0.80-1.40	Oven dry	18	Tap water
2	TP-1-2	1.40-3.00	Oven dry	20	,,
3	TP-2-1	0.80-1.50	Oven dry	30	,,
4	TP-2-2	1.50-3.0	Oven dry	50	,,
5	TP-3-1	0.00-1.00	Oven dry	50	,,
6	TP-3-2	1.0-3.0	Oven dry	50	,,
7	TP-4-1	0.30-2.0	Oven dry	19.5	,,
8	TP-4-2	2.0-3.0	Oven dry	20	,,
9	TP-5-1	0.00-0.80	Oven dry	50	,,
10	TP-5-2	0.80-3.0	Oven dry	30	,,
11	TP-6-1	0.3-1.10	Oven dry	28	,,
12	TP-6-2	1.10-3.0	Oven dry	30	,,
13	TP-7-1	0.00-1,10	Oven dry	30	,,
14	TP-7-2	1.10-2.60	Oven dry	25	,,
15	TP-8-1	0.60-1.45	Oven dry	30	,,
16	TP-8-2	1.45-2.30	Oven dry	40	,,
17	TP-9-1	0.60-1.85	Oven dry	30	,,
18	TP-9-2	1.85-3.0	Oven dry	50	,,
19	TP-10-1	0.00-2.0	Oven dry	30	,,
20	TP-10-2	2.00-3.00	Oven dry	20	,,

4.3. Compaction test

Mechanical compaction is one of the most common and cost effective means of stabilizing soils. During compaction air is expelled from the void spaces. Thus compaction results in an increase in the density of the soil. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density (as a percentage of the "maximum" density measured in a standard laboratory test), and the water content. In general, most engineering properties, such as the strength, stiffness, resistance to shrinkage, and imperviousness of the soil, will improve by increasing the soil density [13]. Results are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction [25].

The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density[13].

Two types of compaction tests routinely performed are: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. In the Standard Proctor Test, the soil is compacted by a 24.4N hammer falling a distance of 0.305meters into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test, except it employs, a 44.5N hammer falling a distance of 0.457meters, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 0.102meters in diameter and has a volume of about 944 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[13].

From the test results the maximum dry density (MDD) of Adama ranges from1.20 to 1.62 g/cm^3 and the optimum moisture content ranges 17.5 to 36.5 percent. The summary of the test result is shown in Table 4.8.

Generally course grained soils can be compacted to a higher dry density than fine gained soils for the some compaction effort. When some fines are added to the coarse grained soils to file the voids, the maximum dry density further increases, but it the amount of fines is too much, more than required to fill the voids, it results in reduction of dry density; well graded soils can attain higher dry density than poorly graded soils. High plasticity clays attain much less dry density than low plasticity clays for the some completive effort. This can also be observed from Table 4.4 and Table 4.8 i.e. as the amount of courser particles increases the dry density will increase too. That is why samples with designation TP-1-1 and TP-6-2 show higher dry density as compared to other samples.

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

Serial No	Designation	Depth(m)	MDD (g/cm3)	OMC (%)
1	TP-1-1	0.80-1.40	1.53	17.5
2	TP-1-2	1.40-3.00	-	-
3	TP-2-1	0.80-1.50	1.29	33
4	TP-2-2	1.50-3.0	1.47	23.5
5	TP-3-1	0.00-1.00	1.27	32
6	TP-3-2	1.0-3.0	1.37	26.75
7	TP-4-1	0.30-2.0	1.37	28.2
8	TP-4-2	2.0-3.0	1.52	23.5
9	TP-5-1	0.00-0.80	-	-
10	TP-5-2	0.80-3.0	1.33	31
11	TP-6-1	0.3-1.10	1.35	27
12	TP-6-2	1.10-3.0	1.62	20.2
13	TP-7-1	0.00-1,10	1.37	22.5
14	TP-7-2	1.10-2.60	1.47	26
15	TP-8-1	0.60-1.45	1.44	25.8
16	TP-8-2	1.45-2.30	1.44	29.25
17	TP-9-1	0.60-1.85	1.2	33.5
18	TP-9-2	1.85-3.0	1.26	36.5
19	TP-10-1	0.00-2.0	1.38	29.5
20	TP-10-2	2.00-3.00	1.44	22.5

4.4. Classification of the Soils

4.4.1. 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 [23]. The soils under investigation have been classified according to AASHTO M-145 and UCSC. These methods are among the widely used classification systems in our country and the classification results are shown in Table 4.9 and 4.10.

Average grain size classification according to ASTM, 1998

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

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

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

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

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

4.4.2. Classification of soils based on Unified soil classification (USC) system

This system describes a system for classifying minerals and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required [3].

Table 4.9 Classifications of soils based on USC Classification system

Serial No	Designation	Depth(m)	Percent amount of particle size				LL (%)	PI (%)	Classification According to USCS
			Gravel	sand	silt	clay			
1	TP-1-1	0.80-1.40	24.78	52.23	17.62	5.37	28.50	0.00	SM
2	TP-1-2	1.40-3.00	6.08	44.85	34.18	15.49	37.50	13.18	SM
3	TP-2-1	0.30-1.50	0.00	27.23	49.76	23.02	48.10	17.18	ML
4	TP-2-2	1.50-3.0	0.64	14.48	54.31	30.57	47.8	13.04	ML
5	TP-3-1	0.00-1.00	9.54	33.59	28.70	28.17	49.00	24.95	CL
6	TP-3-2	1.0-3.0	0.00	30.29	39.18	30.54	72.8	33.94	MH
7	TP-4-1	0.30-2.0	0.00	40.84	47.25	11.91	41.8	9.50	ML
8	TP-4-2	2.0-3.0	0.00	40.84	47.25	11.91	39.1	9.04	ML
9	TP-5-1	0.00-0.80	2.03	20.91	60.72	16.33	47.4	15.29	ML
10	TP-5-2	0.80-3.0	0.00	23.34	59.47	17.18	46.50	13.36	ML
11	TP-6-1	0.3-1.10	0.15	29.97	52.54	17.34	48.10	17.18	ML
12	TP-6-2	1.10-3.0	13.80	47.14	31.19	5.87	28.70	8.14	SM
13	TP-7-1	0.00-1,10	0.00	33.00	52.01	14.99	41.3	10.91	ML
14	TP-7-2	1.10-2.60	1.22	36.80	47.18	14.80	34.65	7.00	ML
15	TP-8-1	0.60-1.45	0.80	24.00	60.78	14.42	41.8	11.14	ML
16	TP-8-2	1.45-2.30	0.00	36.34	49.90	13.76	30.8	6.98	ML
17	TP-9-1	0.60-1.85	0.61	31.52	36.25	31.62	52.40	17.77	MH
18	TP-9-2	1.85-3.0	0.00	17.98	62.14	19.88	57.40	26.00	MH
19	TP-10-1	0.00-2.0	4.86	33.47	44.11	17.56	40.00	10.00	ML
20	TP-10-2	2.00-3.00	1.22	54.60	33.15	11.02	30.28	4.97	SM

According to USC classification scheme most of the soil of the study area falls in ML or MH region, which shows that the soil is non expansive. From the plot of plasticity chart in figure 4.5 and the classification soils on table 4.9 the soils found in Adama town are silt, elastic silt and silty sand.

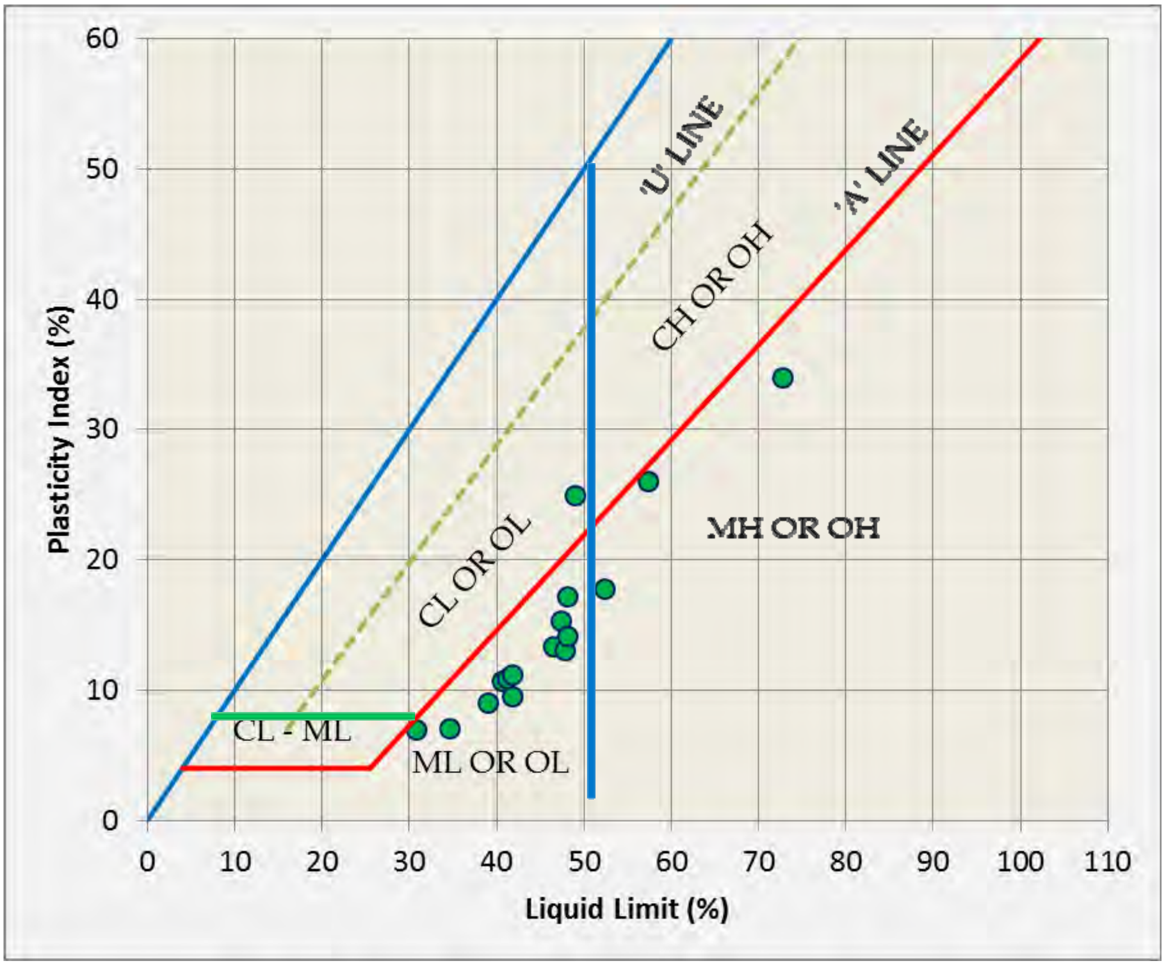

Fig 4.5 Plasticity chart of the study area according to Unified Soil Classification System [3]

4.4.3. Classifications of soils based on AASHTO Classification system

The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30. It generally 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.

As it can be observed from this Classification system (Table 4.10and fig 4.6) the usual types of significant constituent materials are silty and clay soils. Except samples from TP-01 which is Silty or clayey gravel and sand the group classification of all the samples are A-4, A-5, and A-7-5/6.

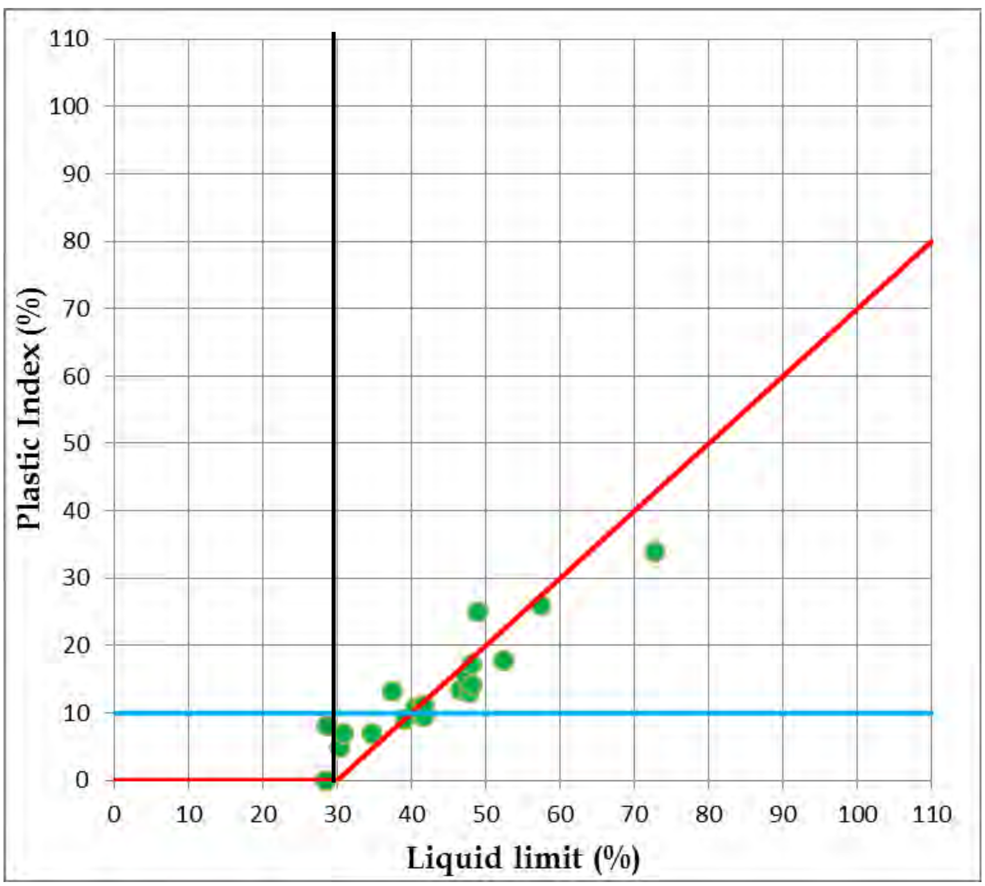

Fig 4.6 Plasticity chart of soil in the study area according to AASHTO system of classification

Table 4.10 Classifications of soils based on AASHTO Classification system

Serial No	Designation	Percent passing on sieve			LL (%)	PI (%)	Group index	Group classification	Usual types of significant constituent materials	General rating as sub-grade materials
		No. 10	No. 40	No. 200						
1	TP-1-1	65.67	49.97	28.27	28.50	0.00	0	A-2-4	Silty or clayey gravel and sand	Good!
2	TP-1-2	90.78	80.71	57.86	37.50	9.44	8(max)	A-4	Silty soils	Fair!
3	TP-2-1	99.39	96.53	80.61	48.10	17.18	20(max)	A-7-5	Clay soils	Poor!
4	TP-2-2	99.07	97.79	94.20	47.8	13.04	20(max)	A-7-5	Clay soils	Poor!
5	TP-3-1	87.55	82.02	61.55	49.00	24.95	20(max)	A-7-6	Clay soils	Poor!
6	TP-3-2	99.39	96.53	80.61	72.8	33.94	20(max)	A-7-6	Clay soils	Poor!
7	TP-4-1	98.60	89.80	67.20	41.8	**9.50**	12(max)	A-5	Silty soils	Fair!
8	TP-4-2	94.65	89.91	77.97	39.1	9.04	8(max)	A-5	Silty soils	Fair!
9	TP-5-1	95.73	91.87	84.76	47.4	15.29	20(max)	A-7-5	Clay soils	Poor!
10	TP-5-2	99.57	96.39	82.69	46.50	13.36	20(max)	A-7-5	Clay soils	Poor!
11	TP-6-1	99.45	95.86	82.51	48.10	17.18	20(max)	A-7-5	Clay soils	Poor!
12	TP-6-2	82.00	74.00	44.40	28.70	8.14	8(max)	A-4	Silty soils	Fair!
13	TP-7-1	99.39	92.06	71.08	41.3	10.91	20(max)	A-7-5	Clay soils	Poor!
14	TP-7-2	96.54	88.62	71.95	34.65	7.00	8(max)	A-4	Silty soils	Fair!
15	TP-8-1	98.80	92.60	75.20	41.8	11.14	20(max)	A-7-5	Clay soils	Poor!
16	TP-8-2	90.59	84.98	70.36	30.8	6.98	8(max)	A-4	Silty soils	Fair!
17	TP-9-1	96.11	85.66	71.31	52.40	17.77	20(max)	A-7-5	Clay soils	Poor!
18	TP-9-2	99.63	97.42	90.80	57.40	26.00	20(max)	A-7-6	Clay soils	Poor!
19	TP-10-1	93.66	87.32	69.77	40.00	10.00	8(max)	A-4	Silty soils	Fair!
20	TP-10-2	95.52	80.65	48.27	30.28	4.97	8(max)	A-4	Silty soils	Fair!

4.5. Consolidation

4.5.1. 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.

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:

I. Compression of the solid matter
II. Compression of water and air within the voids
III. 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 [15]. A study of the compressibility of soils is necessary to be able to forecast the problem settlement of structures on different type of soils [20].

Generally, the volume change in a soil deposit can be divided in to three stages: [2]

- Initial consolidation
- Primary consolidation
- Secondary consolidation

4.5.2. 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.

The sample which field density and natural moisture content already known was air dried. From the air dried sample existing moisture content was taken. The required samples of soils passing sieve number 4.75mm was taken and by quartering the sample it's compacted on a compaction mold in three layers. Finally using sample extruder sample was extruded on to the consolidation ring.

After carefully trimming the soil sample at its top and bottom, it was placed inside the metal ring with porous stone at its top and bottom. A setting load of 5 KPa was applied until the soil saturated fully. Load was applied through the lever arm and compression dial reading was taken at a time interval of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, minutes and 1, 2, 4, 8, and 24hours. The load was doubled every 24 hours strarting from 25kpa to 1600kpa. This procedure was followed for all the samples. The plot of void ratio versues logarizem of pressure and pressure for all the samples is shown in fig 4.9 and 4.10 repectively (see App. C for detail calculations of the consolidation test result).

4.5.2.1. 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 [12].

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log *p* curve of an undisturbed sample as shown in Fig 4.7. From *B,* a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the pre-consolidation pressure, Pc [15].

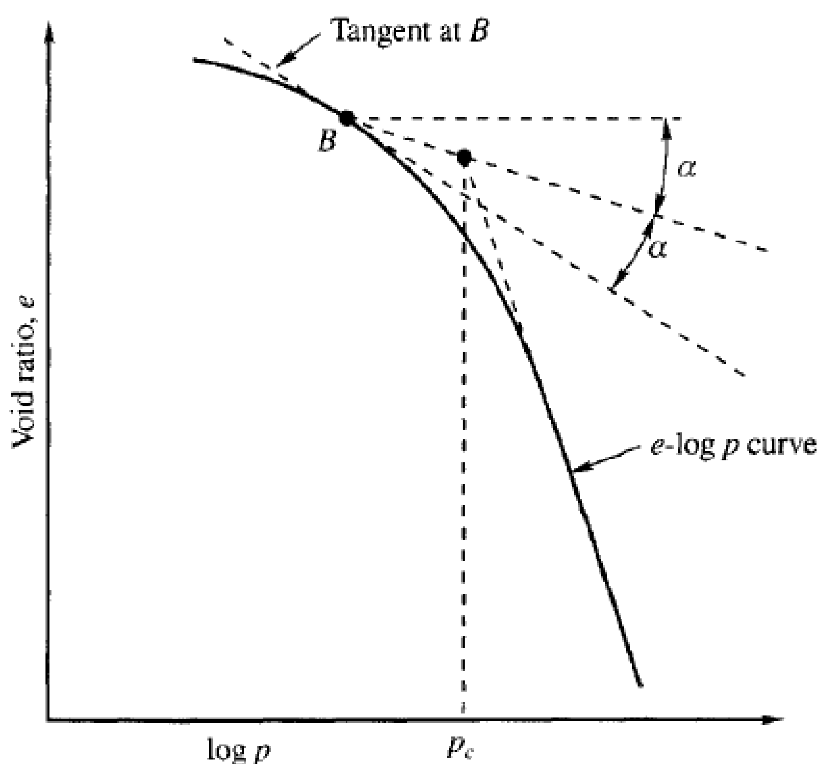

Fig 4.7 Method of determining p_c by Casagrande method [15]

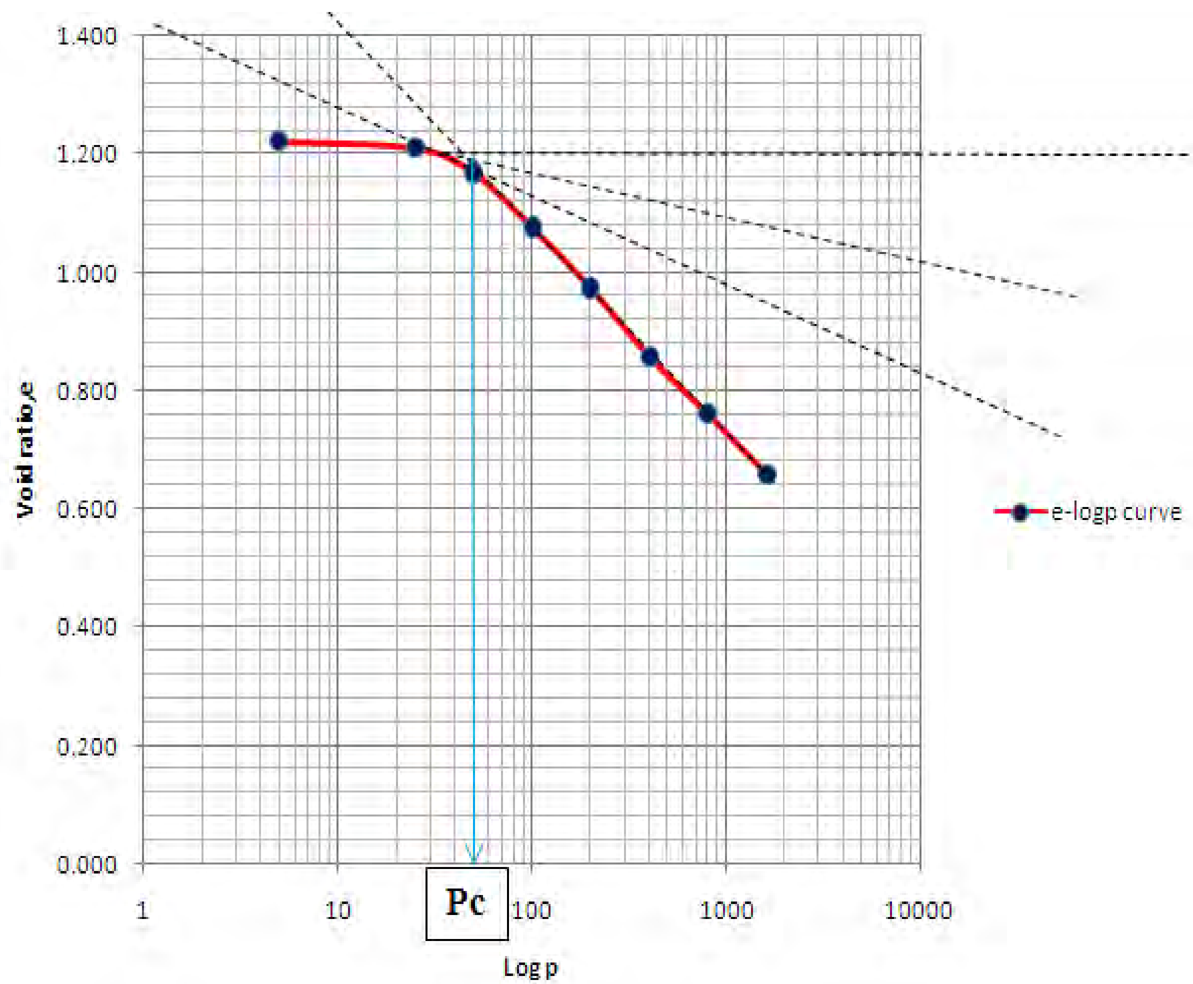

Fig 4.8 Typical void ratio Vs pressure curve to determine Pc

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

$$OCR = \frac{P_c}{P_0} \qquad (4.2)$$

The pre-consolidation pressure for the four samples is determined as depicted in fig 4.9. The results are shown in Table 4.11. From fig 4.9 and 4.10 all the samples has almost similar pre-consolidation pressure.

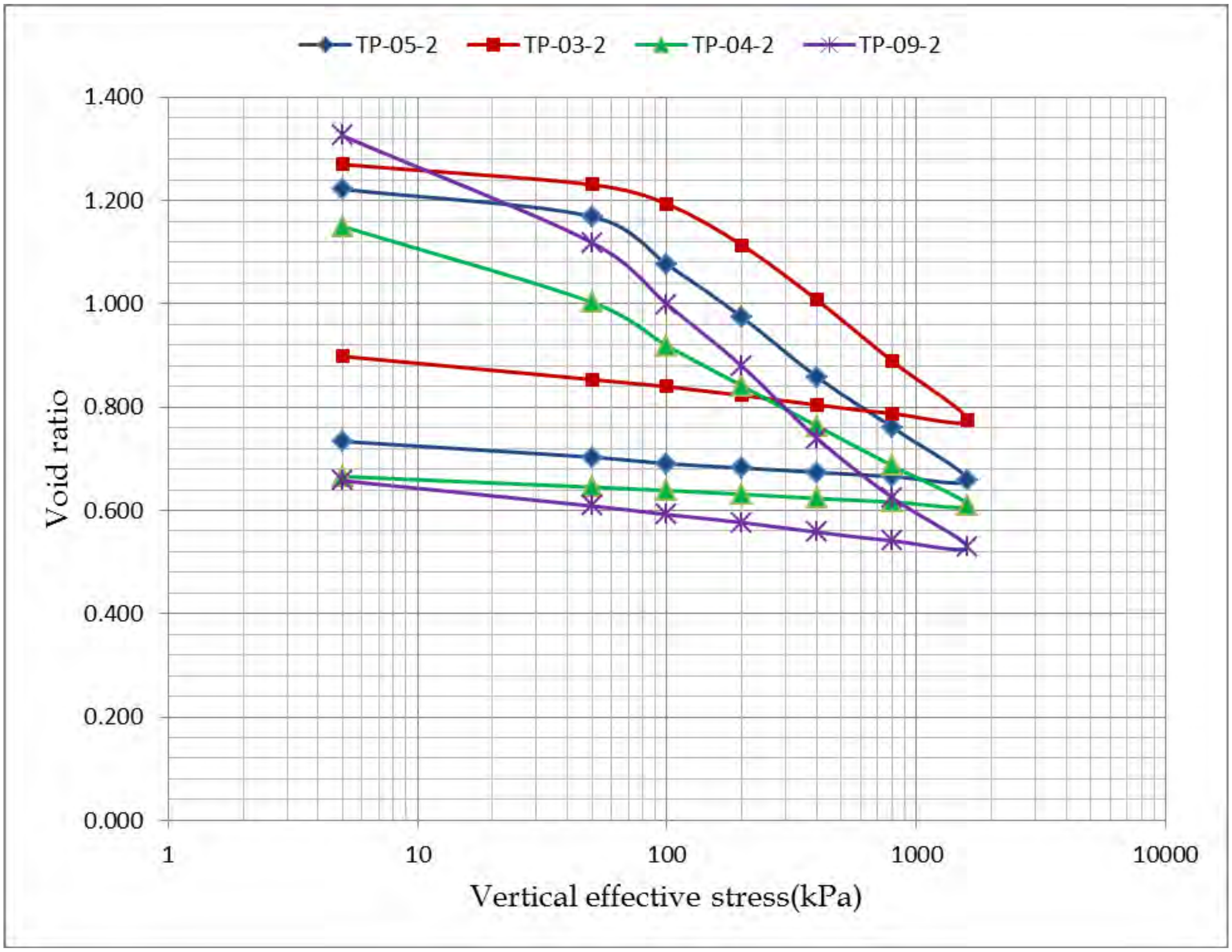

Fig 4.9 Plot of vertical effective stress Vs void ratio on semi-log scale

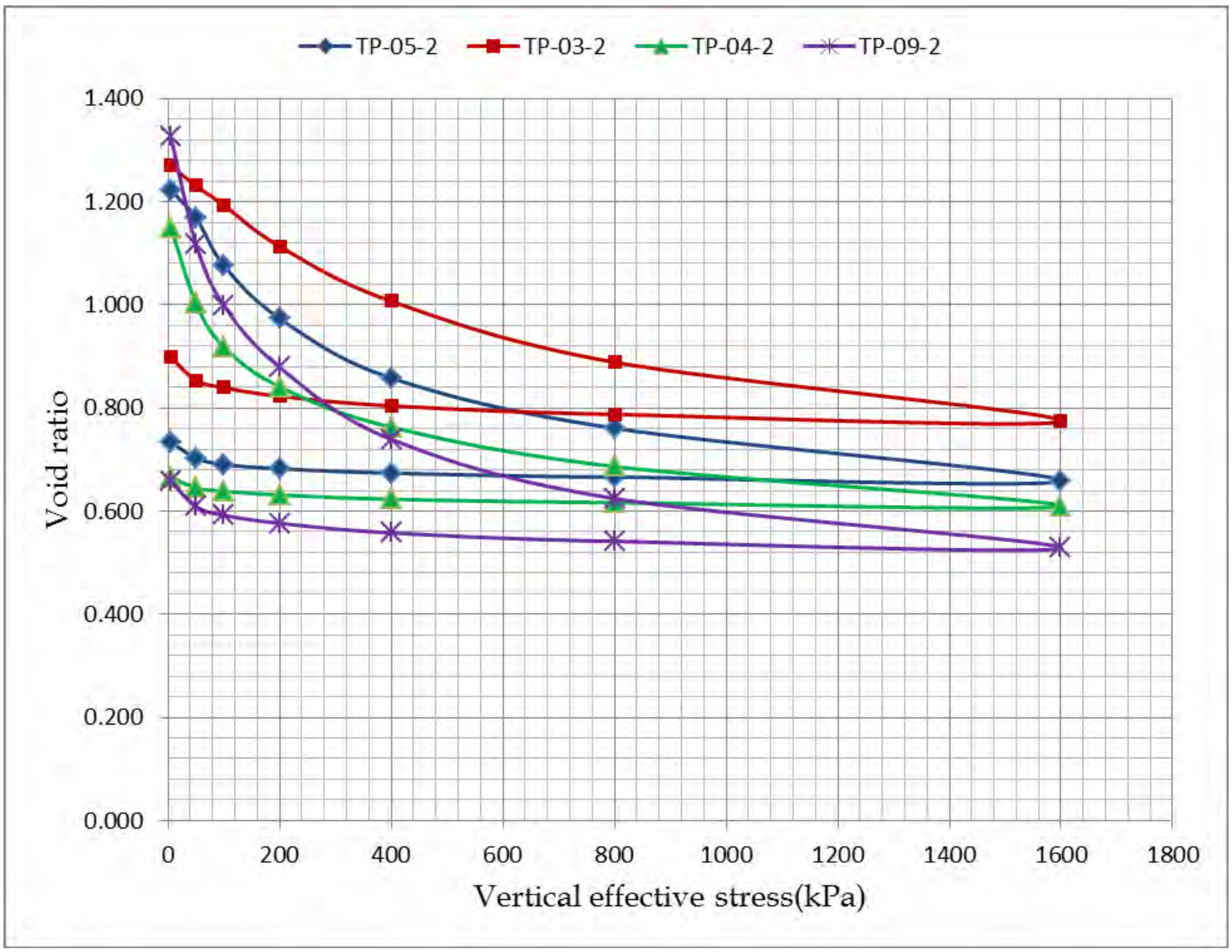

Fig 4.10 Plot of vertical effective stress Vs void ratio on linear scale

4.5.2.2 Compression index

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

$$C_c = \frac{\Delta e}{\log\left(\frac{p_o + \Delta p}{p_o}\right)} \quad \text{.......................................} (4.3)$$

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

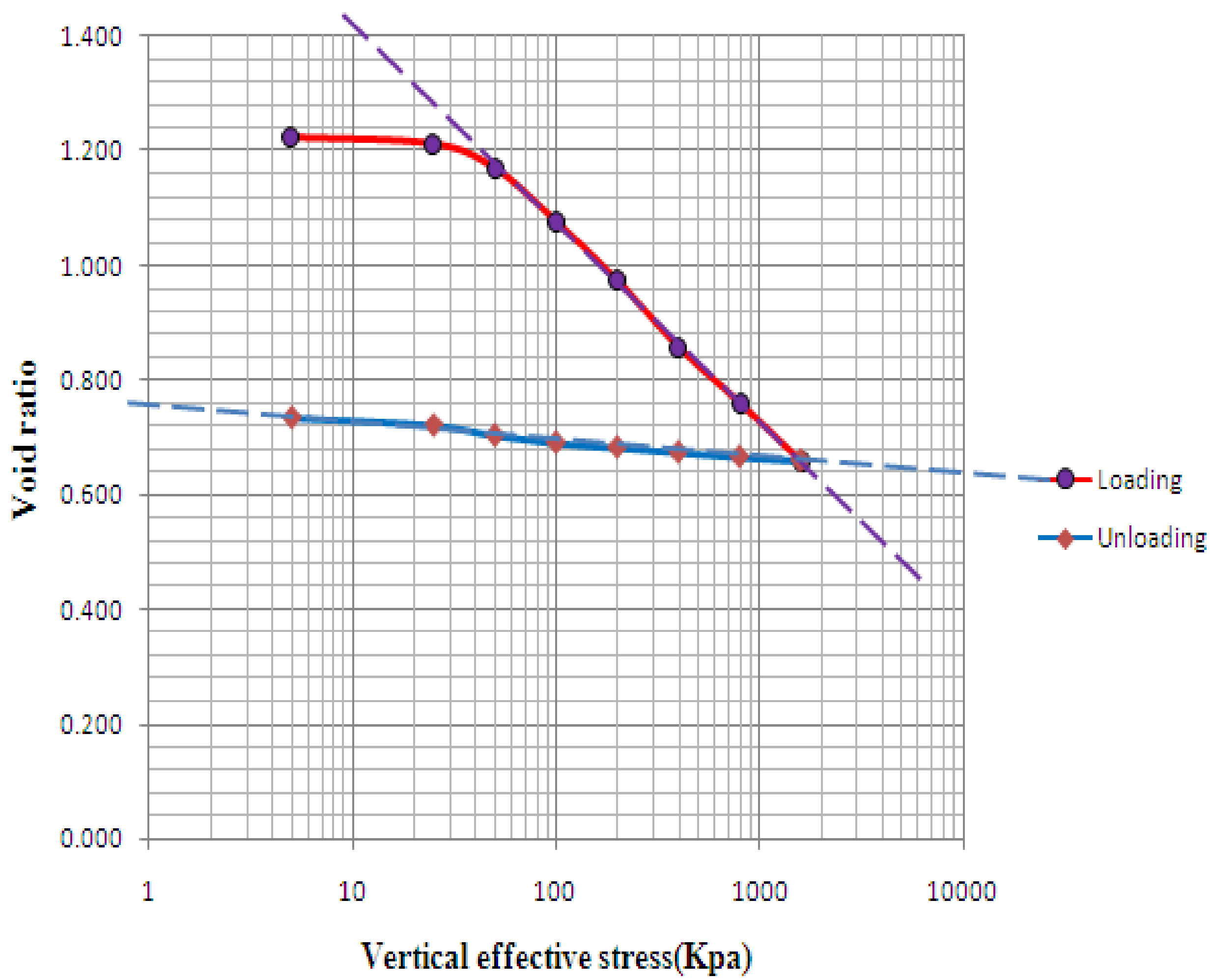

Fig 4.11 Typical loading unloading curve to calculate the coefficient of consolidation and the recompression index

Table 4.11 Summery of the consolidation test results

Test Pit Designation	Depth (m)	Natural Moisture Content (%)	Total Unit weight in (γ) kPa	Pressure P kPa	Void ratio e_f	Coefficient of Consolidation C_v 10^{-3} cm^2/sec	Compression Index C_c	Over-burden pressure Po (kPa)	Pre-consolidation pressure Pc (kPa)	Over-consolidation ratio (OCR)
TP-03-2	3	38	14.7	5	1.270	-	0.360	43.3	103	2.4
				50	1.231	-				
				100	1.193	2.79				
				200	1.114	9.05				
				400	1.007	9.05				
				800	0.889	1.87				
				1600	0.775	0.883				
TP-04-2	3	21	13.8	5	1.150	-	0.327	40.6	60	1.5
				50	1.003	-				
				100	0.919	9.81				
				200	0.841	14.1				
				400	0.763	9.05				
				800	0.687	7.48				
				1600	0.610	6.28				

Table 4.11 (cont'd)

Test Pit Designation	Depth (m)	Natural Moisture Content (%)	Total Unit weight in γ (kPa)	Pressure P kPa	Void ratio e_f	Coefficient of Consolidation C_v 10^{-3} cm^2/sec	Compression Index C_c	Over burden pressure Po (kPa)	Pre-consolidation pressure Pc (kPa)	Over consolidation ratio (OCR)
TP-05-2	3	32	13.4	5	1.222	-	0.335	39.4	50	1.3
				25	1.211	-				
				50	1.169	-				
				100	1.077	9.050				
				200	0.974	11.70				
				400	0.858	9.050				
				800	0.761	5.350				
				1600	0.659	4.610				
TP-09-2	3	27	14.0	5	1.326	-	0.399	41.2	70	1.7
				50	1.119	-				
				100	0.999	-				
				200	0.879	0.513				
				400	0.739	1.340				
				800	0.625	0.883				
				1600	0.529	0.427				

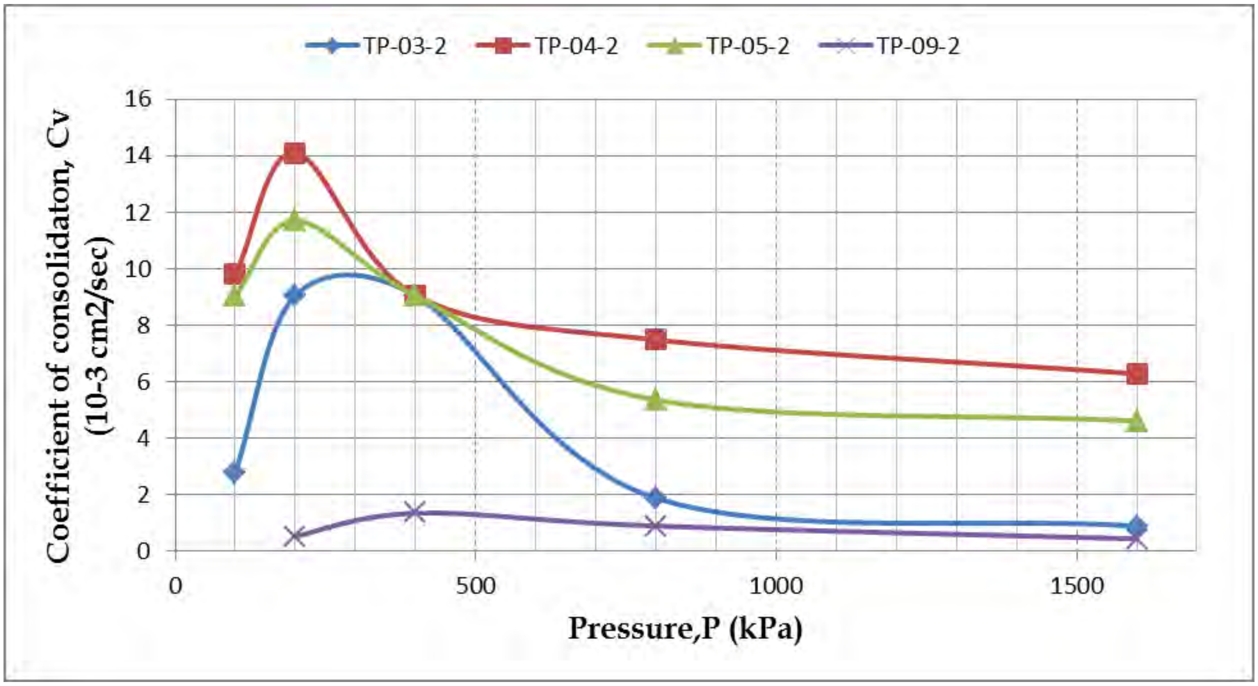

Fig 4.12 Coefficient of Consolidation Vs Pressure

4.5.2.3 Coefficient of consolidation

From those increments of load where time-deformation readings are obtained, two alternative procedures are provided to present the data, determine the end-of-primary consolidation and compute the rate of consolidation [3].

i. Logarithm-of-time-fitting method

A straight line is drawn through the points representing the final readings which exhibit a straight line trend and constant slope. A second straight line is drawn tangent to the steepest part of the deformation-log time curve. The intersection represents the deformation, d_{100}, and time, t_{100}, corresponding to 100% primary consolidation. Compression in excess of the above estimated 100% primary consolidation is defined as secondary compression.

The deformation representing 0% primary consolidation is selected by choosing any two points that have a time ratio of 1 to 4. The deformation at the larger of the two times should be greater than ¼, but less than ½ of the total deformation for the load increment. The deformation corresponding to 0% primary consolidation is equal to the deformation at the smaller time, less the difference in the deformation for the two selected times.

The deformation, d_{50}, corresponding to 50% primary consolidation is equal to the average of the deformations corresponding to the 0 and 100% deformations. The time, t_{50}, requid for 50% consolidation may be found graphically from the deformation-log time curve by observing the time that corresponds to 50% of primary consolidation on the curve.

ii. Square-root-of-time fitting method

A straight line is drawn through the points representing the initial readings that exhibit a straight line trend. Then the line is extrapolated back to t=0 and the deformation ordinate representing 0% primary consolidation is obtained.

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

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

From the measured data and the data obtained from either of the above two methods, the consolidation curve (pressure-void ratio relationship) can be plotted. This data 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 [13].

Because of the fact that during the process of consolidation k and m_v are assumed to be constant, the coefficient of consolidation Cv during the process of consolidation of the clay is constant [12].

The coefficient of consolidation Cv as determined by Casagrande's semi logarithmic plot method is given by:

$$c_v = \frac{(0.196) * H^2}{t_{50}} \quad in \left[\frac{cm^2}{s}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.4)$$

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

$$c_v = \frac{(0.848) * H^2}{t_{90}} \quad in \left[\frac{cm^2}{s}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.5)$$

4.5.2.4 Coefficient of Permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relationship [23].

$$k = \frac{c_v * a_v * \gamma_w}{1 + e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.6)$$

Where: c_v- Coefficient of consolidation

a_v - Coefficient of compressibility

γ_w - Unit weight of water

e_o - Void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.12. It is noted that a_v, the ratio of change in void ratio to change in pressure, was obtained from Fig 4.10. As shown in Table 4.12, the range of values of coefficient of permeability lies between 10^{-7} and10^{-9} cm/sec, which indicates that the soils are practically impervious or have low permeability. In general, void ratio versus log coefficient of permeability is close to a straight line for nearly all soils [13]. As shown in Fig 4.13, for all the soil samples void ratio versus log coefficient of permeability is close to a straight line.

Table 4.12 Relationship between Void ratio and coefficient of permeability

Test Pit Designation	Depth (m)	Pressure P kPa	Void ratio e_f	Coefficient of consolidation C_v 10^{-3} cm^2/sec	Coefficient of compressibility a_v 10^{-5} cm^2/KN	Coefficient of permeability k 10^{-8} cm/sec
TP-03-2	3.0	100	1.193	2.79	78.50	9.98
		200	1.114	9.05	66.72	28.57
		400	1.007	9.05	40.00	18.04
		800	0.889	1.87	30.00	2.97
		1600	0.775	0.883	12.45	0.62
TP-04-2	3.0	100	0.919	9.81	107.81	55.12
		200	0.841	14.1	70.00	53.60
		400	0.763	9.05	50.00	25.67
		800	0.687	7.48	10.00	4.43
		1600	0.610	6.28	3.55	1.38
TP-05-2	3.0	100	1.077	9.05	130.00	56.66
		200	0.974	11.7	88.00	52.16
		400	0.858	9.05	40.00	19.48
		800	0.761	5.35	20.00	6.08
		1600	0.659	4.61	10.35	2.88
TP-09-2	3.0	100	0.999	-	-	-
		200	0.879	0.513	99.5	2.72
		400	0.739	1.34	50	3.85
		800	0.625	0.883	15	0.82
		1600	0.529	0.427	10.64	0.30

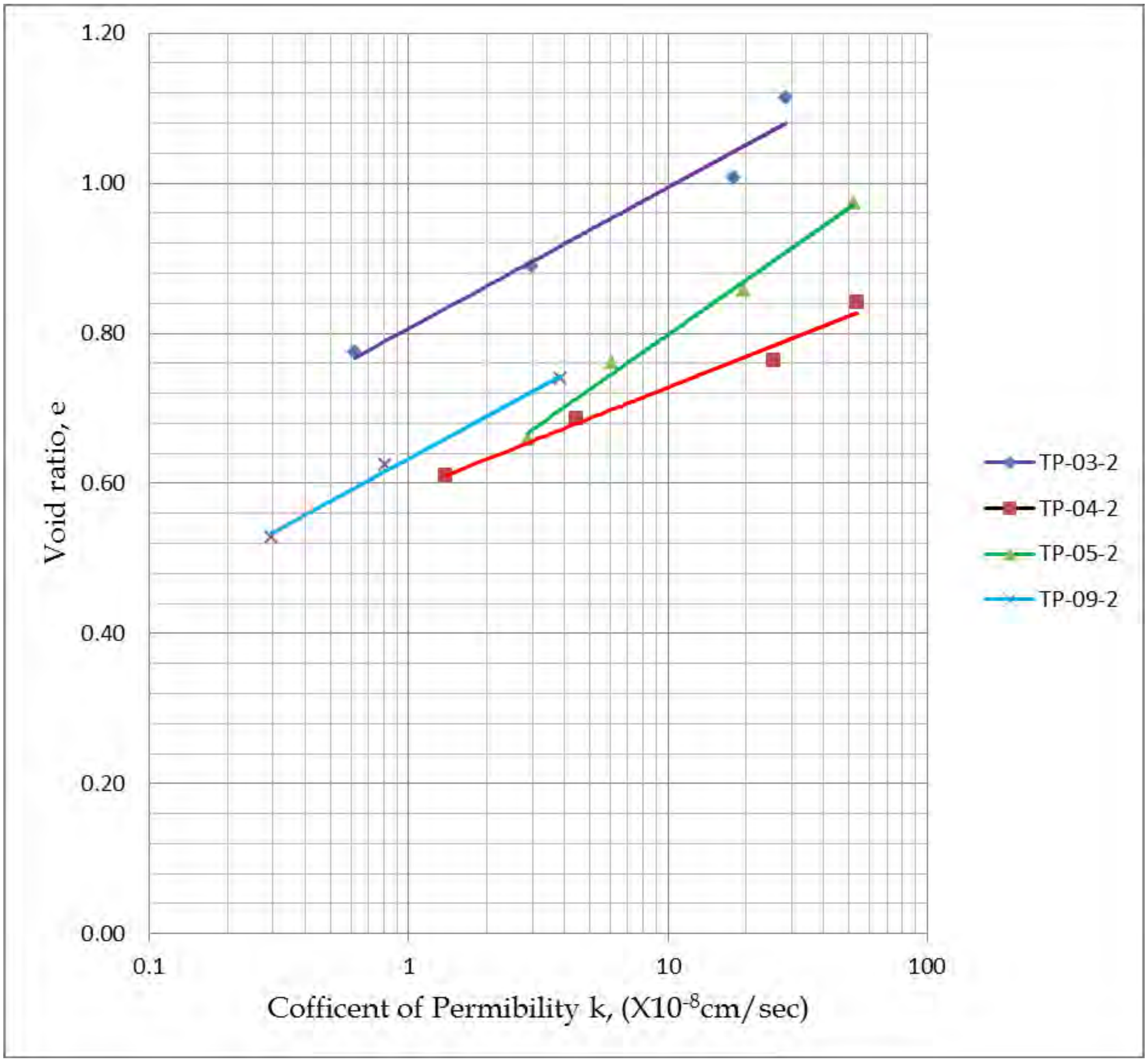

Fig 4.13 Void ratio Vs Log Coefficient of Permeability

4.5.2.5. Modulus of compressibility

As stated in the journal of EAEA [20] the compressibility curve obtained from the consolidation test may be expressed with sufficient accuracy by the equation of Ohde is

$$E_s = \frac{\partial \sigma'}{\partial s'} = \upsilon(\sigma')^{w} \quad \ldots\ldots\ldots (4.7)$$

In the above equations s′ is the relative settlement, υ and w are coefficients where υ has a unit of kPa. It depends on the void ratio, water (moisture) content and consistency of the sample. It could have values ranging from 50 to 30000 KPa.

The coefficient w is a dimensionless quantity which depends on the soil type. It could have values ranging from 0 to 1. The tangent of the compressibility curve, which is a function of σ′ gives the modulus of compressibility Es.

From Eq. (4.7)

$$\frac{\partial s'}{\partial \sigma'} = \frac{1}{\upsilon(\sigma')^{w}} \quad \ldots\ldots\ldots (4.8)$$

$$\partial s' = \frac{(\sigma')^{-w}\partial\sigma}{\upsilon} \quad \ldots\ldots\ldots (4.9)$$

$$s' = \frac{1}{\upsilon}\int(\sigma')^{-w}\partial\sigma \quad \ldots\ldots\ldots 4.10)$$

Case #1: for w ≠ 1

$$s' = \frac{1}{\upsilon(1-w)}(\sigma')^{1-w} + c \quad \ldots\ldots\ldots (4.11)$$

Defining a = $\frac{1}{\upsilon(1-w)}$ and k = 1-w, equ. (4.11) becomes

$$s' = a(\sigma')^k + c \text{..} \quad (4.12)$$

Case #2: for w = 1

$$\frac{\partial \sigma'}{\partial s'} = \frac{1}{v\sigma} \text{..} \quad (4.13)$$

$$s' = \frac{1}{v} \ln\sigma' + c \text{....................................... ....} \quad (4.14)$$

Where: c – constant of integration

As it's observed from figure 4.14, when σ' equal to zero, s′ will also be zero. Therefore equation (4.12) becomes:

$$s' = a(\sigma')^k \text{..} \quad (4.15)$$

The same is true for equation (4.14) and taking common logarithm of equation (4.15) it becomes:

$$\log s' = k \log\sigma' + \log a \text{.................. ....} \quad (4.16)$$

If a plot s′ versus lnσ is made, a straight line relationship is obtained for some cohesive soils. This would mean that the compressibility of the soil is as described in case #2. Other soils give straight line relationship when the results are plotted on double logarithmic scale (the first case).

As shown on figure 4.15, the plot of effective normal stress (lnσ′) Vs relative settlements (s′) shows straight line relationship. This shows that the compressibility of the soil is as described in case #2. The values of the coefficients, v and w, were calculated using Fig 4.15 and tabulated on Table 4.14.

Table 4.13 Effective stress, total compression and relative settlements of samples

Test Pit Designation	Depth (m)	Effective stress s' (KN/m^2)	Total compression ΔH (mm)	Relative settlement s' = ΔH/H	Modulus of compressibility KN/m^2
TP-03-2	3.0	5	0.000	0.000	4910
		50	0.352	0.018	
		100	0.689	0.034	
		200	1.408	0.070	
		400	2.366	0.118	
		800	3.431	0.172	
		1600	4.454	0.223	
TP-04-2	3.0	5	0.000	0.000	7512
		50	1.324	0.066	
		100	2.080	0.104	
		200	2.779	0.139	
		400	3.483	0.174	
		800	4.171	0.209	
		1600	4.860	0.243	
TP-05-2	3.0	5	0.000	0.000	4651
		25	0.139	0.007	
		50	0.516	0.026	
		100	1.349	0.067	
		200	2.272	0.114	
		400	3.315	0.166	
		800	4.191	0.210	
		1600	5.104	0.255	
TP-09-2	3.0	5	0.000	0.000	3680
		50	1.863	0.093	
		100	2.939	0.147	
		200	4.019	0.201	

		400	5.277	0.264	
		800	6.309	0.315	
		1600	7.172	0.359	

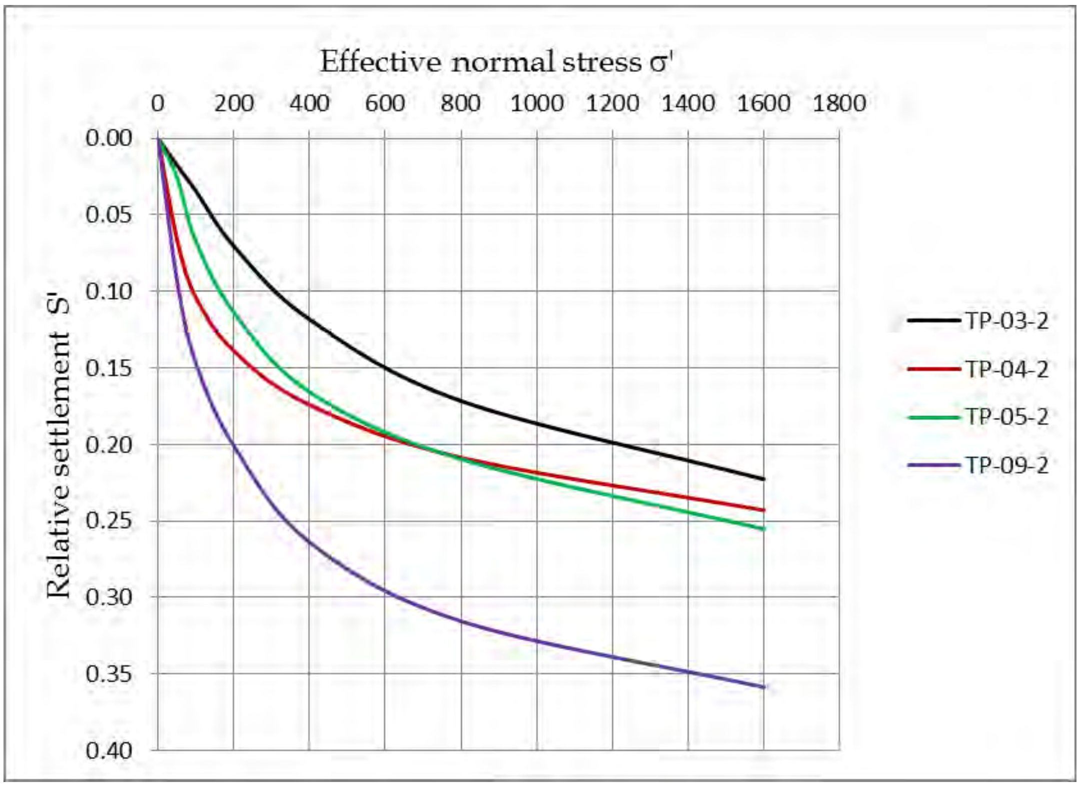

Fig 4.14 Effective normal stress Vs Relative settlements of the samples

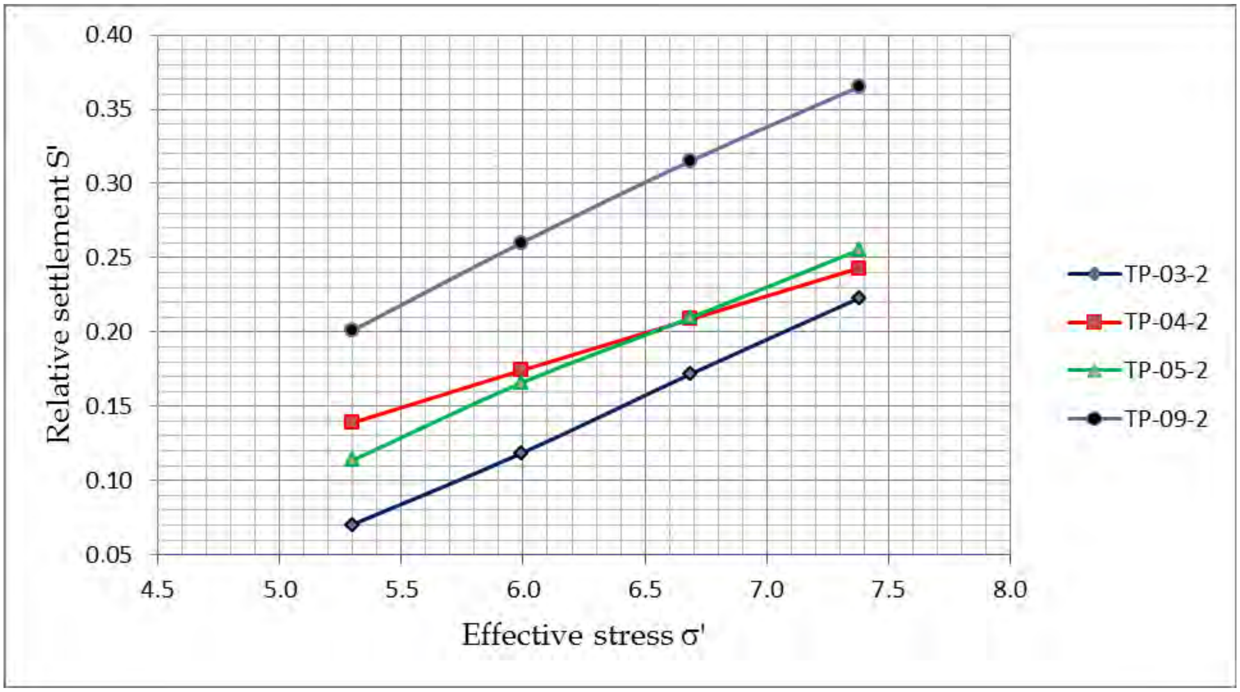

Fig 4.15 Effective normal stress Vs Relative settlements of the samples (lnσ′Vs S′)

Table 4.14 Coefficients, v and w, and equation of the modulus of compressibility

Test Pit Designation	Depth (m)	υ kN/m2	w	Modulus of compressibility, Es KN/m²
TP-03-2	3.0	850	1.0	850(σ′)
TP-04-2	3.0	437	1.0	437(σ′)
TP-05-2	3.0	536	1.0	536(σ′)
TP-09-2	3.0	2396	1.0	2396(σ′)

Table 4.15 Variation of Modulus of compressibility with depth

Sample No.	TP-03-2	TP-04-2	TP-05-2	TP-09-2
Depth (m)	3.0	3.0	3.0	3.0
σ'	Es kN/m2	Es kN/m2	Es kN/m2	Es kN/m2

0	0	0	0	0
100	85000	43700	53600	239600
200	170000	87400	107200	479200
300	255000	131100	160800	718800
400	340000	174800	214400	958400
500	425000	218500	268000	1198000
600	510000	262200	321600	1437600
700	595000	305900	375200	1677200
800	680000	349600	428800	1916800
900	765000	393300	482400	2156400
1000	850000	437000	536000	2396000
1100	935000	480700	589600	2635600
1200	1020000	524400	643200	2875200
1300	1105000	568100	696800	3114800
1400	1190000	611800	750400	3354400
1500	1275000	655500	804000	3594000
1600	1360000	699200	857600	3833600

5. Discussions of the laboratory test results and Comparisons with previously done researches

5.1. Discussions of the laboratory test results

The grain size analysis result is shown in figures 4.2 and 4.3 and the summary of grain size analysis result is shown on Table 4.3. The results obtained from the grain size analyses indicate that the dominant proportion of soil particle in the research area is silt, which has clay content ranging from 5.4 – 40 5%, silt fraction 17.6 - 60.7%, sand fraction 14.5 - 54.6% and gravel content from 0.0 – 24.8%.

The result of Atterberg Limit of the soil samples on air dried and oven dried soil samples is shown on Table 4.5 and 4.6. From these testes the soil under investigation is inorganic. The soil in the research area has liquid limit ranging from 29 – 73%, plastic limit ranging from 21 – 35% and plastic index from 13 – 34%.

Free swell test results for oven dried samples at a temperature of 105$\pm$5 ^{o}c are summarized in Table 4.7. From the test result one can see that the free swell of the soil under investigation ranges from 18% to 50%. Those soils having a free swell less than 50% are considered as low in degree of expansion [23]. Hence all soil samples under investigation are non expansive soils.

Within the depth of exploration, the top layer which is brown silty clay as classified visually has a specific gravity as low as 2.40. The specific gravity of soil becomes lower when it contains organic matter. That is why soil from upper layers of the pit has specific gravity around 2.40. But as excavated deeper it becomes between 2.60 to 2.70. This specific gravity the same as that of found in literatures for silt and silty sand soils.

Fig 4.5 shows plasticity chart of the study area according to Unified Soil Classification System. This chat shows that the soil under investigation lies below the A-line in the region of inorganic silt and inorganic elastic silt. That means inorganic silt with low to high plasticity. This chart also shows that one sample located above A-line, it is because of some organic content of the sample.

Classifications of soils in the study area based on AASHTO Classification system is shown in Table 4.10. And also Fig 4.6 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-4, A-5, and A-7-5/6.

Optimum moisture content and the maximum dry density of the study area is summarized in Table 4.8. From the test results the maximum dry density (MDD) of Adama ranges from1.20 to 1.62 g/cm^3 and the optimum moisture content ranges 17.5 to 36.5 percent.

Figure 4.9 and 4.10 shows the plot of vertical effective stress Vs 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 50-100kPa. 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, C_v, which was calculated from curve of compression dial reading versus square root of time for each incremental loading is plotted as a function of effective stress in Fig 4.12. 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. But one of the samples has slightly different in shape and value of coefficient of consolidation for each incremental loading. This is because the compressibility of any soil type varies with density, history of previous loading, handling prior to and during compression, and in the magnitude of stress increment relative to the existing loading any point.

The compression and recompression index of the soils is calculated from the straight portions of the loading and unloading e-logp curve (Figure 4.9), the typical loading unloading curve as shown in Fig 4.11. This calculation shows that the compression index, Cc, ranges from 0.33-0.40 recompression index, Cr, from 0.017-0.66.

For all the samples the modulus of compressibility varies linearly with effective normal stress. The effective normal stress also shows linear relationship with relative settlements of the samples.

The coefficient of permeability of soil under investigation which is calculated from the test results of consolidation test ranges from 10^{-7} to 10^{-9} cm/sec (table 4.12). The result shows that the soil under investigation is practically impermeable to very low permeability. As shown in fig 4.13 the plot of void ratio versus log permeability coefficient shows straight line.

From Table 4.13 modulus of compressibility of the area ranges from 3600-7500 kPa. As stated in [23], the modulus of compressibility for silt soil ranges from 3000-10000 kPa. Therefore, the test result agrees with this range.

Table 4.15 shows the variation of modulus of compressibility, Es, with effective stress,σ'. From this table it can be observed that, for a given value of effective stress the corresponding value of Es varies from place to place. This is because of the non-uniformity of the degree of stiffness of the soil samples.

5.2. Index property Test Results in different parts of the country.

Table 5.1 Index property Test Results in different parts of the country

	Morin & Parry	Previous Research (Zelalem, 2005)	Previous Research (Haile Mariam,1992)	Previous Research (Wakuma, 2007)	Previous Research (Samuel,1989)			Previous Research (Adiszemen, 2005)	Current Research
Soil type	Red clay	Lateritic	Red clay	Lateritic	Red clay			Black expansive soils	Silt & silt sand
Location	Ethiopia	Nejo-Mendi	Addis Ababa	Asossa	Addis Ababa			Gondor	Adama
					Kolfe	Semen Gebeya	Rufael		
Clay Content (%)	34-76	2.0-20.6	48-73	2.5- 60	58-70	53-68	50-70	41.6-82.25	5.4 – 40 5
Activity		0.97- 0.98		0.62- 1.02				0.76-1.47	
Liquid Limit (%)	44-66	48-67	54-81	41-72	61-75	57-76	56-75	68.89-110.2	29-73
Plasticity Index (%)	14-30	17-27	21-30	20-48	30-43	33-47	29-41	45.85-78.66	5.0-34.0
Shrinkage limit (%)		7.1- 15.7	14-22		15-21	14-25	14-20	2.65-11.56	
Free swell (%)		20-40	10.0-40.0	11.0-45.0	15-45	15-50	30-40	-	18-50
Specific gravity	2.61-2.90	2.78-3.03	2.61-2.79	2.19-2.94	2.66-2.73	2.70-2.77	2.66-2.74		2.4- 2.7
From plasticity chart		MH		CH,SC,MH,CL&SM	CH	CH	CH		SM, ML, MH

6. Conclusions and recommendations

6.1. Conclusion

1. Grain size analysis tests revealed that, starting from few centimeters below the ground level to the depth of investigation which is three meters, the soil in Adama town is mostly silt, and silty sand soil in which the percentage of clay ranges from 5-32%, silt from 18-62%, sand from 15-55% and gravel 0-25%.

2. Due to some organic matters in the sample the specific gravity of the few soil samples is low but mostly the specific gravity of Adama soil is raging from 2.60-2.70.

3. Almost all the samples have free swell value of less than 50%. This shows the soil in the study area is non expansive with free swell value ranging from 18-50%.

4. From consistency limit test results the liquid limit of the area ranges from29-73% plastic limit rages from 21-39% and plastic index from 5-34%.

5. As determined from the one-dimensional consolidation test conducted on remolded soil samples, compression index, Cc, ranges from 0.33-0.40, recompression index, Cr, from 0.017-0.066, coefficient of consolidation, Cv, from 1.63-44.7 ($x10^{-11}$)cm2/year, coefficient of permeability from 0.3-56.7 ($x10^{-8}$ cm/sec) and the modulus of compressibility from 3600-7500 kPa.

6.2. Recommendation.

1. 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.

2. Shear strength property of Adama soil is not studied in this research. Therefore, by studying the Shear strength characteristics soils found Adama, the Correlation of the index property with shear strength parameters may also be done.

7. Reference

1. Adama University, Adama University Guest's Guide, Ethiopia.
2. Arora, K.R., Soil Mechanics and Foundation Engineering, Standard Publishers Distributors, New Delhi.
3. ASTM, 2004, Special Procedures for Testing Soil and Rock for Civil Engineering Purpose, U.S. America.
4. Bowls, J.K., (1978), 'Engineering properties of soil and their measurements', McGraw Hill Book Company, U.S. America.
5. Bowles, J.E., (1996), Foundation Analysis and Design, 5th ed, McGraw-Hill Companies, Inc. New York.
6. Budhu, M., (2000), Soil Mechanics and Foundations, John Wiley and Sons, U.S America.
7. Das, B.M., (1997), Advanced Soil Mechanics, Taylor & Francis, Washington DC.
8. Gilloth, J. E., Clay in Engineering Geology, Elsevier Publishing Company.
9. Girma, R. E., (1962), Applied Clay Mineralogy, McGraw-Hill Book Company, Inc.
10. Giorgis K., T. A. and T. D., Estimating crop water use and simulating yield reduction for Maize and sorghum in Adama and Miesso districts using the cropwat model.
11. James K.M**.,** (1976), Fundamentals of soil behavior, University of California. Berkeley, John Wily& sons, Inc.
12. Jumikis A.R., (1984), Soil Mechanics, Robert E. Krieger Publishing Company, Florida.
13. Krishna R. (2002), Engineering Properties of Soils Based on Laboratory Testing, UIC.
14. Mulugeta M., (2010)**,** Food security attainment role of urban agriculture: a case study from Adama town, central Ethiopia, Journal of Sustainable Development in Africa, Vol.12, No.3.
15. Murthy, V. N. S., (1990), Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering, Marcel Dekker, Inc., New York.
16. NUPI, (1993), Mekelle Development Plan, Final Report: Executive Summary, National Urban Planning Institute (NUPI), Addis Ababa.

17. Punmia B.C., *et al*, Soil Mechanics and Foundations.
18. Robert D.H. and William D. K., (1981), An Introduction to Geotechnical Engineering, PRENTICE HALL, Englewood Cliffs, New Jersey.
19. Stat of Florida Department of Transport, (2000), Soils and Foundation Handbook, State Materials Office, Gainesville, Florida.
20. Tadesse S. (Dr.), Lecture note, Addis Ababa University, Unpublished.
21. Taylor R.M., (1990), Tropical Residual soils, The Quaternary Journal of Engineering Geology, London.
22. Teferra A., (1984), Estimation of settlement of rigid footing, Zede, Journal of the Ethiopian Association of Engineers and Architects (EAEA), Vol.6.
23. Teferra A. and M.Leikun., (1999), Soil Mechanics, Faculty of Technology Addis Ababa University, Addis Ababa.
24. Zelalem A., (2005), Basic Engineering Properties of Lateritic Soils found in Nejo – Mendi Road Construction Area, Welega, Unpublished M.Sc. thesis, A.A.U., Ethiopia.
25. Website,http/en.wikedia.orh/wik/Geotechnical-engineering#geotechnical investigation.
26. Website, (1997), http/www.vulcanhammer.net/.

Appendix – A

(Bore hole-log)

Test pit: 01 Sampled Date: 10/3/2003

Location: Around Adama University

Co-ordinate, x: 37P 0531856, y: UTM 094567 Elevation(m): 1653

As observed from the surface: Small sized gravel with sandy soil

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 0.80		Fill		
∇ - 1.40	+ - +	Reddish silty clay with very small amount of -sand	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	+ +	Silty Sand	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

Test pit: 02 Sampled Date: 14/3/2003

Location: Around Adama city administration

Co-ordinate, x: 37P 0529976, y: UTM 0944650 Elevation(m): 1615

As observed from the surface: Brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ -0.30		Fill	~	~
∇ - 1.00	+ - + + - + - + - + - +	Brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	- + - + - + - + - + -	Reddish silty clay with very small amount of -sand	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

	+ - + - +-			

Test pit: 03 Sampled Date: 13/3/2003

Location: Kebele-09

Co-ordinate, x: 37P 0528655

y: UTM 0943297 Elevation(m): 1612

As observed from the surface: Brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 1.0	+ - + + - + - + - + -	Brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	+ - + - + - + - + - + - + - + - + - + - +	Reddish silty clay with very little fine sand	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

Test pit: 04 Sampled Date: 13/3/2003

Location: Kebele-01

Co-ordinate, x: 37P 0529233

y: UTM 0945691 Elevation(m): 1641

As observed from the surface: Brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 1.00	+ - + + - + - + - + -+	Brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	+ - + - + - + - + - + -	Reddish silty clay with very little fine sand	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

	+ - + - + -			

Test pit: 05 Sampled Date: 13/3/2003

Location: Kebele-03

Co-ordinate, x: 37P 0530258 y: UTM 0941824 Elevation(m): 1598

As observed from the surface: Covered with grass

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 0.80		Fill	~	~
∇ - 3.00	+ - + + - + - + - + -+ - + - + - + - + - + - +	Brown silty clay	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

Test pit: 06 Sampled Date: 13/3/2003

Location: Kebele-05

Co-ordinate, x: 37P 05527055 y: UTM 094385 Elevation(m): 1643

As observed from the surface: Covered with grass

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ -0.30		Fill	~	~
∇ - 1.10	+ - + + - + - + - + - + - + -	Brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	- + - + - + - + - + - + - + -	Reddish silty clay with very small amount of -	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

	+- + - + +	sand		

Test pit: 07 Sampled Date: 14/3/2003
Location: Kebele-14
Co-ordinate, x: 37P 0531379 y: UTM 0944277 Elevation(m): 1604
As observed from the surface: brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 1.10	+ - + + - + - + - + -+	Brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 2.60	+ + O + + + + O	Reddish sandy silt with some gravel	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation
∇ - 3.00	O o o O O o	Hard to drilling with small boulders	~	~

Test pit: 08 Sampled Date: 14/3/2003
Location: Kebele-02
Co-ordinate x: 37P 0530640 y: UTM 0942118 Elevation(m): 1603
As observed from the surface: Brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 0.60	- - - - - - - - -	Brown clay organic soil	~	~
∇ - 1.45	+ + + + + + + + +	Reddish sandy silt	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 2.30	+ + + + + + + + + + + +	Reddish sandy silt which of slightly white	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation
∇ - 3.00	O o O	Hard to drilling	~	~

	O o O o O o	with small boulders		

Test pit: 09 Sampled Date: 15/3/2003

Location: Kebele-04

Co-ordinate, x: 37P 0530683 y: UTM 0945375 Elevation(m): 1634

As observed from the surface: Dark Brown clay

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 0.60		fill		
∇ - 1.85	+ - + + - + - + - + -+	Dark brown silty clay	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	+ + + + + + + +	Brown silty soil with some fine sand	Field density and Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

`` Sampled Date: 15/3/2003

Location: Kebele-03

Co-ordinate, x: 37P 0531303 y: UTM 0939312 Elevation(m): 1633

As observed from the surface: Sandy silt and the area was serving as farm land

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 2.00	++ + + + + + + + +	Reddish sandy silt	Natural moisture content	NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test
∇ - 3.00	+ + O + + + + O	Reddish sandy silt with some gravel	Field density and Natural moisture content	`NMC, Gs, Grain size analysis, Consistency limit test, Compaction test, Free swell test, 1D-consolidation

	+ 0			

Appendix – B

(Meteorological data)

Table B.1. Summary of monthly maximum temperature in ^oc

Element	Monthly Max. Temp. In ^oc	Lat.	Lon.	Alt.
Region	SHOA	39°.17'	8°.33'	1622mt
station	NAZERTH			

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1977	26.4	26.3	29.9	29.4	29.5	28.9	25	25.8	27	27.9	25.4	26.4
1978	27.6	27.7	29.4	32.2	32.5	30.2	25	26.1	27.3	27.5		27.2
1979	26	27.8	30	31.7	30.2	29.3	25.8	28.3	29.4	29.8		
1980										29.1	28.9	28.1
1981	29.6	29.6	28.4	27.6		33	27.4	25.7	25.8	27.9	28.1	27.5
1982	28.2	29.1	31.3	30.4	30.3	31.5		25.1	27.2	25.7	26.2	26.5
1983		28.1	30.7	29.5	30.4	29.8	27.8	25	27.5	28.4	28.5	27.7
1984	27.8	29.3	32.2	33.8	30.5	27.9	26.4		28	30.4	30.1	27.5
1985	29.9		32.1	30.3	30.8	32.4	27.2	25.7	27.9	29.1	30.3	29.3
1986	29.7	31.2	29.7	33.8	34.3	29.6	27.5	28.4	30.8	27.8	28.9	29.8
1987	25	31.7	31.2	29.8	30.2	30.1	29	27	29.8	31.1	30.7	30.6
1988	30.9	32.9	32.3	32.9	33.1	31.6	25.2	27	27.5	27.2	25.2	27.3
1989	28.9	29	28.6	27	32	30.3	24.3	24.7	27	28	28.3	28
1990	26.8	27.5	26.1	26.8	30.3	30	25.6	27.5	26.1	27.3	27.3	25.9
1991	27.7		28.9	29		30.1	25	24.8	26.6	27.5	27.1	26.6
1992	25.8	26.1	29.7	29.7	30.7	30.1	25.3	24.4	25.3	25.9	26.4	25.9
1993	25.3	24.9	29.5	27.9	28.1	28.6	24.8	25.1	26.5			
1994					31.1	28.6	26.4	25.4	25.9	27.1	25.5	25.1
1995	26.9	29	28.7	28.3	30.9	31.6	25.9	25.5	26.5	27.9	27.4	27.5
1996	26.8	29.6	29.5	29	29.2	26.5	26.3	25.5	26.9	27.9	27.5	26.2
1997	28	28.3	30.3	28.5	31.2	29.7	26.1	26.3	28.3	27.3	25.6	25.9
1998	26.9	29.3	28.8	31.6	31.6			25	26.3	26.8	26.5	25.8
1999	27.1	29.9	28.3	31.1	31.5	30.5	24.7	25.2	27.1	25.6	25.9	25.4
2000	27.3	28.3	30.2	30.3	30.8	29.8	25.9	25.6	26.7	25.8	25.8	25.3
2001	24.7	27.8	28.5	29.8	29.6	28.2	25.8	25.3	27.4	28.7	27.3	26.8
2002	26.2	29.2	29.9	31	32.8	30.9	29.7	26.6	28.2	29.8	28.6	26.5
2003	27	29.5	29.8	29.1	32.4	30.1	25.5	25.9	27	28.5	27.5	25.4
2004	27.8	28.2	29.3	28.7		29.6	26.3	26	27.3	26.7	26.8	26.2
2005	26.6	29.6	29.9	30.2	29.1	29.6	25.6	26.7	27.6	28.9	27.5	26.3
2006	27.5	28.7	28.6	28.8	30.6	29.9	25.9	25.2	26.4	27.9	26.9	25.1
2007	26.2	28.4	29.8	28.7	30.9	28.2	25.7	25.4	26.4	26.9	26.0	25.4
2008	26.8	27.1	30.4	29.6	30.1	28.5	25.4	25.1	26.9	27.3	24.7	25.1
2009	25.8	28.0	30.2	29.8	30.0	31.5	26.9	26.2	28.7	27.1	27.3	26.0

Table B.2. Summary of monthly minimum temperature in ^{o}c

Region SHOA

station NAZERTH MON. MIN TEMP

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1977	14.6	12.5	14.5	14.8	15.8	16.7	16	16	15.5	14.6	11.4	10.8
1978	10.9	13.5	15.1	15.4	17.5	17.6	16.3	15.9	15.7	13.1		12.4
1979	13.8	13.4	14.1	15.2	15.9	16.8	15.8	13.5	12.6	9.9		
1980											11.6	9.3
1981	12.1	13.7	15.8	15.1		17.9	15.6	16	15.2	11.6	10.6	10
1982	13.9		15.1	16.2	16.2	17.9		15.5	15.5	11.9	12.9	12.6
1983		15.5	16.7	16.4	16.8	17.6	16.9	16.4	16.6	12.5	10.8	10.6
1984	9.6	10	13.7	15.6	16	16.9	15.5		14	10.9	12.3	11.3
1985	11		14.9	15.7	15.8	17.3	15.1	14.5	15.1	10.2	9.1	10.8
1986	11.2	14.7	13.3	13.5	14.8	16.1	15.7	14.2	15.8	11.2	10.7	16.2
1987	11.8	14.7	15.1	14.1	14.2	18	15.3	16.2	14.9	11.9	9.7	8.2
1988	11.5	15.1	12.6	12.6	14.8	17	11	14.4	15	10	7.6	8.2
1989	8.5	15.5	16.2	14.1	16.7	14.8	12	14.9	14.6	10.7	10.3	13.7
1990	12.7	15.1	14.4	14.4	16.5	17.8	15.7	15.6	15.1	11.9	12.4	11.6
1991	14.4		15.1	14.8		18	15.7	15.6	14.8	13	12.1	11.3
1992	13.6	15.4	16.1	16	15.8	17	15.4	15.5	14	11.8	12.2	13.9
1993	13.7	13.6	13.8	15.1	15.6	16.4	15.4	14.9	14.8			
1994					15.8	16.8	15.7	15.5	13.7	11.2	12.5	11.1
1995	10.5	14.3	14.4	14.8	13.2	15.3	15.6	15.2	13.5	12.9	12.7	14.1
1996	14	13.6	15.4	14.6	14.7	16.6	15.8	15.7	14.6	12.1	13.2	12.1
1997	12.7	15.3						15.9	15.1	15.5	14.7	12.5
1998	14	16.2	16.1	16.3	17	18	16.4	16.1	15	13.5	13	10.6
1999	11.9	13.1	14.9	14.6	16.3	16.9	15.4	15.7	14.8	14.4	9.8	7.9
2000	9.2	13	15	16	16.7	17.9	15.9	16.2	15.2	13.6	12.6	11.5
2001	12.2	13.3	15.1	15.3	17.1	17.3	16.1	16.5	13.9	13.5	12.9	13
2002	14.4	13.9	15.9	16.3	17.9	18.3	17.3	16.4	15	13.6	13.3	15
2003	13.2	14.7	13.3	16	14.7	16.6	15.9	15.8	15.5	13.4	13.7	12.1
2004	14.8	12.9	13.3	15.9	16.5	17.6	16.3	16.3	15.4	13.6	13.6	14
2005	13.3	14.9	16.4	16.4	16.8	17.7	16.4	16.4	16.2	14.5	14.5	12.2
2006	14	15.8	15.4	15.6	16.8	17.8	16.6	16.1	16.1	15.2	14.8	14.7
2007	14.0	15.8	16.1	16.5	17.9	17.6	16.7	15.6	16.5	13.1	13.9	11.6
2008	13.9	13.7	14.0	16.9	17.2	16.8	15.5	15.4	15.9	14.2	12.8	12.1

2009	13.5	15.2	16.8	17.1	17.5	17.8	16.8	16.7	16.3	14.3	13.3	15.0

Table B.3. Summary of monthly Rainfall Data (mm)

Element: Monthly Rainfall in mm
Region: SHOA
Station: NAZERETH

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1977	58.2	10.8	58.6	133.7	67.6	139.8	226.2	172.0	83.5	163.0	63.7	0.0
1978	3.2	99.4	2.7	16.3	15.1	59.1	96.2	199.3	82.3	68.3		5.5
1979	114.3	20.6	60.9	8.1	126.4	115.6	92.4	128.6	21.8	17.4		
1980									120.7	39.4	4.3	0.0
1981	0.0	40.7	166.7	57.5		2.9	246.3	309.8	137.9	4.7	0.0	0.0
1982	8.8	41.6	34.9	29.7	79.1	31.7	127.4	259.8	47.6	104.5	31.3	10.9
1983		43.4	33.8	79.3	188.0	24.7	214.8	221.3	72.4	14.3	0.0	0.0
1984	0.0	0.0	4.3	0.4	173.1	84.6	202.7	148.3	66.9	0.0	0.0	19.8
1985	3.0		23.1	183.4	67.3	8.0	405.4	327.1	169.0	0.0	0.0	0.0
1986	0.0	96.5	41.0	6.2	54.4	152.4	263.3	95.0	20.4	0.0	0.0	0.0
1987	0.0	11.2	80.2	81.1	259.6	0.0	161.6	243.4	30.8	0.0	0.0	0.0
1988	34.0	31.3	6.8	50.9	9.4	50.3	155.4	171.4	186.9	52.9	0.0	0.0
1989	0.0	29.9	21.7	95.4	0.0	54.7	182.5	251.2	80.3	5.7	0.0	3.5
1990	0.7	183.9	83.0	114.7	13.3	12.0	337.8	168.7	153.7	10.8	0.0	0.0
1991	0.0		84.2	13.5	22.3	74.4	322.0	232.8	89.0	13.1	0.0	1.7
1992	41.2	27.8	0.0	46.3	8.6	65.9	232.8	210.0	160.3	43.9	0.0	3.5
1993	15.4	51.9	0.0	102.6	72.7	64.1	345.2	142.4	79.3	20.0	0.0	0.0
1994	0.0	0.0	2.6	49.1	26.5	70.1	229.6	171.5	173.8	13.8	35.6	51.0
1995	0.0	36.5	46.7	127.2	33.0	46.5	203.1	251.4	88.2	14.7	0.0	2.8
1996	27.2	0.0	111.3	65.1	115.2	120.2	220.2	250.0	93.9	0.0	7.9	0.0
1997	14.4	0.0	75.3	28.5	6.9	94.0	193.1	240.9	75.5	116.5	31.5	0.0
1998	11.8	25.6	105.2	19.8	79.3	55.3	196.5	220.6	144.7	132.8	0.0	0.0
1999	8.7	0.0	22.6	1.2	18.6	68.0	283.2	194.4	66.3	164.7	3.1	0.0
2000	0.0	0.0	20.2	16.1	51.5	60.8	352.7	271.0	133.6	85.7	64.8	12.9
2001	0.0	6.2	108.3	28.7	177.0	51.2	253.8	145.3	107.8	1.7	0.0	0.3
2002	20.9	11.1	21.4	51.3	22.5	50.2	129.9	205.7	65.3	1.1	0.0	34.5
2003	46.5	69.1	151.2	88.9	3.6	75.2	235.6	279.7	122.8	0.0	5.3	48.8
2004	28.8	3.3	77.4	53.1	1.9	63.3	114.4	227.3	77.1	58.6	12.8	1.6
2005	72.5	6.3	90.1	41.3	71.1	50.2	144.3	165	68.4	6	5.3	0
2006	17.6	88.4	64.6	88.7	27.8	58.7	173.5	225	128.8	10.1	0.5	28.5
2007	23.1	31.6	82.1	101.7	64.7	62.8	225.7	344.4	138	25.4	7.5	0
2008	9.6	0.0	0.0	79.9	69.5	71.3	353.1	302.2	100.3	37.5	69.5	0.8
2009	62.6	0.0	3.1	2.3	22.1	50.0	156.1	113.3	34.0	132.7	0.4	12.5

Appendix-C

(Detail calculations of consolidation test results)

C.1. Test Sample # 01

Sample Designation…………………………TP-05-2

Location…………………………………Kebele-03

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type ……………………………..……Remolded

Field density (γ) ……………………….....1.34g/cm3

Natural moisture content (ω)…………...31.86%

Specific gravity of the soil (Gs)…………2.57

Existing moisture content (ω_e)…………..6.4%

Volume of the mold (V)…………………942.5cm^3

Mass of the ring (M_R)……………………70.507g

Mass of ring + soil (M_{S+R})……………….130.39g

The height of the ring (Ho) ……………..20mm

The inside diameter of the ring (D) ……50mm

Moisture to be added…………………......25.46%

Therefore using the above data, mass of soil required for remolding is 1263g. Out of this total mass the mass water that must be added to the air dried soil is 256g and the mass of soil solid is 1007g.

Area of the consolidation ring = 19.63cm^2

Volume of the consolidation ring = 39.27cm^3

Mass of soil (Ms) = 59.88g

Table C.1. Dial gauge reading for each incremental loading

		Dial Gauge Reading, mm							
		5 [kPa]	25 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
Time (min)	**√time (min)**								
0	0.00	4.9220	4.9230	5.0621	5.4390	6.2715	7.1950	8.2375	9.1140
6	2.45	4.9220	5.0000	5.2150	5.8900	6.7200	7.7200	8.6300	9.4680
15	3.87	4.9220	5.0020	5.2190	5.9600	6.7920	7.7700	8.6620	9.5020
30	5.48	4.9220	5.0050	5.2240	5.9940	6.8280	7.8038	8.6900	9.5320
1	1.00	4.9220	5.0070	5.2310	6.0290	6.8670	7.8570	8.7260	9.5720
2	1.41	4.9220	5.0100	5.2420	6.0605	6.9070	7.9010	8.7700	9.6190
5	2.24	4.9220	5.0141	5.2600	6.0940	6.9610	7.9690	8.8320	9.6920
8	2.83	4.9220	5.0180	5.2710	6.1110	6.9860	8.0050	8.8670	9.7240
15	3.87	4.9220	5.0220	5.2900	6.1365	7.0205	8.0505	8.9130	9.7895
30	5.48	4.9220	5.0290	5.3090	6.1600	7.0520	8.0910	8.9570	9.8420
60	7.75	4.9220	5.0345	5.3365	6.1840	7.0810	8.1440	8.9940	9.8920
120	10.95	4.9220	5.0420	5.3690	6.2085	7.1100	8.1595	9.0290	9.9365
240	15.49	4.9220	5.0490	5.3965	6.2310	7.1425	8.1875	9.0600	9.9700
480	21.91	4.9225	5.0570	5.4165	6.2520	7.1690	8.2115	9.0850	9.9800
1440	37.95	4.9230	5.0621	5.4390	6.2715	7.1950	8.2375	9.1140	10.0265

Table C.2. Cumulative dial gauge reading at the end of each consecutive unloading

Dial Guage Reading, mm							
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	25 [kPa]	5 [kPa]
10.0265	9.9675	9.8955	9.8195	9.7450	9.6320	9.4765	9.3560

C.1.1. Compression index, Cc

Dry mass of the soil (Md) $= \dfrac{Ms}{(1+\omega)}$

$= \dfrac{59.88}{(1+0.3186)}$

= 45.414g

Height of solid (Hs) $= \dfrac{Md}{Gs * A * \rho_w}$

Where:

G = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w = density of water 1g/cm^3

$$\Rightarrow \quad Hs = \frac{45.414}{2.57 * 19.63 * 1} = 9mm$$

Height of void (H_v) = H - Hs

=20 - 9 = 11mm

Initial void ratio (eo) $= \dfrac{V_v}{V_s} = \dfrac{H_v * A}{H * A} = \dfrac{H_v}{H_s}$

$= \frac{11}{9} = 1.222$

<u>For loading of 25KPa</u>

Initial reading = 4.9230

Final dial reading = 5.0621

$\Delta H_1 = 5.0621 - 4.9230$

= 0.1mm

$\Delta e_1 = \dfrac{\Delta H_1}{Hs}$

$= \frac{0.1}{9} = 0.011$

$e_1 = e_o - \Delta e_1$

$= 1.222 - 0.011$

$= 1.211$

<u>For loading of 50KPa</u>

Initial reading = 5.0621

Final dial reading = 5.4390

$\Delta H_2 = 5.4390 - 5.0621$

$= 0.3769mm$

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

$= \frac{0.3769}{9} = 0.0419$

$e_2 = e_1 - \Delta e_2$

$= 1.211 - 0.0419$

$= 1.1691$

For loading of 100KPa

Initial reading = 5.4390

Final dial reading = 6.2715

$\Delta H_3 = 6.2715 - 5.4390$

$= 0.8325mm$

$\Delta e_3 = \frac{\Delta H_3}{H_S}$

$= \frac{0.8325}{9} = 0.0925$

$e_3 = e_2 - \Delta e_3$

$= 1.1691 - 0.0925$

$= 1.0766$

For loading of 200KPa

Initial reading = 6.2715

Final dial reading = 7.1950

$\Delta H_4 = 7.1950 - 6.2715$

$= 0.9235mm$

$\Delta e_4 = \frac{\Delta H_4}{H_S}$

$= \frac{0.9235}{9} = 0.1026$

$e_4 = e_3 - \Delta e_4$

$= 1.0766 - 0.1026$

$= 0.974$

For loading of 400KPa

Initial reading = 7.1950

Final dial reading = 8.2375

$\Delta H_5 = 8.2375 - 7.1950$

$= 1.0425$ mm

$\Delta e_5 = \frac{\Delta H_5}{H_S}$

$= \frac{1.0425}{9} = 0.1158$

$e_5 = e_4 - \Delta e_5$

$= 0.974 - 0.1158$

$= 0.8582$

For loading of 800KPa

Initial reading = 8.2375

Final dial reading = 9.1140

$\Delta H_6 = 9.1140 - 8.2375$

$= 0.8765$ mm

$\Delta e_6 = \frac{\Delta H_6}{H_S}$

$= \frac{0.8765}{9} = 0.0974$

$e_6 = e_5 - \Delta e_6$

$= 0.8582 - 0.0974$

$= 0.7608$

For loading of 1600KPa

Initial reading = 9.1140

Final dial reading = 10.0265

$\Delta H_7 = 10.0265 - 9.1140$

$= 0.9125mm$

$\Delta e_7 = \frac{\Delta H_7}{H_s}$

$= \frac{0.9125}{9} = 0.1014$

$e_7 = e_6 - \Delta e_7$

$= 0.7608 - 0.1014$

$= 0.6594$

For loading of 1600KPa

Initial reading = 9.1140

Final dial reading = 10.0265

$\Delta H_7 = 10.0265 - 9.1140$

$= 0.9125mm$

$\Delta e_7 = \frac{\Delta H_7}{H_s}$

$= \frac{0.9125}{9} = 0.1014$

$e_7 = e_6 - \Delta e_7$

$= 0.7608 - 0.1014$

$= 0.6594$

For unloading of 800KPa

Initial reading = 10.0265

Final dial reading = 9.9675

$\Delta H_8 = 10.0265 - 9.9675$

$= 0.0590mm$

$\Delta e_8 = \frac{\Delta H_8}{H_s}$

$= \frac{0.0590}{9} = 0.0066$

$e_8 = e_7 - \Delta e_8$

$= 0.6594 + 0.0066$

$= 0.666$

For unloading of 400KPa

Initial reading = 9.9675

Final dial reading = 9.8955

$\Delta H_9 = 9.9675 - 9.8955$

$= 0.0720mm$

$\Delta e_9 = \frac{\Delta H_9}{H_S}$

$= \frac{0.0720}{9} = 0.008$

$e_9 = e_8 - \Delta e_9$

$= 0.666 + 0.008$

$= 0.6740$

For unloading of 200KPa

Initial reading = 9.8955

Final dial reading = 9.8195

$\Delta H_{10} = 9.8955 - 9.8195$

$= 0.076mm$

$\Delta e_{10} = \frac{\Delta H_{10}}{H_S}$

$= \frac{0.076}{9} = 0.0084$

$e_{10} = e_9 - \Delta e_{10}$

$= 0.6740 + 0.0084$

$= 0.6824$

For unloading of 100KPa

Initial reading = 9.8195

Final dial reading = 9.7450

$\Delta H_{11} = 9.8195 - 9.7450$

$= 0.0745mm$

$\Delta e_{11} = \frac{\Delta H_{11}}{H_S}$

$= \frac{0.0745}{9} = 0.00828$

$e_{11} = e_{10} - \Delta e_{11}$

$= 0.6824 + 0.00828$

$= 0.6907$

For unloading of 50KPa

Initial reading = 9.7450

Final dial reading = 9.6320

$\Delta H_{12} = 9.7450 - 9.6320$

$= 0.113mm$

$\Delta e_{12} = \frac{\Delta H_{12}}{H_S}$

$= \frac{0.113}{9} = 0.0126$

$e_{12} = e_{11} - \Delta e_{12}$

$= 0.6907 + 0.0126 = 0.7033$

For unloading of 25KPa

Initial reading = 9.6320

Final dial reading = 9.4765

$\Delta H_{13} = 9.6320 - 9.4765$

$= 0.156mm$

$\Delta e_{13} = \frac{\Delta H_{13}}{H_S}$

$= \frac{0.1555}{9} = 0.0173$

$e_{13} = e_{12} - \Delta e_{13}$

$= 0.7033 + 0.0173 = 0.7206$

For unloading of 25KPa

Initial reading = 9.4765

Final dial reading = 9.3560

$$\Delta H_{14} = 9.4765 - 9.3560$$

$$= 0.1205mm$$

$$\Delta e_{14} = \frac{\Delta H_{14}}{H_s}$$

$$= \frac{0.1205}{9} = 0.0134$$

$$e_{14} = e_{13} - \Delta e_{14}$$

$$= 0.7206 + 0.0134 = 0.7340$$

After calculating the change in specimen height, change in void ratio and final void ratio for each incremental loading and unloading the results are summarized in the table below:

Table C.3. Summary of applied pressure Vs void ratio for loading and unloading

Applied Pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height(ΔHi) (mm)	Change in void ratio Δei	Final Specimen Height (mm)	Void Height, Hv (mm)	Void Ratio, e
LOADING						
5	4.9230	0.0000	0.0000	20.00	11.00	1.2220
25	5.0621	0.1000	0.0111	19.90	10.90	1.2109
50	5.4390	0.3770	0.0419	19.52	10.52	1.1690
100	6.2715	0.8325	0.0925	18.69	9.69	1.0765
200	7.1950	0.9235	0.1026	17.77	8.77	0.9739
400	8.2375	1.0425	0.1158	16.72	7.72	0.8581
800	9.1140	0.8765	0.0974	15.85	6.85	0.7607
1600	10.0265	0.9125	0.1014	14.94	5.94	0.6593
UNLOADIN						
1600	10.0265	0.9125	0.1014	14.94	5.94	0.6593
800	9.9675	0.0590	0.0066	14.99	5.99	0.6658
400	9.8955	0.0720	0.0080	15.07	6.07	0.6738

200	9.8195	0.0760	0.0084	15.14	6.14	0.6823
100	9.7450	0.0745	0.0083	15.22	6.22	0.6906
50	9.6320	0.1130	0.0126	15.33	6.33	0.7031
25	9.4765	0.1555	0.0173	15.49	6.49	0.7204
5	9.3560	0.1205	0.0134	15.61	6.61	0.7338

Using the deformation results (void ratio or strain) corresponding to the end each increment loading or unloading versus logarithm of pressure and pressure respectively is drawn. These graphs are shown in fig 4.9 and fig 4.10. Based on this plot, the compression index, Cc will be the slope of loading curve and recompression index, Cr will be the slope of unloading curve. Therefore, by taking any two points on the straight portions of figure 4.9 for both loading and unloading:

$$\text{The compression index, } C_C = \frac{\Delta e}{\log(\frac{p_o + \Delta p}{p_0})}$$

$$C_C = \frac{1.08 - 0.760}{\log(\frac{100 + 800}{100})}$$

$$= 0.3353$$

$$\text{The recompression index, } C_r = \frac{\Delta e}{\log(\frac{p_o + \Delta p}{p_0})}$$

$$C_r = \frac{0.68 \quad 0.66}{\log(\frac{100 + 1500}{100})}$$

$$=0.0166$$

C.1.2. Coefficient of permeability

The coefficient of permeability can be obtained from the following relationship [23].

$$k = \frac{c_v * a_v * \gamma_w}{1 + e}$$

Where: c_v- Coefficient of consolidation

a_v - Coefficient of compressibility

γ_w - Unit weight of water

e - Void ratio

<u>For loading of 1600 kPa</u>

Void ratio (e) = 0.6593

From figure 4.12 C_v = 4.610 X 10^{-3} cm^2/sec

Unit weight of water (γ_w) = 10 KN/m^3

$$\text{From figure 4.10 } a_v = \frac{0.68 - 0.6593}{1400 - 1600}$$

$\Rightarrow a_v$ = 10.35 x 10^{-5} m^2/kN

$$k = \frac{4.610 X 10^{-3} \frac{cm2}{sec} * 10.35 x 10^{-5} \frac{m2}{kN} * 10 KN/m3}{1 + 0.6593}$$

= 2.876 X 10^{-8} cm/sec

<u>For loading of 800 kPa</u>

Void ratio (e) = 0.7607

From figure 4.12 C_v = 5.35X 10^{-3} cm^2/sec

Unit weight of water (γ_w) = 10 KN/m^3

$$\text{From figure 4.10 } = \frac{0.74 - 0.78}{900 - 700}$$

$\Rightarrow a_v$ = 20.0 x 10^{-5} m^2/kN

$$k = \frac{5.350X 10^{-3} \frac{cm2}{sec} * 20.0 x 10^{-5} \frac{m2}{kN} * 10 KN/m3}{1 + 0.7607}$$

$= 6.077X\ 10^{-8}$ cm/sec

Following the same procedure coefficient of compressibility and coefficient of permeability for each incremental loading is shown on table 4.12.

C.1.3. Modulus of compressibility

Two points, namely σ'_{min} and σ'_{max} are located on relative settlement Vs effective st5ress curve as shown in fig 4.14 and these two points are then joined by straight line gives Es.

$$Es = \tan\alpha = \frac{\Delta\sigma}{\Delta s'}$$

$$\text{From fig 4.12 : } Es = \frac{800 - 100}{0.20955 - 0.06743}$$

$= 4910\ KN/m^2$

As shown on figure 4.15, the plot of effective normal stress (lnσ') Vs relative settlements (s') shows straight line relationship.

$\Rightarrow W = 1$

$$\text{and} \quad s' = \frac{1}{v} \ln\sigma'$$

$$\Rightarrow \quad v = \frac{1}{s'} \ln\sigma'$$

From figure 4.15: υ = 2393 for sample #01, 850 for sample #02.5, 437 for sample #03, and 536for sample #04.#

C.2.Test Sample # 02

Sample Designation …………………………TP-03-2

Location……………………………………...Kebele-09

Sample depth………………………… …….3.0m

Data before commencement of the consolidation test:

Sample type ……………………….…...Remolded

Field density (γ) ………………………..1.47g/cm3

Natural moisture content (ω)…………37.5%

Specific gravity of the soil………………2.67

Existing moisture content………………10.75%

Volume of the mold (V)………………..942.5cm^3

Mass of the ring (M_R)…………………..70.497g

Mass of ring + soil (M_{S+R})……………...133.916g

The height of the ring (Ho) …………….20mm

The inside diameter of the ring (D) ……50mm

Moisture to be added………………......26.75%

Data at the end of the consolidation test:

Mass of ring + wet soil+pan (M_{S+R+p})……………………139.402g

Mass of ring + oven dried soil+pan (M_{S+R+p})……………121.548g

Mass of ring + pan (M_{S+p})……………………………….75.791g

Moisture content at the end of testing………………………..39%

Therefore, using the above data mass of soil required for remolding is 1390g. Out of this total mass the mass water that must be added to the air dried soil is 293g and the mass of soil solid is 1096g.

Area of the consolidation ring = 19.63cm^2

Volume of the consolidation ring = 39.27cm^3

Mass of soil (Ms) = 63.42g

C.2.1.Dial gauge reading for each incremental loading

		Dial Gauge Reading, mm						
Time(min.)	√time	5 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1.560	1.5640	1.9160	2.2530	2.9720	3.9300	4.9950
0.1	0.32	-	1.7440	2.0080	2.5300	3.3400	4.3140	5.2000
0.25	0.50	-	1.7680	2.0260	2.5700	3.3900	4.3420	5.2420
0.50	0.71	-	1.7780	2.0380	2.6010	3.4340	4.3700	5.2900
1	1.00	-	1.7900	2.0540	2.6420	3.4740	4.4030	5.3210
2	1.41	-	1.8020	2.0700	2.6780	3.5210	4.4430	5.3600
4	2.00	-	1.8135	2.0890	2.7180	3.5720	4.4930	5.4200
8	2.83	-	1.8240	2.1120	2.7540	3.6280	4.5580	5.4720
15	3.87	-	1.8330	2.1160	2.7880	3.6790	4.6260	5.5620
30	5.48	-	1.8450	2.1560	2.8250	3.7350	4.7090	5.6800
60	7.75	-	1.8560	2.1750	2.8590	3.7860	4.7940	5.7840
120	10.95	-	1.8700	2.1980	2.8980	3.8460	4.8620	5.8780

240	15.49	-	1.8800	2.2130	2.9260	3.8685	4.9301	5.9435
480	21.91	-	1.8930	2.2340	2.9450	3.8850	4.9630	6.0000
1440	37.95	1.564	1.9160	2.2530	2.9720	3.9300	4.9950	6.0180

C.2.2.Cumulative dials gauge reading at the end of each consecutive unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	50 [kPa]
6.0180	5.9060	5.7550	6.5875	5.4370	5.3120	4.9080

C.2.1. Compression index, Cc

Dry mass of the soil (M_d) $= \frac{Ms}{(1+\omega)} = \frac{63.42}{(1+.375)}$

$= 46.12g$

Height of solid (Hs) = $\frac{Md}{Gs*A*\rho_w}$

Where: G = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w = density of water 1g/cm^3

$\Rightarrow$ Hs = $\frac{46.12}{2.67*19.63*1}$ = 8.81mm

Height of void (H_v) = H - Hs

=20 – 8.81 = 11.19mm

Initial void ratio (e_o) = $\frac{V_V}{V_S}$

$= \frac{H_V*A}{H*A} = \frac{H_V}{H_S} = \frac{11.19}{8.81} = 1.270$

Using this initial void ratio and dial gauge reading similar calculations was done like sample #01. The analysis result is shown in the table C.2.3.

Using the deformation results (void ratio or strain) corresponding to the end each increment loading or unloading versus logarithm of pressure and pressure respectively is drawn. These graphs are shown in fig 4.9 and fig 4.10. Based on this plot, the compression index, Cc will be the slope of loading curve and recompression index, Cr will be the slope of unloading curve. Therefore, by taking any two points on the straight line portion of effective stress Vs void ratio cure on fig 4.9 for both loading and unloading:

$$\text{The compression index, } C_c = \frac{\Delta e}{\log\left(\frac{p_o + \Delta p}{p_0}\right)}$$

$$C_c = \frac{1.08 - 0.80}{\log\left(\frac{250 + 1250}{250}\right)}$$

= 0.3598

$$\text{The recompression index, } C_r = \frac{\Delta e}{\log\left(\frac{p_o + \Delta p}{p_0}\right)}$$

$$C_r = \frac{0.854 - 0.775}{\log\left(\frac{50 + 1550}{50}\right)}$$

=0.0656

Table C.2.3. Summary of applied pressure Vs void ratio for loading and unloading

Applied pressure P	Final Dial Reading	Change In Specimen Height (ΔHi)	Change in void ratio Δei	Final Specimen Height	Void Height, Hv	Void Ratio, e

(kPa)	(mm)	(mm)		(mm)	(mm)	
LOADING						
5	1.5640	0.0000	0.0000	20.00	11.00	1.2700
50	1.9160	0.3520	0.0391	19.65	10.65	1.2309
100	2.2530	0.3370	0.0374	19.31	10.31	1.1934
200	2.9720	0.7190	0.0799	18.59	9.59	1.1136
400	3.9300	0.9580	0.1064	17.63	8.63	1.0071
800	4.9950	1.0650	0.1183	16.57	7.57	0.8888
1600	6.0180	1.0230	0.1137	15.55	6.55	0.7751
UNLOADIN						
1600	6.0180	1.0230	0.1137	15.55	6.55	0.7751
800	5.9060	0.1120	0.0124	15.66	6.66	0.7876
400	5.7550	0.1510	0.0168	15.81	6.81	0.8043
200	5.5875	0.1675	0.0186	15.98	6.98	0.8229
100	5.4370	0.1505	0.0167	16.13	7.13	0.8397
50	5.3120	0.1250	0.0139	16.25	7.25	0.8536
5	4.9080	0.4040	0.0449	16.66	7.66	0.8984

C.2.2. Coefficient of permeability

The coefficient of permeability can be obtained from the following relationship [23].

$$k = \frac{c_v * a_v * \gamma_w}{1 + e}$$

Where: c_v- Coefficient of consolidation

a_v - Coefficient of compressibility

γ_w - Unit weight of water

e - Void ratio

<u>For loading of 1600 kPa</u>

Void ratio (e) = 0.7751

From figure 4.12 $C_v = 0.883 \times 10^{-3}$ cm^2/sec

Unit weight of water (γ_w) = 10 kN/m^3

$$\text{From figure 4.10 } a_v = \frac{0.00 - 0.7751}{1400 - 1600}$$

$\Rightarrow a_v$ = 12.45 x 10^{-5} m^2/kN

$$k = \frac{0.883 X 10^{-3} \frac{cm2}{sec} * 12.45 x 10^{-3} \frac{m2}{kN} * 10 kN/m3}{1 + 0.7751}$$

= 6.2 X 10^{-9} cm/sec

<u>For loading of 800 kPa</u>

Void ratio (e) = 0.8888

From figure 4.12 C_v = 1.87X 10^{-3} cm^2/sec

Unit weight of water (γ_w) = 10 kN/m^3

$$\text{From figure 4.10 } a_v = \frac{0.857 - 0.875}{900 - 700}$$

$\Rightarrow a_v$ = 30.0 x 10^{-5} m^2/kN

$$k = \frac{5.350X 10^{-3} \frac{cm2}{sec} * 30.0 x 10^{-3} \frac{m2}{kN} * 10 kN/m3}{1 + 0.8888}$$

= 2.97X 10^{-8} cm/sec

<u>For loading of 400 kPa</u>

Void ratio (e) = 1.0071

From figure 4.12 C_v = 9.05X 10^{-3} cm^2/sec

Unit weight of water (γ_w) = 10 kN/m^3

$$\text{From figure 4.10 } a_v = \frac{1.06 - 0.98}{900 - 700}$$

$\Rightarrow a_v$ = 40.0 x 10^{-5} m^2/kN

$$k = \frac{9.05X 10^{-3} \frac{cm2}{sec} * 40.0 x 10^{-3} \frac{m2}{kN} * 10 kN/m3}{1 + 1.0071}$$

= 18.04X 10^{-8} cm/sec

Similar calculations were done and Coefficient of compressibility and coefficient of permeability for each incremental loading is summarized on table 4.12.

C.2.3. Modulus of compressibility

Two points, namely σ'_{min} and σ'_{max} are located on relative settlement Vs effective st5ress curve as shown in fig 4.14 and these two points are then joined by straight line gives Es.

$$Es = \tan\alpha = \frac{\Delta\sigma}{\Delta s'}$$

$$\text{From fig 4.14 : } Es = \frac{800 - 400}{0.17155 - 0.1183}$$

$$= 7512 \text{ kN/m}^2$$

<u>C.3. Test Sample # 03</u>

Sample Designation ………………………….TP-04-2

Location……………………………………...Kebele-01

Sample depth………………………… …….3.0m

Data before commencement of the consolidation test:

Sample type ……………………….…….Remolded

Field density (γ) ……………………….....1.38g/cm3

Natural moisture content (ω)……………21%

Specific gravity of the soil…………………2.70

Existing moisture content…………………10%

Volume of the mold (V)…………………..942.5cm^3

Mass of the ring (M_R)……………………...71.16g

Mass of ring + soil (M_{S+R})…………………130.69g

The height of the ring (Ho) ………………20mm

The inside diameter of the ring (D) ……..50mm

Moisture to be added…………………........11%

Data at the end of the consolidation test:

Mass of ring + wet soil+pan (M_{S+R+p})………………………138.77g

Mass of ring + oven dried soil+pan (M_{S+R+p})………………125.14g

Mass of ring + pan (M_{S+p})……………………………………76.55g

Moisture content at the end of testing……………………….28%

Therefore, using the above data mass of soil required for remolding is 1300g. Out of this total mass the mass water that must be added to the air dried soil is 130g and the mass of soil solid is 1170g.

Area of the consolidation ring = 19.63cm^2

Volume of the consolidation ring = 39.27cm^3

Mass of soil (Ms) = 59.53g

Table C.3.1.Dial gauge reading for each incremental loading

		Dial Gauge Reading, mm						
Time(min.)	√time	5 [kPa]	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.860	3.8615	5.1850	5.9415	6.6400	7.3440	8.0320
0.1	0.32	-	4.6700	5.3600	6.1600	6.8800	7.6200	8.3160
0.25	0.50	-	4.7300	5.3720	6.2200	6.9460	7.6580	8.3440
0.50	0.71	-	4.7900	5.4540	6.2600	6.9890	7.6840	8.3680

1	1.00	-	4.8700	5.5250	6.3030	7.0300	7.7180	8.4000
2	1.41	-	4.9460	5.6200	6.3490	7.0720	7.7540	8.4345
4	2.00	-	4.9940	5.6840	6.4100	7.1280	7.8100	8.4880
8	2.83	-	5.0280	5.7215	6.4375	7.1560	7.8370	8.5145
15	3.87	-	5.0640	5.7655	6.4725	7.1920	7.8840	8.5500
30	5.48	-	5.0880	5.8115	6.5085	7.2250	7.9060	8.5860
60	7.75	-	5.1090	5.8510	6.5425	7.2561	7.9370	8.6235
120	10.95	-	5.1320	5.8860	6.5770	7.2855	7.9680	8.6525
240	15.49	-	5.1550	5.9105	6.6035	7.3075	7.9930	8.6780
480	21.91	-	5.1650	5.9280	6.6230	7.3275	8.0125	8.6990
1440	37.95	3.8615	5.1850	5.9415	6.6400	7.3440	8.0320	8.7210

Table C.3.2.Cumulative dials gauge reading at the end of each consecutive unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	5 [kPa]
8.7210	8.6635	8.6030	8.5280	8.4640	8.4060	8.2200

C.3.1. Compression index, Cc

$$\text{Dry mass of the soil (Md)} = \frac{Ms}{(1+\omega)} = \frac{59.53}{(1+0.21)}$$

= 49.2g

$$\text{Height of solid (Hs)} = \frac{Md}{Gs * A * \rho_w}$$

Where: G = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w = density of water 1g/cm^3

$$\Rightarrow \quad Hs = \frac{49.2}{2.7 * 19.63 * 1} = 9.3mm$$

Height of void (H_v) = H - Hs

=20 – 9.30 = 10.70mm

$$\text{Initial void ratio (eo)} = \frac{V_v}{V_s} = \frac{H_v * A}{H * A} = \frac{H_v}{H_s}$$

$$= \frac{10.70}{9.30} = 1.150$$

Using this initial void ratio and dial gauge reading similar calculations was done like sample #01. The analysis result is shown in the table C.3.3.

Using the deformation results (void ratio or strain) corresponding to the end each increment loading or unloading versus logarithm of pressure and pressure respectively is drawn. These graphs are shown in fig 4.9 and fig 4.10. Based on this plot, the compression index, Cc will be the slope of loading curve and recompression index, Cr will be the slope of unloading curve. Therefore, by taking any two points on the straight line portion of effective stress Vs void ratio cure on fig 4.9 for both loading and unloading:

Therefore using equations of consolidation coefficient give above sample calculations:

$$C_c = \frac{0.920 - 0.608}{\log(\frac{100 + 700}{100})}$$

= 0.3267

$$C_r = \frac{0.6101 - 0.6386}{\log(\frac{100 + 1500}{100})}$$

=0.0237

Table C.3.3. Summary of applied pressure Vs void ratio for loading and unloading

Applied	Final	Change	Change	Final	Void	Void

pressure P (kPa)	Dial Reading (mm)	In Specimen Height(ΔHi) (mm)	in void ratio Δei	Specimen Height (mm)	Height Hv (mm)	Ratio e
LOADING						
5	3.8615	0.0000	0.0000	20.00	11.00	1.1500
50	5.1850	1.3235	0.1471	18.68	9.68	1.0029
100	5.9415	0.7565	0.0841	17.92	8.92	0.9189
200	6.6400	0.6985	0.0776	17.22	8.22	0.8413
400	7.3440	0.7040	0.0782	16.52	7.52	0.7631
800	8.0320	0.6880	0.0764	15.83	6.83	0.6866
1600	8.7210	0.6890	0.0766	15.14	6.14	0.6101
UNLOADIN						
1600	8.7210	0.6890	0.0766	15.14	6.14	0.6101
800	8.6635	0.0575	0.0064	15.20	6.20	0.6164
400	8.6030	0.0605	0.0067	15.26	6.26	0.6232
200	8.5280	0.0750	0.0083	15.33	6.33	0.6315
100	8.4640	0.0640	0.0071	15.40	6.40	0.6386
50	8.4060	0.0580	0.0064	15.46	6.46	0.6451
5	8.2200	0.1860	0.0207	15.64	6.64	0.6657

C.2.2. Coefficient of permeability

The coefficient of permeability can be obtained from the following relationship [23].

$$k = \frac{c_v * a_v * \gamma_w}{1 + e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.6)$$

Where: c_v- Coefficient of consolidation

a_v - Coefficient of compressibility

γ_w - Unit weight of water

e - Void ratio

For loading of 1600 kPa

Void ratio (e) = 0.6101

From figure 4.12 $C_v = 6.280 \times 10^{-3}$ cm²/sec

Unit weight of water (γ_w) = 10 kN/m³

$$\text{From figure 4.10 } a_v = \frac{0.6101 - 0.603}{1400 - 1600}$$

$\Rightarrow a_v = 3.55 \times 10^{-5}$ m²/kN

$$k = \frac{6.28 \times 10^{-3} \frac{cm2}{sec} * 3.55 \times 10^{-5} \frac{m2}{kN} * 10\ kN/m3}{1 + 0.6101}$$

$= 1.385 \times 10^{-9}$ cm/sec

Similar calculations were done and Coefficient of compressibility and coefficient of permeability for each incremental loading is summarized on table 4.12.

C.3.3. Modulus of compressibility

Two points, namely σ'_{min} and σ'_{max} are located on relative settlement Vs effective st5ress curve as shown in fig 4.14 and these two points are then joined by straight line gives Es.

$$Es = \tan\alpha = \frac{\Delta\sigma}{\Delta s'}$$

$$\text{From fig 4.14}: Es = (500 - 100)/(0.190 - 0.104) \qquad = 4651\ kN/m^2$$

C.4. Test Sample # 04

Sample Designation………………………………TP-09-2

Location…………………………………………...Kebele-04

Sample depth………………………… ………...3.0m

Data before commencement of the consolidation test:

Sample type ………………………….……….......Remolded

Field density (γ) ……………………...................1.40g/cm3

Natural moisture content (ω)……………………..27.41%

Specific gravity of the soil (Gs)…………………...2.57

Existing moisture content…………………………13.52%

Volume of the mold (V)…………………………...942.5cm^3

Mass of the ring (M_R)……………………………...71.654g

Mass of ring + soil (M_{S+R})………………………...126.943g

The height of the ring (Ho) ………………………20mm

The inside diameter of the ring (D) ………………50mm

Moisture to be added…………………......................13.89%

Data at the end of the consolidation test:

Mass of ring + wet soil+pan (M_{S+R+p})………………136.62g

Mass of ring + oven dried soil+pan (M_{S+R+p})………120.335g

Mass of ring + pan (M_{S+p})……………………………77.0g

Moisture content at the end of testing………………37.6%

Therefore, using the above data mass of soil required for remolding is 1320g. Out of this total mass the mass water that must be added to the air dried soil sample is 161g and the mass of soil solid is 1159g.

Area of the consolidation ring = 19.63cm^2

Volume of the consolidation ring = 39.27cm^3

Mass of soil (Ms) = 55.30g

Table C.4.1Dial gauge reading for each incremental loading

		Dial Gauge Reading, mm						
		5	50	100	200	400	800	1600
Time(min.)	√time	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]

0	0.00	-	1.7170	3.5800	4.6560	5.7360	6.9940	8.0260
0.1	0.32	-	2.9120	4.1000	4.9740	6.2460	7.3000	8.1960
0.25	0.50	-	3.0600	4.1700	4.9980	6.2700	7.3200	8.2200
0.50	0.71	-	3.1400	4.2300	5.0180	6.3000	7.3440	8.2370
1	1.00	-	3.2000	4.2840	5.0420	6.3380	7.3740	8.2620
2	1.41	-	3.2460	4.3420	5.0720	6.3820	7.4120	8.2920
4	2.00	-	3.2820	4.3900	5.1080	6.4400	7.4585	8.3300
8	2.83	-	3.3200	4.4400	5.1440	6.5050	7.5135	8.3800
15	3.87	-	3.3400	4.4820	5.1820	6.5780	7.5760	8.4600
30	5.48	-	3.3700	4.5220	5.2680	6.6740	7.6650	8.5200
60	7.75	-	3.4020	4.5450	5.2740	6.7650	7.7620	8.6200
120	10.95	-	3.4020	4.5838	5.3780	6.8742	7.8440	8.7600
240	15.49	-	3.4420	4.6120	5.4370	6.9145	7.9472	8.8020
480	21.91	-	3.4835	4.6300	5.6880		7.9910	8.8600
1440	37.95	1.717	3.5800	4.6560	5.7360	6.9940	8.0260	8.8890

Table C.4.1Cumulative dials gauge reading at the end of each consecutive unloading

Dial Gauge Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	50 [kPa]
8.8890	8.7770	8.6260	8.4585	8.3180	8.1700	7.7290

C.4.1. Compression index, Cc

$$\text{Dry mass of the soil (Md)} = \frac{Ms}{(1+\omega)} = \frac{55.30}{(1+.2741)}$$

= 43.4g

$$\text{Height of solid (Hs)} = \frac{Md}{Gs * A * \rho_w}$$

Where: G = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w = density of water 1g/cm^3

$$\Rightarrow \quad Hs = \frac{43.4}{2.57 * 19.63 * 1} = 8.6mm$$

Height of void (H_v) = H - Hs

=20 – 8.6 = 11.4mm

$$\text{Initial void ratio (eo)} = \frac{V_v}{V_s} = \frac{H_v * A}{H * A} = \frac{H_v}{H_s}$$

$$= \frac{11.4}{8.6} = 1.3256$$

Using this initial void ratio and dial gauge reading similar calculations was done like sample #01. The analysis result is shown in the table C.3.3.

Using the deformation results (void ratio or strain) corresponding to the end each increment loading or unloading versus logarithm of pressure and pressure respectively is drawn. These graphs are shown in fig 4.9 and fig 4.10. Based on this plot, the compression index, Cc will be the slope of loading curve and recompression index, Cr will be the slope of unloading curve. Therefore, by taking any two points on the straight line portion of effective stress Vs void ratio cure on fig 4.9 for both loading and unloading:

Therefore using equations of consolidation coefficient give above sample calculations:

$$C_c = \frac{1.00 - 0.520}{\log(\frac{100 + 1500}{100})}$$

= 0.3986

$$C_r = \frac{0.5287 - 0.5922}{\log(\frac{100 + 1500}{100})}$$

=0.05274

Table C.4.3. Summary of applied pressure Vs void ratio for loading and unloading

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height(ΔHi) (mm)	Change in void ratio Δei	Final Specimen Height (mm)	Void Height Hv (mm)	Void Ratio e
LOADING						
5	1.7170	0.0000	0.0000	20.00	11.00	1.3256
50	3.5800	1.8630	0.2070	18.14	9.14	1.1186
100	4.6560	1.0760	0.1196	17.06	8.06	0.9990
200	5.7360	1.0800	0.1200	15.98	6.98	0.8790
400	6.9940	1.2580	0.1398	14.72	5.72	0.7393
800	8.0260	1.0320	0.1147	13.69	4.69	0.6246
1600	8.8890	0.8630	0.0959	12.83	3.83	0.5287
UNLOADIN						
1600	8.8890	0.8630	0.0959	12.83	3.83	0.5287
800	8.7770	0.1120	0.0124	12.94	3.94	0.5412
400	8.6260	0.1510	0.0168	13.09	4.09	0.5579
200	8.4585	0.1675	0.0186	13.26	4.26	0.5765
100	8.3180	0.1405	0.0156	13.40	4.40	0.5922
50	8.1700	0.1480	0.0164	13.55	4.55	0.6086
5	7.7290	0.4410	0.0490	13.99	4.99	0.6576

C.2.2. Coefficient of permeability

The coefficient of permeability can be obtained from the following relationship [23].

$$k = \frac{c_v * a_v * \gamma_w}{1 + e}$$

Where: c_v- Coefficient of consolidation

a_v - Coefficient of compressibility

γ_w - Unit weight of water

e - Void ratio

<u>For loading of 1600 kPa</u>

Void ratio (e) = 0.5287

From figure 4.10 $C_v = 0.427\ 10^{-3}\ cm^2/sec$

Unit weight of water (γ_w) = 10 kN/m^3

$$\text{From figure 4.8 } a_v = \frac{0.5287 - 0.55}{1400 - 1600}$$

$\Rightarrow a_v = 10.64x\ 10^{-5}\ m^2/kN$

$$k = \frac{0.427 X 10^{-3} \frac{cm2}{sec} * 10.64 x 10^{-5} \frac{m2}{kN} * 10\ kN/m3}{1 + 0.5287}$$

$= 3.0 X 10^{-9}$ cm/sec

Similar calculations were done and coefficient of compressibility and coefficient of permeability for each incremental loading is summarized on table 4.12.

C.4.3. Modulus of compressibility

Two points, namely σ'_{min} and σ'_{max} are located on relative settlement Vs effective stress curve as shown in fig 4.14 and these two points are then joined by straight line gives Es.

$$Es = \tan\alpha = \frac{\Delta\sigma}{\Delta s'}$$

$$\text{From fig 4.14 : } Es = \frac{700 - 100}{0.31 - 0.14695} = 3680\ kN/m$$

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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