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

Investigations on some of the Engineering properties of soils found in Kemise town

"A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering"

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DECLARATION

I, the undersigned, declare that this thesis is my work and has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

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LIST OF ABBRIVATIONS

AAiT	Addis Ababa institute of Technology
TP	Test Pit
OD	Oven Dry
LL	Liquid Limit
PL	Plastic Limit
PI	Plastic Index
ASTM	American Society for Testing and Materials
A	Activity
G_s	Specific gravity
USCS	Unified Soil Classification System
AASHTO	American Association of State Highway and Transport Officials
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
C	Cohesion
P_c	Preconsolidation pressure
OCR	Over Consolidation Ratio

ABSTRACT

The objective of this research is investigations on some of the engineering properties of soil found in Kemise town. Kemise is one of the reform towns which need many construction works in the future. To find the necessary information from the sub soil, geotechnical investigation on engineering properties of soil is very essential and it is one of the most important parts of Foundation engineering. However, the engineering properties of soils in the area are not studied.

To achieve the central aim of this research nineteen disturbed and undisturbed soil samples were collected from eleven test pits. Thereafter, tests for engineering properties were carried out in the laboratory.

According to engineering properties test results, it is concluded that the type of soils found in Kemise town is clay, silt and silty sand.

Based on the type of soils identified the clay soils have liquid limit ranging from 40 to 84.9%, plastic index ranging from 16.7 to 55.4%. Silt soils have liquid limit ranging from 36.8 to 67.5%, plastic index ranging from 12.1 to 34.7%. While silty sand soil has liquid limit ranging from 30.7 to 37.4%, plastic index ranging from 6.5 to 10%.

The consistency of soils which are stiff to hard, identified based on the liquidity index value. The results obtained from consolidation test, the values of coefficient of consolidation and Coefficient of permeability are reduced with effective stress increment. The coefficient of permeability (10^{-8} to 10^{-9} cm/sec) indicating that the soil sample investigated in study area is practically impermeable (impervious soil).

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1. INTRODUCTION

1.1 General

Kemise town is the administrative center of the Oromiya zone in Amhara region, north eastern part of Ethiopia, 325km far from Addis Ababa.

Field and laboratory investigations are required to obtain the essential information on the subsoil and is one of the most important parts of Foundation Engineering.
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.

The growth of Kemise town has been rapid, since the town became the capital of the new zone of Oromiya and simultaneously became the woreda (sub-zonal administration) town of the Dewa Chefa woreda. However, the engineering properties of soils in the area are not studied. The central aim of the study was to investigating the index properties and consolidation characteristics of soils.

1.2 Objective

The main objective of this research is to investigate some of the engineering properties of soils found in Kemise town.

1.3 Materials and Methods

In order to achieve the objective of this thesis, researches accomplished on engineering properties of soils, documents and some other literatures have been reviewed. In addition, tests have been carried out samples collected from different sites. Sample locations are selected in such a manner as to conform to the sites chosen for the attainment of the proposed objectives. Engineering properties of soils have been determined using laboratory equipment. Because of the difficulty to find samples of different initial moisture content, it was necessary to prepare disturbed soil samples of lower initial moisture content. Material property tests; particle size distribution test, atterberg limit test, specific gravity test, free swell test, compaction tests were conducted on disturbed soil samples. The unconfined compression test and consolidation test were conducted on undisturbed soil samples.

Laboratory work has been carried out in conjunction with data analysis and research report has been written that combines all of the above obtained analysed data to show the engineering properties of soils.

1.4 Scope of the Study

The scope of this study is focused on investigating the engineering properties of soils in the town. Nineteen Soil samples were collected from eleven test pits. Due to budget constraint, the depth of investigation in this research is limited to a maximum of three meters.

1.5 Structure of the thesis

This thesis is organized in to six Chapters. Topics under first Chapter, comprise general, objective, methods and materials, scope of the study and structure of the thesis. The second Chapter deals with literature review. The third Chapter deals with the study area. The topics discussed are soil and geology, borehole locations, topography and drainage conditions and climate (rainfall and temperature). The fourth Chapter deals with field and laboratory tests and their analysis. In Chapter five discussion of laboratory test results are presented. Finally conclusions and recommendations are presented in Chapter six.

2. LITERATURE REVIEW

2.1 Geological Formation and Nature of Soils

Soil is defined as a natural aggregate of mineral grains, with or without organic constituents that can be separated by gentle mechanical means such as agitation in water. By contrast, rock is considered to be a natural aggregate of mineral grains connected by strong and permanent cohesive forces. The process of weathering of the rock decreases the cohesive forces binding the mineral grains and leads to disintegration of bigger masses to smaller and smaller particles. Soils are formed by the process of weathering of the parent rock. The weathering of rocks might be by mechanical disintegration, and/or chemical decomposition [18].

The variety of soil materials encountered in engineering 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 [8].

On the basis of origin of their constituents, soils can be residual and transported. Residual soils are those that remain at the place of their formation as a result of the weathering of parent rocks. The depth of residual soils depends primarily on climatic conditions and the time of exposure. In some areas, this depth might be considerable. In temperate zones residual soils are commonly stiff and stable. An important characteristic of residual soil is that the sizes of grains are indefinite [18]. Transported soils are those developed on unconsolidated sediment that are formed from rock debris which are carried by some natural agent from where they are formed by weathering and erosion to where they now occur [13].

Most of the soil classification systems that have been developed for engineering purposes are based on simple soil-forming processes and index properties. The main factors affecting the formations of soil are: Parent materials i.e. geology of the area, topography and drainage, climate and vegetation cover [8].

2.2 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 [22].

2.3 Topography and Drainage

Topography has a strong and fairly consistent influence on the weathering process, and thus on the type of clay minerals formed, especially in the wet tropics. In hilly and mountainous areas, the soil is well drained and seepage flow has a strong downward component. This leads to the formation of low-activity clay minerals, especially kaolinite. In wide, flat areas, drainage of any sort is much more limited, and moisture movement occurs primarily as a result of seasonal changes [17].

2.4 Climate

The physical properties of soils vary from region to region due to the heterogeneous nature of climate. Moisture and temperature have a major influence on a formation of different type of soil.

2.5 General types of soils

It has been discussed earlier that soil is formed by the process of physical and chemical weathering. The individual size of the constituent parts of even the weathered rock might range from the smallest state (colloidal) to the largest possible (boulders). This implies that all the weathered constituents of a parent rock cannot be termed soil. According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials [18].

2.6 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. Grains finer than 0.075 mm constitute the finer fractions of soils. The molecular structure of particles can be investigated by means of X-ray analysis. The coarser fractions of soils consist of gravel and sand. The individual particles of gravel, which are nothing but fragments of rock, are composed of one or more minerals, whereas sand grains contain mostly one mineral which is 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 [18].

2.7 Mineral composition of soils

Soil has four major components: mineral matter, or broken-down rock; organic matter, or humus, which is the decayed remains of organisms; water; and air. The source of the mineral matter in soil is known as the parent material. The parent material may be either bedrock or unconsolidated deposits, such as those in a river valley [24]. Minerals may be distinguished from one another by their distinctive physical properties. There are two fundamental characteristics of a mineral that together distinguish it from all other minerals are its chemical composition and its crystal structure. Their physical properties, however, are vastly different because of the differences in their internal crystalline structure. Both mineral's composition and crystal structure can usually be determined only by using sophisticated laboratory equipment [13]. The combination of two sheets of silica and gibbsite in different arrangements and conditions lead to the formation of different clay minerals. In the actual formation of the sheet silicate minerals, the phenomenon of isomorphous substitution frequently occurs. Isomorphous (meaning same form) substitution consists of the substitution of one kind of atom for another [18].

X-ray Diffraction method is one of the most important and widely used techniques for clay mineral identification [12]. Of the hundred or so elements known, only eight are abundant at the earth's surface. These, in decreasing order of abundance, are oxygen (O), silicon (Si), aluminum

(Al), iron (Fe), calcium (Ca), sodium (Na), potassium (K) and magnesium (Mg). The common rock-forming minerals are formed mainly of combinations of these important elements, and most of them are silicates [13].

- ✓ 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) [8].

2.8 Related studies around Kemise area

Tesfaye [25] has done index properties, shear strength and dynamic properties of soils found in Dessie town. He has conducted laboratory tests and the free swell value found within the range of marginal to expansive. The value of liquidity index ranging from stiff to hard. He has classified the soil based on unified soil classification system and AASHTO classification system, and the type of soils found in Dessie town were silt and A-7-5 respectively.

Tadesse [26] has done investigation in to some of the engineering properties of soils found in Woldiya town. He has obtained results from laboratory tests and based on the free swell value the soils found in Woldiya town were non-expansive , marginal and expansive. The activity of the soils found within normal to active. He has classified the soil based on unified soil classification system and AASHTO classification system, and the type of soils found in Woldiya town were silty, black clay and A-4, A-7-5 respectively.

3. THE STUDY AREA

3.1 General

Kemise is a town and a separate woreda in northeastern Ethiopia, the administrative center of the Oromiya Zone of the Amhara Region. This town has a latitude and longitude of 10°43′N 39°52′E respectively with elevation of 1450 meters above sea level. The town has a development plan and the growth has is rapid. This growth has been stimulated by two major factors. First, in 1994 Kemise became the capital of the new zone of Oromiya and simultaneously became the woreda (sub-zonal administration) town of the Dewa Chefa *woreda*. This administrative upgrading meant that the settlement attracted many new administrative functions and personnel, and in view of this, a number of good quality houses were built for higher level administrative government employees. Second, and very recently, the boundaries of the town of Kemise were greatly extended, in terms of area and population [14]. The study area map (Kemise Town) is shown below in Fig.3.1.

Fig. 3.1 Location map of Kemise town in Oromiya zone [14]

3.2 Soil and Geology

Soil forms through complex interaction of several factors. The most important factors in soil formation are parent material, time, climate, organisms, and Relief (land forms and topography) [24]. The geological formation i.e., lithology or general composition of rock found in Kemise area is intermediate lava [22].

3.3 Test Pits Location

The area investigated is located between latitudes from 10°42'23'N to 10°43'17'N and longitudes from 39°51'44'E to 39°52'41'E. Eleven boreholes were drilled in the area to collect disturbed and undisturbed natural samples. The test pits locations are shown in Fig.3.2 and the test pit information is listed in Table 3.1.

Table 3.1 : coordinates of sample location

Designation	Location	Latitude	Longitude
TP1	Zone Administration office	10^0 43'01"	39^0 52'03"
TP2	Gelma Abageda Adarash	10^0 43'08"	39^0 52'05"
TP3	Kune	10^0 43'04"	39^0 52'19"
TP4	Green Area	10^0 42'42"	39^0 52'01"
TP5	Fewoz duket Factory	10^0 42'43"	39^0 51'44"
TP6	Around Kemise 02 elementary school	10^0 42'35"	39^0 52'03"
TP7	Amrach Sefer	10^0 42'23"	39^0 52'16"
TP8	Around Mikael Church	10^0 43'08"	39^0 52'30"
TP9	Segno gebeya	10^0 42'51"	39^0 52'27"
TP10	Garagadous (around high school)	10^0 42'52"	39^0 52'41"
TP11	Police memria (around preparatory school)	10^0 43'17"	39^0 51'50"

3.4 Topography and drainage conditions

The Kemise town is located on nearly flat terrain and the broad Cheffa valley area. The part of the town with reference to the main road from Addis Ababa to Dessie is sometimes swampy. The river basin passing near to the town is Borkena.

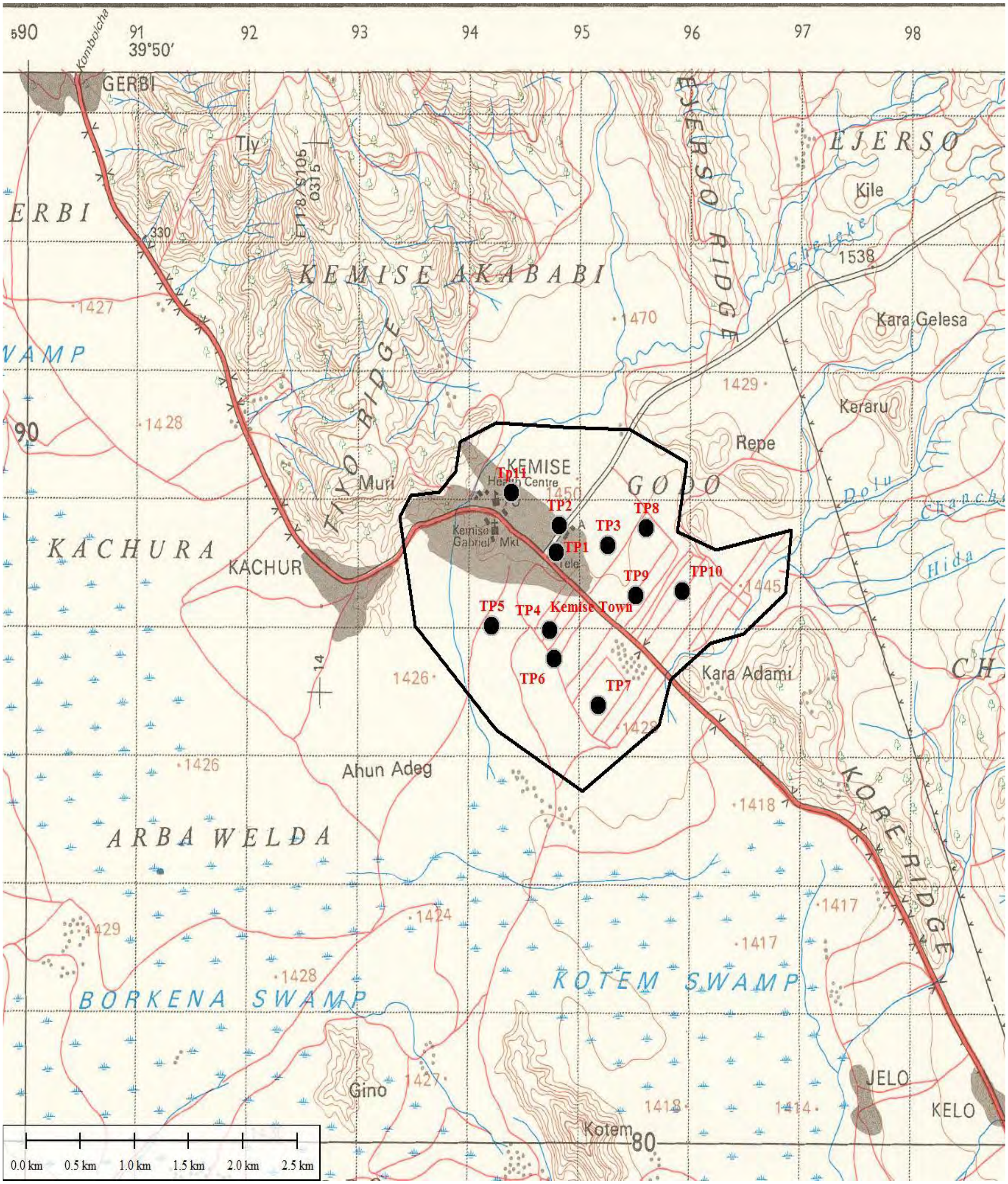

Fig.3.2 Test pits location for Kemise town.

3.5 Climate

3.5.1 Rain Fall and Temperature

The climate in Ethiopia is mainly influenced by altitude and the main climate regions are; Dega (cool to cold temperature) which covers the cool highlands where average temperature falls between 10°c and 16°c with altitude above 2500m above sea level. Weina Dega (warm to cool climate) average annual temperature ranges between 16° and 20°c and comprises much of the highlands between 1500 to 2500m above sea level. Kolla (warm to hot climate) covers the area of the hot lowlands, and the average temperature is between 20°c to 30°c with altitude ranges from 500 to 1500m above sea level. Bereha (hot and arid climate) covers the area of the desert lowlands below 500m above sea level and the average annual temperature is over 30°c [27].

The study area (Kemise town) is found in lowland (Kola) area with latitude of 10°43′0"and longitude of 39°52′0",and elevation of 1450m above mean sea level. Since the altitude is the most significant factor, the climate condition of the town is categorized under warm to hot and it also affects the distribution and seasonal variation of rain fall and temperature.

The recorded data for 1985-2014 from National Meteorological Service Agency shows that the mean monthly rainfall of Kemise ranges from 13.97-332.87mm. From the computed mean monthly rain fall the maximum rain fall takes place for the months of July and August (Fig.3.3).

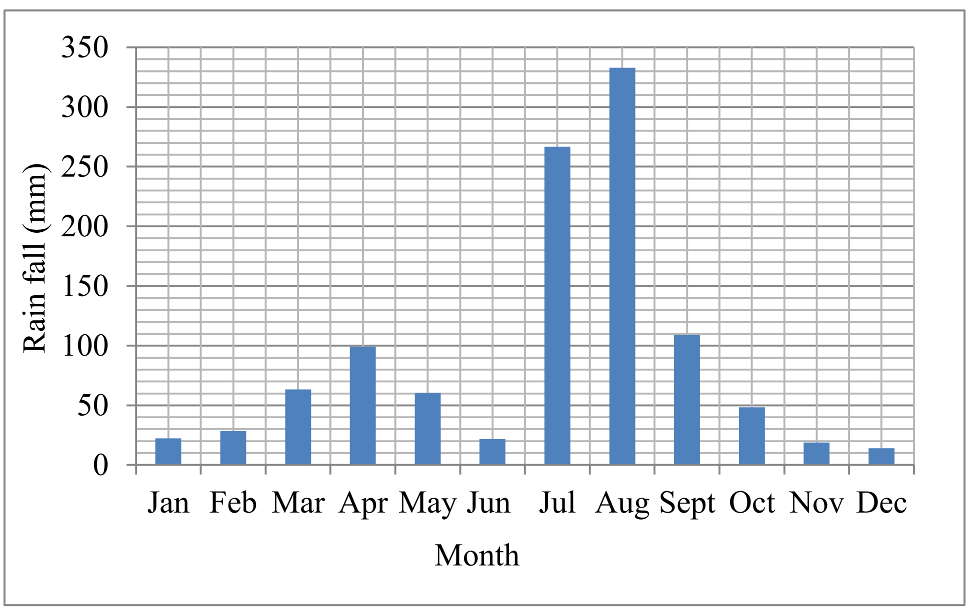

Fig.3.3 Mean monthly rain fall of Kemise (1985-2014)

The recorded temperature data found from National Meteorological Service Agency are shown in Fig.3.4. Based on these data the mean monthly average temperature ranges from 19.51^{0}c to 26.4^{0}c. The hottest months are May and June.

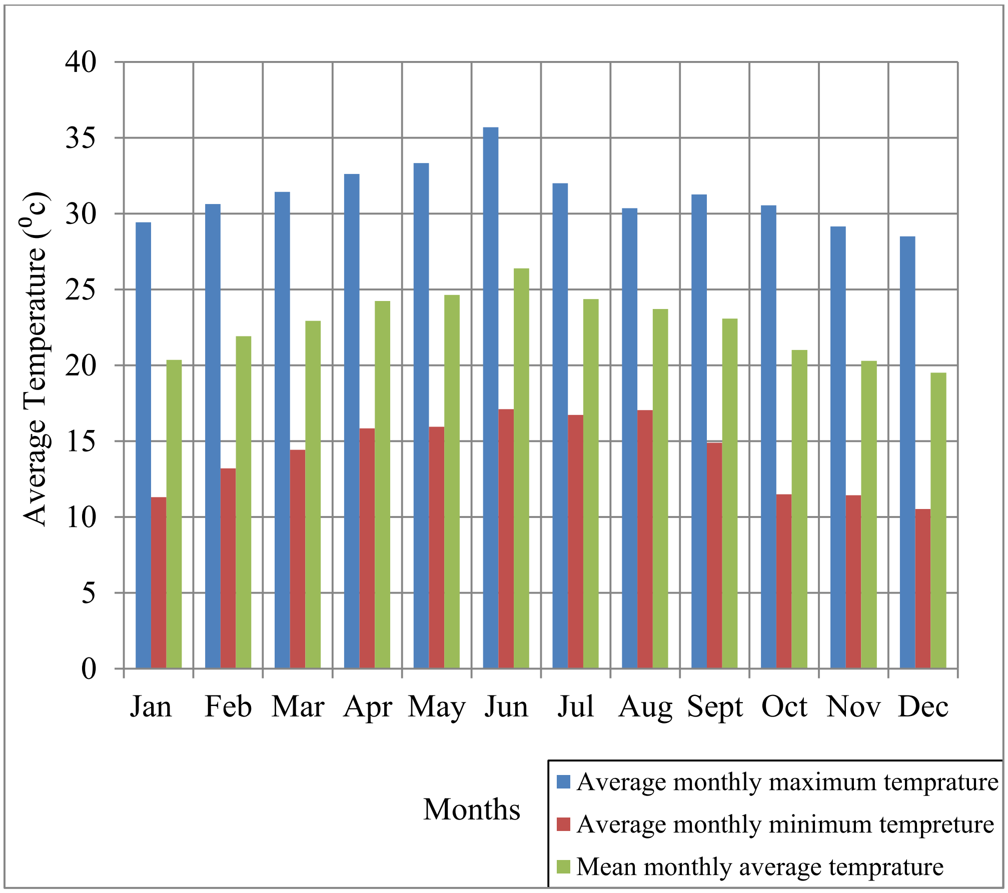

Fig.3.4 Average Monthly Maximum and Minimum, and mean monthly average temperature distribution of Kemise town, (2007-2014G.C)

4. IN-SITU PROPERTIES, LABORATORYTEST RESULTS AND ANALYSIS

4.1 In-situ Properties

4.1.1 Sampling area identification and description

Identification of sampling area has been done by collecting information from walk-over survey. Accordingly eleven test pits have been selected within the town. Test pits were excavated up to depth of 3m. Disturbed and undisturbed soil samples were taken at 1.5m and 3m depths. For test pitsTP6 and TP7, ground water is found at depths of 1.9m and 1.7m respectively. For test pit TP8, a bed rock is found at 1.6m. The study area soil is nearly dark and light gray.

4.2 Laboratory test results and analysis

4.2.1 Index Properties

4.2.1.1 General

The tests required for determination of engineering properties are generally detail and time-consuming. Sometimes, the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting detail tests. This is possible if index properties are determined. Due to the properties of individual particle and soil mass, the index properties should be determined from a remolded, disturbed and undisturbed samples [2].

4.2.1.2 Moisture content

4.2.1.2.1 Effect of temperature on moisture content determination

Water content determinations are made on the recovered soil samples to obtain the natural water content. The water content of a soil is an important property. The characteristics of a soil, especially a fine-grained soil, change to a marked degree with a variation of its water content [2]. Moisture may be present as adsorbed moisture at internal surfaces and as capillary condensed water in small pores. At low relative humidities, moisture consists mainly of adsorbed water. At higher relative humidities, liquid water becomes more and more important, depending on the pore size.

According to ASTM 2216-92 [3], moisture content test were performed in laboratory. Tests were conducted in oven dried condition for the study area. Keeping the soil sample in the oven for 24hr duration and at the average temperature value of 105 °C. If a soil specimen is heated to

105°C most of the water is driven off, although a little will still remain in and around the clay minerals. Heating to a higher temperature would drive off some more water, but the tests were performed at this arbitrary standard temperature. Due to low temperature, under air dried condition limited amount of moisture removed from soil samples. The results are tabulated in Table 4.1.

Table 4.1: The In-situ moisture content

Test pit	**Depth in (m)**	**Moisture content (%)**
TP1	1.5	27.03
	3	32.74
TP2	1.5	21.74
	3	22.45
TP3	1.5	14.01
	3	17.15
TP4	1.5	23.75
	3	31.99
TP5	1.5	27.58
	3	34.85
TP6	1.9	37.21
TP7	1.7	38.4
TP8	1.6	35.77
TP9	1.5	12.9
	3	17.23
TP10	1.5	27.6
	3	36.67
TP11	1.5	23.17
	3	24.85

4.2.1.3 Atterberg Limits

The behavior of all the soils and specially clays differs considerably with the presence of water. Consistency test is one of the index property tests and it is used to determine the degree of firmness of the soil. Based on their mode of formation and mineralogical composition different soils respond differently for the same moisture content.

Atterberg Limits are integral parts of several engineering classification systems to characterize fine grained soil [19]. The different states through which the soil sample passes with the decrease in the moisture content are depicted in Table 4.2

Table 4.2: Different states and consistency of soils with Atterberg limits [18]

States	Limit	Consistency	volume change
Liquid…………………W_i	Liquid limit……….	Very soft	Decrease in volume
		Soft	↓
Plastic………………...W_p	Plastic limit………	Very stiff	
Semi solid…………..…W_s	Shrinkage limit…..	Extremely stiff	
Solid		Hard	Constant volume

4.2.1.3.1 Test results

Atterberg Limits were determined for air-dried soil samples based on ASTM D 4318-98 – Standard Test Method [3]. Effect of mixing time and energy applied will result in more extensive breaking down of cemented bonds between clay clusters [23]. The test result for Atterberg limit is summarized in Table 4.3 below. The detailed test results are given in Appendix A-1.

Table 4.3: Summery of atterberg Limit Result

Test pit	Depth (m)	Liquid limit (LL)	Plastic limit (PL)	Plastic index (PI)
TP1	1.5	53.8	26.2	27.5
	3	64.5	32.5	32.0
TP2	1.5	53.3	30.4	22.8

	3	51.1	33.1	18.0
TP3	1.5	36.8	24.7	12.1
	3	45.3	25.2	20.1
TP4	1.5	61.2	32.4	28.9
	3	45.1	27.5	17.6
TP5	1.5	58.9	31.2	27.7
	3	67.5	33.3	34.2
TP6	1.9	65.8	31.1	34.7
TP7	1.7	59.5	29.2	30.3
TP8	1.6	65.2	31.8	33.5
TP9	1.5	37.4	27.4	10
	3	30.7	24.2	6.5
TP10	1.5	81.4	33.0	48.4
	3	84.9	29.5	55.4
TP11	1.5	52.7	30.9	21.8
	3	40.0	23.3	16.7

4.2.1.3.2 Activity number

The Activity number is given by the following equation.

$$\text{Activity } A = \frac{\text{Plasticity index,} I_p}{\text{Percent finer than 2 micron}} \quad (4.1)$$

Depending upon activity, the soils are classified into three types as given in Table 4.4.

Table 4.4: Classification of soils based on activity [2]

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

The activity numbers were computed by equation 4.1, summarized in Table 4.5. It gives information about the type of clay found in soil.

Table 4.5: Skempton activity number of investigated soils

Test pit	Depth (m)	Plastic Index (PI)	Percent of clay	Activity (A)
TP1	1.5	27.5	36.78	0.75
	3	32.0	39	0.82
TP2	1.5	22.8	36.54	0.62
	3	18.0	33.52	0.54
TP3	1.5	12.1	25.91	0.47
	3	20.1	30.91	0.65
TP4	1.5	28.9	36.11	0.8
	3	17.6	21.78	0.81
TP5	1.5	27.7	32.51	0.85
	3	34.2	45.17	0.76
TP6	1.9	34.7	40.13	0.86
TP7	1.7	30.3	35.36	0.86
TP8	1.6	33.5	46.4	0.72
TP9	1.5	10.0	16.6	0.60
	3	6.5	11.07	0.59
TP10	1.5	48.4	53.39	0.91
	3	55.4	57.16	0.97
TP11	1.5	21.8	30.98	0.7
	3	16.7	50.12	0.33

4.2.1.4 Free Swell

Studying swelling property is to measure the expansive potential of soil [4]. To calculate the volumetric swelling of the soil, the following formula is used.

$$\text{Free Swell}=\frac{\text{Final volume} - \text{Initial volume of the soil}}{\text{Initial volume}} X100 \qquad (4.2)$$

The laboratory test result is summarized in Table 4.6. The results of free swelling test for this specific area ranges from 15% to 97.5%.

Table 4.6: Free swell test results

Test Pit	Depth (m)	Free swell (%)
TP1	1.5	55
	3	57.5
TP2	1.5	40
	3	35
TP3	1.5	35
	3	25
TP4	1.5	35
	3	27.5
TP5	1.5	40
	3	45
TP6	1.9	60
TP7	1.7	85
TP8	1.6	60
TP9	1.5	15
	3	20
TP10	1.5	95
	3	97.5
TP11	1.5	30
	3	75

4.2.1.5 Specific gravity

Specific gravity is defined as the ratio of the unit weight of a given material to the unit weight of water. Thus

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w} \quad (4.3)$$

The type of rock and soil forming mineral can affect the value of specific gravity of the soils. For most minerals G_s values fall within a range of 2.6 to 2.9 [6].

Tests were conducted according to ASTM D 854-92 [3], method-A. The values are summarized in Table 4.7. The typical test result presented in Appendix A-2.

Table 4.7: Specific gravity of study area

Test Pit	Depth (m)	Condition	Specific gravity
TP 1	1.5	OD	2.63
	3	OD	2.64
TP2	1.5	OD	2.63
	3	OD	2.67
TP3	1.5	OD	2.72
	3	OD	2.7
TP4	1.5	OD	2.69
	3	OD	2.64
TP5	1.5	OD	2.69
	3	OD	2.7
TP6	1.9	OD	2.67
TP7	1.7	OD	2.74
TP8	1.6	OD	2.68
TP9	1.5	OD	2.7
	3	OD	2.71
TP10	1.5	OD	2.73
	3	OD	2.7
TP11	1.5	OD	2.67
	3	OD	2.6

4.2.1.6 Grain-size distribution

The distribution of the grain size present in a given soil sample helps to understanding the nature of soil. Two methods generally are used for the determination of grain size distribution: (1) sieve analysis, (2) sedimentation analysis. The results of combined grain size analysis are usually presented in the form of grain size distribution curve [6,2,21].

For all test pits the grain size analysis is done according to ASTM D 422-63 and D1140-97 [3].

From the combined analysis the results are summarized in Table 4.8.The gradation curves are presented in Fig.4.1 and Fig.4.2.

Table 4.8: Summary of the combined grain size analysis result

Test pit	Depth (m)	Percent amount of particle size			
		Gravel	Sand	Silt	Clay
TP1	1.5	0.11	8.138	54.93	36.78
	3	0.1	1.4	59.5	39
TP2	1.5	0.31	3.16	59.99	36.54
	3	0.54	10.77	55.17	33.52
TP3	1.5	1.43	22.78	49.88	25.91
	3	2.4	11.16	55.33	30.91
TP4	1.5	-	2.14	61.75	36.11
	3	11.87	27.02	39.33	21.78
TP5	1.5	0.07	5.44	61.98	32.51
	3	-	2.06	52.77	45.17
TP6	1.9	-	3.11	56.76	40.13
TP7	1.7	0.26	5.82	58.56	35.36
TP8	1.6	0.95	3.94	48.71	46.4
TP9	1.5	-	43.22	40.18	16.6
	3	0.44	54.42	34.07	11.07
TP10	1.5	1.32	3.15	42.14	53.39
	3	0.32	1.4	41.12	57.16

TP11	1.5	-	3.06	65.96	30.98
	3	0.06	6.71	43.11	50.12

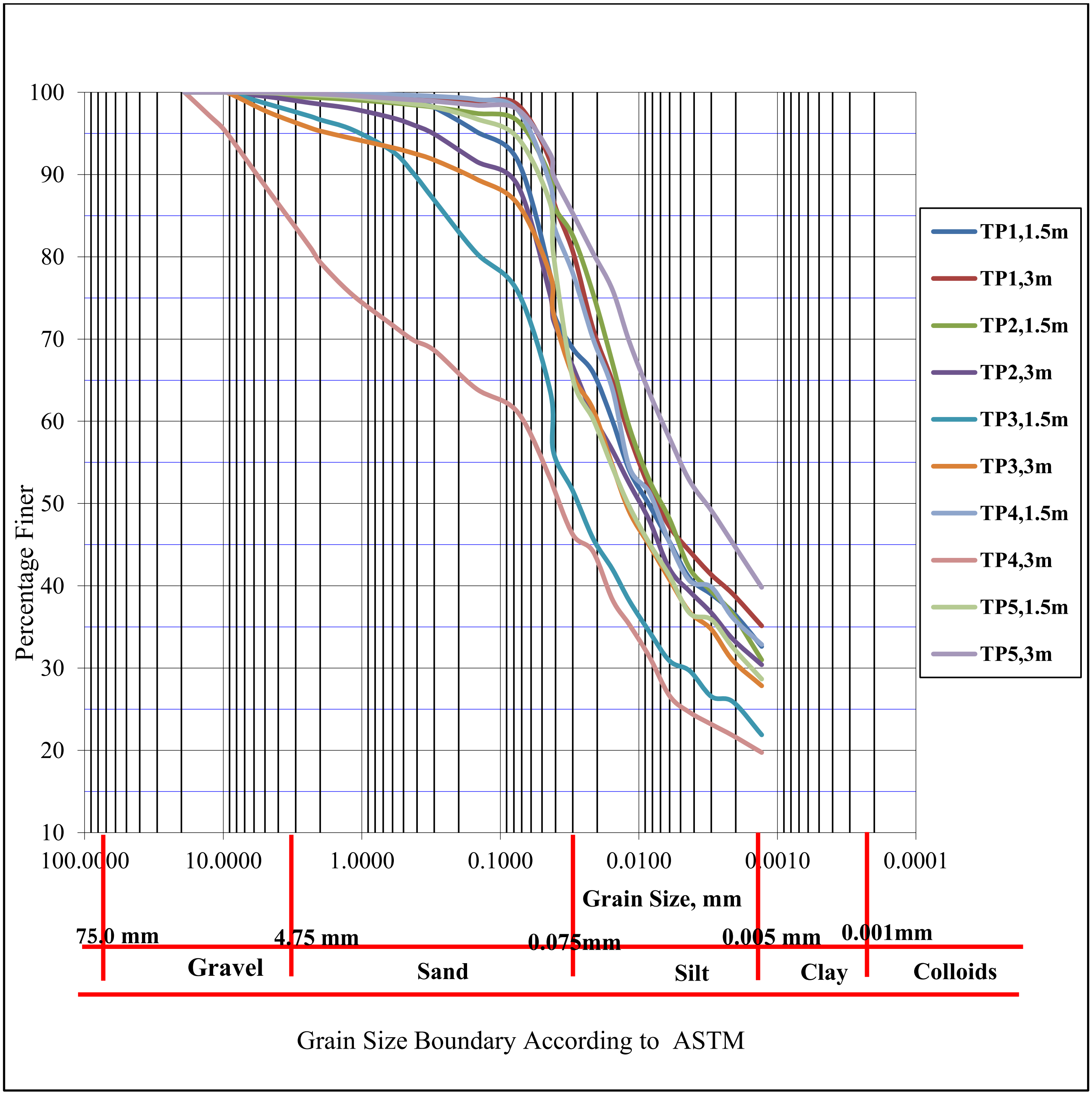

Fig.4.1 Grain size distribution curve for TP1 to TP5

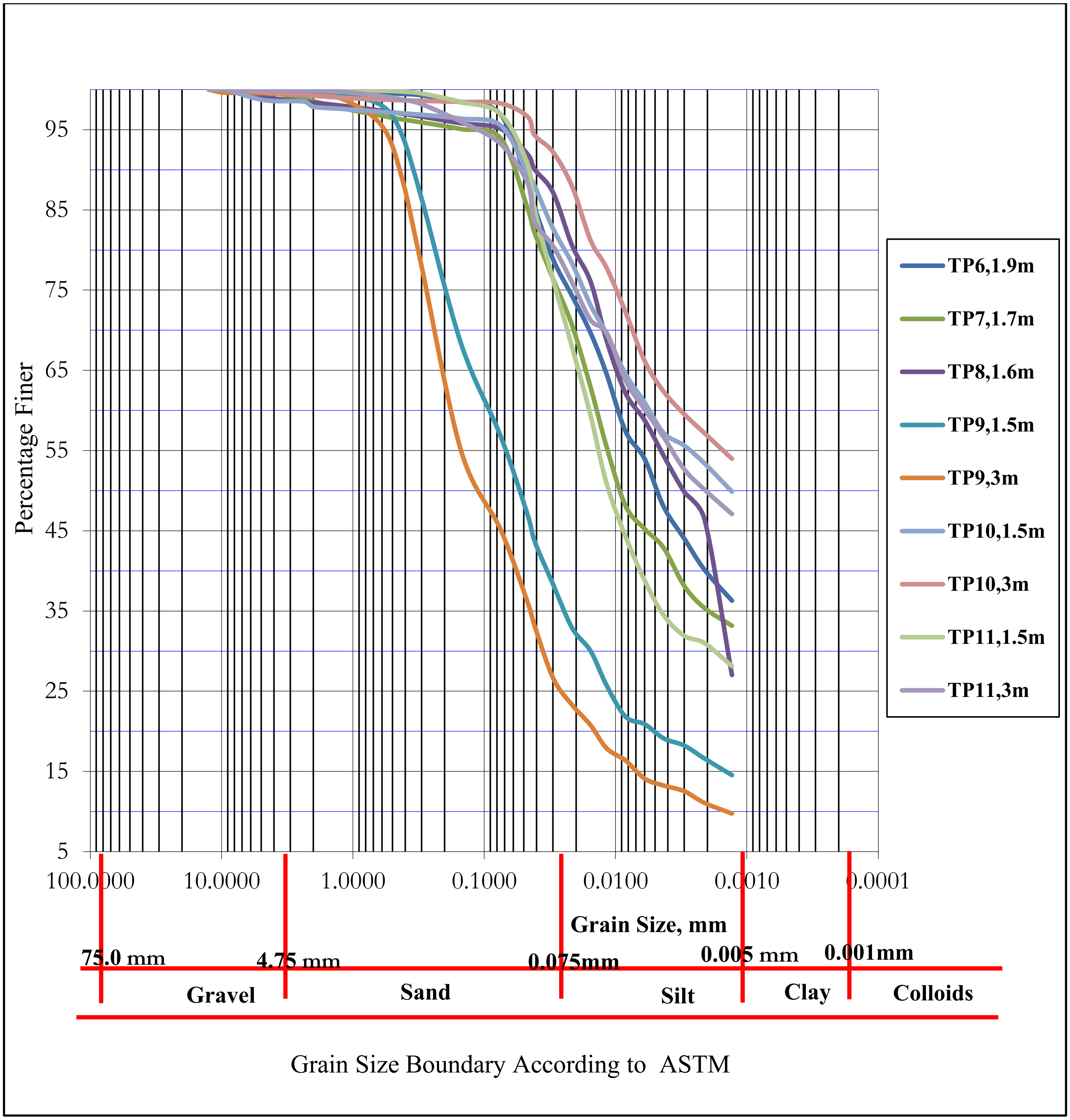

Fig.4.2 Grain size distribution curve for TP6 to TP11.

4.2.2 Soil classification

Different soils with similar properties may be classified in to groups and sub-groups according to their engineering behavior. Classification systems provide a common language to concisely express the general characteristics of soils, which are infinitely varied, without detailed descriptions. Currently two elaborate classification systems are commonly used by soils engineers. Both systems take into consideration the particle-size distribution and atterberg limits. They are the American Association of State Highway and Transportation officials (AASHTO) classification system and the Unified Soil Classification System(USCS) [5].

4.2.2.1 Unified system of soil classification

This classification uses the symbols (GW, GP, GM, GC, SW, SP, SM, SC, ML, CL, OL, MH, CH, OH and Pt) based on the indicated percentages passing the No.200 and No.4 sieves, and the atterberg limit values [15]. The Plasticity Chart is presented in Fig.4.3.

Table 4.9: Classifications of soils based on USCS Classification system [3]

Test pit	Depth (m)	Percent amount of particle size				LL (%)	PI (%)	Classification According to USCS
		Gravel	Sand	Silt	Clay			
TP1	1.5	0.11	8.18	54.93	36.78	53.8	27.5	CH
	3	0.1	1.4	59.5	39	64.5	32	MH
TP2	1.5	0.31	3.16	59.99	36.54	53.3	22.8	MH
	3	0.54	10.77	55.17	33.52	51.1	18.0	MH
TP3	1.5	1.43	22.78	49.88	25.91	36.8	12.1	ML
	3	2.4	11.16	55.33	30.91	45.3	20.1	CL
TP4	1.5	-	2.14	61.75	36.11	61.2	28.9	MH
	3	11.87	27.02	39.33	21.78	45.1	17.6	ML
TP5	1.5	0.07	5.44	61.98	32.51	58.9	27.7	MH
	3	-	2.06	52.77	45.17	67.5	34.2	MH
TP6	1.9	-	3.11	56.76	40.13	65.8	34.7	MH
TP7	1.7	0.26	5.82	58.56	35.36	62.0	28.9	MH

TP8	1.6	0.95	3.94	48.71	46.4	65.2	33.5	MH
TP9	1.5	-	43.22	40.18	16.6	37.4	10.0	SM
	3	0.44	54.42	34.07	11.07	30.7	6.5	SM
TP10	1.5	1.32	3.15	42.14	53.39	81.4	48.4	CH
	3	0.32	1.4	41.12	57.16	84.9	55.4	CH
TP11	1.5	-	3.06	65.96	30.98	52.7	21.8	MH
	3	0.06	6.71	43.11	50.12	40.0	16.7	CL

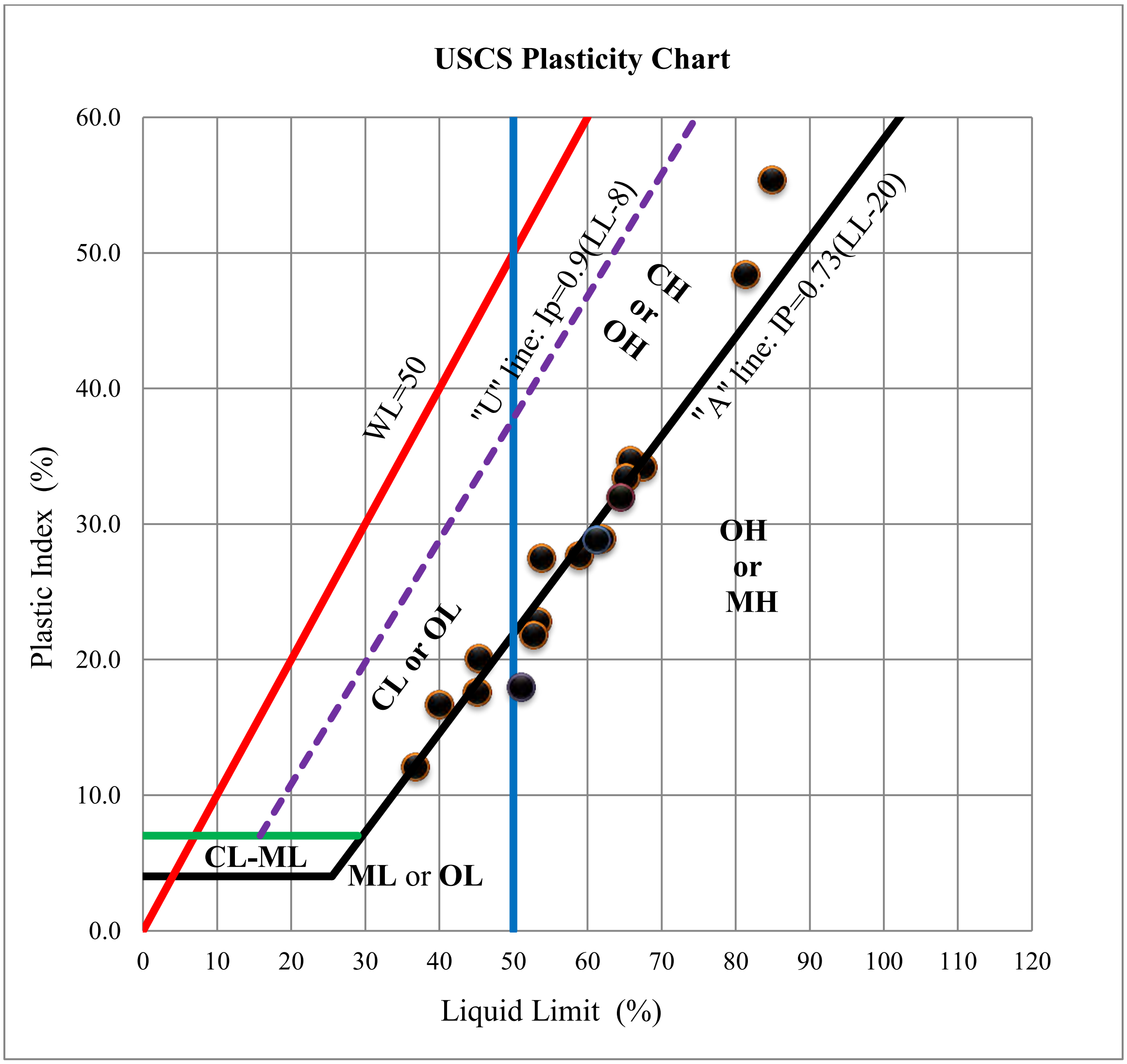

Fig 4.3 Plasticity chart for soil classification based on USCS

4.2.2.2 AASHTO Classification System

The AASHTO system of soil classification was developed in 1929 as the Public Road Administration Classification System. According to this system, soil is classified into seven major groups: A-1 through A-7. Soils classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve. Soils of which more than 35% pass through the No. 200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly clay and silt-type materials [6]. The soil properties that are used for this classification system are; particle size distribution, Liquid limit and Plastic index. To Check the quality of the soil, a group index is defined. The index can help to make general interpretations relating to performance of the soil for engineering uses, such as highway and airfield construction. As far as this thesis concerned, more group index values indicate that only few samples are below 20. The smaller the value of the group index, the better is the soil in that category. A group index of zero indicates a good subgrade, whereas a group index of 20 or greater shows a very poor subgrade. The group index is appended to the soil type determined from AASHTO table. The smaller the value of a group index, the better the soil is in that category. From the Table 4.10 and Fig.4.4 the soils under the study area are categorized in A-4, A-6 and A-7-5/6. According to this classification system, the usual type of material is clayey soils except TP-9 which is silty soils.

Table 4.10: Classifications of soils based on AASHTO Classification system [1]

Test pit	Depth (m)	Percent passing 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						
TP1	1.5	99.83	98.94	91.71	53.8	27.5	20(max)	A-7-6	Clayey Soils	Poor
	3	99.78	99.41	98.5	64.5	32.0	20(max)	A-7-5	Clayey Soils	Poor
TP2	1.5	99.33	98.47	96.53	53.3	30.4	20(max)	A-7-5	Clayey Soils	Poor
	3	98.56	96.04	88.69	51.0	18.0	20(max)	A-7-5	Clayey Soils	Poor
TP3	1.5	96.67	90.16	75.79	36.8	12.1	16(max)	A-6	Clayey Soils	Poor
	3	95.3	92.6	86.44	45.3	20.1	20(max)	A-7-6	Clayey Soils	Poor
TP4	1.5	99.96	99.64	97.86	61.2	28.9	20(max)	A-7-5	Clayey Soils	Poor

	3	79.35	69.87	61.11	45.1	17.6	20(max)	A-7-6	Clayey Soils	Poor
TP5	1.5	99.64	98.58	94.49	58.9	27.7	20(max)	A-7-5	Clayey Soils	Poor
	3	99.8	99.08	97.94	67.5	34.2	20(max)	A-7-5	Clayey Soils	Poor
TP6	1.9	99.95	99.47	96.89	65.8	34.7	20(max)	A-7-6	Clayey Soils	Poor
TP7	1.7	98.57	96.29	93.92	59.5	30.3	20(max)	A-7-6	Clayey Soils	Poor
TP8	1.6	98.37	96.99	95.11	65.2	33.5	20(max)	A-7-5	Clayey Soils	Poor
TP9	1.5	99.68	94.43	56.78	37.4	10.0	8(max)	A-4	Silty Soils	Fair
	3	99.25	89.07	45.14	30.7	6.5	8(max)	A-4	Silty Soils	Fair
TP10	1.5	97.86	97.05	95.53	81.4	48.4	20(max)	A-7-5	Clayey Soils	Poor
	3	99.24	98.67	98.28	84.9	55.4	20(max)	A-7-5	Clayey Soils	Poor
TP11	1.5	99.96	99.76	96.94	52.7	21.8	20(max)	A-7-5	Clayey Soils	Poor
	3	99.87	98.78	93.23	40	16.7	16(max)	A-6	Clayey Soils	Poor

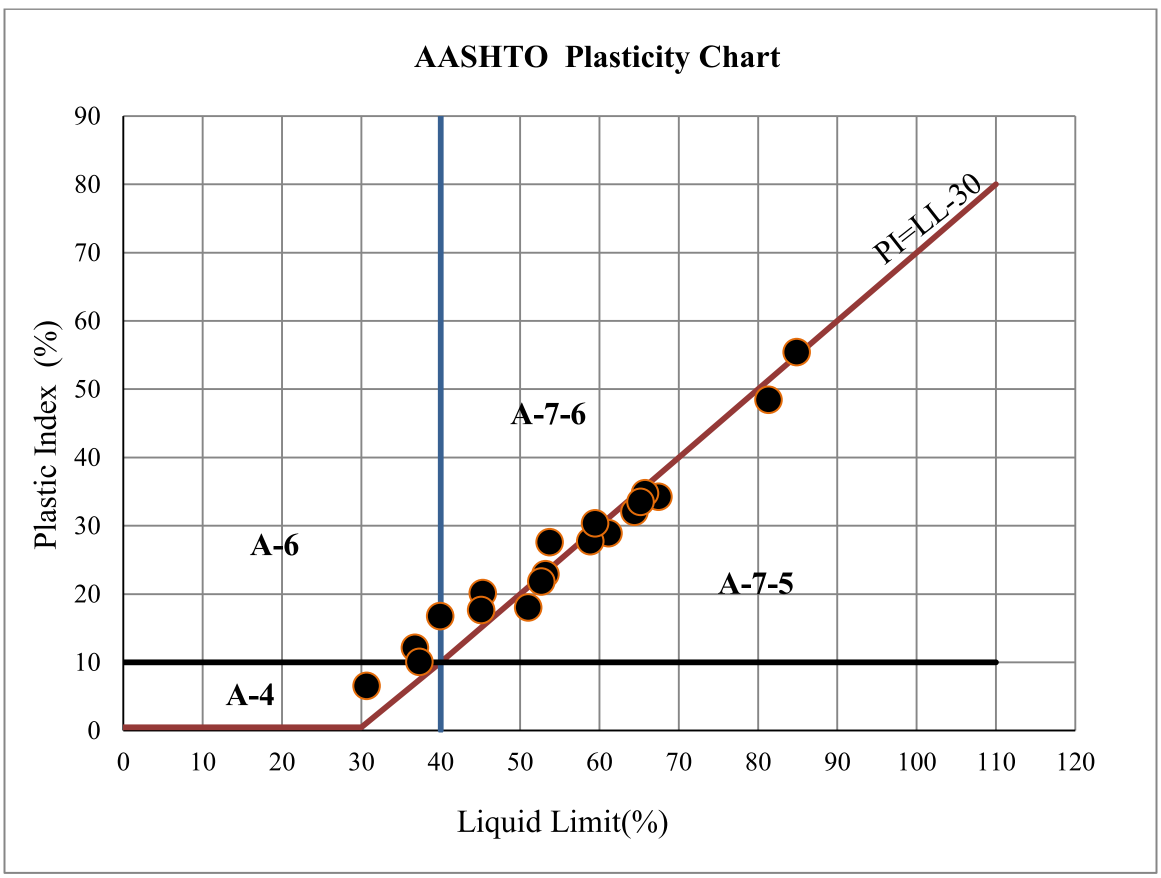

Fig.4.4 Plasticity chart for soil classification based on AASHTO

4.2.3 Classification According to Activity

The potential expansiveness of most of the test pits investigated in this thesis are from low to medium. The results of activity numbers are presented in Table 4.5. The activity chart is presented in Fig.4.5.

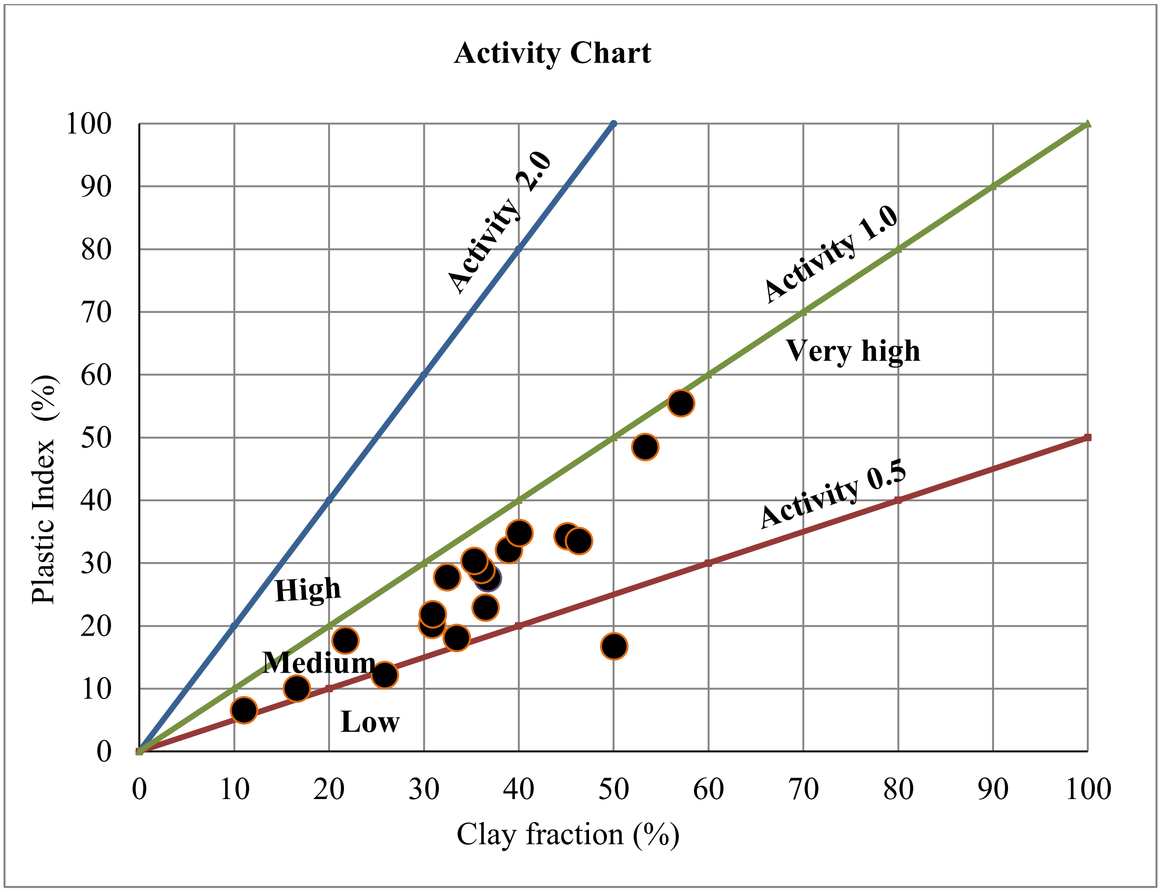

Fig. 4.5 Activity chart for soil classification

4.2.4 Compaction

Compaction, in general, is the densification of soil by removal of air, which requires mechanical energy. The degree of compaction of a soil is measured in terms of its dry unit weight. When water is added to the soil during compaction, it acts as a softening agent on the soil particles. The soil particles slip over each other and move into a densely packed position. The dry unit weight after compaction first increases as the moisture content increases. Compaction increases the strength characteristics of soils, which increase the bearing capacity of foundations and physical properties of soil appropriate for structures constructed over them [6].

For compaction process water plays an important role. As we can see, at the start of the test when the soil is relatively dry, the soil assumes a flocculated structure. Additional mechanical reworking and increasing amounts of water and subsequent expulsion of air and closing of the voids tend to produce a semi flocculated structure with increasing density until a peak is attained. When we keep on increasing the water content in the soil sample to be compacted, compaction process will be very difficult. Here presence of water prevents further compaction and adding water at this stage will make the soil platelets making to be oriented and aligned and the inter particle distances tend to widen the process more difficult. The Standard Proctor Test [3] was used for the compaction. Soil passing No.4 (4.75mm) sieve air-dried soil is prepared and compacted in three layers with each layer compacted by 25 blows.

The dry density achieved depends upon the type of soil. Coarse-grained soils can be compacted to higher dry density than fine-grained soils. If the quantity of fines is increased to a value more than that require to fill the voids of the coarse-grained soils, the maximum dry density decreases. In general cohesionless soils have a relatively higher maximum dry density and require less water than cohesive soils. Therefore,the optimum water content is less for cohesionless soils. Heavy clays of very high plasticity have very low dry density and a very high optimum water content. Compaction test results are summarized in Table 4.11. The detailed test results are presented in Appendix B.

Table 4.11: Compaction test result (MDD and OMC)

Test pit	Depth(m)	OMC (%)	MDD (g/cm3)
TP1	1.5	27.5	1.41
	3	31	1.37
TP2	1.5	27	1.38
	3	26	1.47
TP4	1.5	20	1.53
	3	28.1	1.48
TP9	1.5	22	1.61
	3	22.8	1.59

4.2.5 Unconfined compressive strength

The unconfined compression test is a special type of unconsolidated-undrained triaxial test in which the confining pressure is equal to zero. The tests are carried out only on saturated samples which can stand without any lateral support. The test is, therefore, applicable to cohesive soils only. The test is an undrained test and is based on the assumption that there is no moisture loss during the test. The unconfined compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils. The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM standard [3], the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail through a weak zone in a simple compression test. The undrained shear strength is necessary for the determination of the bearing capacity of foundations structures, and it is basically equal to cohesion(c) and expressed as:

$$Su = c = \frac{q_u}{2} \qquad (4.4)$$

$$q_u = \frac{P}{A} \qquad (4.5)$$

Where: q_u=unconfined compressive strength (kPa)

P=Compressive force (kN)

A=Cross sectional area (m^2)

The cross sectional area A at any stage of loading of the sample may be computed on the basic assumption that the total volume of the sample remains the same [18]. The corrected average cross sectional area is calculated by:

$$A = \frac{A_o}{1-\varepsilon} \qquad (4.6)$$

Where: A=Cross sectional area (m^2)

A_o=Initial cross sectional area (m^2)

ε=Axial strain (mm/mm), $\varepsilon = \Delta L / L_o$

The undisturbed soil samples are used for these tests and the results are shown in Table 4.12 for test pits TP1, TP2, TP4, TP9 and TP10. The detailed test results are presented in Appendix C.

Table 4.12: The consistency of typical soil samples

Test pit	Depth (m)	Liquid Limit	Plastic Limit	Plastic Index	Unconfined compressive strength	Water Content (%)	Liquidity Index	Consistency
TP1	1.5	53.8	26.2	27.5	57	27.03	0.03	Stiff
	3	64.5	32.5	32.0	127	32.74	0.01	stiff
TP2	1.5	53.3	30.4	22.8	130	21.74	-0.38	Hard
	3	51.1	33.1	18.0	166	22.45	-0.59	Hard
TP4	1.5	61.2	32.4	28.9	247	23.75	-0.3	Hard
	3	45.1	27.5	17.6	195	31.99	0.26	stiff
TP9	1.5	37.4	27.4	10.0	36	12.9	-1.45	Hard
	3	30.7	24.2	6.5	30	17.23	-1.07	Hard
TP10	1.5	81.4	33	48.4	186	27.6	-0.11	Hard
	3	84.9	29.5	55.4	85	36.67	0.13	Stiff

The consistency of undisturbed soil varies quantitatively on the basis of its liquidity index. The soil found in stiff to hard state of consistency.

4.2.6 Consolidation

The process whereby soil particles are packed more closely together over a period of time under the application of continued pressure. It is accompanied by drainage of water from the pore spaces between solid particles [11]. The compressibility characteristics of a soil mass might be due to any or a combination of the following factors [18]:

1. Compression of the solid matter.
2. Compression of water and air within the voids.
3. Escape of water and air from the voids.

4.2.6.1 Test Procedure and results

The standard test method used for One-Dimensional Consolidation is based on ASTM D 2435 [3]. In this procedure the specimen is subjected to a series of pre-selected vertical stresses (e.g. 7, 7, 50, 100, 200, 400, 800, 1600kPa) each of which is held constant while dial gauge measurements of vertical deformation of the top of the specimen are made, and until movements cease (normally 24 h). Dial gauge readings are taken at standard intervals of time after the start of the test (i.e. 0, 6, 15 and 30s, 1, 2, 4, 8, 15, 30 and 60min, 2, 4, 8 and 24hr).

4.2.6.2 Pre-consolidation pressure.

The pre-consolidation stress, pc, is the maximum effective stress to which the soil has been exposed in its past geological history. The Casagrande [7] method of obtaining the pre-consolidation pressure from consolidation tests is shown in Fig 4.6. Determining the point of greatest curvature requires care and judgment [9].

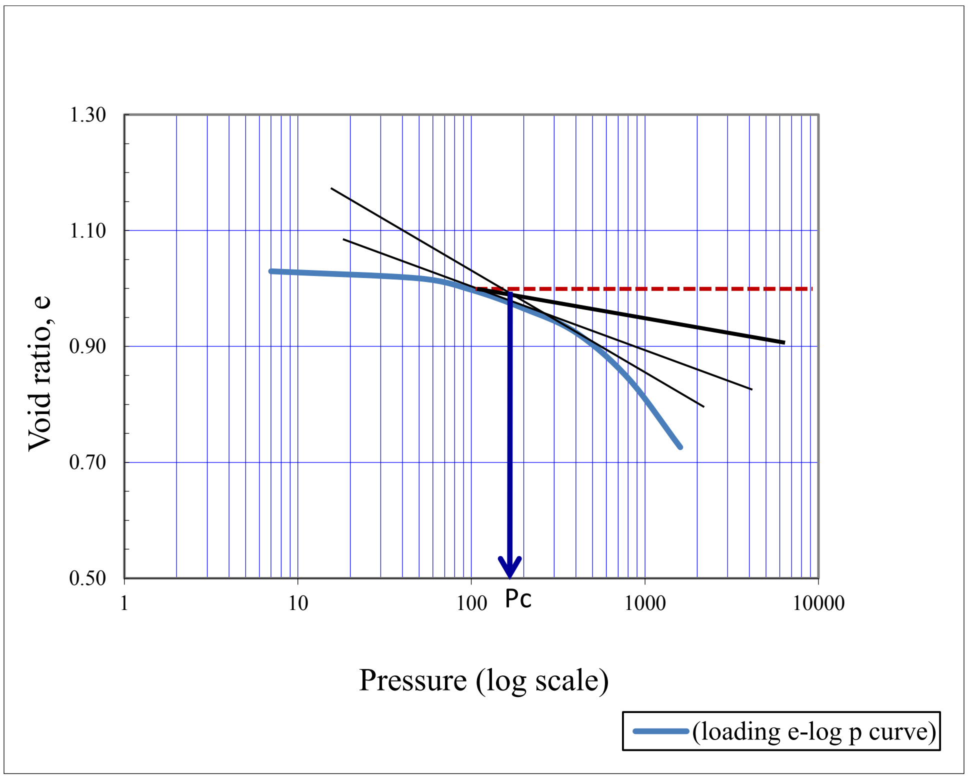

Fig.4.6 Typical curve (TP1,3m) void ratio Vs log pressure to determine P_c

The consolidation tests for samples TP1, TP2, TP4 and TP10 at 3m depth are plotted in Fig. 4.7 and Fig. 4.8.

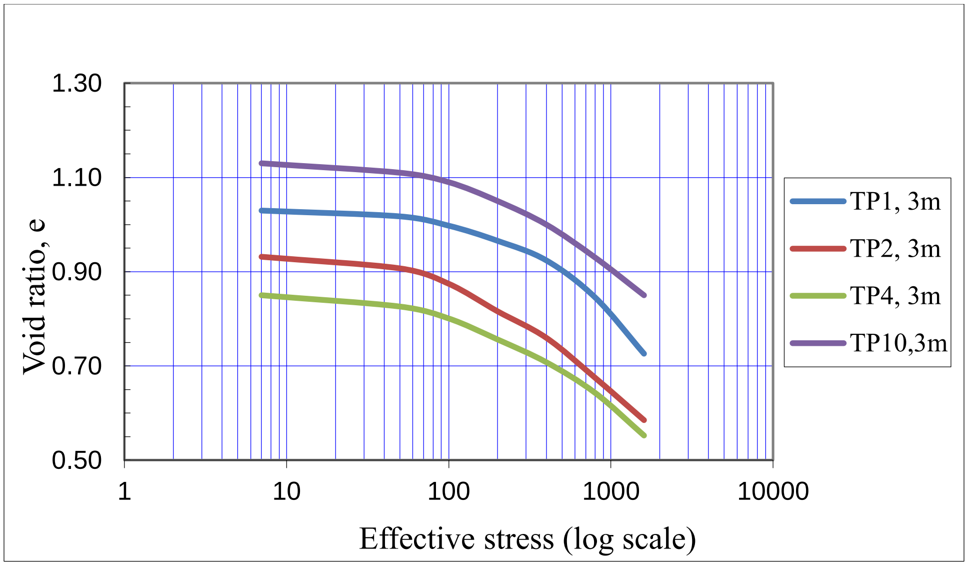

Fig.4.7 Consolidation test results, void ratio vs Effective stress (log scale)

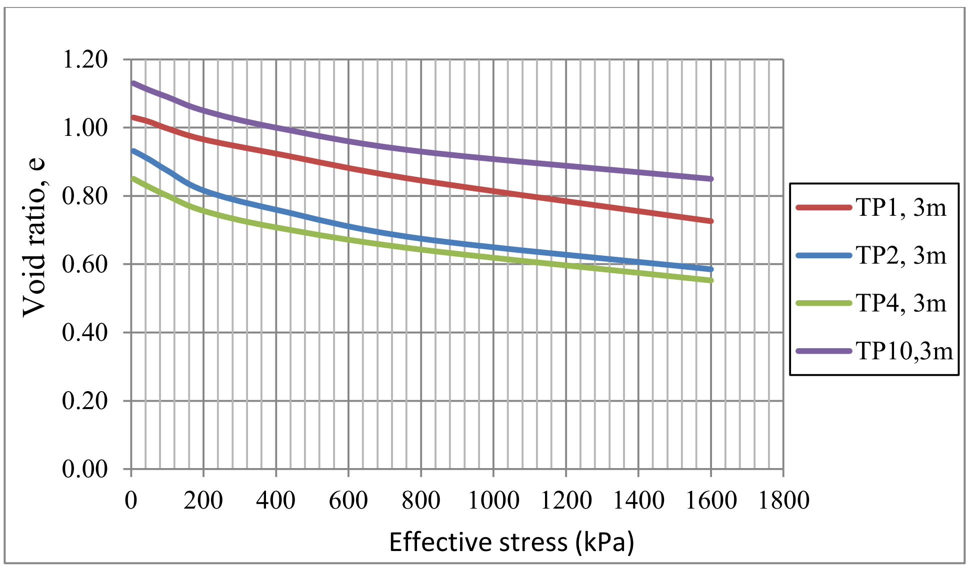

Fig.4.8 Consolidation test results, void ratio Vs pressure (linear scale)

4.2.6.3 Compression Index (C_c)

The compression index for the calculation of field settlement caused by consolidation can be determined by graphic construction after one obtains the laboratory test results for void ratio and pressure. The value of C_c (Table 4.13) is the slope of the linear portion of the e Vs log P curve. Thus:

$$C_c = \frac{e_1 - e_2}{log^{p_2} - log^{p_1}} \quad (4.7)$$

4.2.6.4 Coefficient of consolidation (c_v)

The curve between dial gauge reading and time t obtained in the laboratory by testing the soil sample is similar in shape to the theoretical curve between U and T_v. This similarity between the laboratory curve and the theoretical curve is used for the determination of the coefficient of consolidation (c_v) of the soil. The methods are known as the fitting methods [2].

4.2.6.4.1 Square-root-of-time fitting method

For this thesis the square-root-of-time fitting method is used to compute C_v values. The typical graphical plot is shown in Fig. 4.9 and the results are listed in Table 4.13. The value of c_v is then determined by [10]:

$$C_v = \frac{0.848 * H^2}{t_{90}} \tag{4.8}$$

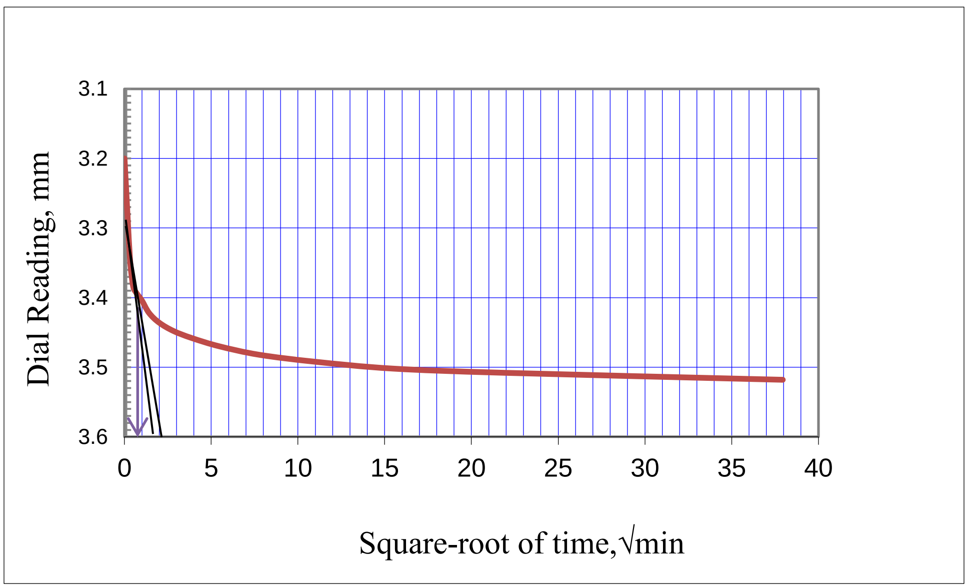

Fig.4.9 Typical Square-root-of-time Vs dial reading curve for TP1, 3m (200Kpa loading)

Table 4.13: Consolidation test results

Test pit	Depth (m)	Natural moisture content (%)	Wet Density, gm/cm^3	Pressure (kPa)	Void ratio (e_f)	coefficient of consolidation,(c_v) (cm^2/sec)	Compression Index, (C_c)	Pre-consolidation pressure, P_c (kPa)	Over burden pressure, kPa	OCR, (%)
TP1	3	32.74	1.75	50	1.02	0.85	0.4	170	51.50	3.3
				100	1	1.71				

				200	0.97	1.01				
				400	0.92	0.55				
				800	0.85	0.33				
				1600	0.73	0.23				
TP2	3	22.45	1.7	50	0.91	0.44	0.3	105	50.03	2.1
				100	0.87	0.83				
				200	0.82	0.53				
				400	0.76	0.33				
				800	0.67	0.26				
				1600	0.59	0.36				
TP4	3	31.99	1.88	50	0.83	0.33	0.3	110	55.33	2.0
				100	0.8	1.21				
				200	0.76	0.31				
				400	0.71	0.33				
				800	0.64	0.17				
				1600	0.55	0.38				
TP10	3	36.67	1.79	50	1.11	0.88	0.26	120	52.7	2.3
				100	1.09	1.35				
				200	1.05	0.7				
				400	1.00	0.31				
				800	0.93	0.23				
				1600	0.85	0.22				

4.2.6.5 Determination of permeability Coefficient (k)

The flow of water through soils depends upon its permeability coefficient. The greater the value of the coefficient of permeability, the greater is the flow. Which means coarse sand and gravel are highly pervious and have correspondingly high permeability coefficients. Clays on the other hand are relatively impervious and hence have low permeability coefficients [20].

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 [21].

$$k = \frac{c_v a_v \gamma_w}{1+e} \tag{4.9}$$

Where: c_v= coefficient of consolidation

a_v= coefficient of compressibility

γ_w=unit weight of water

e= void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results. The results obtained and the relationship between the coefficient of permeability and the consolidation pressure is shown in Table 4.14 and Fig.4.10 respectively. It is noted that a_v, the ratio of change in void ratio to change in pressure, was obtained from Fig.4.8. The values of coefficient of permeability for the tested soils lie between 10^{-8}and10^{-9} cm/sec, which indicates that the soils are practically impervious. In void ratio versus log coefficient of permeability graph all soil samples taken from study area have nearly straight line relationship (Fig.4.10) [16].

Fig.4.11 shows that, consolidation pressure has a direct impact on permeability of soil and the value of 'k' decreases with increasing consolidation pressure.

Table 4.14: Relationship between void ratio and coefficient of permeability

Test pit	**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^{-9} cm/sec**
TP1	3	100	1	1.71	39.55	33.82
		200	0.97	1.01	32.08	16.45
		400	0.92	0.55	20.78	5.95
		800	0.85	0.33	19.67	3.51
		1600	0.73	0.23	14.91	1.98

TP2	3	100	0.87	0.83	65.3	29
		200	0.82	0.53	58.4	17.01
		400	0.76	0.33	28.2	5.3
		800	0.67	0.26	21.2	3.3
		1600	0.59	0.36	11.2	2.54
TP4	3	100	0.8	1.21	50.62	34.03
		200	0.76	0.31	44.52	7.84
		400	0.71	0.33	24.2	4.67
		800	0.64	0.17	16.26	1.68
		1600	0.55	0.38	11.29	2.77
TP10	3	100	1.09	1.35	37.8	24.42
		200	1.05	0.7	35.94	12.3
		400	1	0.31	25.96	4.02
		800	0.93	0.23	17.19	2.05
		1600	0.85	0.22	9.71	1.16

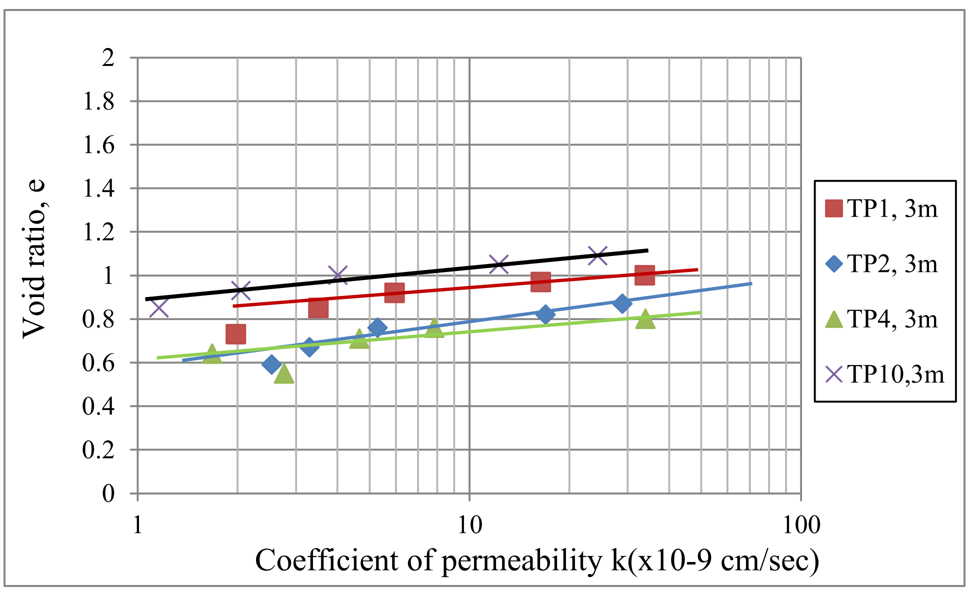

Fig.4.10 Void ratio Vs Log Coefficient of Permeability

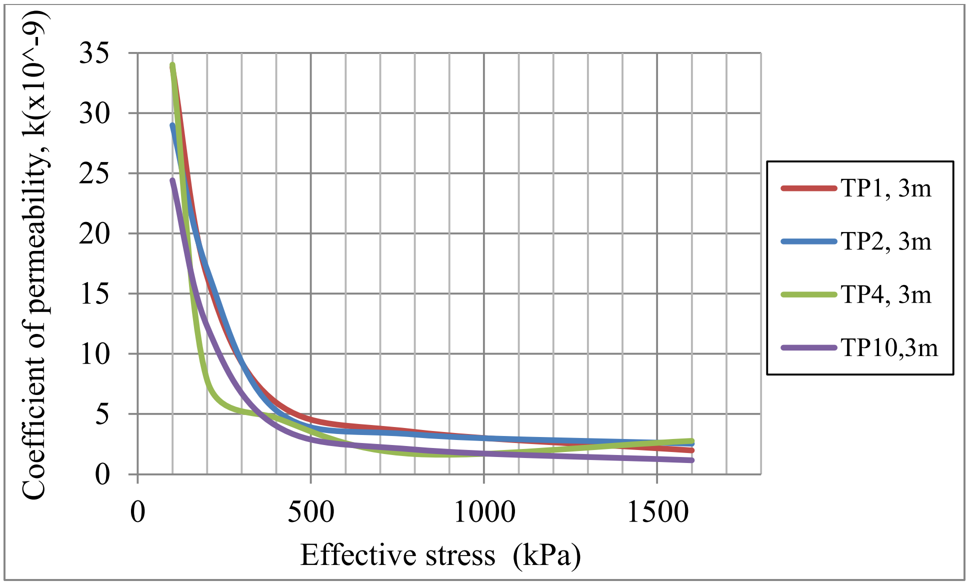

Fig.4.11. Relationship between consolidation pressure and coefficient of permeability

4.2.6.6 Modulus of Compressibility

The compressibility curve is obtained from the relative settlement (s') or void ratio versus the effective stress (σ') in Fig.4.12. The curve may be expressed with sufficient accuracy by the following equation [21]:

$$E_s = \frac{d\sigma'}{ds'} = v(\sigma')^w \tag{4.10}$$

Where: v and w are coefficient; v has a unit of kN/m^2
w =is dimensionless
σ'=is effective normal stress, kN/m^2
s'= relative settlement

In order to make the exponent w dimensionless, it is advisable to make σ' also dimensionless by dividing it by a unit stress σ_e [21].

$$\sigma'_e = \frac{\sigma'}{\sigma_e} \tag{4.11}$$

The coefficient v depends on the void ratio, water (moisture) content and consistency of the Sample. It could have values ranges from 50 to 30000kN/m^2.whereas w depends on the soil type. It could have values ranges from 0 to 1 [21].

The tangent of the compressibility curve, which is a function of effective stress, gives the modulus of compressibility E_s.

From Eq. (4.8)

$$\frac{ds'}{d\sigma'} = \frac{1}{v(\sigma')^{w}} \quad (4.12)$$

$$ds' = \frac{1}{v}(\sigma')^{-w}\sigma' \quad (4.13)$$

$$s' = \frac{1}{v}\int(\sigma')^{-w}\sigma' \quad (4.14)$$

For the case w≠1

$$s' = \frac{1}{v(1-w)}(\sigma')^{1-w} + c \quad (4.15)$$

Defining $a = \frac{1}{v(1-w)}$ and k=1-w, equation (4.13) becomes;

$$s' = a(\sigma')^{k} + c \quad (4.16)$$

For σ'=0, s'=0 then c=0; equation (4.14) becomes

$$s' = a(\sigma')^{k} \quad (4.17)$$

For the case w=1

$$\frac{ds'}{d\sigma'} = \frac{1}{v\sigma'} \quad (4.18)$$

$$s' = \frac{1}{v}\ln\sigma' + c \quad (4.19)$$

Taking common logarithm, equation (4.17) becomes;

$$\log s' = k\log\sigma' + \log a \quad (4.20)$$

If a plot s' versus $\ln\sigma'$ is made, one obtains a straight line relationship for some cohesive soils. This would mean that the compressibility of the soil is described in case(w=1) by equation (4.19). Other soils give straight line relationship when the results are plotted on a double log scale in case (w≠1) by equation (4.16) [21].

Fig.4.13 is plotted based on the result obtained from one-dimensional consolidation test, relative settlement versus effective stress (lnσ′) and shows straight line relationship. The values of the coefficients, v and w, were calculated using Fig.4.13 and tabulated on Table 4.16.

Table 4.15: Effective stress, total compression and relative settlements of samples

Test pit	Depth (m)	Effective stress P (KN/m^2)	Total compression ΔH (mm)	Relative settlement $s' = \Delta H/H$	modulus of compressibility E_s (kN/m^2)
TP1	3	50	0.004	0.0002	5000
		100	0.204	0.0102	
		200	0.724	0.0362	
		400	1.654	0.0827	
		800	3.364	0.1682	
		1600	6.254	0.3127	
TP2	3	50	0.03	0.0015	3529
		100	0.4	0.02	
		200	1.38	0.069	
		400	2.95	0.1475	
		800	5.41	0.2705	
		1600	8.81	0.4405	
TP4	3	50	0.23	0.0115	3600
		100	0.74	0.037	
		200	1.73	0.0865	
		400	3.24	0.162	
		800	5.46	0.273	
		1600	8.65	0.4325	
TP10	3	200	0.25	0.0125	6500
		400	1	0.05	

		800	2.41	0.1205	
		1600	4.57	0.2285	

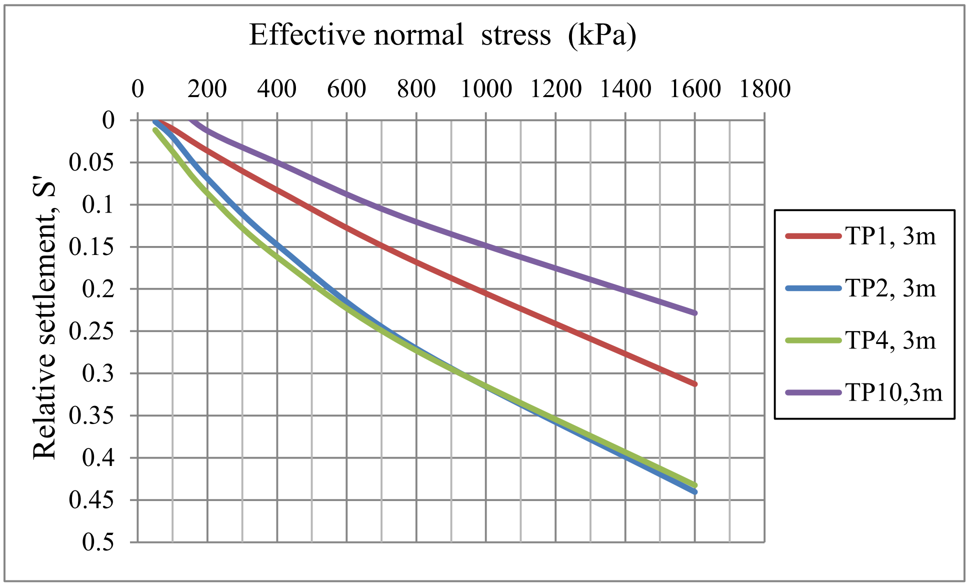

Fig.4.12. Effective normal stress Vs Relative settlements of the samples.

Table 4.16: Coefficients, v and w, and equation of the modulus of compressibility.

Test Pit	**Depth (m)**	**v (kN/m^2)**	**w**	**Es**
TP1	3	3851	1	3851*eff. Stress
TP2	3	3466	1	3466*eff. Stress
TP4	3	3648	1	3648*eff. Stress
TP10	3	5546	1	5546* eff. Stress

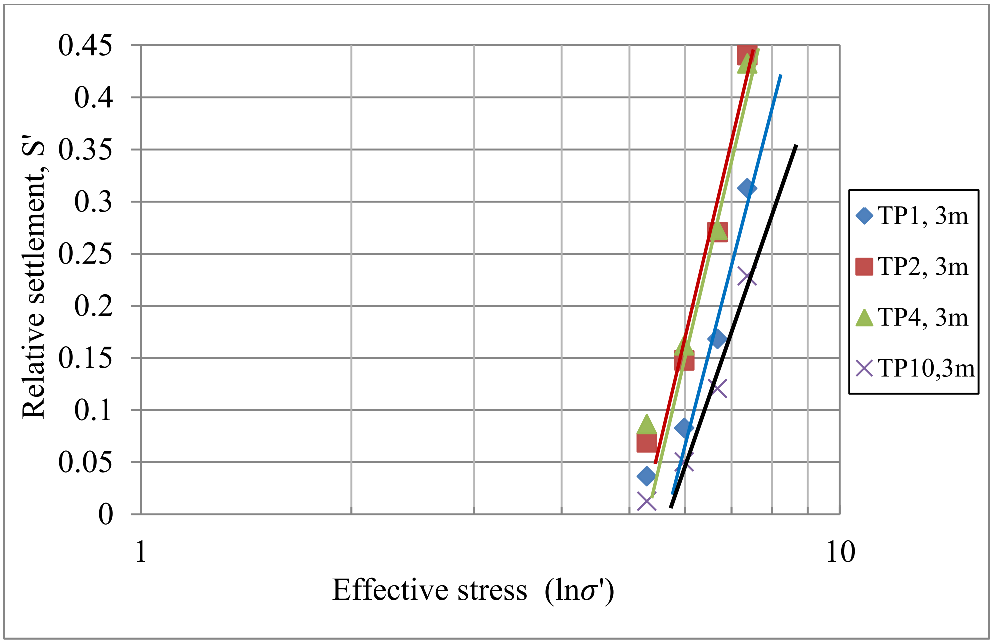

Fig.4.13. Relative settlement Vs Effective Stress (lnσ' vs s')

The results of modulus of compressibility tabulated in Table 4.17 are computed by substituting the value of v and w in to equation (4.10) as shown in Table 4.16.

Table 4.17: Modulus of compressibility for different effective stress

Test Pit	**TP1**	**TP2**	**TP4**	**TP10**
Depth(m)	3	3	3	3
Effective stress	Es(kN/m^2)	Es (kN/m^2)	Es(kN/m^2)	Es(kN/m^2)
0	0	0	0	0
100	385100	346600	364800	554600
200	770200	693200	729600	1109200
300	1155300	1039800	1094400	1663800
400	1540400	1386400	1459200	2218400
500	1925500	1733000	1824000	2773000
600	2310600	2079600	2188800	3327600
700	2695700	2426200	2553600	3882200

800	3080800	2772800	2918400	4436800
900	3465900	3119400	3283200	4991400
1000	3851000	3466000	3648000	5546000
1100	4236100	3812600	4012800	6100600
1200	4621200	4159200	4377600	6655200
1300	5006300	4505800	4742400	7209800
1400	5391400	4852400	5107200	7764400
1500	5776500	5199000	5472000	8319000
1600	6161600	5545600	5836800	8873600

5. DISCUSSIONS OF THE LABORATORY TEST RESULTS

5.1 Discussions

Information obtained from the results tested in laboratory is used to provide an indicator of general engineering behavior of soil in study area.

Based on Unified Soil Classification System, the soils investigated in the research area are classified under CH, CL, MH, ML and SM. According to soil classification on plasticity index values stated in [18], the soil samples investigated under this thesis found within low to high plasticity.

Activity number of the soils ranges from 0.33 to 0.97. This indicates that the soils in the research area are moderate to inactive. The free swell tests tell us how much the increase in volume of soil subject to submergence in water. The soil samples found in study area are classified as marginal (50-100%) and non-expansive value less than 50%.

For clayey and silty soils, the specific gravity vary from 2.6 to 2.9 [6]. The values of specific gravity of the tested soils ranges from 2.6-2.74, which is within this range.

Particle size distribution by mechanical sieve and hydrometer analysis were evaluated using a representative air dried soil sample. The grain size analysis result show that the dominant proportion of soil particles in study area is silt.

According to AASHTO soil classification system, soil investigated in Kemise town is classified under A-4, A-6, A-7-5 and A-7-6. This classification indicates that the suitability of soils as subgrade material are not good.

From the compaction test results obtained the coarser material has higher maximum dry density and lower optimum moisture content, and the finer material has lower maximum dry density and higher optimum moisture content.

On the basis of the general relationship of consistency and liquidity index stated in [18], the approximate consistencies of soil studied fall under stiff to hard. The high UCS values may be due to the cementing effect of particles by sesquioxides.

The e-log p relationship displays a distinct break at approximately the maximum past effective stress (P_c). The values of pre-consolidation pressure (P_c) are ranges from 105-170kPa. Actually investigated values of OCR are greater than one, showing that the soil has been loaded in the past with maximum effective stress.

For most test results obtained under the research shows that, coefficient of compressibility inversely proportional to effective stress, and directly proportional to void ratio and coefficient of permeability. The values of coefficient of permeability in this study indicates that the soil sample is practically impermeable (impervious soil).

The values of modulus of compressibility (E_s) differs with effective stress increment for this soil samples. This difference is attributed to the stiffness of soil. The relationship between coefficient of consolidation and effective pressure is shown in Fig.5.1.

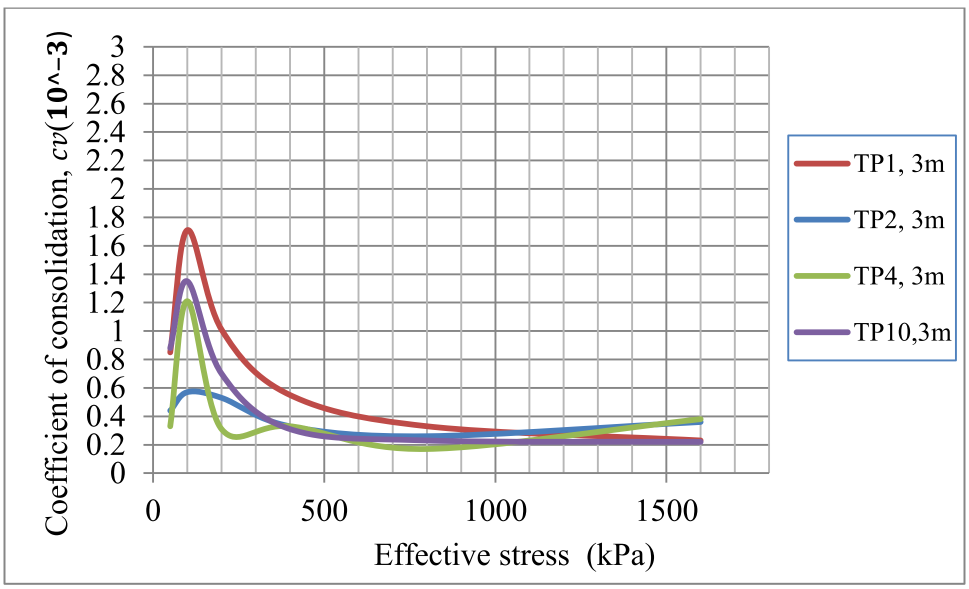

Fig.5.1 Coefficient of consolidation Vs Effective stress curve

Table 5.1: Comparison of test results with previous investigated research near to study area

	Previous Research[25]	**Previous Research[26]**	**Present Research**	**Previous Research[26]**	**Present Research**
Soil type	Silt	Silt	silt	black clay	Clay
Location	Dessie	Woldiya	Kemise	Woldiya	Kemise
Clay content (%)	50.55-70.71	6.16-10.2	21.78-46.4	33.3-49.25	30.91-57.16
Liquid Limit (%)	61-88	34.11-39.97	36.8-67.5	72.77-96.63	40-84.9
Plastic Limit (%)	38-59	28.62-30.64	22.9-33.3	30.97-34.08	23.3-33.0
Plastic Index (%)	22-38	5.49-9.33	12.1-34.7	40.23-62.55	16.7-55.4
Activity	-	0.89-0.91	0.47-0.86	1.08-1.27	0.33-0.97
Free Swell (%)	65-130	39-45	27.5-85	72-130	25-97.5
Specific Gravity	2.65-2.83	2.65-2.67	2.63-2.74	2.75-2.89	2.6-2.73
Unconfined compression strength (kN/m^2)	216-311	-	127-247	-	57-186
AASHTO clasification	A-7-5	A-4	A-6, A-7-5/6	A-7-5	A-6, A-7-5/6

The comparison of test results are tabulated in table 5.1. It indicates that for silt soils the study area have nearly equal specific gravity value with others previously done researches. Both the study area and Woldiya town clay soils have relatively equal clay fraction. But the study area silt soils have clay fraction greater than Woldiya town and less than Dessie town silt soils clay fraction. The free swell value indicates that expansive soils were found in Dessie and Woldiya towns, however, the research area soils are non-expansive and marginal. The Kemise town, Woldiya town and Dessie town soils are not good for subgrade material.

5.2 Preparation of tentative soil map

In the study area soil samples were collected and tested from eleven test pits. The test pits are excavated up to a depth of 3m. The tentative soil map is prepared by assembling the laboratory analysis and field observation. The delineation is done by interpolation between neighboring test pits. The type soil found in Kemise town are mixtures of clay, silt and silty sand.

This soil map doesn't have a detailed list of information about the study area, which misses like topography, land use, morphological characteristics of soils. The tentative soil map is presented in Fig.5.2.

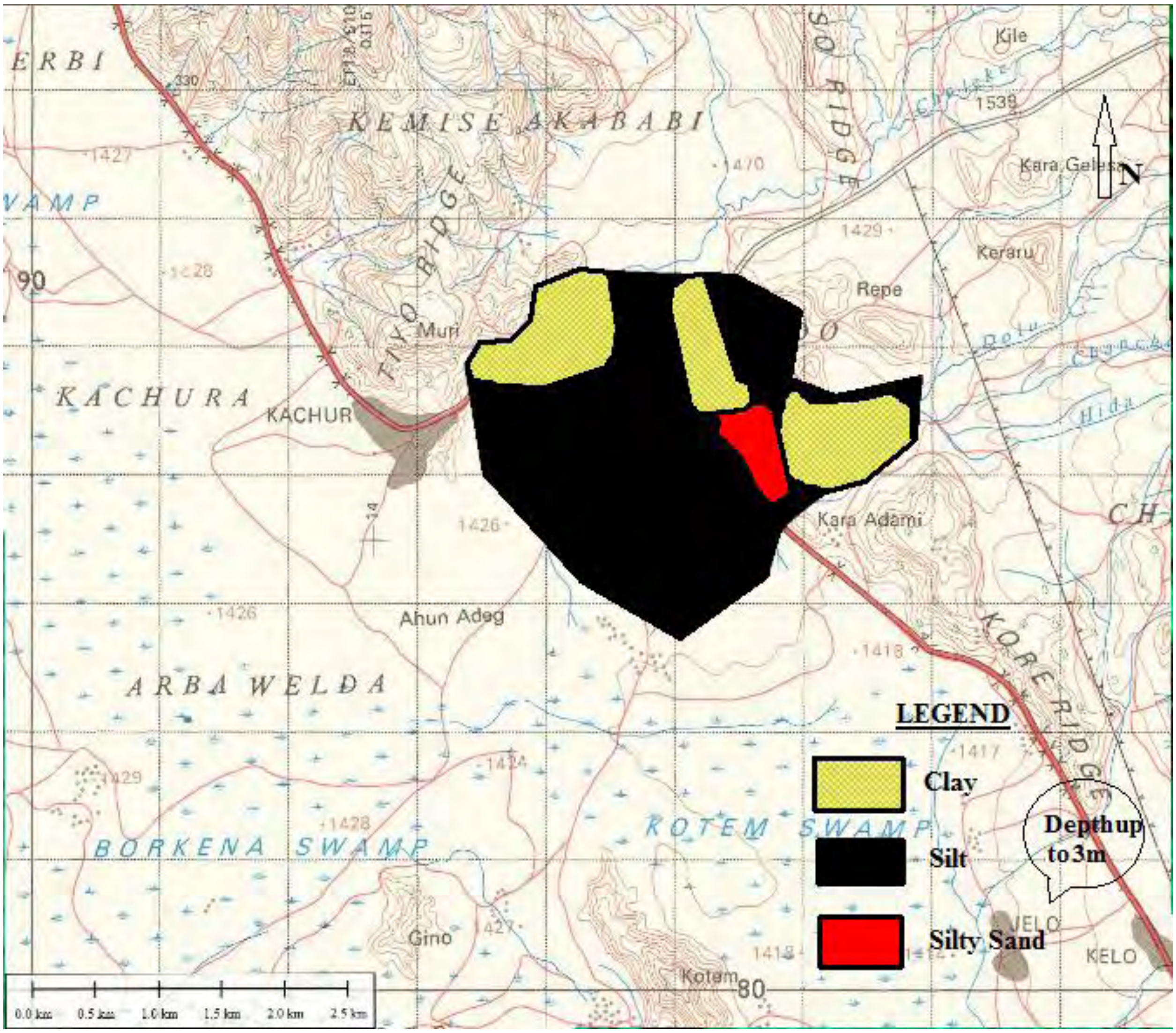

Fig.5.2 Tentative soil map for Kemise town.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. According to this study the most common types of soil found in Kemise town are clay, silt and silty sand .
2. The clay soils have liquid limit ranging from 40 to 84.9%, plastic index ranges from 16.7 to 55.4% and specific gravity ranging from 2.6 to 2.73. The silt soils have liquid limit ranging from 36.8 to 67.5%, plastic index ranging from 12.1 to 34.7% and specific gravity ranging from 2.63 to 2.74. The silty sand soils have liquid limit ranging from 30.7 to 37.4%, plastic index ranging from 6.5 to 10% and specific gravity ranging from 2.7 to 2.71.
3. The free swell for clay soils ranging from 25 to 97.5%, silt soils ranging from 27.5 to 85% and silty soils ranging from 15 to 20%, indicates that the soils are non-expansive to moderate soil. The activity of clay soils ranging from 0.33 to 0.97, silt soils ranging from 0.47 to 0.86, silty sand soils ranging from 0.59 to 0.6.
4. The results obtained from grain size analysis the clay content ranges from 11.07 to 57.16%, silt ranges from 34.07-65.96%, sand ranges from 1.4 to 54.42% and gravel ranges from 0 to 11.87%.
5. The clay and silt soils have consistency of stiff to hard, while silty sand soils are hard. The clay, silt and silty sand soils have unconfined compressive strength of 57 to 186kPa, 127 to 247kPa, 30 to 36kPa with natural moisture content of 17.15 to 36.67%, 14.01 to 38.4%, 12.9 to 17.23% respectively. The results of compaction test; maximum dry density ranges from 1.37 to 1.61gm/cm^3 and optimum moisture content ranges from 20 to 31%.
6. The clay soil has compression index (c_c) of 0.26. While the silt soils have compression index of 0.3 to 0.4. The coefficient of permeability (k) ranging from 10^{-8} to 10^{-9} cm/sec, modulus of compressibility ranging from 3529-6500kPa.

6.2 Recommendations

1. Since the town became the capital of the new zone of oromiya and simultaneously became the woreda (sub-zonal administration) town of the Dawa Chefa woreda; very recently, the boundaries of the town of Kemise are greatly extended, in terms of area and population. The tentative soil map of the study area should be modified by considering the newly incorporated surrounding areas.

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APPENDIX-A

A-1) The detailed test results for Atterberg limit

#<u>For test pit 1</u>

Sample depth…………1.5m

Table A-1-1: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A14	A28	C30	D21	A3	D5
Mass of container,gm	15.6	15.5	15.4	15.5	16.0	15.4
Mass of container +wet soil,gm	39.6	39.2	46.0	36.1	20	20.5
Mass of container +dry soil,gm	31.4	31.0	35.2	28.8	19.2	19.4
Mass of water, gm	8.2	8.2	10.8	7.3	0.8	1.1
Mass of dry soil, gm	15.8	15.5	19.8	13.3	3.2	4.0
Water content(ω), %	51.9	52.9	54.5	54.9	25.0	27.5
No of blows	36	28	24	19	26.25	

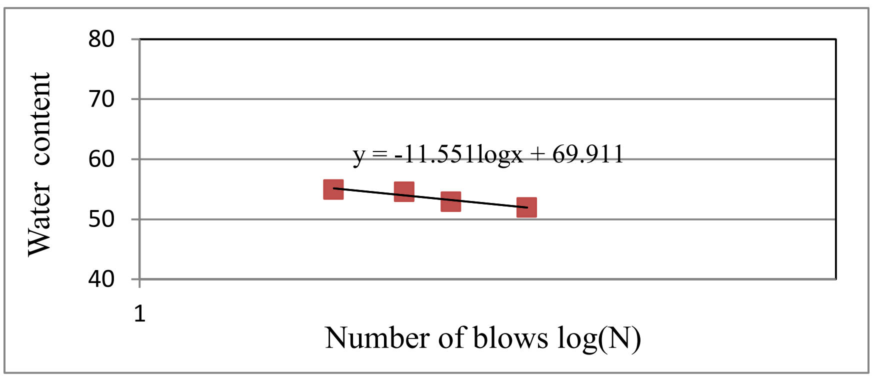

Fig. A-1-1. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-11.55*log(25)+69.911=53.8 %

#<u>For test pit 1</u>

Sample depth…………….3m

Table A-1-2: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2

Container No	A1	C2	U4	D6	T3	D2
Mass of container,gm	15.6	15.5	15.7	15.7	15.5	15.6
Mass of container +wet soil,gm	36.9	37.5	38.5	39.6	20.5	20.4
Mass of container +dry soil,gm	28.8	28.9	29.5	30	19.3	19.2
Mass of water, gm	8.1	8.6	9	9.6	1.2	1.2
Mass of dry soil, gm	13.2	13.4	13.8	14.3	3.8	3.6
Water content(ω), %	61.4	64.2	65.8	67.1	31.58	33.33
No of blows	36	27	23	18	32.46	

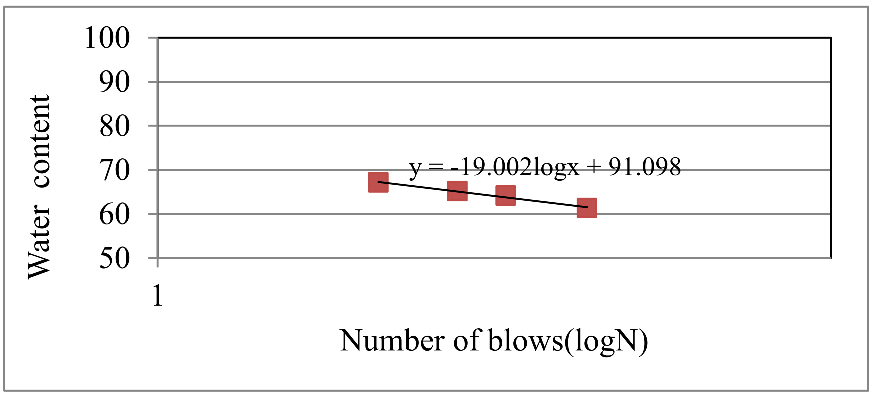

Fig.A-1-2. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-19.002*log(25)+91.098=64.5%

#<u>For test pit 2</u>

Sample depth…………1.5m

Table A-1-3: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A14	A28	C30	D21	A3	D5
Mass of container,gm	15.6	15.6	15.5	15.5	15.5	15.7
Mass of container +wet soil,gm	44.0	45.4	41.7	35.9	21.2	21.6
Mass of container +dry soil,gm	34.5	35.0	32.5	28.7	19.8	20.3
Mass of water, gm	9.5	10.4	9.2	7.2	1.4	1.3
Mass of dry soil, gm	18.9	19.4	17.0	13.2	4.3	4.6
Water content(ω), %	50.3	53.6	54.1	54.5	32.56	28.26
No of blows	35	28	24	18	30.41	

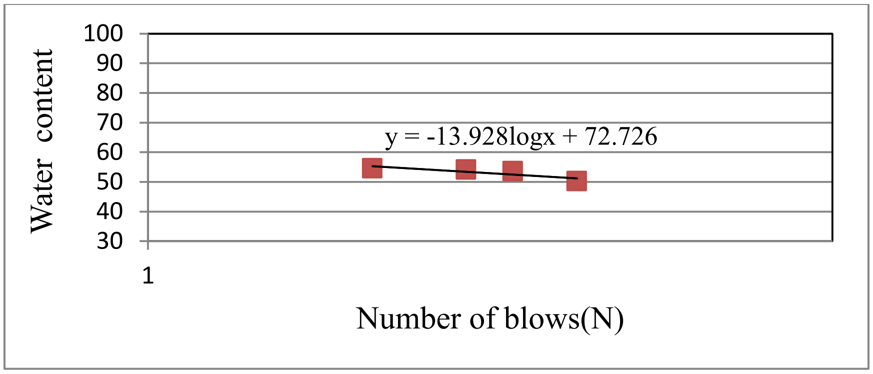

Fig.A-1-3. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-13.929*log(25)+72.726=53.3%

#<u>For test pit 2</u>

Sample depth…………3m

Table A-1-4: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	D8	C5	E9	J9	T3	D2
Mass of container,gm	15.5	15.4	15.7	15.9	15.8	15.7
Mass of container +wet soil,gm	42.1	51.8	42.1	42.6	20	21.5
Mass of container +dry soil,gm	33.4	39.6	33.1	33.2	19	20
Mass of water, gm	8.7	12.2	9.0	9.4	1	1.5
Mass of dry soil, gm	17.9	24.2	17.4	17.3	3.2	4.3
Water content(ω), %	48.6	50.4	51.7	54.3	31.25	34.88
No of blows	33	26	23	18	33.07	

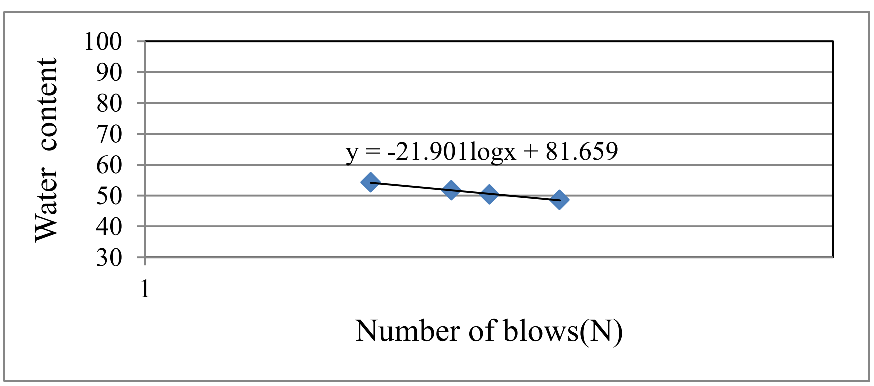

Fig.A-1-4. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-21.901*log(25)+81.659=51.1%

#For test pit 3

Sample depth…………1.5m

Table A-1-5: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	I5	C5	T4	K3	A3	D5
Mass of container,gm	15.6	15.6	15.5	15.5	15.3	15.2
Mass of container +wet soil,gm	38.9	39.0	34.2	36.3	20.4	22.2
Mass of container +dry soil,gm	33.1	32.9	29.1	30.4	19.4	20.8
Mass of water, gm	5.8	6.1	5.1	5.9	1	1.4
Mass of dry soil, gm	17.5	17.3	13.6	14.9	4.1	5.6
Water content(ω), %	33.1	35.3	37.5	39.6	24.39	25.0
No of blows	37	30	24	18	24.7	

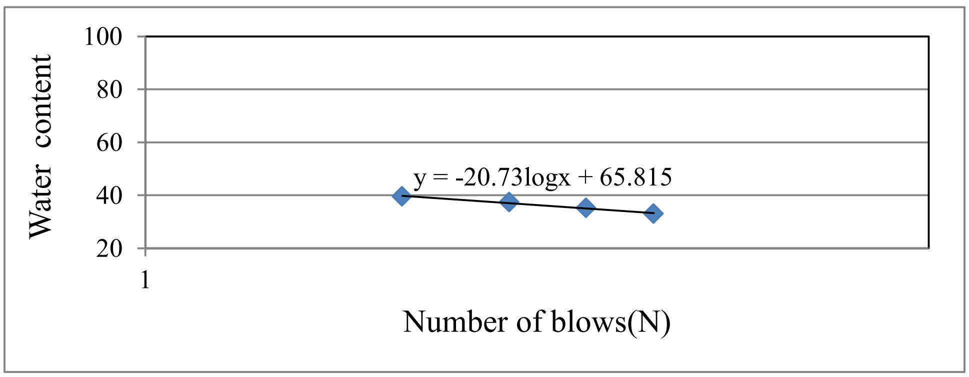

Fig.A-1-5. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-20.73*log(25)+65.815=36.8%

#For test pit 3

Sample depth…………3m

Table A-1-6: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	J3	C5	B4	D2	T3	D2
Mass of container,gm	15.7	15.7	15.6	15.3	15.5	15.5
Mass of container +wet soil,gm	35.4	34.3	35.0	41.5	21.4	23.5

Mass of container +dry soil,gm	29.4	28.6	29.0	32.9	20.2	21.9
Mass of water, gm	6.0	5.7	6.0	8.6	1.2	1.6
Mass of dry soil, gm	13.7	12.9	13.4	17.6	4.7	6.4
Water content(ω), %	43.8	44.2	44.8	48.9	25.53	25.0
No of blows	33	28	24	17	25.24	

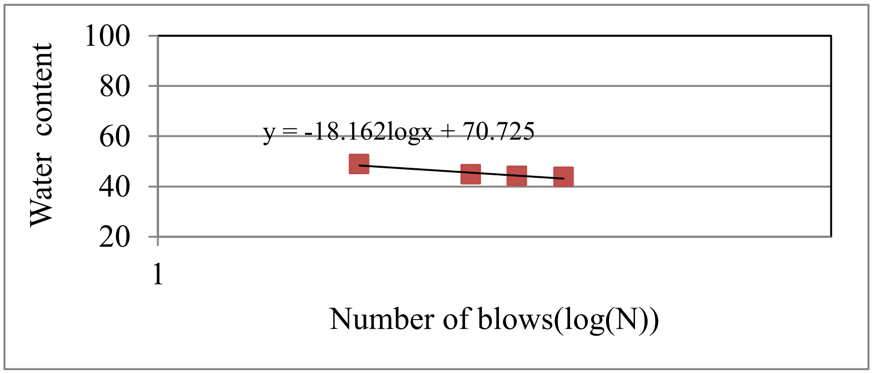

Fig.A-1-6. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-18.162*log(25)+57.725=45.3%

#<u>For test pit 4</u>

Sample depth…………1.5m

Table A-1-7: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A1	C5	C7	H6	T3	D5
Mass of container,gm	15.4	15.7	15.3	14.2	15.9	15.6
Mass of container +wet soil,gm	37.8	42.8	38.2	39.3	22.9	22.1
Mass of container +dry soil,gm	29.5	32.5	29.4	29.6	21.2	20.5
Mass of water, gm	8.3	10.3	8.8	9.7	1.7	1.6
Mass of dry soil, gm	14.1	16.8	14.1	15.4	5.3	4.9
Water content(ω), %	58.9	61.3	62.4	63.0	32.08	32.65
No of blows	33	28	23	16	32.36	

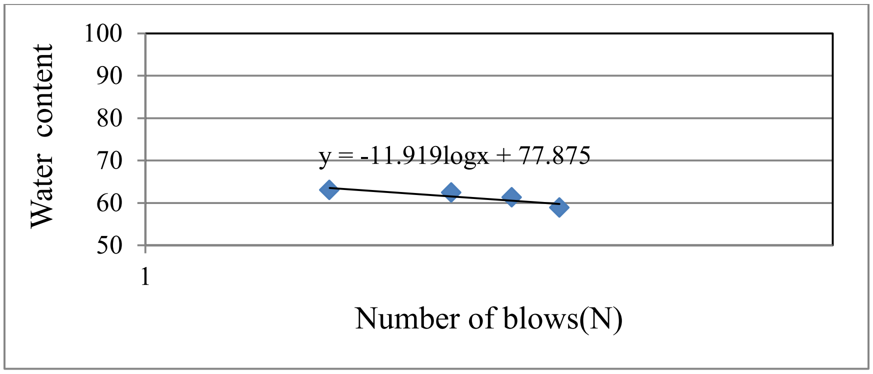

Fig. A-1-7. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-11.919*log(25)+77.875=61.2%

#<u>For test Pit 4</u>

Sample depth…………3m

Table A-1-8: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	D8	C5	E9	J9	T3	D2
Mass of container,gm	15.5	15.5	15.3	15.8	15.5	16.0
Mass of container +wet soil,gm	36.3	36.3	38.8	38.5	23.1	23.7
Mass of container +dry soil,gm	30.0	29.9	31.5	31.3	21.4	222.1
Mass of water, gm	6.3	6.4	7.3	7.2	1.7	1.6
Mass of dry soil, gm	14.5	14.4	16.2	15.5	5.9	6.1
Water content(ω), %	43.4	44.4	45.1	46.5	28.81	26.23
No of blows	37	30	24	19	27.52	

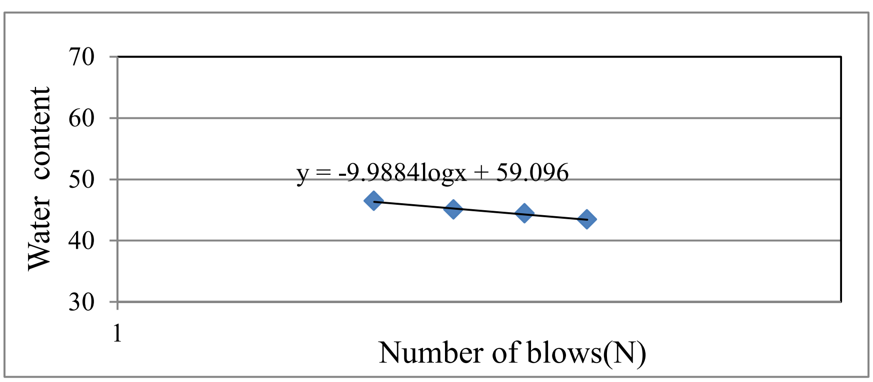

Fig.A-1-8. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-9.9884*log(25)+59.096=45.1%

#For test pit 5

Sample depth…………1.5m

Table A-1-9: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	I5	C5	T4	K3	A3	D5
Mass of container,gm	15.6	15.9	15.6	13.9	15.2	15.6
Mass of container +wet soil,gm	45.5	39.1	42.4	38.6	20.3	21
Mass of container +dry soil,gm	34.6	30.6	32.4	29.3	19.1	19.7
Mass of water, gm	10.9	8.5	10.0	9.3	1.2	1.3
Mass of dry soil, gm	19.0	14.7	16.8	15.4	3.9	4.1
Water content(ω), %	57.4	57.8	59.5	60.4	30.77	31.71
No of blows	37	29	23	18	31.24	

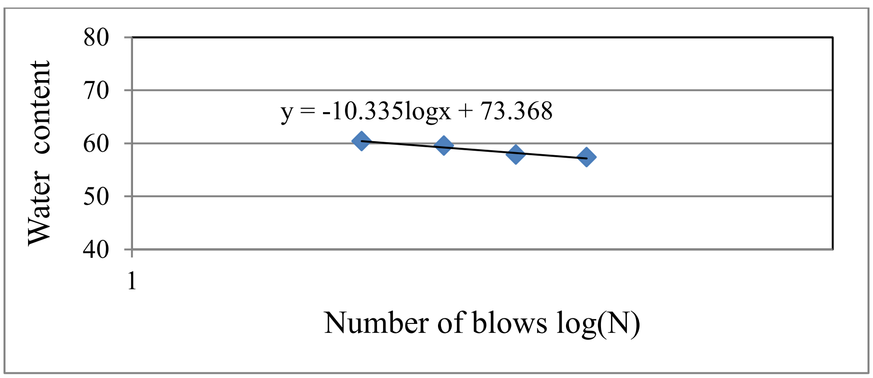

Fig.A-1-9. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-10.335*log(25)+73.368=58.9%

#For test Pit 5

Sample depth…………3m

Table A-1-10: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A4	C5	G6	D2	T3	E5
Mass of container,gm	15.8	15.6	15.7	15.5	15.7	15.6
Mass of container +wet soil,gm	40.7	39.9	41.5	44.0	22.3	23

Mass of container +dry soil,gm	30.9	30.2	31	31.9	20.7	21.1
Mass of water, gm	9.8	9.7	10.5	12.1	1.6	1.9
Mass of dry soil, gm	15.1	14.6	15.3	16.4	5	5.5
Water content(ω), %	64.9	66.4	68.6	73.8	32.0	34.55
No of blows	35	28	24	15	33.27	

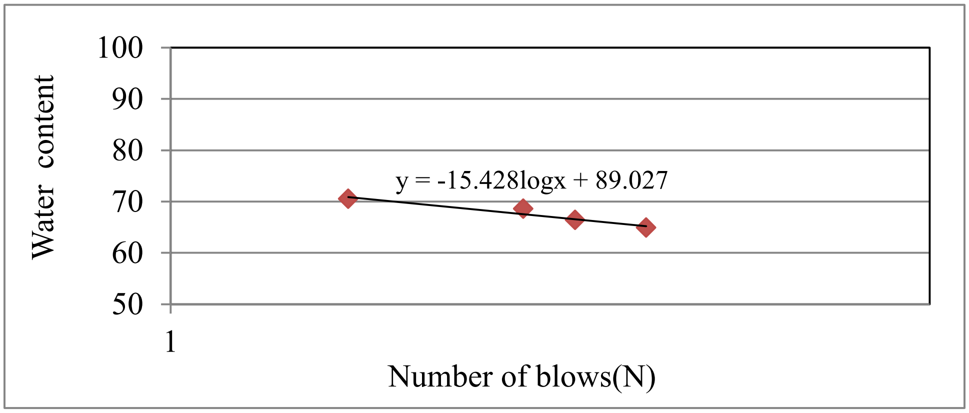

Fig.A-1-10. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-15.428*log(25)+89.025=67.5%

#**<u>For test pit 6</u>**

Sample depth…………1.9m

Table A-1-11: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A4	C5	G6	D2	T3	E5
Mass of container,gm	15.7	15.1	15.6	15.9	15.5	15.9
Mass of container +wet soil,gm	38.8	38.5	42.4	46.3	22.1	21.5
Mass of container +dry soil,gm	29.8	29.3	31.7	34.1	20.5	20.2
Mass of water, gm	9.0	9.2	10.7	12.2	1.6	1.3
Mass of dry soil, gm	14.1	14.2	16.1	18.2	5	4.3
Water content(ω), %	63.8	64.8	66.5	67.0	32.00	30.23
No of blows	37	29	24	19	31.12	

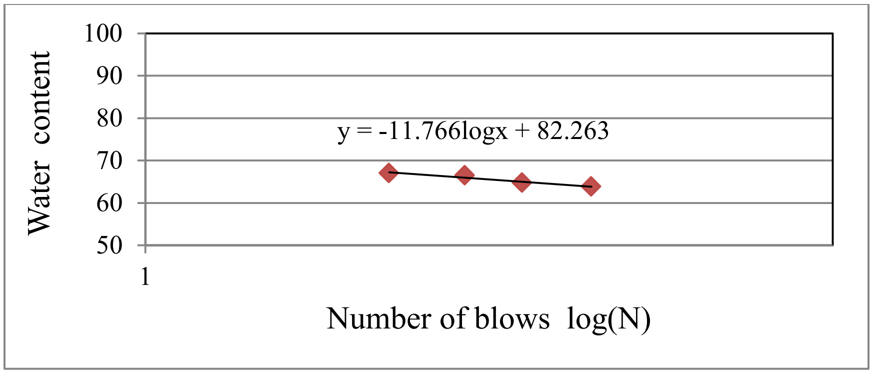

Fig.A-1-11. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-11.766*log(25)+82.263=65.8%

#<u>For test pit 7</u>

Sample depth…………1.7m

Table A-1-12: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A1	C2	U4	D6	T3	D2
Mass of container,gm	15.4	15.7	15.8	15.6	15.9	15.3
Mass of container +wet soil,gm	38.4	36.2	37.4	36	22.8	21.7
Mass of container +dry soil,gm	30.0	28.6	29.3	28.2	21.3	20.2
Mass of water, gm	8.4	7.6	8.1	7.8	1.5	1.5
Mass of dry soil, gm	14.6	12.9	13.5	12.6	5.4	4.9
Water content(ω), %	57.5	58.9	60	61.9	27.78	30.61
No of blows	36	27	22	17	29.2	

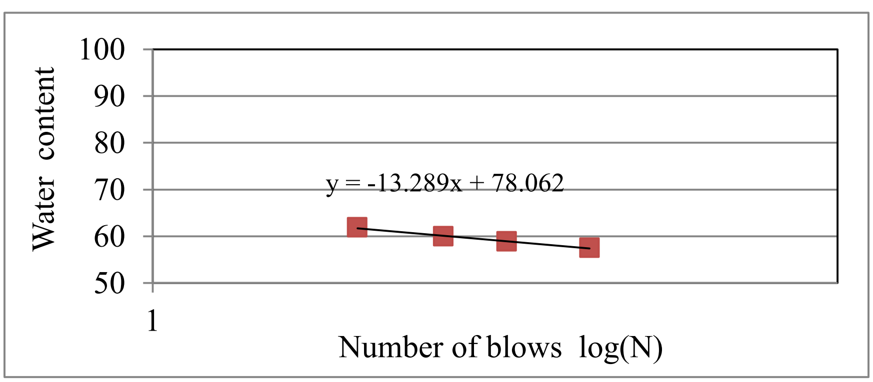

Fig.A-1-12. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-13.289*log(25)+78.7062=59.5%

#For test pit 8

Sample depth…………1.6m

Table A-1-13: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A14	A28	C30	D21	A3	D5
Mass of container,gm	15.7	14.3	15.6	15.6	15.7	15.7
Mass of container +wet soil,gm	40.7	43.0	45.0	39.5	22.7	22.8
Mass of container +dry soil,gm	30.9	31.7	33.4	30.0	21	21.1
Mass of water, gm	9.8	11.3	11.6	9.5	1.7	1.7
Mass of dry soil, gm	15.2	17.4	17.8	14.4	5.3	5.4
Water content(ω), %	64.5	64.9	65.2	66.0	32.08	31.48
No of blows	38	30	23	18	31.78	

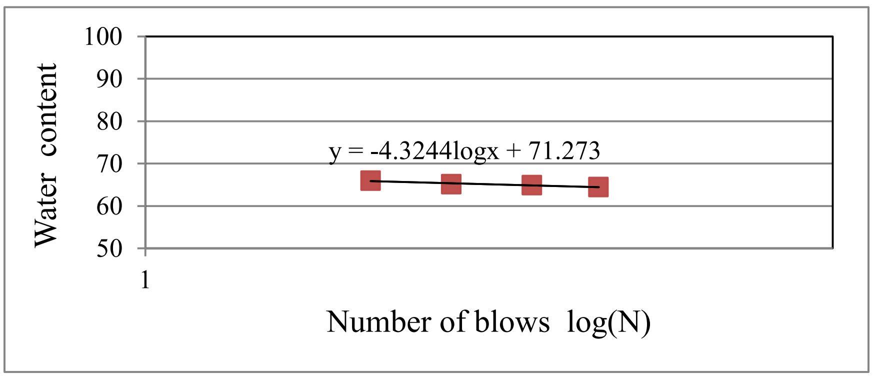

Fig.A-1-13. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-4.3244*log(25)+71.273=65.2%

#For test pit 9

Sample depth…………1.5m

Table A-1-14: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	D8	C5	E9	J9	T3	D2
Mass of container,gm	15.5	15.6	15.3	15.7	15.6	15.6
Mass of container +wet soil,gm	36.0	39.2	39.6	35.8	23.5	23.1

Mass of container +dry soil,gm	30.6	32.9	32.9	30.2	21.9	21.4
Mass of water, gm	5.4	6.3	6.7	5.6	1.6	1.7
Mass of dry soil, gm	15.1	17.3	17.6	14.5	6.3	5.8
Water content(ω), %	35.8	36.4	38.1	38.6	25.40	29.31
No of blows	37	29	23	18	27.35	

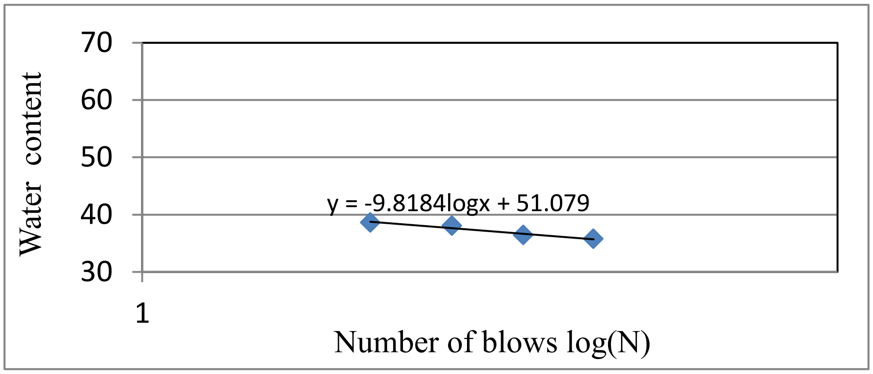

Fig.A-1-14. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-9.8184*log(25)+51.079=37.4%

#<u>For test pit 9</u>

Sample depth…………3m

Table A-1-15: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	I5	C5	T4	K3	A3	D5
Mass of container,gm	15.6	15.5	15.5	15.4	15.6	15.6
Mass of container +wet soil,gm	34.4	35.5	33.3	33.5	23.5	23.1
Mass of container +dry soil,gm	30.2	30.9	29.0	29.1	22	21.6
Mass of water, gm	4.2	4.6	4.3	4.4	1.5	1.5
Mass of dry soil, gm	14.6	15.4	13.5	13.7	6.4	6
Water content(ω), %	28.8	29.9	31.9	32.1	23.44	25.0
No of blows	37	29	24	16	24.22	

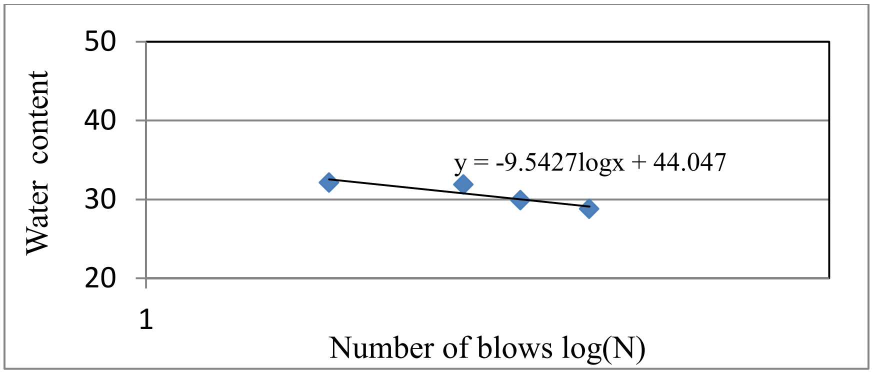

Fig.A-1-15. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-9.5427*log(25)+44.047=30.7%

#<u>For test pit 10</u>

Sample depth…………1.5m

Table A-1-16: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	J3	C5	B4	D2	T3	D2
Mass of container,gm	10.9	15.7	14.9	15.5	15.7	15.8
Mass of container +wet soil,gm	34.5	40.1	38.8	39.3	22.4	23.2
Mass of container +dry soil,gm	24.2	29.3	27.9	28.4	20.8	21.3
Mass of water, gm	10.3	10.8	10.9	10.9	1.6	1.9
Mass of dry soil, gm	13.3	13.6	13.0	12.9	5.1	5.5
Water content(ω), %	77.4	79.4	83.8	84.5	31.37	34.55
No of blows	37	28	23	17	32.96	

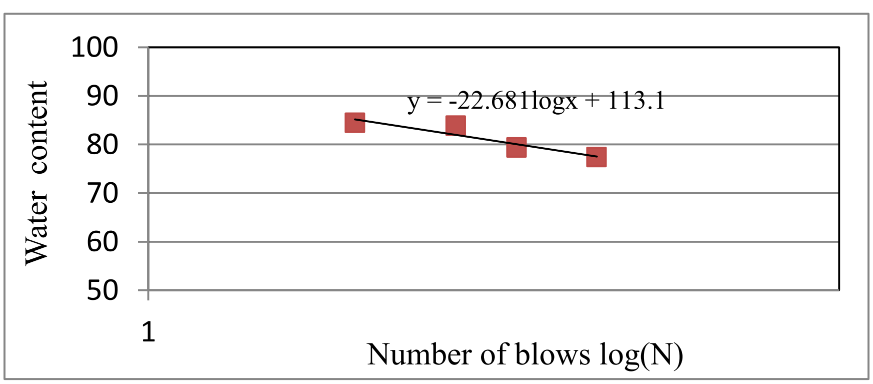

Fig.A-1-16. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-22.681*log(25)+113.1=81.4%

#For test pit 10

Sample depth…………3m

Table A-1-17: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A1	C5	C7	H6	T3	D5
Mass of container,gm	15.4	15.5	15.4	15.3	15.4	10.9
Mass of container +wet soil,gm	36.7	36.7	44.4	43.6	21.6	17.0
Mass of container +dry soil,gm	27.0	27.0	31.1	30.5	20.2	15.6
Mass of water, gm	9.7	9.7	13.3	13.1	1.4	1.4
Mass of dry soil, gm	11.6	11.5	15.7	15.2	4.8	4.7
Water content(ω), %	83.6	84.3	84.7	86.2	29.17	29.79
No of blows	38	29	24	18	29.48	

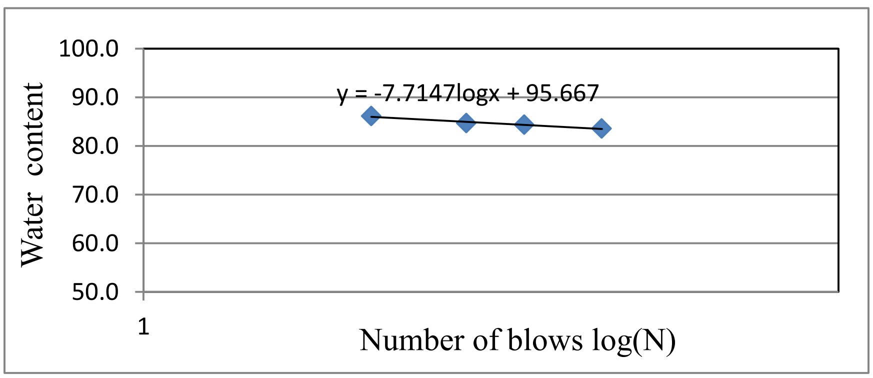

Fig.A-1-17. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-7.7147*log(25)+95.667=84.9%

#For test pit 11

Sample depth…………1.5m

Table A-1-18: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A4	C5	G6	D2	T3	E5
Mass of container,gm	15.7	15.5	15.5	15.7	15.7	15.8
Mass of container +wet soil,gm	35.9	35.0	39.4	40.4	23.2	22.7

Mass of container +dry soil,gm	29.1	28.3	31.1	31.6	21.4	21.1
Mass of water, gm	6.8	6.7	8.3	8.8	1.8	1.6
Mass of dry soil, gm	13.4	12.8	15.6	15.9	5.7	5.3
Water content(ω), %	50.7	52.3	53.2	55.3	31.58	30.19
No of blows	33	28	22	17	30.88	

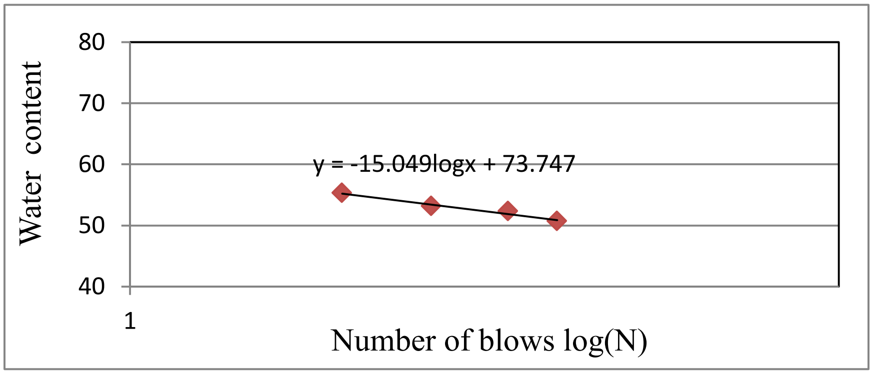

Fig.A-1-18. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-15.049*log(25)+73.747=52.7%

#<u>For test pit 11</u>

Sample depth…………3m

Table A-1-19: Liquid limit and Plastic limit test results

	Liquid Limit				Plastic Limit	
Trail No	1	2	3	4	1	2
Container No	A1	C2	U4	D6	T3	D2
Mass of container,gm	15.7	15.3	15.6	15.5	15.3	15.6
Mass of container +wet soil,gm	37.3	36.2	37.4	39.1	21.4	22.2
Mass of container +dry soil,gm	31.4	30.3	31.1	32	20.3	20.9
Mass of water, gm	5.9	5.9	6.3	7.1	1.1	1.3
Mass of dry soil, gm	15.7	15	15.5	16.5	5	5.3
Water content(ω), %	37.6	39.3	40.6	43	22.00	24.53
No of blows	35	29	22	16	23.26	

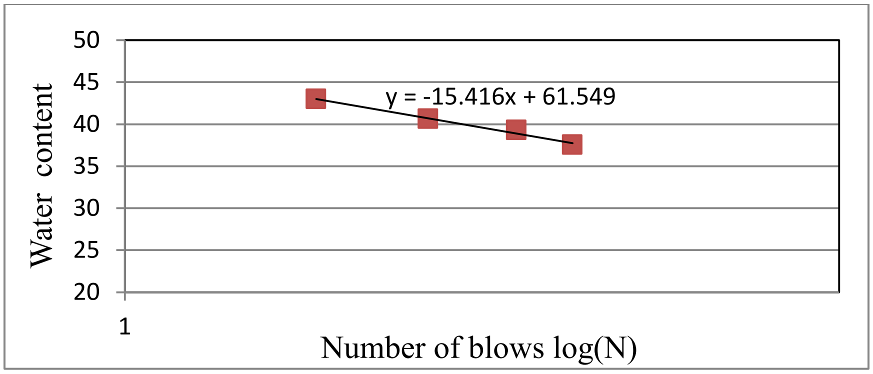

Fig.A-1-19. Water content Vs log(N) graph

The value of liquid limit is determined by: LL=-15.416*log(25)+61.549=40%

A-2) A typical specific gravity test result

Table: A-2-1 Typical specific gravity test result for TP1, 3m

Pycnometer no	A	B
Mass of dry soil(Ms),gm	25.00	25.00
Mass of pycnometer + water + soil (M1),gm	160.30	160.20
Mass of pycnometer + water (M2),gm	144.80	144.60
Test Temp in °C	26.60	27.40
Specific gravity of soil at test temperature	2.63	2.66
Correction factor, K	0.9984	0.9982
Specific gravity of soil at 20°C	2.63	2.65
Average specific gravity of soil at 20°C	2.64	

Appendix-B

B) The detailed compaction test results

#For test pit 1

Sample depth…………1.5m

Table B-1: Moisture content Vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4507.3	4685	4833	4741.7	4731.3
Mass of Compacted soil, g	1369.3	1547	1695	1603.7	1593.3
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.45	1.64	1.80	1.70	1.68
Water Contenet, %	15.29	21.81	28.22	43.49	43.32
Dry density, g/cm3	1.26	1.35	1.40	1.18	1.17

Table B-2: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.6	15.65	15.55	15.55	15.45
Mass of container + wet soil, g	46.9	51.95	70.3	70.65	71.2
Mass of ontainer + Dry soil, g	42.75	45.45	58.25	53.95	54
Mass of Water, g	4.15	6.5	12.05	16.7	17.2
Mass of Dry soil, g	27.15	29.8	42.7	38.4	38.55
Water content, %	15.29	21.81	28.22	43.49	44.62
Dry Unit Weight, g/cm3	1.26	1.35	1.40	1.18	1.16

#For test pit 1

Sample depth…………3m

Table B-3: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138

Mass of mold + Compacted Soil, g	4493.5	4583.5	4827	4779.4	4768.5
Mass of Compacted soil, g	1355.5	1445.5	1689	1641.4	1630.5
Volume of Mold, cm3	944	944	944	945	946
Bulk density, g/cm3	1.44	1.53	1.79	1.74	1.72
Water Contenet, %	17.52	23.41	30.85	40.27	44.90
Dry density, g/cm3	1.22	1.24	1.37	1.24	1.19

Table B-4: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.5	15.5	15.75	15.55	15.25
Mass of container + wet soil, g	47.7	53.45	59.65	67.8	64.3
Mass of ontainer + Dry soil, g	42.9	46.25	49.3	52.8	49.1
Mass of Water, g	4.8	7.2	10.35	15	15.2
Mass of Dry soil, g	27.4	30.75	33.55	37.25	33.85
Water content, %	17.52	23.41	30.85	40.27	44.90
Dry Unit Weight, g/cm3	1.22	1.24	1.37	1.24	1.19

#<u>For test pit 2</u>

Sample depth…………1.5m

Table B-5: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4523.7	4786.4	4883.2	4720	4643
Mass of Compacted soil, g	1385.7	1648.4	1745.2	1582	1505
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.47	1.75	1.85	1.68	1.59
Water Contenet, %	20.19	26.69	40.37	48.28	54.31
Dry density, g/cm3	1.22	1.38	1.32	1.13	1.03

Table B-6: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.75	15.5	15.65	14.8	15.55

Mass of container + wet soil, g	40.75	53	76.85	70.7	72.8
Mass of ontainer + Dry soil, g	36.55	45.1	59.25	52.5	52.65
Mass of Water, g	4.2	7.9	17.6	18.2	20.15
Mass of Dry soil, g	20.8	29.6	43.6	37.7	37.1
Water content, %	20.19	26.69	40.37	48.28	54.31
Dry Unit Weight, g/cm3	1.22	1.38	1.32	1.13	1.03

#<u>For test pit 2</u>

Sample depth…………3m

Table B-7: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4595.7	4890.4	4793.2	4747	4723
Mass of Compacted soil, g	1457.7	1752.4	1655.2	1609	1585
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.54	1.86	1.75	1.70	1.68
Water Contenet, %	20.19	26.69	40.37	48.28	54.31
Dry density, g/cm3	1.28	1.47	1.25	1.15	1.09

Table B-8: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.75	15.5	15.65	14.8	15.55
Mass of container + wet soil, g	40.75	53	76.85	70.7	72.8
Mass of ontainer + Dry soil, g	36.55	45.1	59.25	52.5	52.65
Mass of Water, g	4.2	7.9	17.6	18.2	20.15
Mass of Dry soil, g	20.8	29.6	43.6	37.7	37.1
Water content, %	20.19	26.69	40.37	48.28	54.31
Dry Unit Weight, g/cm3	1.28	1.47	1.25	1.15	1.09

#**For test pit 4**

Sample depth…………1.5m

Table B-9: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4690.3	4838.3	4935.1	4921.7	4880.7
Mass of Compacted soil, g	1552.3	1700.3	1797.1	1783.7	1742.7
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.64	1.80	1.90	1.89	1.85
Water Contenet, %	14.59	18.88	27.23	30.92	32.50
Dry density, g/cm3	1.43	1.52	1.50	1.44	1.39

Table B-10: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.8	15.4	15.65	15.45	15.5
Mass of container + wet soil, g	58.2	54.75	71.95	74.1	89.7
Mass of ontainer + Dry soil, g	52.8	48.5	59.9	60.25	71.5
Mass of Water, g	5.4	6.25	12.05	13.85	18.2
Mass of Dry soil, g	37	33.1	44.25	44.8	56
Water content, %	14.59	18.88	27.23	30.92	32.50
Dry Unit Weight, g/cm3	1.43	1.52	1.50	1.44	1.39

#**For test pit 4**

Sample depth…………3m

Table B-11: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4639.7	4806.3	4924.7	4876.2	4859.1
Mass of Compacted soil, g	1501.7	1668.3	1786.7	1738.2	1721.1

Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.59	1.77	1.89	1.84	1.82
Water Contenet, %	19.05	24.16	28.16	32.90	34.50
Dry density, g/cm3	1.34	1.42	1.48	1.39	1.36

Table B-12: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.55	15.2	15.65	14.85	15.65
Mass of container + wet soil, g	58.05	54	60.25	66.55	69.65
Mass of ontainer + Dry soil, g	51.25	46.45	50.45	53.75	55.8
Mass of Water, g	6.8	7.55	9.8	12.8	13.85
Mass of Dry soil, g	35.7	31.25	34.8	38.9	40.15
Water content, %	19.05	24.16	28.16	32.90	34.50
Dry Unit Weight, g/cm3	1.34	1.42	1.48	1.39	1.36

#For test pit 9

Sample depth…………1.5m

Table B-13: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4707.5	4974.1	4963	4946.1	4898.7
Mass of Compacted soil, g	1569.5	1836.1	1825	1808.1	1760.7
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.66	1.95	1.93	1.92	1.87
Water Contenet, %	15.67	21.38	28.03	29.15	31.84
Dry density, g/cm3	1.44	1.60	1.51	1.48	1.41

Table B-14: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.6	18.1	13.5	15.65	15.65
Mass of container + wet soil, g	49.55	62.1	63.75	57.3	83.35
Mass of ontainer + Dry soil, g	44.95	54.35	52.75	47.9	67

Mass of Water, g	4.6	7.75	11	9.4	16.35
Mass of Dry soil, g	29.35	36.25	39.25	32.25	51.35
Water content, %	15.67	21.38	28.03	29.15	31.84
Dry Unit Weight, g/cm3	1.44	1.60	1.51	1.48	1.41

#For test pit 9

Sample depth…………3m

Table B-15: Moisture content vs dry density computation

Determination No.	1	2	3	4	5
Mass of Mold, g	3138	3138	3138	3138	3138
Mass of mold + Compacted Soil, g	4605	4863	4995.9	4899.3	4869.9
Mass of Compacted soil, g	1467	1725	1857.9	1761.3	1731.9
Volume of Mold, cm3	944	944	944	944	944
Bulk density, g/cm3	1.55	1.83	1.97	1.87	1.83
Water Contenet, %	12.27	18.14	24.68	31.76	33.50
Dry density, g/cm3	1.38	1.55	1.58	1.42	1.37

Table B-16: Water content computation

Conatainer No	77	40	49	B2	C2
Mass of container, g	15.8	14.7	15.6	15.75	15.7
Mass of container + wet soil, g	46.9	50.2	54.25	60.55	94
Mass of ontainer + Dry soil, g	43.5	44.75	46.6	49.75	74.35
Mass of Water, g	3.4	5.45	7.65	10.8	19.65
Mass of Dry soil, g	27.7	30.05	31	34	58.65
Water content, %	12.27	18.14	24.68	31.76	33.50
Dry Unit Weight, g/cm3	1.38	1.55	1.58	1.42	1.37

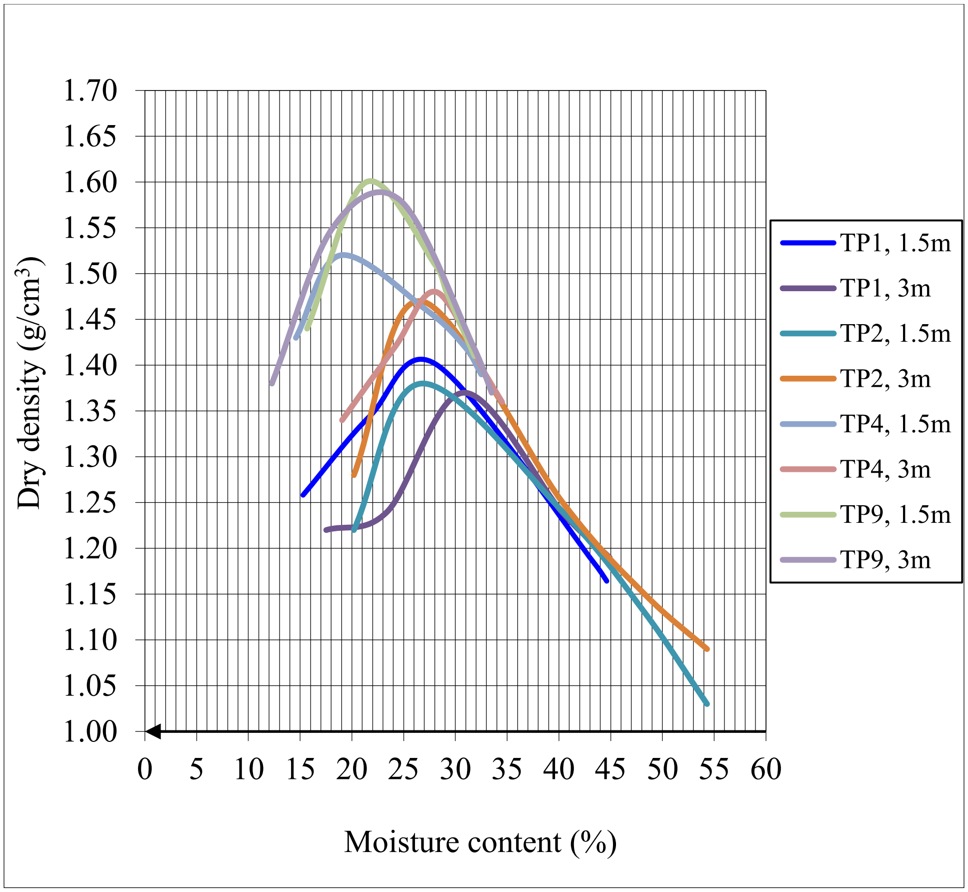

Fig.B-1. Dry density Vs Moisture content graph

Appendix-C

C) The results of Unconfined compressive strength test

#<u>For test pit 1</u>

Sample depth…………1.5m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-1: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.5	0.66	15	0.0207	0.00114	18.13
1	1.32	24	0.0331	0.00115	28.82
1.5	1.97	37	0.0511	0.00116	44.14
2	2.63	48	0.0662	0.00116	56.88
2.5	3.29	58	0.0800	0.00117	68.26
3	3.95	68	0.0938	0.00118	79.48
3.5	4.61	78	0.1076	0.00119	90.55
4	5.26	84	0.1159	0.00120	96.84
4.5	5.92	92	0.1270	0.00121	105.33
5	6.58	98	0.1352	0.00121	111.41
5.5	7.24	101	0.1394	0.00122	114.02
6	7.89	102	0.1408	0.00123	114.33
6.5	8.55	97	0.1339	0.00124	107.95
7	9.21	91	0.1256	0.00125	100.54
7.5	9.87	90	0.1242	0.00126	98.72

#<u>For test pit 1</u>

Sample depth…………3m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-2: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	13	0.0179	0.00114	15.78
0.4	0.53	21.5	0.0297	0.00114	26.03
0.6	0.79	33.5	0.0462	0.00114	40.45
0.8	1.05	44.5	0.0614	0.00115	53.58
1	1.32	56.5	0.0780	0.00115	67.85
1.2	1.58	67.5	0.0932	0.00115	80.85
1.4	1.84	79	0.1090	0.00116	94.37
1.6	2.11	89.5	0.1235	0.00116	106.62
1.8	2.37	99	0.1366	0.00116	117.62
2	2.63	108.5	0.1497	0.00116	128.56
2.2	2.89	118.5	0.1635	0.00117	140.03
2.4	3.16	129	0.1780	0.00117	152.03
2.6	3.42	136	0.1877	0.00117	159.84
2.8	3.68	143.5	0.1980	0.00118	168.20
3	3.95	150.5	0.2077	0.00118	175.92
3.2	4.21	157	0.2167	0.00118	183.01
3.4	4.47	167	0.2305	0.00119	194.14
3.8	4.74	175.5	0.2422	0.00119	203.45
3.8	5.00	182	0.2512	0.00119	210.41
4	5.26	186	0.2567	0.00120	214.44
4.2	5.53	192	0.2650	0.00120	220.74
4.4	5.79	201	0.2774	0.00120	230.44
4.6	6.05	210	0.2898	0.00121	240.09

4.8	6.32	216	0.2981	0.00121	246.26
5	6.58	221	0.3050	0.00121	251.25
5.2	6.84	223	0.3077	0.00122	252.81
5.4	7.11	225	0.3105	0.00122	254.35
5.6	7.37	224	0.3091	0.00122	252.51
5.8	7.63	220	0.3036	0.00123	247.29
6	7.89	214	0.2953	0.00123	239.86
6.2	8.16	202	0.2788	0.00123	225.77

#For test pit 2

Sample depth…………1.5m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-3: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	30	0.0414	0.00114	36.41
0.4	0.53	60	0.0828	0.00114	72.63
0.6	0.79	100	0.1380	0.00114	120.73
0.8	1.05	145	0.2001	0.00115	174.60
1	1.32	185	0.2553	0.00115	222.17
1.2	1.58	210	0.2898	0.00115	251.52
1.4	1.84	218	0.3008	0.00116	260.40
1.6	2.11	205	0.2829	0.00116	244.22
1.8	2.37	180	0.2484	0.00116	213.86
2	2.63	150	0.2070	0.00116	177.74

#<u>For test pit 2</u>

Sample depth…………3m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-4: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	8	0.0110	0.00114	9.71
0.4	0.53	26	0.0359	0.00114	31.47
0.6	0.79	51	0.0704	0.00114	61.57
0.8	1.05	97	0.1339	0.00115	116.80
1	1.32	140	0.1932	0.00115	168.13
1.2	1.58	178	0.2456	0.00115	213.19
1.4	1.84	218	0.3008	0.00116	260.40
1.6	2.11	245	0.3381	0.00116	291.87
1.8	2.37	270	0.3726	0.00116	320.79
2	2.63	280	0.3864	0.00116	331.77
2.2	2.89	279	0.3850	0.00117	329.70
2.4	3.16	255	0.3519	0.00117	300.52
2.6	3.42	218	0.3008	0.00117	256.22
2.8	3.68	170	0.2346	0.00118	199.26

#<u>For test pit 4</u>

Sample depth…………1.5m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-5: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	25	0.0345	0.00114	30.34
0.4	0.53	64	0.0883	0.00114	77.47
0.6	0.79	125	0.1725	0.00114	150.92
0.8	1.05	205	0.2829	0.00115	246.84
1	1.32	310	0.4278	0.00115	372.28
1.2	1.58	382	0.5272	0.00115	457.53
1.4	1.84	401	0.5534	0.00116	479.00
1.6	2.11	410	0.5658	0.00116	488.44
1.8	2.37	416	0.5741	0.00116	494.25
2	2.63	401	0.5534	0.00116	475.15
2.2	2.89	386	0.5327	0.00117	456.14
2.4	3.16	370	0.5106	0.00117	436.05

#<u>For test pit 4</u>

Sample depth………….3m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-6: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	25	0.0345	0.00114	30.34
0.4	0.53	64	0.0883	0.00114	77.47
0.6	0.79	94	0.1297	0.00114	113.49

0.8	1.05	110	0.1518	0.00115	132.45
1	1.32	190	0.2622	0.00115	228.17
1.2	1.58	220	0.3036	0.00115	263.50
1.4	1.84	312	0.4306	0.00116	372.69
1.6	2.11	326	0.4499	0.00116	388.37
1.8	2.37	329	0.4540	0.00116	390.89
2	2.63	307	0.4237	0.00116	363.77
2.2	2.89	278	0.3836	0.00117	328.51
2.4	3.16	243	0.3353	0.00117	286.38

#<u>For test pit 9</u>

Sample depth…………1.5m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138kN/div

Table C-7: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	3	0.0041	0.00114	3.64
0.4	0.53	4	0.0055	0.00114	4.84
0.6	0.79	6	0.0083	0.00114	7.24
0.8	1.05	7	0.0097	0.00115	8.43
1	1.32	10	0.0138	0.00115	12.01
1.2	1.58	14	0.0193	0.00115	16.77
1.4	1.84	19	0.0262	0.00116	22.70
1.6	2.11	23	0.0317	0.00116	27.40
1.8	2.37	33	0.0455	0.00116	39.21
2	2.63	43	0.0593	0.00116	50.95
2.2	2.89	55	0.0759	0.00117	64.99

2.4	3.16	61	0.0842	0.00117	71.89
2.6	3.42	57	0.0787	0.00117	66.99
2.8	3.68	50	0.0690	0.00118	58.60
3	3.95	46	0.0635	0.00118	53.77

#For test pit 9

Sample depth…………3m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-8: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	6	0.0083	0.00114	7.28
0.4	0.53	11	0.0152	0.00114	13.32
0.6	0.79	16	0.0221	0.00114	19.32
0.8	1.05	21	0.0290	0.00115	25.29
1	1.32	29	0.0400	0.00115	34.83
1.2	1.58	37	0.0511	0.00115	44.32
1.4	1.84	44	0.0607	0.00116	52.56
1.6	2.11	49	0.0676	0.00116	58.37
1.8	2.37	51	0.0704	0.00116	60.59
2	2.63	50	0.0690	0.00116	59.25
2.2	2.89	49	0.0676	0.00117	57.90
2.4	3.16	46	0.0635	0.00117	54.21

#For test pit 10

Sample depth…………1.5m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-9: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	18.5	0.0255	0.00114	22.45
0.4	0.53	34	0.0469	0.00114	41.16
0.6	0.79	55	0.0759	0.00114	66.40
0.8	1.05	76.5	0.1056	0.00115	92.12
1	1.32	98	0.1352	0.00115	117.69
1.2	1.58	119	0.1642	0.00115	142.53
1.4	1.84	141	0.1946	0.00116	168.43
1.6	2.11	160.5	0.2215	0.00116	191.21
1.8	2.37	179.5	0.2477	0.00116	213.27
2	2.63	196.5	0.2712	0.00116	232.83
2.2	2.89	211.5	0.2919	0.00117	249.93
2.4	3.16	228.5	0.3153	0.00117	269.29
2.6	3.42	242.5	0.3347	0.00117	285.01
2.8	3.68	257.5	0.3554	0.00118	301.81
3	3.95	268.5	0.3705	0.00118	313.85
3.2	4.21	278.5	0.3843	0.00118	324.65
3.4	4.47	289	0.3988	0.00119	335.96
3.6	4.74	298.5	0.4119	0.00119	346.05
3.8	5.00	305.5	0.4216	0.00119	353.18
4	5.26	311.5	0.4299	0.00120	359.12
4.2	5.53	316.5	0.4368	0.00120	363.87
4.4	5.79	321	0.4430	0.00120	368.02
4.6	6.05	324.5	0.4478	0.00121	370.99
4.8	6.32	326	0.4499	0.00121	371.66
5	6.58	327	0.4513	0.00121	371.76

5.2	6.84	327.5	0.4520	0.00122	371.28
5.4	7.11	322.5	0.4451	0.00122	364.57
5.6	7.37	313.5	0.4326	0.00122	353.40
5.8	7.63	308.5	0.4257	0.00123	346.77

#**For test pit 10**

Sample depth…………3m

Diameter of the sample =38mm

Length of sample =76mm

Ring Calibration Factor =0.00138 kN/div

Table C-10: Computation of UCS values

Axial Deformation (mm)	Axial Strain (%)	Proving Ring Reading (div)	Axial Load (kN)	Corrected Area (m^2)	Axial Stress (kPa)
0	0.00	0	0.0000	0.00113	0
0.2	0.26	1	0.0014	0.00114	1.21
0.4	0.53	7	0.0097	0.00114	8.47
0.6	0.79	16	0.0221	0.00114	19.32
0.8	1.05	33	0.0455	0.00115	39.74
1	1.32	47	0.0649	0.00115	56.44
1.2	1.58	60	0.0828	0.00115	71.86
1.4	1.84	72	0.0994	0.00116	86.01
1.6	2.11	83	0.1145	0.00116	98.88
1.8	2.37	93	0.1283	0.00116	110.49
2	2.63	101	0.1394	0.00116	119.68
2.2	2.89	109	0.1504	0.00117	128.81
2.4	3.16	116	0.1601	0.00117	136.71
2.6	3.42	122	0.1684	0.00117	143.39
2.8	3.68	127	0.1753	0.00118	148.86
3	3.95	131	0.1808	0.00118	153.13
3.2	4.21	135	0.1863	0.00118	157.37

3.4	4.47	139	0.1918	0.00119	161.59
3.6	4.74	142	0.1960	0.00119	164.62
3.8	5.00	144	0.1987	0.00119	166.48
4	5.26	146	0.2015	0.00120	168.32
4.2	5.53	148	0.2042	0.00120	170.15
4.4	5.79	149	0.2056	0.00120	170.83
4.6	6.05	148	0.2042	0.00121	169.20
4.8	6.32	147	0.2029	0.00121	167.59
5	6.58	144	0.1987	0.00121	163.71

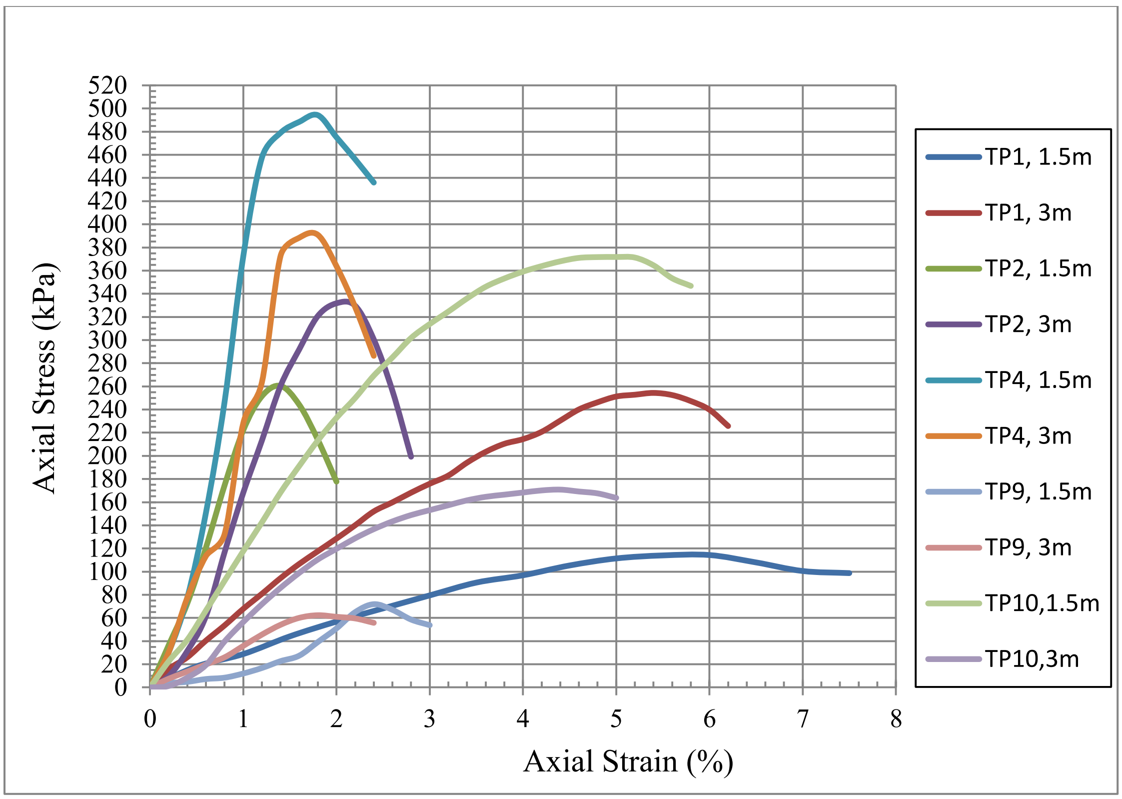

Fig.C-1. Axial stress Vs axial strain graph.

Appendix-D

D) The detailed calculation and plot of consolidation test results

 # <u>For test pit 1</u>

Sample depth……………………3m

Data at the beginning and end of the test

Sample type …………………….Undisturbed

Ring area………………….........19.63 cm^2

Volume of sample………………39.27 cm^3

Mass of empty ring (M_R)………..69.8 gm

Mass of soil+ ring (M_{S+R})……….138.35gm

Specific gravity of soil(G_s)………2.64

Mass of soil (M_s)………………...68.55gm

Mass of dry soil (M_d)……………51.4g

Ring Diameter (D)……………….5cm

Height of sample (H_0)……………20mm

Initial moisture content (ω)……...32.74%

Table D-1: Dial gauge reading for each incremental loading

Dial Gauge Reading, mm							
Square root of time		**50kpa**	**100kpa**	**200kpa**	**400kpa**	**800kpa**	**1600kpa**
	Time (min)	**Dial reading (mm)**	**Dial reading (mm)**	**Dial reading (mm)**	**Dial reading (mm)**	**Dial reading (mm)**	**Dial reading (mm)**
0	0	2.882	3.004	3.2	3.518	3.93	4.71
0.316228	0.1	2.93	3.092	3.344	3.722	4.224	4.984
0.5	0.25	2.942	3.098	3.384	3.734	4.292	5.022
0.707107	0.5	2.952	3.104	3.394	3.744	4.292	5.066
1	1	2.954	3.112	3.404	3.752	4.31	5.09

1.414214	2	2.956	3.128	3.422	3.766	4.338	5.136
2	4	2.958	3.136	3.436	3.776	4.372	5.182
2.828427	8	2.962	3.146	3.448	3.788	4.41	5.25
3.872983	15	2.966	3.156	3.458	3.802	4.452	5.324
5.477226	30	2.97	3.166	3.47	3.822	4.504	5.428
7.745967	60	2.98	3.192	3.482	3.868	4.558	5.532
10.95445	120	2.986	3.194	3.492	3.882	4.624	5.68
15.49193	240	2.99	3.196	3.502	3.902	4.666	5.796
21.9089	480	2.99	3.198	3.508	3.92	4.694	5.85
37.94733	1440	3.004	3.2	3.518	3.93	4.71	5.892

Compression Index (C_c)

Height of solid (H_s) = $\frac{M_d}{\rho_w * A * G_s}$ Where: G_s= Specific gravity of the solids

A = Area of the consolidation ring

ρ_w= density of water 1g/cm^3

$= \frac{51.4}{1*19.63*2.64} = 9.91$mm

Height of void (H_v) = H-Hs

=20-9.91=10.09mm

Initial void ratio (e_o) = $\frac{H_v}{H_s} = \frac{10.09}{9.91}$ =1.018≈1.02

Computation of void ratio for loading of 7kPa

Initial dial reading=3.0

Final dial reading=2.882

ΔH_1=2.882-3.0=-0.118mm

$\Delta e_1 = \frac{\Delta H_1}{H_s}$ =-0.118/9.91=-0.0119

$e_1 = e_o - \Delta e_1$=1.018-(-0.0119)=1.0299≈1.03

Computation of void ratio for loading of 50kPa

Initial dial reading=2.882

Final dial reading=3.004

ΔH_2=3.004-2.882=0.122mm

$\Delta e_2 = \frac{\Delta H_2}{H_s}$=0.122/9.91=0.012

$e_2 = e_1 - \Delta e_2$=1.0299-0.012=1.0179≈1.02

Computation of void ratio for loading of 100kPa

Initial dial reading=3.004

Final dial reading=3.2

ΔH_3=3.2-3.004=0.196mm

$\Delta e_3 = \frac{\Delta H_3}{H_s}$=0.196/9.91=0.0198

$e_3 = e_2 - \Delta e_3$=1.0179-0.0198=0.998≈1.0

Computation of void ratio for loading of 200kPa

Initial dial reading=3.2

Final dial reading=3.518

ΔH_4=3.518-3.2=0.318mm

$\Delta e_4 = \frac{\Delta H_4}{H_s}$=0.318/9.91=0.0321

$e_4 = e_3 - \Delta e_4$=0.998-0.0321=0.966≈0.97

Computation of void ratio for loading of 400kPa

Initial dial reading=3.518

Final dial *reading=3.93*

ΔH_5=3.93-3.518=0.412mm

$\Delta e_5 = \frac{\Delta H_5}{H_s}$=0.412/9.91=0.0416

$e_5 = e_4 - \Delta e_5$=0.966-0.0416=0.924≈0.92

Computation of void ratio for loading of 800kPa

Initial dial reading=3.93

Final dial reading=4.71

ΔH_6=4.71-3.93=0.78mm

$\Delta e_6 = \frac{\Delta H_6}{H_s}$=0.78/9.91=0.0787

$e_6 = e_5 - \Delta e_6$=0.924-0.07871=0.845≈0.85

Computation of void ratio for loading of 1600kPa

Initial dial reading=4.71

Final dial reading=5.892

ΔH_7=5.892-4.71=1.182mm

$\Delta e_7 = \frac{\Delta H_7}{H_s}$ =1.182/9.91=0.1193

$e_7 = e_6 - \Delta e_7$=0.845-0.1193=0.7257≈0.73

The calculated values of final void ratio are presented in Table D-2.

Table D-2: Determining the value of final void ratio for TP1

Applied Pressure, P (kPa)	Final Dial Reading (mm)	Change In Specimen Height, (mm)	Final Specimen Height, (mm)	Void Height, H_v (mm)	Void Ratio, e
Loading					
7	3.000	0.00	20.00	10.09	1.02
7	2.882	-0.12	20.12	10.21	1.03
50	3.004	0.004	20.00	10.08	1.02

100	3.200	0.20	19.80	9.89	1.00
200	3.518	0.52	19.48	9.57	0.97
400	3.930	0.93	19.07	9.16	0.92
800	4.710	1.71	18.29	8.38	0.85
1600	5.892	2.89	17.11	7.20	0.73

The values of compression index (Cc) are the slope of the plot presented in Fig.4.7. Its calculation are presented below.

The compression index, $C_c = \frac{e_{o-}e}{log^{P} - log^{Po}}$

$= \frac{0.845\_0.7257}{log^{1600} - log^{800}} = 0.4$

Coefficient of permeability

The value of obtained from the Fig.4.8 and calculated using the following equation.

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

Where: c_v= coefficient of consolidation

a_v= coefficient of compressibility

γ_w=unit weight of water

e= void ratio

For loading of 100kPa

$c_v = \frac{0.848 * \mathrm{H_d}^2}{\mathrm{t_{90}}}$, t_{90} =0.49 computed from Fig.4.9

$c_v = \frac{0.848*0.99}{0.49} = 1.71$

e=1.0

$\gamma_w = 10$

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{1.0179-0.998}{100-50}$ =3.955x10^{-4} cm^2/sec

$k = \frac{1.71*0.0003955*10}{1+1.0}$ =33.82x10^{-9}cm/sec

For loading of 200kPa

$c_v = \frac{0.848*H_d{}^2}{t_{90}}$, t_{90} =0.81 computed from Fig.4.9

$c_v = \frac{0.848*0.96}{0.81}$=1.01

e=0.97

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.998-966}{200-100}$ =3.208x10^{-4} cm^2/sec

k=$\frac{1.01*0.0003208*10}{1+0.97}$ =16.45x10^{-9}cm/sec

For loading of 400kPa

$c_v = \frac{0.848*H_d{}^2}{t_{90}}$, t_{90} =1.44 computed from fig 4.9

$c_v = \frac{0.848*0.93}{1.44}$=0.55

e=0.92

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.966-0.924}{400-200}$ =2.078x10^{-4} cm^2/sec

k=$\frac{0.55*0.0002078*10}{1+0.93}$ =5.95x10^{-9}cm/sec

For loading of 800kPa

$c_v = \frac{0.848*H_d{}^2}{t_{90}}$, t_{90} =2.25 computed from Fig.4.9

$c_v = \frac{0.848*0.87}{2.25}$=0.33

e=0.85

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.845-0.924}{800-400}$=1.97x$10^{-4}$ cm^2/sec

k=$\frac{0.33*0.000197*10}{1+0.85}$ =3.51x10^{-9}cm/sec

For loading of 1600kPa

$c_v = \frac{0.848*H_d{}^2}{t_{90}}$, t_{90} =2.86 computed from Fig.4.9

$c_v = \frac{0.848*0.78}{2.86}$=0.23

e=0.0.73

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.845-0.7257}{1600-800}$=1.491x$10^{-4}$ cm^2/sec

$$k= \frac{0.23*0.0001491*10}{1+0.73} = 1.98\text{x}10^{-9}\text{cm/sec}$$

Modulus of compressibility

Modulus of compressibility is the slope which determined from Fig.4.12. Thus:

$$E_s = \frac{d\sigma'}{ds'}$$

$$= \frac{1200-700}{0.25-0.15} = 5000 \text{ kN/}m^2$$

The result of v computed for w=1, by using Fig.4.13 is 3851kN/m^2.

For test pit 2

Sample depth……………………..3m

Data at the beginning and end of the test

Sample type ……………………....Undisturbed

Ring area…………………............19.63 cm^2

Volume of sample……………......39.27 cm^3

Mass of empty ring (M_R)…………68.7 gm

Mass of soil+ ring (M_{S+R})………...135.4gm

Specific gravity of soil(G_s)………..2.67

Mass of soil (M_s)…………………66.7gm

Mass of dry soil (M_d)……………...54.9g

Ring Diameter (D)………………....5cm

Height of sample (H_0)……………...20mm

Initial moisture content (ω)…….....22.45%

Table D-3: Dial gauge reading for each incremental loading

Dial Gauge Reading (mm)							
Square root of time		50kpa	100kpa	200kpa	400kpa	800kpa	1600kpa
	Time (min)	Dial reading (mm)	Dial reading (mm)	Dial reading (mm)	Dial reading (mm)	Dial reading (mm)	Dial reading (mm)
0	0	7.97	8.228	8.57	9.182	9.772	10.66
0.316228	0.1	8.002	8.33	8.616	9.264	10.19	10.88
0.5	0.25	8.024	8.348	8.65	9.31	10.032	10.944
0.707107	0.5	8.042	8.364	8.676	9.334	10.058	10.972
1	1	8.064	8.384	8.714	9.356	10.094	11
1.414214	2	8.084	8.406	8.744	9.382	10.134	11.052
2	4	8.1	8.432	8.78	9.404	10.188	11.276
2.828427	8	8.116	8.454	8.816	9.43	10.226	11.318
3.872983	15	8.13	8.476	8.846	9.45	10.25	11.364
5.477226	30	8.148	8.5	8.872	9.472	10.278	11.43
7.745967	60	8.166	8.516	8.952	9.492	10.378	11.496
10.95445	120	8.192	8.534	9.136	9.504	10.382	11.536
15.49193	240	8.208	8.552	9.146	9.522	10.422	11.57
21.9089	480	8.222	8.56	9.162	9.536	10.464	11.588
37.94733	1440	8.228	8.57	9.182	9.772	10.66	11.6

Compression Index, C_c

Height of solid (H_s) = $\frac{M_d}{\rho_w * A * G_s}$

Where: G_s = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w= density of water 1g/cm^3

= $\frac{54.9}{1*19.63*2.67}$ =10.47mm

Height of void (H_v) = H-Hs

=20-10.47=9.53mm

Initial void ratio (e_0) = $\frac{H_v}{H_s}$= $\frac{9.53}{10.47}$ = 0.91

Likewise calculation of void ratio are accomplished similarly and presented in Table D-4.

Table D-4: Determining the value of void ratio for TP2

Applied Pressure, P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, e
Loading					
7	8.200	0.00	20.00	9.53	0.91
7	7.970	-0.23	20.23	9.76	0.93
50	8.228	0.03	19.97	9.50	0.91
100	8.570	0.37	19.63	9.16	0.87
200	9.182	0.98	19.02	8.55	0.82
400	9.772	1.57	18.43	7.96	0.76
800	10.660	2.46	17.54	7.07	0.67
1600	11.600	3.40	16.60	6.13	0.59

The values of compression index (Cc) are the slope of the plot presented in Fig.4.7. Its calculation are presented below.

The compression index, $C_c = \frac{e_{o\_}e}{log^{P-}log^{Po}}$

$$= \frac{0.675\_0.585}{log^{1600-}log^{800}} = 0.3$$

Coefficient of permeability

The value of obtained from the Fig.4.8 and calculated using the following equation.

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

Where: c_v= coefficient of consolidation

a_v= coefficient of compressibility

γ_w=unit weight of water

e= void ratio

For loading of 100kPa

$c_v = \frac{0.848*{H_d}^2}{t_{90}}$, t_{90} =1.0 computed from Fig.4.9

$c_v = \frac{0.848*0.98}{1.0} = 0.83$

e=0.87

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.907-0.8744}{100-50}$ =6.53x10^{-4} cm^2/sec

$$k = \frac{0.83*0.000656*10}{1+0.87} = 29.0\text{x}10^{-9}\text{cm/sec}$$

For loading of 200kPa

$c_v = \frac{0.848*{H_d}^2}{t_{90}}$, t_{90} =1.5 computed from Fig.4.9

$c_v = \frac{0.848*0.93}{1.5} = 0.53$

e=0.82

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.8744-0.816}{200-100}$ =5.84x10^{-4} cm^2/sec

$$k = \frac{0.53*0.000584*10}{1+0.82} = 17.01\text{x}10^{-9}\text{cm/sec}$$

For loading of 400kPa

$c_v = \frac{0.848*{H_d}^2}{t_{90}}$, t_{90} =2.26 computed from Fig.4.9

$c_v = \frac{0.848*0.89}{2.26}$ = 0.33cm2/sec

e=0.76

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.816-0.76}{400-200}$ =2.82x10^{-4} cm^2/sec

$$k = \frac{0.33*0.000282*10}{1+0.76} = 5.3\text{x}10^{-9}\text{cm/sec}$$

For loading of 800kPa

$c_v = \frac{0.848*{H_d}^2}{t_{90}}$, t_{90} =2.6 computed from Fig.4.9

$C_v = \frac{0.848*0.81}{4}$=0.26cm2/sec

e=0.67

γ_w=10

Coefficientof consolidation is obtained from Fig.4.8, $a_v = \frac{0.76-0.675}{800-400}$=2.12x$10^{-4}$ cm^2/sec

k=$\frac{0.26*0.000212*10}{1+0.67}$ =3.3x10^{-9}cm/sec

For loading of 1600kPa

$C_v = \frac{0.848*H_d^2}{t_{90}}$, t_{90} =1.7 computed from Fig.4.9

$C_v = \frac{0.848*0.73}{1.7}$ = 0.36cm2/sec

e=0.6

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.675-0.595}{1600-800}$=1.12x$10^{-4}$ cm^2/sec

k=$\frac{0.51*0.000112*10}{1+0.59}$ =2.54x10^{-9}cm/sec

Modulus of compressibility

Modulus of compressibility is the slope which determined from Fig.4.12. Thus:

$E_s = \frac{d\sigma'}{ds'}$

$= \frac{1600-100}{0.5-0.075}$=3529 kN/$m^2$

The result of v computed for w=1, by using Fig.4.13 is 3466kN/m^2.

 # Test Pit 4

Sample depth…………………...3m

Data at the beginning and end of the test

Sample type ……………………Undisturbed

Ring area………………………..19.63 cm^2

Volume of sample………………39.27 cm^3

Mass of empty ring (M_R)………..68.7 gm

Mass of soil+ ring (M_{S+R})………142.5gm

Specific gravity of soil(G_s)………2.64

Mass of soil (M_s)………………...73.8gm

Mass of dry soil (M_d)……………56.2gm

Ring Diameter (D)……………….5cm

Height of sample (H_0)…………...20mm

Initial moisture content (ω)……...31.99%

Table D-5: Dial gauge reading for each incremental loading for TP4

Dial Gauge Reading, mm							
Square root of time	**Time (min)**	**50kpa Dial reading (mm)**	**100kpa Dial reading (mm)**	**200kpa Dial reading (mm)**	**400kpa Dial reading (mm)**	**800kpa Dial reading (mm)**	**1600kpa Dial reading (mm)**
0	0	4.172	4.432	4.706	5.188	5.712	6.416
0.316228	0.1	4.266	4.486	4.8	5.2	5.796	6.49
0.5	0.25	4.29	4.5	4.858	5.238	5.856	6.734
0.707107	0.5	4.302	4.51	4.876	5.252	5.89	6.758
1	1	4.31	4.52	4.9	5.272	5.932	6.8
1.414214	2	4.324	4.528	4.928	5.302	5.992	6.848
2	4	4.338	4.544	4.964	5.336	6.014	6.918
2.828427	8	4.352	4.604	4.994	5.378	6.046	7.01
3.872983	15	4.366	4.616	5.024	5.418	6.102	7.104
5.477226	30	4.378	4.634	5.04	5.456	6.142	7.208
7.745967	60	4.396	4.65	5.094	5.49	6.366	7.316
10.95445	120	4.404	4.666	5.122	5.52	6.386	7.326
15.49193	240	4.408	4.676	5.144	5.544	6.4	7.356
21.9089	480	4.414	4.69	5.162	5.566	6.406	7.378
37.94733	1440	4.432	4.706	5.188	5.712	6.416	7.394

Compression Index, C_c

Height of solid (H_s) = $\frac{M_d}{\rho_w * A * G_s}$ Where: G_s = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w= density of water 1g/cm^3

$= \frac{56.2}{1*19.63*2.64}$=10.83mm

Height of void (H_v) = H-Hs

=20-10.83=9.17mm

Initial void ratio (e_0) = $\frac{H_v}{H_s} = \frac{9.53}{10.83}$=0.848

Likewise calculation of void ratio are accomplished similarly and presented in Table D-6.

Table D-6: Determining the value of void ratio for TP4

Applied Pressure, P (kPa)	Final Dial, Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, e
Loading					
7	4.200	0.00	20.00	9.17	0.848
7	4.172	-0.03	20.03	9.20	0.850
50	4.432	0.23	19.77	8.94	0.83
100	4.706	0.51	19.49	8.67	0.80
200	5.188	0.99	19.01	8.19	0.76
400	5.712	1.51	18.49	7.66	0.71
800	6.416	2.22	17.78	6.96	0.64
1600	7.394	3.19	16.81	5.98	0.55

The values of compression index (Cc) are the slope of the plot presented in Fig.4.7. Its calculation are presented below.

The compression index, $C_c = \frac{e_{o-}e}{log^{P-}log^{Po}}$

$$= \frac{0.643\_0.552}{log^{1600-}log^{800}} = 0.3$$

Coefficient of permeability

The value of obtained from the Fig.4.8 and calculated using the following equation.

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

Where: c_v= coefficient of consolidation

a_v= coefficient of compressibility

γ_w=unit weight of water

e= void ratio

For loading of 100kPa

$c_v = \frac{0.848 * {H_d}^2}{t_{90}}$, t_{90} =0.67 computed from Fig.4.9

$c_v = \frac{0.848*0.96}{0.673} = 1.21$

e=0.8

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.826-0.801}{100-50}$ =5.062x10^{-4} cm^2/sec

$$k = \frac{1.21*0.0005062*10}{1+0.8} = 34.03x10^{-9} cm/sec$$

For loading of 200kPa

$c_v = \frac{0.848 * {H_d}^2}{t_{90}}$, t_{90} =2.56 computed from Fig.4.9

$c_v = \frac{0.848*0.93}{2.56} = 0.31$

e=0.756≈0.76

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.801-0.756}{200-100}$ =4.452x10^{-4} cm^2/sec

$$k = \frac{0.31*0.0004452*10}{1+0.76} = 7.84x10^{-9} cm/sec$$

For loading of 400kPa

$c_v = \frac{0.848 * {H_d}^2}{t_{90}}$, t_{90} =2.25 computed from Fig.4.9

$c_v = \frac{0.848 * \mathbf{0.88}}{2.25}$=0.33

e=0.708≈0.71

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.756-0.708}{400-200}$ =2.42x10^{-4} cm^2/sec

k=$\frac{0.33*0.000242*10}{1+0.71}$ =4.67x10^{-9}cm/sec

For loading of 800kPa

$c_v = \frac{0.848 * {H_d}^2}{t_{90}}$, t_{90} =4 computed from Fig.4.9

$c_v = \frac{0.848*0.82}{4}$ = 0.17

e=0.64

γ_w=10

Coefficientof consolidation is obtained from Fig.4.8, $a_v = \frac{0.708-0.643}{800-400}$=1.626x$10^{-4}$ cm^2/sec

k=$\frac{0.17*0.0001626*10}{1+0.64}$ =1.68x10^{-9}cm/sec

For loading of 1600kPa

$c_v = \frac{0.848 * {H_d}^2}{t_{90}}$, t_{90} =1.69 computed from Fig.4.9

$c_v = \frac{0.848*0.75}{1.69}$=0.38

e=0.55

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.643-0.552}{1600-800}$=1.129x$10^{-4}$ cm^2/sec

k=$\frac{0.38*0.0001129*10}{1+0.55}$ =2.77x10^{-9}cm/sec

Modulus of compressibility

Modulus of compressibility is the slope which determined from Fig.4.12. Thus:

$E_s = \frac{d\sigma'}{ds'}$

$$=\frac{1600-700}{0.5-0.25}=3600 \text{ kN}/m^2$$

The result of v computed for w=1, by using Fig.4.13 is 3648kN/m^2.

 # Test Pit 10

Sample depth…………………...3m

Data at the beginning and end of the test

Sample type ……………………Undisturbed

Ring area………………………..19.63 cm^2

Volume of sample………………39.27 cm^3

Mass of empty ring (M_R)………..68.7 gm

Mass of soil+ ring (M_{S+R})……….139.1gm

Specific gravity of soil(G_s)………2.7

Mass of soil (M_s)………………...70.4gm

Mass of dry soil (M_d)……………51gm

Ring Diameter (D)……………….5cm

Height of sample (H_0)…………...20mm

Initial moisture content (ω)……...36.67%

Table D-7: Dial gauge reading for each incremental loading for TP10

Dial Gauge Reading, mm							
Square root of time	**Time (min)**	**50kpa Dial reading (mm)**	**100kpa Dial reading (mm)**	**200kpa Dial reading (mm)**	**400kpa Dial reading (mm)**	**800kpa Dial reading (mm)**	**1600kpa Dial reading (mm)**
0	0	3.500	3.722	3.904	4.25	4.75	5.412
0.316228	0.1	3.548	3.73	3.97	4.304	4.79	5.486
0.5	0.25	3.554	3.736	3.994	4.314	4.814	5.49
0.707107	0.5	3.558	3.74	4	4.322	4.824	5.502
1	1	3.566	3.748	4.008	4.332	4.836	5.51

1.414214	2	3.576	3.756	4.016	4.346	4.85	5.526
2	4	3.586	3.762	4.028	4.368	4.872	5.546
2.828427	8	3.596	3.776	4.042	4.388	4.898	5.586
3.872983	15	3.608	3.8	4.062	4.416	4.952	5.618
5.477226	30	3.624	3.814	4.088	4.458	4.98	5.676
7.745967	60	3.634	3.83	4.12	4.514	5.096	5.764
10.95445	120	3.646	3.844	4.158	4.586	5.17	5.796
15.49193	240	3.66	3.866	4.196	4.662	5.284	6.03
21.9089	480	3.672	3.884	4.228	4.718	5.356	6.072
37.94733	1440	3.722	3.904	4.25	4.75	5.412	6.16

Compression Index, C_c

Height of solid (H_s) = $\frac{M_d}{\rho_w * A * G_s}$

Where: G_s = Specific gravity of the solids

A = Area of the consolidation ring

ρ_w= density of water 1g/cm^3

$= \frac{51}{1*19.63*2.7}$=9.63mm

Height of void (H_v) = H-Hs

=20-9.63=10.37mm

Initial void ratio (e_0) = $\frac{H_v}{H_s} = \frac{10.37}{9.63}$=1.08

Likewise calculation of void ratio are accomplished similarly and presented in Table D-8.

Table D-8: Determining the value of void ratio for TP10.

Applied Pressure, P (kPa)	Final Dial, Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height, H_v (mm)	Void Ratio, e
Loading					
7	4.00	0.00	20.00	10.37	1.08

7	3.5	-0.5	20.5	10.87	1.13
50	3.722	-0.28	20.28	10.65	1.11
100	3.904	-0.1	20.1	10.47	1.09
200	4.25	0.25	19.75	10.12	1.05
400	4.75	0.75	19.25	9.62	1
800	5.412	1.41	18.59	8.96	0.93
1600	6.16	2.16	17.84	8.21	0.85

The values of compression index (Cc) are the slope of the plot presented in Fig.4.7. Its calculation are presented below.

The compression index, $C_c = \frac{e_{o} - e}{log^{P} - log^{Po}}$

$$= \frac{0.93 - 0.85}{log^{1600} - log^{800}} = 0.26$$

<u>Coefficient of permeability</u>

The value of obtained from the Fig.4.8 and calculated using the following equation.

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

Where: c_v= coefficient of consolidation

a_v= coefficient of compressibility

γ_w=unit weight of water

e= void ratio

For loading of 100kPa

$c_v = \frac{0.848 * \mathrm{H_d}^2}{\mathrm{t_{90}}}$, t_{90} =0.64 computed from Fig.4.9

$c_v = \frac{0.848 * 1.02}{0.64} = 1.35$

e=1.09

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{1.106 - 1.087}{100 - 50}$ =37.8x10^{-4} cm^2/sec

$$k=\frac{1.35*0.000378*10}{1+1.087}=24.42\text{x}10^{-9}\text{cm/sec}$$

For loading of 200kPa

$c_v=\frac{0.848*\text{H}_\text{d}{}^2}{\text{t}_{90}}$, t_{90} =1.21 computed from Fig.4.9

$c_v=\frac{0.848*0.992}{1.21}$=0.7

e=1.051

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v=\frac{1.087-1.051}{200-100}$ =3.594x10^{-4} cm^2/sec

$$k=\frac{0.7*0.0003594*10}{1+1.051}=12.3\text{x}10^{-9}\text{cm/sec}$$

For loading of 400kPa

$c_v=\frac{0.848*\text{H}_\text{d}{}^2}{\text{t}_{90}}$, t_{90} =2.56 computed from Fig.4.9

$c_v=\frac{0.848*0.951}{2.56}$=0.31

e=0.999≈1.0

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v=\frac{1.051-0.999}{400-200}$ =2.596x10^{-4} cm^2/sec

$$k=\frac{0.31*0.0002596*10}{1+0.999}=4.02\text{x}10^{-9}\text{cm/sec}$$

For loading of 800kPa

$c_v=\frac{0.848*\text{H}_\text{d}{}^2}{\text{t}_{90}}$, t_{90} =3.24 computed from Fig.4.9

$c_v=\frac{0.848*0.89}{3.24}$ = 0.23

e=0.93

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v=\frac{0.999-0.931}{800-400}$=1.719x$10^{-4}$ cm^2/sec

$$k=\frac{0.23*0.0001719*10}{1+0.93}=2.05\text{x}10^{-9}\text{cm/sec}$$

For loading of 1600kPa

$$c_v = \frac{0.848 * H_d^2}{t_{90}},\ t_{90} = 3.24 \text{ computed from fig 4.9}$$

$$c_v = \frac{0.848 * 0.829}{3.24} = 0.22$$

e=0.85

γ_w=10

Coefficient of consolidation is obtained from Fig.4.8, $a_v = \frac{0.931 - 0.853}{1600 - 800}$=0.971x$10^{-4}$ cm^2/sec

$$k = \frac{0.22 * 0.0000971 * 10}{1 + 0.853} = 1.16\text{x}10^{-9}\text{cm/sec}$$

<u>Modulus of compressibility</u>

Modulus of compressibility is the slope which determined from Fig.4.12. Thus:

$$E_s = \frac{d\sigma'}{ds'}$$

$$= \frac{1600 - 300}{0.25 - 0.05} = 6500 \text{ kN}/m^2$$

The result of v computed for w=1, by using Fig.4.13 is 5546kN/m^2.

Graphs for the determination of preconsolidation pressure

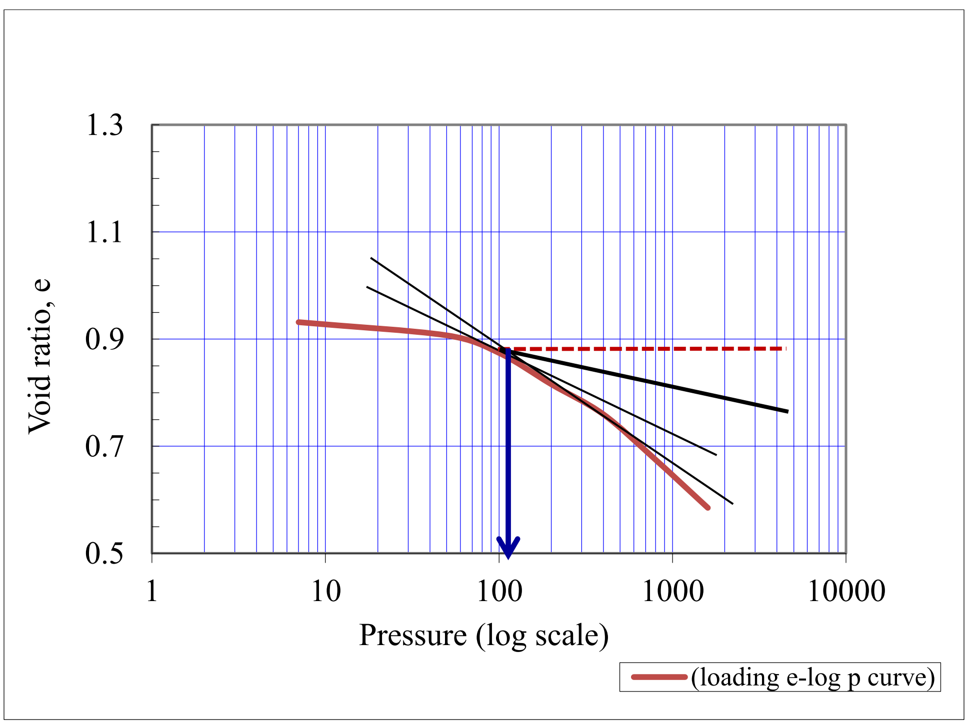

Fig.D-1 void ratio Vs log pressure curve to determine Pc for TP2, 3m

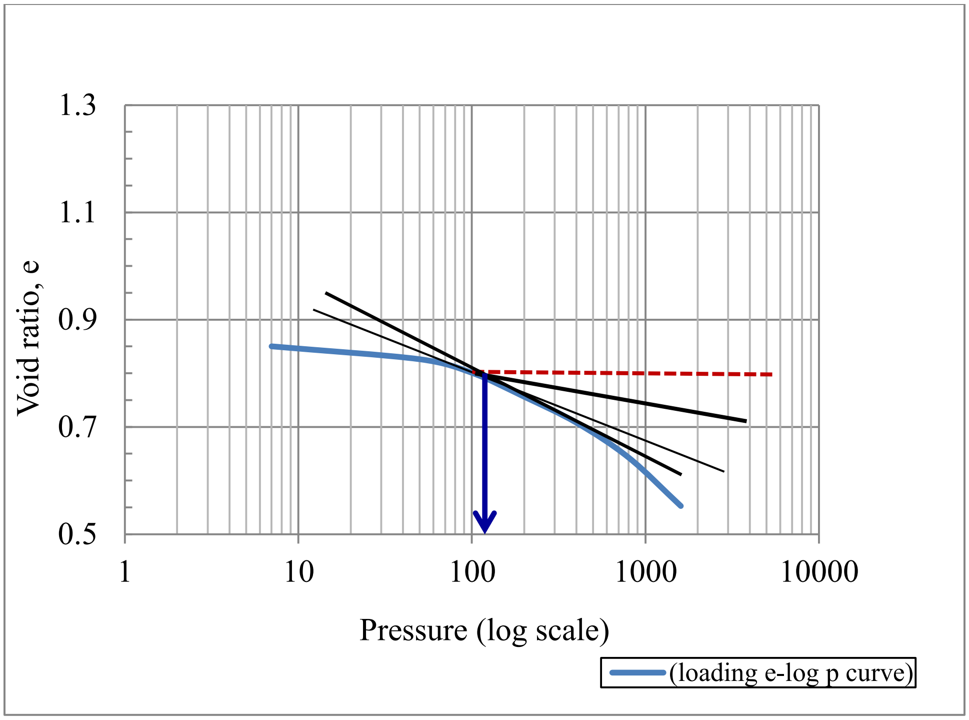

Fig.D-2 Void ratio Vs log pressure curve to determine Pc for TP4, 3m.

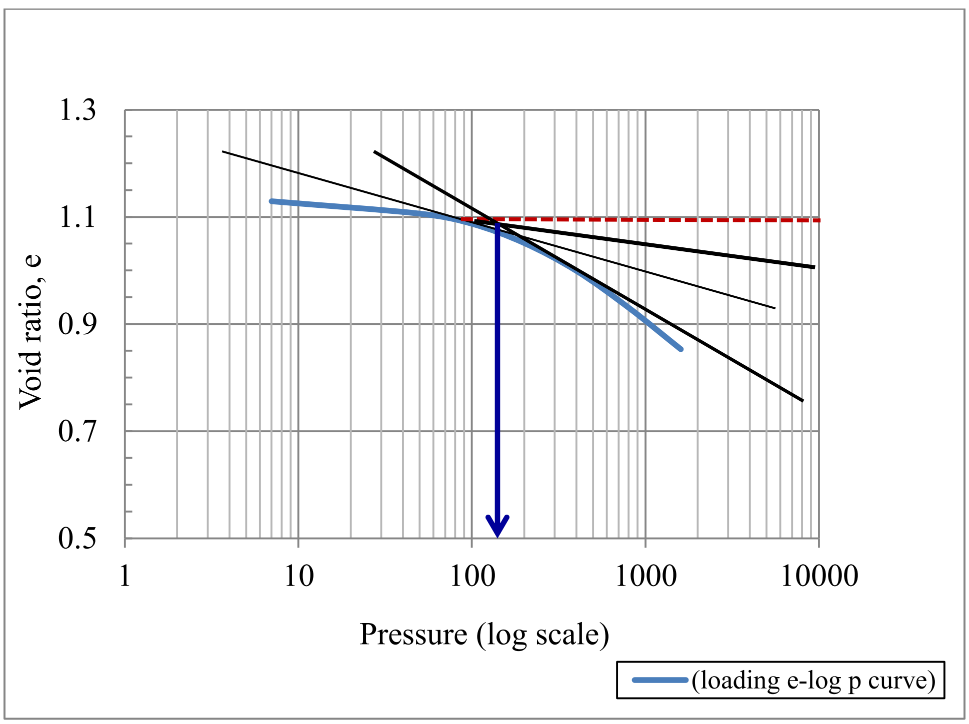

Fig.D-3 void ratio Vs log pressure curve to determine P_c for TP10, 3m