

INVESTIGATION ON THE ENGINEERING CHARACTERISTICS OF SOIL IN GUMER WEREDA AREKIT TOWN

Ephrem Gizaw Abdo



A Thesis submitted to the department of Civil Engineering
School of Civil Engineering and Architecture

Presented in partial fulfillment of the requirement for the degree of Master's in
Civil Engineering (specialization Geo technical Engineering)

Office of Graduate studies
Adama Science and Technology University

Adama
August, 2020

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Under the supervision and Guidance of
Argaw Asha (PhD)



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Approval of Board of examiner

We, the undersigned members of the board of examiner of the final open defense by Ephrem Gizaw Abdo Have read and evaluated his thesis entitled “Investigation on the engineering characteristics of soil Gumer wereda Arekit town” and examine the candidate. This is therefore, to certify that the thesis has been accepted in partial fulfilment of the requirement of the degree of Master of Science in Geotechnical Engineering

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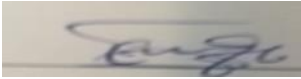
Declaration

I, the undersigned, pronounce that this thesis is my unique work performed under the supervision of my research advisor Dr.Argaw Asha and has not been introduced as thesis for a degree in any other university. All well spring of material utilized for this thesis have been appropriately recognized.

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LIST OF ACRONYMS, SYMBOL AND UNIT

ACRONYMS	DESIGNATION	SI UNIT
AASHTO,	American Association of state high way and transport official	--
ASTM,	American society testing and material	--
Ac,	Activity number	--
a_v ,	Coefficient of compressibility	kN/m ²
C_v ,	Coefficient of consolidation	m ² /day, cm ² /min
Cc,	Compression index	--
Cc,	Consistency index	--
CL,	Inorganic clay of low to medium plasticity	--
CH,	Inorganic clay of high plasticity	--
Cs,	swelling index	--
e,	void ratio	--
GI,	Group index	--
Gs,	Specific gravity	--
Ip,	Plastic index	%
K,	Coefficient of permeability	m/day, cm/min
LI,	liquidity index	%
LL/ w_L ,	liquid limit	%
MDD,	maximum dry density	Kg/m ³ , gm/cc
MH,	Inorganic silt of high plasticity	--
NMC,	natural moisture content	--
OD,	oven dry	--
OMC,	optimum moisture content	%
OCR,	Over consolidation ratio	--
PL/ w_p ,	plastic limit	%
TP	Test pit	--
σ'_c ,and	Pre- consolidation pressure	kPa
$\Delta\sigma_f$.	Stress at failure	kN/m ²

Abstract

Investigating the Engineering property of the soil is a base for design and construction of a physical structure created in the world. In the filed ten test pit at depth of 1.5m and 3m excavated. Then, the disturbed and un-disturbed soil sample collected. Moreover, laboratory test have been carried out. From the index property test result the natural moisture content of the soil varies in the range of 20-45%. But, at T'emren Hayek the natural moisture content was 82.05 % and the specific gravity of the soil ranges, 2.65-2.78 According to ASTM D854-92 which indicate inorganic soil. In addition to this, the LL ranges from 29.33-78.12%, and PL range from 16.60-46.45% and also the Ip ranges from 3.80-31.63% which indicated that the soil in the study area was plastic state. From the grain size analysis test result, silt and clay particles are the major one the clay ranges, 24.92-75.52%, and silt ranges, 22.78-67.95%. As well as, the free swell test result in between 15-50% shows the study area doesn't have expansive nature except in TP-4 with free swell 70%, it shows moderate expansive nature. For the part of engineering property test result, the maximum dry density and optimum moisture content ranges, 1.09 to 1.6gm/cc and 19.5 to 46% respectively for. The triaxial test result shown in the chamber pressure at 100,200,300 KN/m² for TP-4, TP-6 and TP-9 the stress at failure that is the deviator stress was 85.12, 86.4, 87 KN/m², 248.48, 249.5, 256.12 KN/m² and 391.3, 394.76, 399.48 KN/m² respectively. The principal stress become 185.12, 286.4, 387 KN/m², 348.48, 449.5, 556.12 KN/m², and 491.3, 594.76, 699.48 KN/m² respectively. And also, the undrain shear strength, 42.56, 43.2, 43.5 kN/m², 124.24, 124.75, 128.06 kN/m², 195.65, 197.38, 199.74 kN/m² respectively. From the consolidation test result, which explain the over consolidation ratio is greater than one which indicated that the soil was over consolidated in its nature. To generalize, in unified soil classification system most of the soil is CL and CH. For AASHTO classification system, most of the soil in the group falls under A-7-5 and A-7-6. Therefore, the soil in Arekit town was a clay soil.

Key word: Investigation of soil, Liquid limit, optimum moisture content, correlation, regression.

CHAPTER ONE

INTRODUCTION

1.0 Back ground of the study

The development of building structure is creating fast in Arekit town like other fundamental town of Ethiopia. Arekit is found 220kilometre south of Addis Ababa and 58km away from Gurage zone capital welkite. Moreover, A few Administrative organizations and private commerce centers are being set up with in the town. Most of the time, the brown to dark soil colure can be seen within the earth surface. In expansion to this, erosion of soil is major problem. Therefore, it require geotechnical investigation of sub-surface condition of these soil is critical.

Soil is foremost significance that can with stand vertical load as well as horizontal load those load comes from super structure, seismic tremors so on. In arrange to know, the sort and characteristics of soil examining the soil with laboratory test additionally field test is basic to decide the bearing capacity of soil, the depth and type of foundation.

Exploring a given soil before erecting huge construction like building, highway, Dam, retaining wall etc. it may be a fundamental need which makes a difference for financial plan of foundation to be made as well as to know the strength of the soil conjointly which can demonstrate whether the soil in particular zone area needs stabilization or not.

The diverse soil parameters obtained from the field and laboratory test, can be utilized as an input in to the design of a foundation. In this manner, one of the most vital engineering properties of soil is its shear strength or ability to resist sliding along internal surfaces within a soil mass. All stability analyses in the field of geotechnical engineering, whether they relate to foundation, slopes of cuts or earth dams, include essential knowledge of this engineering property of the soils.

The scope of soil investigation depends on the type, size and importance of the structure, the Client the engineer's familiarity with soil at the site and local building code. Structure that are sensitive to settlement such as machine foundation and high rise building usually require deep soil investigation compared to foundation for house [6].

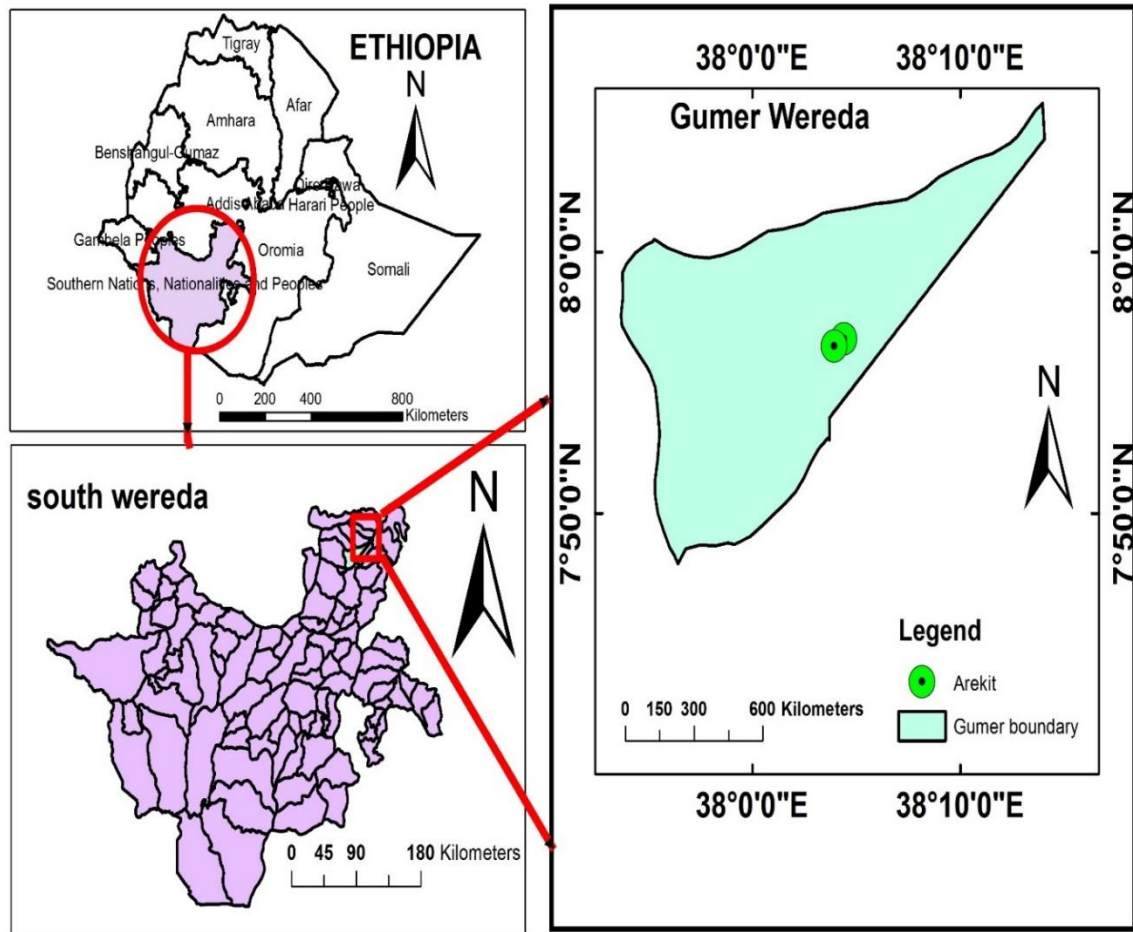


Figure 1.1:-The location of Arekit on Map of Ethiopia

1.1 Statement of the problem

In the Gumer wereda, Arekit town roads are constructed and building being under construction. There is no available soil investigation data. For this reason, improper designs, Excessive cost of construction, delays in construction schedule plan and also, erosion is major problem in Arekit town. The municipality needs such kind of research in order to solve the problem. This research help in conducting proper soil exploration by providing the necessary data within the appropriate soil investigation process. Arekit town, currently developing at high growth rate and which needs numerous construction works in the future, geotechnical investigation on the engineering property of soil is very essential.

1.2 General Objectives

The general objectives of these research is, to study the engineering characteristics of soils in Arekit town.

1.3 Specific objective

- To investigate the engineering and index properties of the soil found in Arekit town
- To determine compression index C_c , coefficient consolidation C_v and permeability of soil.
- To determine geotechnical investigation profile
- To predicted the correlation of Atterberg limit to maximum dry density and optimum moisture content.

1.4 Significance of the study

This research can be used for preliminary design to decide the type and depth of the foundation and also it is important for the Engineers and municipality of the town to plan land use for the area which one is suitable for home building, industry zone etc. Very vital to select site for construction and evaluate special practice to insure performance. Finally, These investigation of the soil express limitation and nature of expansive in the soil where the area need improvement to come up with the solution of limitation and the effect of selected land use on the environment.

1.5 Scope of the study

The study covered the investigation of the index properties, grain size distribution, and compaction and tri axial test of soil and also consolidation of soil.

1.6 Limitation of the research

The depth of investigation in this research is limited to 3m since it is difficult to excavate and do sampling manually beyond because of the financial limitation.

1.7 Organization of the thesis

This research work holds six chapters and each chapter includes a specific topic. In the introductory chapter back - ground of the study, statement of the problem, general objectives, and specific objectives, significance of the study, scope of the study and organization of the thesis are presented. And also chapter two express, literature review in chapter three it explains the methodology and description of the area where this study was done. The fourth chapter deals about the laboratory test result and fifth chapter deals about discussion. In addition to this, the soil map of the study area and correlation on Atterberg limit with maximum dry density and optimum moisture content briefly presented. In chapter six, conclusion and recommendation of the research are drawn. The last but not the least point, detailed calculation on the consolidation test result, triaxial test reading are included in appendix.

CHAPTER TWO

Literature review

2.0 General

Soil is delicate geological deposits extending from the sub soil to bed rock. In some soil there is certain amount of cementation between the grains which influence the physical property of the soil if this cementation is such that a rock- hard material has been produced then the material must be described as rock. A rough rule is that if the material can be excavated by hand or hand tools it is a soil [10].

2.1 Modes of formation of soil

There are three types of soil when considering modes of formation

- i. Transported soil (gravels, sands, silts and clays)

Most soil have been transported by water, as stream or river loses its speed it tends to store some of the particle that is carrying, dropping the bigger, heavier particles to begin with. Consequently, on the higher reaches of river gravel and sand are found whilst on the lower silt and clays prevail especially where the water way enters the ocean or a lake and loses its velocity [10].

- ii. Residual soil (topsoil, laterites)

These soil are formed insitu by chemical weathering and may be found on level rock surface where the action of the components has been created a soil with small affinity to move. Residual soil can also happen whenever the rate of breakup of the rock exceeds the rate of removal. Laterites are formed by chemical weathering under warm, humid tropical condition when the rain water leaches out the soluble rock material leaving behind the insoluble hydroxide of iron and aluminum giving them their characteristics red brown color [10].

- iii. Organic soil

These soil contain enormous amount of deteriorated animal and vegetable matter. They are usually dark in color and give off a distinctive odors [10].

2.2 Parent material

There are to primary factor in parent material that influences soil: grain size and composition. Grain size is the most determinant of soil texture. Texture has impact on the soil structure, consistency, cation exchange capacity, profile drainage, moisture holding capacity and organic content [11].

2.3 Soil classification

Soil classification makes a difference the engineer to assign a soil to one of a number of group based on the fabric property and behavior of soil. Two major soil classification systems that take in to consideration both particle-size distribution and Atterberg limits are available for general engineering use. They are the AASHTO (American association of State Highway and Transport official) classification frame work and USCS [10].

2.4 General type of soil

According to their grain size, soil particles are classified as cobbles, gravel, sand, silt and clay. Grains having diameters in the range of 4.75 to 76.2 mm are called gravel. If the grains are visible to the naked eye, but are less than about 4.75 mm in size the soil is described as sand. The lower limit of visibility of grains for the naked eyes is about 0.075 mm. Soil grains ranging from 0.075 to 0.002 mm are termed as silt and those that are finer than 0.002 mm as clay. This classification is purely based on size which does not indicate the properties of fine grained materials [17].

2.5 Soil particle size and shape

The size of particles may range from gravel to the finest size possible. Their characteristics vary with the size. Soil particles coarser than 0.075mm 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 [17].

2.6 Structure of soil

The orientation of particles in a mass depends on the size and shape of the grains as well as upon the minerals of which the grains are formed. The structure of soils that is formed by natural deposition can be altered by external forces [17].

Generally various types of structures of soil for example

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

2.7 Mineralogy of clay

The minerals of clay are formed by weathering of rock. The clay minerals are made out of oxygen and silicon which are the two most abundant components in the earth. Silicate are a group of mineral with basic unit called the silica tetrahedron a central silica cation is encircled by four oxygen anions one at each edge of the tetrahedron. Silica tetrahedron join to form sheet called silica sheet. Silicate sheet may contain other structural unit for example, Alumina sheet, alumina sheet are formed by a combination of alumina minerals which comprises of an alumina particle encompassed by six oxygen atoms in an octahedron [6]

The principal group of clay crystalline materials that make up clay the minerals kaolinite, illite, montmorillonite

2.7.1 Kaolinite

Kaolinite has a structure that comprises of one silica sheet and one alumina sheet bonded together in to a layer about 0.72nm (10^{-9} m) thick and stacked over and again. The layers are held together by hydrogen bound [5]. Kaolinite is moderately stable and water can't enter between the layers. Therefore, kaolinite shows small swelling on wetting [24]. Kaolinite has a few or no exchangeable cation. The inter layer bonds are relatively strong preventing any hydration between the layer.

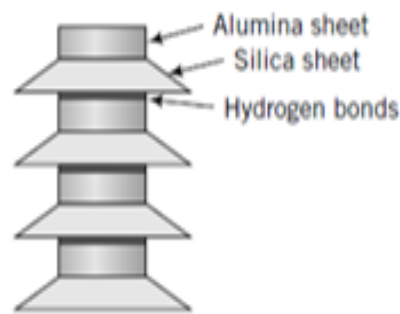


Figure 2.1:- Structure of Kaolinite [27].

2.7.2 Illite

The Illite to some degree like montmorillonite in the structural unit but the layers are strongly bonded by potassium atom. They develop less affinity for water than montmorillonite and illite normally happen as very small flaky particles mixed with other clays non clay material [3].

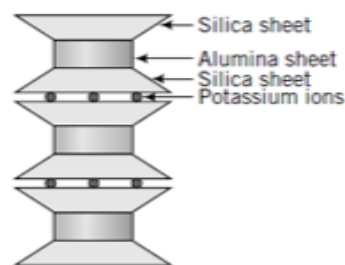


Figure 2.2:- structure of illite [27].

2.7.3 Montmorillonite

Composed of an octahedral sheet between two tetrahedral sheets the bonding between the silica sheets is represented by Vander Waals' forces [3].

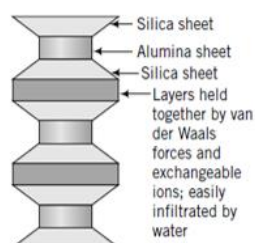


Figure 2.3:- Structure of montmorillonite [27].

2.8. Previously studied on the investigation of engineering property of soil

The direct shear test result indicates that areas near Lake Haromaya are cohesion less soil material with angle of internal friction, $\phi = 27.30$ and cohesion, $C = 2.35$ (KN/m²). According to the Unified Soil Classification System, the soil in Haromaya Town is categorized as CH (high plastic clay soil), CL (low to medium plastic clay soil) and SM (silty sand). The AASHTO Classification System shows that most of soils of study areas categorized on A-7-6 and A-6 therefore those soils have poor quality for use as subgrade [28].

According to [7] test result Grain size analysis tests shows that, starting from few centimeters below the ground level to the depth of investigation which is three meters, the soil in Adama town is mostly silt, and silt sand soil in which the percentage of clay ranges from 5-32%, silt from 18-62%, sand from 15-55% and gravel 0-25%. This value of grain size analysis indicates that the soil is fine grained soils. And the specific gravity of the soils ranging from 2.60-2.70. From consistency limit test results the liquid limit of the area ranges from 29-73% plastic limit ranges from 21-39% and plastic index from 5-34%. This values indicates that the soil is under silt and clay range. From soil classification by method of AASHTO the soils are classified under A-2-4, A-4. & A-7-5/6. The A-2-4 value indicates that the soil is silt and clayey gravel and sand with subgrade rating excellent to good and value of A-4 indicates that silt soil with fair to poor subgrade rating. And also the value of A-7-5/6 indicates that clayey soil with fair to poor subgrade rating. According to Unified Soil Classification System the plasticity chart shows that the soil under investigation lies below the A-line in the region of inorganic silt and inorganic elastic silt. That means inorganic silt with low to high plasticity.

The classification of soil in Debrebirhan town Soil classification for the area under study is made by both USCS and AASHTO. Soils classification by USCS shows that the soil contains around 41% CH, 18% CL, 18% MH, 14% ML, 4.5% SM and 4.5% SC and AASTHO classification system shows the soils are classified in either of A-6 or A-7 [21].

Comparison of different researcher result as shown in table 2.1

Table 2.1 Previous index property test result

Item	Haromaya [28]	Adama [7]	Debre birhan [21]
Gravel ,%	0-6.9	0-24.85	0-24.63
Sand, %	6.88-39.6	14.5-54.6	2.37-38.79
Silt, %	18.3-36.7	17.6-60.7	27.28-55.9
Clay, %	36.1-65.75	5.4-40.5	8.77-67.5
Liquid limit, %	86.4-87.3	29-73	31-80
Plastic limit, %	23-23.8	21-35	18-43
Plastic index, %	63.3-63.4	13-34	13-46
Specific gravity, %	2.7-2.75	2.4-2.7	2.62-2.81
Free swell, %	20-113	18-50	50-100

From table 2.1 the index property of the soil in different location or town was different.

2.9 Summary of the literature review

The soil can be formed from physical, chemical, and biological weathering of rock. As a result of this, the factor that influence soil parent material is its grain size and the composition of the soil material. 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. In order to classify the soil USCS and AASHTO soil classification system is used for common understanding among the professional. Moreover, the structure of soil orientation particles mass depends on the size, shape and minerals where grains formed. The principal group of clay crystalline materials that make up clay the minerals kaolinite, illite, montmorillonite In general, investigation of the soil in different location its index and engineering property of the soil become different.

CHAPTER THREE

Methodology and Description of study area

3.1 Methodology

To fulfill the research objective ten sampling area were selected. From the chosen sampling area subsurface investigation was done by hand excavation with posthole diggers test pit 3m depth and then disturbed and undisturbed soil sample is collected for laboratory testing. In the field GPS reading was taken to locate the coordinate of sampling area. And also the following activity were done

- In processing all the design and analysis, identify literature review of research, and data gathered evaluated to come up with the research output.
- The specific gravity test used in order to evaluate the unit weight of the soil.
- To determine the engineering property of soil Atterberg test were done.
- Modified compaction test is performed to determine the relationship between the moisture content and the dry density of a specific soil sample at a specified effort.
- In order to use the triaxial test Samples of soil for the test is remolded using the maximum dry density (MDD) and optimum moisture content (OMC) for disturbed soil sample. The test was done at submerged (saturated) condition. This test method provides data useful in determining strength of the soils such as Mohr strength envelopes.
- Required materials: - Group of sieves test apparatus, Compaction test Apparatus Atterberg limits Apparatus, triaxial test Apparatus, Vehicle, Sample bag, Sample taking instrument.



Figure 3.1:- Schematic diagram

3.2 Description of laboratory test

3.2.1 Atterberg limit of soil

The engineering properties of fine-grained soils vary with the amount of water present. As the water content changes, the consistency of these soils will also change as the material passes through the various physical states (namely liquid, plastic, and semisolid, solid). In 1911, A. Atterberg established limiting water contents for the different physical states engineering behaviors. These limits are known as the Atterberg limits and consist of the liquid limit, the plastic limit and the shrinkage limit. The Atterberg limits are water contents (expressed as percentages) where the soil behavior changes [14].

Above the liquid limit, LL, the soil-water system is a suspension. Below the liquid-limit and above the plastic limit, PL, the soil-water system is said to be in a plastic state. In this state the soil may be deformed or remolded without the formation of cracks and without change in volume. The range of water content over which the soil-water system acts as a plastic material is frequently referred to as the plastic range, and the number difference is called the plastic index.

$$I_p = LL - PL \dots\dots\dots (4.2)$$

Somewhat below the plastic limit, the soil-water system reaches the shrinkage limit, SL. Reduction of the water content by drying below the shrinkage limit is not accompanied by a decrease in volume; instead air enters the voids of the system and the material become unsaturated. The Atterberg limits vary with the amount of clay present in a soil, the type of clay mineral, and the nature of the ions absorbed on the clay surface.

The most common use of the Atterberg test results is for soil classification. Soils with comparable limits and indices are classed together. The number is used to classify fine grained soils and the indices to characterize soil behavior. Generally, soils with high liquid limits are clays with poor engineering properties. A low plasticity index indicates a soil with little or no cohesion and plasticity, such as a granular type soil. Both the liquid limit and plasticity index are used to some degree as a quality measuring device for pavement materials [14]

3.2.2 Grain size analysis test result

In this system soils are split into coarse-grained non-cohesive and fine-grained cohesive soils. Most systems of soil classification depend to some extent upon the distribution of various sized particles in the soil. For coarse-grained material this distribution may be determined by sieving, and for finer particles a method of measuring the rate of settlement in water is used [3].

- Sample – Disturbed but representative

Grain size analysis of a soil is carried out in two stages:

- ✓ Sieve analysis for coarse grained particles
- ✓ Hydrometer analysis for fine grained particles

A. Sieve Analysis.

It is carried out by sieving a known dry weight of sample through the set of sieves placed one below the other, so that the openings decrease in size from the top sieve down, with a pan at the bottom of the stock. The whole set of sieves is given a horizontal shaking for about ten minutes until the weight of soil remaining on each sieve reaches a constant value. By determining the weight of soil sample left on each sieve, then the calculations can be made [3].

B. Hydrometer Analysis

Soil particle sizes smaller than 0.075 mm (passing 200 mesh sieve) are determined by the so-called hydrometer method it is based on the process of sedimentation of soil particles in water by gravity. Sedimentation by gravity is the separation of particles of various sizes by their velocity. Coarse soil suspension settles out more rapidly than the finer ones of the same specific gravity the hydrometer method of analysis is based on Stoke's law of settlement i.e. small spheres in a liquid settle at different rate according to the size of the sphere. The terminal velocity of a spherical soil particle settling in water is expressed by Stoke's law as [14]

$$v = \frac{\gamma_s - \gamma_w}{18\eta} D^2 \dots\dots\dots (4.5)$$

Where γ_s = unit weight of spherical grain (g/cm^3), γ_w = unit weight of water (g/cm^3),

v = velocity (cm/s), D = diameter of the soil particle, η = Viscosity of water ($\frac{\text{g.s}}{\text{cm}^2}$)

3.2.3 Test Procedure and results

By using ASTM D422-63 test procedure soil had been tested. The soil particle diameter less than $75\mu\text{m}$ was done by hydrometer test and for coarser soil i.e. for particle diameter greater than $75\mu\text{m}$ was done by using mechanical sieve first of all the soil sample that is collected from the site air dried then it had been washed on sieve NO. 200. Oven dry the soil samples then take 50gm of soil for hydrometer analysis Characterizing particle size of soil like silt and clay. material for the test was mixing contained to mix the soil for 60 second, 125ml of dispersing agent, graduated cylinder, distilled water which was mix with slurry filled up to 1000ml on the graduated cylinder, rubber gloves the top seal and mixing the solution for a minute then a hydrometer instrument was inserted on the solution. After that reading taken at the bottom of the meniscus that was the curve in the upper surface of the container caused by surface tension. The elapsed time was 2, 5, 15, 30, 60, 250, 1440 minutes.

3.2.4 Triaxial test

The triaxial test is one among the for most versatile and widely performed geotechnical laboratory tests, allowing the shear strength and stiffness of soil and rock to be determined to be used in geotechnical design Advantages over simpler procedures, like the direct shear test, include the power to control specimen drainage and take measurements of pore water pressures. Primary parameters obtained from the test may include the angle of shearing resistance ϕ' , cohesion c' , and undrained shear strength c_u , although other parameters like the shear stiffness G , compression index C_c , and permeability k may also be determined [20].

The triaxial test typically involves placing a cylindrical specimen of soil, starting from 38mm to 100mm diameter, into a cell which will be pressurized. Most specimens have an approximate 2:1 height-to-diameter ratio, and are sealed within a rubber membrane. Following this first preparation the specimen is saturated, consolidated, and sheared, allowing the soil response to be observed under conditions which will approximate those insitu. During the shear stage the soil is loaded axially, either in compression, or less-commonly in extension [20].

There are three types of triaxial tests conducted with in the laboratory, each allowing the soil response for differing engineering applications to be observed [20]. These are

- Unconsolidated Undrained test (UU)
- Consolidated Undrained test (CU)
- Consolidated Drained test (CD)

For this research the Unconsolidated undrained test (UU) had been used the test is the simplest and fastest procedure, with soil specimens loaded whilst only total stresses are controlled and recorded. This allows the undrained shear strength c_u to be determined, which is suitable for assessing soil stability within the short-term (e.g. during or directly following a construction project). This test is generally performed on cohesive soil specimens. This test method covers determination of the strength and stress-strain relationships of a cylindrical specimen of either undisturbed or remolded cohesive soil. Specimens are subjected to a confining fluid pressure in a triaxial chamber. No drainage of the specimen is permitted during the test. The specimen is sheared in compression without drainage at a continuing rate of axial deformation [20].

3.2.5 Test procedure

Place the membrane on the specimen, and then place the membrane onto the cap or base. After that Place the specimen on the base. Finally Place the rubber membrane around the specimen and seal it at the cap and base with O-rings or other positive seals at each end. A thin coating of silicon grease on the vertical surfaces of the cap or base will aid in sealing the membrane [4].

With the specimen encased in the rubber membrane, which is sealed to the specimen cap and base and positioned in the chamber, assemble the triaxial chamber. Bring the axial load piston into contact with the specimen cap several times to permit proper seating and alignment of the piston with the cap. When the piston is brought into contact the final time, record the reading on the deformation indicator to 3 significant digits [4]. During this procedure, look out not to apply an axial stress to the specimen exceeding approximately 0.5 % and look after the estimated compressive strength. If the load of the piston is sufficient to use an axial stress exceeding approximately 0.5 % of the estimated compressive strength, lock the piston in place above the specimen cap after checking the seating and alignment and keep locked until application of the chamber pressure. Place the chamber in position in the axial loading device. Be careful to align the axial loading device, the axial load-measuring device, and the triaxial chamber to stop the appliance of a lateral force to the piston during testing [4].

Attach the pressure-maintaining and measurement device and fill the chamber with the confining liquid. Adjust the pressure maintaining and measurement device to the desired chamber pressure and apply the pressure to the chamber fluid. Wait approximately 10 min after the appliance of chamber pressure to allow the specimen to stabilize under the chamber pressure prior to application of the axial load. Apply the axial load to produce axial strain at a rate of approximately 1 %/min that achieve maximum deviator stress at approximately 3 to 6 % strain. At these rates, the time period to succeed maximum deviator stress are going to be approximately 15 to twenty min. Continue the loading to fifteen % axial strain, except loading could also be stopped when the deviator stress has peaked then dropped 20 % or the axial strain has reached 5 % well beyond the strain at which the highest in deviator stress occurred. Record load and deformation values to three significant digits at about 0.1, 0.2, 0.3, 0.4, and 0.5 % strain; then at increments of about 0.5 % strain to three %; and, thereafter at every 1 %. Take sufficient readings to define the stress-strain curve; hence, more frequent readings could also be required within the early stages of the test and as failure is approached [4].

3.2.6 Consolidation

The time dependent dissipation of excess pore water pressure and long term reduction in void ratio, e , caused by the loading of fine grained soil is referred to as consolidation. Consolidation is a coupled process between flow and volume change. The aim of consolidation test is to quantify both the ultimate amount of settlement in the layer, a one dimensional consolidation test is performed in the laboratory and using laboratory derived parameter field settlement behavior of the soil layer can be predicted. At high void ratios the curve is concave down word but it becomes almost straight line. The nearly straight line portion of the curve is known as the virgin consolidation line or virgin branch. When stress is removed from a consolidation soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process if the stress is re applied the soil will consolidate again along re-compression curve, defined by the re- compression index the soil which had its load removed is considered to be to be over consolidated. The highest stress that it has been subjected to is termed as the pre-consolidation stress. A load increases imposed by the construction of foundation or other compressible soil layer load [13]. The compression is caused by

- Deformation of soil particles
- Relocation of soil particles
- Expulsion of water or air from the void space.

Generally the soil settlement imposed by loads may be divided in to three broad categories

- Elastic settlement
- Primary consolidation settlement
- Secondary consolidation settlement

When a saturated compressible clay layer is subjected to load increase elastic settlement happen immediately because the hydraulic conductivity of clay is significantly smaller than that of sand, the excess pore water pressure generated by loading gradually dissipated over a long period. Thus the associated volume change in the clay may continue long after the elastic settlement. When foundation constructed on very compressible clays the consolidation settlement can be several times greater than the elastic settlement [11].

3.2.7 Test procedure

The one dimensional consolidation testing procedure was first suggested by terzaghi .This test is performed on ASTM D2435 standard. The test performed in consolidometer (sometimes referred to as an oedometer). The soil specimen is placed inside a metal ring with two porous stones one at the top of specimen and another at the bottom. The Load was applied through the lever arm and compression dial reading was taken at a time interval of 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, minutes and 1, 2, 4, 8, and 24hours and a compression was measured by a micrometer dial gauge the specimen is kept under water during the test. Each load usually kept for 24 hours after that the load usually doubled starting from 111kpa to 1776kpa the pressure on the specimen and the compression measurement is continued at the end of the test.

3.2.8 Description of study area

The study area is located in Gumer woreda, Guragea zone, in the Southern Nations, Nationalities, and Peoples' Region of Ethiopia. Gumer woreda is one of the fifteen woredas of the Guragea zone. The wored's capital, Arekit is located 220 kilometers south of Addis Ababa. This woreda is named after one of the sub- groups of the Sebat Bet Gurage, the Gumer. Part of the Gurage Zone, Gumer is bordered on the southeast by the Silt'e Zone, on the southwest by Geta, on the northwest by Cheha, and on the north by Ezha .The Arkit town has a total area of 23,555 hectares .

The geographical location of this town area is around 7° 54' 59.99" N, 38° 04' 60" E, with altitudinal range from 2700–3078 meter a.m.s.l. And also in winter time the water permeability is minimum .In addition, the major soil types observed along the area of the proposed Arekit town are red clayey soil and also in some areas there are Brown to dark clayey

Soil. so this, erosion of soil is major problem and also the soil is very sticky and very plastic when wet. The dominant land use or land cover of the area is characterized by intensively cultivated and settled rural settings. Perennial crops like inset, coffee and fruit trees are intensively cultivated along the area,

3.2.9 Soil and geology

"The geology of the area is assessed using 1:2,000,000 Geological Map of Ethiopia, 1996. According to this Map of Ethiopia, the geology of the area is divided into the following rock formations:

➤ The Nazret Series

It consists of Ignimbrites, unwelded tuffs, ash flows, rhyolite flows, and trachyte.

➤ Quaternary formation.

It consists of rhyolitic volcanic centers, obsidian pitchstone, pumice, ignimbrite, tuff, subordinate trachytic flows (predominantly peralkaline in composition). This area also consists field of small Baltic vents. The major soil cover observed in the town are red clayey soil and also in some areas there are Brown to dark clayey soils" [16].

3.2.10 Climate

The climate data of the area is not readily available there is no metrological station in Arekit town but from Ethiopian road authority environmental impact assessment report as shown in Table 3.1. This will help to know the working environment of the area by estimating temperature distribution of the area. Climatic data will also be employed in the specifying parameters affected by amount of rain such as plasticity index of embankment and wearing course materials. The Arekit town when categorized climatically lies in between Cool Temperate (Dega). The altitude of Arkit town is about 2700–3078 meter above sea level.

Table3.1:-Climate of the study area [16].

Altitude (m)	Description	Local equivalent	Major towns or village along Wolkite - Hossaina road project
>3300	Cool		
2300-3300	Cool temperate	Dega	Dakuna, moche, yowere, jembero, Arekit, Kebul Mugo, Bilalu, lera, Damala, Shurmo, Hossaina
1500-2300	Temperate	Weina Dega	Wolkite, Gubre, Amora Meda, Jefek, Terek, Indibir

CHAPTER FOUR

Laboratory test result

4.0 Visual description of soil in the study area

In order to select sampling area visual site investigation, information from the existing community and agricultural authority were asked to consider the different soil type and to take sample randomly from the area. After assessing the area ten test pit excavated to a maximum depth of 3m. And, eight undisturbed soil sample and disturbed soil sample taken in February 13, 2012 Ethiopian calendar. From the visual description most part of the town are covered with red clay soil and some are brown to dark clay soil. The global coordinate of sampling location are shown in the table 4.0 below.

Table 4.0 Global coordinate of sampling area

Test pit	Location	Polarization	Coordinate		Altitude (m)
			Easting	Northing	
TP-1	Menahariya	37N	38°06'30"	7°92'70"	2893
TP-2	Selam	37N	38°09'20"	7°94'40"	2889
TP-3	Gurguzer	37N	38°05'80"	7°94'44"	2901
TP-4	T'emren Hayk	37N	38°08'10"	7°94'60"	2889
TP-5	High school	37N	38°06'80"	7°94'80"	2896
TP-6	Stadium	37N	38°06'90"	7°93'10"	2879
TP-7	Koto	37N	38°05'10"	7°93'00"	2891
TP-8	Agro-industry	37N	38°08'80"	7°93'00"	2892
TP-9	Deche	37N	38°07'70"	7°92'80"	2888
TP-10	Jame	37N	38°07'00"	7°94'40"	2874

4.1. Natural moisture content

ASTM D2216 is used to determine the NMC. The moisture content of soil is to assign the amount of water present in soil. By definition, moisture content is the ratio of the mass of water to the mass of solids in the sample expressed as a percentage [4]. From table 4.1 the natural moisture content of the soil ranges 20-45%. But, at TP-4 the natural moisture content was 82.05 %, it express soft organic clay [14].

4.2. Specific gravity of soil

In general, the term specific gravity is defined as the ratio of the mass of a given volume of a material to the mass of an equal volume of water. In effect, it tells us how much the material is heavier than (or lighter) than water. The particular specific gravity of a soil actually denotes the specific gravity of the solid matter of the soil and refers, therefore, to the ratio of the mass of solid matter of a given soil sample to the mass of an equal volume of water. The specific gravity of the soil ranges 2.65-2.78 According to ASTM D854-92 which is inorganic soil.

Table 4.1:- Natural moisture content and specific gravity of soil

Test pit	Depth (m)	NMC (%)	Specific gravity
TP-1	1.5	24	2.78
	3	29	2.65
TP-2	1.5	25	2.74
	3	28	2.75
TP-3	1.5	27	2.75
	3	33	2.71
TP-4	1.5	67	2.75
	3	82.05	2.78
TP-5	1.5	24	2.75
	3	45	2.67
TP-6	1.5	57	2.68
	3	34	2.78
TP-7	1.5	35	2.70
	3	42	2.78
TP-8	1.5	26	2.75
	3	20	2.76
TP-9	1.5	27	2.78
	3	25	2.71
TP-10	1.5	25	2.78
	3	20	2.77

4.3. Free swell test

IS2720 standard test for the determination of free swell index of soil which helps used to identify the potential of a soil to swell which might need further detailed investigation regarding swelling and swelling pressures under different field conditions. Free swell test is the simplest test conducted to study the swelling property of 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:

$$\text{Free swell test} = \frac{\text{Final volume} - \text{initial volume}}{\text{initial volume}} * 100 \dots\dots\dots (4.1)$$

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

Table 4.2:- Free swell soil test result for the selected soil sample

Test pit	Depth (m)	Free swell in %
TP-1	3	15
TP-2	3	45
TP-3	3	30
TP-4	3	70
TP-5	3	38
TP-6	3	10
TP-7	3	30
TP-8	3	50
TP-9	3	30
TP-10	3	20

The free swell test result shows the study area doesn't have expansive nature except in TP-4 with free swell 70% it shows moderate expansive nature.

4.4. Atterberg limit test

This test was done according to ASTM D 4318. Atterberg proposed four state of soil and a corresponding three boundaries. It is one of the index property test and used to determine the degree of firmness of the soil.

Table 4.3:- Atterberg limit test result

Test pit	Depth (m)	LL (%)	PL (%)	Ip (%)
TP-1	1.5	36.77	16.60	20.17
	3	29.33	25.57	3.80
TP-2	1.5	34.32	19.73	14.59
	3	49.81	27.24	22.57
TP-3	1.5	50.82	26.55	24.27
	3	43.48	24.88	18.61
TP-4	1.5	72.95	44.12	28.83
	3	78.12	46.45	31.67
TP-5	1.5	58.97	27.49	31.48
	3	56.90	36.87	20.03
TP-6	1.5	55.11	37.29	17.82
	3	37.37	22.58	14.79
TP-7	1.5	61.28	29.64	31.63
	3	50.64	29.47	21.17
TP-8	1.5	37.15	18.62	18.53
	3	46.95	29.59	17.36
TP-9	1.5	47.58	20.60	26.98
	3	42.26	24.37	17.89
TP-10	1.5	45.17	24.01	21.17
	3	37.22	23.87	13.35

From the table the LL ranges from 29.33-78.12%, PL ranges from 16.60-46.45 % and Ip ranges from 3.80-31.48%. According to [14] if the liquid limit in the range 35-100 the clay mineral they contain become kaolinite. As a result most of the soil found in study area falls under clay minerals kaolinite. In addition to this based on [14] the plastic index ranges from 20-40 the clay minerals became kaolinite so in table 4.4 the soil found in Arekit town its plastic index in most of the test pit under the clay mineral kaolinite.

4.5 Liquidity Index

The liquidity index of a soil indicates the nearness of its water content to its liquid limit. When the soil is at its liquid limit, its liquidity index is 100% and it behaves as a liquid. When soil is at the plastic limit, its liquidity index is zero. A negative value of the liquidity index indicates water content smaller than the plastic limit. The soil is then in hard state. The liquidity index is also called Water-Plasticity ratio [3]. Liquidity index is given by

$$I_l = \frac{w - w_p}{I_p} \dots\dots\dots (4.3)$$

Where: - I_l = Liquidity index, w = water content, w_p = plasticity limit and I_p = plasticity index

When $I_l < 0$: soil is in semi-plastic solid state or solid state, $I_l = 0$: soil is very stiff state,

$0 < I_l < 1$: soil is in plastic state and $I_l > 1$: soil is in liquid state [6].

4.5.1 Consistency index

"The consistency of soil indicate the firmness of the soil. it shows the nearness of the water content of the soil to its plastic limit. When a soil with consistency index is between 0-25 is very soft, 25-50 is soft, 50-75 medium firm, and 75-100 stiff and also greater than 100 is Very stiff. Consistency index is given by

$$I_c = \frac{w_l - w}{I_p} * 100 \dots\dots\dots (4.4)$$

Where I_c = consistency index, w_l = liquid limit, w = natural moisture content, I_p = plastic index" [3].

Table 4.4:- Liquidity and consistency index of the study area

Test pit	Depth (m)	LL (%)	PL (%)	Ip (%)	Consistency Index (%)	Liquidity index	Remark
TP-1	1.5	36.77	16.60	20.17	63.31	0.37	Plastic state
	3	29.33	25.57	3.80	8.68	0.90	Plastic state
TP-2	1.5	34.32	19.73	14.59	63.88	0.36	Plastic state
	3	49.81	27.24	22.57	96.63	0.03	Plastic state
TP-3	1.5	50.82	26.55	24.27	98.15	0.02	Plastic state
	3	43.48	24.88	18.61	56.31	0.44	Plastic state
TP-4	1.5	72.95	44.12	28.83	20.64	0.79	Plastic state
	3	78.12	46.45	31.67	-12.41	1.12	Liquid state
TP-5	1.5	58.97	27.49	31.48	111.09	-0.11	Semisolid state
	3	56.90	36.87	20.03	59.41	0.41	Plastic state
TP-6	1.5	55.11	37.29	17.82	-10.61	1.11	Liquid state
	3	37.37	22.58	14.79	22.79	0.77	Plastic state
TP-7	1.5	61.28	29.64	31.63	83.09	0.17	Plastic state
	3	50.64	29.47	21.17	40.81	0.59	Plastic state
TP-8	1.5	37.15	18.62	18.53	60.17	0.40	Plastic state
	3	46.95	29.59	17.36	155.24	-0.55	Semisolid state
TP-9	1.5	47.58	20.60	26.98	76.28	0.24	Plastic state
	3	42.26	24.37	17.89	96.48	0.04	Plastic state
TP-10	1.5	45.17	24.01	21.17	95.28	0.05	Plastic state
	3	37.22	23.87	13.35	136.48	-0.36	Semisolid state

4.6. Grain size analysis test result

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. In order to classify the soil USCS and AASHTO soil classification system is used with standard test method of ASTM D422.

Table 4.5:- Grain size analysis test result

Test pit	Depth (m)	Percentage amount of particle size			
		Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TP-1	1.5	0.00	8.59	48.25	43.16
	3	0.00	11.83	36.95	51.22
TP-2	1.5	0.13	12.71	42.94	44.22
	3	0.00	9.81	30.28	60.55
TP-3	1.5	0.00	10.33	29.92	59.75
	3	0.00	17.17	33.26	49.57
TP-4	1.5	0.16	8.07	32.76	59.00
	3	0.00	4.90	22.78	73.33
TP-5	1.5	0.10	5.84	34.49	59.57
	3	1.62	15.04	36.10	47.24
TP-6	1.5	0.00	0.73	23.75	75.52
	3	0.00	5.37	67.93	26.70
TP-7	1.5	0.00	22.58	40.26	37.17
	3	0.00	25.45	32.87	41.68
TP-8	1.5	11.98	22.53	40.57	24.92
	3	6.29	26.03	42.13	25.55
TP-9	1.5	0.00	8.29	37.01	54.71
	3	0.00	8.35	44.82	46.83
TP-10	1.5	2.86	26.96	36.94	33.24
	3	0.00	11.12	43.33	45.55

As mention in above table 4.6 of the grain size distribution test result that there is range of variation of the particle size. Therefore, silt and clay particles are major one. The range of the content in the grain size analysis test result are clay content ranging from 24.92-75.52%, silt fraction 22.78-67.93%, sand fraction 0.73-26.96 % and gravel content from 0-11.98%.

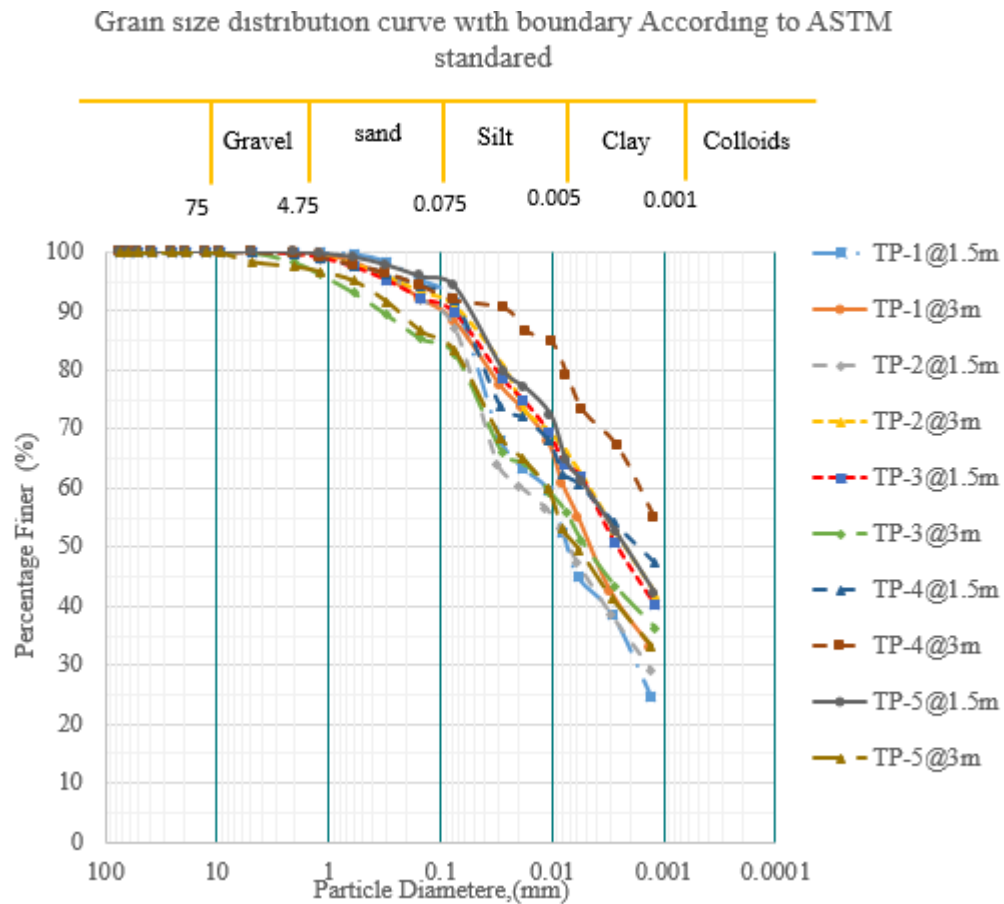


Figure 4.1 Grain size distribution curve for sample test pit 1 up to 5

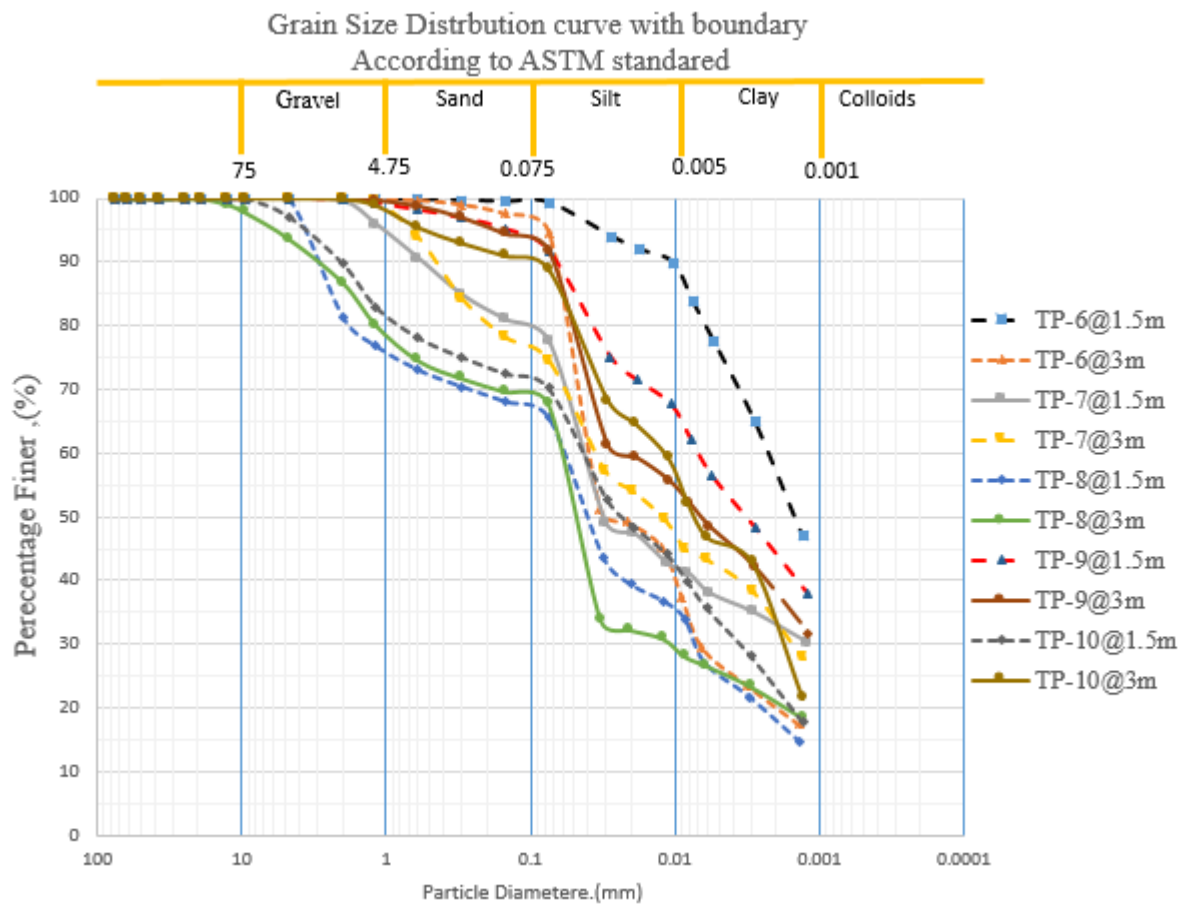


Figure 4.2 Grain size Distribution curve for sample test pit 6 up to 10

4.6.4. Classification of the soil

Soil classification enables the engineer to assign a soil to one of a limited number of groups based on the material property and characteristics of the soil. The classification groups are then used as a system of reference for soils [10].

There are different method of soil classification the most widely used classification system by engineers are USCS and AASHTO. For this research purpose the investigation of the soil is classified using AASTHO M-145 and USCS ASTM D2487-11.

4.6.5. Classification of soil based on unified soil classification system

The unified soil classification system of which can classify two types of soil when percentage passing through 200 sieve less than 50% as coarse grained soil which include gravel and sand and for percentage of passing through 200sieve greater than 50% as fine grained soil which include inorganic silt(M), inorganic clay (C) and in organic silt and clay (O). The name and symbols used to distinguish between the typical and boundary soil groups are GW, GP, GM, GC, SW, SP, SM and SC. The symbols that started with a prefix G stands for gravel or gravelly soil and symbol that started with S are sand or sandy soil. These symbols and their representations are: G-gravel, S-sand, M-silt and C-clay. These are combined with other symbols expressing gradation characteristics. ‘W’ for well graded and ‘P’ for poorly graded and plasticity characteristics ‘H’ for high and ‘L’ for low and symbol [3].

In order to use the unified soil classification table the following are essential.

- % of gravel passing or retained through sieve 4
- % of sand or silt and clay passing or retained through sieve 200
- Uniformity coefficient and coefficient of gradation
- Liquid limit and plastic index of soil passing through sieve 40.

The result of the unified soil classification of soil as shown in table 4.7.

Table 4.6:- Unified soil classification system

Assignable	Depth (m)	LL (%)	PI (%)	Percentage amount of particle size				Classification According to USCS
				Gravel (%)	Sand (%)	Silt (%)	Clay (%)	
TP-1	1.5	36.77	20.17	0.00	8.59	48.25	43.16	CL
	3	29.33	3.80	0.00	11.83	36.95	51.22	ML
TP-2	1.5	34.32	14.59	0.13	12.71	42.94	44.22	CL
	3	49.81	22.57	0.00	9.81	30.28	60.55	CL
TP-3	1.5	50.82	24.27	0.00	10.33	29.92	59.75	CH
	3	43.48	18.61	0.00	17.17	33.26	49.57	CL
TP-4	1.5	72.95	28.83	0.16	8.07	32.76	59.00	CH
	3	78.12	31.67	0.00	4.90	22.78	73.33	CH
TP-5	1.5	58.97	31.48	0.10	5.84	34.49	59.57	CH
	3	56.90	20.03	1.62	15.04	36.10	47.24	MH
TP-6	1.5	55.11	17.82	0.00	0.73	23.75	75.52	MH
	3	37.37	14.79	0.00	5.37	67.93	26.70	CL
TP-7	1.5	61.28	31.63	0.00	22.58	40.26	37.17	CH
	3	50.64	21.17	0.00	25.45	32.87	41.68	MH
TP-8	1.5	37.15	18.53	11.98	22.53	40.57	24.92	CL
	3	46.95	17.36	6.29	26.03	42.13	25.55	CL
TP-9	1.5	47.58	26.98	0.00	8.29	37.01	54.71	CL
	3	42.26	17.89	0.00	8.35	44.82	46.83	CL
TP-10	1.5	45.17	21.17	2.86	26.96	36.94	33.24	CL
	3	37.22	13.35	0.00	11.12	43.33	45.55	CL

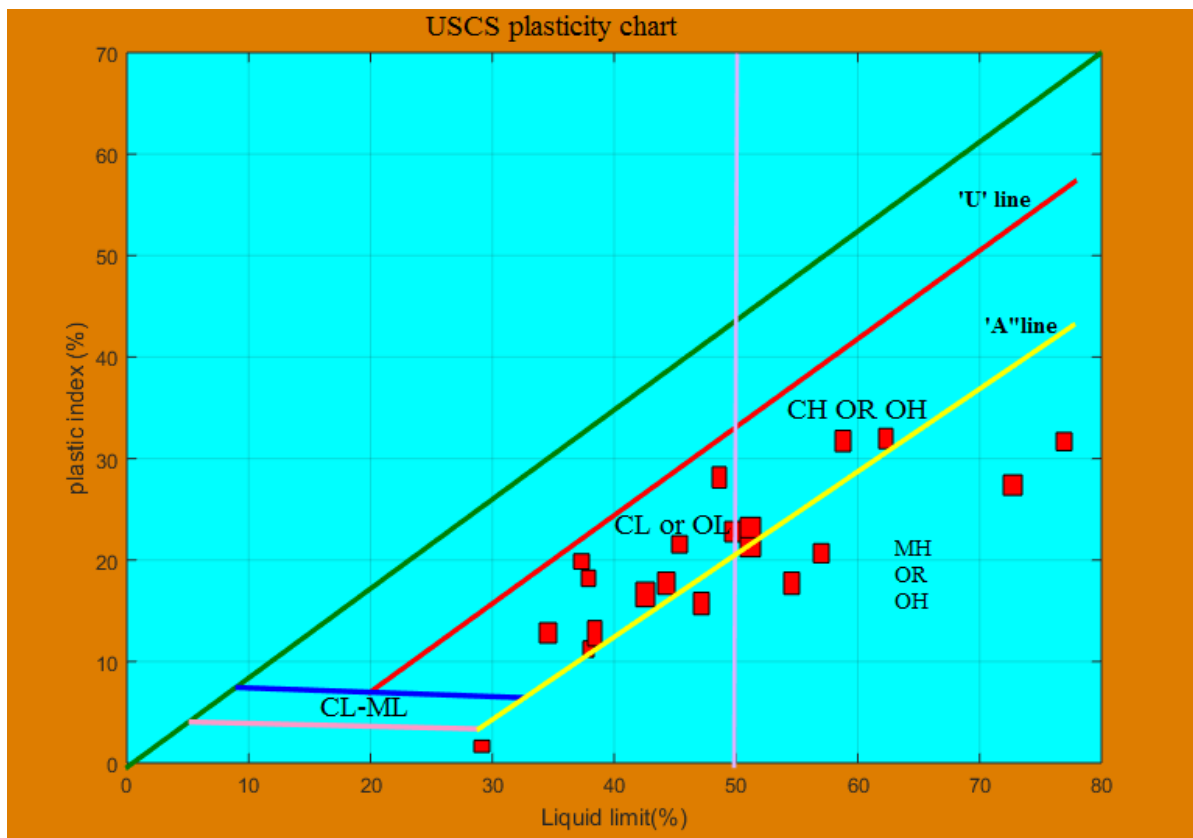


Figure 4.3 Plasticity chart of the study area according to unified soil classification system

From table 4.7 and figure 4.3 unified soil classifications result indicated that most of the soil found in Arekit town under CL and CH. Except in TP-1 at 3m depth it was ML and TP-5 at 1.5m , TP-6 at 3m ,TP-7 at 3m depth it was MH.

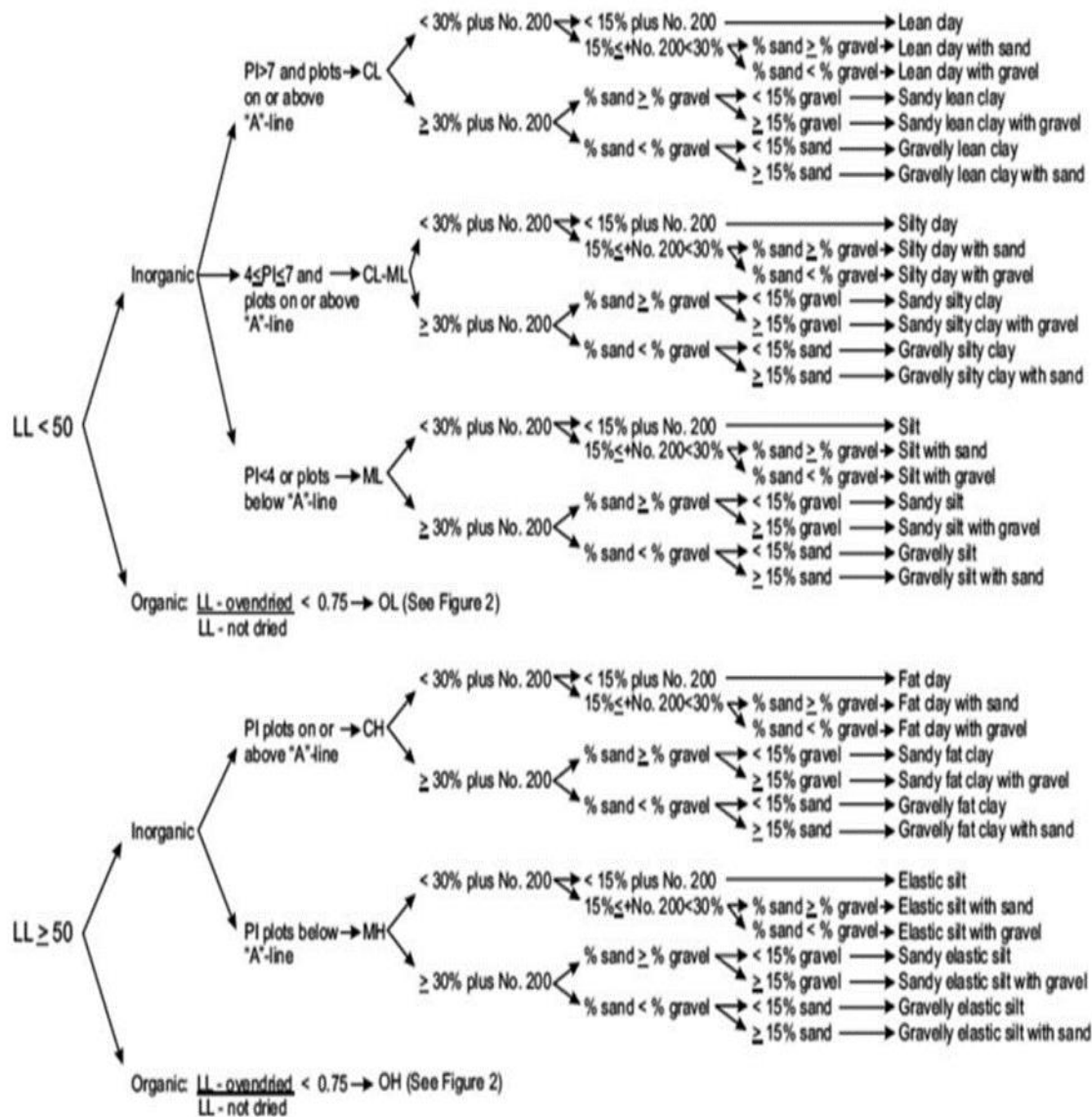


Figure 4.4 Flow chart for classifying fine grained soil using ASTM D2487-11

Table 4.7:- For classifying fine grained soil using ASTM D2487-11

Test pit	Depth (m)	LL (%)	PI (%)	Gravel (%)	Sand (%)	Group symbols	Group name
TP-1	1.5	36.77	20.17	0.00	8.59	CL	Sandy lean clay
	3	29.33	3.80	0.00	11.83	ML	Silt
TP-2	1.5	34.32	14.59	0.13	12.71	CL	Sandy lean clay
	3	49.81	22.57	0.00	9.81	CL	Sandy lean clay
TP-3	1.5	50.82	24.27	0.00	10.33	CH	Sandy fat clay
	3	43.48	18.61	0.00	17.17	CL	Sandy lean clay
TP-4	1.5	72.95	28.83	0.16	8.07	CH	Sandy fat clay
	3	78.12	31.67	0.00	4.90	CH	Sandy fat clay
TP-5	1.5	58.97	31.48	0.10	5.84	CH	Sandy fat clay
	3	56.90	20.03	1.62	15.04	MH	Sandy elastic silt
TP-6	1.5	55.11	17.82	0.00	0.73	MH	Sandy elastic silt
	3	37.37	14.79	0.00	5.37	CL	Sandy lean clay
TP-7	1.5	61.28	31.63	0.00	22.58	CH	Sandy fat clay
	3	50.64	21.17	0.00	25.45	MH	Sandy elastic silt
TP-8	1.5	37.15	18.53	11.98	22.53	CL	Sandy lean clay
	3	46.95	17.36	6.29	26.03	CL	Sandy lean clay
TP-9	1.5	47.58	26.98	0.00	8.29	CL	Sandy lean clay
	3	42.26	17.89	0.00	8.35	CL	Sandy lean clay
TP-10	1.5	45.17	21.17	2.86	26.96	CL	Sandy lean clay
	3	37.22	13.35	0.00	11.12	CL	Sandy lean clay

By using the flow chart for classifying fine grained soil stated in table 4.8 most of the soil in the area was sandy lean clay except the soil found in test pit one at depth of 3m it was silt and TP-3 ,TP-4,TP-5,TP-7 at 1.5m Sandy fat clay. Moreover, TP-5,TP-6 at 3m depth it was sandy elastic clay.

4.6.6 AASHTO soil classification system

AASHTO classification also included under ASTM D3284. Two type of soil can be classified in AASHTO when percentage of passing through 200 sieve is less than equal to 35% which is called a granular material then this granular material is further sub classified in to three groups those are A-1, A-2 and A-3. When the percentage of passing through sieve number 200 greater than 35% which is called silt and clay materials can be grouped in to four namely A-4, A-5, A-6 and A-7. Those points below are necessary in order to use the AASHTO table.

- %passing through sieve number 200
- Liquid limit and plastic index
- Group index (GI)

Except for TP-1 1.5m and 3m depth, TP-6 3m depth, TP-8 1.5m depth, and TP-10 3m depth most of the soil in the study area under A-7-6 and A-7-5 from the classification system of AASHTO. It is clear that A-7-6, A-7-5, A-6 material in order to use for high way construction can precipitate poor performance. Additionally, the group index shows 50% of soil in the area was below 20 which can be used as sub grade material. For the rest of the soil when the group index of the soil above 20 need stabilization or geotechnical solution. Generally according to AASHTO classification system indict the soil in the study area for the usual type of significant constituent material are clayey soil and also the general rating as sub grade material is fair to poor.

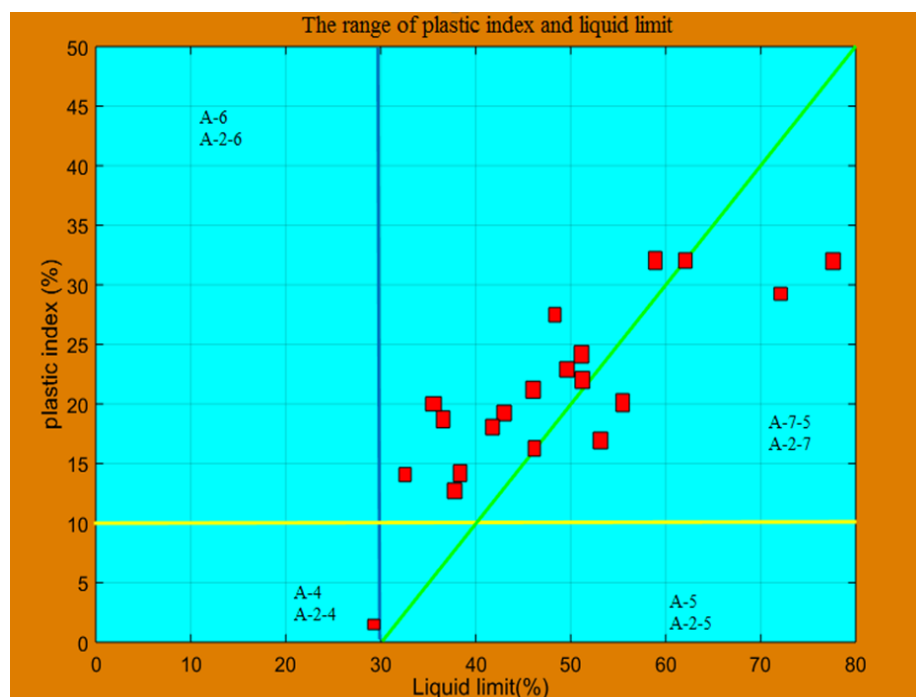


Figure 4.5:-Plasticity chart according to AASHTO soil classification system

Table 4.8:- AASHTO soil classification system

Designation	Depth (m)	%passing sieve NO.200	LL (%)	PI (%)	GI	Group Classification	Usual type of significance constituent material	General rating as sub grade material
TP-1	1.5	91.41	36.77	20.17	18	A-6	Clayey soils	Fair to poor
	3	88.17	29.33	3.80	3	A-4	Silty soil	Fair to poor
TP-2	1.5	87.16	34.32	14.59	12	A-6	Clayey soils	Fair to poor
	3	90.82	49.81	22.57	23	A-7-6	Clayey soils	Fair to poor
TP-3	1.5	89.67	50.82	24.27	25	A-7-6	Clayey soils	Fair to poor
	3	82.83	43.48	18.61	16	A-7-6	Clayey soils	Fair to poor
TP-4	1.5	91.77	72.95	28.83	35	A-7-5	Clayey soils	Fair to poor
	3	95.10	78.12	31.67	41	A-7-5	Clayey soils	Fair to poor
TP-5	1.5	94.06	58.97	31.48	35	A-7-6	Clayey soils	Fair to poor
	3	83.34	56.90	20.03	21	A-7-5	Clayey soils	Fair to poor
TP-6	1.5	99.27	55.11	17.82	24	A-7-5	Clayey soils	Fair to poor
	3	94.63	37.37	14.79	15	A-6	Clayey soils	Fair to poor
TP-7	1.5	77.42	61.28	31.63	27	A-7-6	Clayey soils	Fair to poor
	3	74.55	50.64	21.17	17	A-7-6	Clayey soils	Fair to poor
TP-8	1.5	65.49	37.15	18.53	10	A-6	Clayey soils	Fair to poor
	3	67.68	46.95	17.36	12	A-7-6	Clayey soils	Fair to poor
TP-9	1.5	91.71	47.58	26.98	27	A-7-6	Clayey soils	Fair to poor
	3	91.65	42.26	17.89	18	A-7-6	Clayey soils	Fair to poor
TP-10	1.5	70.18	45.17	21.17	14	A-7-6	Clayey soils	Fair to poor
	3	88.88	37.22	13.35	13	A-6	Clayey soils	Fair to poor

From figure 4.4 and table 4.9 AASHTO soil classification most of the soil falls under A-7-6 and A-7-5 except TP-1 at depth 1.5m, TP-6, TP-8 and TP-6 at depth of 3m which was A-6. Also TP-1 at depth of 3m it was A-4.

4.7. Activity of soil

Skempton classify the varies swelling characteristics of soils based on a number called as activity number, A_c [17]. The activity is expressed as

$$A_c = \frac{\text{plasticity index } I_p}{\text{percent finer than two micron}} \dots\dots\dots (4.6)$$

Table 4.9 Soil classification based on activity number

A_c	Soil type
< 0.75	In active
0.75-1.4	Normal
> 1.4	Active

Table 4.10:- The value of soil based on activity number

Test pit	Depth	I_p	Clay fraction (%)	A_c	Remark
TP-1	1.5	20.17	43.16	0.47	In active
	3	3.8	51.22	0.07	In active
TP-2	1.5	14.59	44.22	0.33	In active
	3	22.57	60.55	0.37	In active
TP-3	1.5	24.27	59.75	0.41	In active
	3	18.61	49.57	0.38	In active
TP-4	1.5	28.83	59	0.49	In active
	3	31.67	73.33	0.43	In active
TP-5	1.5	31.48	59.57	0.53	In active
	3	20.03	47.24	0.42	In active
TP-6	1.5	17.82	75.52	0.24	In active
	3	14.79	26.7	0.55	In active
TP-7	1.5	31.63	37.17	0.85	Normal
	3	21.17	41.68	0.51	In active
TP-8	1.5	18.53	24.92	0.74	In active
	3	17.36	25.55	0.68	In active
TP-9	1.5	26.98	54.71	0.49	In active
	3	17.89	46.83	0.38	In active
TP-10	1.5	21.17	33.24	0.64	In active
	3	13.35	45.55	0.29	In active

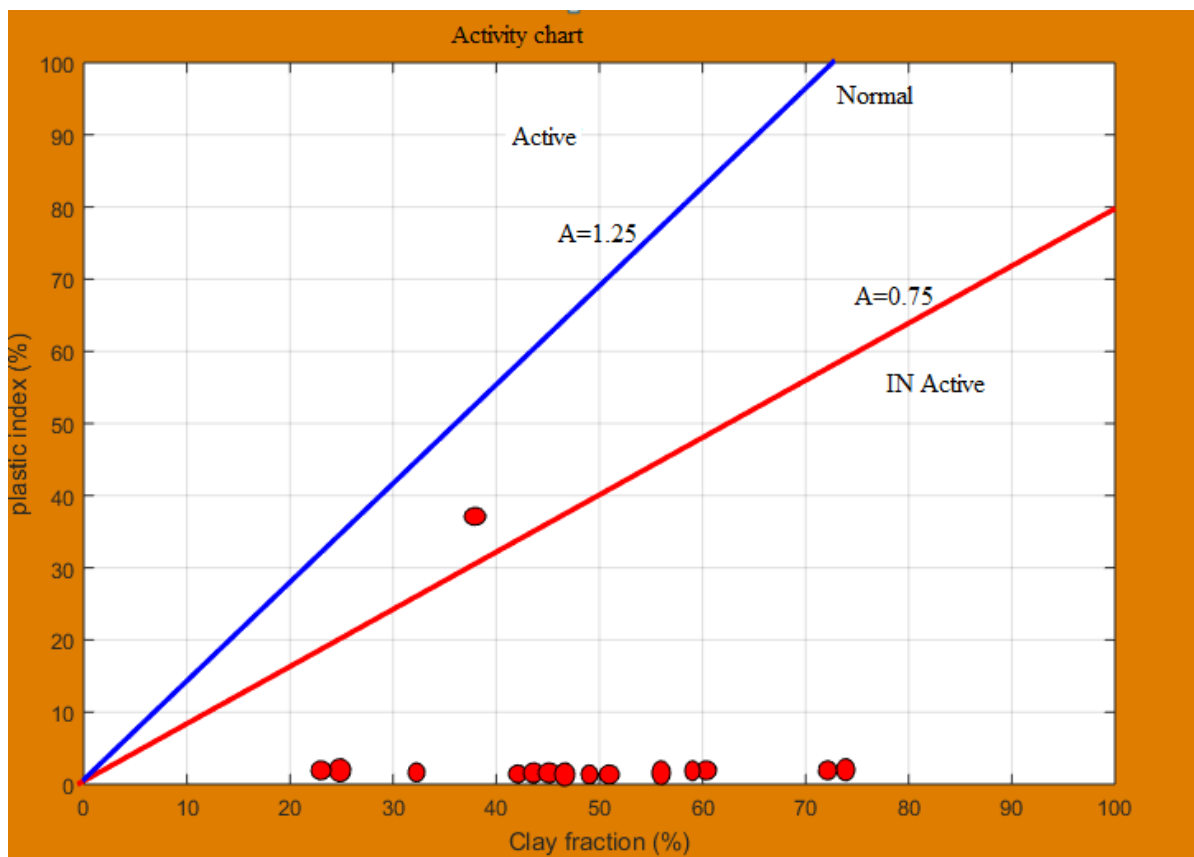


Figure 4.6:- Activity chart of soil in the study area

From table 4.11 and figure 4.5 most of the soil in the study area was in active except the soil found in TP-7 at 1.5m depth its activity number indicate normal.

4.8. Compaction test result

When soil is used as a construction material such as highways, embankment fills and for many other purposes it has to be compacted in an artificial way to:

- Decrease settlement
- To decrease permeability
- And finally to increase its shearing and compressive strength.

Through this compaction process water plays an important role. Up to certain water content the presence of water helps the compaction process as a lubricant for the grains rearrangement, at this stage water plays a positive role for compaction when we keep on increasing the water content in the soil sample to be compacted, compaction process will be very difficult, here presence of water prevents further compaction and adding water at this stage will make the process more difficult. The purpose of this laboratory test is to determine the maximum dry density and optimum moisture content. The results of this laboratory test will be used for the field compaction for example for highway construction, dam construction and so on.

The extent of compaction depends on the moisture content of the soil and the compactive effort used. In a compaction test the objective is to determine the optimum moisture content and maximum dry density achievable with a given compactive effort. A plot of dry density versus moisture content indicates that compaction becomes more efficient up to a certain moisture content, after which the efficiency decreases. The maximum dry density is obtained at this optimum moisture content.

If the compaction process were completely efficient, it would be possible (but not necessarily desirable) to expel all the air from the voids, in which case the dry density would correspond to a zero-air voids state (i.e. the sample would be saturated with water). Since perfect compaction is not possible (except at high moisture contents and low dry density) the compaction curve will always fall below the ideal or zero-air voids curve. It should be noted that there are a number of standards for compaction tests, each differing in the amount of energy input into compaction. For a given soil the different tests will produce different maximum dry densities and optimum moisture contents (i.e. these parameters are NOT soil properties). The maximum dry density and optimum moisture content are only relevant for a specified compaction procedure which should be stated when presenting the results. In earthworks it is common to specify a dry density within a certain percentage of the maximum

Determined from a specified compaction test. For this to be a sensible procedure it is important that the compactive effort used in the laboratory is comparable to that supplied by the field equipment [1].

Table 4.11:- Modified compaction test result

Test Pit	Depth (m)	MDD(gm/cc)	OMC (%)
TP-1	1.5	1.60	19.5
	3	1.59	20
TP-2	1.5	1.58	21
	3	1.41	28.5
TP-3	1.5	1.43	30.90
	3	1.41	32.00
TP-4	1.5	1.09	46.00
	3	1.18	30.89
TP-5	1.5	1.44	28.00
	3	1.28	36.00
TP-6	1.5	1.36	25.50
	3	1.53	23.00
TP-7	1.5	1.30	37.00
	3	1.41	27.10
TP-8	1.5	1.54	24.00
	3	1.31	34.00
TP-9	1.5	1.49	23.00
	3	1.47	27.00
TP-10	1.5	1.47	28.00
	3	1.38	32.30

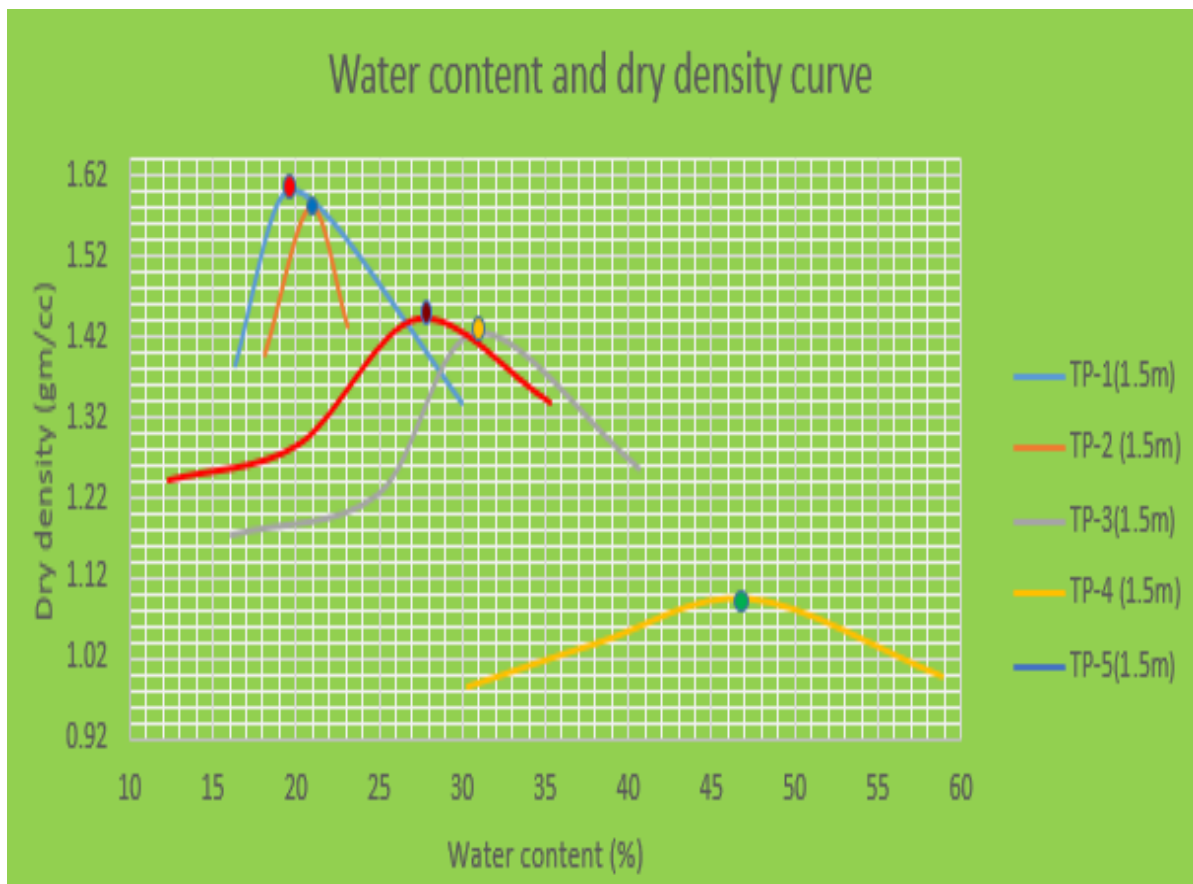


Figure 4.7:- Typical TP-1 up to TP-5 (1.5m) compaction curve

From the modified Compaction test result stated in table 4.12 the maximum dry density and optimum moisture content ranges from 1.01-1.6gm/cc and 19.5 - 46 % respectively

4.9 Triaxial test result

When the void of the soil contain both air and water the soil is said to be partially saturated. UU test is very vital for the case of the construction of a building on a surface foundation that increase the stress in the soil which means that the consolidation increase and a soil strength increases [5]. Unconsolidated undrained triaxial test of soils of the study area is conducted for undisturbed and remolded soil samples taken at a depth of 3m. The result are as shown in table 4.13.

The triaxial test was done in ASTM D 2850 the soil specimen subjected to three compressive stress at right angle to each other one of the three stress being increased until the sample faille in shear the soil sample placed on the pedestal of the base of the triaxial cell. And a thin rubber membrane is then placed over the sample and encircled by a triaxial cylinder then fix the rod in the triaxial base in order to avoid movement. And a loading cape is placed on the top. Finally assemble cell filled with water and a water subjected to a predetermined value of pressure and the additional axial stress is created by an axial load applied through a load transducer.

The stress–strain relations obtained from the triaxial tests provide the basis for determination of deformations in the field. This may be done by closed form solutions or by employing the results of the triaxial tests for calibration of a constitutive model used with a numerical method in finite element or finite difference computer programs. Usually, it is desirable to know how much change in load the soil can sustain without failing and how much deformation will occur under normal working conditions.

Table 4.12:- Triaxial soil test result

Test pit	Type of test	Depth (m)	Moisture content (%)	Chamber pressure (KN/m ²)			Deviator stress (KN/m ²)		
TP-4	disturbed	3	18.45	100	200	300	85.12	86.4	87
Tp-6	UN disturbed	3	27.85	100	200	300	248.48	249.5	256.12
TP-9	UN disturbed	3	28.86	100	200	300	391.3	394.76	399.48

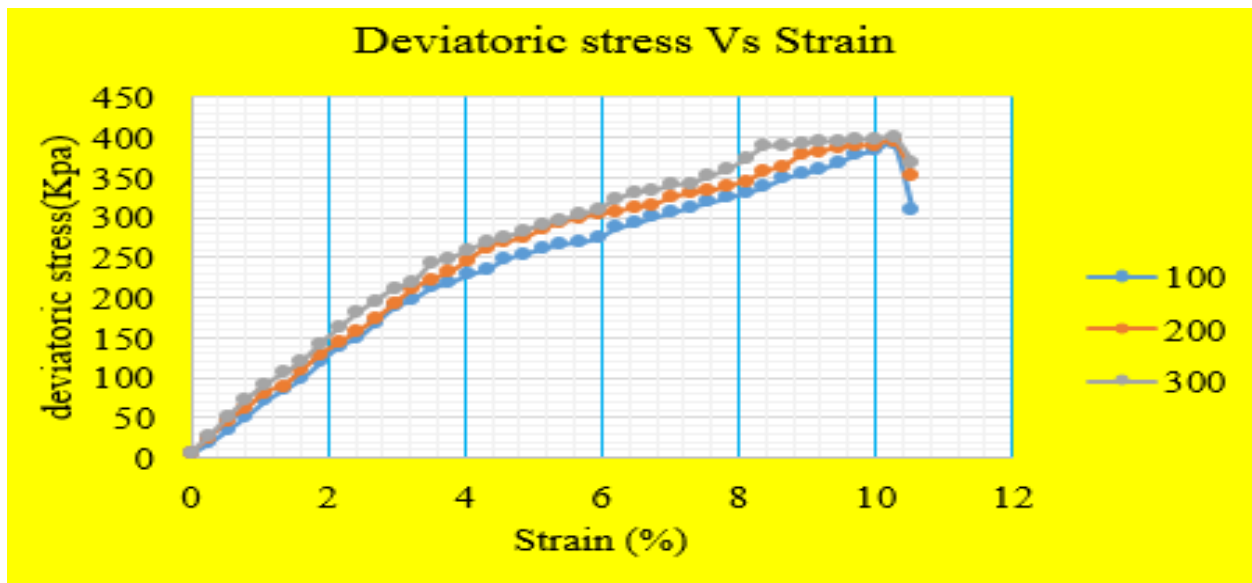


Figure 4.8:- typical stress Vs strain curve of TP-9

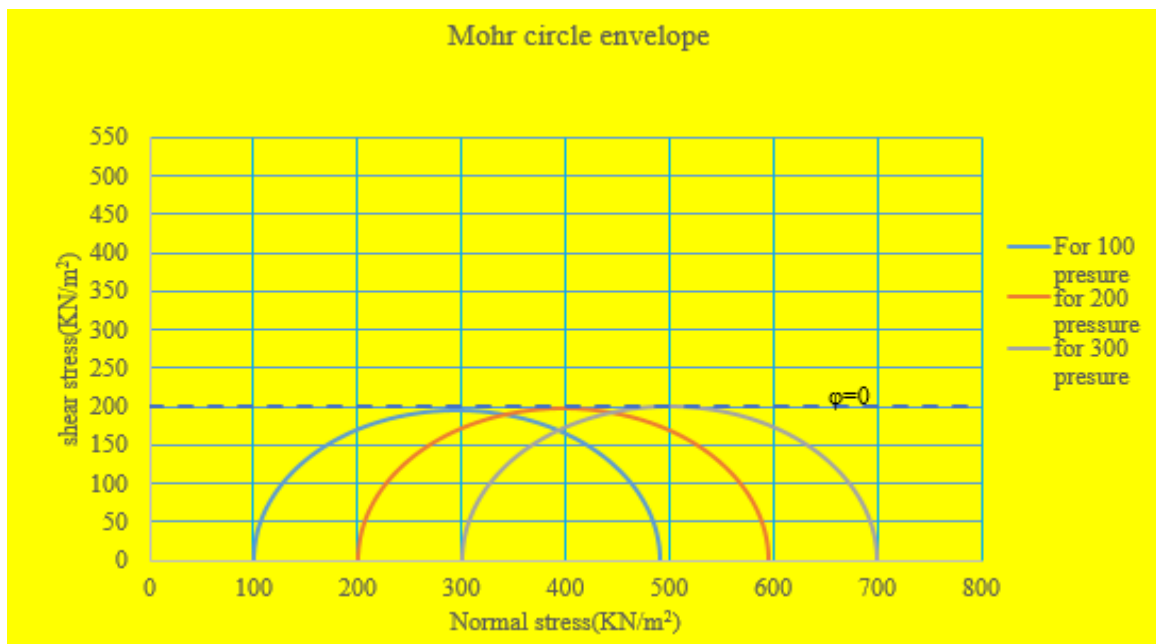


Figure 4.9:- Mohr circle envelope For TP-9

To explain the triaxial test result shown in table 4.13 the chamber pressure at 100,200,300 KN/m² for TP-4, TP-6 and TP-9 the stress at failure that is the deviatoric stress was 85.12,86.4,87KN/m²,248.48, 249.5,256.12KN/m² and 391.3,394.76,399.48KN/m² respectively. To get the principal stress that is $\sigma_1 = \sigma_3 + \Delta\sigma_f$ for TP-4, TP-6, TP-9 then it become 185.12,286.4,387KN/m²,348.48,449.5,556.12KN/m² and 491.3,594.76,699.48KN/m² respectively. The undrain shear strength in TP-4, TP-6, TP-9 are $S_u = \frac{1}{2}(\sigma_1 - \sigma_3)$, that is 42.56, 43.2, 43.5kN/m², 124.24, 124.75, 128.06kN/m, 195.65, 197.38, 199.74kN/m² respectively

4.10 Consolidation test

A one-dimensional consolidation test is done ASTM D 2435 which is widely used to obtain the settlement and time parameters. The test confines the soil laterally in a metal ring so that settlement and drainage can occur only in the vertical direction [5].



Figure 4.10 consolidation test apparatus

For the test of consolidation in the disturbed type of sample it began by compacting the soil sample on compaction mold about five layer and determine the optimum moisture content and maximum dry density then by using optimum moisture content the soil in the mold can be saturated in order to make the soil as if in the field. Then, the soil sample extruded in order to use for consolidation test. First determine the empty ring weight, diameter, height then, place the ring in the extruded soil and trim the soil spaceman in order to make smooth then place the porous stone and filter paper at the bottom and top of the soil spaceman. After that, put in to the consolidation cell and insert collar in order to make center. Next to this, place the bolt to make it tighter the collar and place the cap and ball on the top of porous stone after this, place in to the loading frame and level the frame. Finally, place the loading desk at the bottom as vertical load for different loading desk starting from 2.5kg, 5kg, 10kg, 20kg, 40kg each vertical loading when it is place one after the other for 24hours sequence it become doubled.

4.10.1 Pre-consolidation pressure

The pre-consolidation pressure σ'_c is the maximum past effective overburden pressure to which the soil specimen has been subjected. Casagrande (1936) proposed to determine the pre-consolidation pressure using simple graphical procedure [15]. That is

- Draw a horizontal line through the same point on the curve.
- Draw a line tangent to the interpreted curve that passes through the selected point
- Finally extend a line that represent the steepest portion of the interpreted normally consolidated compression curve.
- The intersection of the extension of the normally consolidated line and bisecting line gives the pre-consolidation stress.

4.10.2 over consolidation

"A soil is said to be normally consolidated if the current overburden pressure (column of soil overlying the plane of consideration) is the largest to which the mass has ever been subjected. It has been found by experience that prior stresses on a soil element produce an imprint or stress history that is retained by the soil structure until a new stress state exceeds the maximum previous one. The soil is said to be over consolidated (or pre consolidated) if the stress history involves a stress state larger than the present overburden pressure. Over consolidated cohesive soils have received considerable attention. Only more recently has it been recognized that over consolidation may be of some importance in cohesion less soils" [5].

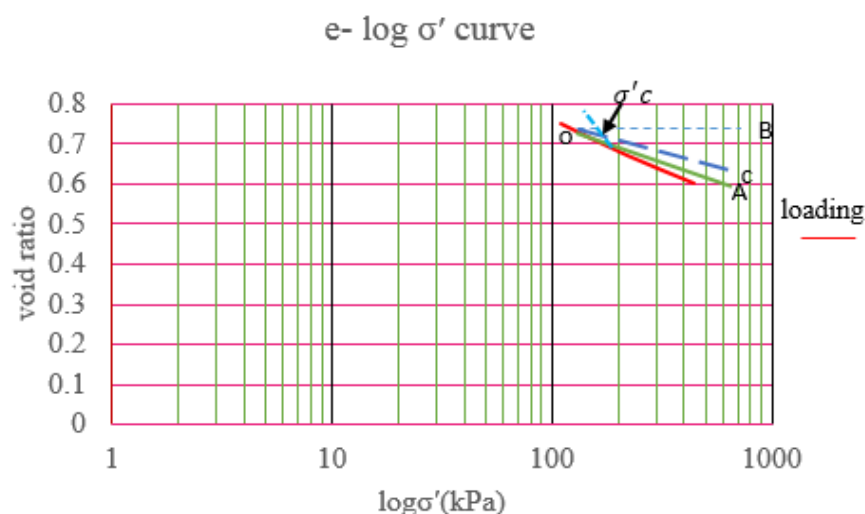


Figure 4.11:- Typical void ratio Vs pressure curve for TP-4

$$OCR = \frac{\sigma'_c}{\sigma_o} \dots\dots\dots (4.7)$$

σ'_c : - pre consolidation pressure, σ_o : - Overburden Pressure, OCR = over consolidation.

Table 4.13:-Pre consolidation pressure of the soil test result

Test pit	Type	Depth (m)	γ ($\frac{KN}{m^2}$)	Overburden Pressure (σ_o)	Compression Index (cc)	$\sigma' c$ (kPa)	OCR
TP-4	Disturbed	3	27.8	83.4	0.261	170	2
TP-6	undisturbed	3	27.8	83.4	0.291	190	2
TP-9	undisturbed	3	27.1	81.3	0.230	165	2.2

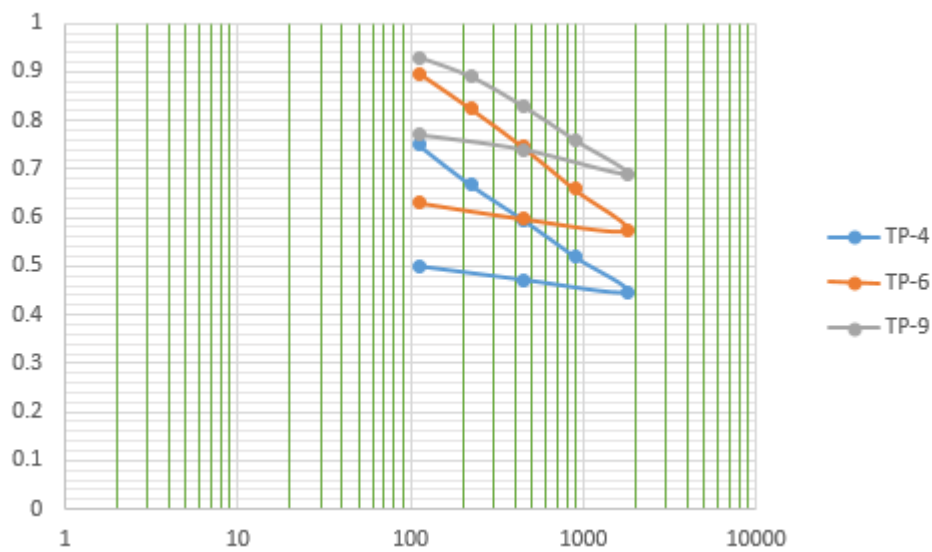


Figure 4.12:- Pressure Vs void ratio log scale

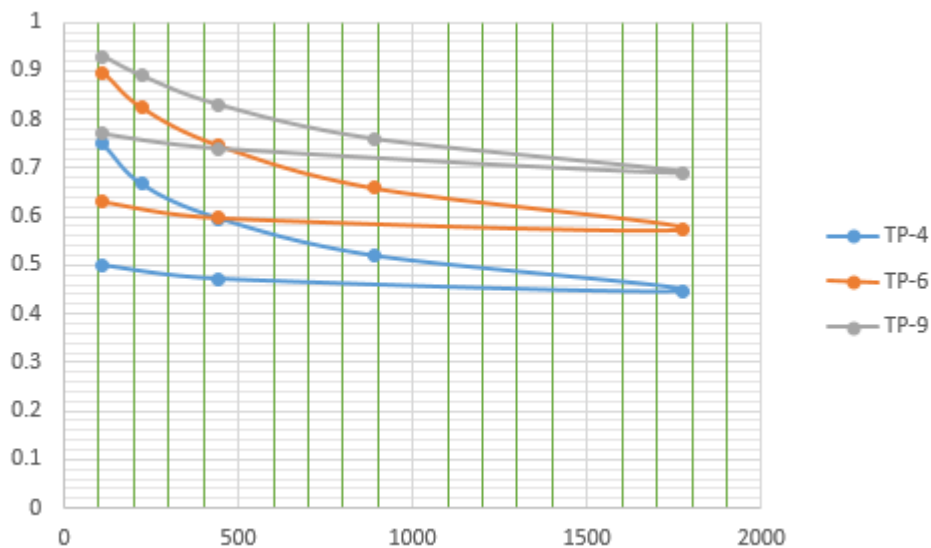


Figure 4.13:- Pressure Vs void ratio linear scale

4.10.3. Compression index

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

$$C_c = \frac{-\Delta e}{\log_{10}\left(\frac{\sigma}{\sigma_0}\right)} \dots\dots\dots (4.8)$$

C_c is important for the determination of settlement of foundation in the field.

4.10.4. Recompression index

Recompression is the compression of soil which had already been loaded and unloaded. The load during recompression is less than the load to which the soil has been subjected previously. The slope of the recompression curve obtained during reloading when plotted void ratio versus log pressure curve.

$$C_r = \frac{-\Delta e}{\log_{10}\left(\frac{\sigma + \Delta\sigma}{\sigma}\right)} \dots\dots\dots (4.9)$$

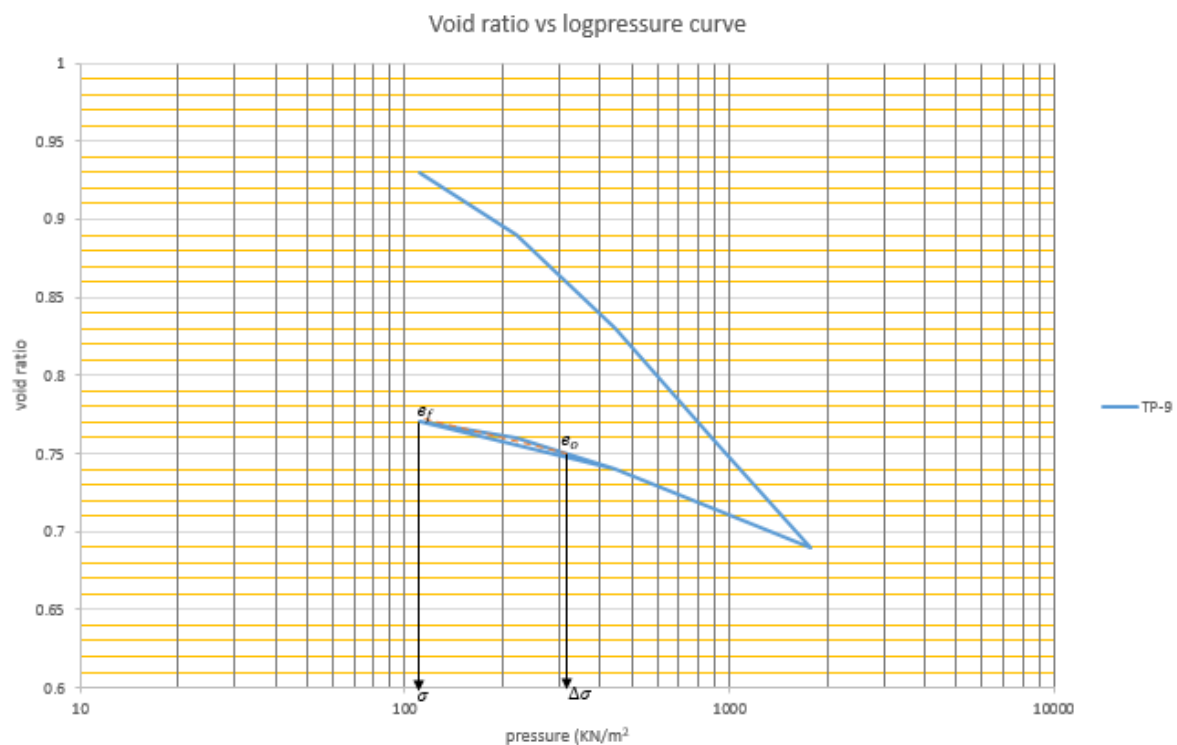


Figure 4.14:- Typical recompression curve for TP-9

4.10.5 Coefficient of consolidation (CV)

The Coefficient of Consolidation is a parameter used to describe the rate at which saturated clay or other soil undergoes consolidation when subjected to an increase in pressure. The coefficient of consolidation C_v can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical. There are two laboratory methods that are in common use for the determination of C_v . They are: Casagrande logarithm of time fitting method and Taylor square root of time fitting method. In Casagrande logarithm of time fitting method, the interval between 0 and 100% consolidation is divided into equal intervals of percent consolidation. Since it has been found that the laboratory and theoretical curves have better correspondence at the central portion, the value of C_v is computed by taking the time t and time factor T at 50% consolidation [17].

$$C_v = 0.197 \frac{H_{dr}^2}{t_{50}} \dots\dots\dots (4.9)$$

Where H_{dr}^2 = drainage path

Square root of time fitting method was devised by Taylor. The fitting method consists of first drawing the straight line which best fits the early portion of the laboratory curve. Next a straight line is drawn which at all points has abscissa 1.15 times as great as those of the first line. The intersection of this line and the laboratory curve is taken as 90% consolidation point. Its value may be read and is designated as t_{90} [5].

$$C_v = 0.848 \frac{H_{dr}^2}{t_{90}} \dots\dots\dots (4.10)$$

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.

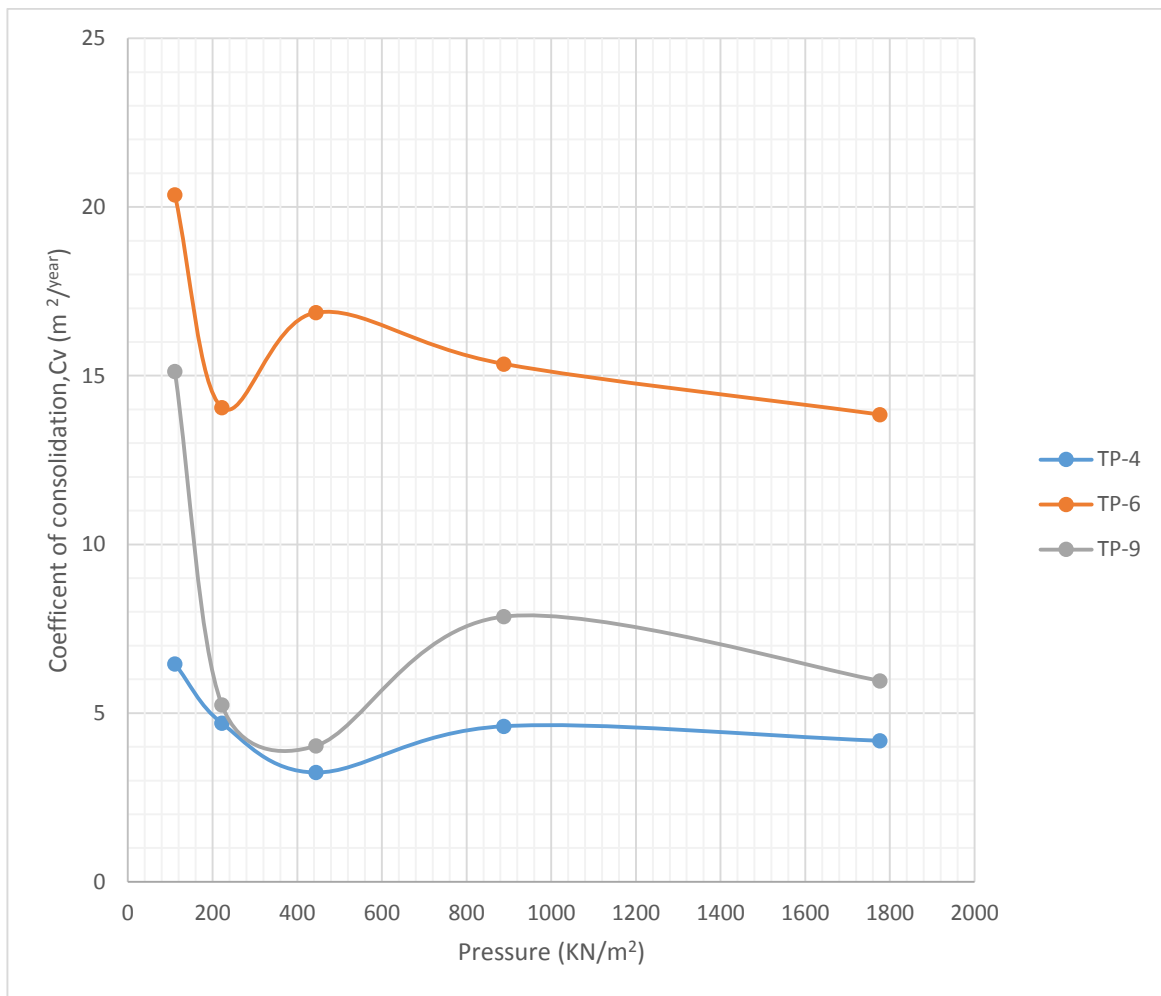


Figure 4.15:- coefficient of consolidation verses pressure

Table 4.14:- One dimensional consolidation test result

Test pit	Depth (m)	NMC	Pressure (KN/m ²)	Void ratio	Cv (10 ⁻³ cm ² /sec)	Loading Cc	Re loading Cr	Unloading Cs
TP-4	3	82.05	111	0.75	2.03	0.261		0.249
			222	0.67	1.49			
			444	0.60	1.02			
			888	0.51	1.43			
			1776	0.45	1.30			
TP-6	3	34	111	0.89	6.40	0.291		0.0498
			222	0.82	4.41			
			444	0.74	5.35			
			888	0.66	4.82			
			1776	0.57	4.34			
Tp-9	3	25	111	0.93	4.75	0.221	0.021	0.038
			222	0.89	1.65			
			444	0.83	1.30			
			888	0.76	2.47			
			1776	0.69	1.87			
			222	0.728	9.390			
			444	0.709	2.908			

4.11 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 [24].

$$K = \frac{C_v \cdot a_v \cdot \gamma_w}{1+e} \dots\dots\dots (4.11)$$

Where: C_v - coefficient of consolidation, a_v- coefficient of compressibility, γ_w- unit weight of water and e - void ratio

Table 4.15:- Relationship between void ratio and coefficient of Permeability

Test pit	Depth (3m)	Pressure (KN/m ²)	Void ratio	Cv (10 ⁻³ cm ² /sec)	coefficient of compressibility(a _v) (10 ⁻⁵ m ² /KN)	K 10 ⁻⁹ cm/sec
TP-4	3	111	0.75	2.03	0.6	68.8
		222	0.67	1.49	0.4	35.1
		444	0.60	1.02	0.267	16.85
		888	0.51	1.43	4.44	4.192
		1776	0.45	1.30	0.114	10.25
TP-6	3	111	0.89	6.40	0.74	24.7
		222	0.82	4.41	0.250	71.92
		444	0.74	5.35	0.4	12.02
		888	0.66	4.82	0.15	43.2
		1776	0.57	4.34	0.142	45.95
TP-9	3	111	0.93	4.75	5.89	1.47
		222	0.89	1.65	0.6	68.8
		444	0.83	1.30	0.5	15.1
		888	0.76	2.47	0.26	37.12
		1776	0.69	1.87	6.88	7.77

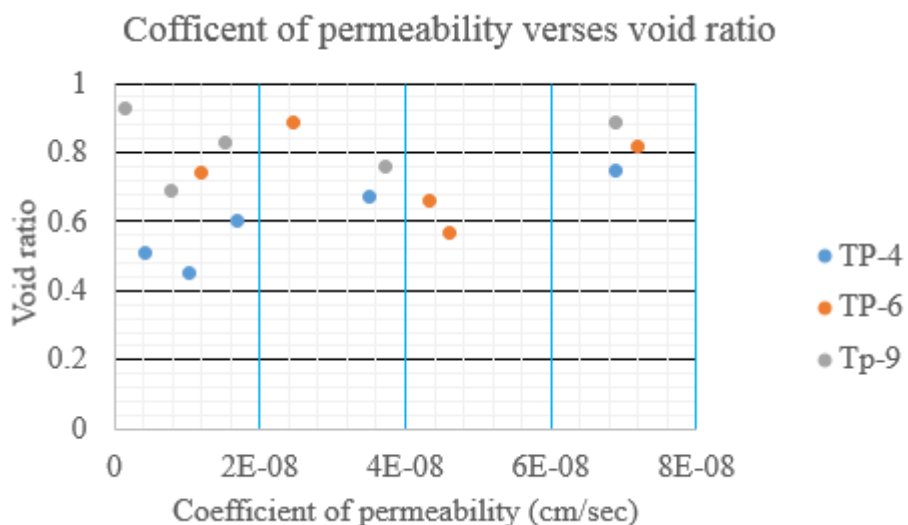


Figure 4.16 void ratio verses log coefficient of permeability

CHAPTER FIVE

Discussion and correlation analysis

5.1 Discussion of the laboratory test result

The specific gravity of the soil in the study area is different in different test pit even across the depth. The range of specific gravity of soil in the study area is 2.65-2.78. As a result of this, the indication of the specific gravity laboratory test result was inorganic soil.

According to the grain size test result as mentioned in the Table 4.6 and figure 4.1 and also figure 4.2 the silt and clay particle are the major one the range of the clay 24.92-75.52%, silt 22.78-67.93%, sand 0.73-26.96% and gravel content 0-11.98%.

The free swell test of the soil shows that the range of free swell is less than 50% except for TP-4. So, the soil doesn't have expansive nature. But, TP-4 the free swell was 70% which is in between 50-100% it shows moderate expansive nature.

The range of the liquid limit 34.32-78.12%, plastic limit 16.60-46.45% and the plastic index 3.8-31.63% more over the liquidity index of the soil as shown in table 4.4 most of the soil is in plastic state as well as the consistency index of the study area which was expressed in table 4.5 tell as more than 50% it's consistency indicate that stiff in nature.

According to unified soil classification system from the test result shown in table 4.7 more than 50% of the soil is CL and CH that is inorganic clay of low medium plastic to high plastic and from the figure 4.3 shows that most of the soil is above A-line in the region of inorganic clay. And from ASTM D2487-11 using unified soil classification system flow chart for fine grained soil indicated that the group name of the soil more than 85% was sandy lean clay.

To say about the classification of the soil in the study area using AASHTO classification system from the table 4.9 and figure 4.4 most of the soil in the group classification was

A-7-5 and A-7-6 in addition to that the significant constituent material was clay soil for the general rating for the sub grade material was fair to poor.

Except for TP-6, TP-10 at 3m depth and TP-2, TP-1 at 1.5m depth which was A-6 and also TP-1 at 3m depth shows A-4 and the minimum group index of the soil has at TP-1 3m depth with the group index is equal to three. Therefore, it can be a good source of sub grade material.

Activity classification in the study area in table 4.11 and figure 4.5 the value of the soil based on activity number assign the range of in active. The compaction test result express in table 4.12 that the maximum dry density and optimum moisture content 1.09 to 1.6 gm/cc and 19.5 to 46% respectively.

The triaxial test result shown in table 4.13 the chamber pressure at 100, 200, 300 KN/m² for TP-4, TP-6 and TP-9 the stress at failure that is the deviator stress was 85.12, 86.4, 87 KN/m², 248.48, 249.5, 256.12 KN/m² and 391.3, 394.76, 399.48 KN/m² respectively. To get the principal stress that is $\sigma_1 = \sigma_3 + \Delta\sigma_f$ for TP-4, TP-6, TP-9 then it become 185.12, 286.4, 387 KN/m², 348.48, 449.5, 556.12 KN/m² and 491.3, 594.76, 699.48 KN/m² respectively. The undrain shear strength in TP-4, TP-6, TP-9 are $S_u = \frac{1}{2}(\sigma_1 - \sigma_3)$, that is 42.56, 43.2, 43.5 kN/m², 124.24, 124.75, 128.06 kN/m, 195.65, 197.38, 199.74 kN/m² respectively.

The consolidation test was done for one disturbed and two undisturbed soil sample from figure 4.10 and 4.11 for vertical pressure verses void ratio on semi log and linear scale except for the different in initial void ratio the figure shows similar curvature for all the samples then the Pre-consolidation pressure for TP-4, TP-6, TP-9, it become 1000, 1200, 1025 kPa respectively and also swelling pressure for TP-4, TP-6, TP-9 200, 300, 350 kPa respectively. More over the over consolidation ratio of the soil is greater than one this indicate that the soil in the study area is over consolidation in its natural state. From figure 4.13 observed that coefficient of consolidation C_v decreases with increasing effective pressure. The compression index C_c of a soil is manipulated from the slope of the straight portion the void ratio verses log pressure ranges from 0.23-0.291

5.2 comparing different researcher test result with recent researcher

Thus researcher listed in table 5.1 that is [7], [22], [21], and [27] and [28]. They took ten test pit with one point five to three meter depth. Except the researcher done by [23] There are five test pit with two meter hand auger tool used In order to evaluate the engineering property of the soil in their study area.

Table 5.1:- comparison of the researcher test result with current research

Item	Haromaya [28]	Adama [7]	Debre birhan [21]	Burayu [27]	Adet [22]	AL- Najaf city Iraq [23]	Arekit recent
Gravel (%)	0-6.9	0-24.85	0-24.63	0-0.8	0-64.05		0-11.98
Sand (%)	6.88-39.6	14.5-54.6	2.37- 38.79	2.8-16.7	0.98-33.48		0.73- 26.98
Silt (%)	18.3-36.7	17.6-60.7	27.28- 55.9	18.7-34.1	8.87-58.41		22.73- 67.95
Clay (%)	36.1- 65.75	5.4-40.5	8.77-67.5	55.7-73.8	9.52-90		24.92- 75.52
Liquid limit (%)	86.4-87.3	29-73	31-80	66.1-72.1	50.14-85.3	21-29	29.33- 78.12
Plastic limit (%)	23-23.8	21-35	18-43	30.6-34.1	31.01- 35.65	11-15	16.6- 46.45
Plastic index (%)	63.3-63.4	13-34	13-46	35.5-39.8	29-50	-	3.8-31.63
Specific gravity	2.7-2.75	2.4-2.7	2.62-2.81	2.7-2.82	2.62- 2.85	2.61- 2.64	2.65-2.78
Free swell (%)	20-113	18-50	50-100	42.5-55	67-100		15-70
OMC (%)	13-23.4	-	-	30.4-35.0	-	8.6-13.2	19.5-46
MDD (gm/cc)	1.39-1.52	-	-	1.31-1.38	-	17.6-19	1.09-1.6
UN confined compression (kN/m ²)	132.2- 273.4	-	75-233	306.68- 465.18	101.35- 442.7	-	-
Triaxial UU (kN/m ²)	-	-	-	-	-	-	80-445
Coefficient of compression	0.13-0.33	0.33-0.40	0.2-0.38	0.099-0.133	0.19-0.299	-	0.23- 0.291

From the table 5.1 shown the soil of Arekit town compared with the previously researcher done like Burayu and Adama the free swell test result indicate that they don't have expansive nature. However, with other index and engineering property of the soil test result is different.

5.3 Soil Map of the study area

The soil map of Arkit town was prepared based on information collected from GPS data and filed visual observation of different area and laboratory test result. Visual observation which is made during reconnaissance survey test pit location shows that most part of the town are covered with red clay soil in some area brown to dark clay soil. The visual description of the ten test pit are from the vertical profile of the test pit shows that most of the soil found in Arekit town 0-0.4m brown to dark black in colure. Exceptionally, seen that the soil found in T'emren Hayk and stadium Keble 0-1m depth the soil was black in colure. After that in most of the soil found in ten test pit from 0.5-3m the colure of the soil become reddish clay soil.

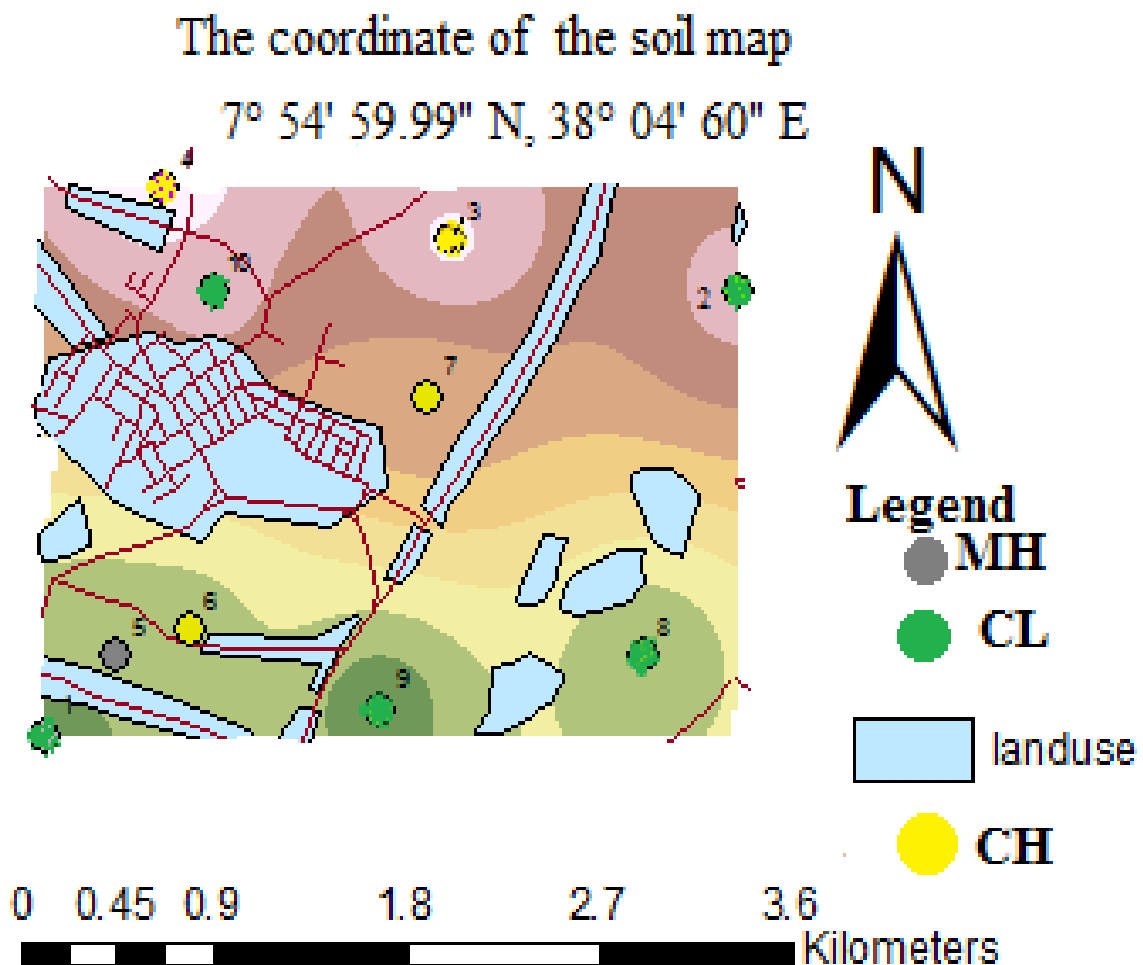


Figure 5.1:- The proposed soil map of the study area

5.4 Further analysis by using correlation and regression

By using the mathematical expression relation can be formed between two or more variable to determine equation on the connected two variable. In this research paper the primary data is collected from Arekit town from ten test pit 20 sample for both Atterberg and compaction test collected. Then, for the collected data laboratory experiment is utilized. After that possible relationship among the parameter have been developed. For analysis purpose the scatter plot, correlation and linear regression analysis is used. To form relation between the dependent variable (OMC and MDD) and the independent variable (LL, PL, PI) [18].

5.5 Scatter plot and Best-Fit curve

Generally in analysis system for this research the estimation of OMC and γ_{dmax} were taken as the dependent variable whereas the Atterberg limit (LL, PL, Ip) are the independent variable. In completing the statistical analysis MS-excel spread sheet are utilized to decide the scatter plot, correlation and regression. MS – excel spread sheet is seen as the most remarkable and reasonable device for scatter plot analysis and determination of correlation between two variable[18].The relationship between the dependent and independent variable analyzed independently across different depth for essential information as introduced in figure 5.2 and 5.3. "Scatter diagram generated by applying the excel spread sheet in order to study the relationship developed between the dependent variable and the regression variable so as to determine the model that best suit the test result" [8].

"For the scatter plot evaluation the following point are essential

- If all point in the scatter plot are very close to each other a fairly good correlation can be expected between the dependent and independent variable, like wise if those point are widely scattered, a poor correlation data can be expected.
- If the point are scattered and they reveal no upward or downward trend then we say the variable are UN correlated.
- However, if there is an increasing trend from the lower left hand corner and going upward to the right hand corner the correlation indicate from the graph is said to be positive and where a down ward trend from upper-left hand corner is said to be negative. " [9].

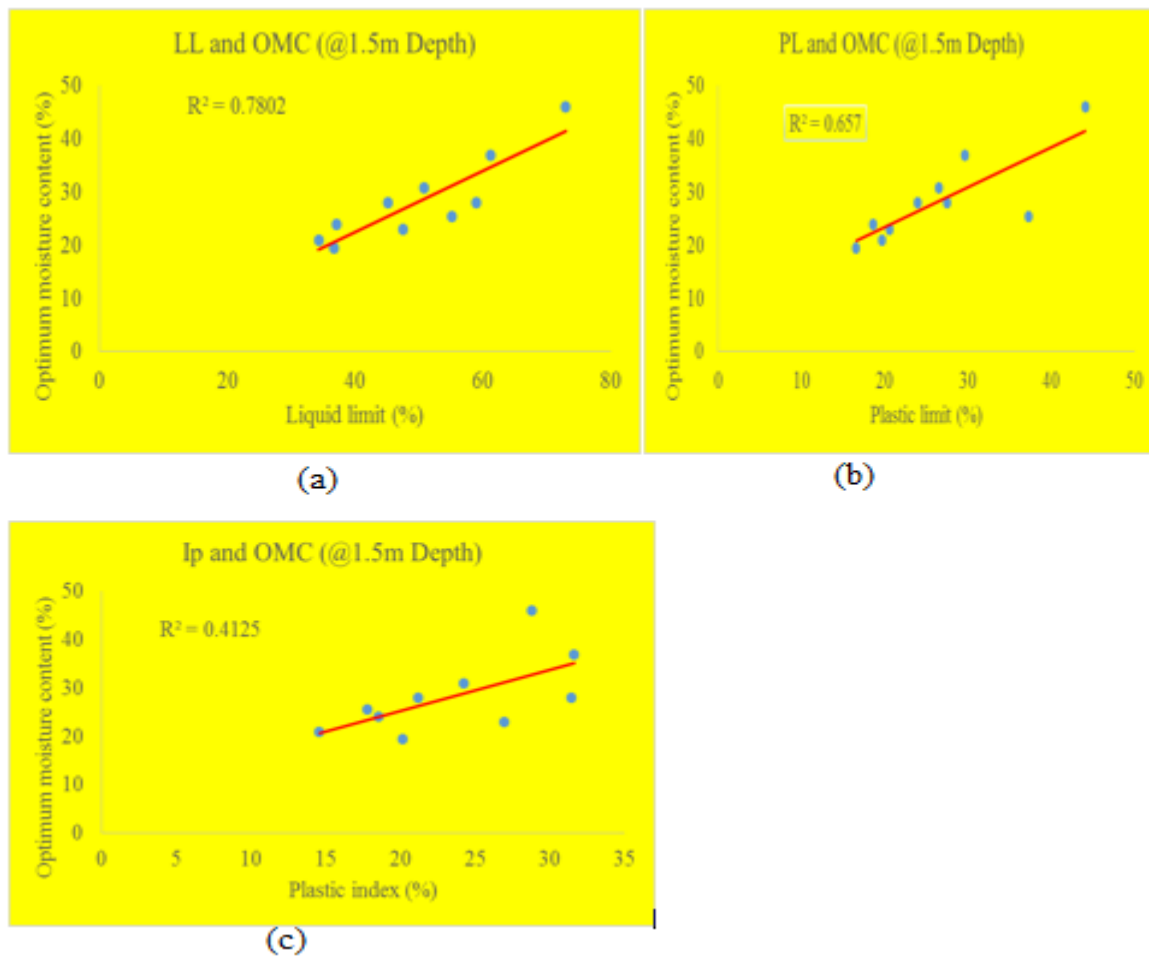


Figure 5.2 scatter plot relation between (a) for LL with OMC, (b) for PL with OMC, and (c) Ip with OMC @1.5 m depth.

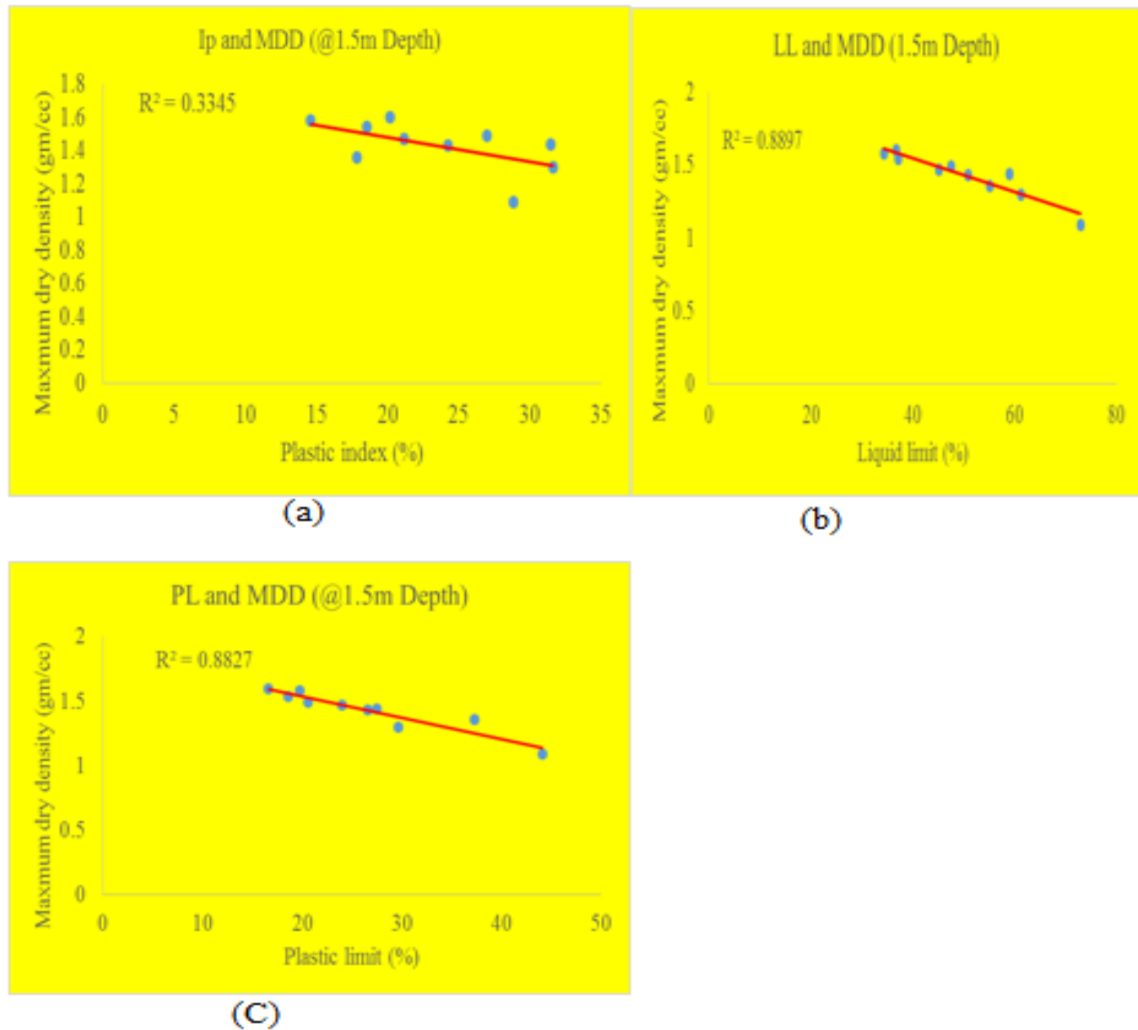


Figure 5.3 scatter plot relation between (a) Ip with MDD, (b) LL with MDD, and (c) Ip with MDD @1.5 m depth.

For the Depth of 3m the scatter plot is shown in appendix G. From the above scatter diagram shows that the points are scattered randomly around straight line which express a linear response. Hence, a linear regression model can explain the association between the parameter.

5.6 Correlation and regression

Correlation is a combination of two term Co-“ together” and relation “connection” it is a statistical technique’s which show the representation of strength of connection between two variable. Correlation can be positive or negative when the two variable moving the same direction that is an increase in one variable and resulting corresponding increase in another variable and vice versa then the variable consider to be positively correlated.

On the contrary, when the two variable moving in different direction in such way that an increase in one variable will result decrease in another variable and vice versa this situation known as negative correlation. For correlation analysis the degree of association is measured by correlation coefficient denoted by "r" and sometimes called as Pearson correlation coefficient and is measure of linear association. Complete correlation expressed either +1 or -1 and complete absence of correlation is expressed by 0.

Table 5.2 Correlation of Liquid limit for @1,5m depth

	LL (%)	PL (%)	Ip (%)	OMC (%)	MDD (gm/cc)
LL (%)	1				
PL (%)	0.89472	1			
Ip (%)	0.792604	0.47524	1		
OMC (%)	0.883262	0.81055	0.728187	1	
MDD (gm/cc)	-0.94324	-0.93953	-0.63225	-0.945544921	1

From table 5.2 the liquid limit have better correlation with OMC than MDD and also relatively seen from others plastic limit have good relation with O MC than other because it approximately approach 1 that is 0.89472. For at depth of 3m the correlation and linear regression equation become in the appendix G available. The significance of F and P value which is essential to check if the data is reliable that significance F if it is less than 0.05 value its ok. But, if significance F is greater than 0.05 value it's probably better to stop using the set of independent variable. "Before the analytical method introduced some important term are express below

T-test value: - the probability of making a mistake to reject a hypothesis when it happen to be true at the level of significance. In practice, it is usual to use the 5% level of significance this means that we are 95% confidence that we can make the right decision and we cannot go wrong with a probability of 5% and you can get the t- value by dividing the standard error of the coefficient through its independent variable.

P- value :- it is the most important term in the opinion of the statistical significance of the independent variable it also represent a significant predictive power of the model of P- value simply the ratio of the model mean square error to the mean square.

Standard error: - the average error of each measurement sample point of the line of the Best fit out of all the curve, the best fit curve through the standard error smaller and it is important because it is used to calculate other measure such as confidence interval and margin of error." [9].

5.7 Linear regression and multiple linear regression

Linear regression attempt to model the relationship between two variable by fitting a linear equation to observe data one variable consider to be an explanatory variable and the other is consider to be a dependent variable. Multiple linear regression it is also a type of linear regression where the mean and variance of the dependent variable will be a function of the values of several variables." the representation of single and multiple linear regression models expressed below

$$Y = \beta_0 + \beta_1x + \varepsilon \dots\dots\dots (5.1)$$

$$Y = \alpha_0 + \alpha_1x_1 + \alpha_2x_2 + \dots + \alpha_nx_n + \varepsilon \dots\dots\dots (5.2)$$

Where the slope (β_1) and intercept (β_0) of the single linear regression model are called regression coefficient and in the same while $\alpha_0, \alpha_1, \alpha_2, \dots, \alpha_n$ are multiple linear regression coefficient" [8]. The early established [18],[24],[2] and [25] correlation equation for soil that relate OMC and MDD with Atterberg limit thus values are summarize in table 5.3.

Table 5.3:- Previously established correlation equation

Researchers name	Equation	R ²	R
Atsbeha Near,(2012)	OMC=0.751PL+ 5.73	0.960	0.980
	MDD=-0.19PL +19.67	0.865	0.930
TesfamichaelTsegaye, (2017)	OMC=0.916*PL-0.030*PI-0.875	0.807	0.898
	MDD=-0.18*PL-0.027*PI+21.182	0.835	0.914
WaqasHassen&Hafizqasim,(2017)	OMC=5.53 +0.662PL	0.905	0.95
	MDD=0.0098PL+1.8541	0.868	0.932
Twinkle Tom ,(2016)	OMC=0.186PL + 16.55	0.893	0.944

5.8 The regression analysis for the data

The developed linear regression equation and output values are tabulated in the table below. For depth of 1.5 the developed linear and multiple linear regression as shown below

$$\text{MDD} = 2.00 - 0.0115 \text{LL} \dots\dots\dots (5.3)$$

$$\text{OMC} = -0.412 + 0.57 \text{LL} \dots\dots\dots (5.4)$$

$$\text{MDD} = 1.96 - (0.0017 * \text{LL}) - (0.0089 * \text{OMC}) - (0.0075 * \text{PL}) \dots\dots\dots (5.5)$$

If the moisture content is exceeding abundantly the space that occupied by solid particle is occupied by water and it precipitate the lower value of the dry unit weight. As a result, of this OMC and MDD have good relation with liquid limit. "In statistics, regression validation is the process of deciding whether the numerical result quantifying hypothesized relationship between variable, obtained from regression analysis model it may be important to confirm the goodness of the fit of the model and the statistical significance of the estimated parameters. A well-fitting regression model results in predicting values close to the observed value to check goodness of fit include R^2 value, F-test, t-test and residual plot" [25].

Summary output

Regression Statistics

Multiple R	0.943235346
R Square	0.889692917
Adjusted R Square	0.875904532
Standard Error	0.053321338
Observations	10

Table 5.4:-summery output for linear regression @1.5m

Predictor	Coefficient	Standard error	t stat	P-value	Significance F
intercept	2.0	0.0738	27.189	3.61E-09	4.24029E-05
LL (%)	-0.01154	0.001437	-8.033	4.24E-09	

Summary output

Regression Statistics

Multiple R	0.991935477
R Square	0.983935991
Adjusted R Square	0.975903987
Standard Error	0.023496094
Observations	10

Table 5.5:- summary output for multiple linear regression @1.5m

Predictor	coefficient	Standard error	t stat	P value	Significance F
Intercept	1.9656	0.03411	57.63	1.83E-09	9.01376E-06
LL (%)	-0.0017	0.001778	-0.968	0.37	
OMC (%)	-0.0089	0.00209	-4.253	0.0054	
PL (%)	-0.0074	0.00202	-3.710	0.010	

Generally for the result of correlation and regression analysis optimum moisture content and maximum dry density have strong correlation with liquid limit in this research for the data collected from ten test pit. But, at a depth of three meter it's not satisfactory to say they have strong correlation because the "R" value is much less than one. In addition to this, the scatter plot shows slightly the liquid limit was close relation with OMC and MDD than plastic limit. The independent variable (LL, PL, Ip) from the scatter plot can be easily understand that they have positive correlation with optimum moisture content and negative correlation with maximum dry density,

CHAPTER SIX

Conclusion and Recommendation

5.1 Conclusion

- In unified soil classification system most of the soil is CL and CH. For AASHTO soil classification the soil in the group under A-7-5 and A-7-6. Therefore, the soil found in Arekit town is clay soil.
- The Atterberg limit test result shows the soil found in town in plastic state except the soil found in TP-4 at 3m depth, TP-6 at 1.5m depth in liquid state and also TP-8, TP-10 at 3m depth solid state.
- The triaxial test result indicate that the stress at failure in between 85kN/m^2 - 400kN/m^2 . As a result, the consistency of the soil in the study area varies in between stiff to hard.
- The one dimensional consolidation test express that the soil was over consolidated in its nature with $\text{OCR} > 1$.
- Optimum moisture content and maximum dry density have strong correlation with liquid limit and from scatter plot diagram (LL, PL, and Ip) have positive correlation with OMC and negative correlation with MDD.

5.2 Recommendation

- The mineralogical composition of the soil test was essential in order to find out the effect on the engineering property of the soil. It is better to do such kind of test
- By increasing the number of sample collection in the town correlation will be studied with different parameter of soil in this study area.

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APPENDICES

Appendix A :- Grain size analysis test result

Sample ID:-TP-3

Depth (m):- 3

A. sieve analysis

Test type	Hydrometer+ wet sieve
Sample type	Disturbed

Table A- 1:- Sieve analysis test result for TP-3 (3m)

Sieve size	Retained for 1 st sieving		Percent passing	Wet sieving Hydrometer NO =152 H Specific gravity= 2.71 Dry mass of a soil, W= 50 Alpha (a) = 0.995 Zero correction factor CZ 6.00 Zero correction factor CZ 5.82 $D = KSQRT(L/t)$ $P=(Ra/W)*100$
(mm)	weight	%Rtd.	%	
3" 75	0.00	0.00	100.00	
2 1/2" 63	0.00	0.00	100.00	
2" 50	0.00	0.00	100.00	
1 1/2" 37.5	0.00	0.00	100.00	
1" 25	0.00	0.00	100.00	
3/4" 19	0.00	0.00	100.00	
1/2" 12.5	0.00	0.00	100.00	
3/8" 9.5	0.00	0.00	100.00	
N ^o 4 4.75	0.00	0.00	100.00	
N ^o 10 2	3.51	1.72	98.28	
N ^o 16 1.18	4.16	2.04	96.23	
N ^o 30 0.6	6.20	3.05	93.19	
N ^o 50 0.3	7.80	3.83	89.36	
N ^o 100 0.15	8.16	4.01	85.35	
N ^o 200 0.075	5.13	2.52	82.83	
Pan	168.64	82.83		
Total	204	100		

Hydrometer

Specific gravity	2.71
------------------	------

Table A -2:-Hydrometer Analysis test result for TP-3 (3m)

Elapsed Time, t,min	Actual Hydrometer reading	Temp (°C)	Effective depth L (mm)	K From Table	Temperature correction Factor Ct	Modified hydrometer Reading $R=R_a-C_z+C_t$	Particle Diameter D (mm)	%pass	Corrected % pass
0									
2	46	20	8.54	0.01355	-0.04	39.959	0.0280	79.56	65.90
5	45	20	8.71	0.01355	-0.04	38.959	0.0179	77.57	64.25
15	42	20	9.20	0.01355	-0.04	35.959	0.0106	71.59	59.30
30	40	20	9.53	0.01355	-0.04	33.959	0.0076	67.61	56.00
60	37	20	10.03	0.01355	-0.04	30.959	0.0055	61.64	51.05
250	32	21	10.85	0.01355	0.18	26.356	0.0028	52.47	43.46
1400	28	20	11.51	0.01355	-0.04	21.959	0.0012	43.47	36.21

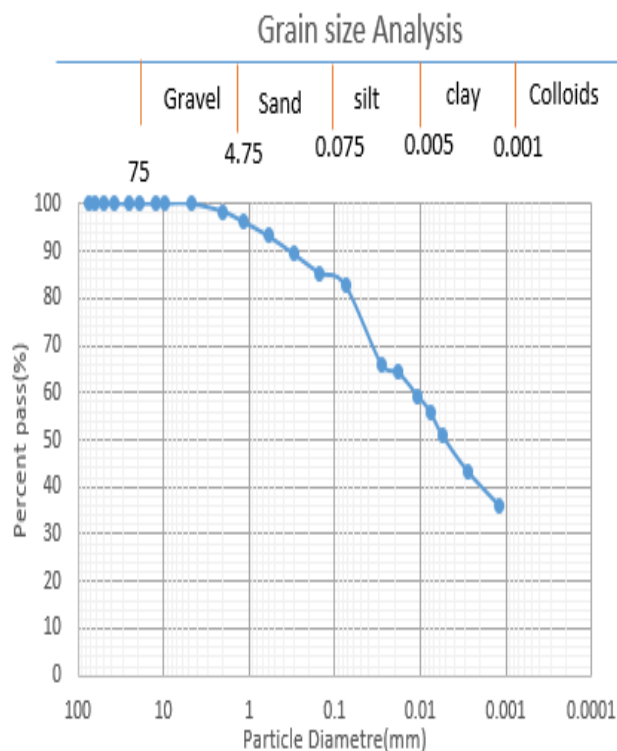


Figure A:-Typical grain size analysis test result for TP-3

2. Sample ID:-TP-4

Depth (m):- 3

Test type	Hydrometer+ wet sieve
Sample type	Disturbed

A. sieve analysis

Table A -3:- Sieve analysis test result for TP-4

Sieve size	Retained for 1 st sieving		Percent passing	Wet sieving Hydrometer NO =152 H Specific gravity= 2.78 Dry mass of a soil, W= 50 Alpha (a) = 1.012 Zero correction factor CZ 6.00 Zero correction factor CZ 5.6 $D = K\sqrt{L/t}$ $P=(Ra/W)*100$
(mm)	weight	%Rtd.	%	
3" 75	0.00	0.00	100.00	
2 1/2" 63	0.00	0.00	100.00	
2" 50	0.00	0.00	100.00	
1 1/2" 37.5	0.00	0.00	100.00	
1" 25	0.00	0.00	100.00	
3/4" 19	0.00	0.00	100.00	
1/2" 12.5	0.00	0.00	100.00	
3/8" 9.5	0.00	0.00	100.00	
N ^o 4 4.75	0.00	0.00	100.00	
N ^o 10 2	0.07	0.02	99.98	
N ^o 16 1.18	0.14	0.04	99.93	
N ^o 30 0.6	1.96	0.62	99.32	
N ^o 50 0.3	5.51	1.73	97.58	
N ^o 100 0.15	4.59	1.45	96.14	
N ^o 200 0.075	3.28	1.03	95.10	
Pan	302.09	95.10		
Total	318	100		

B. Hydrometer

Specific gravity	2.78
------------------	------

Table A -4:- Hydrometer analysis test result for TP-4 @3m

Elapsed Time, t,min	Actual Hydrometer reading	Temp (°C)	Effective depth L (mm)	K From Table	Temperature correction Factor Ct	Modified hydrometer Reading $R=R_a-C_z+C_t$	Particle Diameter D (mm)	%pass	Corrected % pass
0									
2	53	20	7.39	0.01384	-0.04	46.959	0.0266	95.03	90.37
5	51	20	7.72	0.01384	-0.04	46.959	0.0172	90.98	86.53
15	50	20	7.88	0.01384	-0.04	43.959	0.0100	88.96	84.60
30	47	20	8.38	0.01384	-0.04	40.959	0.0073	82.88	78.83
60	44	20	8.87	0.01384	-0.04	37.959	0.0053	76.81	73.05
250	40	22	9.53	0.01384	0.41	34.809	0.0026	70.44	66.99
1400	34	20	10.44	0.01384	-0.04	28.459	0.0012	57.59	54.77

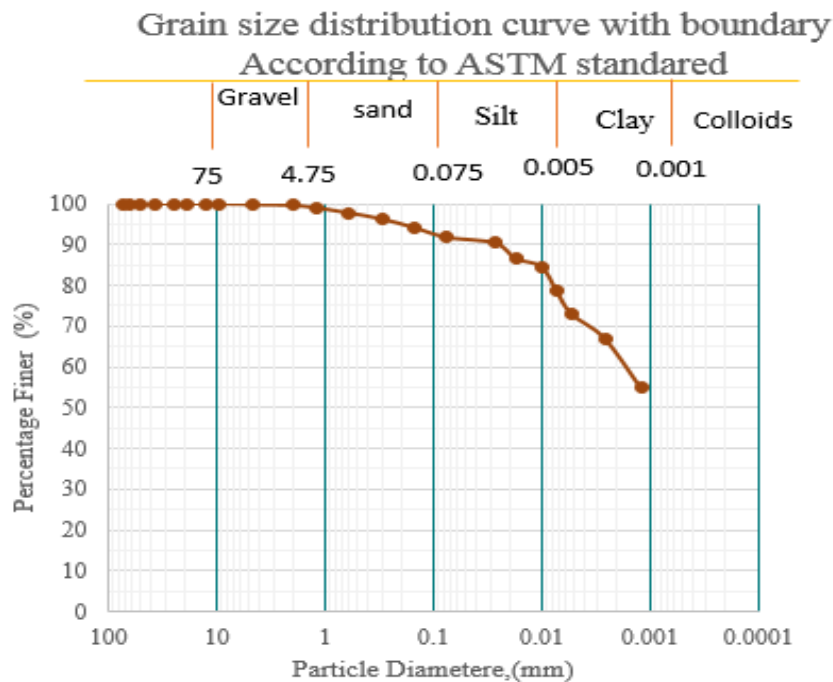


Figure A: - Typical gran size analysis test result for TP-4 @ 3m

Sample ID TP-7

Depth 1.5m

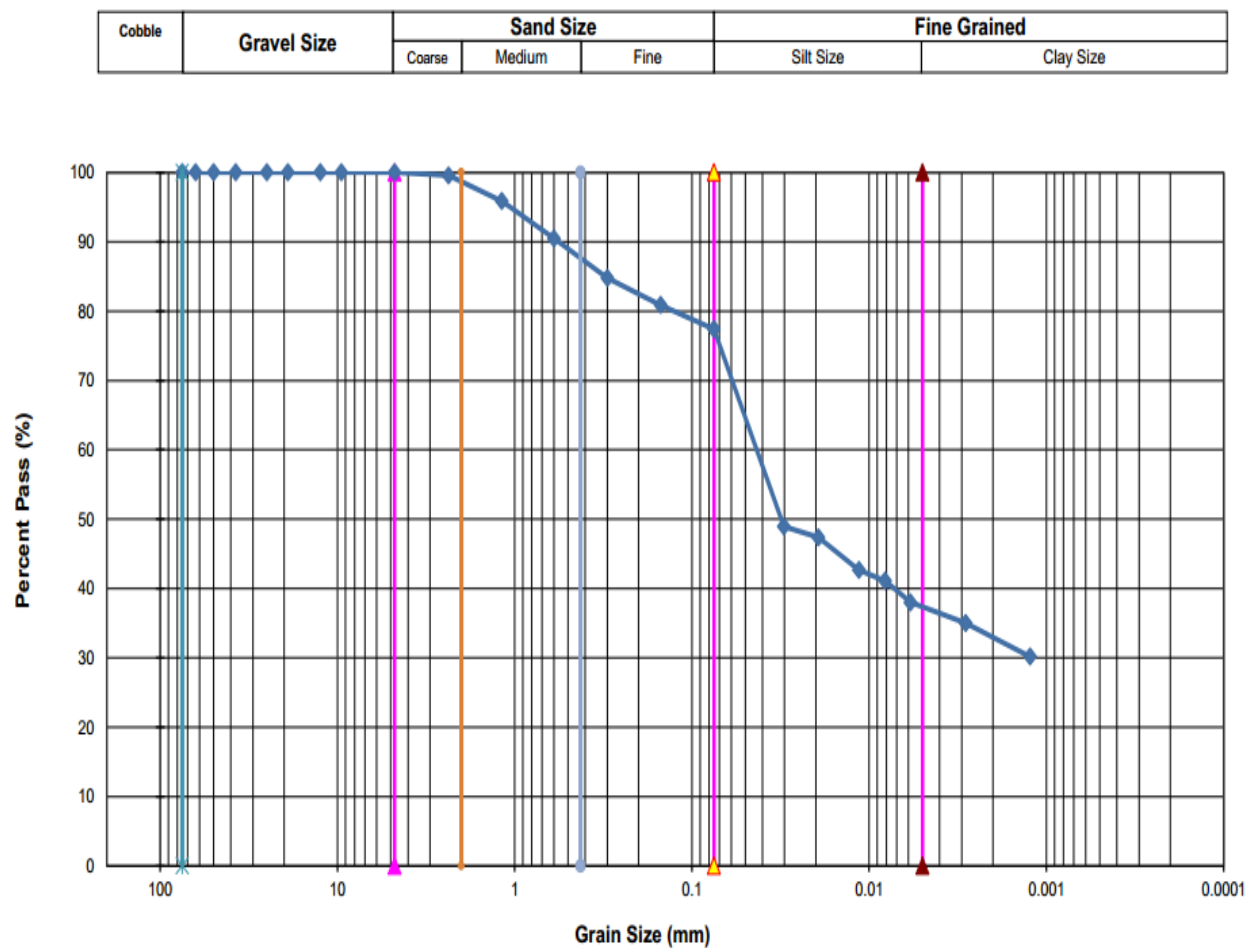
Test type	Hydrometer+ wet sieve
Sample type	Disturbed

A. sieve analysis

Sieve size	Retained for 1 st sieving		Percent passing	Wet sieving Hydrometer NO =152 H Specific gravity= 2.75 Dry mass of a soil, W= 50 Alpha (a) = 1.009 Zero correction factor CZ 5.6 Zero correction factor CZ 5.04 $D = K\sqrt{L/t}$ $P=(Ra/W)*100$
(mm)	weight	%Rtd.	%	
3" 75	0.00	0.00	100.00	
2 1/2" 63	0.00	0.00	100.00	
2" 50	0.00	0.00	100.00	
1 1/2" 37.5	0.00	0.00	100.00	
1" 25	0.00	0.00	100.00	
3/4" 19	0.00	0.00	100.00	
1/2" 12.5	0.00	0.00	100.00	
3/8" 9.5	0.00	0.00	100.00	
No 4 4.75	0.00	0.00	100.00	
No10 2	0.5	0.46	99.54	
N ^o 16 1.18	4.02	3.67	95.87	
N ^o 30 0.6	5.89	5.38	90.49	
N ^o 50 0.3	6.26	5.72	84.78	
N ^o 100 0.15	4.26	3.89	80.89	
N ^o 200 0.075	3.79	3.46	77.42	
Pan	84.78	77.42		
Total	204	100		

B. Hydrometer

Elapsed Time, t,min	Actual Hydrometer reading	Temp (°C)	Effective depth L (mm)	K From Table	Temperature correction Factor Ct	Modified hydrometer Reading $R=R_a-C_z+C_t$	Particle Diameter D (mm)	%pass	Corrected % pass
0									
2	36.5	20	10.11	0.01346	0.41	31.309	0.0303	63.20	48.94
5	35.5	20	10.27	0.01346	0.41	31.309	0.0193	61.19	47.37
15	32.5	20	10.77	0.01346	0.41	27.309	0.0114	55.13	42.68
30	31.5	20	10.93	0.01346	0.41	26.309	0.0081	53.11	41.12
60	29.5	20	11.26	0.01346	0.41	24.309	0.0058	49.07	37.99
250	26.5	22	11.76	0.01346	0.41	22.411	0.0029	45.24	35.03
1400	24.5	20	12.09	0.01346	0.41	19.309	0.0012	38.98	30.18



Appendix B AASHTO soil classification chart

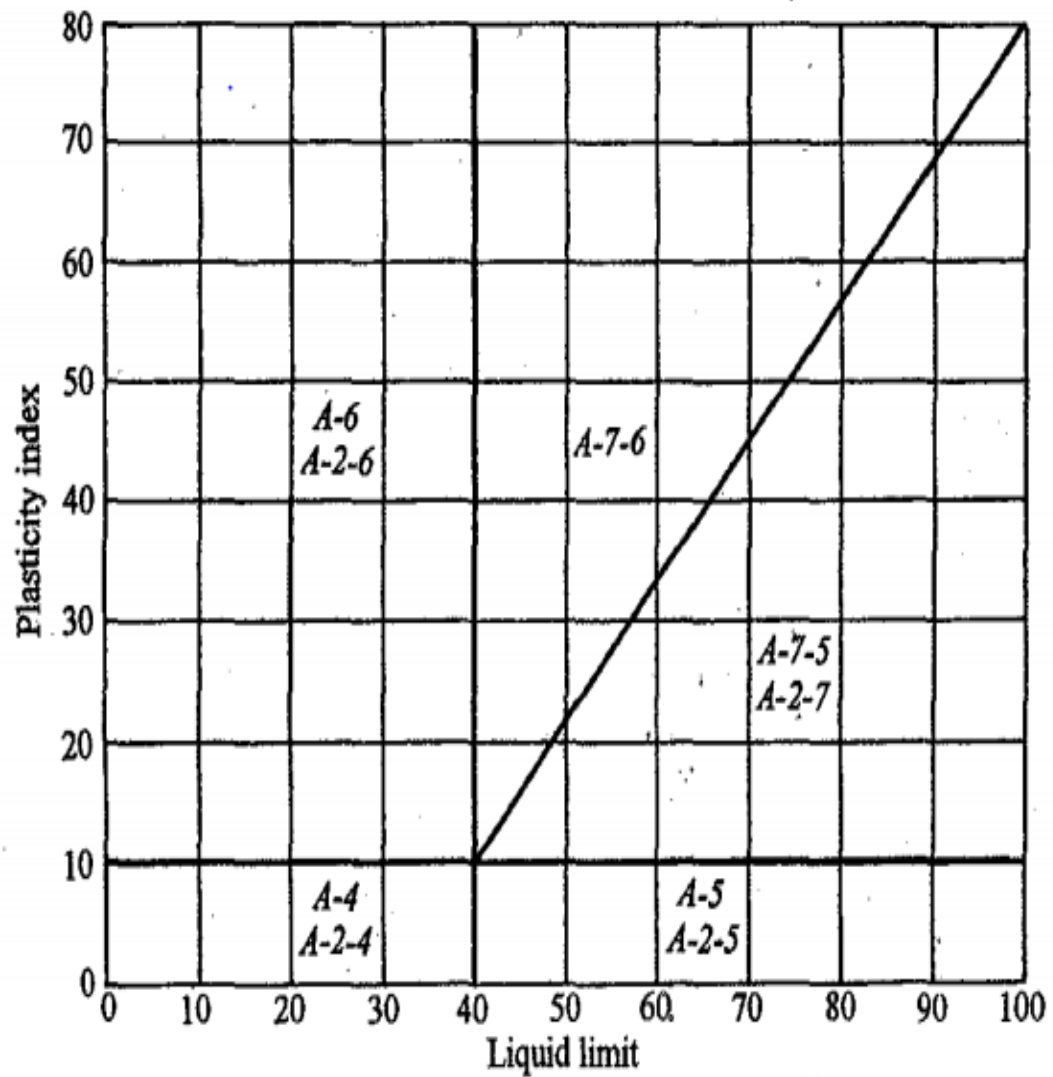


Figure B :- AASHTO classification chart.

Appendix C:- Liquid limit and plastic limit test result

1. Sample ID:-TP-6

Depth (m); - 3

Test type	Atterberg
Sample type	Disturbed

Table C -1:- Liquid limit and test result for TP-6@3m

Type of test		Liquid limit test			
		Trial -1	Trial-2	Trial-3	Trial-4
No. of Blows	N	40	30	24	16
Mass of moist sample +can	M ₁	33.01	32.34	30.42	30.87
Mass of dried sample +can	M ₂	28.14	27.72	26.02	26.32
Mass of Can	M _c	14.80	15.20	14.27	14.37
Mass of moist	M _m =M ₁ -M ₂	4.87	4.62	4.40	4.55
Mass of Dried sample	M _d =M ₂ -M _c	13.34	12.52	11.75	11.95
Moisture content	=M _m /M _d *100	36.51	36.90	37.45	38.08

Table C- 2:- Plastic limit test result for TP-6 @3m

Type of test		Plastic limit test	
		Trial-1	Trial- 2
Mass of moist sample + can	M ₁	26.56	28.62
Mass of Dried sample +Can	M ₂	24.43	26.05
Mass of can	M _c	14.85	14.84
Mass of moist	M _m =M ₁ -M ₂	2.13	2.57
Mass of Dried sample	M _d =M ₂ -M _c	9.58	11.21
Moisture content	=M _m /M _d *100	22.23	22.93

PL= 22.58

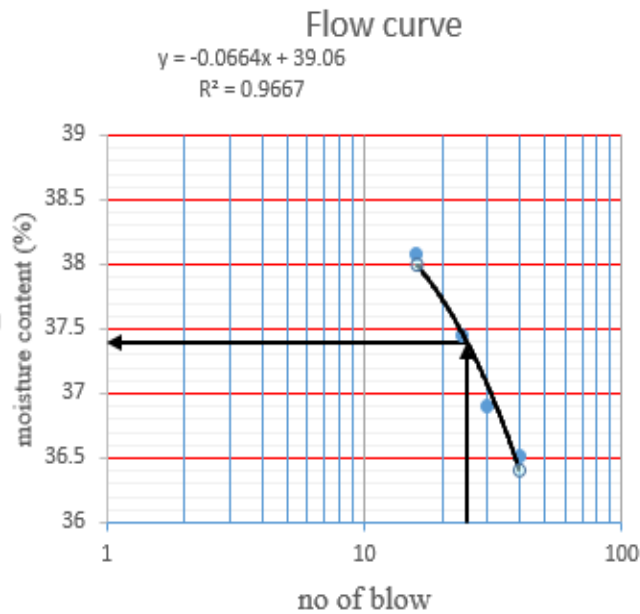


Figure C:-Typical liquid limit test result for TP-6 @3m

Result: - Liquid limit (LL) =37.37

Plastic index=LL-PL=14.79

2. Sample ID:-TP-9

Depth (m); - 3

Test type	Atterberg
Sample type	Disturbed

Table C -3:- Liquid limit test result for TP-9 @3m

Type of test	Liquid limit test			
	Trial -1	Trial-2	Trial-3	Trial-4
No. of Blows N	34	28	22	16
Mass of moist sample +can M ₁	24.78	29.27	31.95	34.75
Mass of dried sample +can M ₂	21.94	25.08	26.83	27.98
Mass of Can M _c	14.79	14.91	14.75	13.43
Mass of moist M _m =M ₁ -M ₂	2.84	4.19	5.12	6.77
Mass of Dried sample M _d =M ₂ -M _c	7.15	10.17	12.08	14.55
Moisture content =M _m /M _d *100	39.72	41.20	42.38	46.53

Table C- 4:-plastic limit test result for TP-9 @3m

Type of test		Plastic limit test	
		Trial-1	Trial- 2
Mass of moist sample + can	M_1	26.31	26.37
Mass of Dried sample +Can	M_2	24.11	24.15
Mass of can	M_c	15.10	15.02
Mass of moist	$M_m = M_1 - M_2$	2.20	2.22
Mass of Dried sample	$M_d = M_2 - M_c$	9.01	9.13
Moisture content	$= M_m / M_d * 100$	24.42	24.31

PL= 24.37

Flow curve

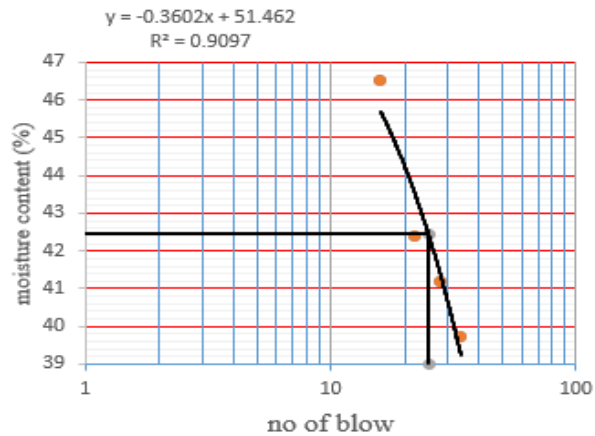


Figure C: - Typical liquid limit test result for TP-9 @3m

Result:-Liquid Limit 42.26

Plastic index = LL-Pl=17.89

Appendix D :- Compaction test result

Sample ID:-TP-10

Depth (m); - 1.5

Test type	Modified compaction
Sample type	Disturbed

Table D-1:-Compaction test result for TP-10@1.5m

	Trial 1	Trial 2	Trial 3	Trial 4
water in cc				
wt.mould+wet.sample (g)	3009.9	3185.9	3408.5	3332.2
wt.mould(g)	1622.5	1622.5	1622.5	1622.5
wt.wet soil(g)	1387.4	1563.4	1786	1709.7
volume of mould cm ³	944	944	944	944
wet density gm/cm ³	1.470	1.656	1.892	1.811
moisture content				
wet soil+tin (g)	105.5	128.8	153.7	200
Dry soil+tin (g)	93.6	107.8	123.1	145.7
Wt of tin(g)	16.1	9.6	16.5	10.9
Wt of water (g)	11.9	21	30.6	47.8
Wt dry soil in (g)	77.5	98.2	106.6	134.8
Moisture content in (%)	15.35	21.38	28.71	48.52
Dry density in gm/cm ³	1.274	1.364	1.470	1.302

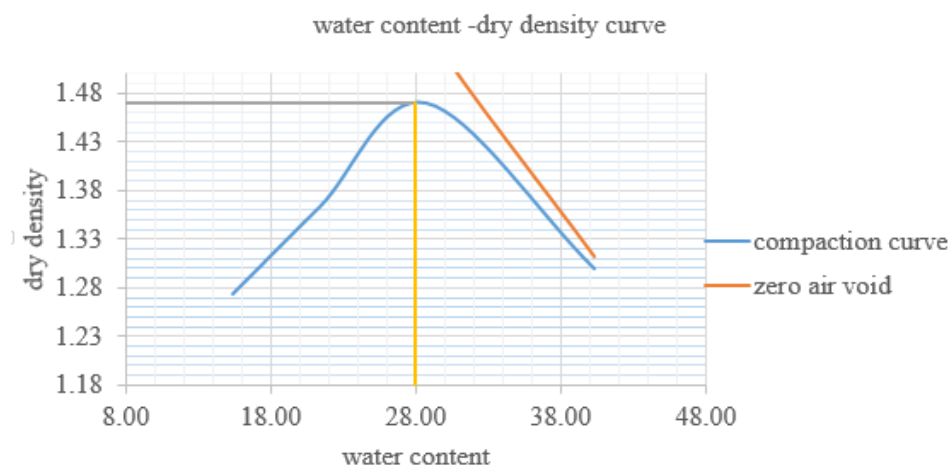


Figure D compaction test result for TP-10 @1.

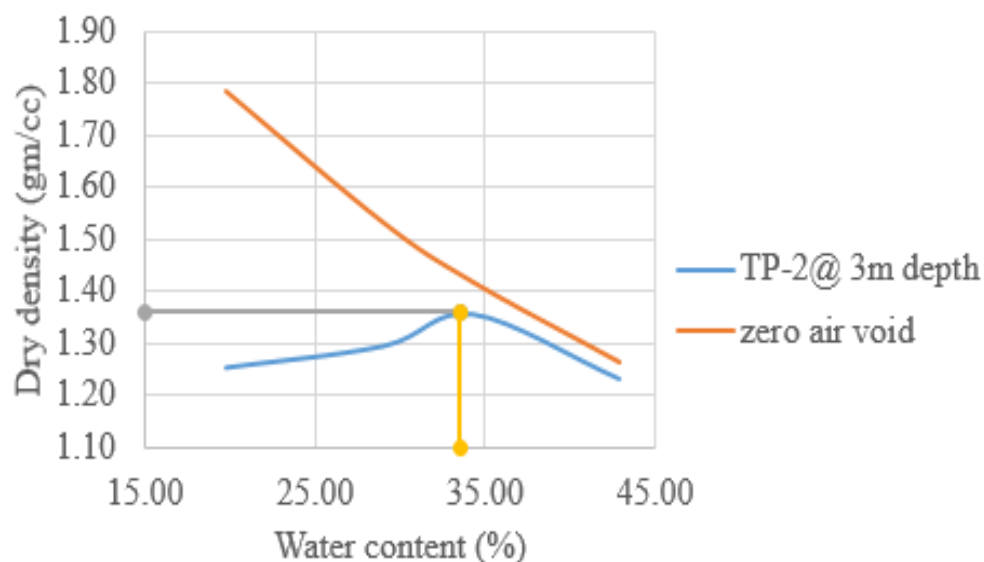
Sample ID TP-2

Depth (m):- 3

Test type	Modified compaction
Sample type	Disturbed

	Trial 1	Trial 2	Trial 3	Trial 4
water in cc				
wt.mould+wet.sample (g)	5537.8	5698.4	5841.1	5781.3
wt.mould(g)	4124.3	4124.3	4124.3	4124.3
wt.wet soil(g)	1413.5	1574.1	1716.8	1657
volume of mould cm ³	944	944	944	944
wet density gm/cm ³	1.497	1.667	1.819	1.755
moisture content				
wet soil+tin (g)	117.5	136.3	162.7	164.8
Dry soil+tin (g)	100.9	109.4	125.5	120.3
Wt of tin(g)	16.5	16.5	16.8	16.4
Wt of water (g)	16.6	26.9	37.2	44.5
Wt dry soil in (g)	84.4	92.9	108.7	103.9
Moisture content in (%)	19.67	28.96	34.22	42.83
Dry density in gm/cm ³	1.251	1.293	1.355	1.229

Water content -dry dencity curve



Appendix E: - Triaxial test result

Table E-1:- For TP-9 triaxial test dial reading

$\Delta L(\text{mm})$	Vertical strain	Dial reading			Pisto Load, P			Corrected	Devitory stress		
	$\varepsilon = \frac{\Delta L}{L_o}(\%)$	100	200	300	Cal.f* Column 3			$A = \frac{A_o}{1 - \varepsilon}$	$\sigma = \frac{p}{A}$		
0	0										
0.02	0.2702703	4.5	4.67	5	0.0065	0.0067	0.007	0.001134	5.734127	5.9507	6.37125
0.04	0.5405405	15	19.00	21	0.02168	0.0275	0.03	0.001134	19.113757	24.211	26.7593
0.06	0.8108108	28	36.80	40	0.04046	0.0532	0.058	0.001134	35.679012	46.892	50.97
0.08	1.0810811	41	47.90	57	0.05925	0.0692	0.082	0.001134	52.244268	61.037	72.6323
0.1	1.3513514	56	63.80	71	0.08092	0.0922	0.103	0.001134	71.358025	81.297	90.4718
0.12	1.6216216	67	70.32	83	0.09682	0.1016	0.12	0.001134	85.37478	89.605	105.763
0.14	1.8918919	78	85.56	94	0.11271	0.1236	0.136	0.001134	99.391534	109.02	119.78
0.16	2.1621622	94	101.00	112	0.13583	0.1459	0.162	0.001134	119.77954	128.7	142.716
0.18	2.4324324	110	112.40	127	0.15895	0.1624	0.184	0.001134	140.16755	143.23	161.83
0.2	2.7027027	118	123.40	142	0.17051	0.1783	0.205	0.001134	150.36155	157.24	180.944
0.22	2.972973	133	135.90	154	0.19219	0.1964	0.223	0.001134	169.47531	173.17	196.235
0.24	3.2432432	148	150.80	166	0.21386	0.2179	0.24	0.001134	188.58907	192.16	211.526
0.26	3.5135135	156	165.00	172	0.22542	0.2384	0.249	0.001134	198.78307	210.25	219.171
0.28	3.7837838	167	174.80	190	0.24132	0.2526	0.275	0.001134	212.79982	222.74	242.108
0.3	4.0540541	172	181.40	195	0.24854	0.2621	0.282	0.001134	219.17108	231.15	248.479
0.32	4.3243243	180	193.80	204	0.2601	0.28	0.295	0.001134	229.36508	246.95	259.947
0.34	4.5945946	185	205.70	211	0.26733	0.2972	0.305	0.001134	235.73633	262.11	268.867
0.36	4.8648649	195.4	211.60	216	0.28235	0.3058	0.312	0.001134	248.98854	269.63	275.238
0.38	5.1351351	200	215.00	222	0.289	0.3107	0.321	0.001134	254.85009	273.96	282.884
0.4	5.4054054	205.5	223.80	229	0.29695	0.3234	0.331	0.001134	261.85847	285.18	291.803
0.42	5.6756757	209.9	230.00	233	0.30331	0.3324	0.337	0.001134	267.46517	293.08	296.9
0.44	5.9459459	211.7	235.00	238	0.30591	0.3396	0.344	0.001134	269.75882	299.45	303.272
0.46	6.2162162	215.8	238.50	244	0.31183	0.3446	0.353	0.001134	274.98325	303.91	310.917
0.48	6.4864865	225.4	241.00	253	0.3257	0.3482	0.366	0.001134	287.21605	307.09	322.385
0.5	6.7567568	230.2	245.00	259.9	0.33264	0.354	0.376	0.001134	293.33245	312.19	331.178
0.52	7.027027	235.8	247.80	261.7	0.34073	0.3581	0.378	0.001134	300.46825	315.76	333.471
0.54	7.2972973	240.8	254.90	267.3	0.34796	0.3683	0.386	0.001134	306.83951	324.81	340.607
0.56	7.5675676	245.7	258.90	268.7	0.35504	0.3741	0.388	0.001134	313.08333	329.9	342.391
0.58	7.8378378	250.6	260.90	275.8	0.36212	0.377	0.399	0.001134	319.32716	332.45	351.438
0.6	8.1081081	255	265.90	282.6	0.36848	0.3842	0.408	0.001134	324.93386	338.82	360.103
0.62	8.3783784	260.1	270.80	292.9	0.37584	0.3913	0.423	0.001134	331.43254	345.07	373.228
0.64	8.6486486	265.8	280.40	304.8	0.38408	0.4052	0.44	0.001134	338.69577	357.3	388.392
0.66	8.9189189	273.9	285.40	305.6	0.39579	0.4124	0.442	0.001134	349.0172	363.67	389.411
0.68	9.1891892	278.8	296.50	307.8	0.40287	0.4284	0.445	0.001134	355.26102	377.82	392.214
0.7	9.4594595	283	299.70	309.7	0.40894	0.4331	0.448	0.001134	360.61287	381.89	394.635
0.72	9.7297297	289.8	304.00	310.2	0.41876	0.4393	0.448	0.001134	369.27778	387.37	395.272
0.74	10	297.6	305.30	311.6	0.43003	0.4412	0.45	0.001134	379.21693	389.03	397.056
0.76	10.27027	301.5	306.50	312.4	0.43567	0.4429	0.451	0.001134	384.18651	390.56	398.076
0.78	10.540541	307.1	309.80	313.5	0.44376	0.4477	0.453	0.001134	391.32231	394.76	399.478
0.8	10.810811	243.7	276.50	289.6	0.35215	0.3995	0.418	0.001134	310.53483	352.33	369.023

Table E-2:- for TP-6 triaxial test dial reading

$\Delta L(\text{mm})$	Vertical strain	Dial reading			Piston Load, P			corrected area	Devitory stress		
	$\varepsilon = \Delta L/L_0(\%)$	100	200	300	Cal.f * Column 3			$A = A_0/(1-\varepsilon)$	$\sigma = p/A$		
0	0	0	0	0	0	0	0	0.001134	0	0	0
0.02	0.27027027	11	12.67	16	0.02	0.02	0.02	0.001134	14.02	16.14	20.388
0.04	0.540540541	20	21.33	24	0.03	0.03	0.03	0.001134	25.49	27.18	30.582
0.06	0.810810811	36	44.33	61	0.05	0.06	0.09	0.001134	45.87	56.49	77.729
0.08	1.081081081	71	80	100	0.1	0.12	0.14	0.001134	90.47	101.9	127.55
0.1	1.351351351	94	111.8	120	0.14	0.16	0.17	0.001134	119.8	142.5	152.91
0.12	1.621621622	105	117.9	130	0.15	0.17	0.19	0.001134	133.8	150.2	166.16
0.14	1.891891892	116	134.6	151	0.17	0.19	0.22	0.001134	147.8	171.5	191.84
0.16	2.162162162	120	143.8	166	0.17	0.21	0.24	0.001134	152.9	183.2	211.27
0.18	2.432432432	132.2	165.6	180	0.19	0.24	0.26	0.001134	168.5	211	229.87
0.2	2.702702703	145.8	169.4	181	0.21	0.24	0.26	0.001134	185.8	215.9	230.64
0.22	2.972972973	151.2	174.5	182	0.22	0.25	0.26	0.001134	192.7	222.4	231.91
0.24	3.243243243	164.6	178.6	183	0.24	0.26	0.26	0.001134	209.7	227.6	233.19
0.26	3.513513514	173.4	180.4	184	0.25	0.26	0.27	0.001134	221	229.9	234.46
0.28	3.783783784	180.4	182.3	185	0.26	0.26	0.27	0.001134	229.9	232.3	235.74
0.3	4.054054054	181.4	184.7	186	0.26	0.27	0.27	0.001134	231.1	235.4	237.01
0.32	4.324324324	182.4	185.6	187	0.26	0.27	0.27	0.001134	232.4	236.5	238.28
0.34	4.594594595	183.4	186.7	188	0.27	0.27	0.27	0.001134	233.7	237.9	239.56
0.36	4.864864865	184	188.5	189	0.27	0.27	0.27	0.001134	234.5	240.2	240.83
0.38	5.135135135	185	189.5	190	0.27	0.27	0.27	0.001134	235.7	241.5	242.11
0.4	5.405405405	186	190.3	191	0.27	0.27	0.28	0.001134	237	242.5	243.38
0.42	5.675675676	187	191.2	192	0.27	0.28	0.28	0.001134	238.3	243.6	244.66
0.44	5.945945946	188	192.3	193	0.27	0.28	0.28	0.001134	239.6	245	245.93
0.46	6.216216216	189	193.4	194	0.27	0.28	0.28	0.001134	240.8	246.4	247.2
0.48	6.486486486	190	195.6	196	0.27	0.28	0.28	0.001134	242.1	249.2	249.63
0.5	6.756756757	191	197.4	196	0.28	0.29	0.28	0.001134	243.4	251.5	249.88
0.52	7.027027027	192	198.5	198	0.28	0.29	0.29	0.001134	244.7	252.9	251.66
0.54	7.297297297	193	194.3	199	0.28	0.28	0.29	0.001134	245.9	247.6	253.7
0.56	7.567567568	194	196.4	200	0.28	0.28	0.29	0.001134	247.2	250.3	254.85
0.58	7.837837838	195	195.8	201	0.28	0.28	0.29	0.001134	248.5	249.5	256.12
0.6	8.108108108	148.2	151.3	160	0.21	0.22	0.23	0.001134	188.8	192.8	203.88

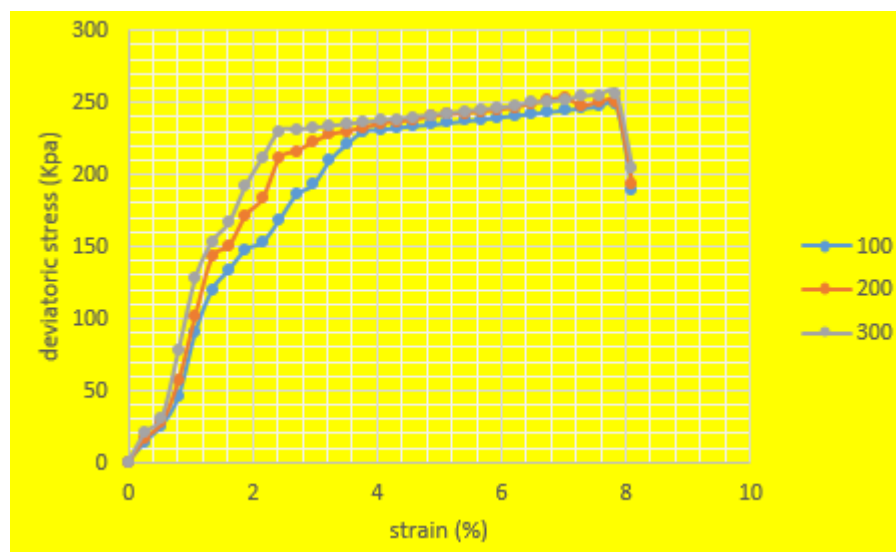


Figure E-2:- Typical Deviator stress Vs strain curve for TP-6@3m

Table E-3:- For TP-4 Dial reading

$\Delta L(mm)$	Vertical strain	Dial reading			Piston Load, P		corrected Area		Devitory stress		
	$\epsilon = \Delta L/L_o(\%)$	100	200	300	Cal.f * Column 3		$A = A_o/(1-\epsilon)$		$\sigma = p/A$		
0	0	0	0	0	0	0	0	0.00113	0	0	0
0.02	0.27027027	4.5	12.3	13.3	0.0065	0.01777	0.01922	0.00113	5.73413	15.6733	16.9475
0.04	0.54054054	7.2	15.3	19.8	0.0104	0.02211	0.02861	0.00113	9.1746	19.496	25.2302
0.06	0.81081081	10.5	18.3	23.6	0.01517	0.02644	0.0341	0.00113	13.3796	23.3188	30.0723
0.08	1.08108108	14.4	21.3	26.7	0.02081	0.03078	0.03858	0.00113	18.3492	27.1415	34.0225
0.1	1.35135135	18.2	24.3	30.1	0.0263	0.03511	0.04349	0.00113	23.1914	30.9643	38.3549
0.12	1.62162162	21.3	27.3	35.6	0.03078	0.03945	0.05144	0.00113	27.1415	34.787	45.3633
0.14	1.89189189	24.6	30.3	38.9	0.03555	0.04378	0.05621	0.00113	31.3466	38.6098	49.5683
0.16	2.16216216	27.6	33.3	43.2	0.03988	0.04812	0.06242	0.00113	35.1693	42.4325	55.0476
0.18	2.43243243	30.6	36.3	47.3	0.04422	0.05245	0.06835	0.00113	38.9921	46.2553	60.272
0.2	2.7027027	33.6	39.3	50.3	0.04855	0.05679	0.07268	0.00113	42.8148	50.078	64.0948
0.22	2.97297297	36.6	42.3	53.4	0.05289	0.06112	0.07716	0.00113	46.6376	53.9008	68.045
0.24	3.24324324	39.6	45.3	55.7	0.05722	0.06546	0.08049	0.00113	50.4603	57.7235	70.9757
0.26	3.51351351	42.6	48.3	56.5	0.06156	0.06979	0.08164	0.00113	54.2831	61.5463	71.9951
0.28	3.78378378	45.6	51.3	57.3	0.06589	0.07413	0.0828	0.00113	58.1058	65.369	73.0146
0.3	4.05405405	48.6	54.3	58.1	0.07023	0.07846	0.08395	0.00113	61.9286	69.1918	74.034
0.32	4.32432432	51.6	57.3	58.9	0.07456	0.0828	0.08511	0.00113	65.7513	73.0146	75.0534
0.34	4.59459459	54.6	57.8	61	0.0789	0.08352	0.08815	0.00113	69.5741	73.6517	77.7293
0.36	4.86486486	56.2	60.1	62.7	0.08121	0.08684	0.0906	0.00113	71.6129	76.5825	79.8955
0.38	5.13513514	57.3	62.3	63.5	0.0828	0.09002	0.09176	0.00113	73.0146	79.3858	80.9149
0.4	5.40540541	60.4	64	65.2	0.08728	0.09248	0.09421	0.00113	76.9647	81.552	83.0811
0.42	5.67567568	63.5	65	66.2	0.09176	0.09393	0.09566	0.00113	80.9149	82.8263	84.3554
0.44	5.94594595	66.8	67.8	68.3	0.09653	0.09797	0.09869	0.00113	85.1199	86.3942	87.0313
0.46	6.21621622	40	45.7	54.7	0.0578	0.06604	0.07904	0.00113	50.97	58.2332	69.7015

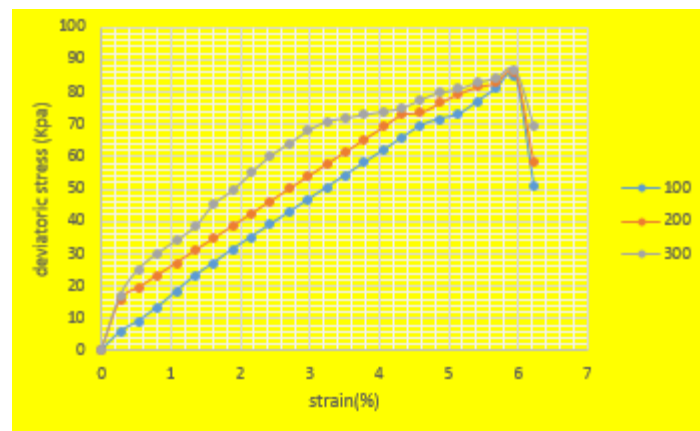


Figure E-3:- Typical Deviator stress Vs strain curve for TP-4

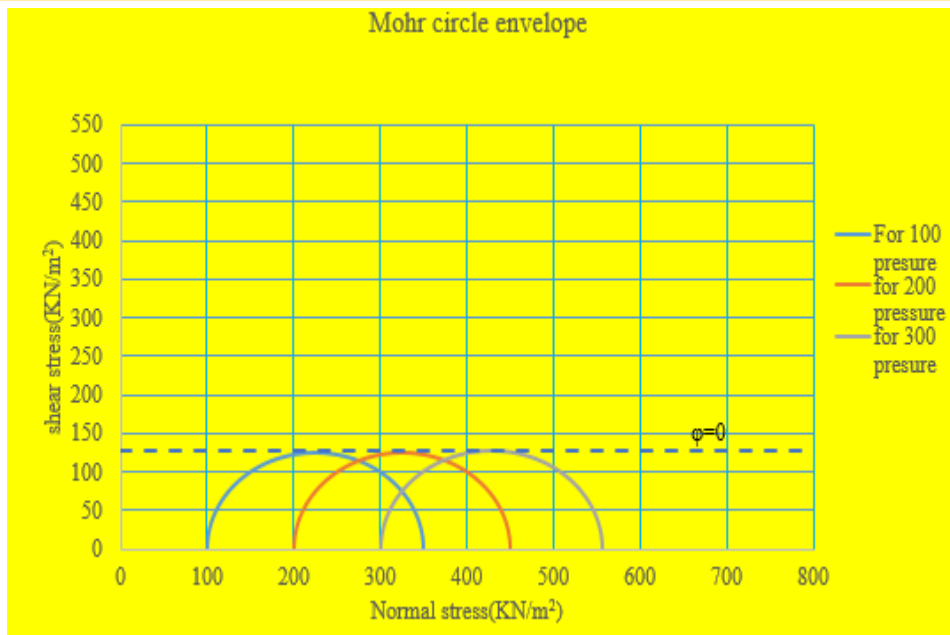


Figure:-Mohr circle envelope for TP-6

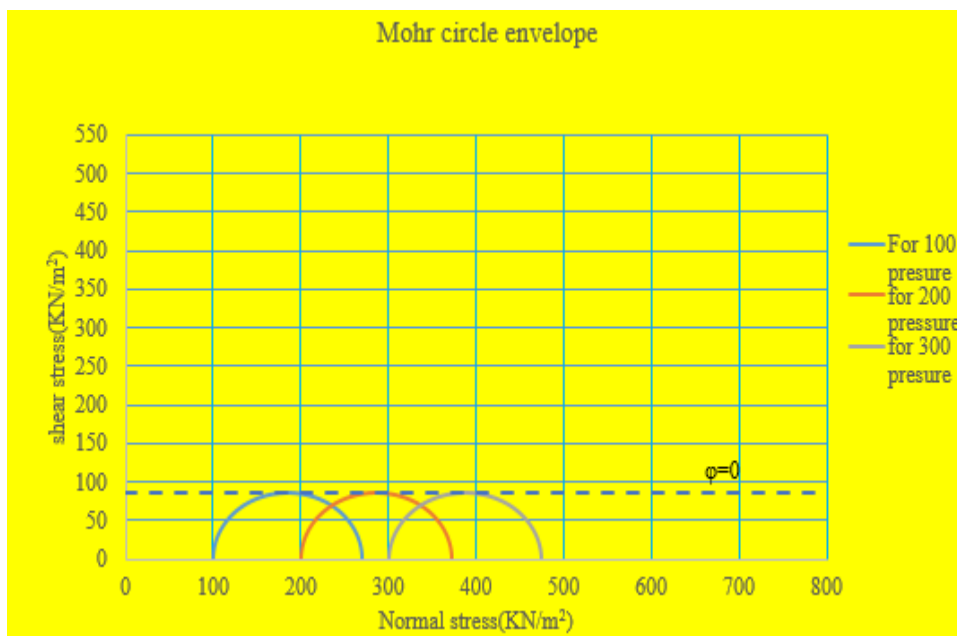


Figure: - Mohr circle envelope for TP-4

Appendix F consolidation test Result

F-1 compression index (Cc) for loading

From the e-log σ' Figure 4.10

Sample ID: - TP-9

Void ratio $e_1=0.89$ and $e_2=0.69$ Pressure $\sigma_1 = 222 \text{ KN/m}^2$ and $\sigma_2 = 1776 \text{ KN/m}^2$

$$Cc = \frac{-\Delta e}{\log_{10}\left(\frac{\sigma}{\sigma_0}\right)}$$

$$\begin{aligned} Cc &= \frac{-(e_2 - e_1)}{\log_{10}\left(\frac{\sigma_2}{\sigma_1}\right)} \\ &= \frac{-(0.658 - 0.865)}{\log_{10}\left(\frac{1776}{222}\right)} \\ &= 0.221 \end{aligned}$$

F-2 swelling index (Cs) for unloading

Sample ID:-TP-4

Void ratio $e_1=0.75$ and $e_2=0.6$ Pressure $\sigma_1 = 111 \text{ KN/m}^2$ and $\sigma_2 = 444 \text{ KN/m}^2$

$$\begin{aligned} Cs &= \frac{-\Delta e}{\log_{10}\left(\frac{\sigma}{\sigma_0}\right)} \\ &= \frac{-(0.573 - 0.602)}{\log_{10}\left(\frac{444}{111}\right)} \\ &= 0.249 \end{aligned}$$

F-3 Recompression (Cr) loading

Sample ID: - TP-9

From the e-log σ' Figure 4.12

$$Cr = \frac{-\Delta e}{\log_{10}\left(\frac{\sigma + \Delta\sigma}{\sigma}\right)}$$

Void ratio $e_1=0.72$ and $e_2=0.71$

$$= \frac{-(0.71-0.72)}{\log_{10}\left(\frac{125+285}{125}\right)}$$

Pressure $\sigma_1=125\text{KN/m}^2$ and $\sigma_2=285\text{ KN/m}^2$

$$=0.02$$

➤ One Dimensional consolidation test calculation and reading

Sample ID- TP-4

Lab No.	1635/12
Sample type	Disturbed
Test type	consolidation

Specimen data

Before test

Table F-1 TP-4 consolidation test table before test specimen data

Weight of sample and ring (gm)	265.5
Weight of ring (gm)	107.5
Weight of sample (gm)	158
Weight of dry sample (gm)	118.7
Weight of initial moisture (gm)	39.3
Initial moisture content (Mo), (%)	33.11
Initial void ratio $e_o = (Gs/rd)-1$	1.07
Height of solid $H_s = H_o/1+e_o$	9.66
Initial saturation : $S_o = Mo*Gs/1+e_o$	92.01
Specific gravity (Gs)	2.78
Sample Diameters (D) (mm)	75
Sample thickness (H _o) (mm)	20
Area (A) (cm ²)	44.18
volume (V) (cm ³)	88.37
Bulk density (rb) (gm/cm ³)	1.76
Dry density (rd) (gm/cm ³)	1.34

After test

Table F-2 consolidation test table after test specimen data

Weight of sample + ring+ can (gm)	261.3
Weight of sample + ring+ can (gm)	226.2
Weight of ring +can (gm)	107.5
Weight of weight sample (gm)	153.8
Weight of dry sample (gm)	118.7
Final moisture content (Mf) (%)	29.57
overall settlement (mm)	4.676
volume change (cm ³)	20.66
Final volume (cm ³)	67.71
Final bulk density (gm/cm ³)	2.272
Final dry density (gm/cm ³)	1.753

Table F-3 consolidation test specimen data

Pressure (kPa)	Cumulative Compression (ΔH) mm	Consolidated height $H=H-\Delta H$ mm	Void Ratio $e=H-H_s/H_s$	δH	δp	M_v	t_{90} (min)	$H=0.5(H_1+H_2)$	$C_v (0.84 \cdot H_2)/t_{90} m^2/year$
0	0	20	0.94			0.87			
				1.9	110.98				
111	1.92	18.08	0.75			0.43			6.4
				0.9	110.98		47.61	19.04	
222	2.776	17.224	0.67			0.19			4.7
				0.7	221.96		56.25	17.652	
444	3.514	16.486	0.60			0.12			3.2
				0.9	443.91		74.39	16.855	
888	4.3706	15.6294	0.51			0.05			4.5
				0.7	887.83		47.61	16.0577	
1776	5.058	14.942	0.45						4.1
							47.61	15.2857	
444	4.8	15.2	0.47						
111	4.516	15.484	0.50						
222	4.568	15.432	0.49			0.03			5.0
				0.1	221.96		40.07	15.458	
444	4.676	15.324	0.48		110.98	0.87			4.2

Sample ID: - TP-6

Lab No.	1635/12
Sample type	Undisturbed
Test type	consolidation

Specimen data

Before test

Table F-4 consolidation test table before test specimen data

Weight of sample and ring (gm)	250.3
Weight of ring (gm)	109.6
Weight of sample (gm)	140.7
Weight of dry sample (gm)	123.9
Weight of initial moisture (gm)	30.8
Initial moisture content (Mo), (%)	28.03
Initial void ratio $e_o = (G_s/r_d) - 1$	0.98
Height of solid $H_s = H_o / (1 + e_o)$	10.08
Initial saturation : $S_o = (M_o * G_s) / (1 + e_o)$	70.2
Specific gravity (G_s)	2.78
Sample Diameters (D) (mm)	75
Sample thickness (H_o) (mm)	20
Area (A) (cm ²)	44.18
volume (V) (cm ³)	88.37
Bulk density (rb) (gm/cm ³)	1.76
Dry density (rd) (gm/cm ³)	1.40

After test

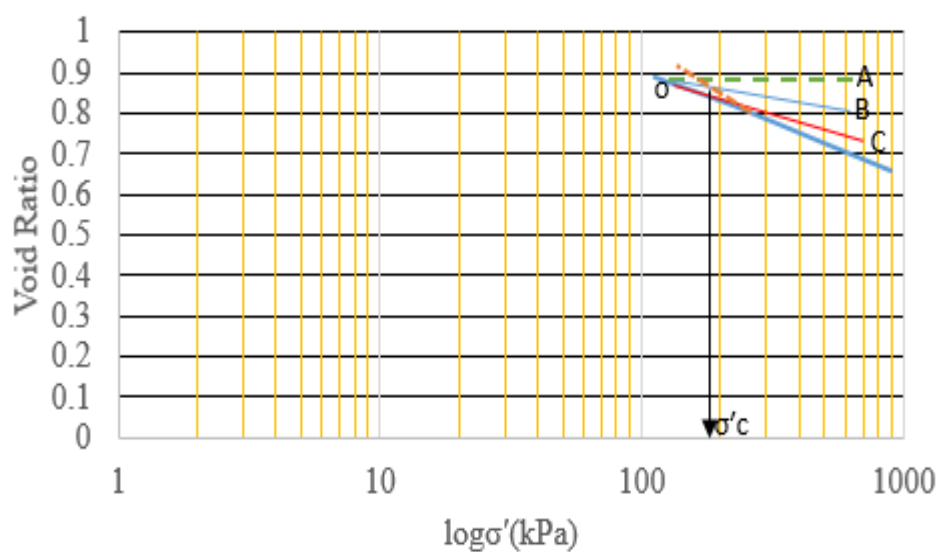
Table F-7 consolidation test table after test specimen data

Weight of sample + ring+ can (gm)	249.2
Weight of sample + ring+ can (gm)	216.5
Weight of ring +can (gm)	109.6
Weight of weight sample (gm)	139.6
Weight of dry sample (gm)	109.9
Final moisture content (Mf) (%)	27.02
overall settlement (mm)	3.822
volume change (cm ³)	16.89
Final volume (cm ³)	71.48
Final bulk density (gm/cm ³)	1.953
Final dry density (gm/cm ³)	1.537

Table F-10 consolidation test specimen data

Pressure (kPa)	Cumulative Compression (ΔH) mm	Consolidated height $H=H-\Delta H$ mm	Void Ratio $e=H-H_s/H_s$	δH	δp	M_V	t_{90} (min)	$H=0.5(H_1+H_2)$	$C_v (0.84 \cdot H_2)/t_{90} \text{m}^2/\text{year}$
0	0	20	0.98						
				0.9	110.98	0.41			
111	0.902	19.098	0.89						
				0.7	110.98	0.34	15.92	19.549	20.2
222	1.632	18.368	0.82						
				0.8	221.96	0.19	21.16	18.733	13.9
444	2.414	17.586	0.74						
				0.9	443.91	0.11	16.24	17.977	16.7
888	3.296	16.704	0.66						
				0.8	887.83	0.06	16.24	17.145	15.2
1776	4.136	15.864	0.57						
							16.24	16.284	13.7
444	3.906	16.094	0.60						
111	3.578	16.422	0.63						
222	3.655	16.345	0.62						
				0.2	221.96	0.05	40.07	16.3835	5.6
444	3.822	16.178	0.60		110.98				

Determination of preconsolidation pressure



Sample ID: - TP-9

Lab No.	1635/12
Sample type	Undisturbed
Test type	consolidation

Specimen data

Before test

Table F-11 consolidation test table before test specimen data

Weight of sample and ring (gm)	264.9
Weight of ring (gm)	109.1
Weight of sample (gm)	155.8
Weight of dry sample (gm)	116.8
Weight of initial moisture (gm)	39
Initial moisture content (Mo), (%)	33.39
Initial void ratio $e_o = (G_s/r_d) - 1$	1.0
Height of solid $H_s = H_o / (1 + e_o)$	9.75
Initial saturation : $S_o = (M_o * G_s) / (1 + e_o)$	70.2
Specific gravity (G_s)	2.71
Sample Diameters (D) (mm)	75
Sample thickness (H_o) (mm)	20
Area (A) (cm ²)	44.18
volume (V) (cm ³)	88.37
Bulk density (rb) (gm/cm ³)	1.768
Dry density (rd) (gm/cm ³)	1.32

After test

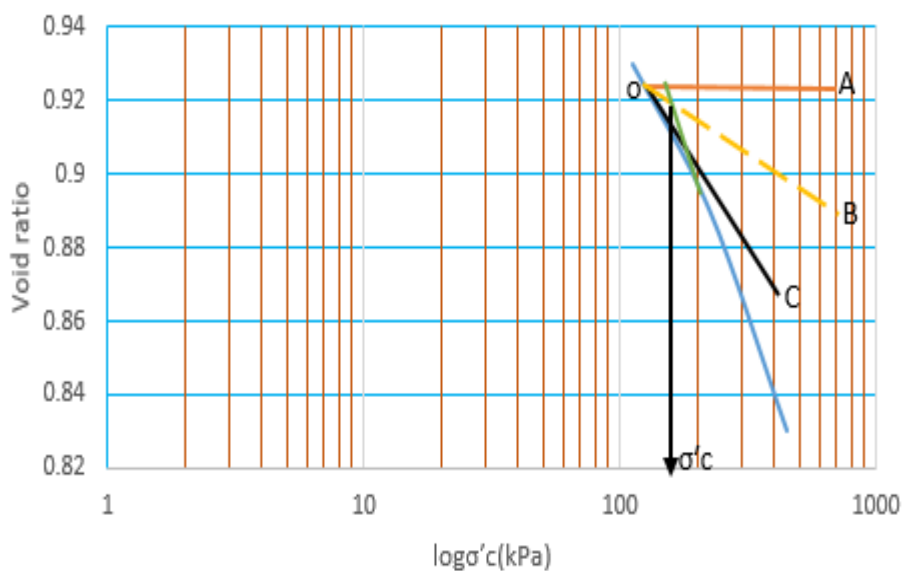
Table F-13 consolidation test table after test specimen data

Weight of sample + ring+ can (gm)	258.3
Weight of sample + ring+ can (gm)	225.9
Weight of ring +can (gm)	109.1
Weight of weight sample (gm)	149.2
Weight of dry sample (gm)	116.8
Final moisture content (Mf) (%)	27.74
overall settlement (mm)	3.062
volume change (cm ³)	13.53
Final volume (cm ³)	74.84
Final bulk density (gm/cm ³)	1.994
Final dry density (gm/cm ³)	1.561

Table F-15 consolidation test table after test specimen data

Pressure (kPa)	Cumulative Compression (ΔH) mm	Consolidated height $H=H-\Delta H$ mm	Void Ratio $e=H-H_s/H_s$	δH	δp	M_v	t_{90} (min)	$H=0.5(H_1+H_2)$	$C_v (0.84 \cdot H_2)/t_{90} \text{m}^2/\text{year}$
0	0	20	1.05		110.98	0.52			
				1.1					
111	1.144	18.856	0.93		110.98	0.20	21.16		
				0.4				19.428	15.0
222	1.56	18.44	0.89		221.96	0.14	56.25		
				0.6				18.648	5.2
444	2.138	17.862	0.83		443.91	0.09	69.39		
				0.7				18.151	4.0
888	2.86	17.14	0.76		887.83	0.05	33.06		
				0.7				17.501	7.8
1776	3.57	16.43	0.69				40.07		
								16.785	5.9
444	3.008	16.992	0.74						
111	2.782	17.218	0.77						
222	2.864	17.136	0.76		221.96	0.05	84.64		
				0.2				17.177	2.9
444	3.062	16.938	0.74		110.98	0.52	26.83		

Determination of preconsolidation pressure



Appendix G Coefficient of permeability

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

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

C_v =coefficient of consolidation

a_v =coefficient of compressibility

γ_w = unit weight of water

e = void ratio

For TP-6 pressure of 1776 for pressure 1600 read from figure (4.13) =0.6

$$e = 0.57$$

$$C_v = 4.34 * 10^{-3} \text{ cm}^2/\text{sec}$$

$$\text{From figure (4.13)} \quad a_v = \frac{0.57 - 0.6}{1600 - 1776}$$

$$= 0.17 * 10^{-5} \text{ m}^2/\text{KN}$$

$$K = \frac{4.34 * 10^{-3} \frac{\text{cm}^2}{\text{sec}} * 0.17 * 10^{-5} \frac{\text{m}^2}{\text{KN}} * 9.81 \frac{\text{KN}}{\text{m}^3}}{1 + 0.57}$$

$$= 45.95 * 10^{-9} \text{ cm/sec}$$

For TP-6 pressure of 444 and for pressure 350 read from figure (4.11) =0.64

$$e = 0.74$$

$$C_v = 5.35 * 10^{-3} \text{ cm}^2/\text{sec}$$

$$\text{From figure (4.13)} \quad a_v = \frac{0.64 - 0.74}{350 - 550}$$

$$= 0.5 * 10^{-5} \text{ m}^2/\text{KN}$$

$$K = \frac{5.35 * 10^{-3} \frac{\text{cm}^2}{\text{sec}} * 0.5 * 10^{-5} \frac{\text{m}^2}{\text{KN}} * 9.81 \frac{\text{KN}}{\text{m}^3}}{1 + 0.74}$$

$$= 15.08 * 10^{-9} \text{ cm}$$

Appendix H Correlation and regression analysis @ depth of 3m

H-1:- The scatter plot

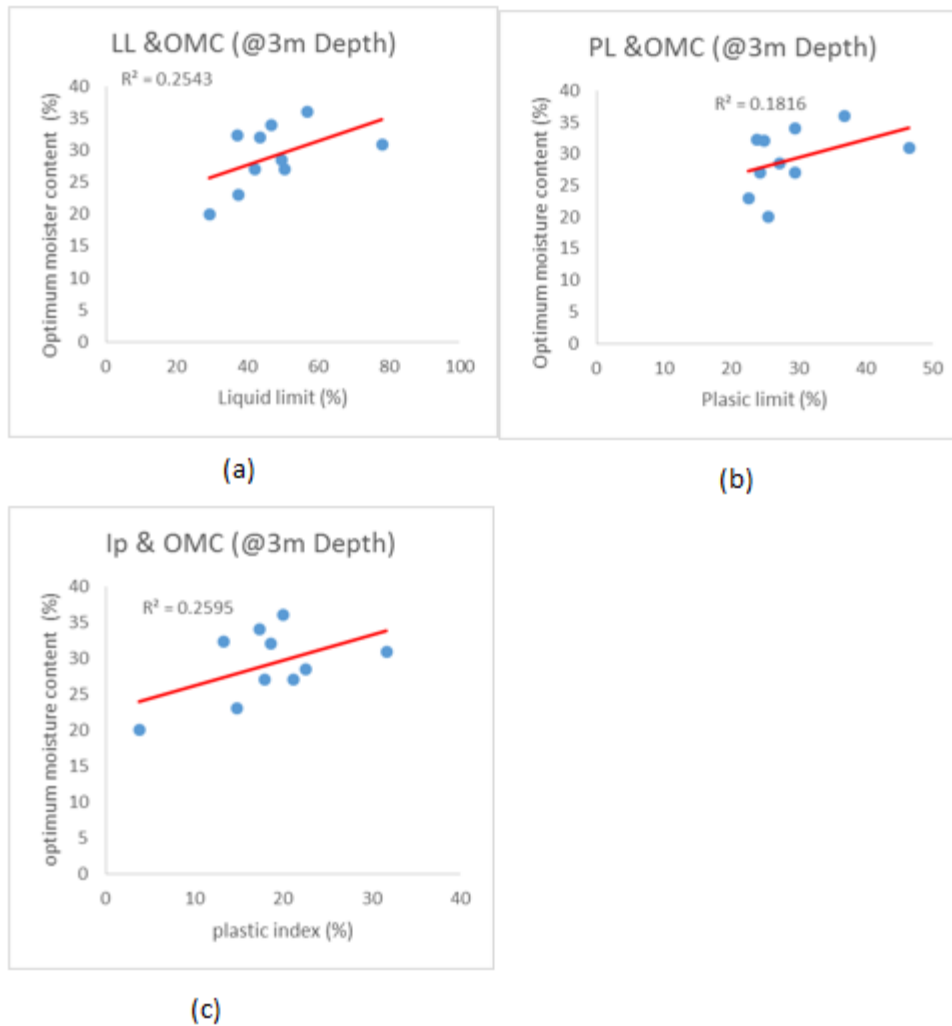


Figure H-1 the scatter plot diagram (a) for LL with OMC (b) for PL with OMC and (c) for Ip with OMC

Table H-2 correlation table

	LL (%)	PL (%)	Ip (%)	OMC (%)	MDD (gm/cc)
LL (%)	1				
PL (%)	0.92911	1			
Ip (%)	0.92448	0.71796	1		
OMC (%)	0.50427	0.42612	0.50944	1	
MDD (gm/cc)	-0.8792	-0.8302	-0.7987	-0.8191991	1

H-3:- The equation, linear regression

$$MDD = 1.97 - (0.0197 * OMC) \dots\dots\dots (4.18)$$

SUMMARY OUTPUT

Regression Statistics

Multiple R	0.819199051
R Square	0.671087085
Adjusted R Square	0.629972971
Standard Error	0.072942289
Observations	10

Table H-3 linear regression table

Predictor	Coefficient	Standard error	t stat	P value	Significance F
Intercept	1.9718	0.144	13.68	7.85	0.00373
OMC (%)	-0.0197	0.00489	-4.04	0.0037	