ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF THECHNOLOGY

DEPARPMENT OF CIVIL ENGINEERING

INVESTIGATION INTO SOME OF THE ENGINEERING PROPERTIES OF SOILS IN DEBRE MARKOS TOWN

BY: ADEM EBRAHIM

"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 (Geotechniques)"

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INVESTIGATION INTO SOME OF THE ENGINEERING PROPERTIES OF SOILS IN DEBRE MARKOS TOWN, ETHIOPIA

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Symbols and Abbreviations

AASHTO	-	American Association of State Highway and Transportation Officials
ASTM	-	American Society for Testing Materials standard
C_c	-	Compression index
CH	–	Highly plastic clay
CL	–	Low plastic clay
C_r	-	Recompression index
C_v	-	Coefficient of consolidation
e	-	Void ratio
E_s	-	Modulus of compressibility
K	-	Modulus of Permeability
LL	-	Liquid limit
MDD	–	Maximum dry density
MH	–	Inorganic Elastic silt
ML	–	Inorganic Silt
NMC	-	Natural moisture content
OMC	-	Optimum moisture content
OCR	-	Over-consolidation ratio
P_c	–	Pre-consolidation pressure
P_o	-	Over burden pressure
PI (I_p)	-	Plastic Index
PL	-	Plastic limit
TP	–	Test pit
USCS	-	Unified Soil Classification System
γ_d	-	Dry unit weight
γ_w	-	Wet unit weight

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Abstract

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

Investigating the engineering properties of soil found in Debre Markos town is the objective of this research. Because now a days Debre Markos is under development but there is no investigation work for most of the undertaking construction. To achieve the objective nine representative test pits were excavated to take representative disturbed and undisturbed samples at the average depth of around 1.5m and 3m.

Study of engineering properties such as the grain size distributions, consistency limit, compaction, UCS and one dimensional consolidation parameters is important to understand the behavior of soils.

The grain size analysis test result showed that the dominant proportion of soil particle in the research area is clay, which have clay content ranging from 50 % – 73% , silt faction 15.88% – 40.21 % and sand fraction 0.36% - 13.28%.

The result of Atterberg Limit test on the soil in the research area showed a liquid limit ranging from 45% – 68%, plastic limit ranges from 18% – 38% and plastic index from 15% – 40%.

The Specific Gravity ranges from 2.69 to 2.84. The Free swell test result on the samples collected shows range from 30 – 50% except TP 9 which was 180% because a sample on TP9 is expansive soil.

From the compaction test results the maximum dry density (MDD) of Debre Markos soil ranges from 13.5kN/m^3 to 14.2kN/m^3 and the optimum moisture content ranges 28% to 36%.

The values of unconfined compressive strength (UCS) range from 320.21kPa to 382.96kPa at natural moisture content of 41.2% to 26.7% respectively.

According to the AASHTO Classification System the usual types of significant constituent materials of the Debre Markos soil is clayey soil. In Unified Soil Classification System, the soil is categorized as silty clay.

Finally one-dimensional consolidation test was done. It showed that the area under investigation is over consolidated in its natural state, have compression index ranging from 0.14 to 0.32 recompression indexes, C_r, from 0.003 to 0.07, coefficient of permeability ranging from 10^{-7} to 10^{-9} cm/sec.

1. INTRODUCTION

PROBLEM BACKGROUND

A detailed and comprehensive geotechnical investigation is an essential requirement for the Design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, high ways, etc. requires adequate knowledge of sub surface conditions at the sites of the proposed structures. Many damages to buildings, roads and other structures founded on soils are mainly due to the lack of proper investigation of substructure condition [11].

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

Therefore, in a country like Ethiopia which is developing at a high growth rate, geological investigation on engineering properties of soil is very essential. These data are very important for geotechnical engineers while designing foundation, pavement, retaining structures, etc. in future construction works.

Debre Markos, the capital of East Gojjam Administrative Zone, located in the North West part of Ethiopia, at a distance of 300Kms from Addis Ababa and 265 kms to the capital of Amhara Nation Regional State Bahir Dar respectively. Debre Markos is 10^0-21' longitude North and 37^0-43' Latitude East. Its elevation is estimated to be 2420mts above sea level. During its founding the total extent of the area was 272 hectors today this extent increased 23 fold and reached 6160 hectors. As regards the climate of Debre Markos, the annual average temperature is 18.5^0C and its annual average rainfall is 1380mm [8]. Debre Markos town is one of the area in which lot of construction works is undertaken at present.

Engineering properties such as mechanical properties, index properties and consolidation characteristics of the Debre Markos soil is not studied well yet. Therefore, investigating the engineering properties, identifying the characteristic of the soil and preparing soil map of the city is very important for construction works as well as for further studies in the future as an input. No systematic soil investigation has been carried out on this town prior to this work.

OBJECTIVE OF THE STUDY AREA

- To investigate some of the engineering properties such as index properties (Grain Size distribution, Atterburg limits, Swelling Potential, Specific gravity, Moisture content), Unconfined Compression Strength (UCS) and consolidation characteristics of soil in Debre Markos town.
- To determine the range of the value of the index properties of soil in different parts of the city.
- To determine the consolidation characteristics of the soil in the city.
- To prepare soil map to the city.

METHODOLOGY

Nine representative sampling areas, where the soil visually shows different characteristic were selected. From the selected areas representative disturbed and undisturbed soil samples were collected from open pits by direct excavation manually at the average depths of 1.5m and 3m.Then these samples were taken to the laboratory for taking care for the undisturbed samples.

The soil samples recovered from the site will be subjected to the following tests:

✓ Specific gravity Test
✓ Moisture content Test
✓ Consistency(Atterberg) Limit Tests
  - Plastic Limit
  - Liquid Limit
✓ Grain Size distribution Test

- ✓ Free Swell Test
- ✓ UCS (Unconfined Compressive Strength Test)
- ✓ One dimensional consolidation test

The tests were done according to ASTM standard.

2. LITERATURE REVIEW

2.1. GENERAL

A bulk soil, as it exists in nature, is more or less randomly assembled of soil particles, water and air. The properties of soils are complex and variable. Every civil engineering work involves the determination of soil type and its associated engineering application; certain properties are more significant than others. The common problems faced by civil engineers are related to bearing capacity and compressibility of soil and seepage through the soil. The possible solution to these problems is arrived at based on the study of the physical and index properties of the soil [6].

Soil index properties are used extensively by engineers to discriminate between the different kinds of soil within a broad category, e.g. clay will exhibit a wide range of engineering properties depending upon its composition. Classification tests to determine index properties will provide the engineer with valuable information when the results are compared against empirical data relative to the index properties determined [9].

Soil is a heterogeneous material. The properties and characteristics of soils vary from place to place laterally or vertically. The tests required for determination of engineering properties are generally elaborate and time consuming. Sometimes the geotechnical engineer is interested to have some rough assessment of the engineering properties without conducting elaborate tests. This is possible if index properties are determined. The properties of soils which are not of primary interest to the geotechnical engineer but which are indicative of the engineering properties are called index properties [1]. According to Dr. B.C. Punimia, et al the index properties of soils are water content, specific gravity, particle size distribution, consistency limits, in-situ density, free- swell and density index.

In nature, Soils occur in a large variety. However, soils exhibiting similar behavior can be grouped together to form a particular group. Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail, and precision required with engineering properties tests. These simplified tests provide indirect information about the engineering properties of soils and are, therefore, called index tests [20].

Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include particle size and distribution, specific gravity, and water content. Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limit, and activity. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [11].

2.2. SOIL FORMATION AND SOIL DEPOSITS

Soils are formed by the process of weathering of the parent rock. The weathering of the rocks might be by mechanical disintegration, and/or chemical decomposition. The properties of the soil materials depend upon the properties of the rocks from which they are derived [16].

The variety of soil materials encountered in engineering problems is almost limitless, ranging from hard, dense, large pieces of rock through to gravel, sand, silt, and clay to organic deposits of soft compressible peat. To compound the complexity, all of these materials may occur over a range of densities and water contents. At any given site, a number of different soil types may be present, and the composition may vary over intervals of a little as a few inches [6].

It has long been appreciated that the engineering classification of soils is greatly facilitated by taking into account the soil-forming processes by which nature has created the various types of soil conditions. Similar combinations of soil-forming processes in different parts of the world have been found to lead to materials of similar index properties and similar engineering characteristics [6]. The main factors affecting the formations of soil are: Parent materials i.e. geology of the area, topography and drainage, climate and vegetation cover.

2.2.1. Parent materials

There are two main variables in parent materials that affect soils: grain size and composition. Grain size is the main determinant of soil texture. Texture influences the soil structure, consistency, cation exchange capacity, profile drainage, moisture retaining capacity and organic content [6].

2.2.2. Topography and Drainage

Topography has a major influence on drainage characteristics which in turn is known to have major effect on soil mineralogy. Its control over soil properties is particularly strong in tropical environment reflecting the importance of lateral movement of water and soil materials [6].

2.2.3. Climate

Climate is the principal factor governing the rate and type of soil formation. The two important components of climate are the amount and distribution of precipitation, and temperature. The temperature variable is adequately represented by mean annual temperature, which doesn't differ greatly from the nearly constant temperature in the lower part of the regolith. According to Van's Hoff's principle the velocity of a chemical reaction increases by a factor of 2 or 3 for every 10 °c rise of temperature [6].

The two main rain fall parameters most widely available are the mean annual total and the length of the dry season. The amount and distribution of precipitation affects the availability of moisture and the relative humidity of the soil atmosphere; it influences the concentration or chemical activities of solutions in the system [6].

2.3. GENERAL TYPES OF SOILS

According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials [16].

2.3.1. Soil particle size and shape

The size of particles may range from gravel to the finest size possible. Their characteristics vary with the size. Soil particles coarser than 0.075 mm are visible to the naked eye or may be examined by means of a hand lens. They constitute the coarser fractions of the soils. The coarser fractions of soils consist of gravel and sand. The individual particles of gravel, which are fragments of rock, are composed of one or more minerals, whereas sand grains contain mostly one mineral which is usually quartz. The individual grains of gravel and sand may be angular, sub angular, sub-rounded, rounded or well-rounded. Gravel may contain grains which may be flat. Some sands contain a fairly high percentage of mica flakes that give them the property of elasticity. Silt and clay constitute the finer fractions of the soil. Any one grain of this fraction generally consists of only one mineral. The particles may be angular, flake-shaped or sometimes needle-like [16].

2.3.2. Soil mineralogical composition [6]

Mineral particles are inorganic materials derived from rocks and minerals. They are extremely variable in size and composition.
Primary minerals: present in original rock from which soil is formed. These occur predominantly in sand and silt fractions, and are weathering resistant (quartz, feldspars).

Secondary minerals: formed by decomposition of primary minerals, and their subsequent weathering and recomposition into new ones (clay minerals). Humus or organic matter (decomposed organic materials).

3. DESCRIPTION OF THE STUDY AREA

GENERAL

Before one and half centuries ago, in 1853, Dejazmach Tedla Gualu governor of Gojjam found Menkorer, presently known by Debre Markos: He ruled Menkorer for nearly three decades (1853-1881). In 1881 the first Church-Saint Markos was built in Menkorer. Just a year after and onwards, the town got a name Debre Markos after the church of St Markos. [8]
The former Menkorer, the present Debre Markos, became the historical, administration and commerce center of Gojjam for long period of time. The city did not show any significant development till 1997 EC. At present the city has shown some progress. [10]

Debre Markos with twelve kebeles and one woreda is one of the oldest historical medium towns of Ethiopia. It is found 300 kilometers Northwest of Addis Ababa and 265 kilometers Southeast of the Amhara National Regional State capital city-Bahir Dar. The geographical coordinates of the town are $10^{o}\ 21^{1}$ latitude north and $37^{o}43^{1}$ longitude east. Situated at 2420 meters above sea level, the weather condition, most of the time is, 'Woinadega'. The town enjoys a tropical climate with a mean annual rainfall of 1308 mm, temperature 16^{o}c, while the maximum and minimum recorded temperature being 24^{o}c and 4^{o}c respectively. [18]

3.2. HYDROGEOLOGY

3.2.1. Climate

The mean annual rainfall at Debre Markos town is 1308 mm. The main rainy season is from June to October and most of the annual rainfall is precipitating in these four months time. The second annual rainfall precipitates from March to May in three months time. The remaining months are relatively dry (Figure 3.4) [8].

The maximum monthly temperature is recorded in February to May which is more than 25 ^{o}C and the lowest maximum monthly average temperature is recorded June to October which is 17 ^{o}C to 21 ^{o}C (Figure 3.5) [8].

3.2.2. Geomorphology

Debre Markos town is located on a plateau of northwestern highlands of Ethiopia. The plateau where the town is built is surrounded by Abay River's deep gorge on the eastern and southern sides and western lowlands on the western side. The general elevation of the region is over 2,000 m.a.s.l. and Choke volcanic mountain that rises to over 4,000 m.a.s.l. is situated on the plateau.

The surroundings of Debre Markos town are undulating hills and valleys but between the hills there area wide marshy plane areas. Some of these planes are over 10 km. across. These plains form swamps and water logged areas during the rainy months [8].

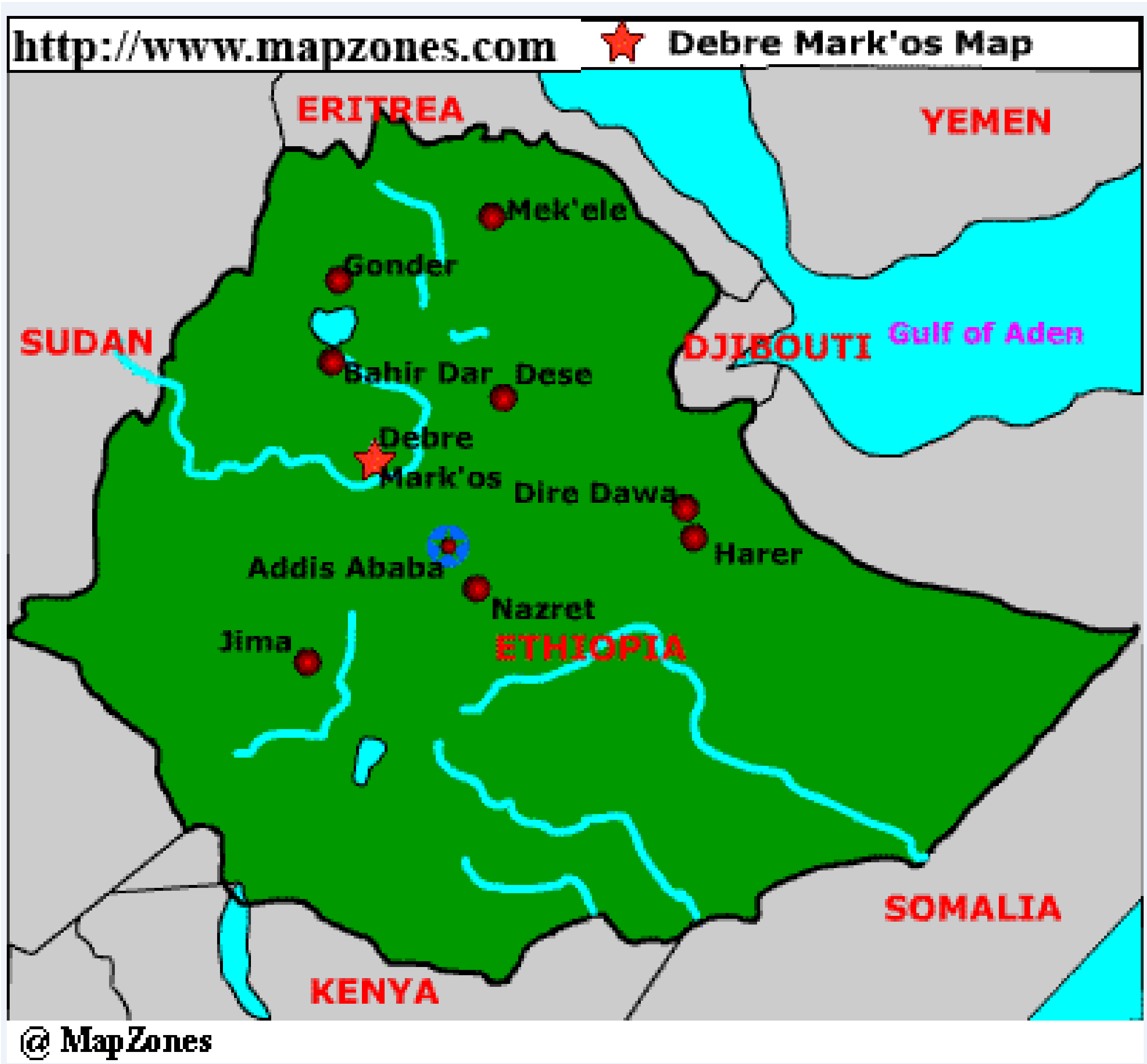


Fig 3.1 Location of the research area on the map of Ethiopia

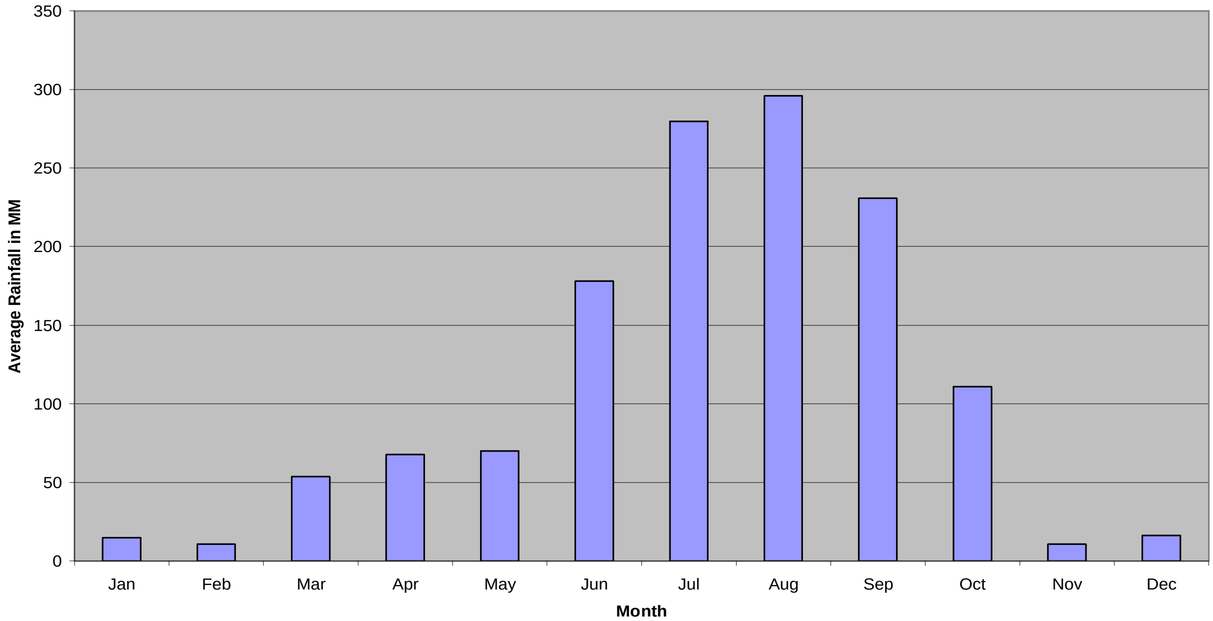


Figure 3.2 Average annual rain fall of Debre Markos 1998-2007 E.C (Source Ethiopian Metrological Agency)

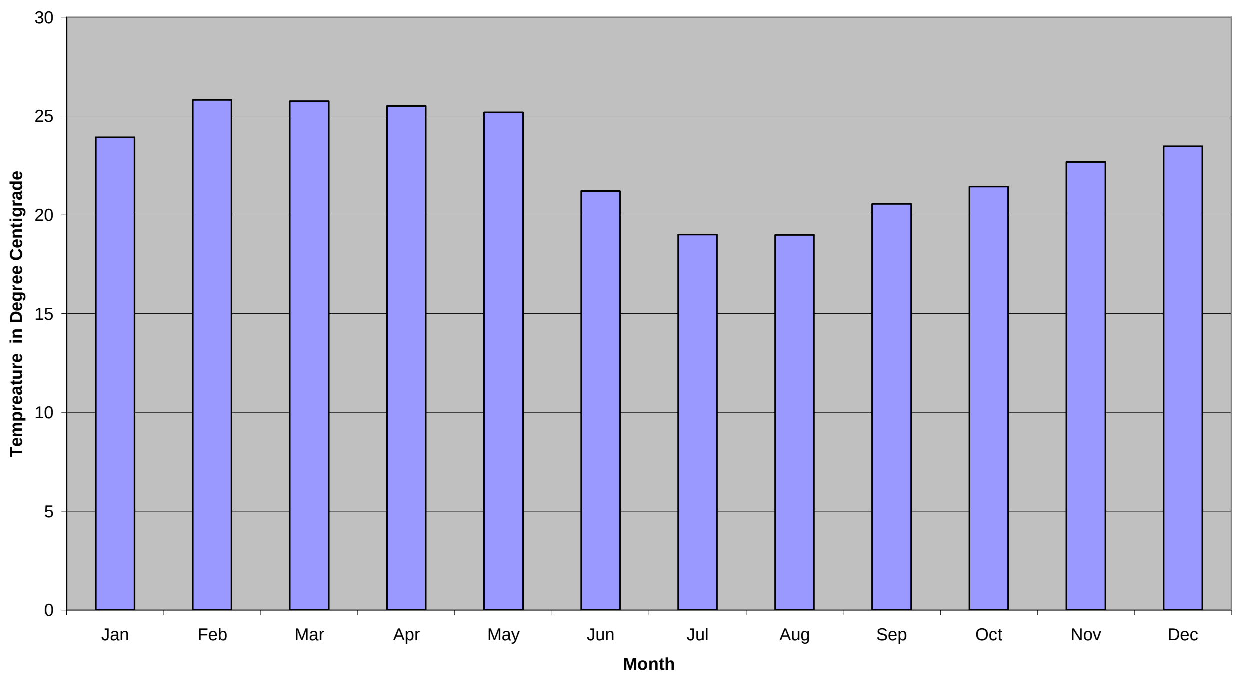


Figure 3.3 Average annual temperature of Debre Markos town 1999-2007 E.C (Source Ethiopian Metrological Agency)

4. IN-SITU PROPERTIES AND LABORATORY TESTS RESULTS

4.1. IN-SITU PROPERTIES

4.1.1. Identification of Soil in the Study Area

Before selecting sampling areas, visual site investigation and information from resident, and construction firms were collected in order to consider the different soil types and to take sample evenly. Accordingly, nine sampling areas (Fig. 4.1) were selected from different locations of the town. Pits were excavated to a maximum depth of three meters. Both disturbed and undisturbed soil samples were collected. In the field visual soil description was made and samples were collected for laboratory testing.

The global coordinates of sampling location i.e. northing, easting and elevations are shown in Table 4.1.

Table 4.1 Global coordinates of sampling areas

Test Pit	Location	Northing	Easting	Elevation (m)
TP-1	Amist kilo	1144599	0361130	2481
TP-2	Kebt gebeya	1142830	0361391	2437
TP-3	Gudukasa1	1142661	0361329	2438
TP-4	Gudukasa2	1142661	0361348	2435
TP-5	Gebeya	1142891	0361228	2455
TP-6	Feresbet/oilybia	1142697	0360802	2460
TP-7	Hamle 30	1142713	0361172	2472
TP-8	Dibza	1142412	0360965	2469
TP-9	DMU			

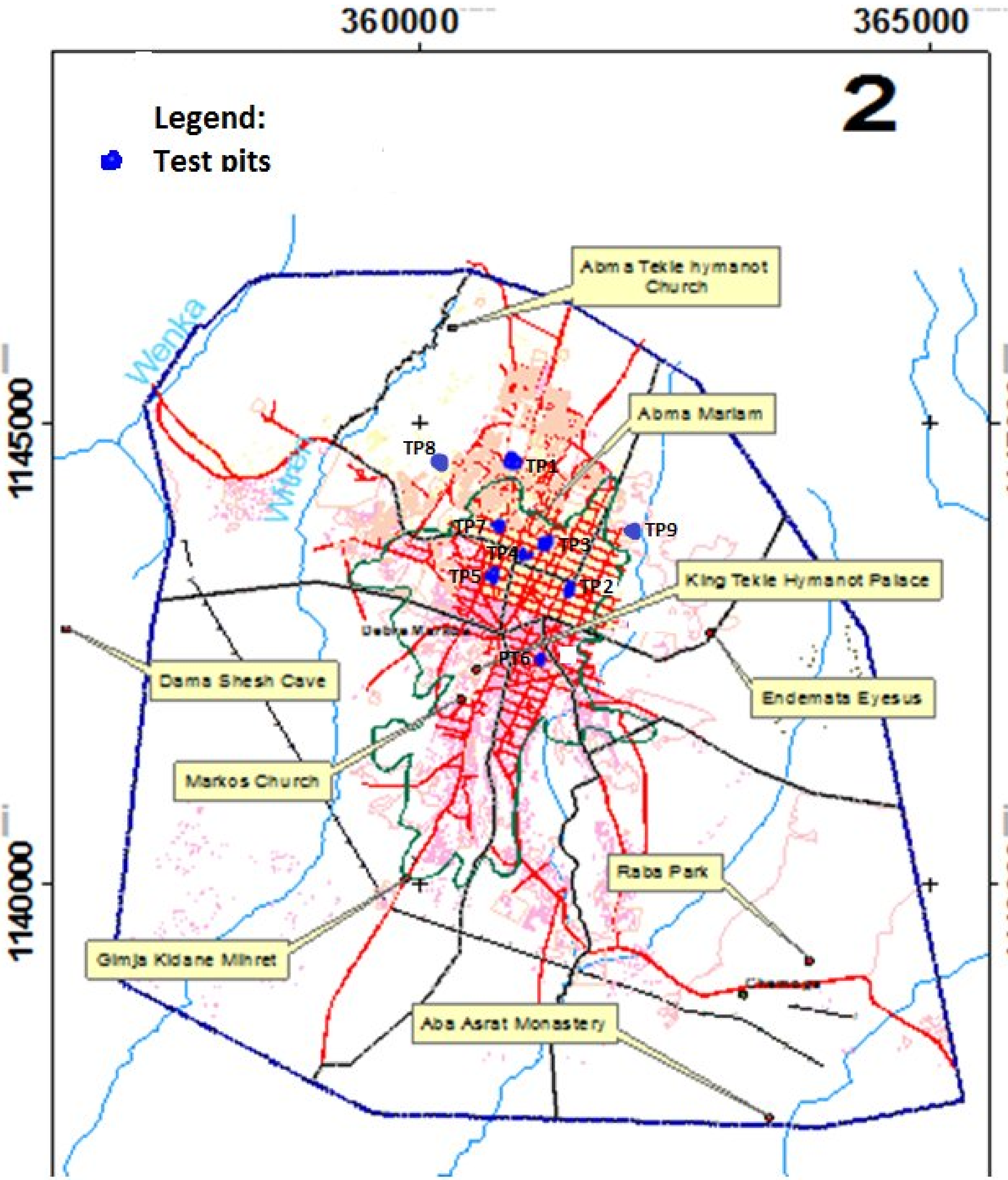


Fig. 4.1 Location of sampling areas shown on map of Debre Markos town

The global coordinates of sampling location i.e. northing, easting and elevations are shown in Table 4.1.

4.2. INDEX PROPERTIES

4.2.1. General

A bulk soil, as it exists in nature, is more or less randomly assembled of soil particles, water and air. The properties of soils are complex and variable. Every civil engineering work involves the determination of soil type and its associated engineering application; certain properties are more significant than others. The common problems faced by civil engineers are related to bearing capacity and compressibility of soil and seepage through the soil. The possible solution to these problems is arrived at based on the study of the physical and index properties of the soil [6].

Basic soil properties and parameters can be subdivided into physical, index, and engineering categories. Physical soil properties include particle size and distribution, specific gravity, and water content. Index parameters of cohesive soils include liquid limit, plastic limit, shrinkage limit, and activity. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties [11].

4.2.2. Specific gravity

Particle density or specific gravity is a measure of the actual particles which make up the soil mass and is defined as the ratio of the mass of the particles to the mass of the water they displace. Knowledge of the particle density is essential in relation to other soil tests. It is used when calculating porosity and voids ratio and is particularly important when compaction and consolidation properties are being investigated. The majority of apparatus used for the various tests is general laboratory equipment [9].

The specific gravity of the minerals affects the specific gravity of soils derived from them. The specific gravity of most rock and soil forming minerals varies from 2.50 (some Feldspars) and 2.65 (Quartz) to 3.5 (Augite or Olivine). Gypsum has a smaller value of 2.3 and salt (NaCl) has 2.1. Some iron minerals may have higher values, for instance, Magnetite has 5.2 [16].

For the study area the specific gravity of the samples is determined using ASTM D 854-00(ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer) and the result is given in Table 4.2. In the study area the specific gravity of soil ranges from 2.69 to 2.84. As we can see the specific gravity of the soil is relatively high. The higher value of Specific Gravity is indicative of higher Iron content.

Table 4.2 Specific Gravity of the Soil of the Study Area

Serial No	Designation	Depth(m)	Specific Gravity
1	TP-1-1	1.5	2.81
2	TP-1-2	3	2.75
3	TP-2-1	1.5	2.81
4	TP-2-2	3	2.84
5	TP-3-1	1.5	2.69
6	TP-3-2	3	2.84
7	TP-4-1	1.5	2.80
8	TP-4-2	3	2.83
9	TP-5-1	1.5	2.75
10	TP-5-2	3	2.78
11	TP-6-1	1.5	2.81
12	TP-6-2	3	2.84
13	TP-7-1	1.5	2.78
14	TP-7-2	3	2.82
15	TP-8-1	1.5	2.70
16	TP-8-2	3	2.74
17	TP-9-1	1.5	2.79
18	TP-9-2	3	2.81

4.2.3. Grain-Size Distribution of soil

4.2.3.1. General

The analysis of soils by particle size provides a useful engineering classification system from which a considerable amount of empirical data can be obtained.

Two separate and different procedures are used. Sieving is used for gravel and sand size particles and sedimentation procedures are used for the finer soils. For soil containing a range of coarse and fine particles it is usual to employ a composite test of sieving and sedimentation procedures [9]. Figure 4.2 is a general grain size distribution curve.

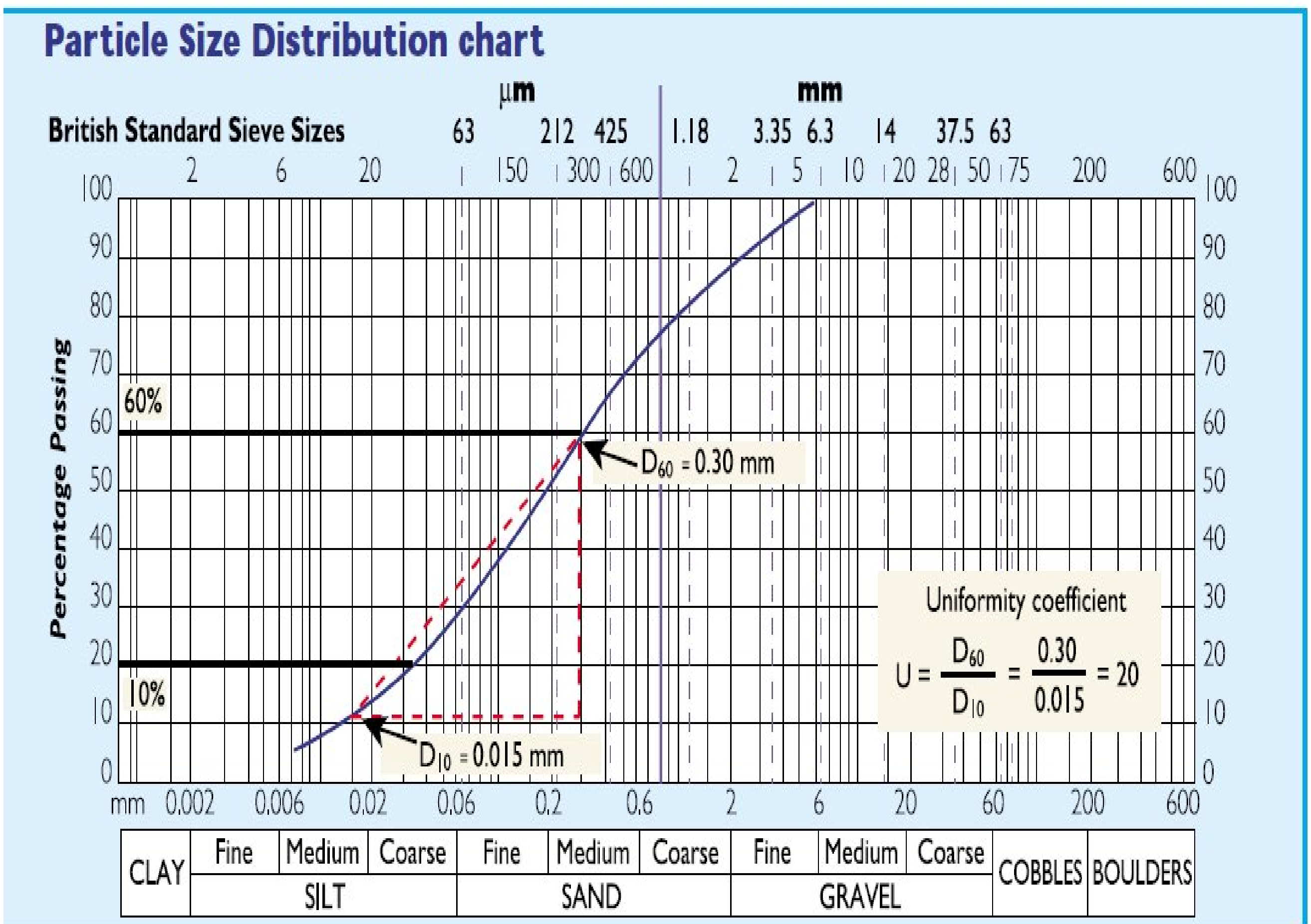


Fig 4.2 General grain size distribution curve

The analysis of soils by particle size provides a useful engineering classification system from which a considerable amount of empirical data can be obtained.

The particle size distribution of a soil (also called a gradation curve) is primarily used for classification purposes. The distribution of particle sizes larger then 0.075 mm (retained on the No. 200 sieve) is determined by sieving, while distribution of particles sizes smaller then 0.075 mm is determined by sedimentation process using a hydrometer.

The size of the sample (i.e., the amount of soil) will depend on the maximum size of the particles present in the sample itself, according to the following table [12].

Table 4.3 Size of the sample (i.e., the amount of soil) [12]

Nominal diameter of largest particles in.(mm)	Approximate Minimum Mass of Portion G
3/8 (9.5)	500
3/4 (19.0)	1000
1 (25.4)	2000
1.5 (38.1)	3000
2 (50.8)	4000
3 (76.2)	5000

For a basic understanding of the nature of soil, the distribution of the grain size present in a given soil mass must be known. The distribution of different grain sizes affects the engineering properties of soil. Grain size analysis provides the grain size distribution required in classifying the soil. Grain size Analysis test is used to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles. The test method covers the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 75µm (retained on the No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75µm is determined by a sedimentation process, using a hydrometer to secure the necessary data [2].

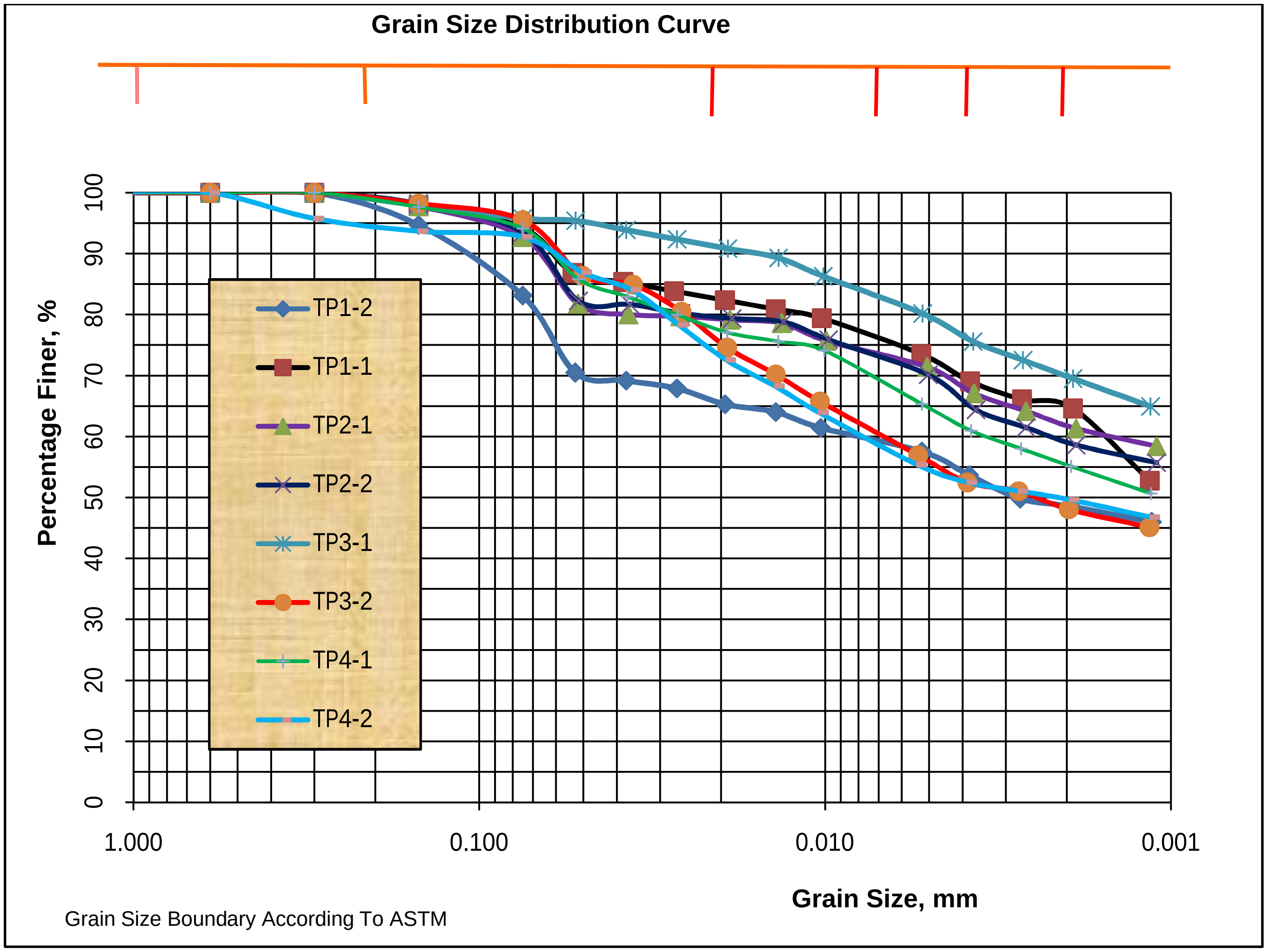
The common laboratory method to determine the particle size distribution of fine-grained soils is a hydrometer test. More than 95% pass sieve no. 200. So that, it is reasonable to use hydrometer test method for grain size analysis. The procedure followed to run this test is according to ASTM standard with designations D422-63 and D1140-97.

The gradation of soils in the study area varies considerably (Table 4.4, Figure 4.3 and Figure 4.4). From the grain size analysis result clay content ranging from 50.32% – 72.58%, silt faction 15.88% – 40.21% and sand fraction 0.36% - 13.28%. The detail tests are presented in Appendix A.1.

Table 4.4 Summary of grain size analysis test result

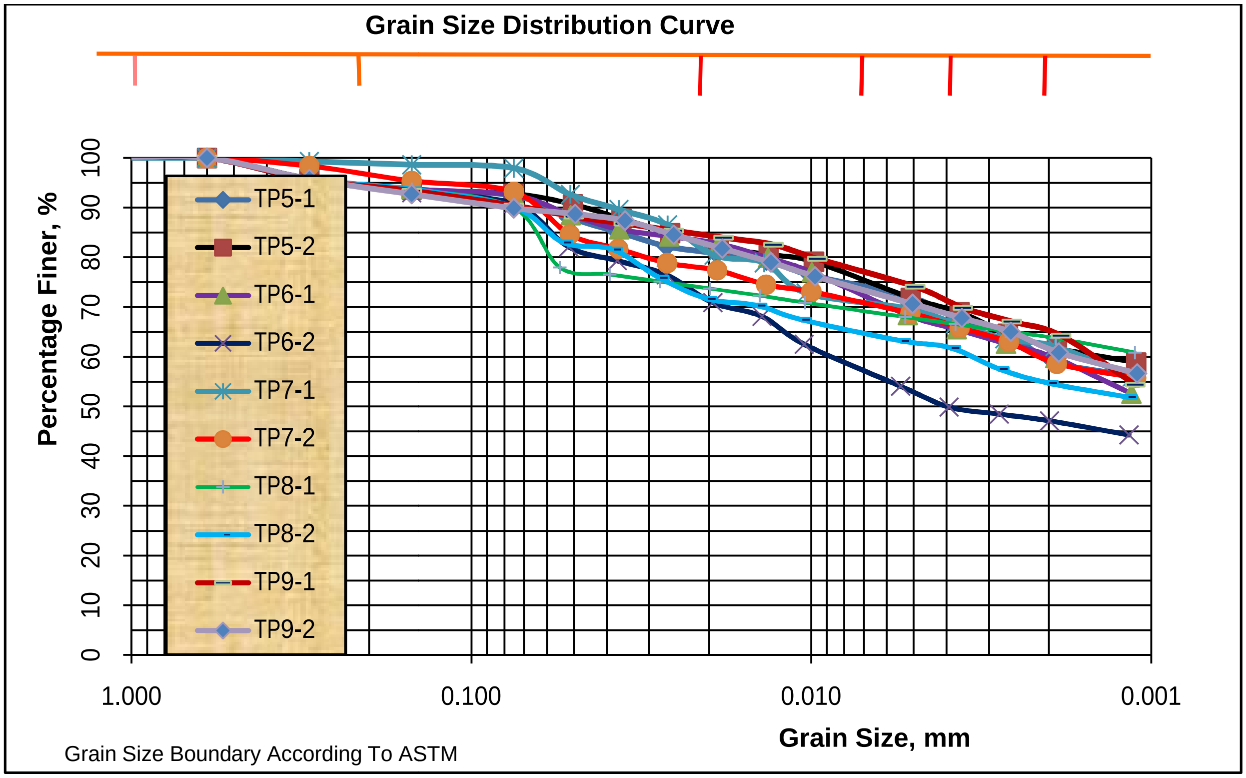
Serial No	Designation	Depth(m)	Percent amount of particle size		
			Sand	Silt	Clay
1	TP-1-1	1.5	7.91	22.58	69.51
2	TP-1-2	3.0	15.56	26.07	58.37
3	TP-2-1	1.5	11.82	22.05	66.14
4	TP-2-2	3.0	12.02	25.31	62.67
5	TP-3-1	1.5	0.36	27.06	72.58
6	TP-3-2	3.0	9.55	40.13	50.32
7	TP-4-1	1.5	8.84	32.67	58.49
8	TP-4-2	3.0	6.29	40.21	53.51
9	TP-5-1	1.5	4.77	31.43	63.80
10	TP-5-2	3.0	2.23	31.24	66.53
11	TP-6-1	1.5	4.38	31.05	64.58
12	TP-6-2	3.0	9.55	38.59	51.86
13	TP-7-1	1.5	5.36	31.24	63.41
14	TP-7-2	3.0	9.2	27.89	62.91
15	TP-8-1	1.5	13.28	15.88	70.84
16	TP-8-2	3.0	7.91	31.43	60.78
17	TP-9-1	2.0	2.43	26.5	72.30
18	TP-9-2	3.0	1.26	31.05	67.74

The sample in the study area is fine clay so that hydrometer test were done for all samples to secure the neesary data.



coarse	medium	fine	coarse	medium	fine	Clay
Sand			Silt			

Fig 4.3 Grain size distribution curve for samples from test pits 1 to 4



coarse	medium	fine	coarse	medium	fine	Clay
Sand			Silt			

Fig 4.4 Grain size distribution curve for samples from test pits 5 to 9

4.2.4. Atterberg Limits

4.2.4.1. General

The condition of a soil can be altered by changing the moisture content. Atterberg Limits are defined as water contents at certain limiting or critical stages in soil behavior. They, along with the natural water content, are the most important items in the description of fine grained soils.

The Liquid Limit, Plastic Limit, and the Plasticity Index of soils are used extensively to correlate a soil with engineering behavior such as compressibility, hydraulic conductivity, shrink-swell, and shear strength. Atterberg defined four possible states of consistency for soils: liquid, plastic, semi-solid and solid. The Liquid Limit divides the plastic and liquid states and is defined as the water content at which the soil flows to close a standard size groove when shaken in a standardized device. The Plastic Limit separates plastic and semi-solid states. At water contents below the Plastic Limit the soil cannot be molded without cracking [9].

The Liquid Limit is the empirically established moisture content at which a soil passes from the plastic to the liquid state. Knowledge of the Liquid Limit allows the engineer to correlate several engineering properties with the soil.

The Liquid Limit is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second.

The Plastic Limit is defined as the lowest moisture content of a soil that will permit a sample to be rolled into threads of 3 mm diameter without the threads breaking [9].

The different states and consistencies through which the soil sample passes with the decrease in the moisture content are depicted in Fig 4.5.

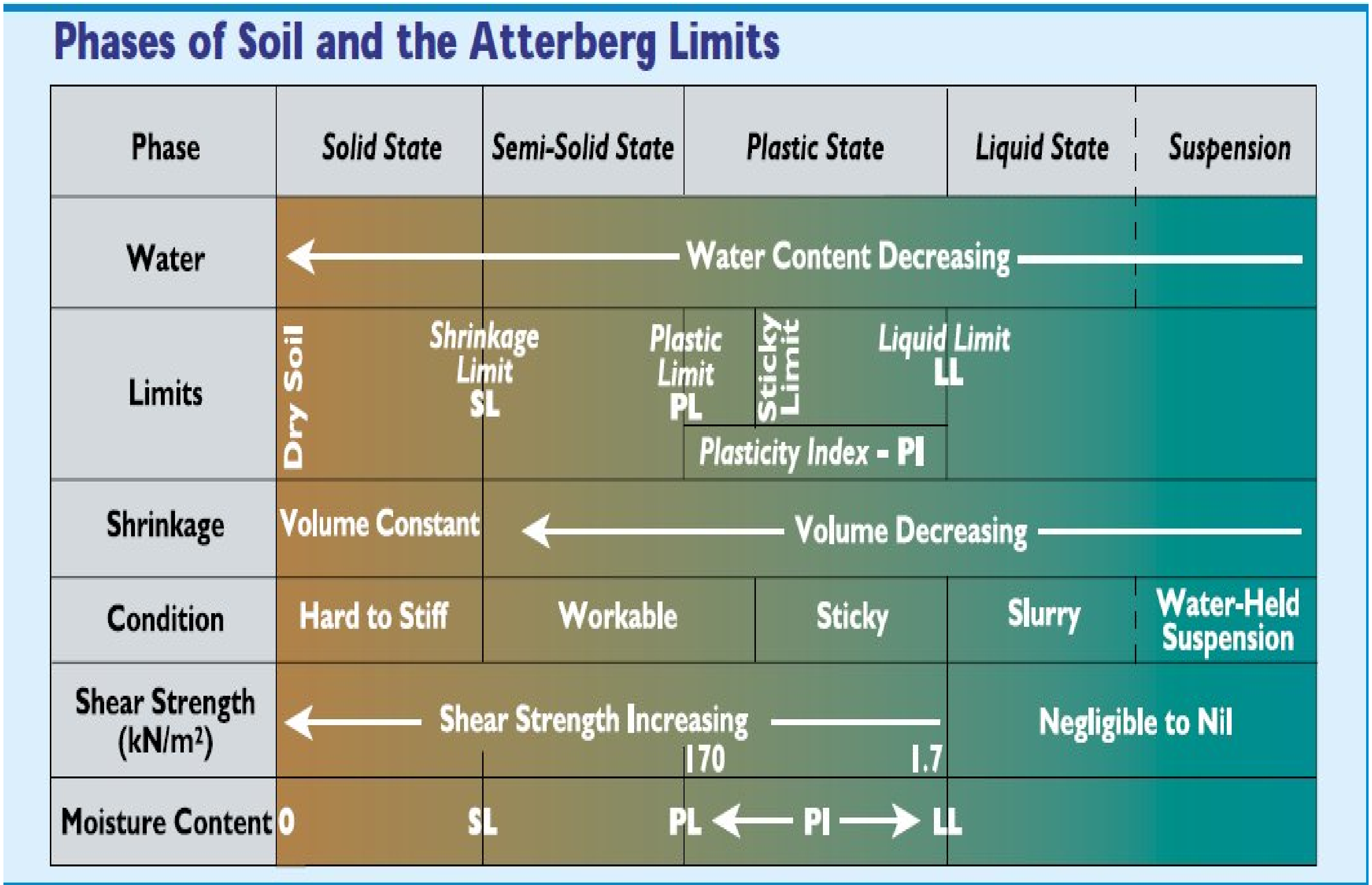


Fig. 4.5 General phases of Soil and the Atterberg Limits (Different states and consistency of soils with Atterberg limits) [9].

4.2.4.2. Test procedure and results

Atterberg Limits were determined for air-dried samples. It was done based on the Standard Reference: ASTM D 4318-98 –Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of soils. The air- dried samples were prepared by spreading the specimen in the air until it dried. The room temperature was about 18-23^{o}c. The portions of the samples passing the No. 40 (0.425mm) sieve were used for the preparation of the sample for this test.

The Atterberg Limits for soil in Debre Markos town are summarized in Table 4.5. From this we can observe that Liquid Limit ranges from 45% – 68%, Plastic Limit ranges from 18% – 38% and Plastic Index from 15% – 40%. The detail tests are presented in Appendix A.2.

Table 4.5 Summary of Atterberg Limit Results

Serial No	Designation	Depth(m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
1	TP-1-1	1.5	58	34	24
2	TP-1-2	3.0	48	33	15
3	TP-2-1	1.5	67	33	34
4	TP-2-2	3.0	63	35	28
5	TP-3-1	1.5	68	28	40
6	TP-3-2	3.0	56	35	21
7	TP-4-1	1.5	66	30	36
8	TP-4-2	3.0	58	33	25
9	TP-5-1	1.5	62	28	34
10	TP-5-2	3.0	63	27	36
11	TP-6-1	1.5	49	18	31
12	TP-6-2	3.0	45	20	25
13	TP-7-1	1.5	61	28	33
14	TP-7-2	3.0	64	28	36
15	TP-8-1	1.5	68	36	32
16	TP-8-2	3.0	63	36	27
17	TP-9-1	1.5	83	35	48
18	TP-9-2	3.0	81	38	43

4.2.5. Free Swell

Both the amount of swelling and the magnitude of swelling pressure are known to be dependent on the clay minerals, the soil mineralogy and structure, fabric and several physico-chemical aspects of the soil. Among clay minerals Montimorillonite influence the magnitude of swelling maximally as compared to Illites and Kaolinites [17].

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

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

Free Swell test results for oven dried samples at a temperature of105$\pm$5 ^{o}c are summarized in Table 4.6. From the test result one can see that the free swell of the soil under investigation ranges from 30% to 50%. Those soils having a free swell less than 50% are considered as medium in degree of expansion [22]. Hence all soil samples under investigation have medium swelling potential.

Table 4.6 Free Swell Test results of the study area

Serial No	Designation	Depth(m)	Free swell (%)	Water used for testing
1	TP-1-1	1.5	50	Tap water
2	TP-1-2	3.0	40	,,
3	TP-2-1	1.5	35	,,
4	TP-2-2	3.0	35	,,
5	TP-3-1	1.5	30	,,
6	TP-3-2	3.0	40	,,
7	TP-4-1	1.5	30	,,
8	TP-4-2	3.0	45	,,
9	TP-5-1	1.5	40	,,
10	TP-5-2	3.0	30	,,
11	TP-6-1	1.5	45	,,
12	TP-6-2	3.0	40	,,
13	TP-7-1	1.5	40	,,
14	TP-7-2	3.0	45	,,
15	TP-8-1	1.5	40	,,
16	TP-8-2	3.0	30	,,
17	TP-9-1	1.5	180	,,
18	TP-9-2	3.0	175	,,

4.3. COMPACTION TEST

The term density refers to mass per unit volume. The density of a mass of soil is of interest to the engineer for a variety of reasons including the design of earthworks and foundations and in slope stability analysis [9].

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

The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density [6].

Two types of compaction tests routinely performed are: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. In the Standard Proctor Test, the soil is compacted by a 24.4N hammer falling a distance of 0.305meters into a soil filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test, except it employs, a 44.5N hammer falling a distance of 0.457meters, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 0.102meters in diameter and has a volume of about 944 cm^3, and the larger type is 0.152meters in diameter and has a volume of about 2123 cm^3. If the larger mold is used each soil layer must receive 56 blows instead of 25[6].

From the test results the maximum dry density (MDD) of D/Markos ranges from1350 kg/m3 to 1420 kg/m3 and the optimum moisture content ranges 28% to 36%. The summary of the test result is shown in Table 4.7. The detail tests are presented in Appendix A.3.

Generally course grained soils can be compacted to a higher dry density than fine gained soils for the some compaction effort. When some fines are added to the coarse grained soils to fill the voids, the maximum dry density further increases, but if the amount of fines is too much, more than required to fill the voids, it results in reduction of dry density; well graded soils can attain higher dry density than poorly graded soils. High plasticity clays attain much less dry density than low plasticity clays for the some completive effort [6].

Table 4.7 Summary of Optimum moisture content and the maximum dry density

Serial No	Designation	Depth(m)	OMC (%)	MDD (g/cm^3)
1	TP-1-1	1.50	35	1.39
2	TP-1-2	2.45	36	1.37
3	TP-2-1	1.5	36	1.35
4	TP-2-2	3.00	31	1.41
5	TP-3-1	1.5	28	1.42
6	TP-3-2	3.00	34	1.36
7	TP-4-1	1.5	28	1.40
8	TP-4-2	3.0	32	1.36
9	TP-5-1	1.5	34	1.37
10	TP-5-2	3.0	34	1.39
11	TP-6-1	1.5	29	1.38
12	TP-6-2	3.0	30	1.42
13	TP-7-1	1.5	32	1.37
14	TP-7-2	3.0	28	1.39
15	TP-8-1	1.5	35	1.38
16	TP-8-2	3.0	33	1.41

4.4. CLASSIFICATION OF THE SOILS

4.4.1. General Considerations for Classification of Soils

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

When a soil consists of the various constituents in different proportions, the mixture is then given the name of the constituents that appear to have significant influence on its behavior, and then other constituents are indicated by adjectives. Thus sandy clay has most of the properties of clay but contains a significant amount of sand [16].

The individual constituents of a soil mixture can be separated and identified as gravel, sand, silt and clay on the basis of mechanical analysis. The clay mineral that is present in a clay soil is sometimes a matter of engineering importance. According to the mineral present, the clay soil can be classified as Kaolinite, Montmorillonite or Illite. The minerals present in clay can be identified by either X-ray diffraction or differential thermal analysis. Buildings, bridges, dams etc. are built on natural soils (undisturbed soils), whereas earthen dams for reservoirs, embankments for roads and railway lines, foundation bases for pavements of roads and airports are made out of remolded soils. Sites for structures on natural soils for embankments, etc, will have to be chosen first on the basis of preliminary examinations of the soil that can be carried out in the field. An engineer should therefore be conversant with the field tests that would identify the various constituents of a soil mixture [16].

The behavior of a soil mass under load depends upon many factors such as the properties of the various constituents present in the mass, the density, the degree of saturation, the environmental conditions etc. If soils are grouped on the basis of certain definite principles and rated according to their performance, the properties of a given soil can be understood to a certain extent, on the basis of some simple tests.

Many systems are in use that is based on grain size distribution and limits of soil. The systems that are quite popular amongst engineers are the AASHTO Soil Classification System and the Unified Soil Classification System (USCS).

4.4.2. Classifications of soils based on AASHTO Classification system

There are seven groups of inorganic soils, A-l to A-7 with 12 subgroups in all. The system is based on Particle-Size Distribution, Liquid Limit and Plasticity Index.

The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form PI= LL-30. It generally classifies a soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2 and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6 and A-7.

As it can be observed from this Classification system (Table 4.8 Fig 4.6 and Fig 4.7) the usual types of significant constituent material is clayey soil A-7-5 and A-7-6.

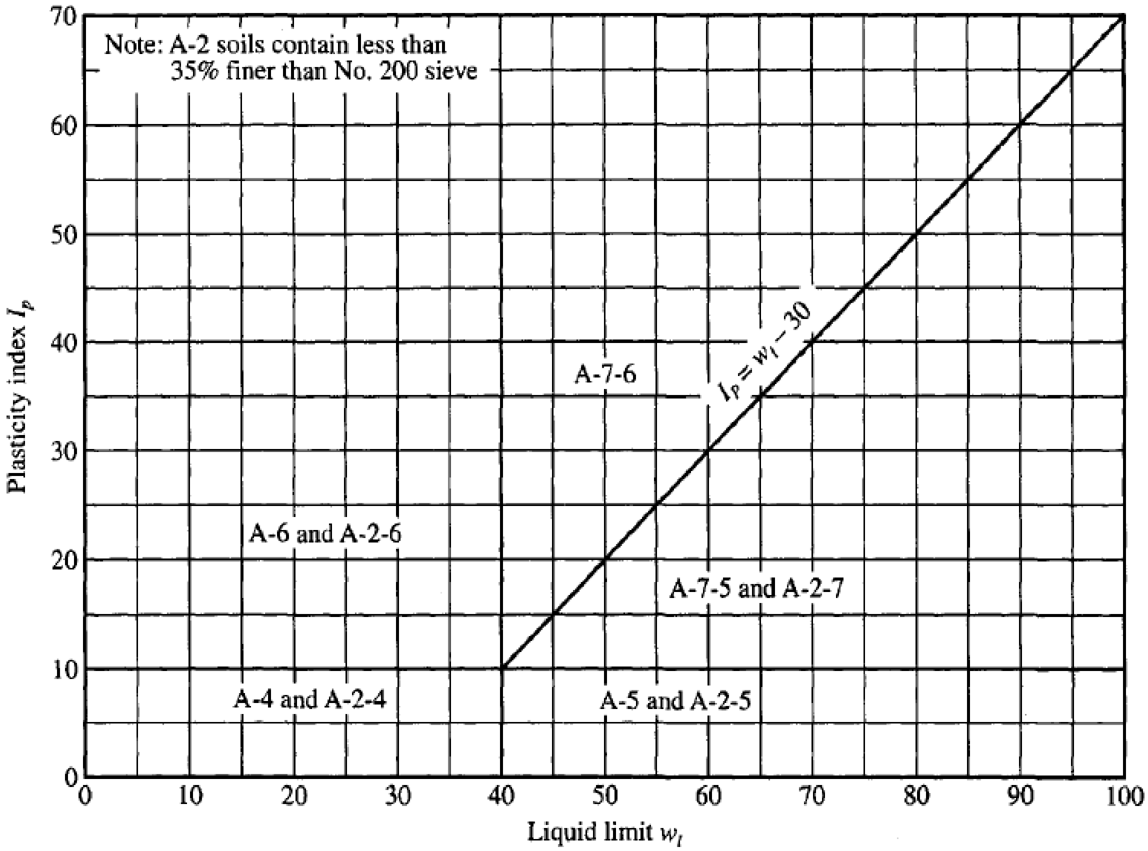


Fig. 4.6 Chart for use in AASHTO soil classification system [16]

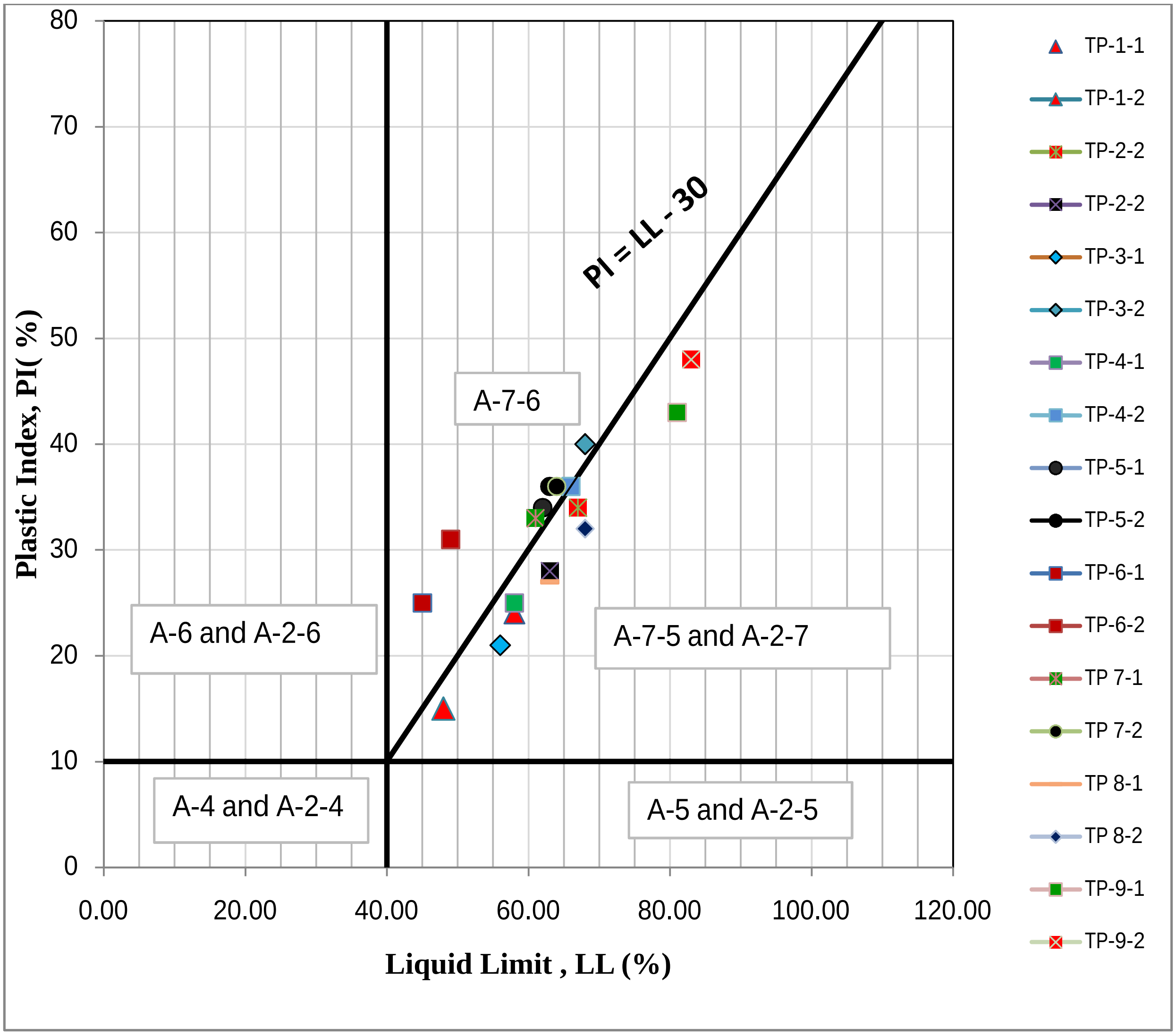


Fig. 4.7 Plasticity chart of soil in the study area according to AASHTO classification system

Table 4.8 Classifications of soils based on AASHTO Classification system

Serial No.	Designation	Depth(m)	Percent passing on sieve #200	LL (%)	PI (%)	Group classification	Usual types of significant constituent materials	General rating as sub-grade materials
1	TP-1-1	1.5	98.8	58	24	A-7-5	Clayey soils	Poor!
2	TP-1-2	3.0	>95	48	15	A-7-5	Clayey soils	Poor!
3	TP-2-1	1.5	>95	67	34	A-7-5	Clayey soils	Poor!
4	TP-2-2	3.0	>95	63	28	A-7-5	Clayey soils	Poor!
5	TP-3-1	1.5	>95	68	40	A-7-6	Clayey soils	Poor!
6	TP-3-2	3.0	>95	56	21	A-7-6	Clayey soils	Poor!
7	TP-4-1	1.5	>95	66	36	A-7-6	Clayey soils	Poor!
8	TP-4-2	3.0	>95	58	25	A-7-5	Clayey soils	Poor!
9	TP-5-1	1.5	>95	62	34	A-7-6	Clayey soils	Poor!
10	TP-5-2	3.0	>95	63	36	A-7-6	Clayey soils	Poor!
11	TP-6-1	1.5	>95	49	31	A-7-6	Clayey soils	Poor!
12	TP-6-2	3.0	>95	45	25	A-7-6	Clayey soils	Poor!

13	TP-7-1	1.5	>95	61	33	A-7-6	Clayey soils	Poor!
14	TP-7-2	3.0	>95	64	36	A-7-6	Clayey soils	Poor!
15	TP-8-1	1.5	>95	68	32	A-7-5	Clayey soils	Poor!
16	TP-8-2	3.0	>95	63	27	A-7-5	Clayey soils	Poor!
17	TP-9-1	1.5	>95	83	48	A-7-5	Clayey soils	Poor!
18	TP-9-2	3.0	>95	81	43	A-7-5	Clayey soils	Poor!

4.4.3. Classification of soils based on Unified Soil Classification System (USCS)

The Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. It divides soil into three major divisions: coarse-grained soils, fine grained soils, and highly organic (peaty) soils. In the field, identification is accomplished by visual examination for the coarse-grained soils and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used. The peat soils are readily identified by color, odor, spongy feel and fibrous texture [16].

Table 4.9 Classifications of soils based on USC Classification system

Serial No.	Designation	Depth(m)	Particle size passing no. 200(%)	LL (%)	PI (%)	Classification According to USCS
1	TP-1-1	1.5	98.8	58	24	MH
2	TP-1-2	3.0	>95	48	15	ML
3	TP-2-1	1.5	>95	67	34	CH
4	TP-2-2	3.0	>95	63	28	MH
5	TP-3-1	1.5	>95	56	21	MH
6	TP-3-2	3.0	>95	68	40	CH
7	TP-4-1	1.5	>95	58	25	MH
8	TP-4-2	3.0	>95	66	36	CH
9	TP-5-1	1.5	>95	62	34	CH
10	TP-5-2	3.0	>95	63	36	CH
11	TP-6-1	1.5	>95	45	25	CL
12	TP-6-2	3.0	>95	49	31	CL
13	TP-7-1	1.5	>95	61	33	CH
14	TP-7-2	3.0	>95	64	36	CH
15	TP-8-1	1.5	>95	63	27	MH
16	TP-8-2	3.0	>95	68	32	MH
17	TP-9-1	1.5	>95	81	43	CH
18	TP-9-2	3.0	>95	83	48	CH

According to USC classification scheme most of the soil of the study area falls in MH, CH and CL region. From the plot of plasticity chart in Figure 4.9 and the classification soils on Table 4.9 most of the soils found in Debre Markos town is highly plastic inorganic clay and inorganic silt (MH, CH and CL). That means inorganic silt with low to high plasticity.

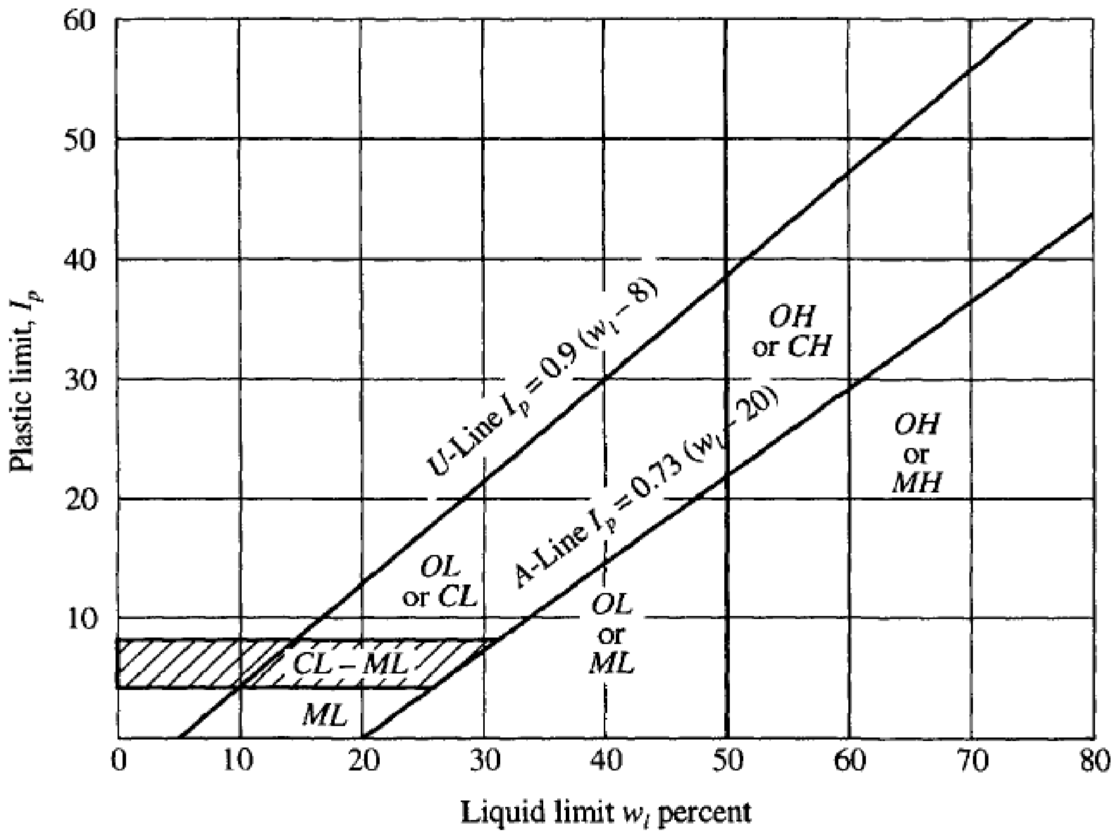


Fig 4.8 Plasticity chart of for fine grained soils [16]

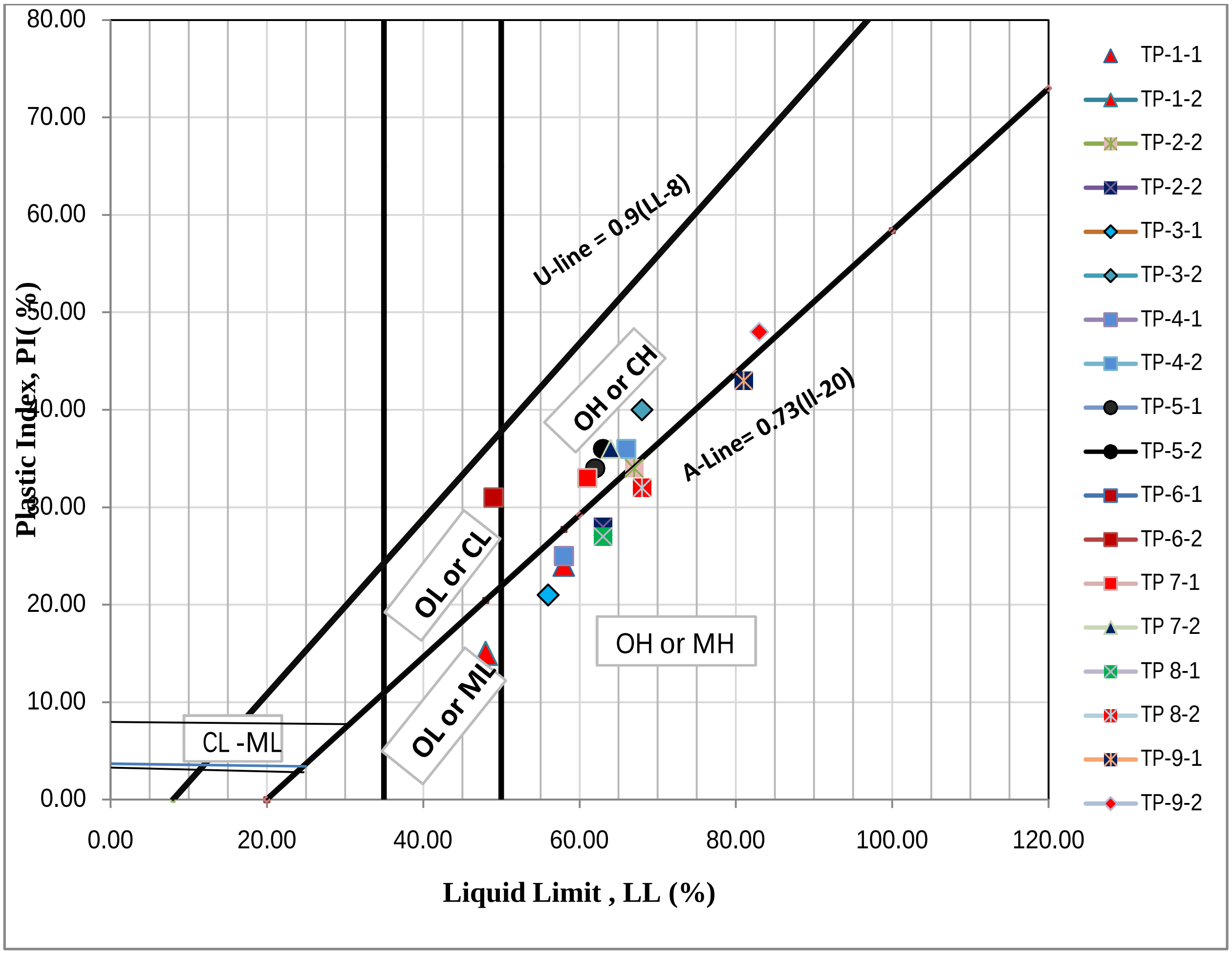


Fig 4.9 Plasticity chart of the study area according to Unified Soil Classification System

4.4.4. Chen method of classification of expansive soil

As per Chen soil classification system soil in TP-9 has high degree of expansion.

Table 4.10 Chen method of classification of expansive soil [17]

Plasticity index I_p (%)	Swelling potential
0–15	Low
10–35	Medium
20–55	High
35 and above	Very high

Table 4.11 Relation between Swelling Potential and Plasticity Index, I_p [17]

Serial No.	Designation	Depth(m)	PI (%)	Swelling Potential
1	TP-1-1	1.5	24	Medium
2	TP-1-2	3.0	15	Low
3	TP-2-1	1.5	34	Medium
4	TP-2-2	3.0	28	Medium
5	TP-3-1	1.5	40	High
6	TP-3-2	3.0	21	Medium
7	TP-4-1	1.5	36	High
8	TP-4-2	3.0	25	Medium
9	TP-5-1	1.5	34	High
10	TP-5-2	3.0	36	High
11	TP-6-1	1.5	31	Medium
12	TP-6-2	3.0	25	Medium
13	TP-7-1	1.5	33	Medium
14	TP-7-2	3.0	36	High
15	TP-8-1	1.5	32	Medium
16	TP-8-2	3.0	27	Medium
17	TP-9-1	1.5	48	Very high
18	TP-9-2	3.0	43	Very high

Here the classification is done based on the method proposed by Chen [17] that uses plasticity index as its basis of classification. My test result is not fit with Chain method of classification especially TP 3-1 and TP 3-2.

The result of Debre Markos soil based on Chen classification system is presented in Table 4.11. Majority samples are categorized as medium to high swelling potential for samples from TP 1 to TP 8. But sample on TP 9 is very high swelling potential. On the other hand there is a contradiction inTP3 on the relation between PI and Free Swell.

4.5. UNCONFINED COMPRESSION STRENGTH (UCS) TEST

The Unconfined Compression Test (UCS) is a special case of the Unconsolidated Undrained Triaxial Test. In this case, no confining pressure is applied to the specimen. The UCS test is one of the easiest and simplest tests for a quick estimate of the shear strength of cohesive soils. The test provides an immediate approximate value of the compressive strength of the soil, either in the undisturbed or the remolded condition. It is also widely used to determine the consistency of saturated clays and other cohesive soils [11]. In this study the UCS tests were carried out on three representative undisturbed samples obtained by tube sampling based on the classification, from the field. Details of testing procedures are given in BS 1377: Part 7: 1990. The results of UC tests are shown in Table 4.12. The unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test. The detailed tests are presented in Appendix A.4.

For soils, the Undrained Shear Strength (Su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (S_u) of clays is commonly determined from an Unconfined Compression Test. The Undrained Shear Strength (S_u) of a cohesive soil is equal to one-half the Unconfined Compressive Strength (q_u) when the soil is under the $\emptyset$= 0 condition ($\emptyset$ = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion (c). This is expressed as [13]:

$$S_u = C = q_u \qquad \text{4.2}$$

Table 4.12 Summery of UCS test results

TP.NO.	q_u(kPa)	C_u(kPa)	ω(%)
5	382.96	191.48	26.67
6	359.34	179.67	33.33
7	320.21	160.10	41.18

4.5. CONSOLIDATION

4.5.1. General

A surface load, for example due to the construction of building, results in increased stresses in the underlying soils. The increase in stress generally causes settlements. When the soils are fine grained and saturated the increase in total stress is carried by the water, as excess pore pressure. Since these soils have low hydraulic conductivity the excess pore pressure will dissipate slowly and the settlement will be delayed in time. The consolidation test, or odometer test, is used to determine the parameters that can be used to estimate both the magnitude and the time rate of the settlements [12].

The effect of the loads is felt by the soil normally up to a depth of about two to three times the width of the foundation. The soil within this depth gets compressed due to the imposed stresses. The compression of the soil mass leads to the decrease in the volume of the mass which results in the settlement of the structure.

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

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

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

Soil engineering problems are of two types. The first type includes all cases wherein there is no possibility of the stress being sufficiently large to exceed the shear strength of the soil, but wherein the strains lead to what may be a serious magnitude of displacement of individual grains leading to settlements within the soil mass. The second type includes cases in which there is danger of shearing stresses exceeding the shear strength of the soil. Problems of this type are called stability Problems [16].

In the case of cohesive soils, the dry state of the soils is not considered as this state is only of a temporary nature. When the soil becomes saturated during the rainy season, the soil becomes more compressible under the same imposed load. Settlement characteristics of cohesive soils are, therefore, considered only under completely saturated conditions [16].

When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting thereby in an excess pore water pressure. If drainage is permitted, the resulting hydraulic gradients initiate a flow of water out of the clay mass and the mass begins to compress. A portion of the applied stress is transferred to the soil skeleton, which in turn causes a reduction in the excess pore pressure. This process, involving a gradual compression occurring simultaneously with a flow of water out of the mass and with a gradual transfer of the applied pressure from the pore water to the mineral skeleton is called consolidation [16].

The total compression of saturated clay strata under excess effective pressure may be considered as the sum of

1. Immediate compression,
2. Primary consolidation, and
3. Secondary compression.

4.5.2. Test procedure and results

The consolidation test is used to determine the parameters that can be used to estimate both the magnitude and the time rate of the settlements. The test is performed on a cylindrical specimen, constrained laterally by a ring and allowed to compress under a constant load. The load is held on the sample for 24 hours or until all excess pore pressure is dissipated. During this time the change in height is measured. The load is usually doubled at the end of the 24 hour period and the process repeated. Usually 5 or 6 load increments are applied and then data are taken during one unloading step. The measurements are used to determine the relationship between the effective stress and void ratio or strain, and the rate at which consolidation can occur [12].

This test method uses conventional consolidation theory based on Terzaghi's consolidation equation to compute the coefficient of consolidation, c_v.

The analysis is based on the following assumptions [12]:

- The soil is saturated and has homogeneous properties.
- The flow of pore water is in the vertical direction.
- The compressibility of soil particles and pore water is negligible compared to the compressibility of the soil skeleton.

- The stress-strain relationship is linear over the load increment.
- The ratio of soil permeability to soil compressibility is constant over the load increment.
- Darcy's law for flow through porous media applies.

This test was done according to ASTM standard, Designation D2435-96. The standard load increment duration is 24 h. Record the height or change in height, d, at time intervals of approximately 0.1, 0.25, 0.5, 1, 2, 4, 8, 15 and 30 min, and 1, 2, 4, 8 and 24 h. Take sufficient readings near the end of the pressure increment period to verify that primary consolidation is completed. The load was doubled every 24 hours starting from 50kpa to 1600kpa. This procedure was followed for all the samples. The plot of void ratio versus logarizem of pressure and pressure for all the samples is shown in fig 4.10 and 4.11 respectively (see App.A for detail calculations of the consolidation test result).

The loading varied between 50 kpa to 1600 kPa by doubling the loading after 24 hrs. For each loading starting from 50 kpa to 1600 kPa the compression was recorded from the dial gage at intervals of: 0.1, 0.25, 0.5, 1, 2, 4 … 1440minuts for twenty-four hours. Unloading was also done by steps to examine the unloading behavior. A total of three consolidation tests were run on samples of three undisturbed samples on different test pits. Refer appendix B.

4.5.2.1. Pre-consolidation pressure

Pre-consolidation pressure was determined from void ratio versus log pressure curve by simplified method and Casagrande's method [4].

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or 40 excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another. The practical significance of the pre-consolidation load appears in calculating settlements of structures [6].

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. No suitable criteria exist for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande [4]. The method involves locating the point of maximum curvature, B, on the laboratory e-log p curve of an undisturbed sample as shown in Fig 4.10. From B, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the preconsolidation pressure, P_c. [16]

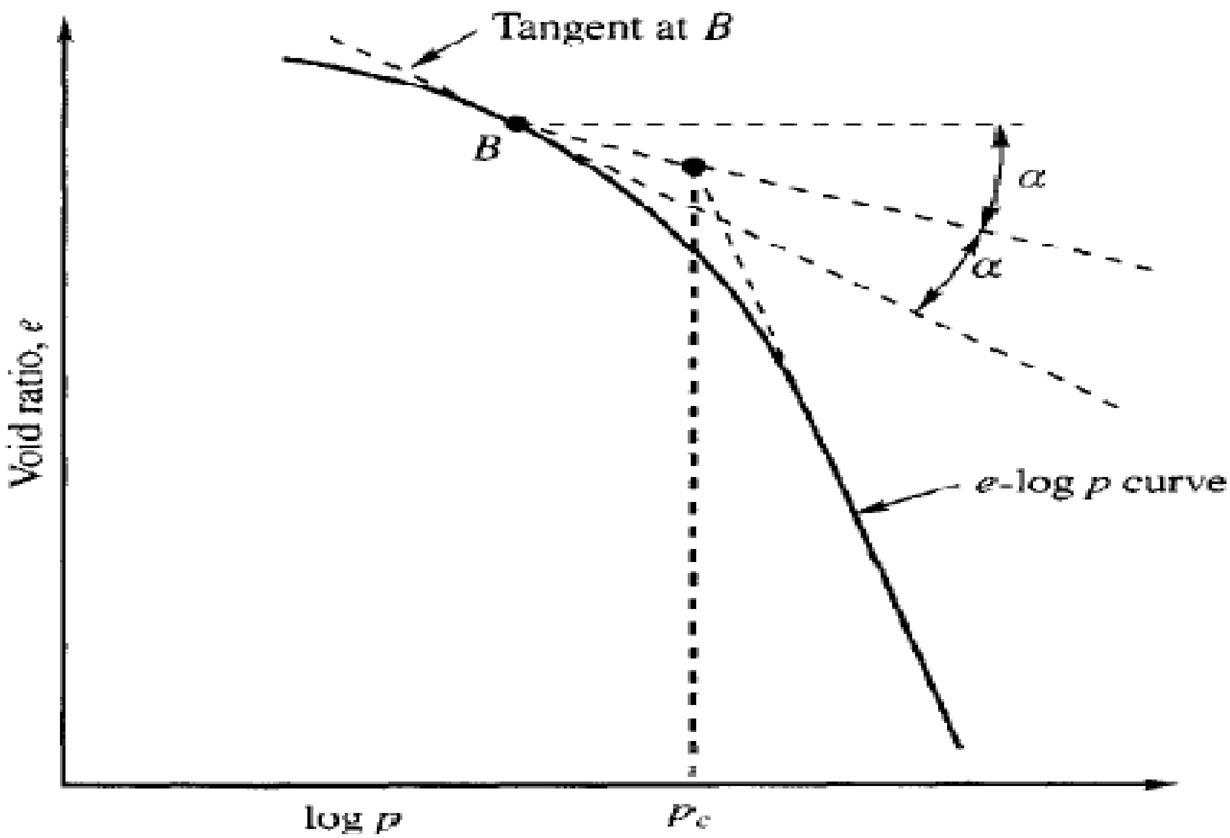


Fig 4.10 Method of determining P_c by Casagrande method [16]

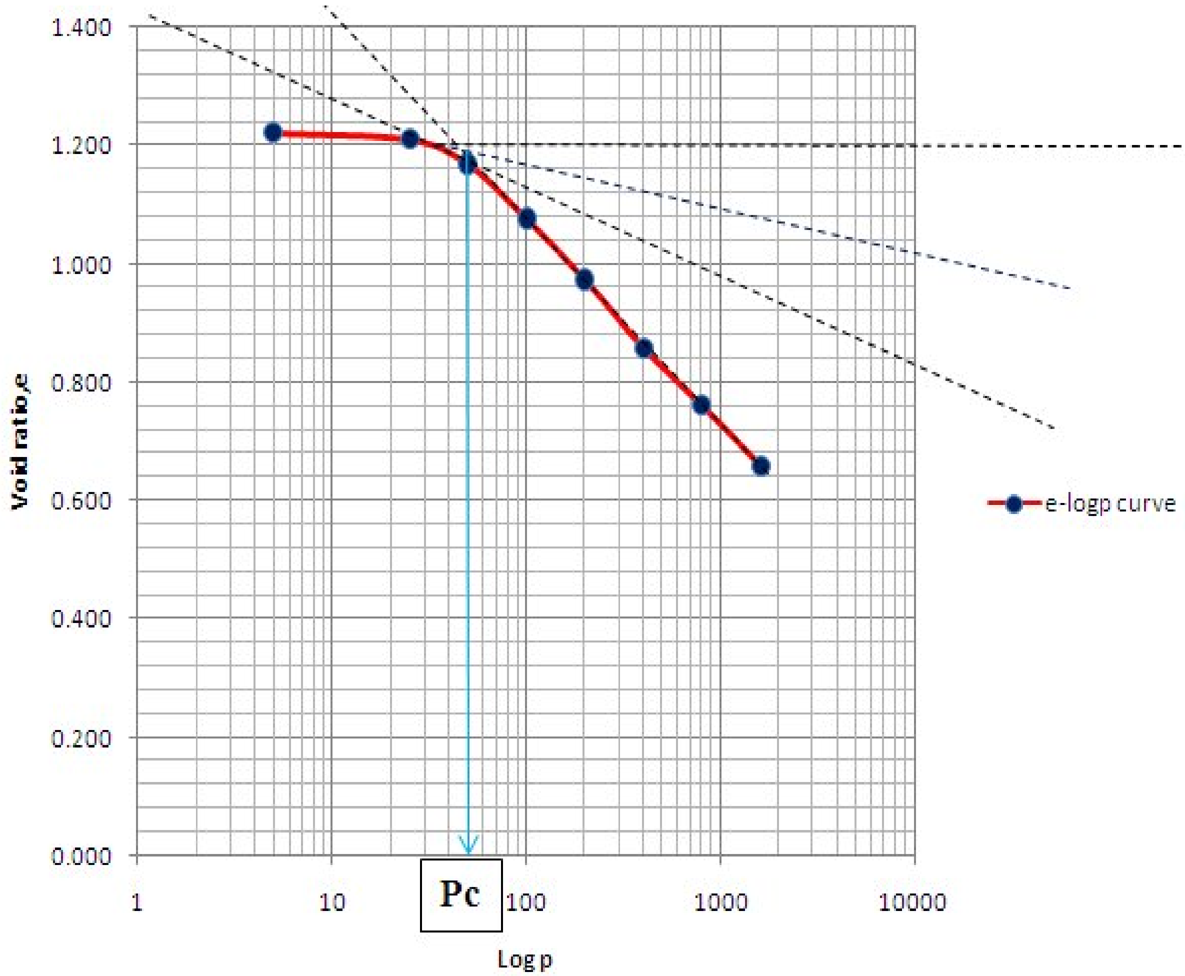


Fig 4.11 Typical void ratio Vs pressure curve used to determine P_c

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

OCR =— ……………………………………………………………... (4.3)

The pre-consolidation pressure for the three samples are determined as depicted in Fig 4.13. The results are shown in Table 4.12. From Fig 4.12 and 4.13. The detailed tests are presented in Appendix B.

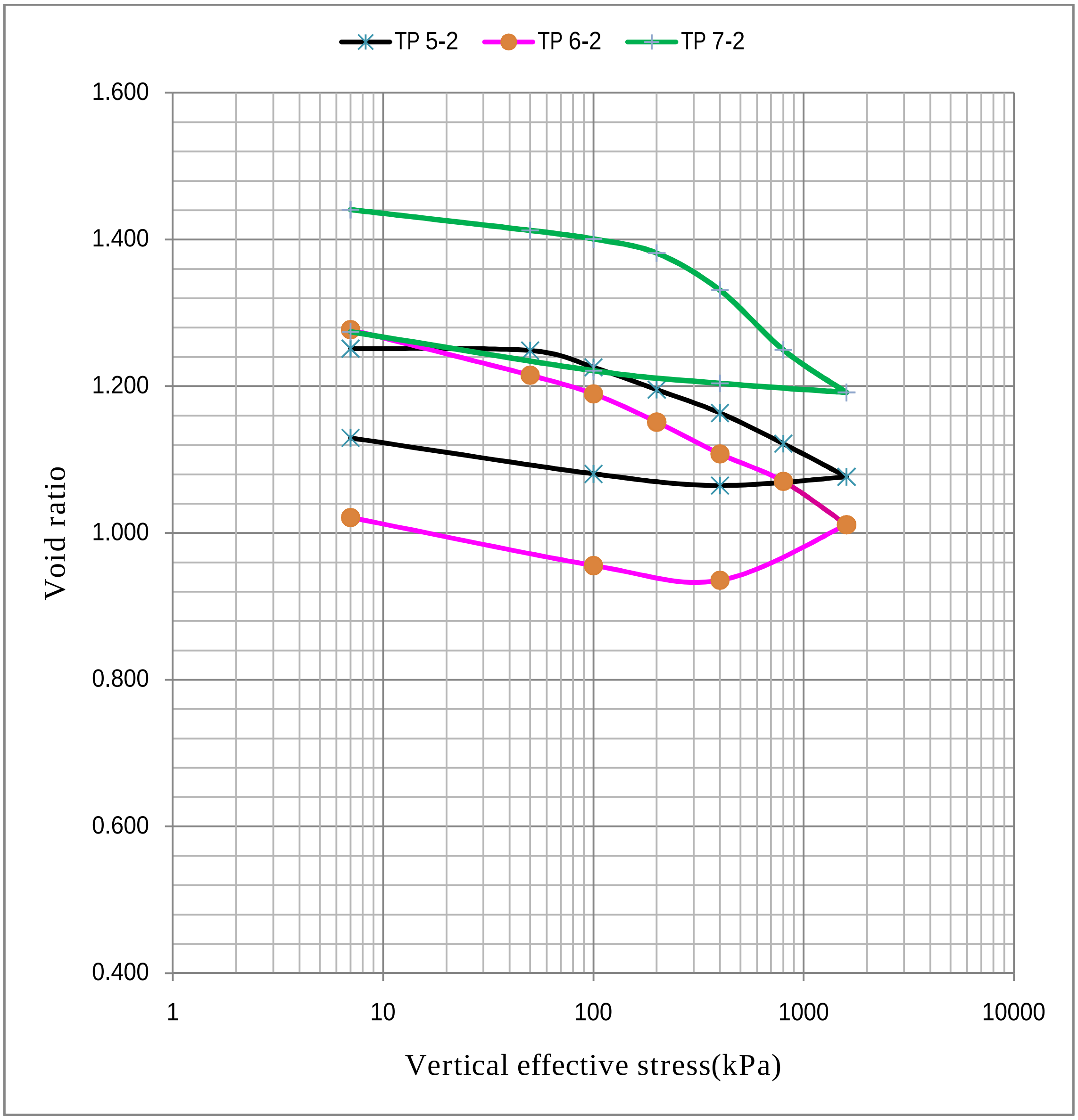


Fig 4.12 Plot of vertical effective stress Vs void ratio on semi-log scale

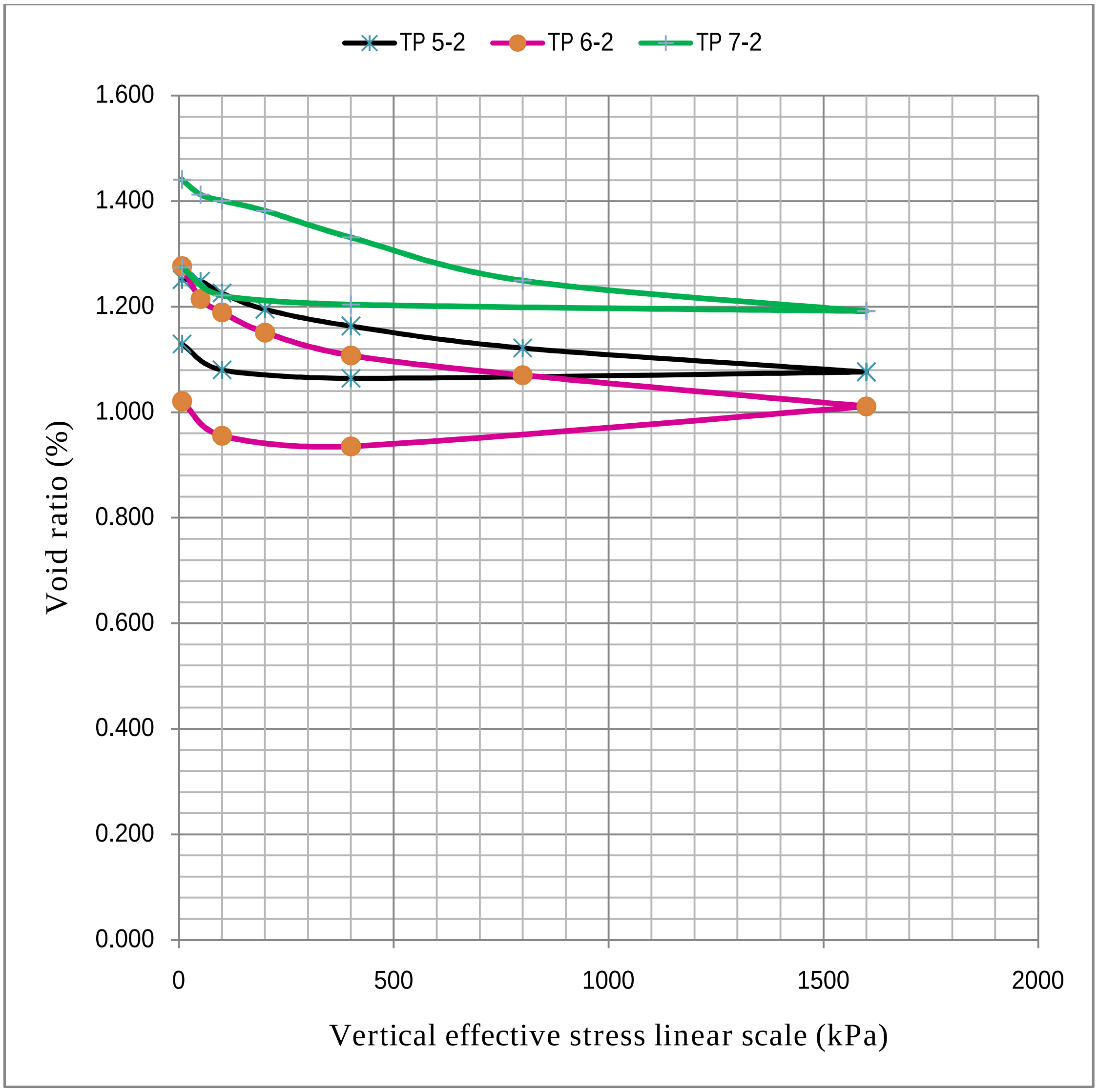


Fig 4.13 Plot of vertical effective stress Vs void ratio on linear scale

4.5.2.2 Compression index

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

$$= \frac{\Delta}{-(\Delta)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (\quad . \quad)$$

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

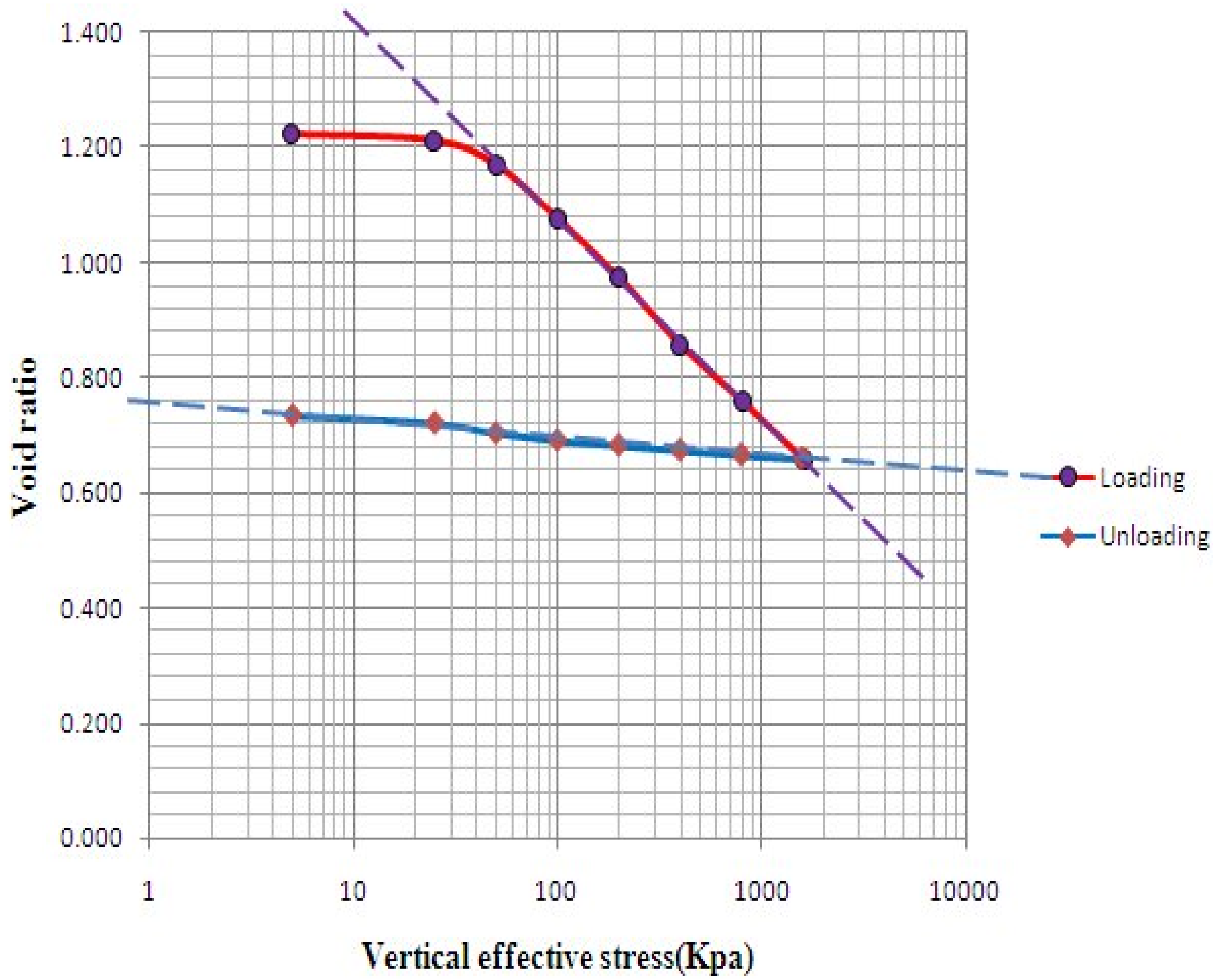


Fig 4.14 Typical loading unloading curves used to calculate the coefficient of consolidation and the recompression index

Table 4.13 Summery of the consolidation test results

Test Pit Designation	Depth (m)	Natural Moisture Content (%)	Total Unit weight in (γ) kPa	Pressure P kPa	Void ratio, e_f	Coefficient of Consolidation, C_v 10-3 cm^2/sec	Compression Index, C_c	Re compression Index, C_s	Over-burden pressure, P_o (kPa)	Pre-consolidation pressure, P_c (kPa)	Over-consolidation ratio (OCR)
TP-5-2	3	33.7	16.2	7	1.25	-	0.144	0.003	48.6	85	1.749
				50	1.25	3.533					
				100	1.23	4.743					
				200	1.20	3.533					
				400	1.16	2.174					
				800	1.12	1.570					
				1600	1.08	1.002					
TP-6-2	3	32.4	17.9	7	1.277	-	0.223	0.008	53.7	120	2.2
				50	1.215	4.615					
				100	1.190	6.281					
				200	1.151	4.615					
				400	1.108	2.792					
				800	1.070	1.570					

				1600	1.011	0.801					
TP-7-2	3	49.6	16.7	7	1.44	-	0.316	0.069	50.1	200	4
				50	1.41	1.570					
				100	1.40	3.533					
				200	1.38	2.672					
				400	1.33	1.710					
				800	1.25	1.187					
				1600	1.19	0.883					

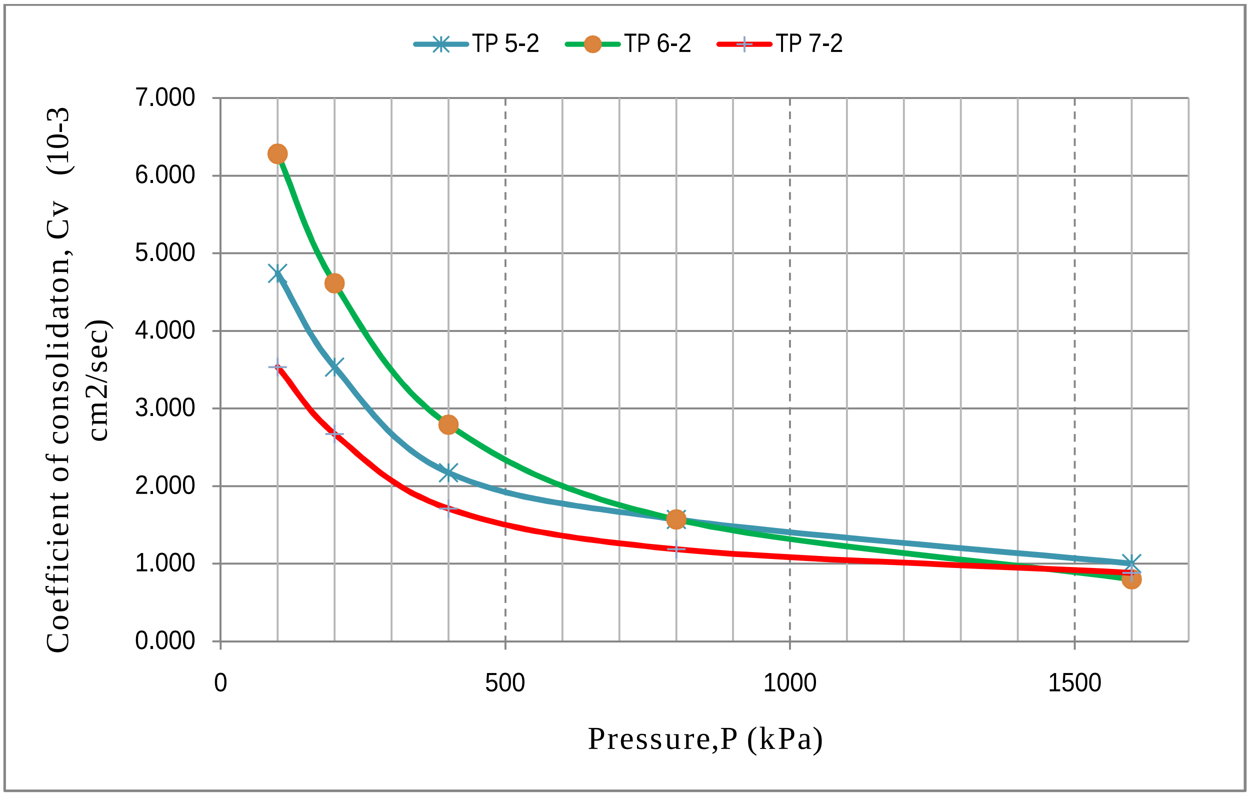


Fig 4.15 Coefficient of Consolidation Vs Pressure

4.5.2.3 Coefficient of consolidation

From those increments of load where time-deformation readings are obtained, two alternative procedures are provided to present the data, determine the end-of-primary consolidation and compute the rate of consolidation [2].

i. Logarithm-of-time-fitting method

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

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

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

ii. Square-root-of-time fitting method

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

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

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

From the measured data and the data obtained from either of the above two methods, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil [13].

During the process of consolidation k and are assumed to be constant, the coefficient of consolidation during the process of consolidation of the clay is constant [20].

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

$$= \frac{(0.196) *}{} \quad \frac{}{} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . (4.5)$$

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

$$= \frac{(0.848) *}{} \quad \frac{}{} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . (4.6)$$

4.5.2.4 Coefficient of Permeability

The coefficient of permeability can be measured using field tests, or tests conducted in the laboratory. Permeability is sometimes also estimated from one dimensional consolidation test. The coefficient of permeability can be obtained from the following relationship [20].

$$k = \frac{c \;\; * a \;\; * \gamma}{1 + e} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.7)$$

Where: - Coefficient of consolidation

- Coefficient of compressibility

γ - Unit weight of water

- Void ratio

Using the above equation, the coefficient of permeability as the function of void ratio was calculated from the consolidation test results and shown in Table 4.13. It is noted that a , the ratio of change in void ratio to change in pressure, was obtained from Fig 4.13. As shown in Table 4.13, the range of values of coefficient of permeability lies between 10^{-9} and10^{-11} m/sec, which indicates that the soils are practically impervious or have low permeability. In general, void ratio versus log coefficient of permeability is close to a straight line for nearly all soils [13]. As shown in Fig 4.16, for all the soil samples void ratio versus log coefficient of permeability is close to a straight line.

Table 4.14 Relationship between Void ratio and coefficient of permeability

Test Pit Designation	Depth (m)	Pressure P kPa	Void ratio e_f	Coefficient of consolidation C_v 10^{-7} m^2/sec	Coefficient of compressibility a_v 10^{-4}m2/kN	Coefficient of permeability k 10^{-2}m/sec
TP-5-2	3.0	50	1.251	3.533		
		100	1.251	4.743	4.548	2.13066E-07
		200	1.249	3.533	3.062	1.06903E-07
		400	1.226	2.174	1.587	3.41133E-08
		800	1.195	1.570	1.041	1.61695E-08
		1600	1.163	1.002	0.566	5.60929E-09
TP-6-2	3.0	50	1.277	4.615		
		100	1.277	6.281	5.080	3.15372E-07
		200	1.215	4.615	3.850	1.75662E-07
		400	1.190	2.792	2.160	5.96512E-08
		800	1.151	1.570	0.939	1.45868E-08
		1600	1.108	0.801	0.741	5.8736E-09
TP-7-2	3.0	50	1.441	1.570		
		100	1.441	3.533	2.343	8.16436E-08
		200	1.412	2.672	1.904	5.01685E-08
		400	1.401	1.710	2.526	4.26259E-08
		800	1.382	1.187	2.044	2.39721E-08
		1600	1.331	0.883	0.723	6.31169E-09

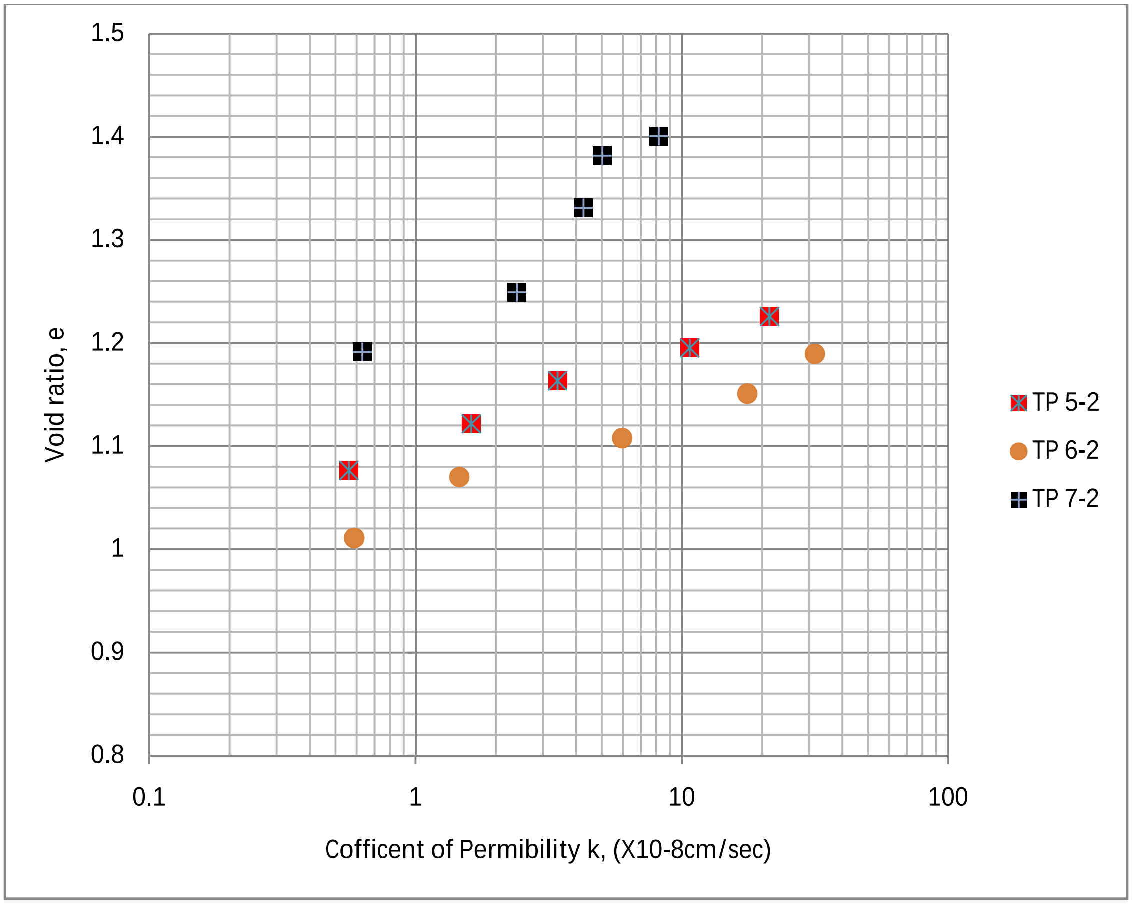


Fig 4.16 Void ratio Vs Log Coefficient of Permeability

Figure 4.16 shows the coefficient of permeability increase as the void ratio increases.

It must be noted that each plot in Fig. 4.16 is for a given soil. The permeability of a soil at a given void ratio may not have any relationship with that of another soil at the same void ratio. Paradoxically, the soil with the largest void ratio (i.e. clay) is the list pervious. This is due to the fact that the individual void passages in clay are extremely small through which water cannot flow easily [1]. From the above figure we can notice that the permeability of a soil is directly proportional to that of void ratio.

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

5.1. Discussions Of The Laboratory Test Results

The Specific Gravity lies in the range between 2.69 to 2.84, Clay Content lies in the range 50% to 72%, Plasticity Index ranges between 14% to 40%, and those values are similar to the results obtained by Samuel, 1989, Addiszemen, 2005.

The Grain Size analysis result is shown in Figures 4.3 and 4.4 and the summary of grain size analysis result is shown on Table 4.4. The results obtained from the grain size analyses indicate that the dominant proportion of soil particle in the research area is clay, which have clay content ranging from 50 % – 73% , silt faction 15.88% – 40.21 % and sand fraction 0.36% - 13.28%.

The result of Atterberg Limit of the soil samples is shown on Table 4.5. From these testes the soil under investigation is inorganic clay soil. The soil in the research area has Liquid Limit ranging from 45% – 68%, Plastic Limit ranges from 18% – 38% and Plastic Index from 15% – 40%. This means the consistency of the soil is medium and highly plastic [17].

Free Swell test results for air dried samples are summarized in Table 4.6. From the test result one can see that the Free Swell of the soil under investigation ranges from 30% to 50% except TP9 which is an expansive soil having Free Swell of 180%. Those soils having a Free Swell less than 50% are considered as low in degree of expansion [20]. Hence all soil samples other than TP9 under investigation are non expansive soils.

The Specific Gravity lies in the range between 2.69 to 2.84, which is similar to the results obtained by Morin and a previous study.

Optimum Moisture Content and the Maximum Dry Density of the study area is summarized in Table 4.8 from compaction test results. From the test results the Maximum Dry Density (MDD) of Debre Markos soil ranges from1.35 g/cm3 to 1.42 g/cm3 and the Optimum Moisture Content ranges 28% to 36%.

The values of Unconfined Compressive Strength from UCS Test range from 320.21kpa to 382.96kpa at natural moisture content of 41.2% to 26.7% respectively.

Classifications of soils in the study area based on AASHTO Classification system is shown in Table 4.10. And also Fig 4.6 shows plasticity chart of soil in the study area according to AASHTO system of classification. From this table and chart it can be observed that soil in the study area is classified in group A-7-5/6. This implies the soil is poor for subgrade material.

Fig 4.9 shows plasticity chart of the study area according to Unified Soil Classification System. This chart shows that the majority of the soil sample under investigation is highly plastic inorganic clay and inorganic silt (MH, CH and CL). That means inorganic silt with low to high plasticity.

Figure 4.12 and 4.13 show the three representative undisturbed sample plot of vertical effective stress Vs void ratio on semi-log and linear scale respectively. Except their variation in initial void ratio the plot shows similar curvature for all the samples. The soil has a pre consolidation pressure of 80-200kPa. Over-consolidation ratios of the soils are more than one, so the soil in the study area is over consolidated in its natural state. It was also recognized larger amount of deformation for steps of higher loadings, especially for loadings beyond Pc.

The coefficient of consolidation, C_v, which is calculated from curve of compression dial reading versus square root of time for each incremental loading is plotted as a function of effective stress in Figure 4.15. From this figure it can be observed that, the shapes of the curves for three samples are almost similar and the same is true for value of coefficient of consolidation for each incremental loading. But there is slightly different in shape and value of coefficient of consolidation for each incremental loading. This is because the compressibility of any soil type varies with density, history of previous loading, handling prior to and during compression, and in the magnitude of stress increment relative to the existing loading any point.

The compression and recompression index of the soils is calculated from the straight portions of the loading and unloading e-log p curve (Figure 4.12), the typical loading unloading curve as shown in Fig 4.14. This calculation shows that the compression index, Cc, ranges from 0.14-0.32 recompression index, Cr, from 0.003-0.07.

The coefficient of permeability of soil under investigation which is calculated from the test results of consolidation test ranges from 10^{-7} to 10^{-9} cm/sec (table 4.13). The result shows that the soil under investigation is practically impermeable to very low permeability. As shown in fig 4.16 the plot of void ratio versus log permeability coefficient shows straight line.

5.2. INDEX PROPERTY TEST RESULTS IN DIFFERENT PARTS OF THE COUNTRY.

The laboratory test results of this investigation can be compared with the other research data as shown in the Table 5.1.

The Specific Gravity lies in the range between 2.69 to 2.84, Clay Content lies in the range 50% to 72%, Plasticity Index ranges between 14% to 40%, and those values are similar to the results obtained by Morin [16] and a previous study carried out in Addis Ababa [19].

As shown in Table 5.1, the ranges of values are close to the results obtained by Morin [16] and previous research on red clay soil [19].

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

	Morin & Parry	Previous Research (Fasil A.,2003)	Previous Research (Haile Mariam,1992)	Previous Research (Samuel,1989)			Previous Research (Adiszemen,2005)	Current Research
Soil type	Red clay	Red clay	Red clay	Red clay			Black expansive soils	Red clay soil
Location	Ethiopia	Bahir Dar	Addis Ababa	Addis Ababa			Gondor	Debre Markos
				Kolfe	Semen Gebeya	Rufael		
Clay Content (%)	34-76	74.4-81.8	48-73	58-70	53-68	50-70	41.6-82.25	50 – 72
Activity							0.76-1.47	
Liquid Limit (%)	44-66	60-68	54-81	61-75	57-76	56-75	68.89-110.2	45-68
Plasticity Index (%)	14-30	25 - 31	21-30	30-43	33-47	29-41	45.85-78.66	14-40
Shrinkage limit (%)		14 - 18	14-22	15-21	14-25	14-20	2.65-11.56	
Free swell (%)		8 - 13	10.0-40.0	15-45	15-50	30-40	-	30-180
Specific gravity	2.61-2.90	2.75 – 2.83	2.61-2.79	2.66-2.73	2.70-2.77	2.66-2.74		2.69- 2.84
From plasticity chart				CH	CH	CH		MH, CH, CL

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusion

1. The grain size analysis test result showed that the dominant proportion of soil particle in the research area is clay, which have clay content ranging from 50 % – 73% , silt faction 15.88% – 40.21 % and sand fraction 0.36% - 13.28%.

2. From the index property test results the majority soil type of the study area is red clay having a specific gravity ranges from 2.69 to 2.84.

3. All the samples have free swell value of less than 50% except sample from one test pit. This implies large area of the town cover by non expansive red clay soil and only small areas covered with expansive soil following Wiseta River. The majority soil in the study area is non expansive with free swell value ranging from 30-50% and the sample which is found around Debre Markos University have free swell value of 180%.

4. From consistency limit test results the liquid limit of the area ranges 45% – 68%, plastic limit ranges from 18% – 38% and plastic index from 15% – 40%.

5. The values of unconfined compressive strength from UCS Test range from 320.21kPa to 382.96kPa at natural moisture content of 41.2% to 26.7% respectively.

6. From the one-dimensional consolidation test result, compression index, C_c, ranges from 0.14 to 0.32, recompression index, C_r, from 0.003 to 0.07, coefficient of consolidation, C_v, from 0.801 to 6.281 ($x10^{-7}$) m^2/sec, coefficient of permeability from 5.61 $x10^{-9}$ to 6.31$x10^{-11}$ m/sec.

6.2. RECOMMENDATION.

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

2. Red clay soil is the major soil type of Debre Markos town. But there is some area which is covered with black cotton soil so that it is recommended to have a detail investigation of both red clay and black cotton soil.

3. In depth Shear strength property of Debre Markos soil is not studied in this research. Therefore, by studying the detail Shear strength characteristics soils found in Debre Markos, the Correlation of the index property with shear strength parameters may also be done.

4. It is recommended that the above values should be refined with larger size of data.

7. References

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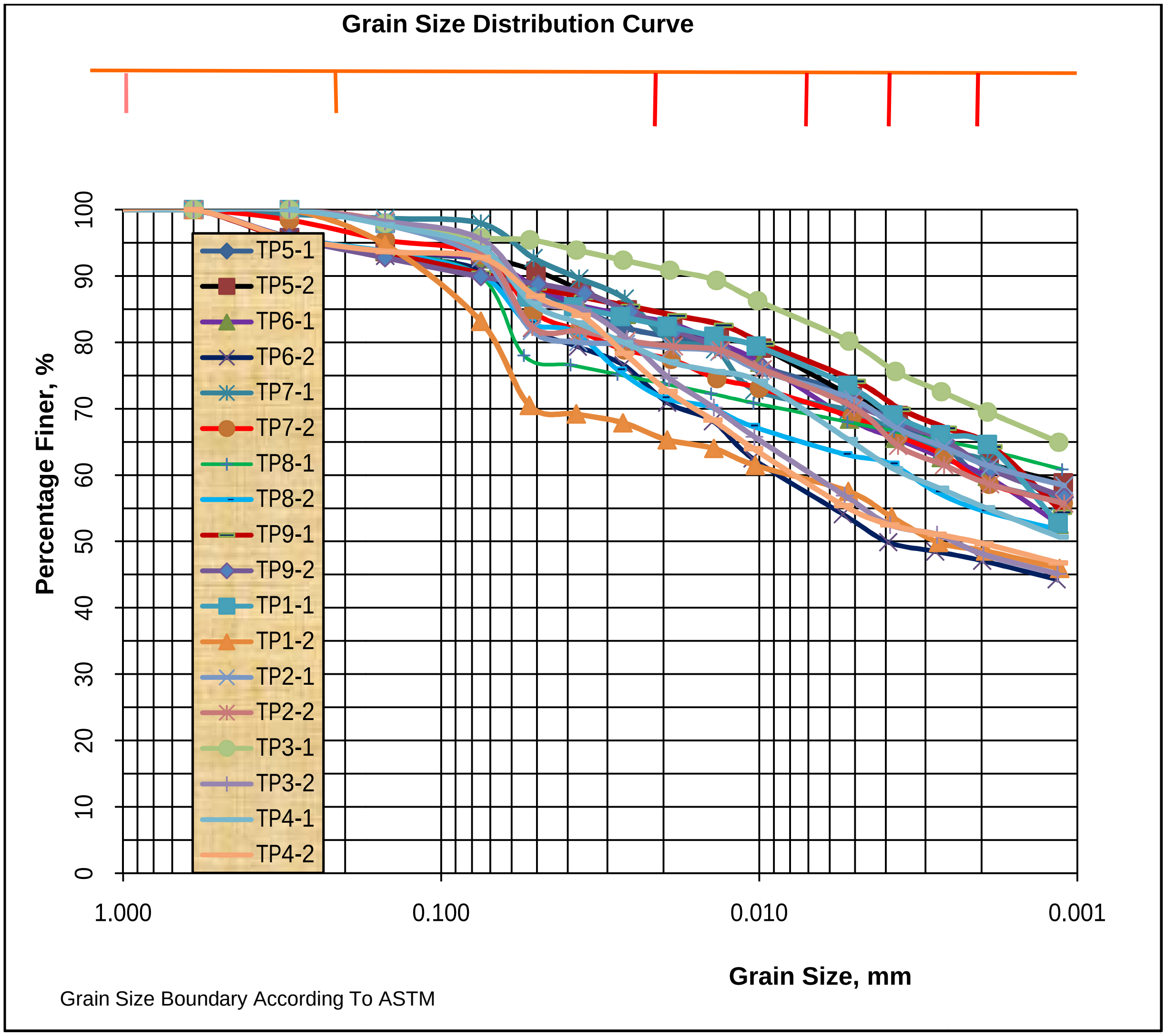
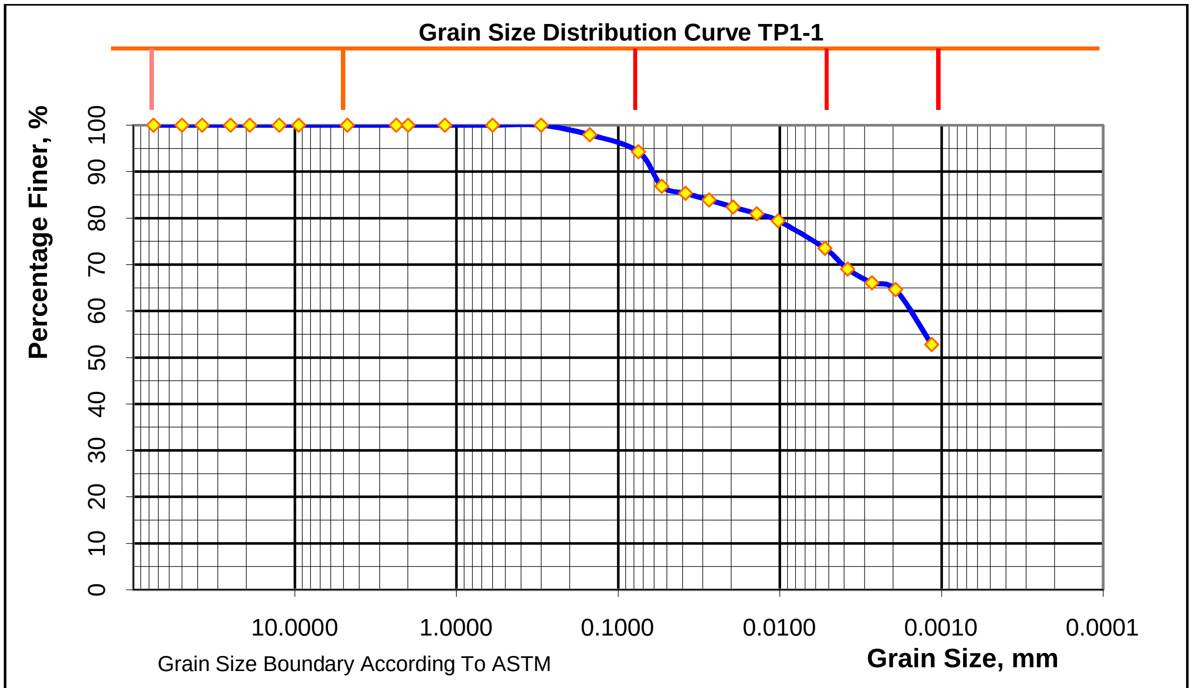
Appendix-A

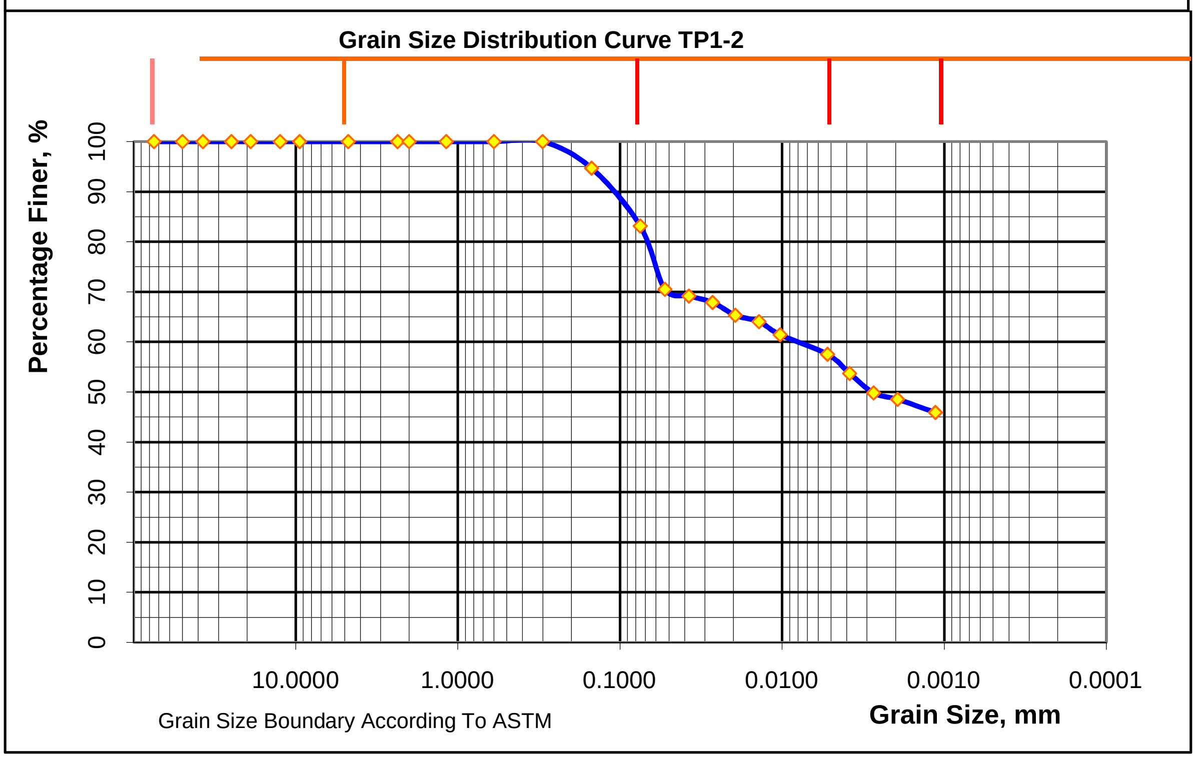
(Laboratory tests)

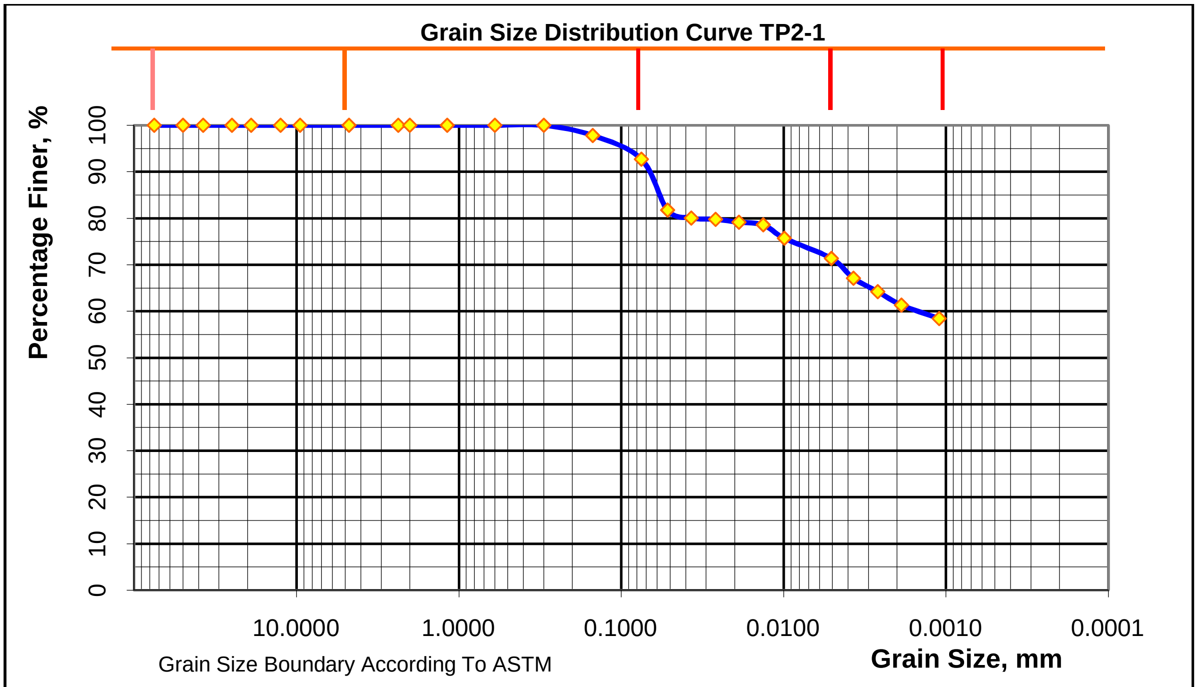
Appendix A.1. Particle Size Distribution Curves:

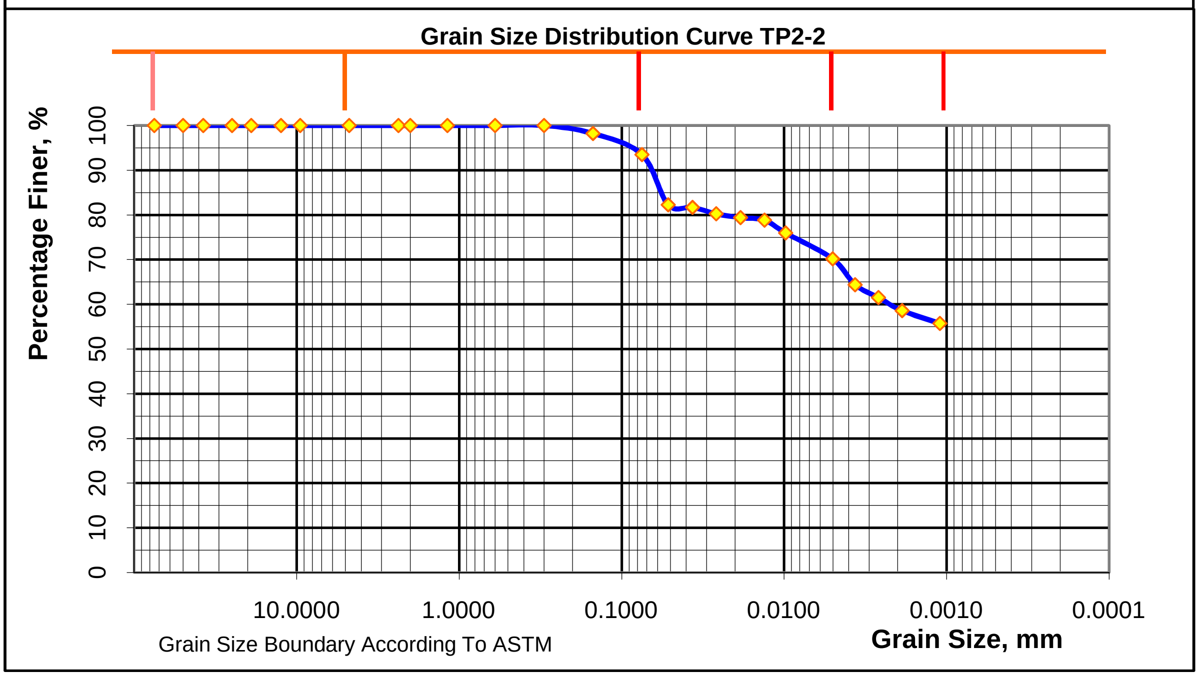


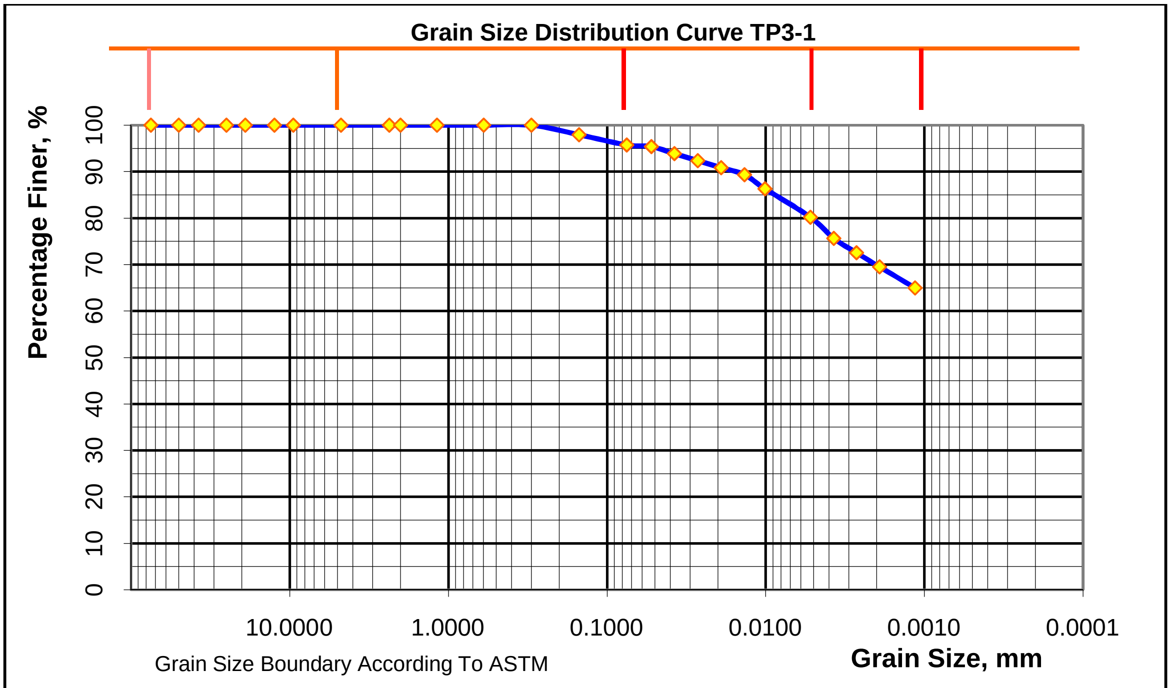
coarse	medium	fine	coarse	medium	fine	Clay
Sand			Silt			

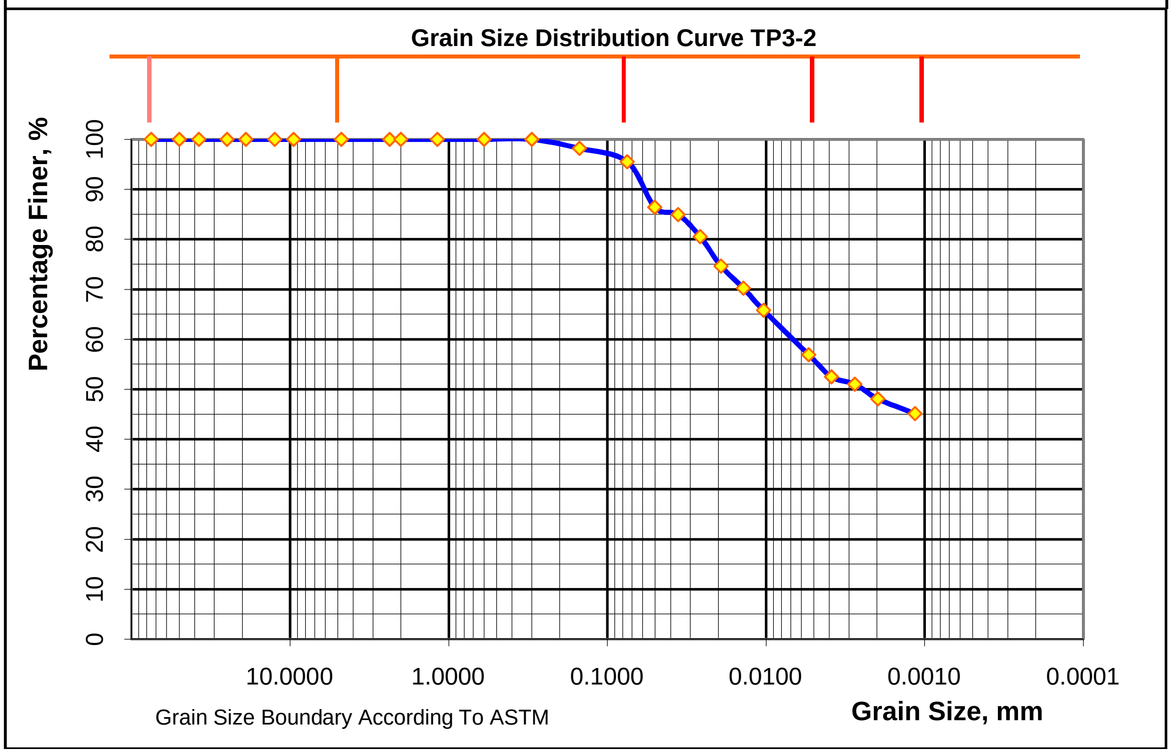

Grain Size Distribution Curve TP1-1
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm

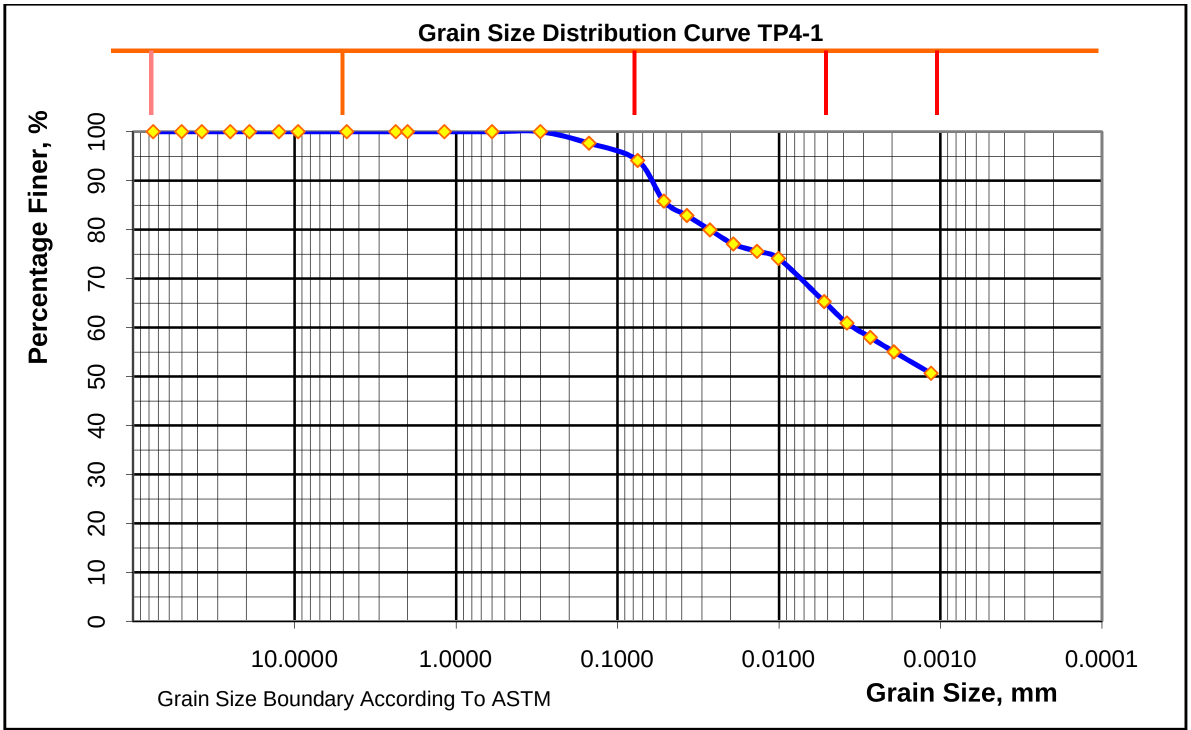

Grain Size Distribution Curve TP1-2
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm

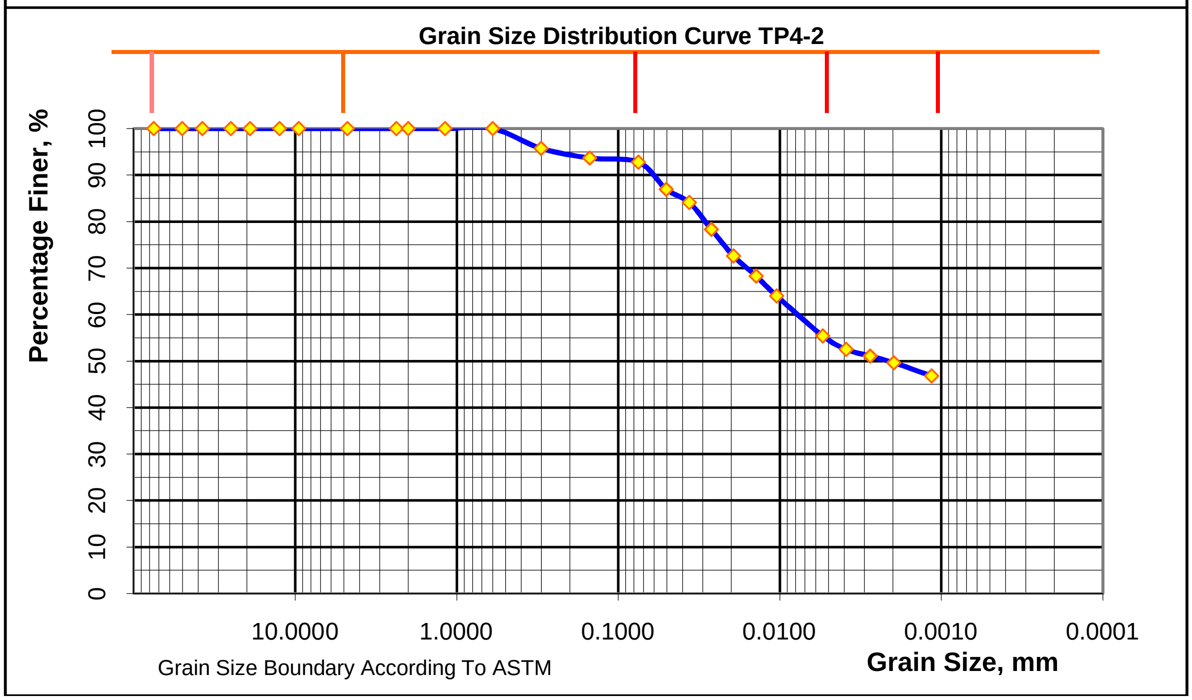

Grain Size Distribution Curve TP2-1
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP2-2
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP3-1
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP3-2
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP4-1
Percentage Finer, %
0
10
20
30
40
50
60
70
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP4-2
Percentage Finer, %
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20
30
40
50
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70
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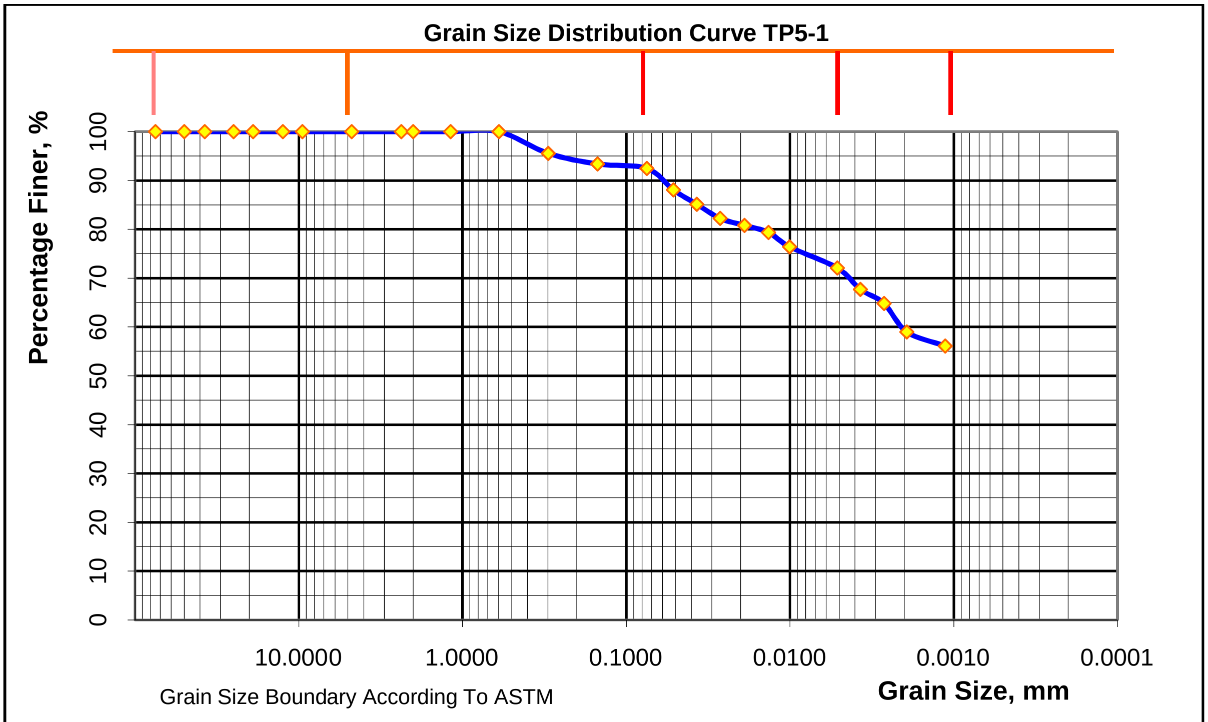
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP5-1
Percentage Finer, %
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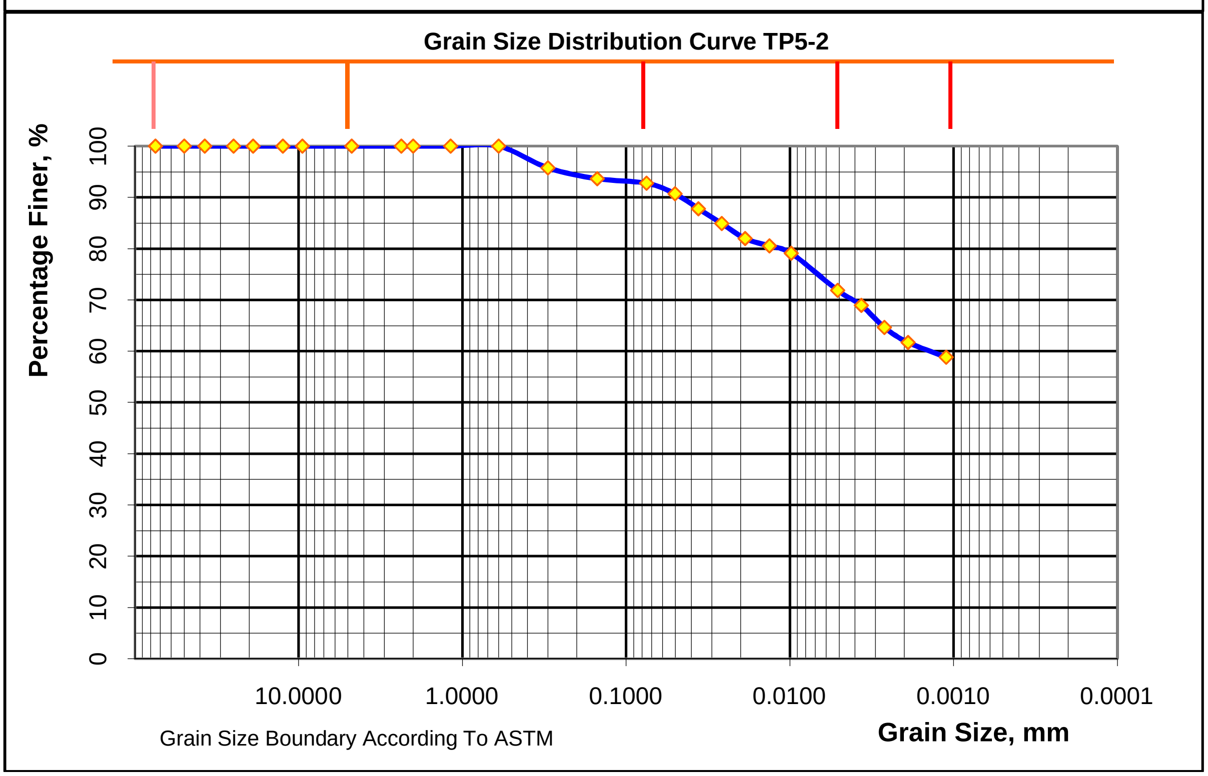
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm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Percentage Finer, %
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30
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10.0000
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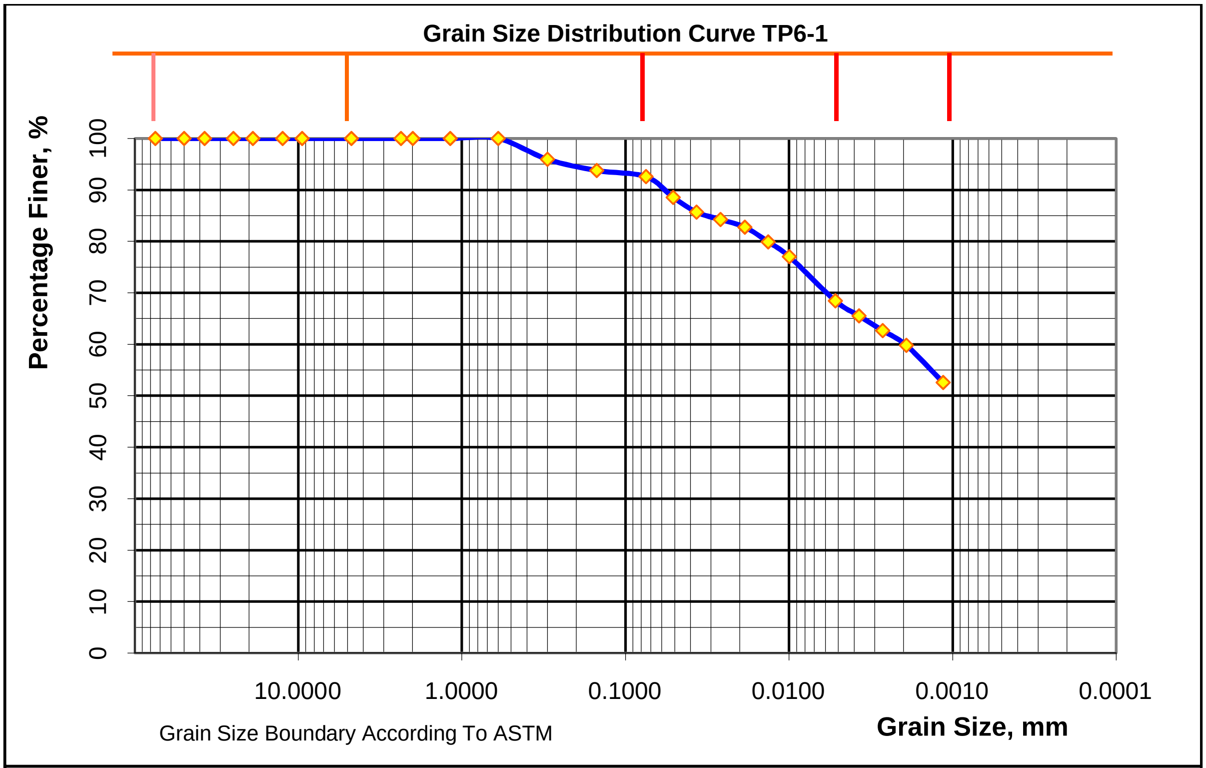
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP6-1
Percentage Finer, %
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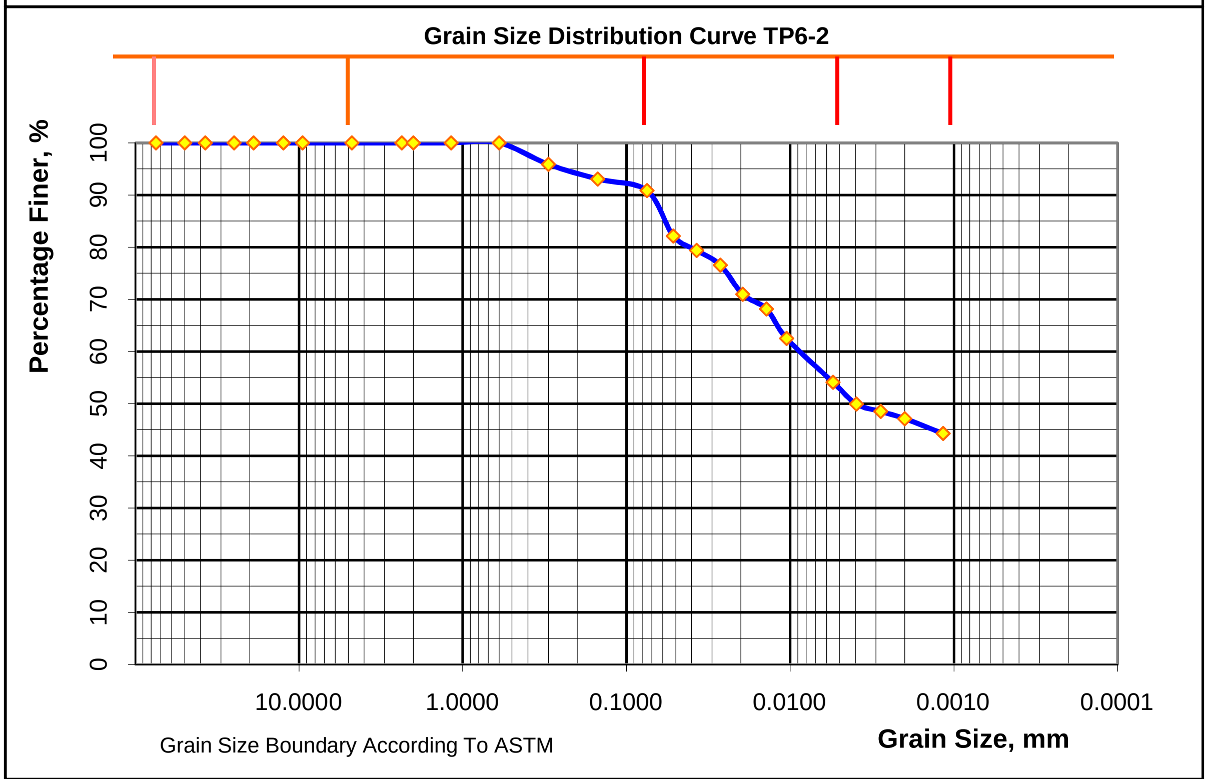
Grain Size Boundary According To ASTM
Grain Size, mm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Percentage Finer, %
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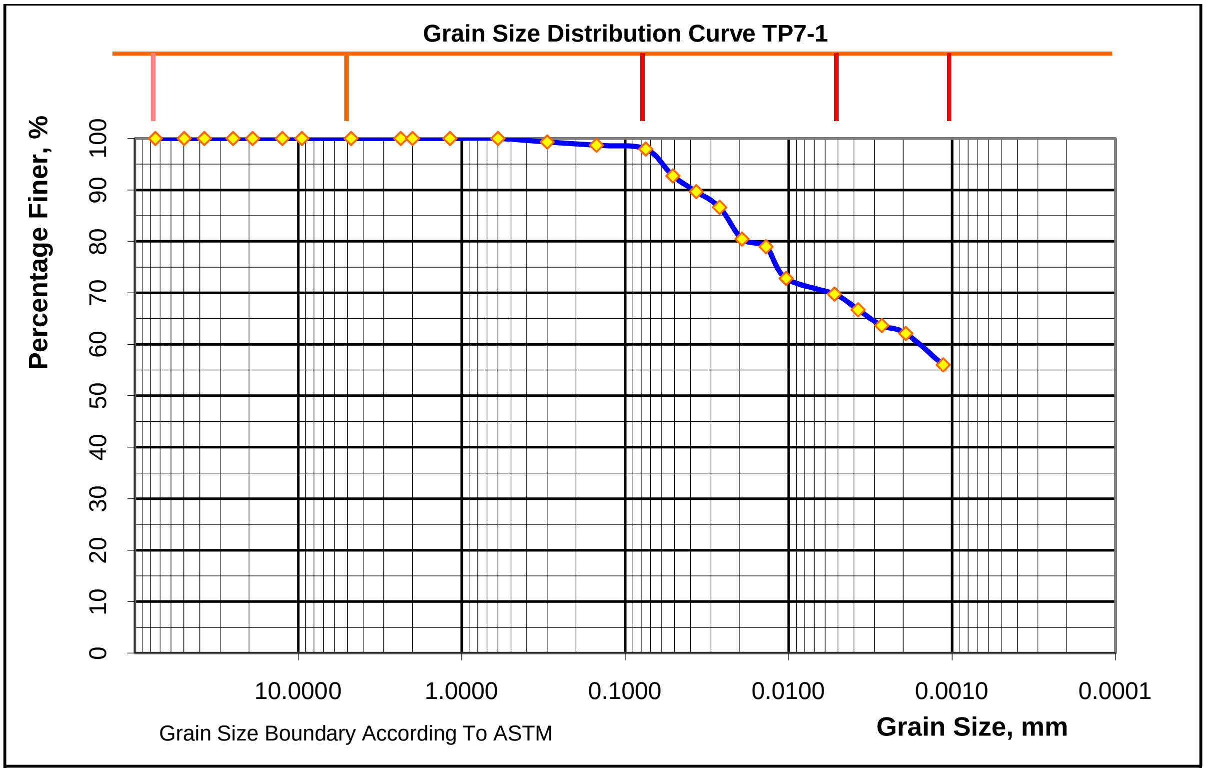
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP7-1
Percentage Finer, %
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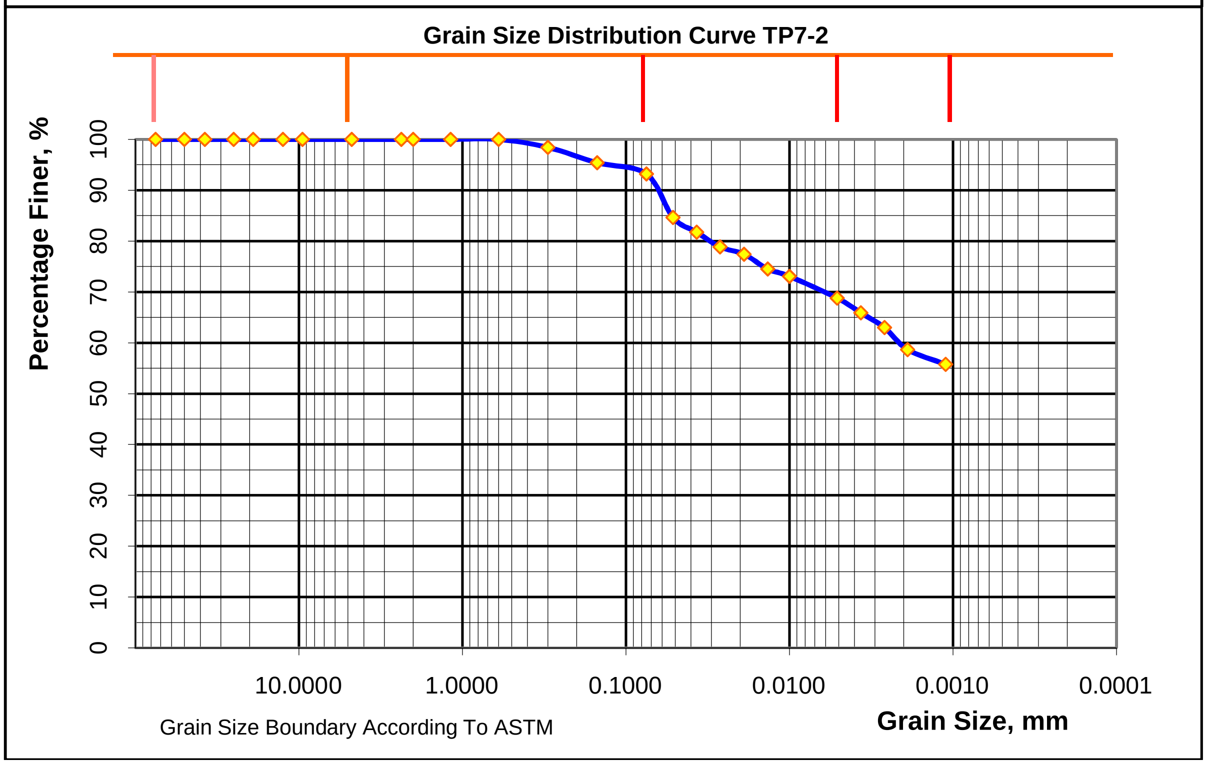
Grain Size Boundary According To ASTM
Grain Size, mm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Percentage Finer, %
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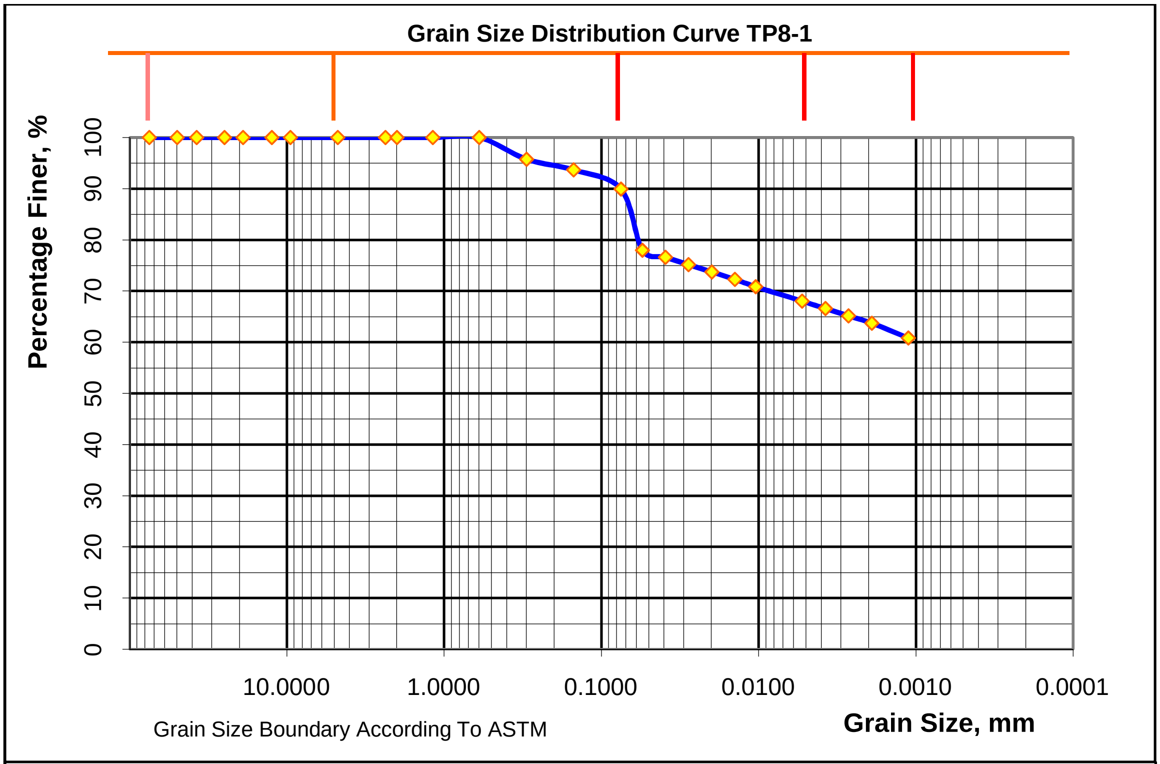
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP8-1
Percentage Finer, %
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0.1000
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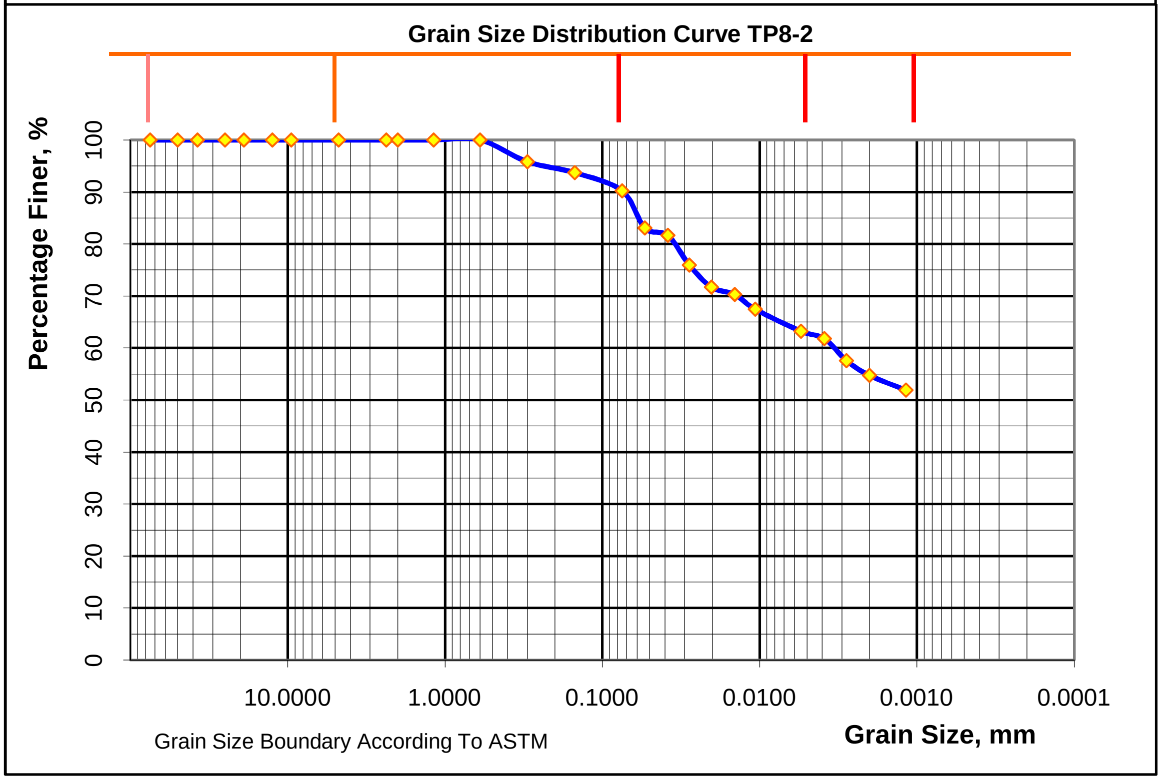
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP8-2
Percentage Finer, %
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10
20
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1.0000
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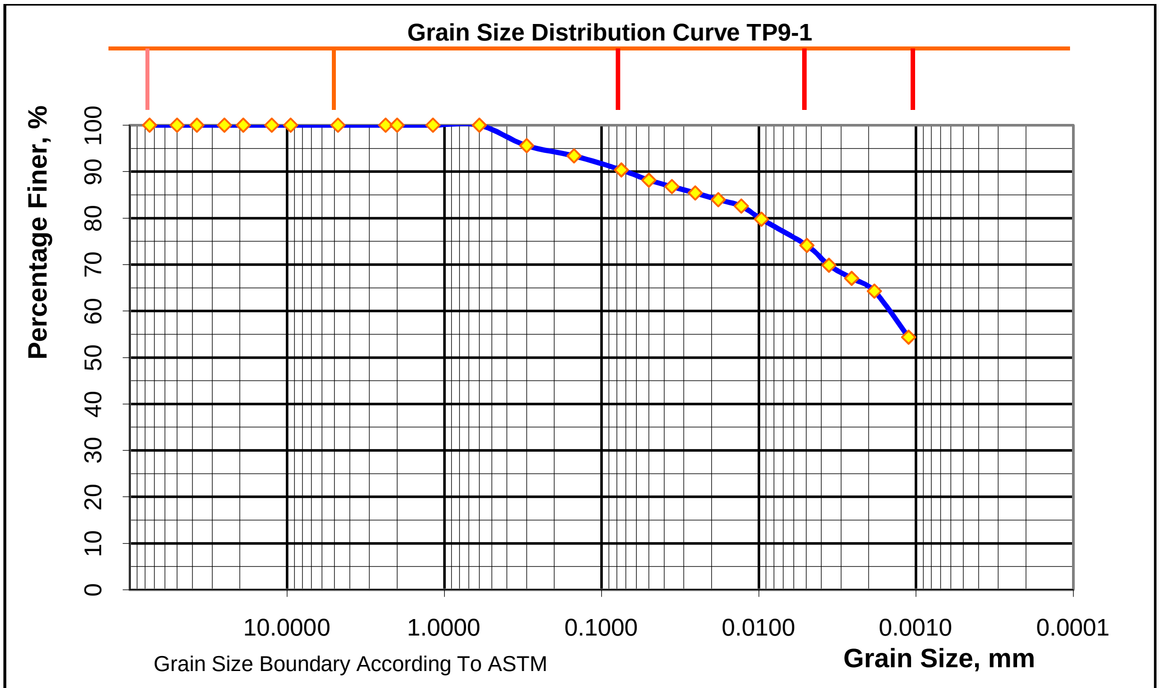
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP9-1
Percentage Finer, %
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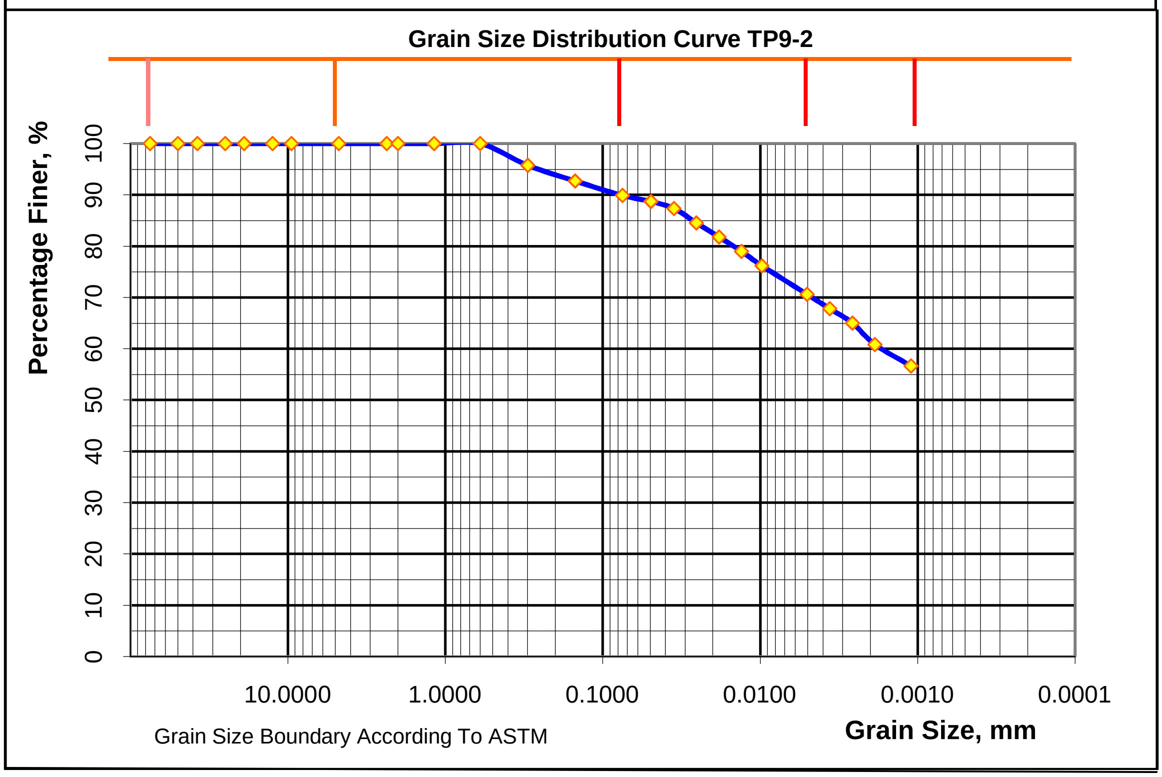
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm


Grain Size Distribution Curve TP9-2
Percentage Finer, %
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10
20
30
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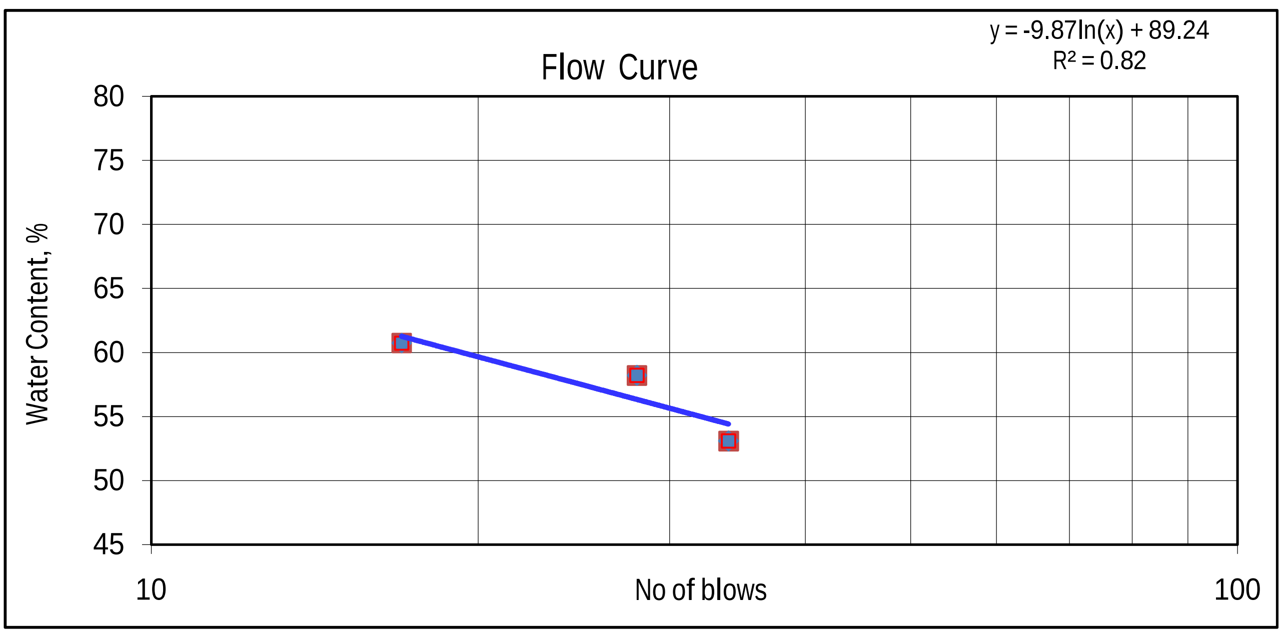
80
90
100
10.0000
1.0000
0.1000
0.0100
0.0010
0.0001
Grain Size Boundary According To ASTM
Grain Size, mm

Appendix A.2. Atterberg Limits Determination

Liquid Limit and Plastic Limit Test

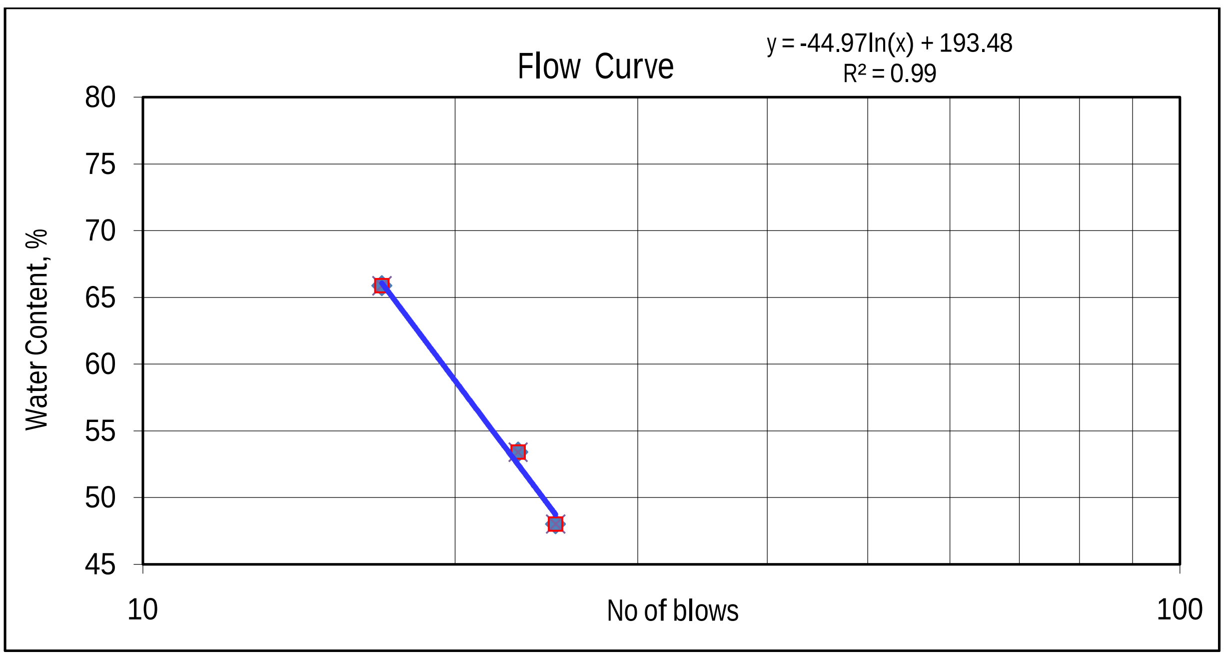
Project	Thesis research		Date	2/6/2012
Sample No.	TP1-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	3	
Container NO.	H2	B7	D4	15.8	15.9	
Mass of container (m1)g	9.3	16	15.9	9.3	9.4	
Mass of container + wet soil (m2)g	23.2	31.9	33.1	11.4	11.1	
Mass of container + dry soil (m3)g	18.38	26.05	26.6	10.9	10.65	
Mass of water (m2-m3)g	4.82	5.85	6.5	0.5	0.45	
Mass of dry soil (m3-m1)	9.08	10.05	10.7	1.6	1.25	
Water content (m2-m3)/m3-m1)*100%	53.08	58.21	60.75	31.25	36.00	
Number of blows	34	28	17			
	LL(%)=	58		PL(%)	33.6	24.4



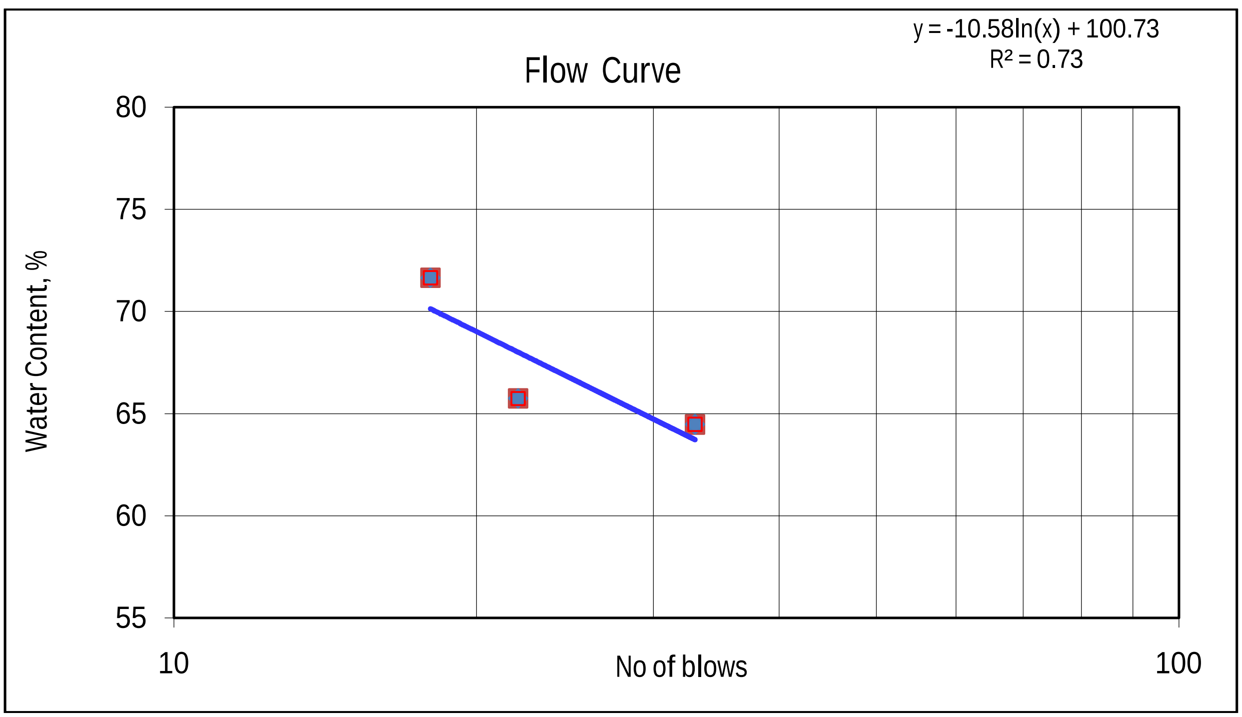
Project	Thesis research		Date	2/6/2012
Sample No.	TP1-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	C4	C6	C1	A2	A4	
Mass of container	9.5	9.4	16	9.6	9.5	
Mass of container + wet soil	23	22.1	30.1	12.4	11.9	
Mass of container + dry soil	18.3	17.98	24.5	11.7	11.3	
Mass of water	4.7	4.12	5.6	0.7	0.6	
Mass of dry soil	8.8	8.58	8.5	2.1	1.8	
Water content	53.4	48.0	65.9	33.3	33.3	
Number of blows	23	25	17		**33.333**	
	LL(%)=	48		PL(%)	33.3	14.7



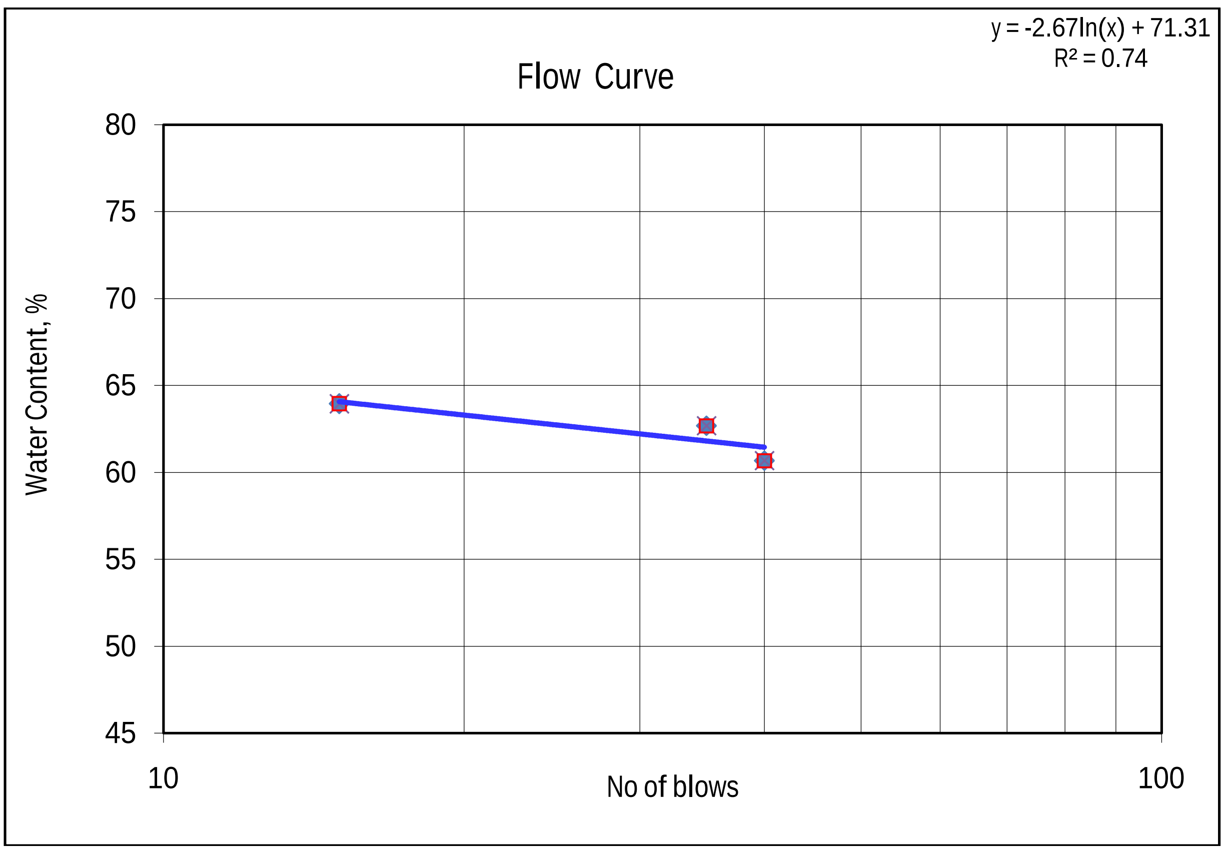
Project	Thesis research		Date	2/6/2012
Sample No.	TP2-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	E	J	
Mass of container	9.5	15.9	15.65	15.9	16	
Mass of container + wet soil	22	28	26.55	17.6	18.1	
Mass of container + dry soil	17.1	23.2	22	17.25	17.5	
Mass of water	4.9	4.8	4.55	0.35	0.6	
Mass of dry soil	7.6	7.3	6.35	1.35	1.5	
Water content	64.5	65.8	71.7	25.9	40.0	
Number of blows	33	22	18			
	LL(%)=	67		PL(%)	33.0	34.0



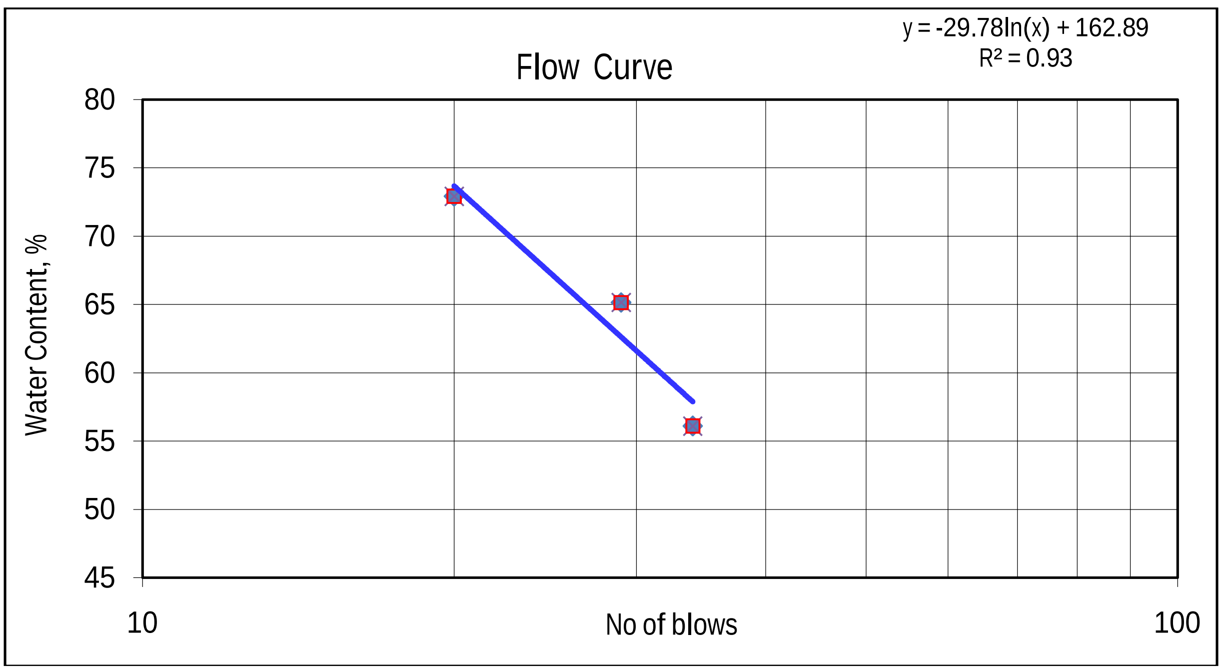
Project	Thesis research		Date	2/6/2012	
Sample No.	TP2-2		Depth	3m	

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	E	J	
Mass of container	15.62	15.8	15.75	15.6	15.8	
Mass of container + wet soil	27.72	28.1	30.67	19.5	19.1	
Mass of container + dry soil	23.15	23.36	24.85	18.5	18.25	
Mass of water	4.57	4.74	5.82	1.0	0.9	
Mass of dry soil	7.53	7.56	9.1	2.9	2.45	
Water content	60.7	62.7	64.0	34.5	34.7	
Number of blows	40	35	15			
	LL(%)=	63		PL(%)	34.6	28.4



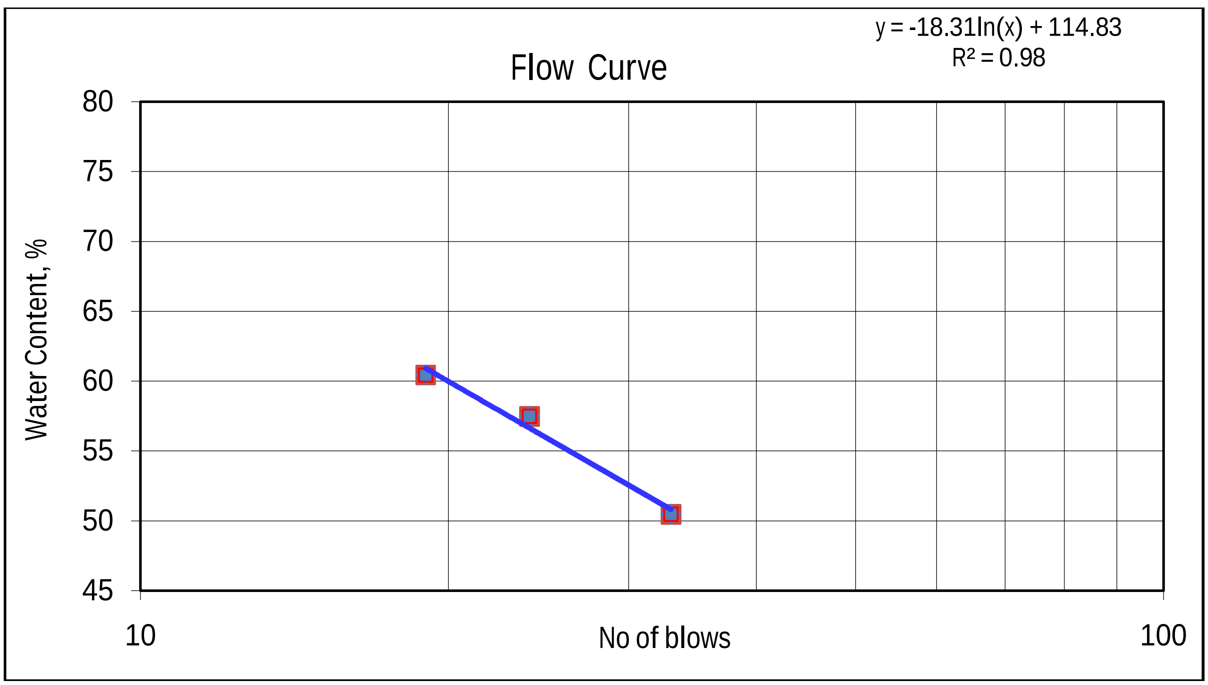
Project Sample No.	Thesis research TP3-1		Date Depth	2/6/2012 1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	A4	C4	C6	15.6	15.7	
Mass of container	9.6	9.5	9.3	9.2	15.7	
Mass of container + wet soil	22.4	20.4	17.6	10.7	17.4	
Mass of container + dry soil	17.8	15.9	14.1	10.4	17	
Mass of water	4.6	4.5	3.5	0.3	0.4	
Mass of dry soil	8.2	6.4	4.8	1.2	1.3	
Water content	56.10	70.31	72.92	25.00	30.77	
Number of blows	34	29	20			
	LL(%)=	68		PL(%)	**27.9**	40.1



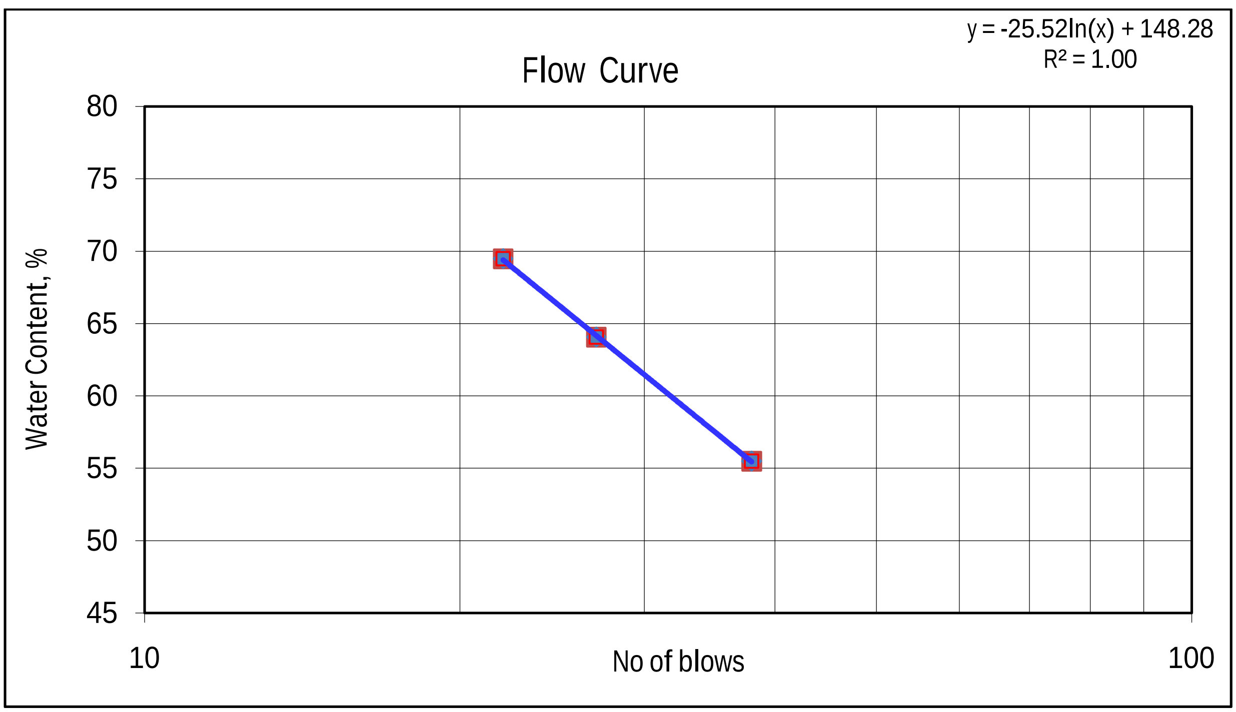
Project	Thesis research		Date	2/6/2012
Sample No.	TP3-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	H	K	
Mass of container	9.5	15.8	15.6	9.2	15.9	
Mass of container + wet soil	25.9	29.5	30.2	11.3	18.9	
Mass of container + dry soil	20.4	24.5	24.7	10.7	18.2	
Mass of water	5.5	5	5.5	0.6	0.7	
Mass of dry soil	10.9	8.7	9.1	1.5	2.3	
Water content	50.46	57.47	60.44	40.00	30.43	
Number of blows	33	24	19			
	LL(%)=	56		PL(%)	**35.2**	20.8



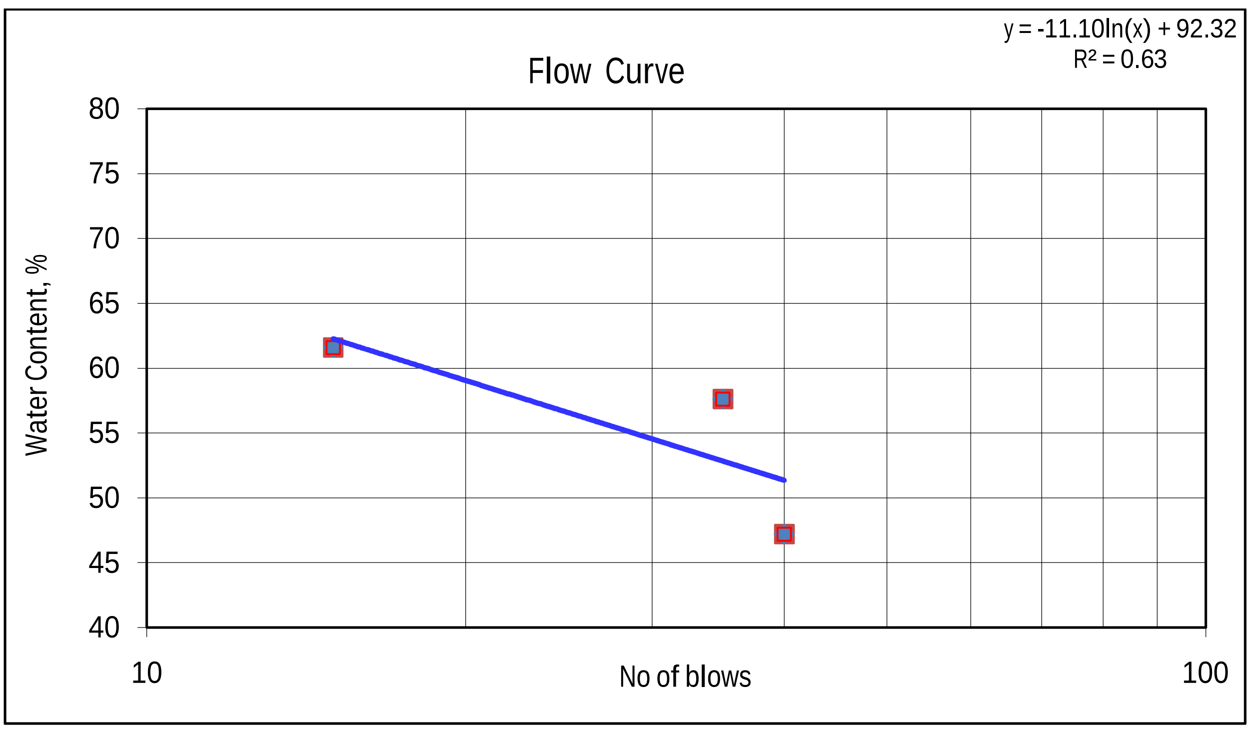
Project	Thesis research		Date	2/6/2012
Sample No.	TP4-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	E	J	
Mass of container	13.56	15.66	15.63	15.32	15.59	
Mass of container + wet soil	32.67	29.49	38.61	17.27	17.87	
Mass of container + dry soil	25.85	24.09	28.97	16.81	17.35	
Mass of water	6.82	5.4	9.64	0.46	0.52	
Mass of dry soil	12.29	8.43	13.34	1.49	1.76	
Water content	55.5	64.1	72.3	30.9	29.5	
Number of blows	38	27	22			
	LL(%)=	66		PL(%)	30.2	35.8



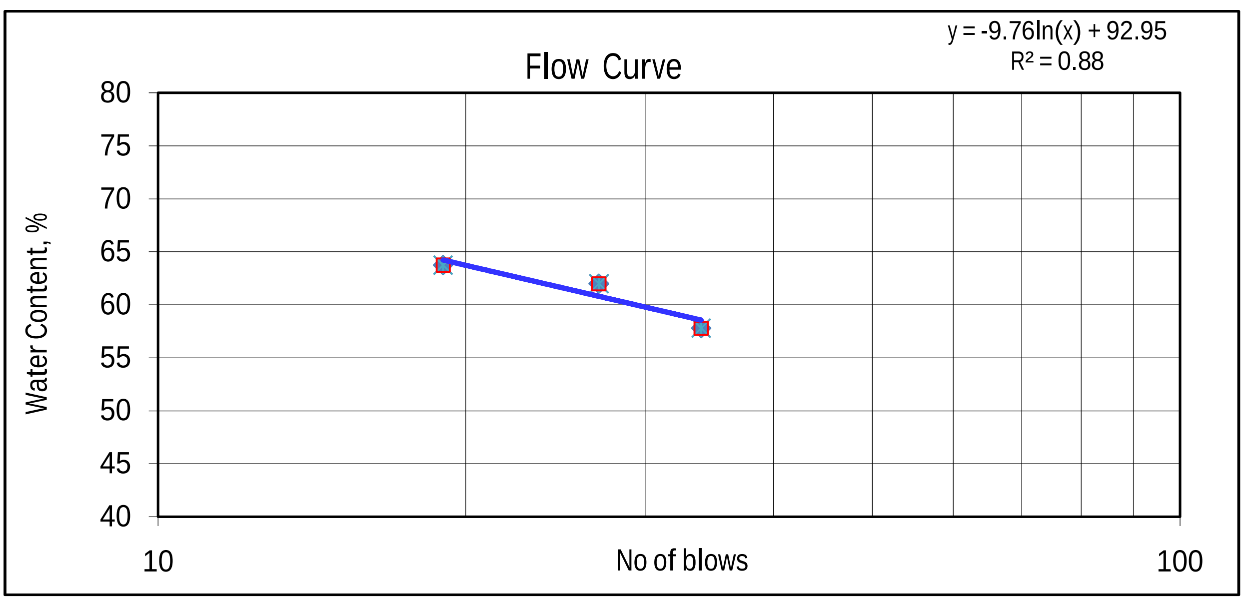
Project	Thesis research		Date	2/6/2012
Sample No.	TP4-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	E	J	
Mass of container	15.71	15.56	15.57	15.67	15.45	
Mass of container + wet soil	36.47	39.16	38.42	17.66	18.02	
Mass of container + dry soil	28.88	31.59	29.71	17.15	17.4	
Mass of water	7.59	7.57	8.71	0.51	0.62	
Mass of dry soil	13.17	16.0	14.14	1.48	1.95	
Water content	57.6	47.2	61.6	34.5	31.8	
Number of blows	35	40	15			
	LL(%)=	58		PL(%)	33.1	24.9



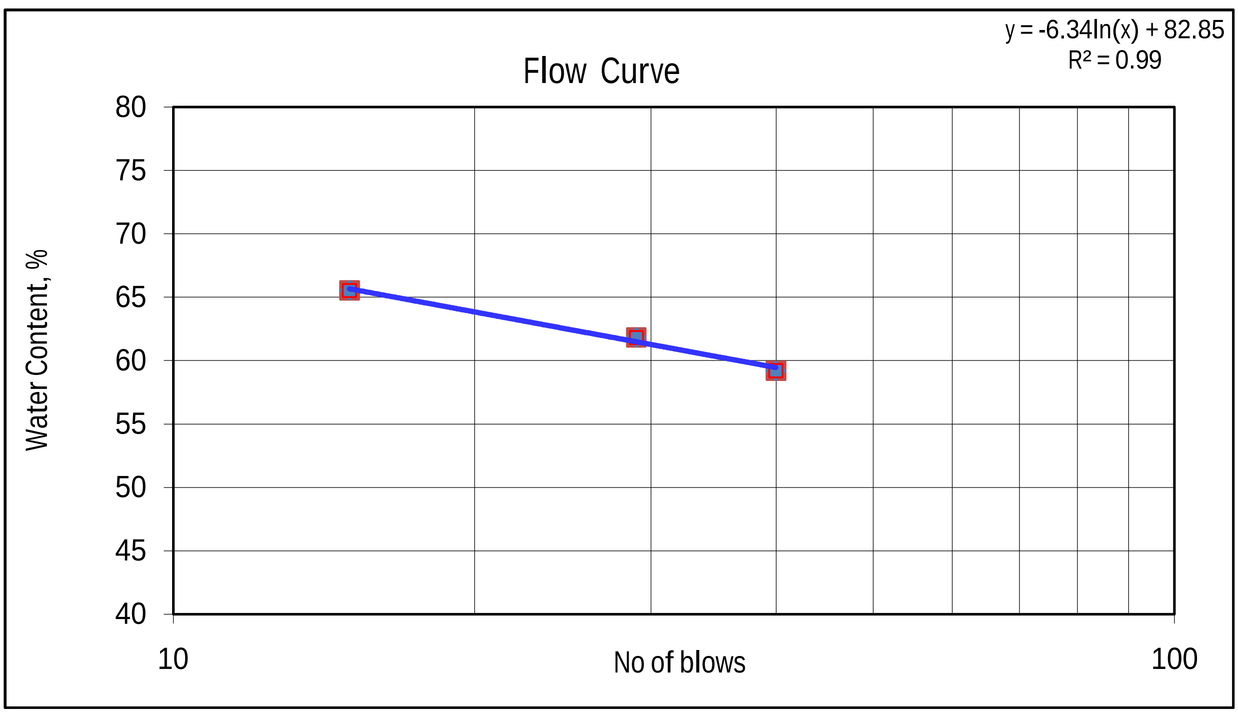
Project	Thesis research		Date	2/6/2012
Sample No.	TP5-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	B1	B2	B3	Y1	Y2	
Mass of container	9.3	9.4	9.2	7.5	7.4	
Mass of container + wet soil	43.7	45.2	40.8	14.8	15.3	
Mass of container + dry soil	31.1	31.5	28.5	13.2	13.6	
Mass of water	12.6	13.7	12.3	1.6	1.7	
Mass of dry soil	21.8	22.1	19.3	5.7	6.2	
Water content	57.8	62.0	63.7	28.1	27.4	
Number of blows	34	27	19			
	LL(%)=	62		PL(%)	27.7	34.3



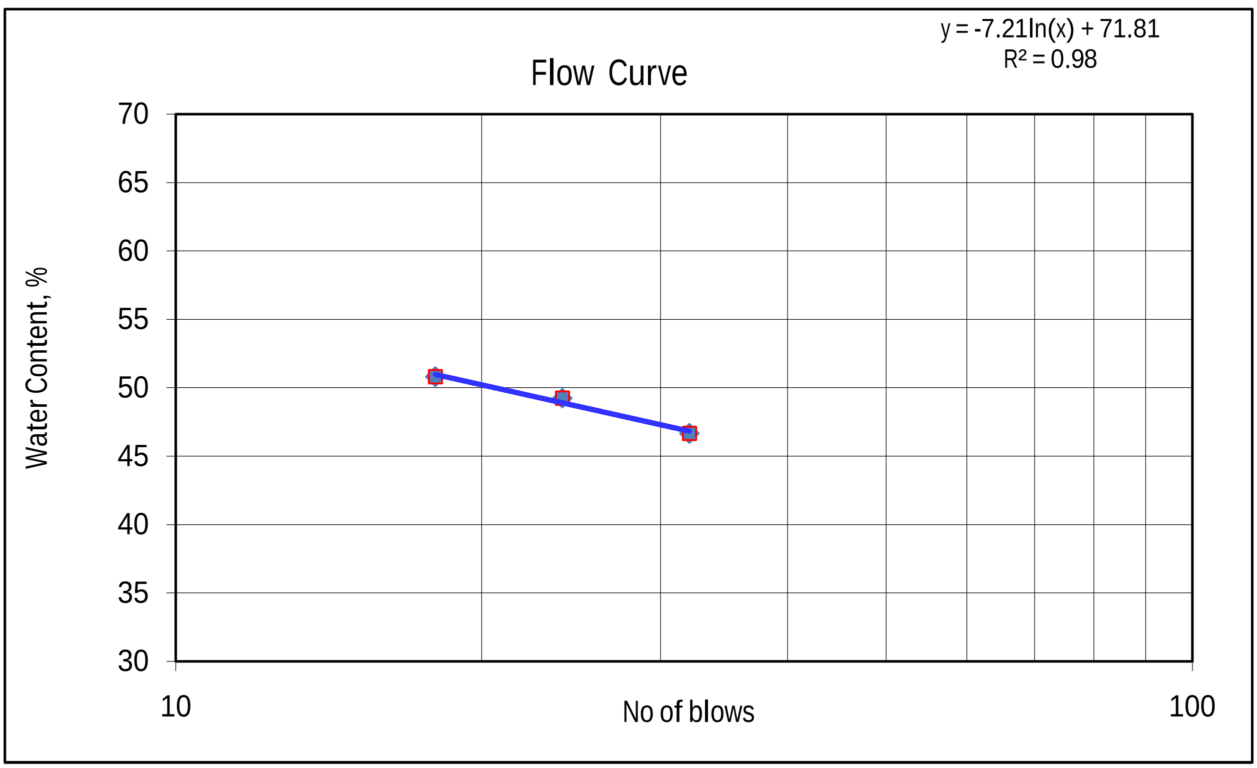
Project	Thesis research		Date	2/6/2012
Sample No.	TP5-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	B5	B4	B5	Y3	Y6	
Mass of container	15.6	15.8	15.8	7.3	7.1	
Mass of container + wet soil	27.7	28.1	30.7	15.9	16.3	
Mass of container + dry soil	23.2	23.4	24.8	14.1	14.3	
Mass of water	4.5	4.7	5.9	1.8	2.0	
Mass of dry soil	7.6	7.6	9	6.8	7.2	
Water content	59.2	61.8	65.6	26.5	27.8	
Number of blows	40	29	15			
	LL(%)=	63		PL(%)	27.1	35.9



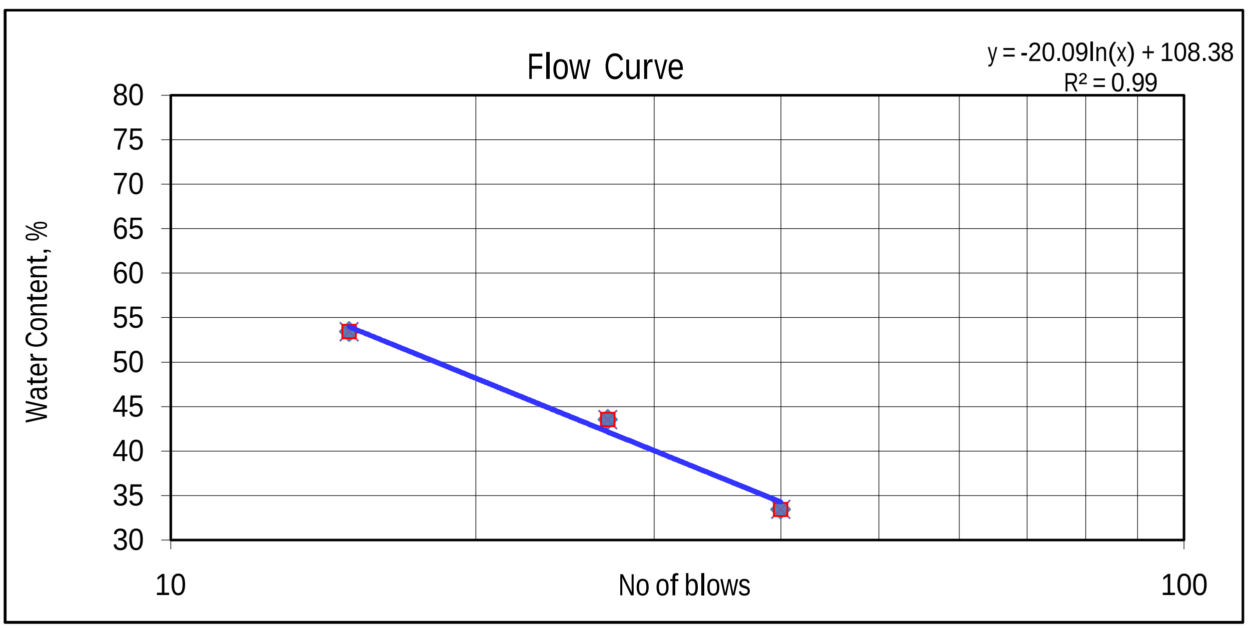
Project	Thesis research		Date	2/6/2012
Sample No.	TP6-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	B1	B2	B5	E	F	
Mass of container	9.2	9.4	9.2	7.4	7.3	
Mass of container + wet soil	42.2	39.4	46.3	10.6	10.7	
Mass of container + dry soil	31.7	29.5	33.8	10.2	10.1	
Mass of water	10.5	9.9	12.5	0.4	0.6	
Mass of dry soil	22.5	20.1	24.6	2.8	2.8	
Water content	46.7	49.3	50.8	14.3	21.4	
Number of blows	32	24	18			
	LL(%)=	48.80		PL(%)	17.9	31



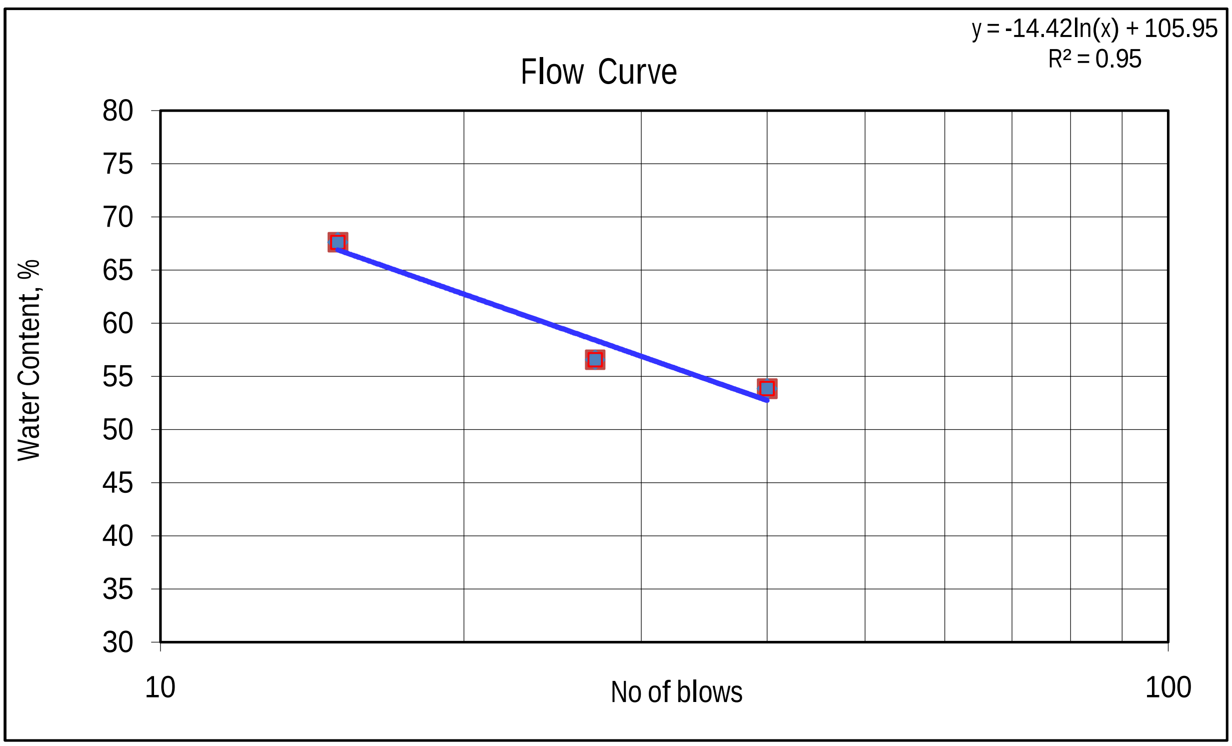
Project	Thesis research		Date	2/6/2012
Sample No.	TP6-2		Depth	3.0m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	B5	B3	B1	A	B	
Mass of container	10.1	10.1	10.1	7.2	7.2	
Mass of container + wet soil	43.2	36.8	39.1	12.5	12.1	
Mass of container + dry soil	34.9	28.7	29	11.7	11.2	
Mass of water	8.3	8.1	10.1	0.8	0.9	
Mass of dry soil	24.8	18.6	18.9	4.5	4	
Water content	33.5	43.5	53.4	17.78	22.50	
Number of blows	40	27	15			
	LL(%)=	45.35		PL(%)	20.1	25



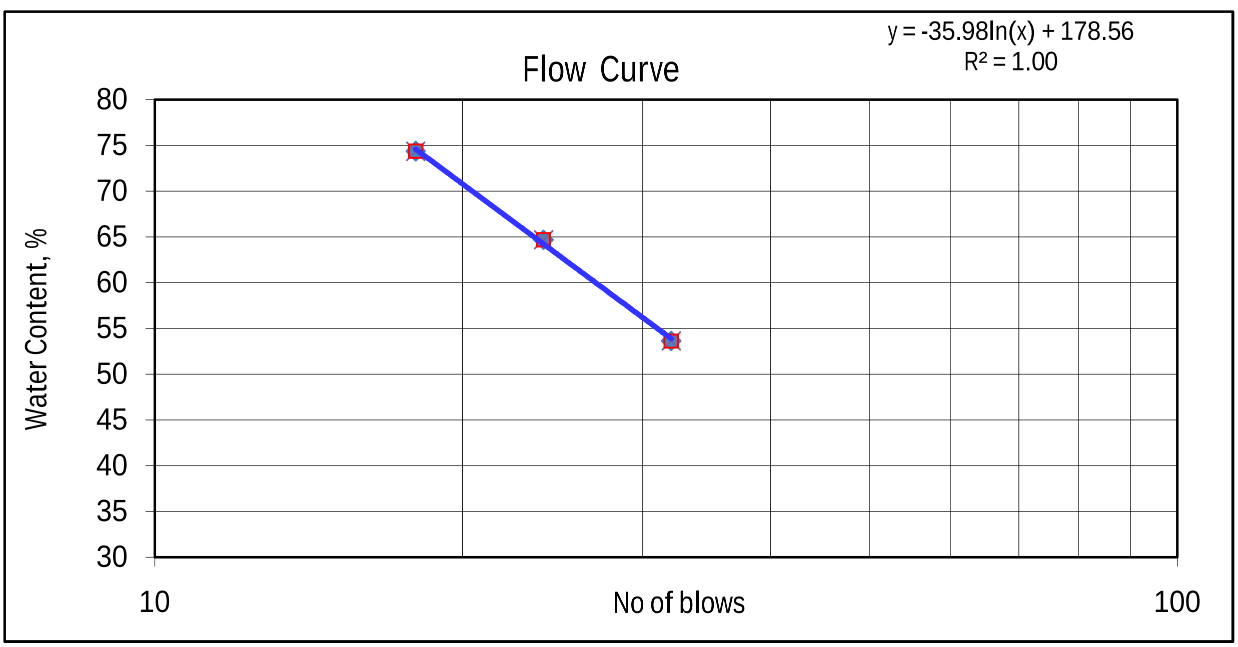
Project	Thesis research		Date	2/6/2012
Sample No.	TP7-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	AA1	AA2	AA3	B1	B2	
Mass of container	14.52	15.6	15.75	7.3	7.6	
Mass of container + wet soil	27.72	29.3	30.87	14.4	14.9	
Mass of container + dry soil	23.1	24.35	24.77	12.9	13.24	
Mass of water	4.62	4.95	6.1	1.5	1.66	
Mass of dry soil	8.58	8.75	9.02	5.6	5.64	
Water content	53.8	56.6	67.6	26.79	29.43	
Number of blows	40	27	15			
	LL(%)=	61		PL(%)	28.1	32.9



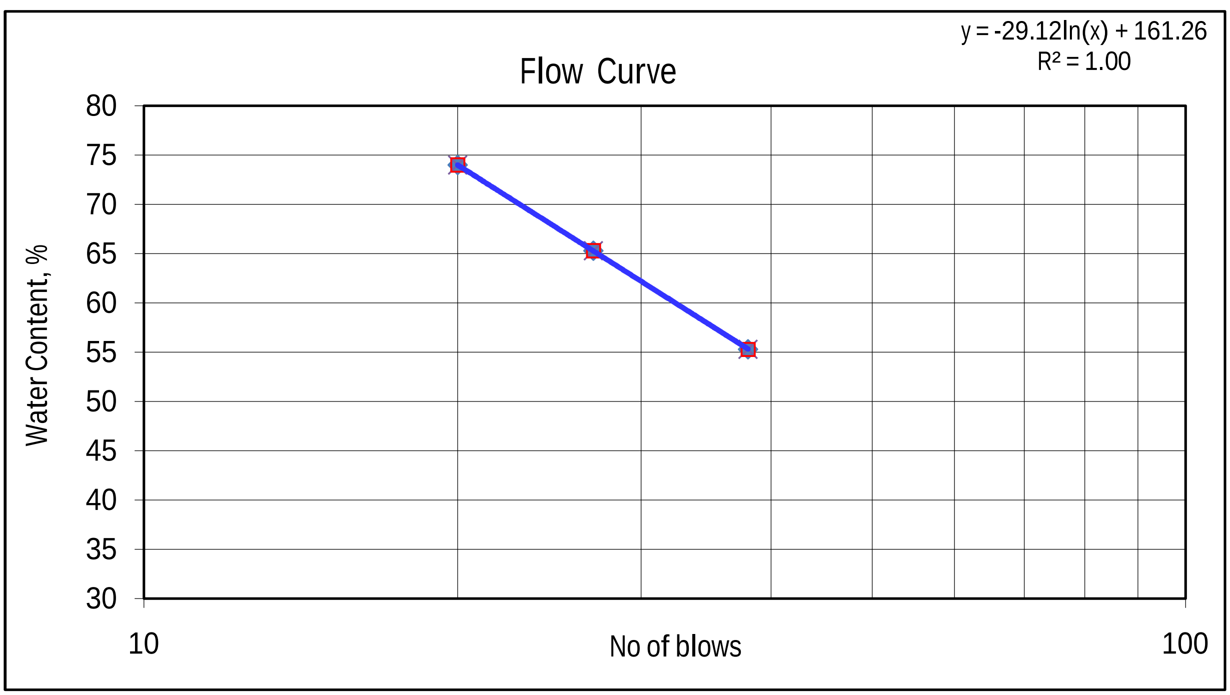
Project	Thesis research		Date	2/6/2012
Sample No.	TP7-2		Depth	3.0m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	DE1	DE2	DE3	D1	D2	
Mass of container	9.7	10.5	13.2	7.9	8.1	
Mass of container + wet soil	39.2	43.6	46.5	12.3	15	
Mass of container + dry soil	28.9	30.6	32.3	11.4	13.4	
Mass of water	10.3	13	14.2	0.9	1.6	
Mass of dry soil	19.2	20.1	19.1	3.5	5.3	
Water content	53.6	64.7	74.3	25.7	30.2	
Number of blows	32	24	18			
	LL(%)=	64		PL(%)	28.0	36.0



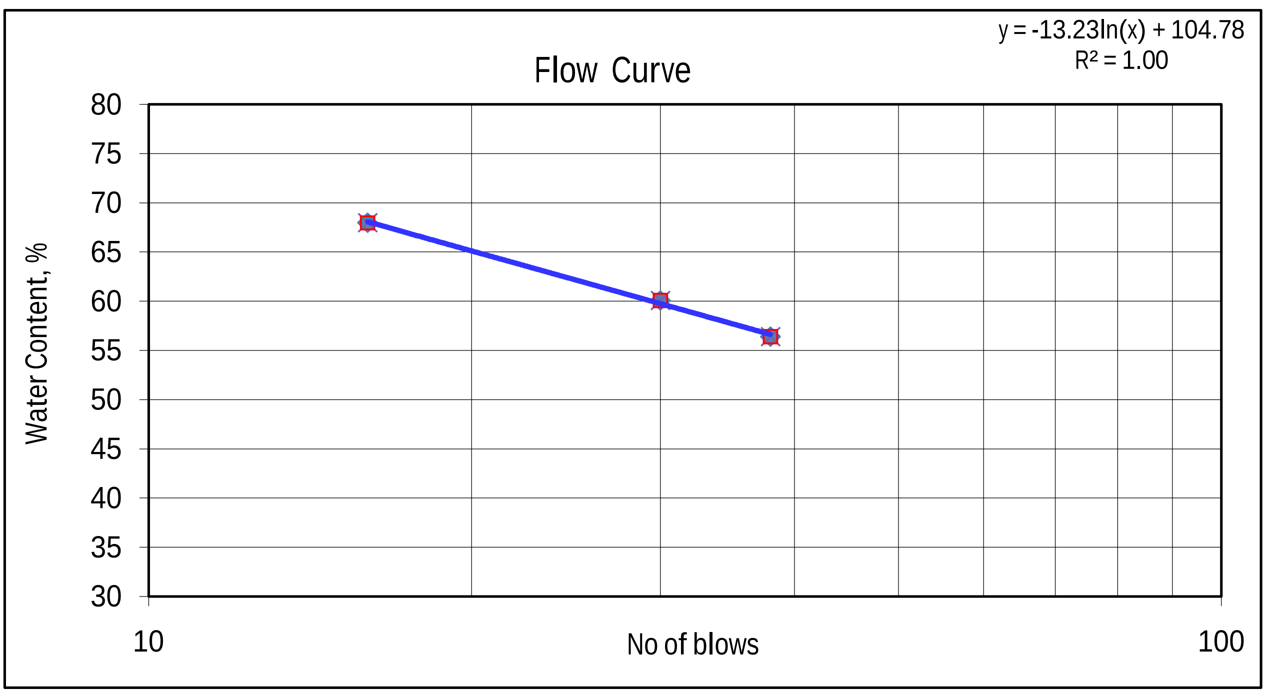
Project	Thesis research		Date	2/6/2012
Sample No.	TP8-1		Depth	1.5m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	m1	m2	m3	a	c	
Mass of container	10	10	10	8.2	7.4	
Mass of container + wet soil	43.7	46.2	48.8	13.6	15.9	
Mass of container + dry soil	31.7	31.9	32.3	12.2	13.6	
Mass of water	12	14.3	16.5	1.4	2.30	
Mass of dry soil	21.7	21.9	22.3	4	6.2	
Water content	55.3	65.3	74.0	35.0	37.1	
Number of blows	38	27	20			
	LL(%)=	68		PL(%)	36.0	32.0



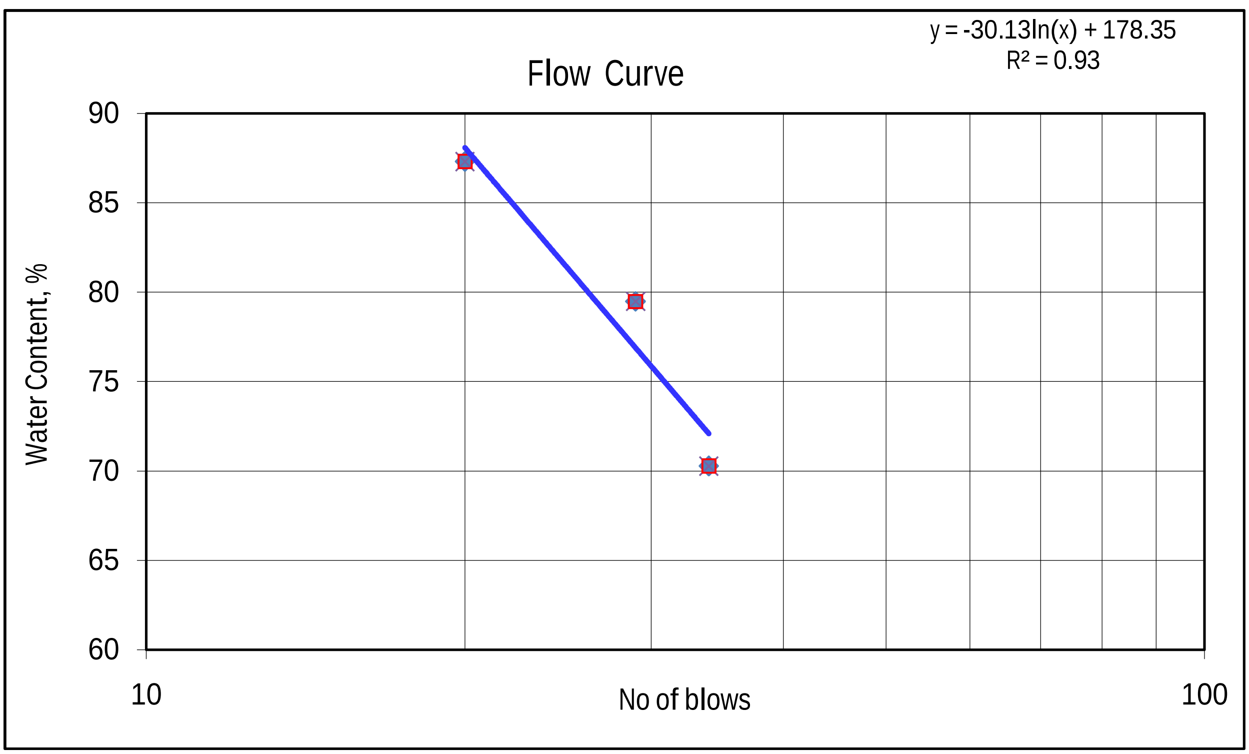
Project	Thesis research		Date	2/6/2012
Sample No.	TP8-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	bb1	bb2	bb3	d	g	
Mass of container	9.3	9.4	9.3	7.6	7.3	
Mass of container + wet soil	45.9	49.9	51.3	12.7	13.1	
Mass of container + dry soil	32.7	34.7	34.3	11.4	11.5	
Mass of water	13.2	15.20	17	1.30	1.60	
Mass of dry soil	23.4	25.3	25	3.8	4.2	
Water content	56.4	60.1	68.0	34.2	38.1	
Number of blows	38	30	16			
	LL(%)=	63		PL(%)	36.2	26.8



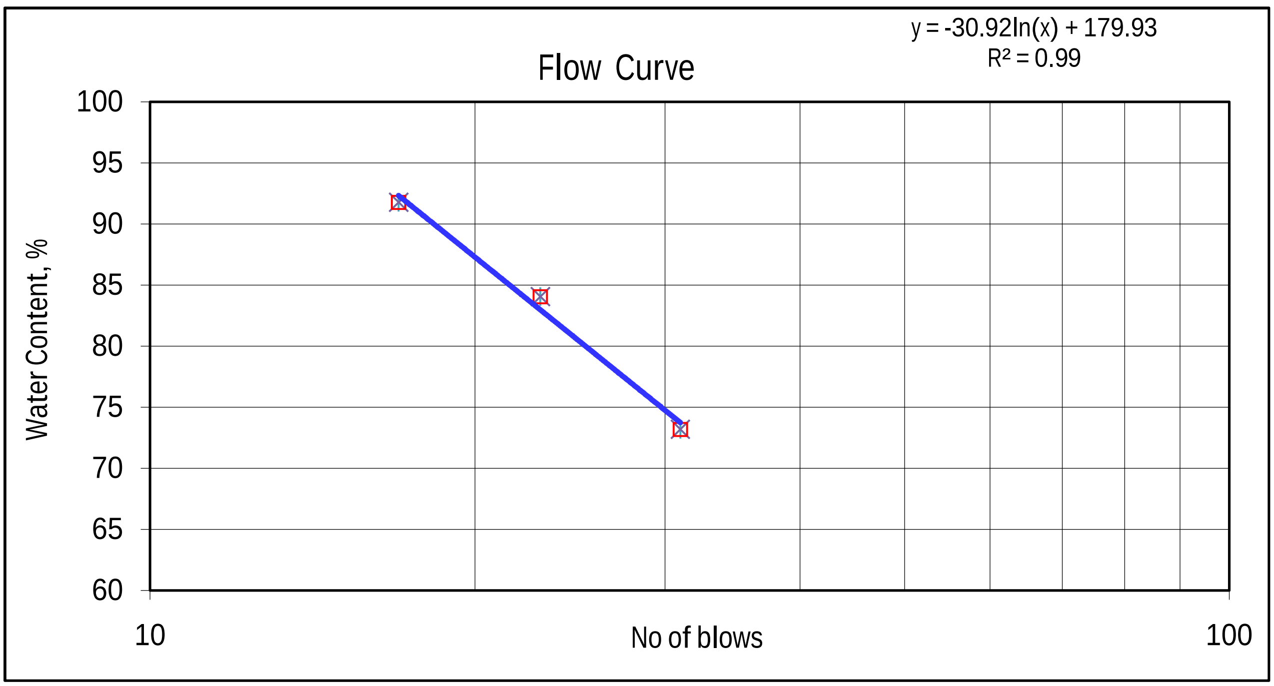
Project	Thesis research		Date	2/6/2012	
Sample No.	TP9-1		Depth	1.5m	

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	A4	C4	C6	D1	D2	
Mass of container	9.8	9.6	9.6	9.2	10.1	
Mass of container + wet soil	22.4	16.6	21.4	11.2	12.3	
Mass of container + dry soil	17.2	13.5	15.9	10.7	11.7	
Mass of water	5.2	3.1	5.5	0.5	0.6	
Mass of dry soil	7.4	3.9	6.3	1.5	1.6	
Water content	70.27	79.49	87.30	33.33	37.50	
Number of blows	34	29	20			
	LL(%)=	83		PL(%)	35.4	47.6



Project	Thesis research		Date	2/6/2012
Sample No.	TP9-2		Depth	3m

	LL			PL		PI(%)= LL - PL
Trial NO.	1	2	3	1	2	
Container NO.	L1	L2	L3	H	K	
Mass of container	9.5	15.8	15.5	9.3	10	
Mass of container + wet soil	26.3	28.5	29.5	10.8	12.5	
Mass of container + dry soil	19.2	22.7	22.8	10.4	11.8	
Mass of water	7.1	5.8	6.7	0.4	0.7	
Mass of dry soil	9.7	6.9	7.3	1.1	1.8	
Water content	73.20	84.06	91.78	36.36	38.89	
Number of blows	31	23	17			
	LL(%)=	81		PL(%)	**37.6**	43.4



Appendix A.3. Standard Proctor Compaction Test

Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP1-1 d= 1.5m

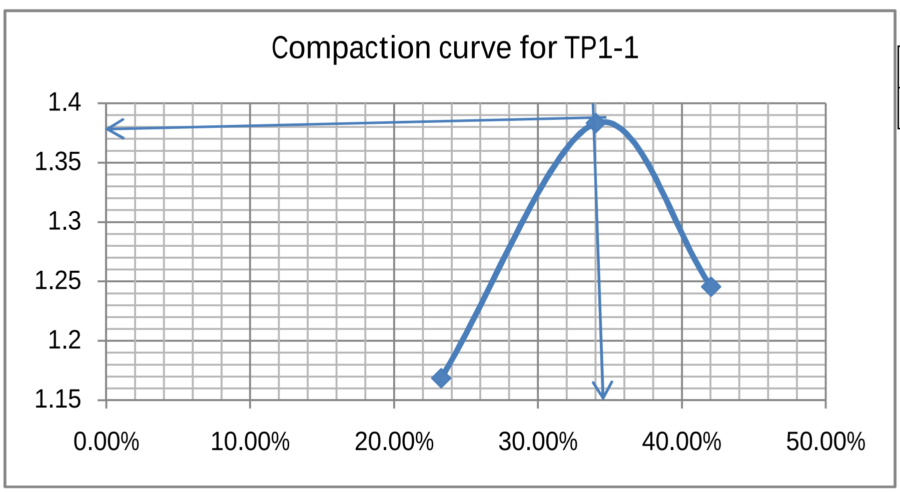
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	A1	A2	A3	
M_C = Mass of empty, clean can + lid (grams)	9.3	9.6	15.9	
M_{CMS} = Mass of can, lid, and moist soil (grams)	37.9	28.9	38.2	
M_{CdS} = Mass of can, lid, and dry soil (grams)	32.5	24	31.6	
M_S = Mass of soil solids (grams)	23.2	14.4	15.7	
M_w = Mass of pore water (grams)	5.4	4.9	6.6	
W = Water content, w%	23.28	34.03	42.04	
	23.28%	34.03%	42.04%	

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5670	6060	5980	
Mass of mold (grams)	4310	4310	4310	
Wet mass of soil in mold (grams)	1360	1750	1670	
Wet density, ρ , (kg/m)	1.440677966	1.853814	1.769068	
Dry density, ρd , (kg/m)	1.16862262	1.383133	1.245472	



OMC	35.00%
MDD	1.39g/cc

w(%)	23.28%	34.03%	42.04%	
DD(g/cc)	1.16862262	1.383133	1.245472	

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP1-2 d= 3m

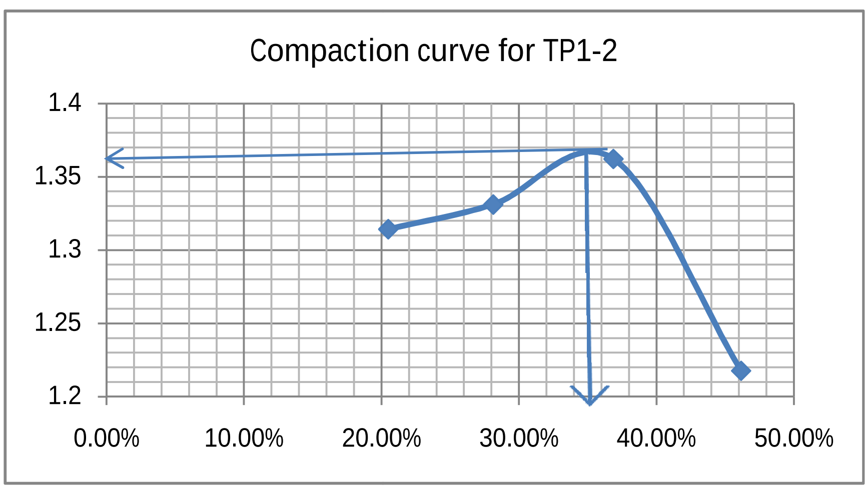
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	J	A2	A1	A6
M_C = Mass of empty, clean can + lid (grams)	15.9	9.5	9.6	15.7
M_{CMS} = Mass of can, lid, and moist soil (grams)	44.7	38.2	34.1	25.2
M_{CdS} = Mass of can, lid, and dry soil (grams)	39.8	31.9	27.5	22.2
M_S = Mass of soil solids (grams)	23.9	22.4	17.9	6.5
M_w = Mass of pore water (grams)	4.9	6.3	6.6	3
W = Water content, w%	20.50	28.13	36.87	46.15
	20.50%	28.13%	36.87%	46.15%

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5805	5920	6070	5990
Mass of mold (grams)	4310	4310	4310	4310
Wet mass of soil in mold (grams)	1495	1610	1760	1680
Wet density, ρ , (kg/m)	1.5836864	1.705508	1.864407	1.779661
Dry density, ρd , (kg/m)	1.3142626	1.331077	1.362173	1.217695



OMC	36.00%
MDD	1.365g/cc

w(%)	20.50%	28.13%	36.87%	46.15%
DD(g/cc)	1.3142626	1.331077	1.362173	1.217695

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP2-1 Depth= 1.5m

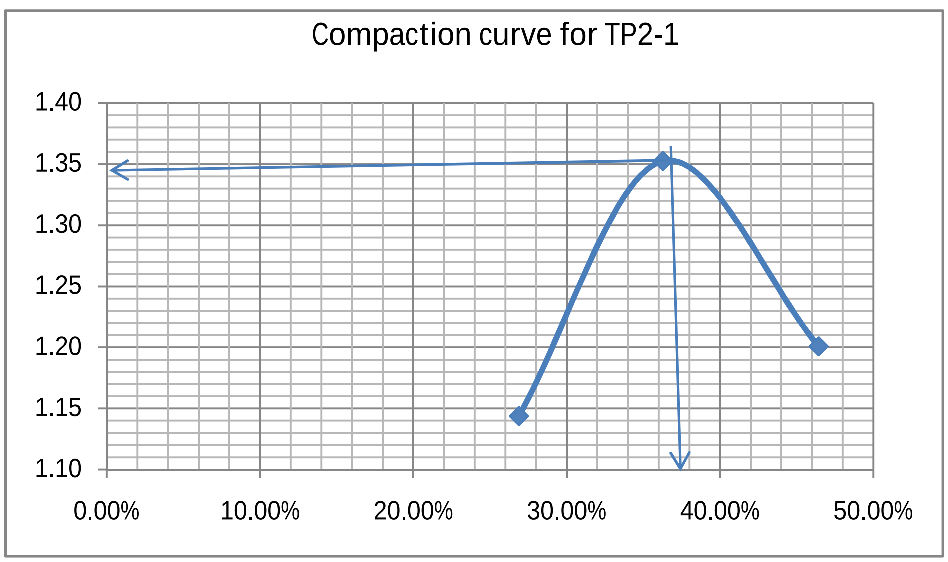
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	C1	A4	D	
M_C = Mass of empty, clean can + lid (grams)	16.1	9.5	9.2	
M_{CMS} = Mass of can, lid, and moist soil (grams)	31.2	24.9	26.7	
M_{CdS} = Mass of can, lid, and dry soil (grams)	28	20.8	21.15	
M_S = Mass of soil solids (grams)	11.9	11.3	11.95	
M_w = Mass of pore water (grams)	3.2	4.1	5.55	
W = Water content, w%	26.89	36.28	46.44	
	26.89%	36.28%	46.44%	

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5700	6070	5990	
Mass of mold (grams)	4330	4330	4330	
Wet mass of soil in mold (grams)	1370	1740	1660	
Wet density, ρ , (kg/m)	1.451	1.843	1.758	
Dry density, ρd , (kg/m)	1.144	1.353	1.201	



OMC	36%
MDD	1.35g/cc

w(%)	26.89%	36.28%	46.44%	
DD(g/cc)	1.143724	1.352524	1.200816	

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP2-2 Depth= 3m

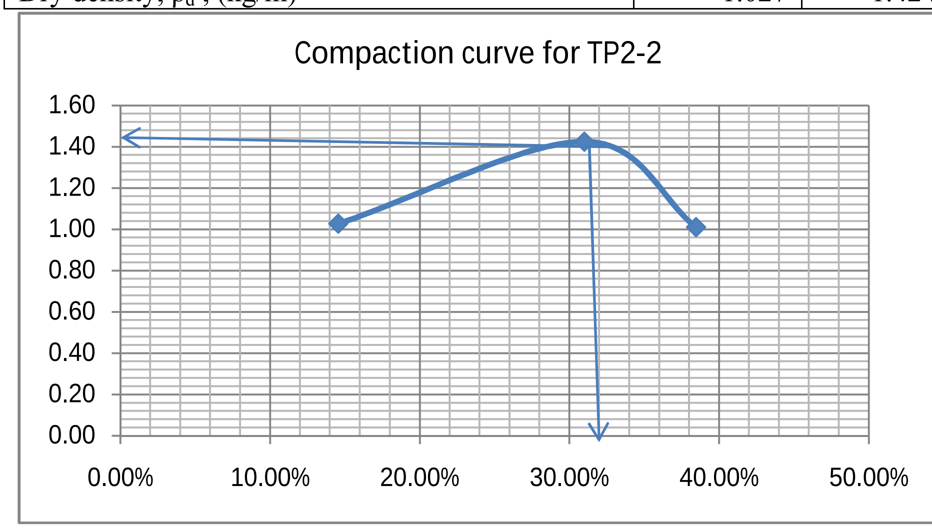
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	L1	L2	L3	
M_C = Mass of empty, clean can + lid (grams)	5.32	186	174	
M_{CMS} = Mass of can, lid, and moist soil (grams)	194	524	570	
M_{CdS} = Mass of can, lid, and dry soil (grams)	170	444	460	
M_S = Mass of soil solids (grams)	164.68	258	286	
M_w = Mass of pore water (grams)	24	80	110	
W = Water content, w%	14.57	31.01	38.46	
	14.57%	31.01%	38.46%	

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5600	5870	5650	
Mass of mold (grams)	4330	4330	4330	
Wet mass of soil in mold (grams)	1270	1540	1320	
Wet density, ρ , (kg/m)	1.345	1.631	1.398	
Dry density, ρ_d , (kg/m)	1.027	1.424	1.010	



OMC	31.00%
MDD	1.41g/cc

w(%)	14.57%	31.01%	38.46%	
DD(g/cc)	1.026916	1.423848	1.009887	

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP3-1	Depth=	1.5m	

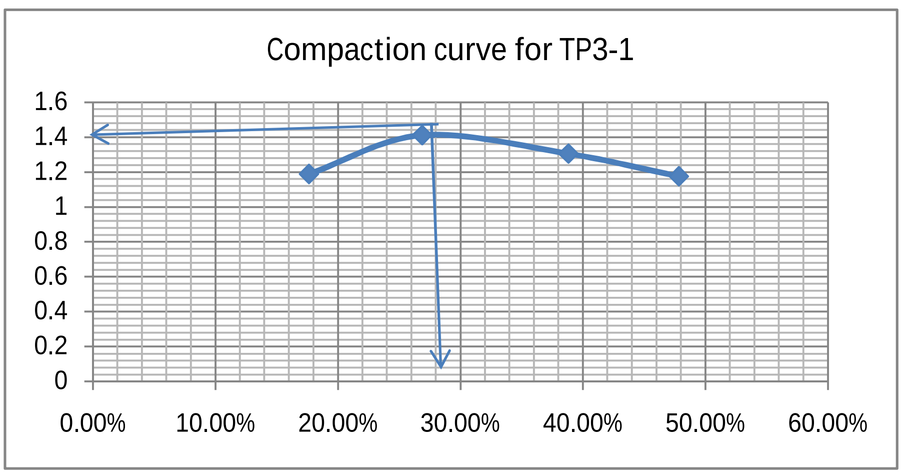
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	C1	B	A4	D
M_C = Mass of empty, clean can + lid (grams)	16.1	9.3	9.5	9.2
M_{CMS} = Mass of can, lid, and moist soil (grams)	30.1	27	24.7	29.6
M_{CdS} = Mass of can, lid, and dry soil (grams)	28	23.25	20.45	23
M_S = Mass of soil solids (grams)	11.9	13.95	10.95	13.8
M_w = Mass of pore water (grams)	2.1	3.75	4.25	6.6
W = Water content, w%	17.65	26.88	38.81	47.83
	17.65%	26.88%	38.81%	47.83%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5650	6020	6040	5970
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1320	1690	1710	1640
Wet density, ρ , (kg/m)	1.398	1.790	1.811	1.737
Dry density, ρd , (kg/m)	1.189	1.411	1.305	1.175



OMC	28%
MDD	1.42g/cc

w(%)	17.65%	26.88%	38.81%	47.83%
DD(g/cc)	1.188559322	1.4109631	1.3049523	1.1752243

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP3-2 Depth= 3m

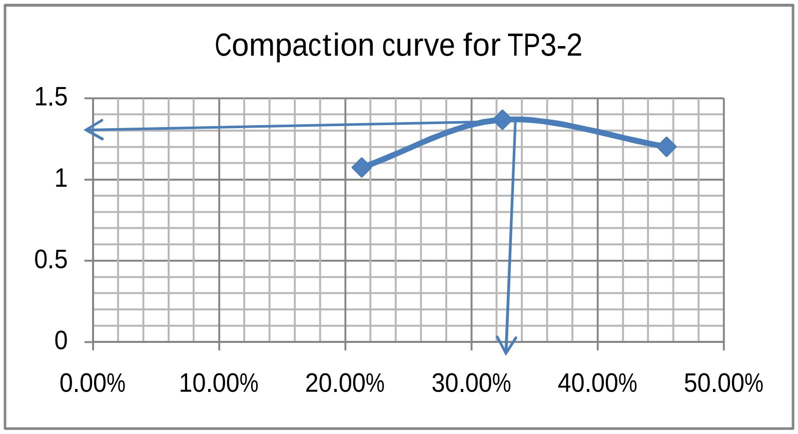
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	C1	J	A1	
M_C = Mass of empty, clean can + lid (grams)	16	16	16	
M_{CMS} = Mass of can, lid, and moist soil (grams)	42.2	36	36.8	
M_{CdS} = Mass of can, lid, and dry soil (grams)	37.6	31.1	30.3	
M_S = Mass of soil solids (grams)	21.6	15.1	14.3	
M_w = Mass of pore water (grams)	4.6	4.9	6.5	
W = Water content, w%	21.30	32.45	45.45	
	21.30%	32.45%	45.45%	

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5550	6030	5970	
Mass of mold (grams)	4320	4320	4320	
Wet mass of soil in mold (grams)	1230	1710	1650	
Wet density, ρ , (kg/m)	1.303	1.811	1.748	
Dry density, ρd , (kg/m)	1.074	1.368	1.202	



OMC	34%
MDD	1.36g/cc

w(%)	21.30%	32.45%	45.45%	
DD(g/cc)	1.074201061	1.3676377	1.2016684	

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP4-1 d= 1.5m

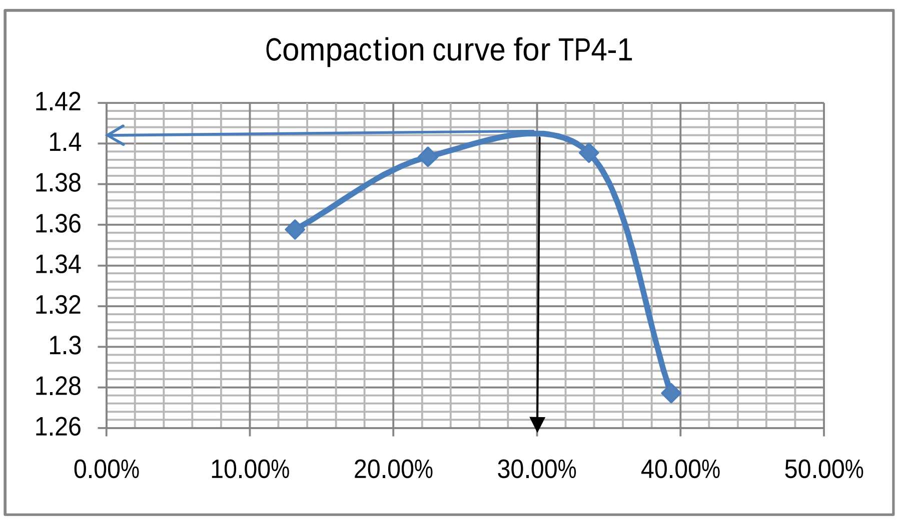
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	25
MCMS = Mass of can, lid, and moist soil (grams)	181	179	185	195
MCDS = Mass of can, lid, and dry soil (grams)	163	151	145	147
MS = Mass of soil solids (grams)	137	125	119	122
MW = Mass of pore water (grams)	18	28	40	48
W = Water content, w%	13.14	22.40	33.61	39.34
	13.14%	22.40%	33.61%	39.34%

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5760	5920	6070	5990
Mass of mold (grams)	4310	4310	4310	4310
Wet mass of soil in mold (grams)	1450	1610	1760	1680
Wet density, ρ , (kg/m)	1.5360169	1.705508	1.864407	1.779661
Dry density, ρd , (kg/m)	1.3576408	1.393389	1.395374	1.277168



OMC	28.00%
MDD	1.40g/cc

w(%)	13.14%	22.40%	33.61%	39.34%
DD(g/cc)	1.3576408	1.393389	1.395374	1.277168

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP4-2 d= 3.0m

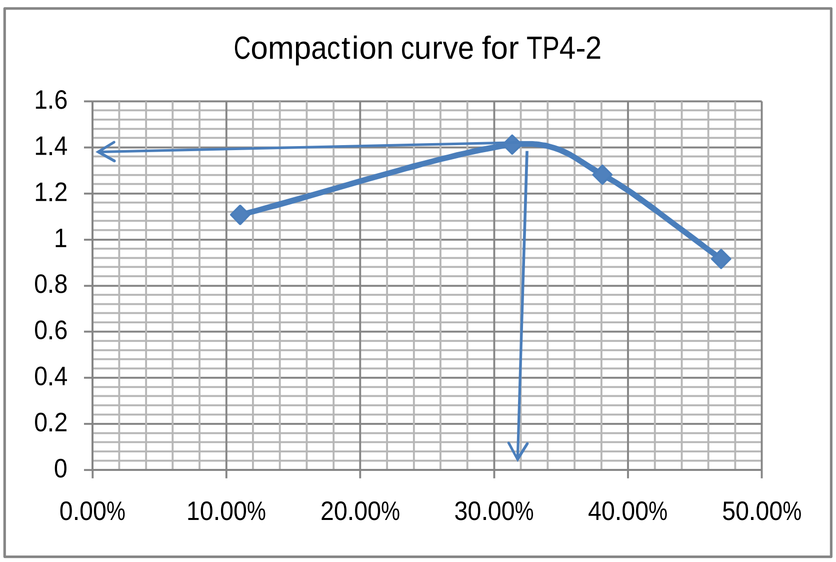
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	25	26
MCMS = Mass of can, lid, and moist soil (grams)	187	181	199	195
MCDS = Mass of can, lid, and dry soil (grams)	171	144	151	141
MS = Mass of soil solids (grams)	145	118	126	115
MW = Mass of pore water (grams)	16	37	48	54
W = Water content, w%	11.03	31.36	38.10	46.96
	11.03%	31.36%	38.10%	46.96%

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5470	6060	5980	5580
Mass of mold (grams)	4310	4310	4310	4310
Wet mass of soil in mold (grams)	1160	1750	1670	1270
Wet density, ρ , (kg/m)	1.228813559	1.853814	1.769068	1.345339
Dry density, ρd , (kg/m)	1.106695442	1.41129	1.281049	0.915467



OMC	32.00%
MDD	1.36g/cc

w(%)	11.03%	31.36%	38.10%	46.96%
DD(g/cc)	1.106695442	1.41129	1.281049	0.915467

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP5-1	Depth=	1.5m	

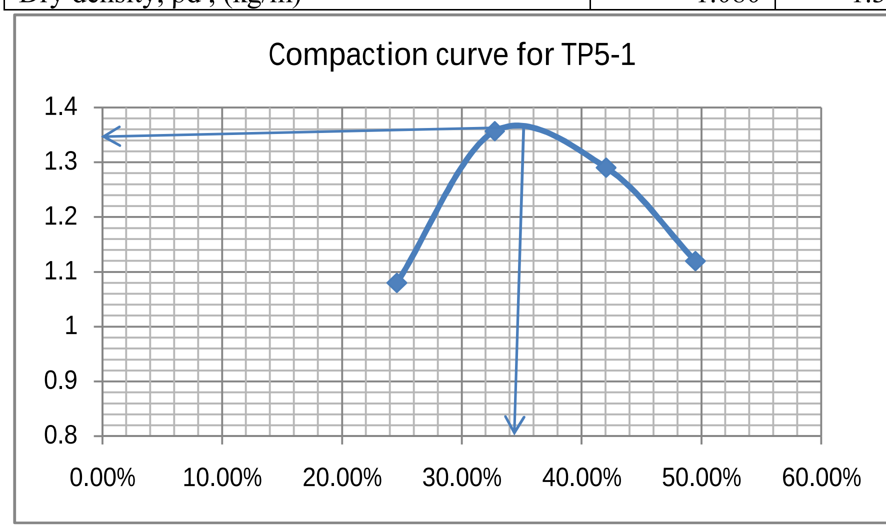
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	e1	e2	e3	e4
MC = Mass of empty, clean can + lid (grams)	25	25	26	25
MCMS = Mass of can, lid, and moist soil (grams)	177	175	178	179
MCDS = Mass of can, lid, and dry soil (grams)	147	138	133	128
MS = Mass of soil solids (grams)	122	113	107	103
MW = Mass of pore water (grams)	30	37	45	51
W = Water content, w%	24.59	32.74	42.06	49.51
	24.59%	32.74%	42.06%	49.51%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5600	6030	6060	5910
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1270	1700	1730	1580
Wet density, ρ , (kg/m)	1.345	1.801	1.833	1.674
Dry density, ρd , (kg/m)	1.080	1.357	1.290	1.119



OMC	34%
MDD	1.37g/cc

w(%)	24.59%	32.74%	42.06%	49.51%
DD(g/cc)	1.079811552	1.3566384	1.290073	1.119442

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP5-2 Depth= 3.0m			

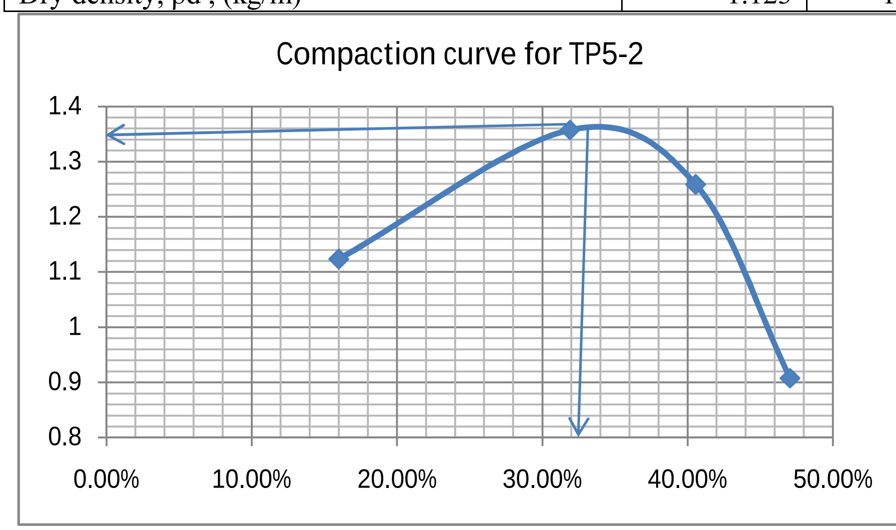
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	m1	m2	m3	m4
MC = Mass of empty, clean can + lid (grams)	26	26	25	25
MCMS = Mass of can, lid, and moist soil (grams)	171	179	174	175
MCDS = Mass of can, lid, and dry soil (grams)	151	142	131	127
MS = Mass of soil solids (grams)	125	116	106	102
MW = Mass of pore water (grams)	20	37	43	48
W = Water content, w%	16.00	31.90	40.57	47.06
	16.00%	31.90%	40.57%	47.06%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5550	6010	5990	5580
Mass of mold (grams)	4320	4320	4320	4320
Wet mass of soil in mold (grams)	1230	1690	1670	1260
Wet density, ρ , (kg/m)	1.303	1.790	1.769	1.335
Dry density, ρd , (kg/m)	1.123	1.357	1.259	0.908



OMC	34%
MDD	1.39g/cc

w(%)	16.00%	31.90%	40.57%	47.06%
DD(g/cc)	1.123246639	1.3573169	1.259	0.908

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP6-1	Depth=	1.5m	

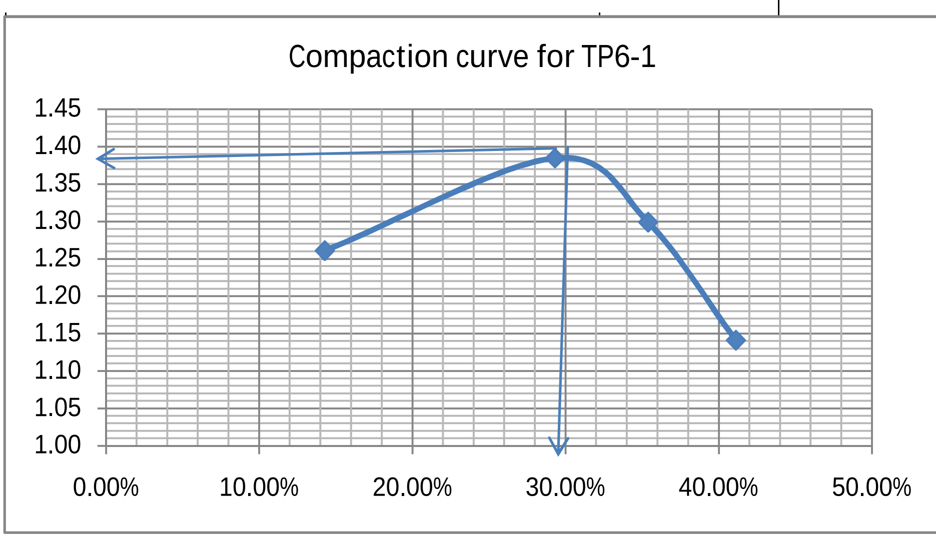
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	25	26	26	26
MCMS = Mass of can, lid, and moist soil (grams)	169	176	179	177
MCDS = Mass of can, lid, and dry soil (grams)	151	142	139	133
MS = Mass of soil solids (grams)	126	116	113	107
MW = Mass of pore water (grams)	18	34	40	44
W = Water content, w%	14.29	29.31	35.40	41.12
	14.29%	29.31%	35.40%	41.12%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5690	6020	5990	5850
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1360	1690	1660	1520
Wet density, ρ , (kg/m)	1.441	1.790	1.758	1.610
			1.299	1.141
			OMC	29%
			MDD	1.38g/cc



w(%)	14.29%	29.31%	35.40%	41.12%
DD(g/cc)	1.260593	1.384463	1.298743	1.14098

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP6-2	Depth=	3m	

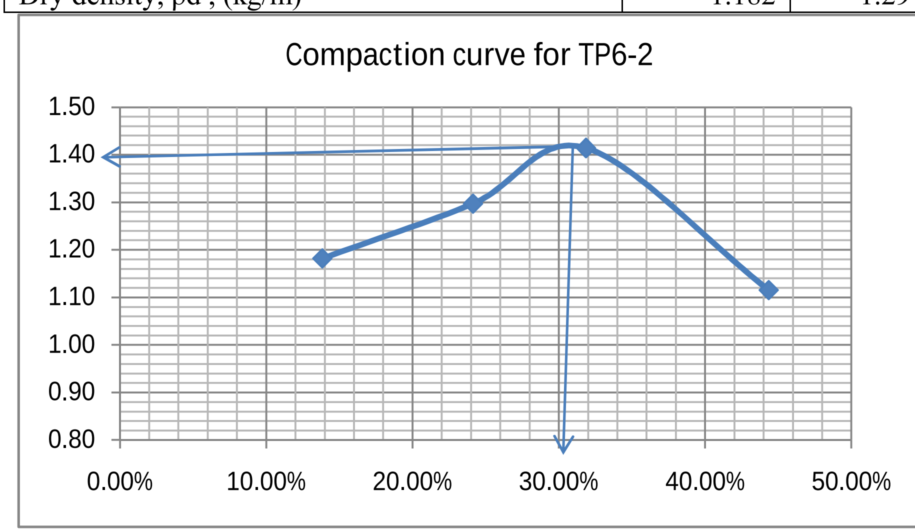
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	25
MCMS = Mass of can, lid, and moist soil (grams)	166	170	175	178
MCDS = Mass of can, lid, and dry soil (grams)	149	142	139	131
MS = Mass of soil solids (grams)	123	116	113	106
MW = Mass of pore water (grams)	17	28	36	47
W = Water content, w%	13.82	24.14	31.86	44.34
	13.82%	24.14%	31.86%	44.34%

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5600	5850	6090	5850
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1270	1520	1760	1520
Wet density, ρ , (kg/m)	1.345	1.610	1.864	1.610
Dry density, ρd , (kg/m)	1.182	1.297	1.414	1.116



OMC	30.00%
MDD	1.42g/cc

w(%)	13.82%	24.14%	31.86%	44.34%
DD(g/cc)	1.181976	1.297081	1.413946	1.115542

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP7-1 Depth= 1.5m

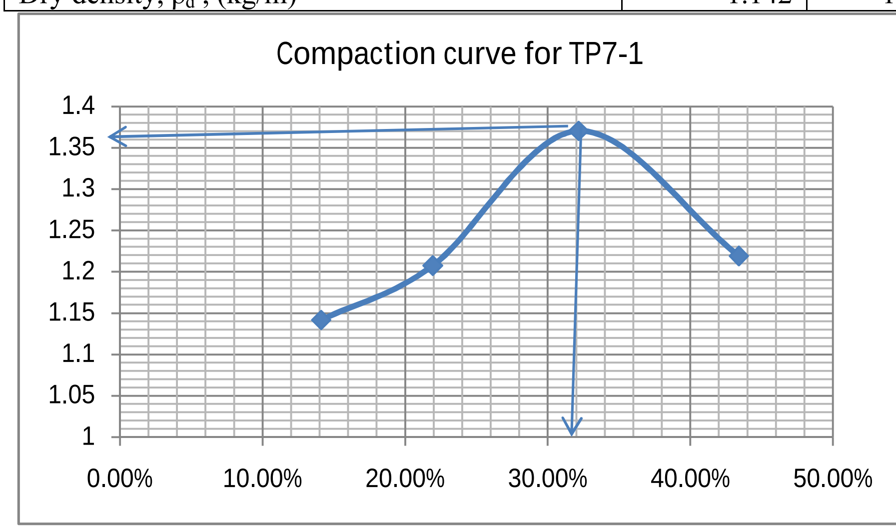
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	25	26	26
MCMS = Mass of can, lid, and moist soil (grams)	167.5	175	178	178
MCDS = Mass of can, lid, and dry soil (grams)	150	148	141	132
MS = Mass of soil solids (grams)	124	123	115	106
MW = Mass of pore water (grams)	17.5	27	37	46
W = Water content, w%	14.11	21.95	32.17	43.40
	14.11%	21.95%	32.17%	43.40%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5550	5710	6030	5970
Mass of mold (grams)	4320	4320	4320	4320
Wet mass of soil in mold (grams)	1230	1390	1710	1650
Wet density, ρ , (kg/m)	1.303	1.472	1.811	1.748
Dry density, ρ_d , (kg/m)	1.142	1.207	1.370	1.219



OMC	32%
MDD	1.37g/cc

w(%)	14.11%	21.95%	32.17%	43.40%
DD(g/cc)	1.141821884	1.2074153	1.3704979	1.218917

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP7-2 Depth= 3.0m			

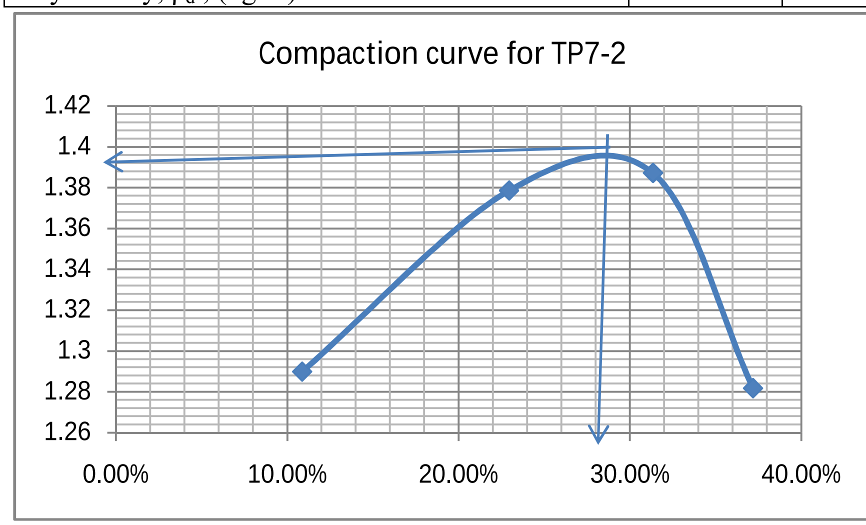
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	26
MCMS = Mass of can, lid, and moist soil (grams)	179	176	181	192
MCDS = Mass of can, lid, and dry soil (grams)	164	148	144	147
MS = Mass of soil solids (grams)	138	122	118	121
MW = Mass of pore water (grams)	15	28	37	45
W = Water content, w%	10.87	22.95	31.36	37.19
	10.87%	22.95%	31.36%	37.19%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5660	5910	6030	5970
Mass of mold (grams)	4310	4310	4310	4310
Wet mass of soil in mold (grams)	1350	1600	1720	1660
Wet density, ρ , (kg/m)	1.4300847	1.69492	1.8220	1.75848
Dry density, ρ_d , (kg/m)	1.2898804	1.37853	1.38710	1.28178



OMC	28%
MDD	1.39g/cc

w(%)	10.87%	22.95%	31.36%	37.19%
DD(g/cc)	1.289880	1.3785	1.3871	1.28178

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP8-1	Depth=	1.5m	

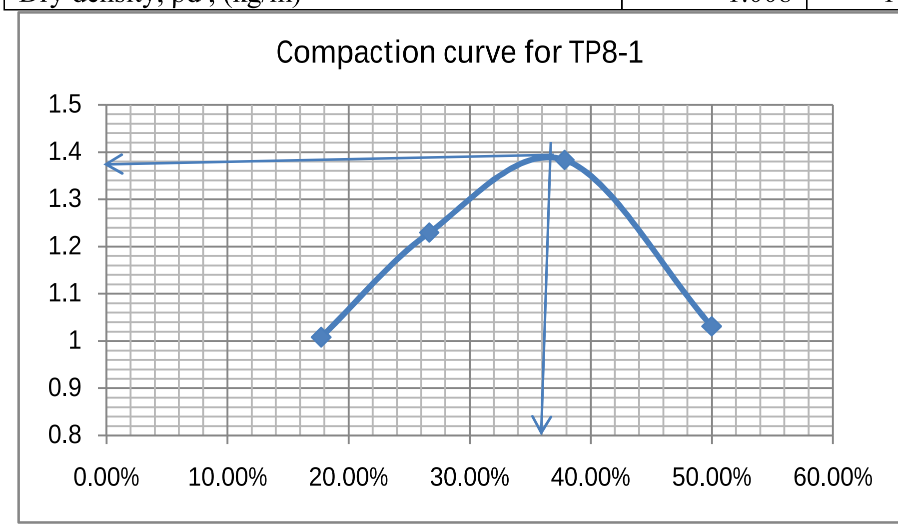
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	26
MCMS = Mass of can, lid, and moist soil (grams)	172	178	179	173
MCDS = Mass of can, lid, and dry soil (grams)	150	146	137	124
MS = Mass of soil solids (grams)	124	120	111	98
MW = Mass of pore water (grams)	22	32	42	49
W = Water content, w%	17.74	26.67	37.84	50.00
	17.74%	26.67%	37.84%	50.00%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5440	5790	6120	5780
Mass of mold (grams)	4320	4320	4320	4320
Wet mass of soil in mold (grams)	1120	1470	1800	1460
Wet density, ρ , (kg/m)	1.186	1.557	1.907	1.547
Dry density, ρd , (kg/m)	1.008	1.229	1.383	1.031



OMC	35%
MDD	1.38g/cc

\

w(%)	17.74%	26.67%	37.84%	50.00%
DD(g/cc)	1.007661946	1.2293711	1.383	1.031

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP8-2 Depth= 3.0m

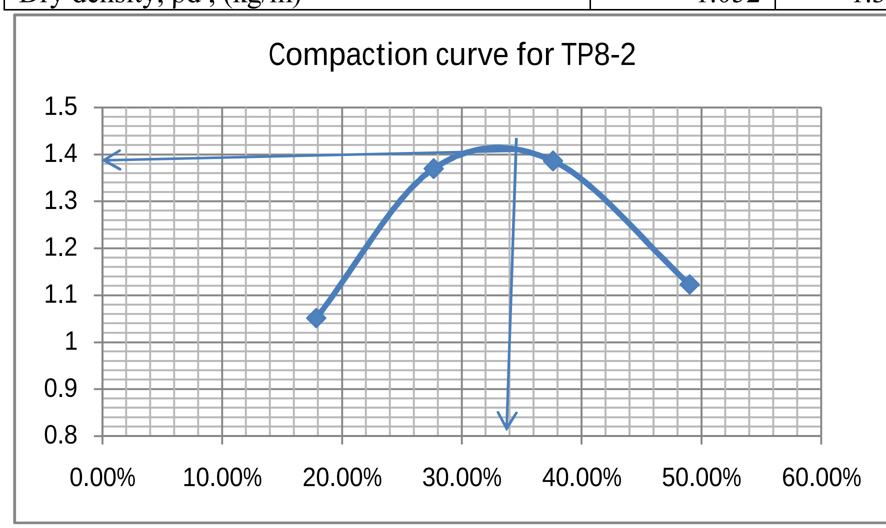
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	26
MCMS = Mass of can, lid, and moist soil (grams)	191	183	176	178
MCDS = Mass of can, lid, and dry soil (grams)	166	149	135	128
MS = Mass of soil solids (grams)	140	123	109	102
MW = Mass of pore water (grams)	25	34	41	50
W = Water content, w%	17.86	27.64	37.61	49.02
	17.86%	27.64%	37.61%	49.02%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5500	5980	6130	5910
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1170	1650	1800	1580
Wet density, ρ , (kg/m)	1.239	1.748	1.907	1.674
Dry density, ρd , (kg/m)	1.052	1.369	1.386	1.123



OMC	34%
MDD	1.37g/cc

w(%)	17.86%	27.64%	37.61%	49.02%
DD(g/cc)	1.051617874	1.3693593	1.3855932	1.1231601

Standard Proctor compaction test	
Date Tested:	11/7/2012
Project Name:	Thesis research
Sample Number:	TP9-1 Depth= 1.5m

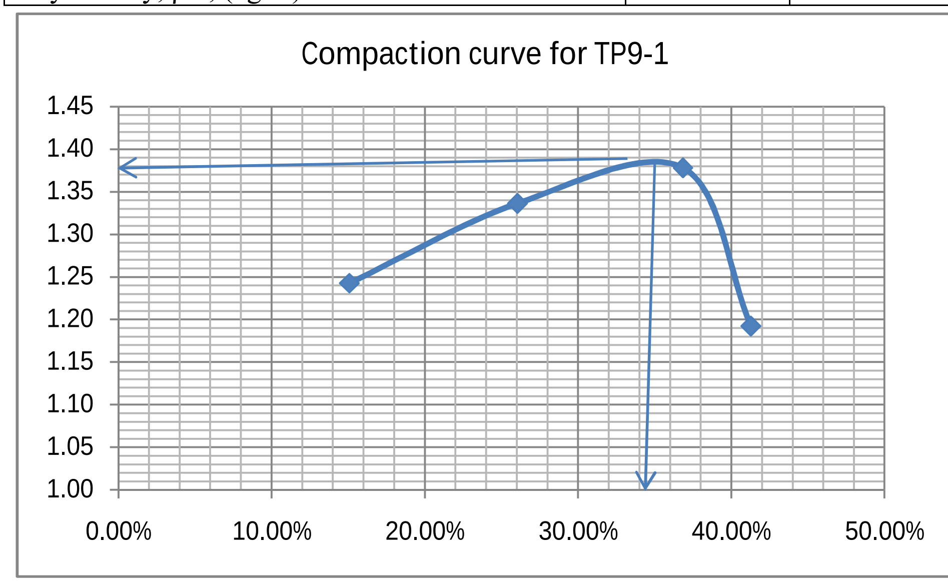
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	26	26	26	25
MCMS = Mass of can, lid, and moist soil (grams)	171	176	182	179
MCDS = Mass of can, lid, and dry soil (grams)	152	145	140	134
MS = Mass of soil solids (grams)	126	119	114	109
MW = Mass of pore water (grams)	19	31	42	45
W = Water content, w%	15.08	26.05	36.84	41.28
	15.08%	26.05%	36.84%	41.28%

Density Determination:

Volume of mold= **944m3**

Compacted Soil - Sample no.	1	2	3	
Mass of compacted soil and mold (grams)	5680	5920	6110	5920
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1350	1590	1780	1590
Wet density, ρ , (kg/m)	1.430	1.684	1.886	1.684
Dry density, ρd , (kg/m)	1.243	1.336	1.378	1.192



OMC	35%
MDD	1.38g/cc

w(%)	15.08%	26.05%	36.84%	41.28%
DD(g/cc)	1.242694	1.336229	1.377934	1.19215

Standard Proctor compaction test				
Date Tested:	11/7/2012			
Project Name:	Thesis research			
Sample Number:	TP9-2	Depth=	3m	

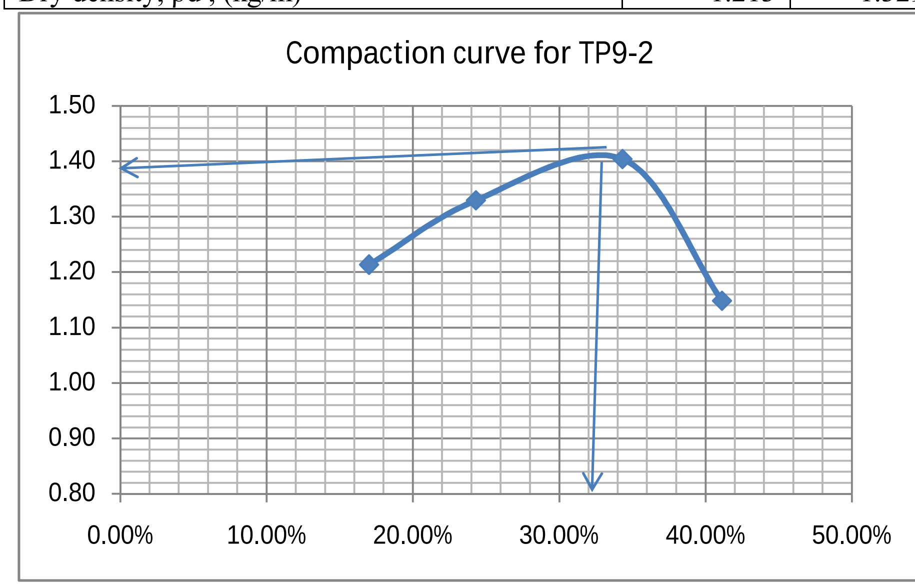
Water Content Determination:

Compacted Soil - Sample no.	1	2	3	4
Moisture can number - Lid number	M1	M2	M3	M4
MC = Mass of empty, clean can + lid (grams)	25	25	25	25
MCMS = Mass of can, lid, and moist soil (grams)	142	158	162	176
MCDS = Mass of can, lid, and dry soil (grams)	125	132	127	132
MS = Mass of soil solids (grams)	100	107	102	107
MW = Mass of pore water (grams)	17	26	35	44
W = Water content, w%	17.00	24.30	34.31	41.12
	17.00%	24.30%	34.31%	41.12%

Density Determination:

Volume of mold= **944cm3**

Compacted Soil - Sample no.	1	2	3	4
Mass of compacted soil and mold (grams)	5670	5890	6110	5860
Mass of mold (grams)	4330	4330	4330	4330
Wet mass of soil in mold (grams)	1340	1560	1780	1530
Wet density, ρ , (kg/m)	1.419	1.653	1.886	1.621
Dry density, ρd , (kg/m)	1.213	1.329	1.404	1.148



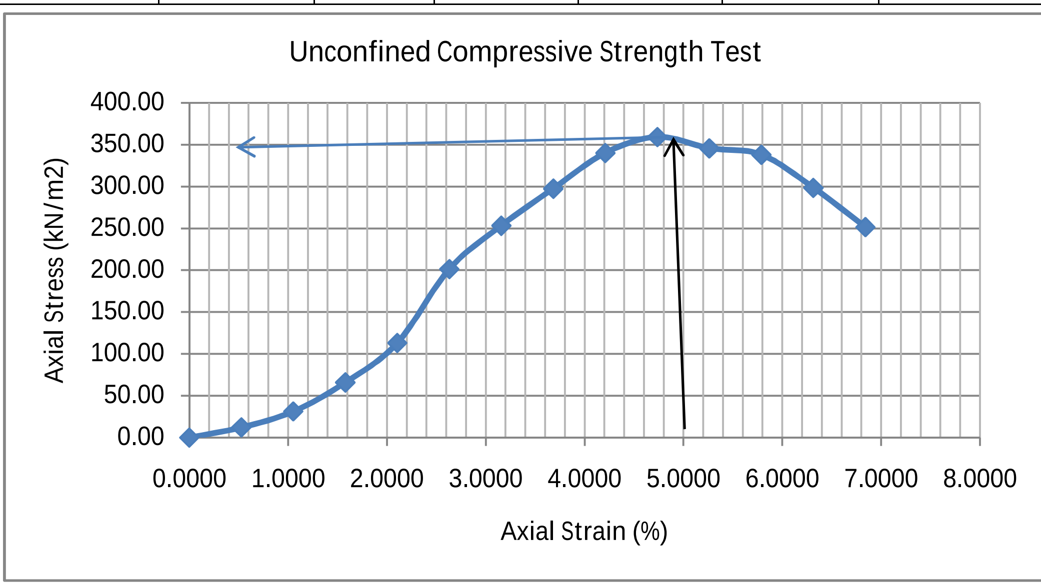
OMC	33.00%
MDD	1.41g/cc

w(%)	17.00%	24.30%	34.31%	41.12%
DD(g/cc)	1.213241	1.329489	1.403872	1.148487

Appendix A.4. Unconfined Compressive Strength Test

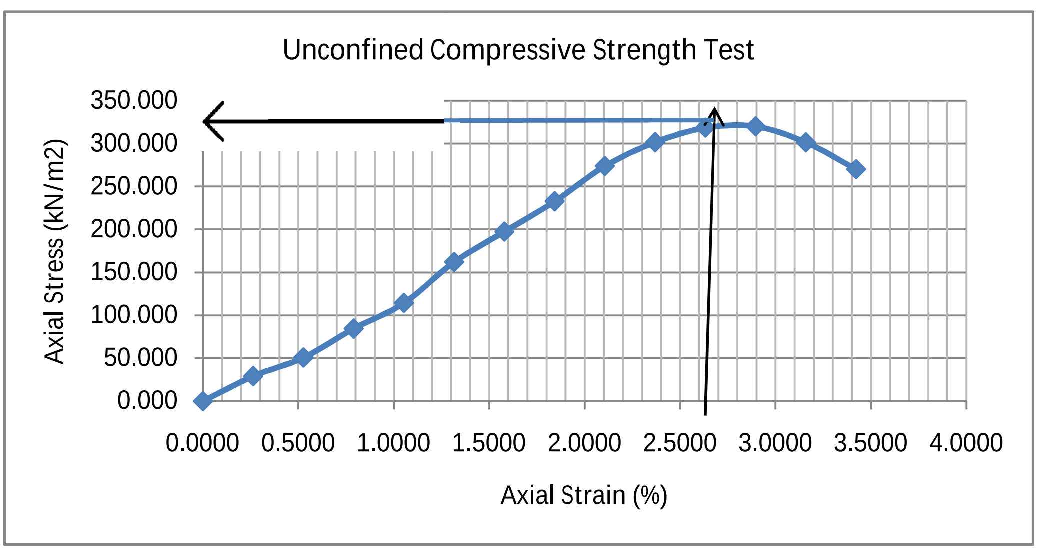
Diameter of Sample (mm):	38.0	Sample :- Undisturbed sample	
Height of Sample (mm):	76.0	Project :- Thesis research	
Area of Sample (m^2):	0.001134	Sample No. TP5-2	
Volume of Test Sample (m^3):	0.1		

Deformation dial reading(DDR)	load dial reading(LDR)	ΔL=DDR * 0.01 (mm)	LDR * 0.00138 *10000	%ε=ΔL/ Lo * 100	Ao(m^2)	corrected area (m^2) Ac=Ao/(1-ε)	Stress (kN/m^2)
0	0	0	0	0.00	0.001134	0.001134	0.00
20	10	0.2	0.0138	0.52	0.001134	0.001140	12.10
40	26	0.4	0.03588	1.05	0.001134	0.001146	31.3
60	55	0.6	0.0759	1.57	0.001134	0.001152	65.87
80	95	0.8	0.1311	2.10	0.001134	0.001159	113.16
100	170	1	0.2346	2.63	0.001134	0.001165	201.41
120	215	1.2	0.2967	3.16	0.001134	0.001171	253.35
140	254	1.4	0.35052	3.68	0.001134	0.001177	297.68
160	292	1.6	0.40296	4.21	0.001134	0.001184	340.35
180	310	1.8	0.4278	4.74	0.001134	0.001191	359.34
200	300	2	0.414	5.26	0.001134	0.001197	345.83
220	295	2.2	0.4071	5.80	0.001134	0.001204	338.18
240	262	2.4	0.36156	6.32	0.001134	0.001211	398.67
260	222	2.6	0.30636	6.84	0.001134	0.001217	251.65



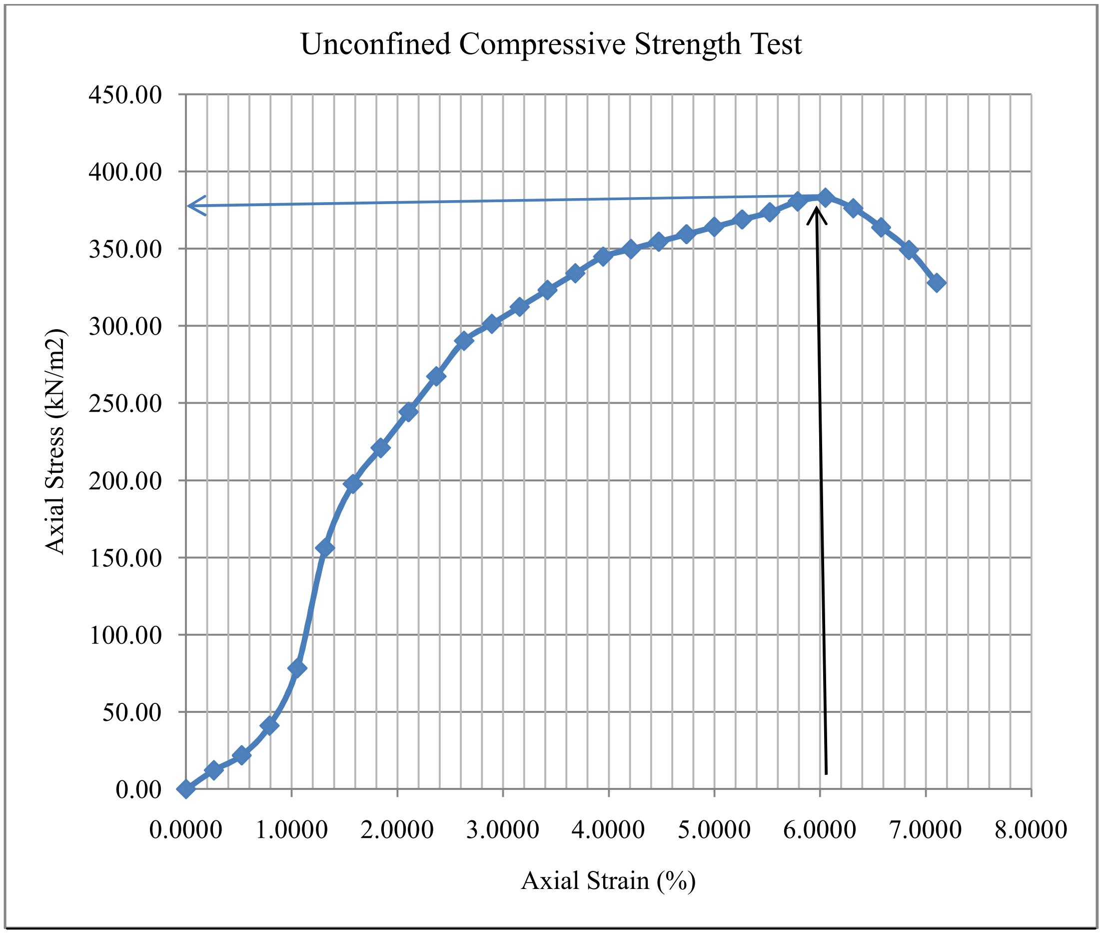
Diameter of Sample (mm):	38.0	Sample :- Undisturbed sample	
Height of Sample (mm):	76.0	Project :- Thesis research	
Area of Sample (m^2):	0.00113	Sample N.	TP6-2
Volume of Test Sample (m^3):	0.1		

deformation dial reading(DDR)	load dial reading(LDR)	ΔL=DDR * 0.01 (mm)	LDR * 0.00138 *10000	%ε=ΔL/Lo * 100	Ao(m^2)	corrected area (m^2) Ac=Ao/(1-ε)	stress (kN/m^2)
0	0	0	0	0.0000	0.0011	0.001134	0.000
20	24	0.2	0.03312	0.2632	0.0011	0.001137	29.127
40	42	0.4	0.05796	0.5263	0.0011	0.001140	50.837
60	70	0.6	0.0966	0.7895	0.0011	0.001143	84.504
80	95	0.8	0.1311	1.0526	0.0011	0.001146	114.380
100	135	1	0.1863	1.3158	0.0011	0.001149	162.108
120	165	1.2	0.2277	1.5789	0.0011	0.001152	197.603
140	195	1.4	0.2691	1.8421	0.0011	0.001155	232.907
160	230	1.6	0.3174	2.1053	0.0011	0.001159	273.974
180	254	1.8	0.35052	2.3684	0.0011	0.001162	301.749
200	269	2	0.37122	2.6316	0.0011	0.001165	318.708
220	271	2.2	0.37398	2.8947	0.0011	0.001168	320.209
240	256	2.4	0.35328	3.1579	0.0011	0.001171	301.666
260	230	2.6	0.3174	3.4211	0.0011	0.001174	270.291



Diameter of Sample:	38.0(mm)	Sample :- Undisturbed sample
Height of Sample (mm):	76.0	Project :- Thesis research
Area of Sample (m^2):	0.00113	Sample No. TP7-2
Volume of Test Sample:	0.1(m^3)	

deformation dial reading(DDR)	load dial reading(LDR)	ΔL=DDR * 0.01 (mm)	LDR * 0.00138 *10000	%ε=ΔL/Lo * 100	Ao(m^2)	corrected area (m^2) Ac=Ao/(1-ε)	stress (kN/m^2)
0	0	0	0	0.0000	0.001134	0.001134	0.00
20	10	0.2	0.0138	0.2632	0.001134	0.001137	12.14
40	18	0.4	0.02484	0.5263	0.001134	0.001140	21.79
60	34	0.6	0.04692	0.7895	0.001134	0.001143	41.04
80	65	0.8	0.0897	1.0526	0.001134	0.001146	78.26
100	130	1	0.1794	1.3158	0.001134	0.001149	156.10
120	165	1.2	0.2277	1.5789	0.001134	0.001152	197.60
140	185	1.4	0.2553	1.8421	0.001134	0.001155	220.96
160	205	1.6	0.2829	2.1053	0.001134	0.001159	244.19
180	225	1.8	0.3105	2.3684	0.001134	0.001162	267.30
200	245	2	0.3381	2.6316	0.001134	0.001165	290.27
220	255	2.2	0.3519	2.8947	0.001134	0.001168	301.30
240	265	2.4	0.3657	3.1579	0.001134	0.001171	312.27
260	275	2.6	0.3795	3.4211	0.001134	0.001174	323.17
280	285	2.8	0.3933	3.6842	0.001134	0.001177	334.01
300	295	3	0.4071	3.9474	0.001134	0.001181	344.79
320	300	3.2	0.414	4.2105	0.001134	0.001184	349.67
340	305	3.4	0.4209	4.4737	0.001134	0.001187	354.52
360	310	3.6	0.4278	4.7368	0.001134	0.001191	359.34
380	315	3.8	0.4347	5.0000	0.001134	0.001194	364.13
400	320	4	0.4416	5.2632	0.001134	0.001197	368.88
420	325	4.2	0.4485	5.5263	0.001134	0.001200	373.61
440	332	4.4	0.45816	5.7895	0.001134	0.001204	380.59
460	335	4.6	0.4623	6.0526	0.001134	0.001207	382.96
480	330	4.8	0.4554	6.3158	0.001134	0.001211	376.19
500	320	5	0.4416	6.5789	0.001134	0.001214	363.76
520	308	5.2	0.42504	6.8421	0.001134	0.001217	349.13
540	290	5.4	0.4002	7.1053	0.001134	0.001221	327.80


Unconfined Compressive Strength Test
Axial Stress (kN/m2)
450.00
400.00
350.00
300.00
250.00
200.00
150.00
100.00
50.00
0.00
0.0000
1.0000
2.0000
3.0000
4.0000
5.0000
6.0000
7.0000
8.0000
Axial Strain (%)

Appendix-B

(Sample calculations of consolidation test results)

A. Test Sample #5

Sample Designation…………………………TP-05-2

Location…………………………………Kebele-03

Sample depth…………………………… 3.0m

Data before commencement of the consolidation test:

Sample type …………………………..……Undisturbed

Natural moisture content (ω)…………...33.7%

Specific gravity of the soil (Gs)…………2.78

Mass of the ring (M_R)……………………70.50g

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

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

Area of the consolidation ring = 19.63cm^2

Volume of the consolidation ring = 39.25cm^3

Mass of soil (Ms) = 48.47g

Data at the end of the consolidation test:

Mass of ring + soil (M_{S+R})……………….135.30g

Mass of oven dried soil (Ms)………………48.47g

Moisture content at the end of testing……40.8%

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

		Dial Gauge Reading, mm						
Time (min)	**√time (min)**	**7 [kPa]**	**50 [kPa]**	**100 [kPa]**	**200 [kPa]**	**400 [kPa]**	**800 [kPa]**	**1600 [kPa]**
0	0.00	4.042	4.042	4.066	4.268	4.54	4.822	5.192
0.1	0.32	4.042	4.044	4.142	4.318	4.56	4.85	5.37
0.25	0.50	4.042	4.046	4.15	4.32	4.562	4.854	5.374
0.50	0.71	4.042	4.046	4.152	4.322	4.566	4.86	5.378
1	1.00	4.042	4.05	4.154	4.324	4.57	4.866	5.382
2	1.41	4.042	4.05	4.156	4.332	4.576	4.87	5.386
4	2.00	4.042	4.052	4.158	4.336	4.58	4.892	5.388
8	2.83	4.042	4.052	4.16	4.34	4.584	4.904	5.394
15	3.87	4.042	4.054	4.162	4.342	4.59	4.912	5.48
30	5.48	4.042	4.056	4.164	4.344	4.596	4.928	5.526
60	7.75	4.042	4.058	4.168	4.352	4.608	4.946	5.548
120	10.95	4.042	4.06	4.17	4.366	4.612	4.952	5.573
240	15.49	4.042	4.062	4.174	4.372	4.616	4.958	5.576
480	21.91	4.042	4.064	4.176	4.398	4.624	5.164	5.584
960	30.98	4.042	4.064	4.22	4.42	4.628	5.17	5.588
1440	37.95	4.042	4.066	4.268	4.54	4.822	5.192	5.594

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

Dial Guage Reading, mm						
1600 [kPa]	800 [kPa]	400 [kPa]	200 [kPa]	100 [kPa]	50 [kPa]	7 [kPa]
5.594		5.7		5.56		5.124

A.1.1. Compression index, Cc

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

$= \frac{.}{(\quad . \quad)}$

=48.47g

Height of solid (Hs) $= \frac{}{* \quad \rho*}$

Where:

G = Specific gravity of the solids

A = Area of the consolidation ring

ρ = density of water 1g/cm^3

$\Rightarrow \quad = \frac{48.47}{2.78 * 19.63 * 1} = 8.88$

Height of void (H_v) = H - Hs

=20 – 8.88 = 11.12mm

Initial void ratio (eo) $= - = \frac{*}{*} = -$

$= \frac{.}{.} = 1.25$

For loading of 50KPa

Initial reading = 4.042

Final dial reading = 4.066

$\Delta H_1 = 4.066 - 4.042$

= 0.024mm

$\Delta e_1 = \frac{\Delta}{}$

$= \frac{.}{.} = 0.0027$

$e_1 = e_o - \Delta e_1$

= 1.25 -0.0027

= 1.25

For loading of 100KPa

Initial reading = 4.066

Final dial reading = 4.268

$\Delta H_2 = 4.268 - 4.066$

$= 0.202$ mm

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

$= \frac{.}{.} = 0.023$

$e_2 = e_1 - \Delta e_2$

$= 1.25 - 0.023$

$= 1.227$

<u>For loading of 200KPa</u>

Initial reading = 4.268

Final dial reading = 4.540

$\Delta H_3 = 4.540 - 4.268$

$= 0.272$mm

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

$= \frac{.}{.} = 0.031$

$e_3 = e_2 - \Delta e_3$

= 1.227- 0.0.1

$= 1.196$

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

Initial reading = 4.540

Final dial reading = 4.822

$\Delta H_4 = 4.822 - 4.540$

$= 0.282$

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

$= \frac{.}{.} = 0.032$

$e_4 = e_3 - \Delta e_4$

$= 1.196 - 0.032$

$= 1.164$

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

Initial reading = 4.822

Final dial reading = 5.192

$\Delta H_5 = 5.192 - 4.822$

$= 0.370$ mm

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

$= \frac{.}{.} = 0.042$

$e_5 = e_4 - \Delta e_5$

$= 1.164 - 0.042$

$= 1.122$

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

Initial reading = 5.192

Final dial reading = 5.594

$\Delta H_6 = 5.594 - 5.192$

$= 0.402$ mm

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

$= \frac{.}{.} = 0.045$

$e_6 = e_5 - \Delta e_6$

$= 1.122 - 0.045$

$= 1.077$

<u>For unloading of 400KPa</u>

Initial reading = 5.594

Final dial reading = 5.700

$\Delta H_7 = 5.700 - 5.594$

$= 0.106$mm

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

$= \frac{.}{.} = 0.012$

$e_7 = e_6 - \Delta e_7$

$= 1.077 - 0.012$

$= 1.065$

For unloading of 100KPa

Initial reading = 5.700

Final dial reading = 5.56

$\Delta H_8 = 5.56 – 5.700$

$=- 0.140$

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

$= \frac{.}{.} = -0.016$

$e_8 = e_7 - \Delta e_8$

$= 1.065 + 0.016$

$= 1.081$

For unloading of 7KPa

Initial reading = 5.560

Final dial reading = 5.124

$\Delta H_9 = 5.124 – 5.56$

$= -0.436mm$

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

$= \frac{.}{.} = -0.049$

$e_9 = e_8 - \Delta e_9$

$=1.081+ 0.049$

$= 1.130$

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

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

Applied Pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height(ΔHi) (mm)	Change in void ratio Δei	Final Specimen Height (mm)	Void Height, Hv (mm)	Void Ratio, e
LOADING						
7	4.042	0.000	0.0000	20.00	11.12	1.251
50	4.066	0.024	0.0027	19.98	11.09	1.249
100	4.268	0.226	0.023	19.77	10.89	1.226
200	4.540	0.498	0.031	19.50	10.62	1.195
400	4.822	0.780	0.032	19.22	10.34	1.163
800	5.192	1.150	0.042	18.85	9.97	1.122
1600	5.594	1.552	0.045	18.45	9.56	1.077
UNLOADING						
1600	5.594	1.552	0.045	18.45	9.56	1.077
400	5.7	1.658	0.012	18.34	9.46	1.065
100	5.56	1.518	0.016	18.48	9.60	1.080
7	5.124	1.082	0.049	18.92	10.03	1.129

Using the deformation results (void ratio or strain) corresponding to the end each increment loading or unloading versus logarithm of pressure and pressure respectively is drawn. These graphs are shown in fig 4.12 and fig 4.13. Based on this plot, the compression index, Cc will be the slope of loading curve and recompression index, Cr will be the slope of unloading curve. Therefore, by taking any two points on the straight portions of figure 4.12 for both loading and unloading:

$$\text{The compression index,} \quad = \frac{e1 - e2}{(\log\ p2 - \log\ p1)}$$

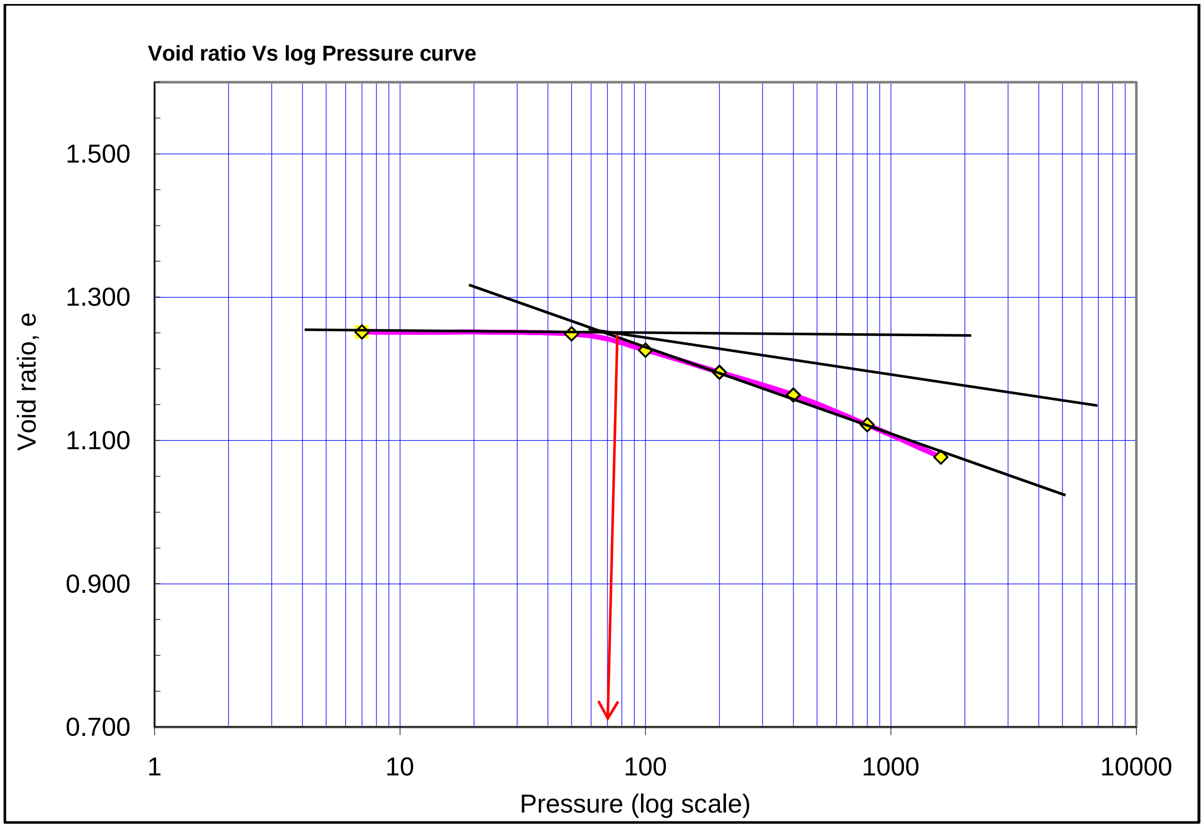
$$= \frac{1.163 - 1.077}{(\log 1600 - \log 400)}$$

$$= 0.144$$

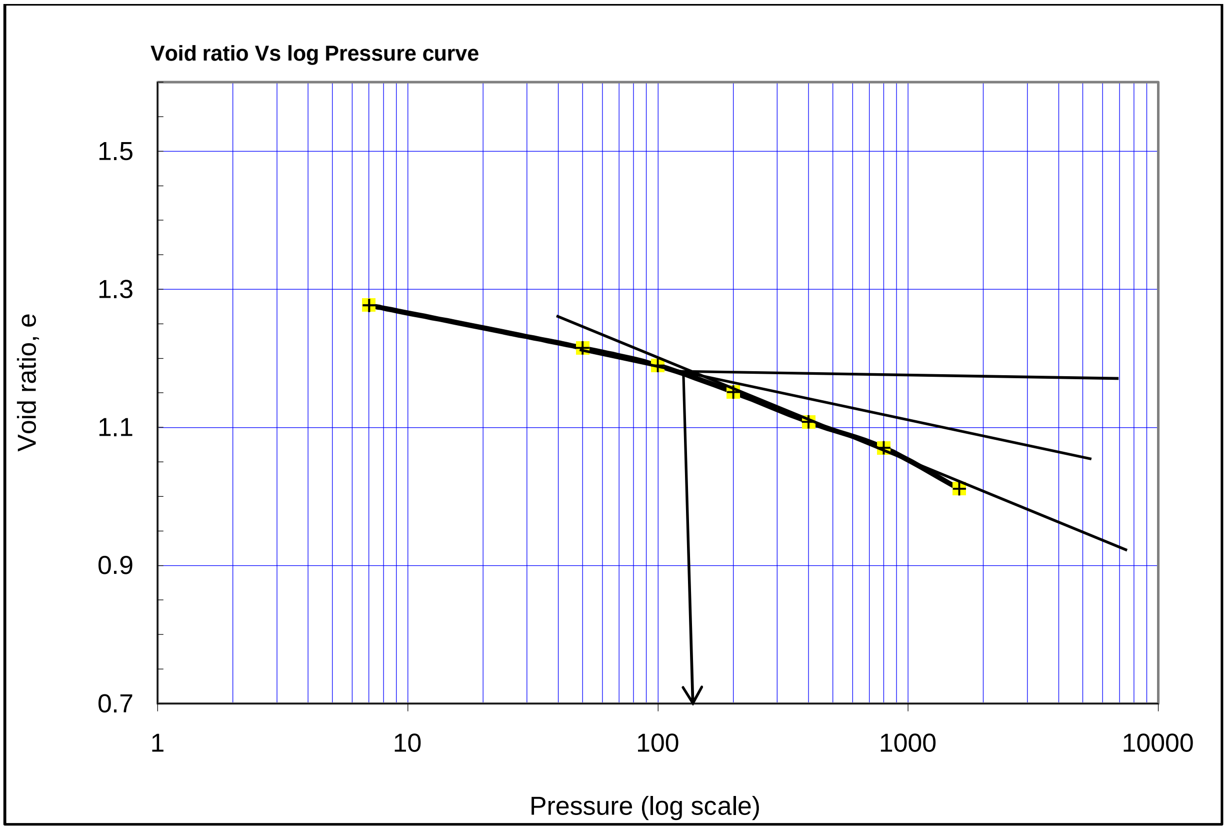
$$\text{The recompression index,} \quad = \frac{e1 - e2}{(\log\ p2 - \log\ p1)}$$

$$= \frac{1.080 - 1.077}{(\log 1600 - \log 100)}$$

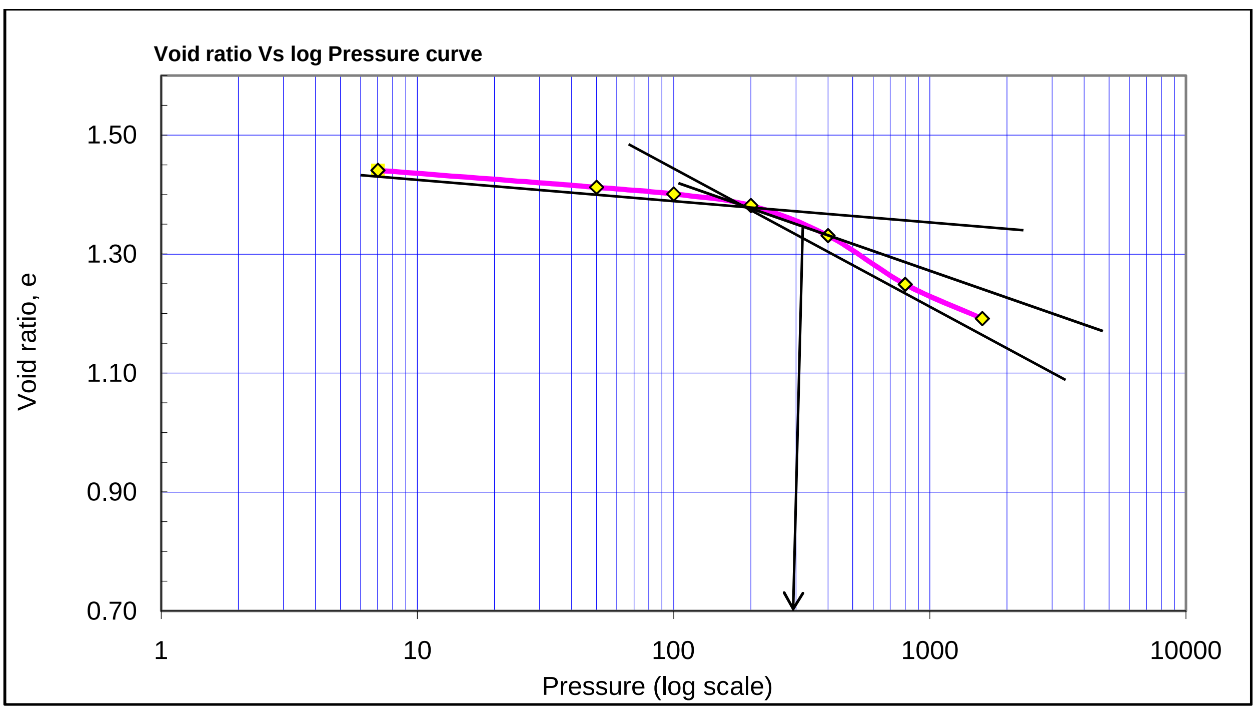
=0.003



Determination of "P_c" for TP- 5



Determination of "P_c" for TP- 6



Determination of "P_c" for TP- 7

A.1.2. Coefficient of permeability

The coefficient of permeability can be obtained from the following relationship [20].

$$k = \frac{c \;\; * a \;\; * \gamma}{1 + e}$$

Where: - Coefficient of consolidation

- Coefficient of compressibility

γ - Unit weight of water

e - Void ratio

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

Void ratio (e) = 1.077

From figure 4.15 $C_v = 1.002 \text{ X } 10^{-3}$ cm^2/sec

Unit weight of water (γ) = 10 KN/m^3

$$\text{From} \quad \text{igure 4.10} \quad = \frac{1.126 - 1.0766}{1600 - 800}$$

= 0.566 cm2/kN

$$k = \frac{1.002\text{X } 10 \quad —— * 0.566 —— * \; 10 * 10\text{^}(-6) \text{ KN/cm3}}{1 + 1.077}$$

= 5.61X 10^{-9} cm/sec

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

Void ratio (e) = 1.122

From figure 4.12 $C_v = 1.57\text{X } 10^{-3}$ cm^2/sec

Unit weight of water (γ) = 10 KN/m^3

$$\text{From} \quad \text{igure 4.10} \quad = \frac{1.1635 - 1.123}{800 - 400}$$

= 1.041c m^2/kN

$$k = \frac{1.57\text{X } 10 \quad —— * 1.041 —— * \; 10 * 10\text{^}(-6) \text{ KN/m3}}{1 + 1.122}$$

= 1.617X 10^{-8} cm/sec

Following the same procedure coefficient of compressibility and coefficient of permeability for each incremental loading is shown on table 4.13.

DECLARATION

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

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