ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF GRADUATE STUDIES

CIVIL ENGINEERING DEPARTMENT

Correlation between Undrained Shear Strength and Index properties of Red Clay Soils in southern Ethiopia (in case of Bule Hora town)

A thesis submitted to School of Graduate Studies of Adama Science and Technology University in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering under Geotechnical Engineering.

BY: YOHANNES ANOLE

ADVISOR: Dr. ARGAW ASHA

OCTOBER, 2018

ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF GRADUATE STUDIES

CIVIL ENGINEERING DEPARTMENT

Master's Thesis

Name:	**Yohannes Anole**
School:	**Civil Engineering and Architecture**
Department:	**Civil Engineering**
Major:	**Geotechnical Engineering**
Advisor:	**Dr. Argaw Asha**
Thesis Title:	**Correlation Between Undrained Shear Strength And Index Properties Of Red Clay Soils In Southern Ethiopia (In Case Of Bule Hora Town)**

APPROVAL PAGE

This thesis entitled with **"Correlation between Undrained Shear Strength and Index properties of Red Clay Soils in southern Ethiopia (in case of Bule Hora town)"** has been approved by the following examiners in partial fulfillment of the requirement for the degree master of science in Civil Engineering (Geotechnical Eng.)

Approved by:

Dr. Argaw Asha______	______________	________________
Advisor	signature	Date
______________________	______________	________________
Internal examiner	signature	Date
______________________	______________	________________
External examiner	signature	Date
______________________	______________	________________
Chair Person	signature	Date
______________________	______________	________________
Department head	signature	Date
______________________	______________	________________
School Dean (SoCEA)	signature	Date
______________________	______________	________________
School of post Graduate Studies	signature	Date

ACKNOWLEDGMENT

First of all, I would like to thank almighty God. I wish to express my deep appreciation and gratitude to my advisor Dr. Argaw Asha for immense guidance and supervision all the way through this research work.

I gratefully acknowledge to Adama Science and Technology University Geotechnical laboratory staff, Ethiopian construction Design & supervision works corporation laboratory staffs for their cooperation and for making the work environment amiable.

Last but not least, my heartfelt gratitude goes to those who assisted me this research work to reach final, particularly my beloved family and friends for their unreserved support and encouragement throughout my journey.

TABLE OF CONTENTS

Content Page

LIST OF TABLES

LISTS OF FIGURES

SYMBOLS AND ABBREVIATIONS

ASTM;	American Society for testing and Materials
AASHTO;	American Association of State Highway and Transportation Officials.
SPSS;	Statistical package for the social science
ML;	Inorganic low plasticity silts
CL;	Inorganic low plasticity clay
OL;	Organic low plasticity silts
MH;	Inorganic high compressible of silts
P200;	percentage of soil pass through No. 200 sieve
CH;	Inorganic high plasticity clay
OH;	Organic high plasticity silts
LL;	Liquid limit
LI;	Liquidity Index
PI;	Plasticity Index
PL;	Plastic limit
OMC;	Optimum moisture content for compaction test
MDD;	Maximum dry density
Su;	Undrained shear strength
$\sigma_{v'}$;	Effective overburden pressure
c;	Cohesion
σ;	Total Stress
$\emptyset$;	Angle of internal friction
R^2;	Correlation Coefficient
MLRA;	Multiple Linear Regression Analysis
SLRA;	Simple Linear Regression Analysis
a_1, a_2, a_3;	partial regression coefficients

ANOVA and Analysis of Variance

Df	degrees of freedom

ABSTRACT

Undrained shear strength is one of the useful parameters in order to take engineering decisions. Most often the bearing capacity is estimated based on undrained shear strength to make conservative estimates. Some laboratory tests needed to obtain these values are expensive and time consuming, while soil properties like moisture content and Atterberg limits can be performed faster and cheaper. Many empirical formulae are available to estimate the undrained shear strength for fine grained soils like clay or silt. Determining of undrained shear strength parameters in laboratory are really tedious and time consuming. Therefore, a correlation between undrained shear strength and index properties is useful for restraint of testing number and costs. In this thesis, attempts have been made to obtain valid correlations between undrained shear strength of Red clay soils with their index properties. For this purpose, 18 samples were collected from Bule Hora town and a series of laboratory tests conducted as a primary data for this work from these two samples considered as Control Point. In the analysis part, both the MS excel spreadsheet and the SPSS software has been used for the scatter plot, correlation, and regression analysis. Attempts were made to obtain the relationships of all the parameters (water content, liquid limit, liquidity index, plastic limit, plasticity index, optimum moisture content, and maximum dry density).The analysis results show that S_u has strong negative relation with LI and relatively weak positive relation with MDD. From the regression analysis it can be observed that the combination of w, LL, LI, PI and OMC gives better prediction of Su with Adjusted R^2 value of 0.809. This means the developed equation can be explain, 80.9% variation of Su depends on the variation of the predictors. The outcome of this thesis may be applicable in different civil Engineering sectors, especially for preliminary investigations and prefeasibility study of Civil Engineering works such as Building, Earthdams, Earthfills, and other works that involve soil.

Keywords: *Undrained shear strength, Liquid Limit, Plastic Limit, Plasticity Index, Liquidity index, Optimum moisture content and Maximum Dry Density.*

CHAPTER-1

INTRODUCTION

1.1 Background

Empirical correlations are widely used in geotechnical engineering practice as a tool to estimate the engineering properties of soils. The inherent nature and diversity of geological processes involved in the soil formation are majorly responsible for the wide variety of soil states. Innovative approaches are needed to predict the soil behavior from simple properties that are obtained in routine soil investigations. These approaches help engineers to verify the test results independently and to use design parameters in preliminary calculations.

Useful correlations exist between the index properties obtained from simple routine testing and the strength and deformations properties of cohesive soils among others. For practical purposes the results of routine index tests and correlations can be used as a first approximation of the soil parameters for use in preliminary design of geotechnical structures, and later as a mean to validate the results of laboratory tests.

Accurate measurement of shear strength parameters, coefficient of consolidation, and compressibility can be difficult, time consuming and costly. As a result of this there is now a tendency in countries all over the world towards building up correlation equations between the above soil properties and the so-called soil indices in order to speed-up the design process. This is most pertinent in third world countries where up-to-date testing equipment are lacking together with the trained manpower needed to operate them. **[1]**

One of the most common index properties of fine-grained soils is the undrained shear strength (Su), which differs for natural and remolded states. The undrained shear strength of remolded soils is important in many applications, including submarine investigations of offshore structures, pile design and studies of glacial soils.

This thesis is aiming to provide a conservative correlation between the undrained shear strength and index properties of Red clay soil. This soil is formed due to weathering of igneous and metamorphic rocks. It is highly porous and less fertile but where it is deep it is fertile, less crystalline and red in colour due to the presence of iron in it. This soil somewhat difficult to differentiate from Laterite soil. But lateritic soil is formed by the leaching process in the heavy rainfall areas of tropical areas and it is red in colour due to little clay and much gravel of red sand-stones.

1.2 Statement of the problem

As the Engineering behavior of soils vary from place to place and even with time, accurate prediction of parameters that properly characterize it depends on how much representative samples in terms of both space and time are gathered. Though various attempts have been made to predict the Su value by different researchers from samples of their locality, adopting those developed prediction methods without adjustment leads us to misinterpretation of soil behavior. On the other hand, the disadvantage of an empirical method is that it can be applied only to a given set of environmental, material, and loading conditions. If these conditions are changed, the design is no longer valid, and a new method must be developed through trial and error to be conformant to the new conditions. Therefore, this paper is intended to fill this gap and to minimize the time required to conduct the unconsolidated undrained triaxial shear test by predicting the Su value from the index properties of Red clay soils of the study area.

1.3 Objective of the Study

1.3.1 General Objective

- The main objective of the study is predicting the undrained shear strength from index properties of Red clay soil of Bule Hora town.

1.3.2 Specific Objectives

- Perform Laboratory soil testing and sampling at the selected study areas of red clay soil;
- To develop appropriate empirical correlations among the corresponding soil parameters; and
- To examine the validity of the correlations, and to draw appropriate conclusions from the relationships of each empirical equation.

1.4 Scope of the Study

The subject study is desired to conduct a localized research particularly on samples recovered from around Bule Hora town. In order to conduct the proposed correlation eighteen laboratory test results are used in this research work.

With regard to the regression analysis, depending on the trends of the scattering of test results the correlation is analyzed using a linear regression model. The required correlation is carried out by applying a single linear regression model and multiple linear regression models with the aid of SPSS Software. Furthermore, the scope of the developed correlation is limited to the test procedures followed in the subject research work.

1.5 Organization of the Study

The thesis is organized and presented under six Chapters. The first Chapter highlights introduction of the subject study. Chapter two deals with review of published literature. In Chapter three, discussions Field investigation and soil sampling were made. In Chapter four, Laboratory soils testing were conducted and Chapter five focuses on Data Analysis, Result and Discussion. Under Chapter six, the conclusion and recommendation were presented. At the end, details of the regression and laboratory test results enclosed under appendix section. Organization of the thesis work is presented with a flow chart as follow:

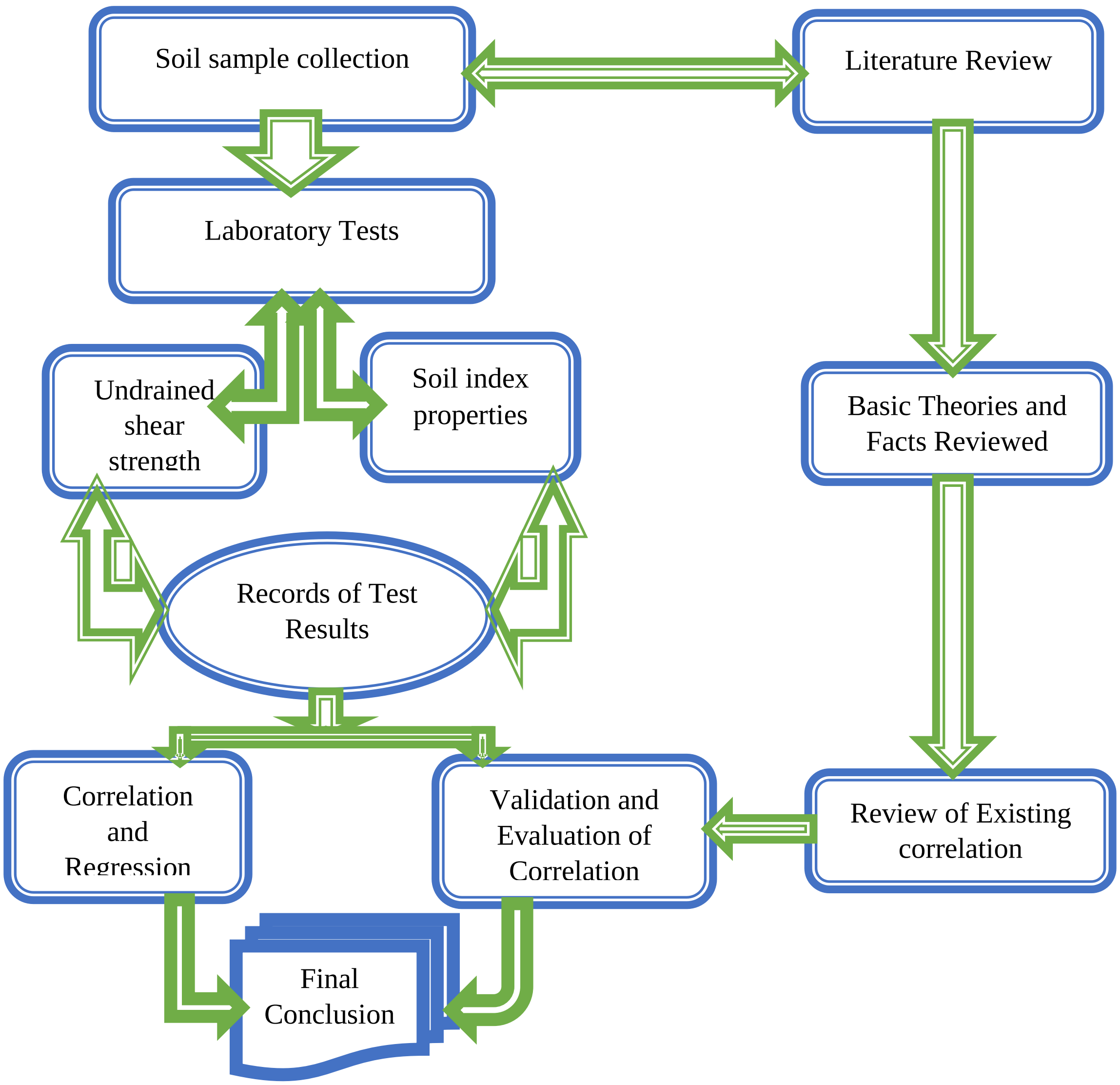


Figure 1.1: Flow Chart of the Study

CHAPTER-2

LITERATURE REVIEW

2.1 Shear strength of soil

2.1.1 General

One of the most important engineering properties of the soil is its shearing strength. This is because all stability analyses in the field of geotechnical engineering, whether they relate to foundation, slopes of cuts or earth dams, involve a basic knowledge of this engineering property of the soil. Shearing strength is defined as the resistance to shearing stresses and a consequent tendency for shear deformation. Shearing strength of a soil is the most difficult to comprehend in view of the multitude of factors known to affect it. A lot of maturity and skill may be required on the part of the engineer in interpreting the results of the laboratory tests for application to the conditions in the field.

Basically speaking, a soil derives its shearing strength from the following:

- Resistance due to the interlocking of particles.
- Frictional resistance between the individual soil grains, which may be sliding friction, rolling friction, or both.
- Adhesion between soil particles or 'cohesion'.

Granular soils of sands may derive their shear strength from the first two sources, while cohesive soils or clays may derive their shear strength from the second and third sources. Highly plastic clays, however, may exhibit the third source alone for their shearing strength. Most natural soil deposits are partly cohesive and partly granular and as such, may fall into the second of the three categories just mentioned, from the point of view of shearing strength.

The fundamental shear strength equation proposed by the French engineer Coulomb (1776) is **[2]**

$$\boxed{s = c + \sigma tan\emptyset \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (2.1)}$$

$$\boxed{\text{Su= c, For undrained shear strength}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. (2.2)}$$

In Coulomb's equation C and $\emptyset$ are empirical parameters, the values of which for any soil depend upon several factors; the most important of these are:

i. The past history of the soil.
ii. The initial state of the soil, i.e., whether it is saturated or unsaturated.
iii. The permeability characteristics of the soil.
iv. The conditions of drainage allowed to take place during the test.

2.2. Properties of Red clay soils

Clays are the plastic fines. Thus, they plot above the `A' line on the plasticity chart. They have low resistance to deformation when wet, but become hard cohesive masses when they dry. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristics of clays. The higher the liquid limit of a clay, the more compressible it will be, and hence, in the most cases the liquid limit is used to distinguish between clays of high compressibility (H) and those of low compressibility (L) **[3].**

In general, the higher the liquid limit and thus the plasticity index the more cohesive is the clay. Field differentiation among clays is accomplished by the toughness test in which the moist soil is molded and rolled into threads until crumbling occurs and by the dry strength test which measures the resistance of the clay to breaking and pulverizing **[3].**

- **Shearing characteristics of clays**

The understanding of the fundamentals of shearing strength is much more important in the case of cohesive soils or clays in view of their troublesome nature with regard to stability. In fact, the most complex physical property of clays is the shearing strength, as it is dependent on a multitude of inter-related factors. One of the most difficult tasks is to interpret results of laboratory shearing strength tests to the shearing strength of natural clay deposits.

- Source and Nature of Shearing Strength of Clays
  - Cohesion

This is a characteristic of true clay. This is sometimes referred to as no-load shear strength and is responsible for the strength of unconfined specimens. Cohesion in clays is a property which varies considerably with consistency. Cohesion therefore varies with both the type of clay and condition of clay. It is a kind of surface attraction among particles.

- Adhesion

Whereas cohesion is the mutual attraction of two different parts of a clay mass to each other, clay often also exhibits the property of ‘adhesion’, which is a propensity to adhere to other materials at a common surface. This has no relation to normal pressure. This is of particular interest in relation to the supporting capacity of friction piling in clays and to the lateral pressures on retaining walls.

- Viscous friction

Solid friction effects are of relatively minor importance and the effects of viscous friction are quite pronounced. The laws of viscous friction are, in general, opposite to those of solid friction. The total frictional resistance is independent of normal force, but varies directly with the contact area. It varies with some power of the relative velocity of adjacent layers of fluid or with the rate of shearing. The well-established fact that the strength of saturated clays varies with consistency also is in accord with the concept that strength is due to viscous rather than solid friction.

- Tensile strength

In varying degrees and for different periods, many types of clay are capable of developing a certain amount of tensile strength. This may affect the magnitude of normal stresses on failure planes.

.

2.3 Laboratory test method

2.3.1 Shear strength test

The triaxial shear test is one of the most reliable methods available for determining the shear strength parameters. It is widely used for both research and conventional testing. The test is considered reliable for the following reasons:

- It provides information on the stress–strain behavior of the soil that the direct shear test does not.
- It provides more uniform stress conditions than the direct shear test does with its stress concentration along the failure plane.
- It provides more flexibility in terms of loading path.

In the triaxial shear test, a soil specimen about 36 mm in diameter and 76 mm long is generally used. The specimen is encased by a thin rubber membrane and placed inside a plastic cylindrical chamber that is usually filled with water or glycerine. The specimen is subjected to a confining pressure by compression of the fluid in the chamber. (Note that air is sometimes used as a compression medium.) To cause shear failure in the specimen, axial stress is applied through a vertical loading ram (sometimes called deviator stress**) [4].** Stress is added in one of two ways:

- Application of dead weights or hydraulic pressure in equal increments until the specimen fails. (Axial deformation of the specimen resulting from the load applied through the ram is measured by a dial gauge.)
- Application of axial deformation at a constant rate by a geared or hydraulic loading press. This is a strain-controlled test. The axial load applied by the loading ram corresponding to a given axial deformation is measured by a proving ring or load cell attached to the ram.

2.3.1.1 Types of shear tests based on drainage conditions

A cohesive or fine-grained soil is usually tested in the saturated condition. Depending upon whether drainage is permitted before and during the test, shear tests on such saturated soils are classified as follows:

I. Unconsolidated-undrained test or undrained test (UU test)
II. Consolidated-drained test or drained test (CD test)
III. Consolidated-undrained test (CU test)

I. Unconsolidated Undrained (UU) Test

The purpose of a UU test is to determine the undrained shear strength of a saturated soil. The test consists of applying a cell pressure to the soil sample without drainage of porewater followed by increments of axial stress. The cell pressure is kept constant and the test is completed very quickly because in neither of the two stages—consolidation and shearing—is the excess porewater pressure allowed to drain. The undrained shear strength, s_u, is obtained from a UU test. The UU tests are useful in preliminary analyses for the design of slopes, foundations, retaining walls, excavations, and other earthworks **[5].**

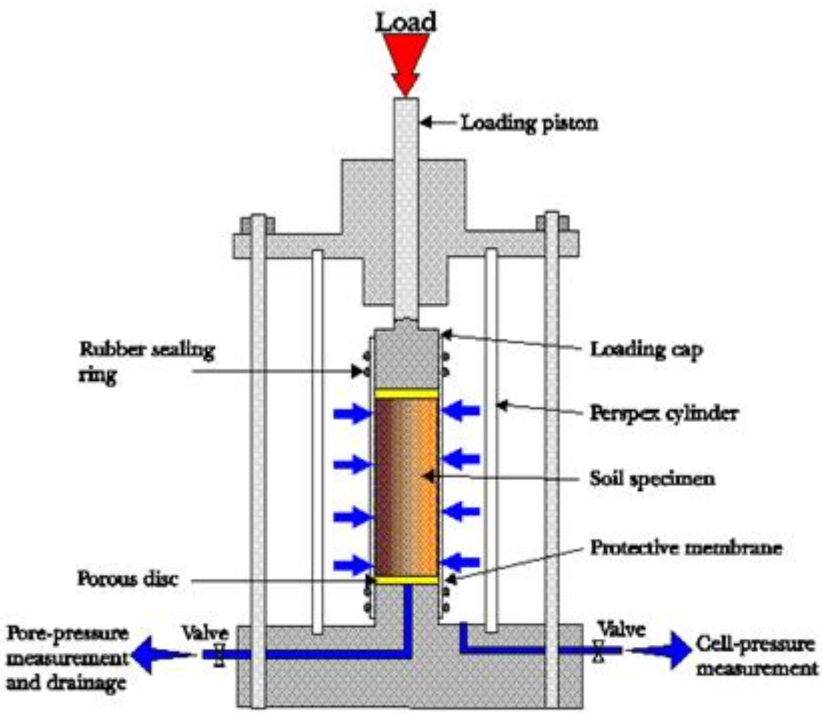


Figure 2.1 Triaxial Apparatus. **[5]**

S_u is the radius of the Mohr's Circle of Total Stress:

$$S_u = \frac{(\sigma_1)_{f-}\ (\sigma_3)_f}{2} = \frac{(\sigma'_1)_{f-}\ (\sigma_3)_f}{2}$$

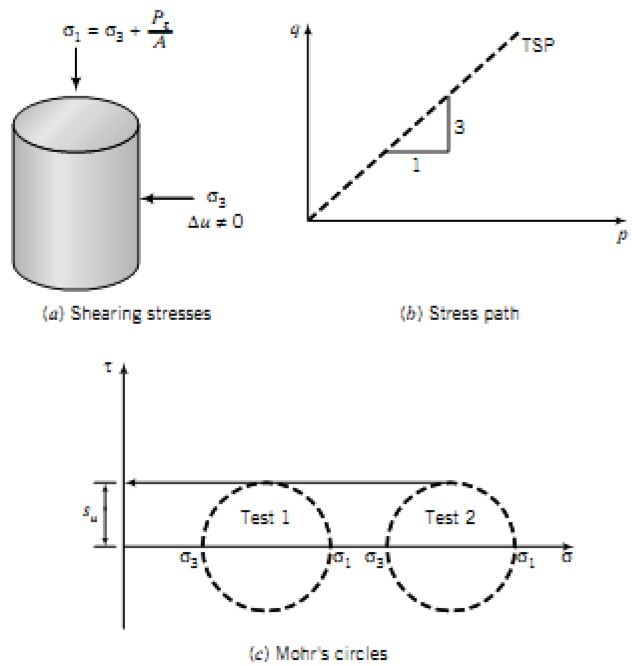


Figure 2.2 Stresses, stress path, and Mohr's circles for UU tests**. [5]**

II. consolidated drained (CD) test

Consolidated drained test is applicable to describing long-term loading response, providing strength parameters determined under effective stress control (i.e. ϕ' and c'). The test can however take a significant time to complete when using cohesive soil, given the shear rate must be slow enough to allow negligible pore water pressure changes.

III. consolidated undrained (CU) test

Consolidated undrained (CU) test is the most common triaxial procedure, as it allows strength parameters to be determined based on the effective stresses (i.e. ϕ' and c') whilst permitting a faster rate of shearing compared with the CD test. This is achieved by recording the excess pore pressure change within the specimen as shearing takes place.

2.3.2 Index Property Tests

In nature soil occurs in a large variety. Engineers are continually searching for simplified tests that will increase their knowledge of soils by employing a simple and rapid soil tests. These simplified tests which are indicative of the engineering properties of soils are called index properties **[6].** Index properties of cohesive soils are used to characterize the physical and mechanical behavior of soils by making use of parameters such as moisture content, specific gravity, particle size distribution, Atterberg limits and moisture-density relationships. Such parameters are useful to classify cohesive soils and provide correlations with engineering soil properties **[7].**

2.3.2.1 Grain Size Analysis

Grain-size analysis is a process in which the proportion of material of each grain size present in a given soil (grain-size distribution) is determined. The grain- size distribution of coarse-grained soils is determined directly by sieve analysis, while that of fine-grained soils is determined indirectly by hydrometer analysis. A sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve, Sieves are constructed of wire screens with square openings of standard sizes. The sieve analysis is performed on material retained on a U. S. Standard No. 200 sieve.

For coarse grained materials, the grain size distribution is determined by passing soil sample either by wet or dry shaken through a series of sieves placed in order of decreasing standard opening sizes and a pan at the bottom of the stock. Then the percent passing on each sieve is used for further identifying the distribution and gradation of different grain sizes **[8]**. Particle size analysis tests are carried out in accordance to ASTM D 422-63. Besides, the distribution of different soil particles in a given soil is determined by a sedimentation process using hydrometer test for soil passing 0.075mm sieve size. For a given cohesive soil having the same moisture content, as the percentage of finer material or clay content decreases the shear strength of the soil possibly increases.

2.3.2.2 Specific Gravity of Soils

The specific gravity of soil, G_s, is defined as the ratio of the mass in air of a given volume of soil particles to the mass in air of an equal volume of gas free distilled water at a stated temperature (typically 68°F{20°C}). The specific gravity is determined by means of a calibrated pycnometer, by which the mass and temperature of a deaired soil/distilled water sample is measured. Tests shall be performed in accordance with ASTM D 854 (AASHTO T 100). This method is used for soil samples composed of particles less than the No. 4 sieve (4.75 mm). For particles larger than this sieve, use the procedures for Specific Gravity and Absorption of Coarse Aggregate (ASTM C 127 or AASHTO T 85).

The specific gravity of soils is needed to relate a weight of soil to its volume, and it is used in the computations of other laboratory tests.

2.2.2.2 Moisture Content

Change in moisture content is the most influential parameter that affects the property of soils. Moisture content is defined as the ratio expressed as a percentage of mass of water to mass of soil solids. The moisture content test is carried out in laboratory as per the procedures of AASHTO T 265 or ASTM D 2216 and in field according to AASHTO T 217.

2.3.2.3 Atterberg Limits

Based on their mode of formation and mineralogical composition different soils respond differently for the same moisture content. Albert Atterberg, a Swedish Scientist in 1911 gave an idea of the consistency limit of cohesive soils and proposed a number of tests for defining their properties. The three Atterberg limits which are liquid limit, plastic limit and shrinkage limits are the boundary between each of the two consecutive states of the soil-water phases. Their test is performed only on that portion of a soil which passes the 425mm (No. 40) sieve **[9].** A description of phases of soil-water system is shown with schematic diagram in Figure 2.3.

Liquid Limit: The liquid limit of a soil is defined as the water content above which the soil behaves as a viscous liquid with little measurable shear strength. The liquid limit corresponds approximately to a water content at which the soil has shear strength of about 2.5 kPa**.** However, subsequent studies have indicated that the liquid limit for all fine-grained soils corresponds to shearing resistance of about 1.7-2.0 kPa **[10].** In other way, Liquid Limit is the water content, expressed as a percentage of the weight of oven-dried soil at which two halves of a soil pat separated by a groove of standard dimensions will close at the bottom of the groove along a distance of 13cm. under the impact of 25 blows in a standard liquid limit device. The liquid limit of a soil highly depends upon the clay mineral present. The conventional liquid limit test is carried out in accordance of test procedures of AASHTO T 89 or ASTM D 4318. A soil containing high water content is in the liquid state and it offers no shearing resistance.

Plastic Limit: The plastic limit is determined by rolling a small clay sample into threads and finding the water content at which threads approximately 3 mm in diameter will just start to crumble (Figure 2.6). Two or more determinations are made, and the average water content is reported as the plastic limit. The conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318. The soil in the plastic state can be remolded into different shapes. When the water content is reduced the plasticity of the soil decreases changing into semisolid state and it cracks when remolded.

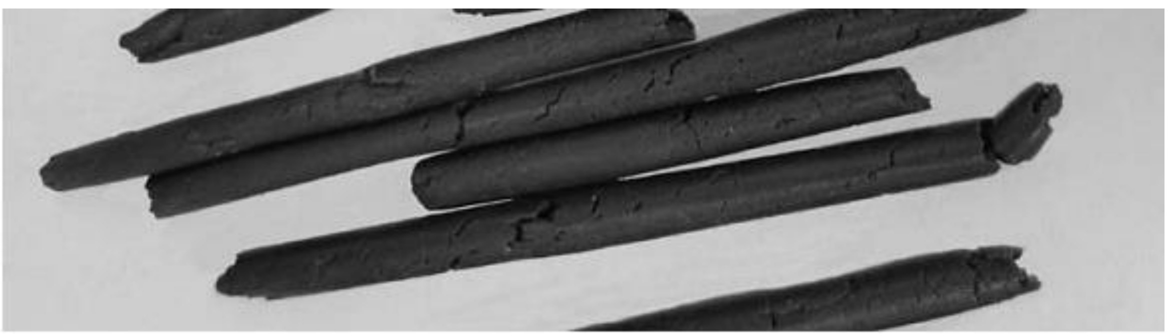


Figure 2.3 Soil at plastic limit**. [10]**

Shrinkage Limit: Is defined as the water content in which the soil changes from semi-solid state to solid state. In other word, it is the moisture content below which no soil volume change occurs with further reduction of water content. The test is carried out as per the procedure of AASHTO T 92 or ASTM D 427.

The amount of water which must be added to change a soil from its plastic limit to liquid limit is an indication of the plasticity of the soil. The degree of plasticity is measured by the plasticity index (PI), which is the numerical difference between liquid limit and plastic limit (PI=LL – PL). The greater the plasticity index means that the soil is more plastic, compressible and the greater volume change characteristic of the soil.

The liquid limit, plastic limit and plasticity index parameters are an integral part of several engineering properties and characterize the fine grained fractions of construction materials. The shear strength of a fine grained soil mainly depends on the consistency of the soil. As shown in

Figure 2.3, the shear strength of fine grained soil increases as the moisture content in the soil decreases from the liquid state up to the solid state.

2.3.2.4 Compaction

General

Compaction, in general, is the densification of soil by removal of air, which requires mechanical energy. The degree of compaction of a soil is measured in terms of its dry unit weight. When water is added to the soil during compaction, it acts as a softening agent on the soil particles. The soil particles slip over each other and move into a densely packed position. The dry unit weight after compaction first increases as the moisture content increases (Figure 2.4) **[4].**
Note that at a moisture content $w = 0$, the moist unit weight (γ) is equal to the dry unit weight (γ_d), When the moisture content is gradually increased and the same compactive effort is used for compaction, the weight of the soil solids in a unit volume gradually increases.

Beyond a certain moisture content $w = w_2$, any increase in the moisture content tends to reduce the dry unit weight. This is because the water takes up the spaces that would have been occupied by the solid particles. The moisture content at which the maximum dry unit weight is attained is generally referred to as the optimum moisture content.

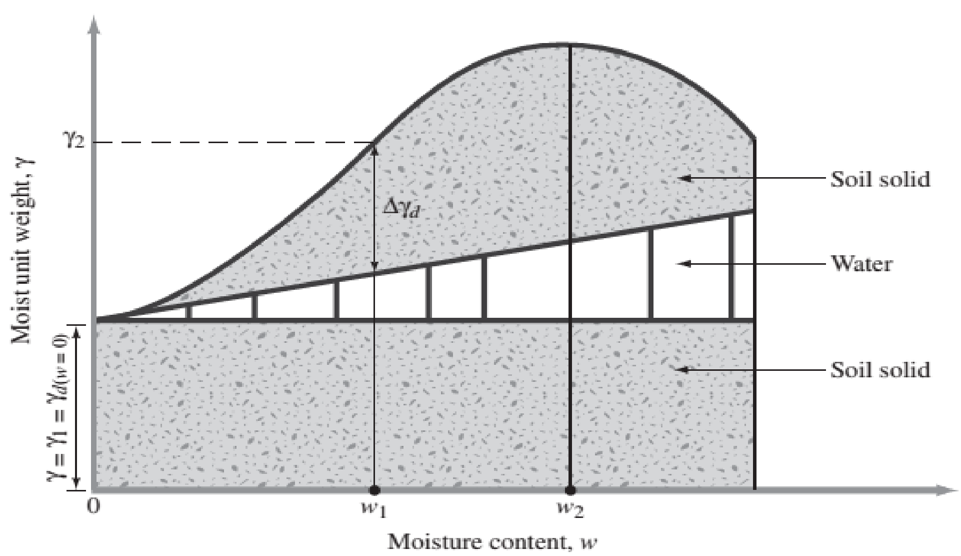


Figure 2.4: Principles of compaction **[4]**

Purpose of Compaction:

Soil compaction is widely used in geo-engineering and is important for the construction of roads, dams, landfills, airfields, foundations, hydraulic barriers, and ground improvements. Compaction is applied to the soil, with the purpose of finding optimum water content to maximize its dry density, and therefore, to decrease soil's compressibility, increase its shearing strength, and in some cases, to reduce its permeability. Proper compaction of materials ensures the durability and stability of earthen constructions.

At lower water content than the optimum the soil is rather stiff and has a lot of void spaces and hence, the dry density is low. On the other hand, at water content more than the optimum the additional water reduces the dry density as it occupies the space that might have been occupied by solid particles **[6].**

Factors affecting compaction:

Effect of Soil Type: The soil type—that is, grain-size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay minerals present—has a great influence on the maximum dry unit weight and optimum moisture content. Lee and Suedkamp (1972) studied compaction curves for 35 different soil samples. They observed four different types of compaction curves. These curves are shown in Figure 2.5. Type A compaction curves are the ones that have a single peak. This type of curve is generally found in soils that have a liquid limit between 30 and 70. Curve type B is a one and one-half peak curve, and curve type C is a double peak curve. Compaction curves of types B and C can be found in soils that have a liquid limit less than about 30. Compaction curves of type D are ones that do not have a definite peak. They are termed odd-shaped. Soils with a liquid limit greater than about 70 may exhibit compaction curves of type C or D. Soils that produce C- and D-type curves are not very common.

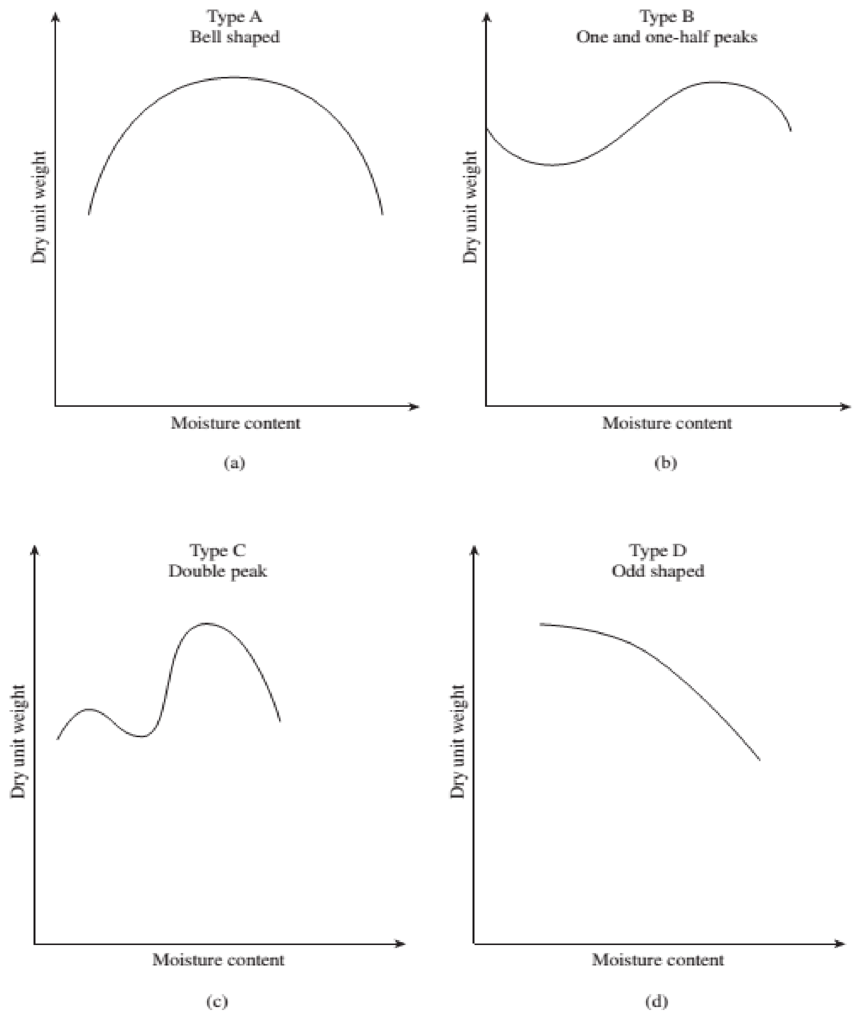


Figure.2.5 various types of compaction curves encountered in soils **[4]**

Compaction methods:

The laboratory standard proctor and modified proctor tests are performed as per (AASHTO T 99 or ASTM D 698) and (AASHTO T 180 or ASTM D 1557) respectively. The tests are performed on disturbed samples of soil particles passing sieve sizes 4.75mm or 19mm mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in three or five layers at 25 blows per layer in accordance with the specified nominal compaction energy of standard or modified proctor test respectively. Dry density is determined based on the moisture content and the unit weight of compacted soil. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content [8].

Grading and Atterberg limits alone are not sufficient to qualify the performance of construction materials since variation of moisture content and density play a considerable role. Different researches show that the moisture content and density conditions have a greater influence, on the value of shear strength of a soil, on coarser materials than fine grained materials.

2.4 Soil Classification

Soils exhibiting similar behavior can be grouped together to form a particular group under different standardized classification systems. A classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics. There are different classification devises such as USCS and AASHTO classification systems, which are used to specify a certain soil type that is best suitable for a specific application. These classification systems divide the soil into two groups: cohesive or fine-grained soils and cohesion-less or coarse-grained soils.

I. AASHTO Classification System

This system of soil classification was developed in 1929 as the Public Road Administration Classification System. It has undergone several revisions, with the present version proposed by the Committee on Classification of Materials for Subgrades and Granular Type Roads of the Highway Research Board in 1945 (ASTM Test Designation D-3282; AASHTO method M145).

According to this system, soil is classified into seven major groups: A-1 through A-7. Soils classified into groups A-1, A-2, and A-3 are granular materials, where 35% or less of the particles pass through the No. 200 sieve. Soils where more than35% pass through the No. 200 sieve are classified into groups A-4, A-5, A-6, and A-7. These are mostly silt and clay-type materials. The classification system is based on the following criteria:

i. Grain size

Gravel: fraction passing the 75 mm sieve and retained on the No. 10 (2 mm)U.S. sieve

Sand: fraction passing the No. 10 (2 mm) U.S. sieve and retained on the No. 200(0.075 mm) U.S. sieve

Silt and clay: fraction passing the No. 200 U.S. sieve

ii. **Plasticity**: The term silty is applied when the fine fractions of the soil have a plasticity index of 10 or less. The term clayey is applied when the fine fractions have a plasticity index of 11or more.

iii. If cobbles and boulders (size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample on which classification is made. However, the percentage of such material is recorded.

Figure 2.6 shows a plot of the range of the liquid limit and the plasticity index for soils which fall into groups A-2, A-4, A-5, A-6, and A-7.

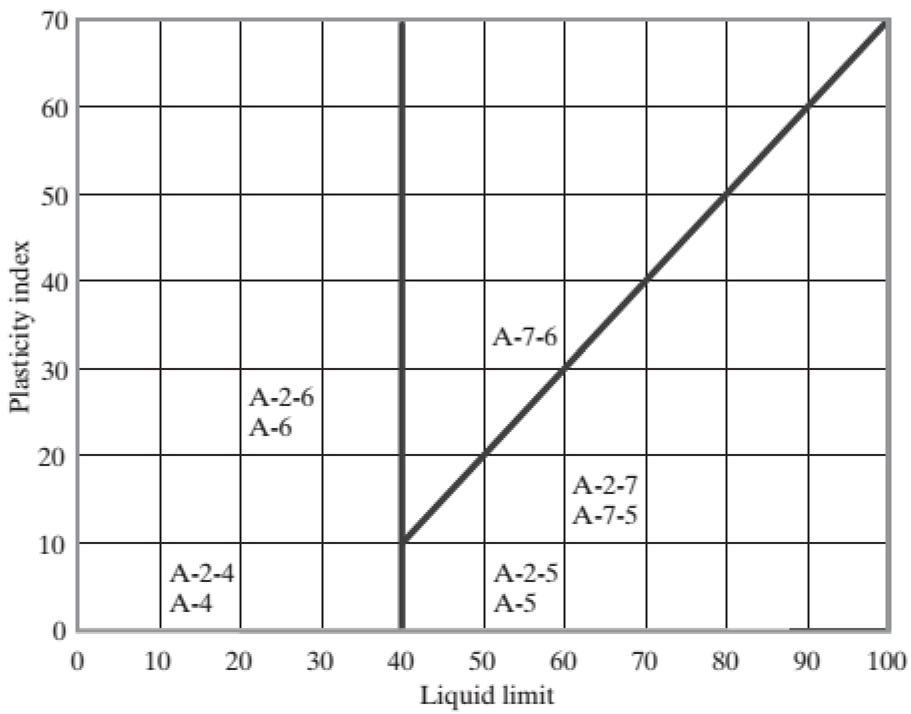


Figure 2.6 Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6, and A-7 **[8]**

Table 2.1 AASHTO Classifications for Fine-Grained Materials

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	*A-4*	*A-5*	*A-6*	*A-7* *A-7-5** *A-7-6*†
Sieve analysis (percent passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

*For A-7-5, $PI \leq LL - 30$
†For A-7-6, $PI > LL - 30$

II. Unified Soil Classification System

The original form of this system was proposed by Casagrande in 1942 for use in the airfield construction works undertaken by the Army Corps of Engineers during World War II. In cooperation with the U.S. Bureau of Reclamation, this system was revised in 1952. At present, it is widely used by engineers (ASTM Test Designation D–2487).The USCS is neither too elaborate nor too simplistic. The USCS uses symbols for the particle size groups. 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 a symbol, O, indicating the presence of organic material. A typical classification of CL means a clay soil with low plasticity, while SP means a poorly graded sand. The flowcharts shown in Figures 2.9 provide systematic means of classifying a soil according to the USCS.

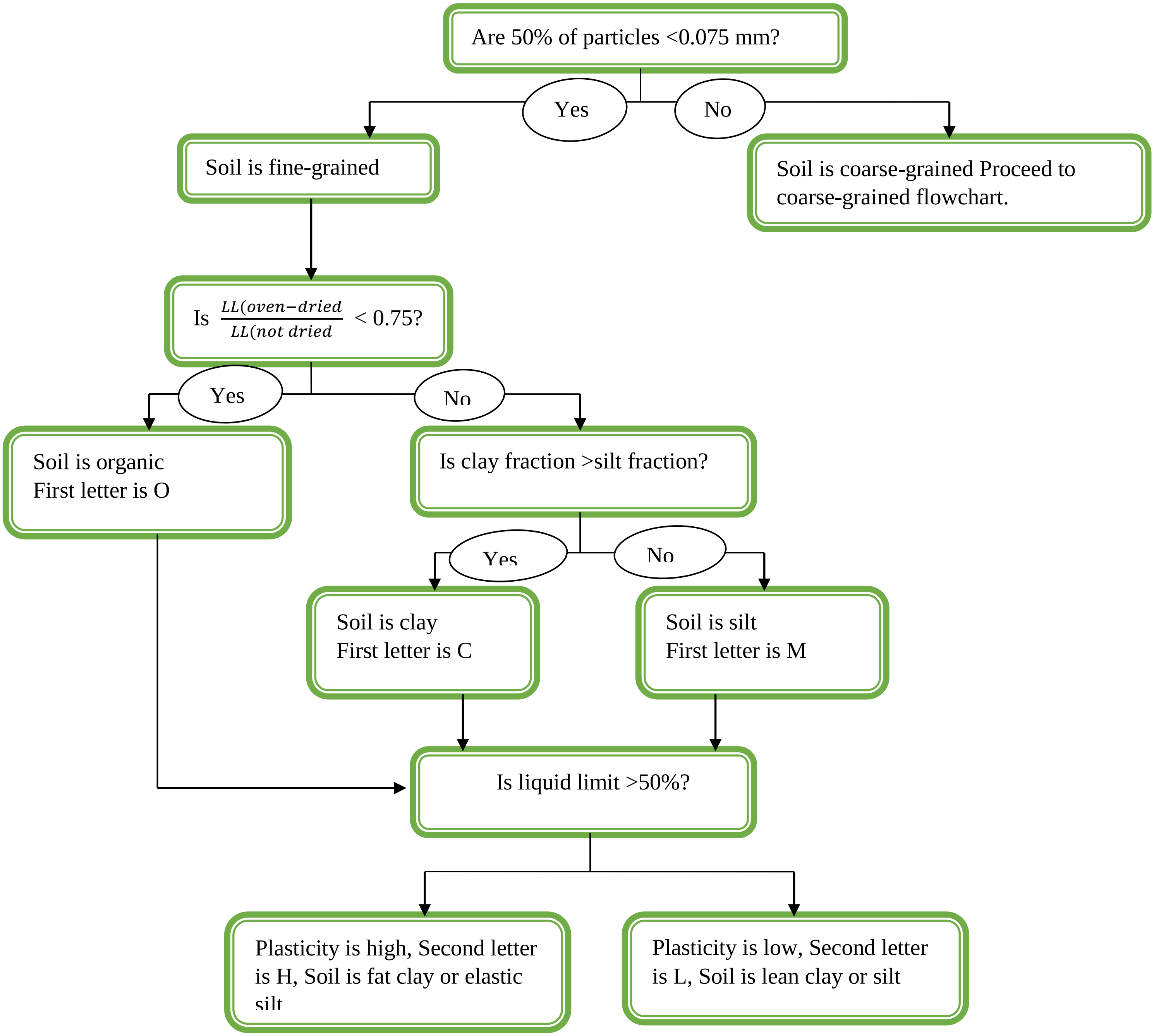


Figure 2.7 Unified Soil Classification System flowchart for fine-grained soils.

American Society for Test This system classifies soils into two broad categories:

I. Coarse-grained soils that are gravelly and sandy in nature with less than50% passing through the No. 200 sieve. The group symbols start with a prefix of either G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.

II. Fine-grained soils with 50% or more passing through the No. 200 sieve. The group symbols start with a prefix of M, which stands for inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils.

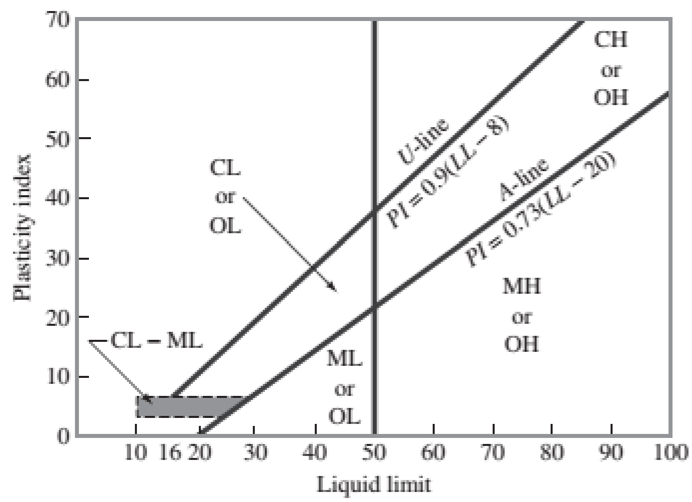


Figure 2.8 Plasticity chart **[8]**

2.5 Existing Correlations

Engineers and laboratory technicians have made numerous attempts to correlate the engineering properties of soils with their index properties since the early 1900's. Early attempts at correlation consisted of rather crude field methods developed for construction control. Researchers have been compiling and analyzing geotechnical data for many years to provide supporting evidences for new theories, develop new useful empirical correlations, or validate existing theories/relationships. Several different mathematical functions (or models) were applied to best represent the correlations existing among geotechnical data.

Correlations are very important to estimate engineering property of soils, especially for preliminary investigation of projects. Correlations may be also used for projects where there is financial limitation, lack of test equipments and limited time **[10].**

It should be noted that correlation between soil properties and the index properties had been initiated more than an half a century ago and some of these will now be mentioned.

Terzaghi and Peck had tabulated a correlation between the unconfined compressive strength of clays and their standard penetration resistances. But when the exhorbitant cost of the standard penetration test is considered no advantage is gained therefrom. For a truly cohesive soil the undrained shear strength is half the unconfined compressive strength.

2.5.1 Plasticity index and undrained shear strength

There exist in the technical literature a number of empirical correlations between S_u/σ_v' (the normalised undrained shear strength) and the Atterberg indices for normally consolidated clays, namely:

Skempton and Henkel presented a curve for the variation of $Su/\sigma_{v'}$ with plasticity index which was later approximated by the linear equation. **[8]**

$S_u/\sigma_{v'} = 0.11 + 0.37(PI/100)$…………………………. (2.3)

Bjerrum and Simons (1960) gave the following regression equations as fitting their experimental data best.

$Su/\sigma_v' = 0.45(PI/100)$ ………………………………… (2.4)

For PI > 50%; which had a deviation range (or scatter) of ± 25% or

Su $/\sigma_v$' = 0.18(LI/100) /………………………………….(2.5)

For LI > 50%; which had a deviation range of ±30%

Linear functions were used to represent the relationships between the plasticity index and the liquid limit in the plasticity chart (Casagrande 1932), between the plasticity index and % clay (Skempton 1953), Terzaghi et al. (1996) examined the relationship between the effective angle of friction and the plasticity index for a wide range of fine-grained soils and summarized the results by a nonlinear function. Between the plasticity index and % clay (Skempton 1953), between the specific discharge and the hydraulic gradient for clean sands in the laminar flow domain (Darcy 1856), and between the shear strength and normal stress in the Mohr-Coulomb failure criterion**. [8]**

Semi-log functions were relied upon to describe the relationships between the moisture content and the blows by the falling cup device (for the determination of liquid limit) and between the void ratio and effective stress for clays.

The major factor affecting the S_u is the moisture content. The variation of the S_u with the water content has been well-documented in the geotechnical literature. Numerous researchers attempted to relate the undrained strength of remolded fine-grained soils to the Atterberg limits. There has been an ongoing debate regarding the Su at the Atterberg limits resulting in two distinctive views. One claims that the undrained shear strengths at the plastic limit and liquid limit are fixed, and the ratio of the former to the latter is about 100. Another claims that the range of the variation of the Su at both the liquid and plastic limits is extremely large, and an assertion cannot be made that the Su has a unique value at either the LL or PL**. [10]**

The earliest assignment of the undrained shear strength to a consistency limit was done in 1939 by Casagrande, who suggested that the Su at the LL is 2.65 kPa. Skempton and Northey reported that the Su at the liquid limit ranged from 0.75 kPa to 1.75 kPa. Wroth and Wood proposed a mean value of 1.7 kPa for the cu at the LL and further assumed that the cu at the PL is 100 times higher than what it is at the LL. This 100-fold variation in the undrained shear strength has also been verified by other researchers. A great majority of researchers found that the cu at the LL ranges from 1–2 kPa. Regarding the Su at PL, most researchers have proposed that it is around 110-170 kPa and mostly towards the lower bound.

2.5.2 Liquid index and undrained shear strength

Many attempts have been made to obtain the correlation equations for fine-grained soils that relate undrained shear strength with index properties. One of author's who was studied the unique relationship between remolded undrained shear strength and the liquidity index, is Terzaghi et al., 1996. This plot includes soft clay soil and silt deposits obtained from different parts of the world**. [6]**

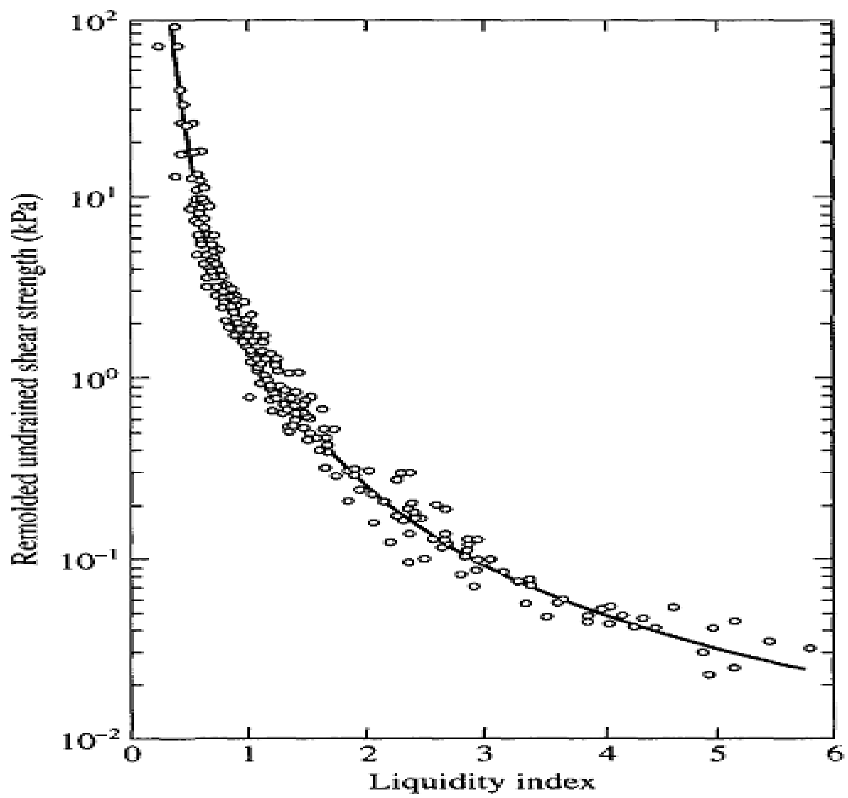


Figure 2.9 Relation between undrained shear strength and liquidity index of clays from around the world **[6]**

The other way of predicting undrained shear strength is by using simple laboratory tests like Atterberg limits. It is known that the liquid and plastic limits are moisture contents at which soil has specific values of undrained shear strength. It therefore follows that, for a remoulded soil, the shear strength depends on the value of the natural moisture content in relation to the liquid and plastic limit values. This can be conveniently expressed by using the concept of liquidity index.

Curves relating remoulded undrained shear strength to liquidity index $LI = \frac{w_n - \mathrm{pL}}{PI}$ have been established by Skempton and Northey (1952) as cited by [10] and these are given in Figure below.

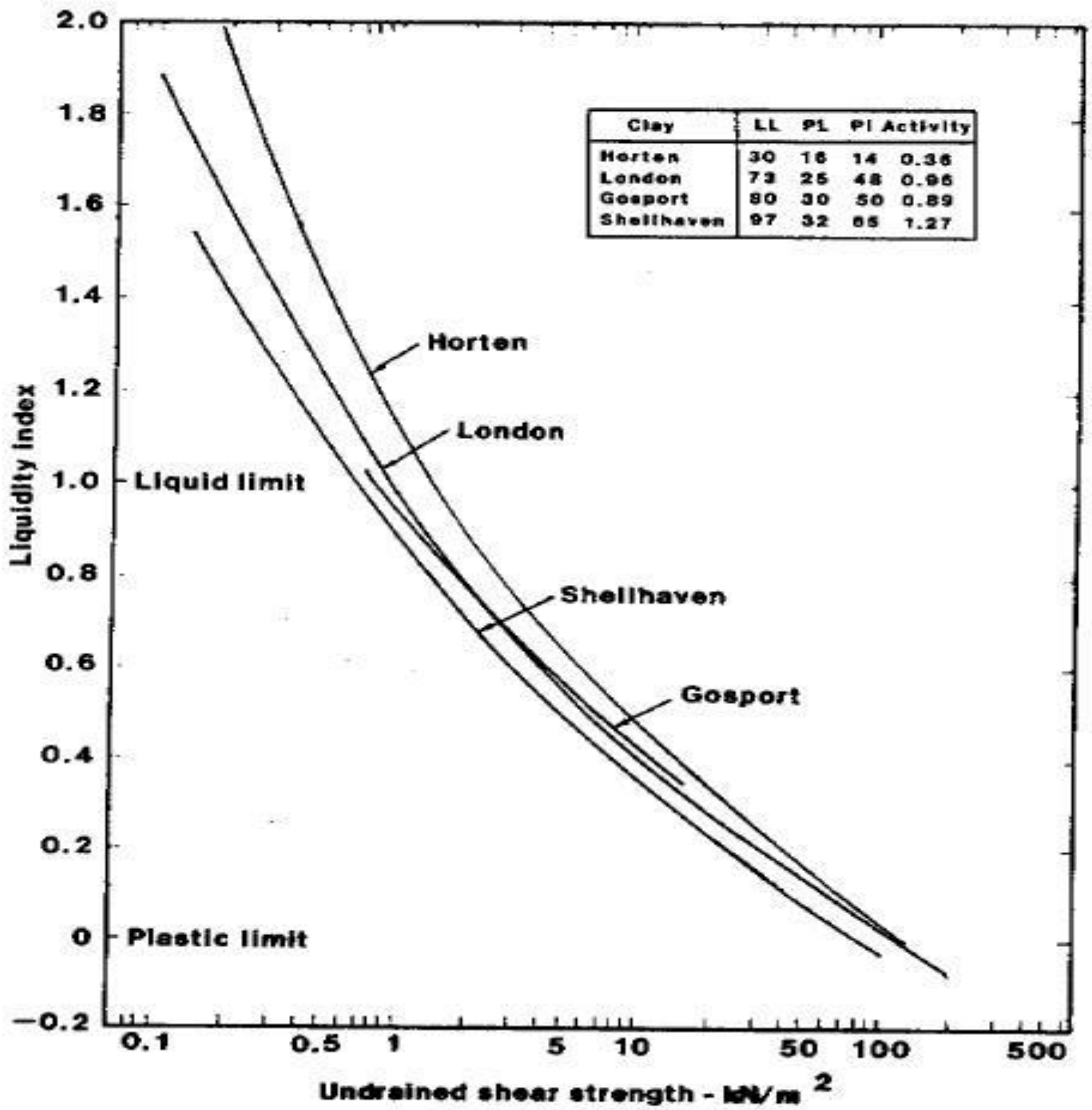


Figure 2-10 Correlation between shear strength and liquidity index established by Skempton and Northey (1952) **[8]**

Several authors have proposed relationships between strength and liquidity index, while not explicitly recognizing the band of strengths which exists in the data. Schofield & Wroth (1968) upon examination of vane shear test data from Skempton & Northey (1952) made the observation that ‘*from these data it appears that the liquid limit and plastic limit do correspond approximately to fixed strengths which are in the proposed ratio of 1:100’*. Based on this observation, Wroth & Wood, (1978) proposed equation (2.6): **[11]**

$$S_u = 170e^{-4.6I_L} = 1.7 * 10^{2(1-I_L)}\ Kpa \qquad (2.6)$$

This equation implies that the undrained shear strength of soil should be 1.7 kPa at the liquid limit and 170 kPa at the plastic limit. An alternative correlation (equation 2.7) was proposed by Leroueil et al. (1983):

$$S_u = \frac{1}{(I_L - 0.21)^2}\ Kpa \quad , 0.5 < I_L < 2.5 \qquad (2.7)$$

Whilst this equation fits the collected data presented in Leroueil et al (1983) well between liquidity indices of 0.5 and 2.5, it should obviously not be extrapolated below these values, as it predicts infinite strength at a liquidity index of 0.21. Locat & Demers (1988) suggested equation for computing strengths at high liquidity indices. This equation was said to be valid for the range 1.5 < I_L < 6.0

$$S_u = \left(\frac{1.167}{I_L}\right)^{2.44}\ \text{Kpa} \qquad (2.8)$$

CHAPTER-3

FIELD INVESTIGATION AND SOIL SAMPLING

3.1 Study area and sampling of soils

The study was conducted at Bule Hora town which is Capital of western Guji Zone Located in the southern part of Oromia Regional state and 465km from Addis Ababa. The town is capital of western Guji Zone. It is located at an elevation of 1,826 meters above sea level and its population amounts to 152,449 from this 79,369 male and 72,980 female.

Its coordinates are 5°37'60" N and 38°13'60" E in DMS (Degrees Minutes Seconds) or 5.63333 and 38.2333 (in decimal degrees). Its UTM position is DG12 and its Joint Operation Graphics reference is NB37-10.

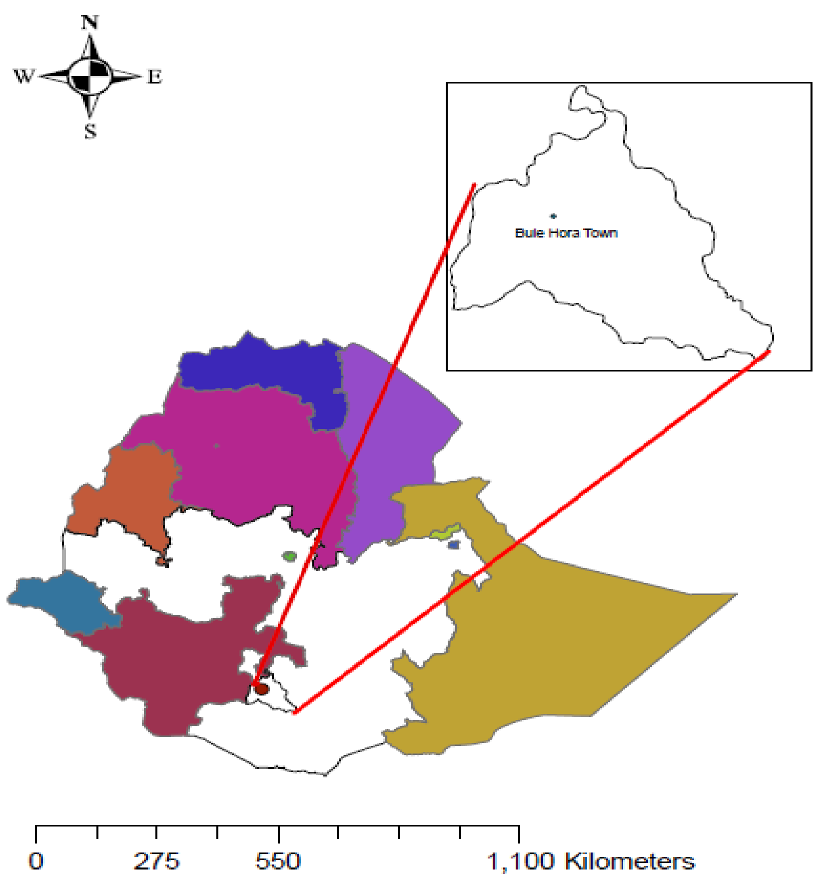


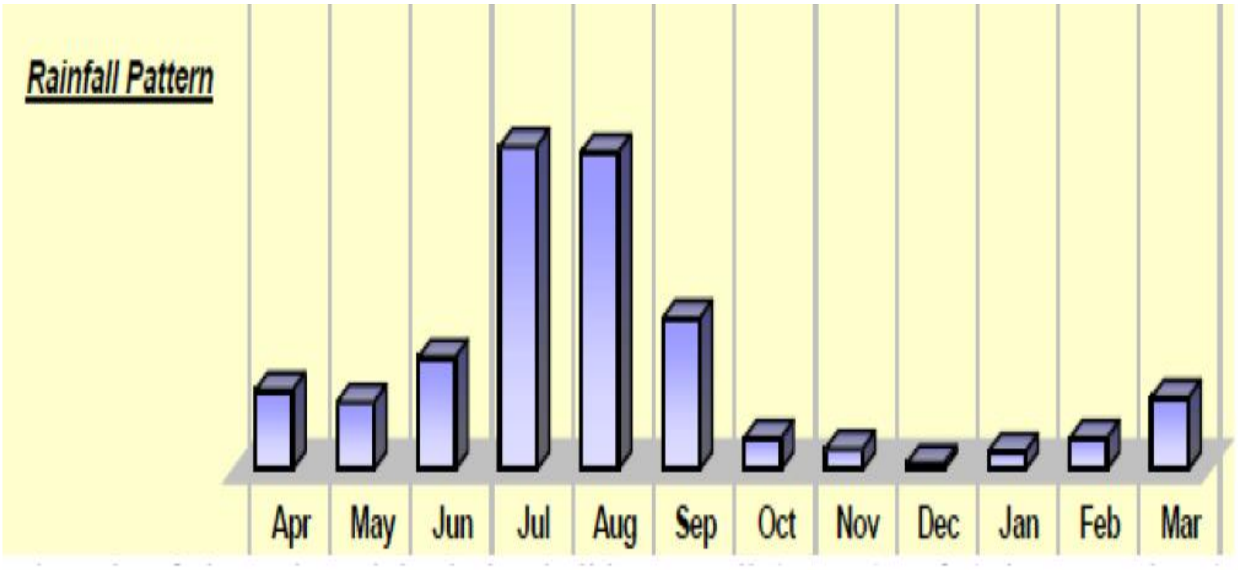
Figure 3.1 Study Area Location Map

Table 3.1 Sample Location.

S/N	Area of sampling	Northing (m)	Easting(m)	Altitude (m)
1	B/Hora Teachers college	620788.49	416085.228	1943.90
2	Kidiste Mariam school	623179.129	416162.087	1920.97
3	Abayi junior school	625042.146	416185.80	1851
4	Mekane Iyesus church	621809.303	416460.708	1912.90
5	B/Hora University*	622491.364	413556.166	1846.82
6	preparatory school	624463.45	415972.08	1860.56
7	Robi Gebeya	622276.693	415087.25	1869.30
8	Around Madiya	626134.32	416247.613	1904.03
9	Arsema	623954.29	414536.635	1796.20
10	Rhobot church	624930.885	415466.915	1863.90
11	Algi church	624189.68	415940.445	1850.45
12	Around Bule Hora Hotel	623934.48	415449.18	1885.30
13	B/Hora University**	623053.715	414139.201	1867.24
14	Tsahay Hora Hotel	623122.32	415579	1911
15	B/Hora Woreda Finance office	623149.367	415885.985	1911.40
16	Around Hospital	621685.96	415751.976	1932.86
17	Zone administration office	626216.58	416418.405	1908.04
18	B/Hora Woreda Court office	621566.608	415980.435	1908.39

3.2 Topography, climate and Agriculture

The rainfall is unimodal and so people depend only on the *meher* harvest. The land preparation commences from February and continues until the sowing time in May-June for the long cycle sorghum and June-August for short cycle crops. Harvesting continues for three months (October-December). In most cases, cattle births occur July-August and that of shots in February to April. Milking continues for five months, July-November. **[12]**



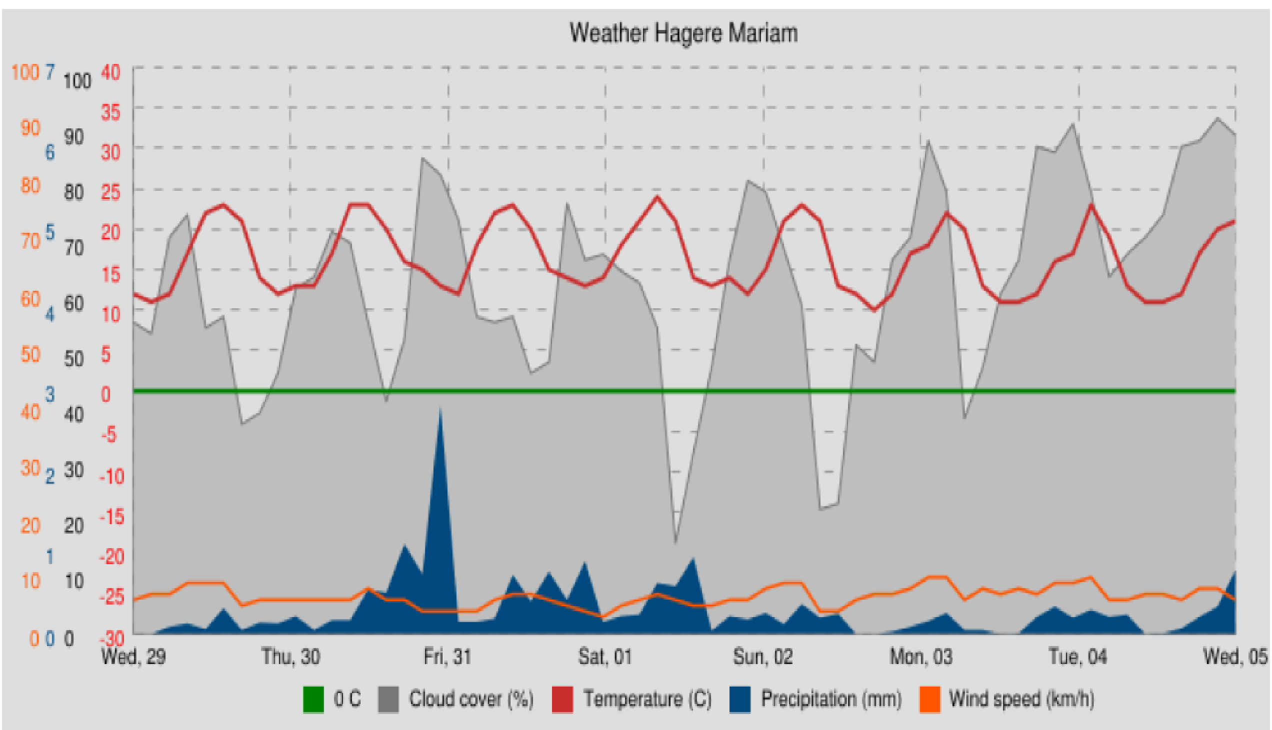


Figure 3.2 Bule Hora Rainfall pattern and weather

The mean seasonal maximum temperature of Bule Hora (Hagere Mariam) ranges from 15°C - 25°C and occurs during February - May (Belg).**[12]**

3.3 Visual identification of soils in the field

The identification of soils on the field was made based on procedures described in ASTM D 2488-00 (Standard practices for Description and identification of soils) standards. The first step used in identifying the type of soil was to determine whether it is fine or coarse grained soil based on the size, texture and distribution of soil particles. After the fine grained soils are identified, the simple and important test methods are used for detail identification of the soil types. These are:

Dry strength test: is a measure of the character and quality of colloidal fraction of soil. If the sample, which is 1/8-inch ball size mold, cannot be broken the soil is highly plastic and samples of low plasticity will break easily.

Dilatancy test: is observing rapidity of water rising to the surface when sample is shaken and when the sample is squeezed between fingers. Speed of appearance during shaking and disappearance assists in identifying the fines in the sample.

Plasticity (toughness) test: is the method of testing the plasticity of the soil. When water is added to the fines portion of the soil sample and the sample is then rolled into 3mm diameter thread.

3.4 Sampling methods and sample preservation.

Soil sampling is made from different locations of the town by taking about 50 kg of disturbed soils. To keep the natural moisture content as well as to avoid the exposition of soils from foreign materials like excessive heat and water, the bags which is used to hold the soil samples made to be water proof like plastic type.

For identification purpose the name of the sampling area, depth of sampling and color of the sample were written on the outside part of the bag.

The transportation of the soil samples was by vehicle to Geotechnical laboratory of Adama science and Technology University (ASTU). Eventhough it is long distance, the soil sample is not exposed to any obstacle.

3.5 Sample size and sample preparation

For correlation purpose the required number of sample size is large. But due to shortage of permeation time for using the laboratory as well as to get accurate results the soil sample size is decided to be eighteen.

Preparation of soil samples for the required tests starts by air drying. Before go to the next step the air dried soil samples were thoroughly pulverized by fingers and wood hammer.

For particle size distribution test representative 500gram air dried (passing no.4) and 50gram oven dried (passing no.200) sample are prepared for wet sieve and hydrometer analysis respectively according to ASTM D1140-97 (Standard Test Method for Amount of Material in the Soils Finer than the No. 200 (0.075mm) Sieve) procedure.

For atterberg limit tests about 200 gram airs dried (passing no.40) sample taken and soaked for hours by small amount of water which is used to moist the soil grains. In order to get precise results multipoint method is chosen and prepared to test according to ASTM D 4318-00 (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils) procedures.

For compaction test, method A of compaction is chosen because less than 20 % by mass of material is retained on the 4.75mm (no.4) sieve. About 16kg soil sample is prepared for five trails of each soil sample. The test is then proceeding according to ASTM 698-00 (Standard Test Methods for Laboratory Compaction Characteristics of Soil) procedures.

For Unconsolidated Undrained Triaxial test about 2.5kg airs dried passing 4.75mm (no.4) sieve soil sample is prepared from each sample in the Geotechnical laboratory.

CHAPTER 4

METHODOLOGY

4.1 Laboratory Soil Testing

4.1.1 Introduction

This chapter presents the methods of soil investigations to determine soil index properties and compaction characteristics of 18 soil samples. The laboratory testing program includes: determination of grain size, Atterberg limits, compaction characteristics, specific gravity and unconsolidated undrained triaxial test. Laboratory tests were performed at Adama Science and Technology University and Ethiopian Construction Design & Supervision Works Corporation Research, Laboratory and Training Center.

In this Chapter, details of test procedures to determine the Atterberg limits, compaction and undrained shear strength are presented. The validity of the correlations depends on the accuracy of the test procedures used. To improve the reliability of such correlations, careful understanding of test procedures and practice are required. In this case, attempt was made to obtain accurate laboratory results by careful following of proper testing procedures as described by ASTM standards.

4.1.2 Natural moisture content

Water content of a material is the ratio expressed as a percent of the mass of “pore” or “free” water in a given mass of material to the mass of the solid material.

Sample for natural moisture content was taken directly from test pit in air tight plastic bags. ASTM D 2216 - Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil was used to carry out the test.

4.1.3 Grain Size Distribution

Grain size analysis is a process in which the proportion of material of each grain size present in a given soil is determined. The grain- size distribution of coarse –grained soils is determined directly by sieve analysis, while that of fine-grained soils is determined indirectly by hydrometer analysis **[3].**

However, determination of amount of materials finer than No. 200(0.075mm) sieve can be separated from coarser particles much more efficiently and completely by wet sieving method than through the use of the dry sieving **[13].**

In this particular study, the wet sieving method is used in the laboratory to determine the fine content of the soil samples. Representative sample was obtained by using the quartering method, and representative 100gram of oven dried soils was taken, socked in water and it was washed over 0.075mm sieve. The amount of material passing 0.075mm during washing process was obtained and recorded. Finally the percentage of fine content was computed. The test was performed according to the procedure described by ASTM D1140-97, Standard Test Method for Amount of Material in the Soils Finer than the No. 200 (0.075mm) Sieve.

4.1.4 Specific Gravity

The specific gravity of selected samples was measured in accordance with ASTM D 854-98 (Standard Test Method for Specific Gravity of Soils). The mass of a clean dry pycnometer was obtained. The pycnometer was filled with clean water, weighed, and the temperature was measured. The water was then poured out until the flask was half full and oven-dried soil of a known mass was carefully placed in the pycnometer. The contents were allowed to be free from any trapped air. After removing all the air, distilled water was carefully added making sure that no air was reintroduced to the contents.

4.1.5 Atterberg limit

These test methods cover the determination of the liquid limit, plastic limit, and the plasticity index of soils. Taking 200gram of 0.425mm sieve passing air dried soil considerable water is added to moist the soil grains and placed for hours. From this about 20 gram of the mixed soil is set aside for determination of plastic limit. Water contents are determined by separating the blows number in to four parts from 15 up to 35.the soil mix at each blow number are taken and oven dried for 24 hours from which water contents are calculated. liquid limit was determined by locating the water content at 25 blows.

For plastic limit 1/3 of 20gram was taken and rolled into 3mm thread on glass plate. The thread is rolled until the soil crumbled when it reached 3mm diameter. At this stage the soil is taken and oven dried for 24 hours and water content is measured. Then the average value of the water contents from two trails taken as plastic limit. The tests are done according to ASTM D 4318-00 procedures or AASHTO T 90. The plastic index is obtained by subtracting the plastic limit from liquid limit.

4.1.6 Compaction

The standard laboratory compaction method is used to determine the relationship between water content and dry unit weight of soils (compaction curve).

A soil at selected water content was placed in three layers into a mold of given dimensions, with each layer compacted by 25 blows of 24.4kN rammer dropped from a distance of 305mm, subjecting the soil to a total compaction effort of about 600 kN/m2. The resulting dry unit weight is determined. The procedure was repeated for sufficient number of water contents to establish a relationship between the dry unit weight and the water content. The values of optimum water content and maximum dry unit weight were determined from the compaction curve. The test is conducted according to ASTM 698-00 or AASHTO T 99.

4.1.7 Unconsolidated Undrained triaxial test.

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 drainage conditions of the sample is generally the deciding factor in choosing a particular type of the test in the laboratory since the test specimen should conform as closely as possible to the conditions under which the soil will be stressed in the field. Since consolidation of clay soil take long period of time, UU tri-axial test have been selected and ASTM D 2850- 95 is used for determination of strength parameters and stress-strain relationships for the soils of the study area.

CHAPTER 5

DATA ANALYSIS, RESULTS AND DISCUSSIONS

5.1. Introduction

The purpose of correlation analysis is to measure and interpret the strength of a linear or nonlinear relationship between two continuous variables. Before obtaining the relations between two or more variables, valid data must be collected that may be obtaining from experiment (primary data). In this Chapter analysis have been done to develop possible relationships among the parameters.

To obtain a relation between two or more variables many methods can be used. But for this thesis two common methods are used. Namely, Scatter plot and Regression Analysis. Detail discussion and analysis of each method is presented below.

5.2. Data Analysis Methods

There are many methods that we can use to check the validity of the relationships between two or more variables **[14]**. However, in this study the two common methods are used, namely: scatter plot and linear regression analysis. Before the application of the analysis methods some important terms are discussed below.

1. Level of significance: The probability of making an error to reject a hypothesis while it happens to be true is called the level of significance. In practice it is customary to use 5% level of significance **[14]**. This means that we are 95% confident that we could make the right decision and we could wrong with probability of 5%.

2. One tailed and two tailed Tests: When a hypothesis is tested assuming that one process is better or worse than the other, then it is called one tailed or one sided test. However, if the hypothesis is tested assuming that the extreme values of the statistics score on both sides of the mean in both tails of the distribution, the tests are called two tailed or two sided tests.

3. Standard error: standard error is the average measure of error of each sample points about the best-fit line. Out of all curves, the best-fit curve has the smallest standard error.

4. Correlation coefficient(R): the coefficient of correlation (sometimes called coefficient of regression) is the measures of how well the least-square regression line (best fit line) fits the sample data. Value of $R = 1\ or - 1$ (R^2 =1) shows that there is a perfect linear correlation and also perfect linear regression. On the other hand R = 0 or approaches to zero shows no valid relationship can be obtained between the variables **[14].**

5.3 Scatter plot and Best-Fit curve

Scatter plots of the variables are done by using SPSS software. This software is chosen because it is found to be the most powerful and manageable tool for scatter plot analysis and determination of correlation between two variables.

According to aim of the thesis, the scatter plots are done to express the degree of correlation strength of the dependent variable Su by each independent variable of LL, PL, PI, OMC, w, LI and MDD for all collected data. The figures of scatter plot are shown below.

5.3.1 Scatter plot for primary data

The scatter plots of the primary data are done by collecting the results of the laboratory tests of 18 soil samples. These are presented on fig 5.1-5.6 as shown below.

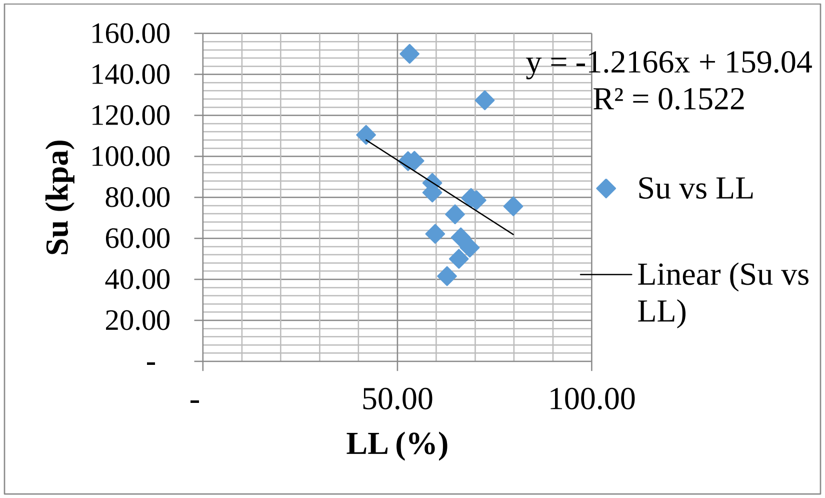
Specific to this research, a statistical package for social science software (SPSS) is employed to investigate the significance of individual regressor variables. Accordingly, the eighteen laboratory test results of the independent and dependent variables are used in the following regression analysis from this one is considered as control Point. The statistical information's of the test results are presented in Table 5.1:

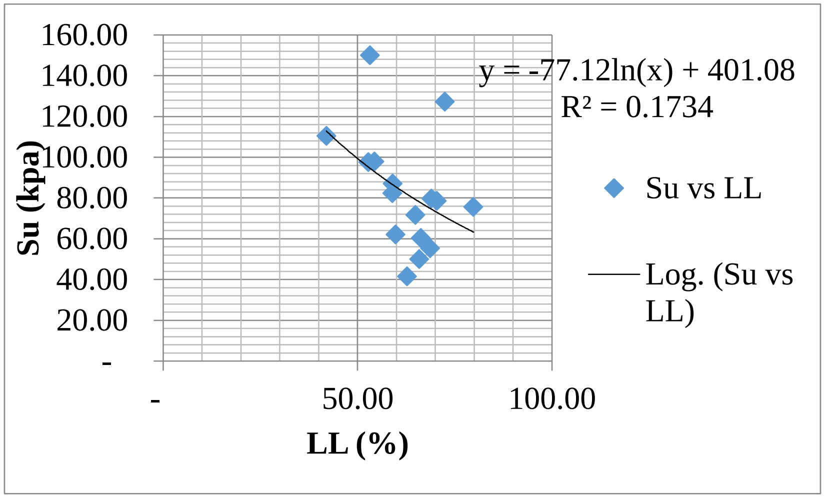
Table 5.1: Statistical Information of Dependent and Independent Variables

Variable Type	Variable Name	Unit of Measurement	No. of Samples	Ranges		Mean	Standard Deviation
				Min.	Max.		
Dependent	**Su**	Kpa	16	41.52	150.00	82.947	28.741
Independent	**LL**	%	16	42.00	79.84	62.547	9.216
	PL	%	16	16.97	38.34	24.885	5.678
	PI	%	16	23.25	48.57	37.661	7.355
	OMC	%	16	22.00	33.00	26.671	3.140
	MDD	g/cc	16	1.22	1.59	1.385	.1075
	LI	No	16	.13	.71	.3975	.1764
	w	%	16	24	52	40.156	8.058

5.3.1.2 Scatter plot and best-fit curve for Undrained shear strength and index properties.

A. Relation between LL and Su





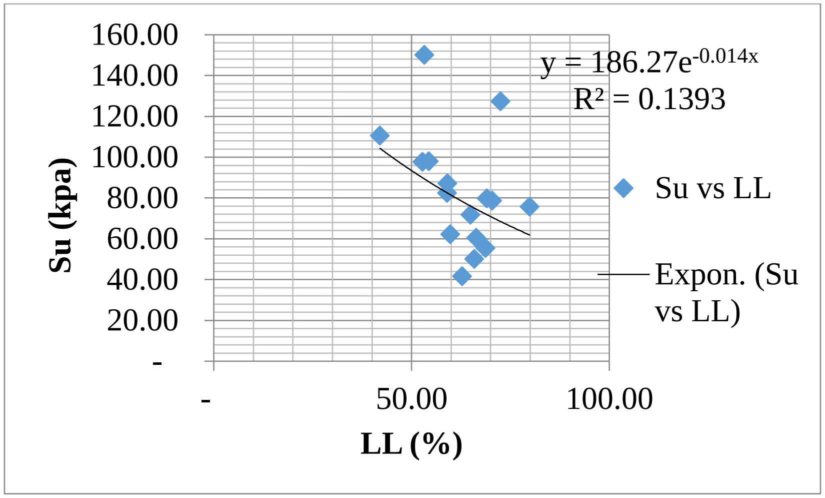
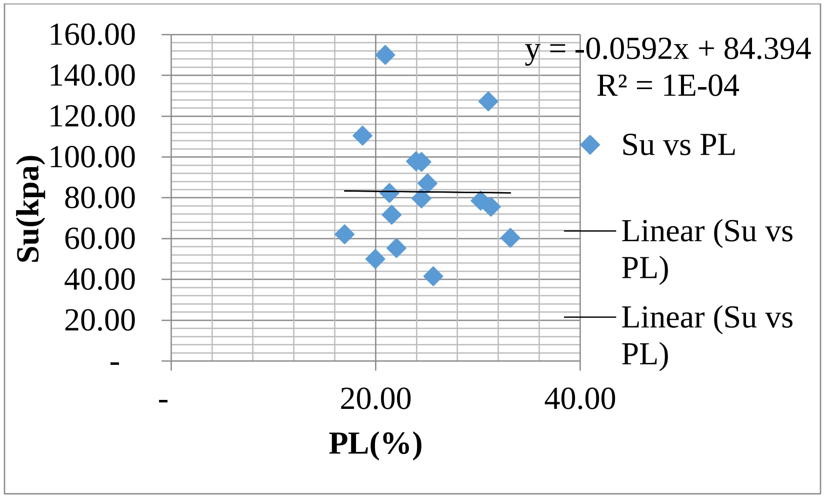
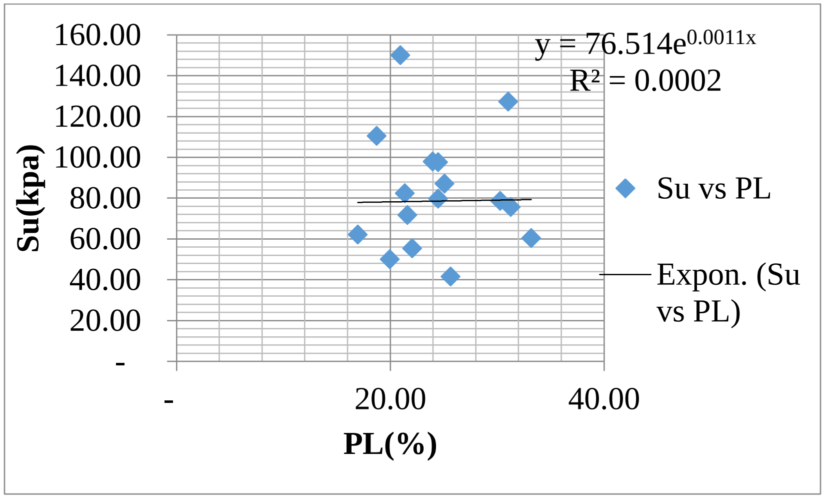


Figure 5.1 Scatter plot and best-fit curve of LL and Su using Linear, Exponential and Logarithmic relation.

B. Relation between PL and Su





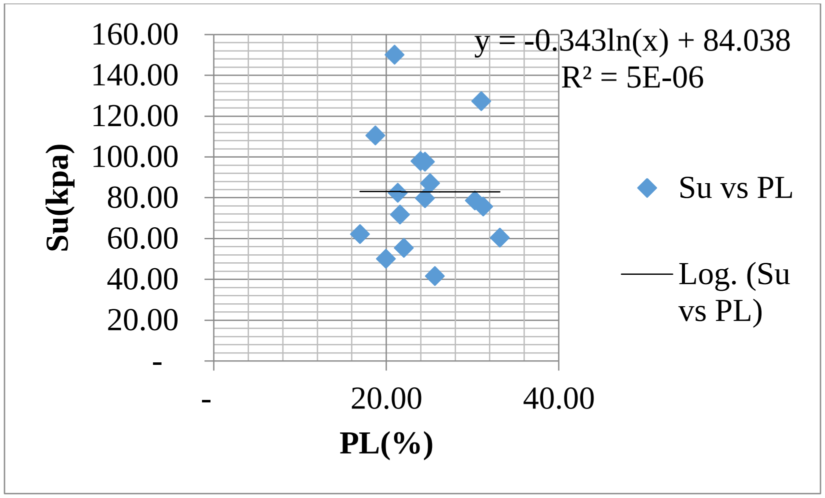
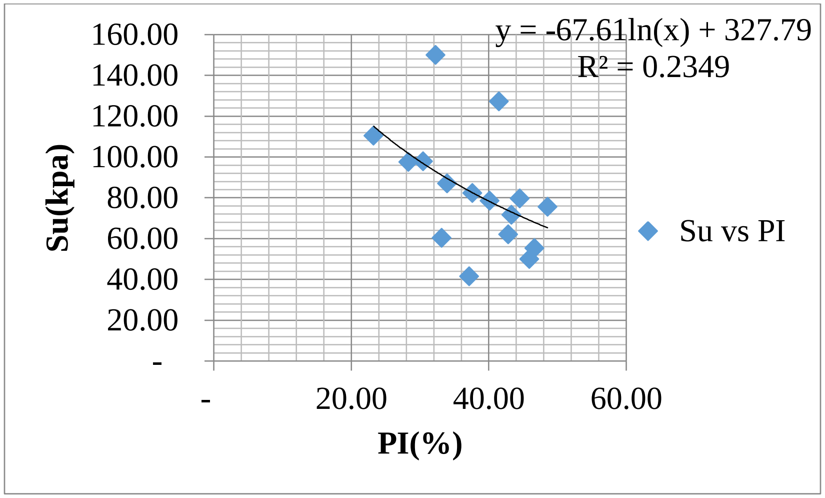
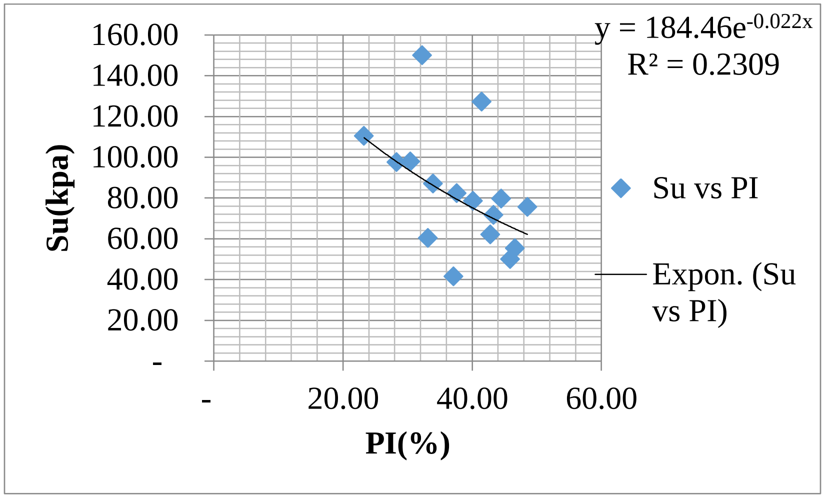


Figure 5.2 Scatter plot and best-fit curve of PL and Su using Linear, Exponential and Logarithmic relation.

C. Relation between PI and Su





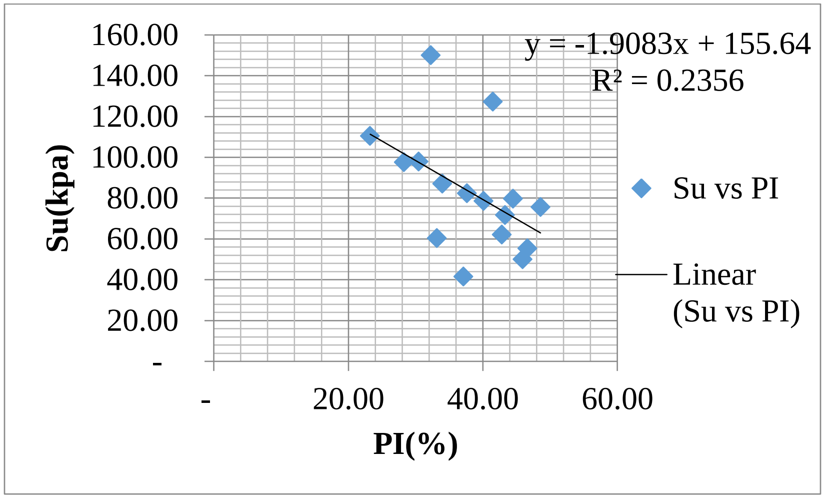
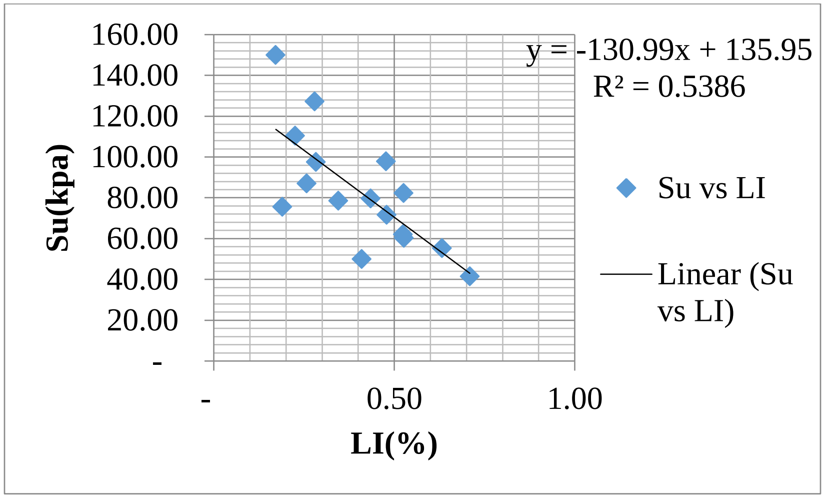
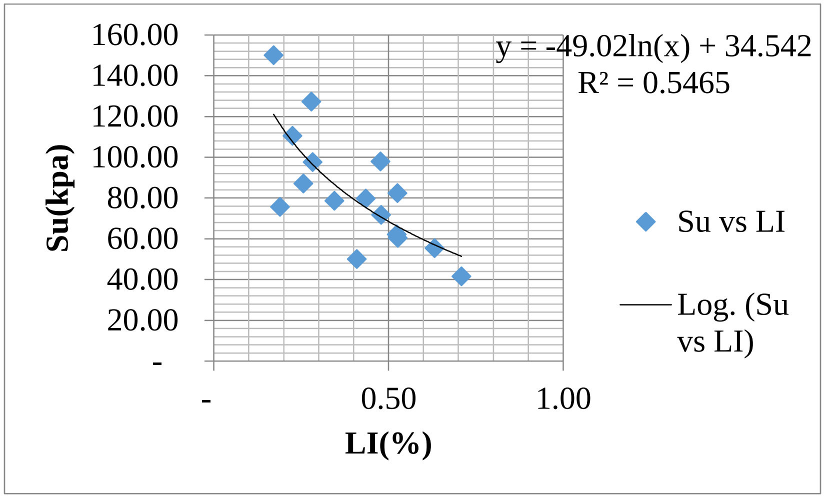


Figure 5.3 Scatter plot and best-fit curve of PI and Su using Linear, Exponential and Logarithmic relation.

D. Relation between LI and Su





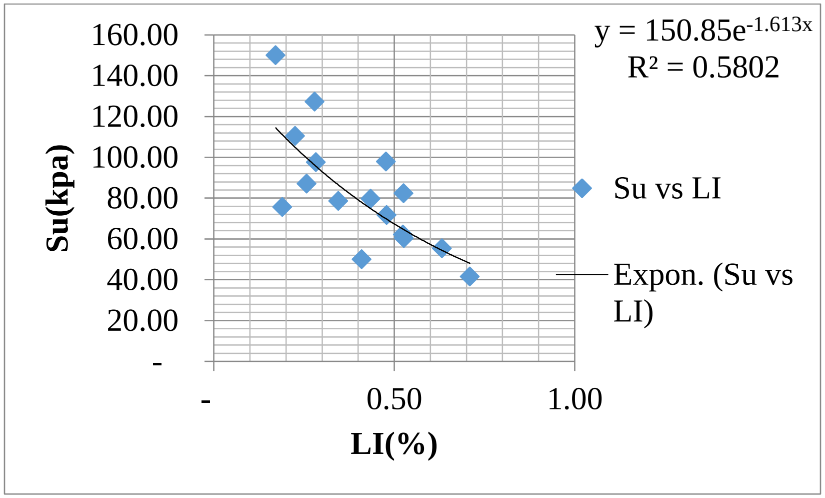


Figure 5.4 Scatter plot and best-fit curve of LI and Su using Linear, Exponential and Logarithmic relation.

E. Relation between OMC and Su

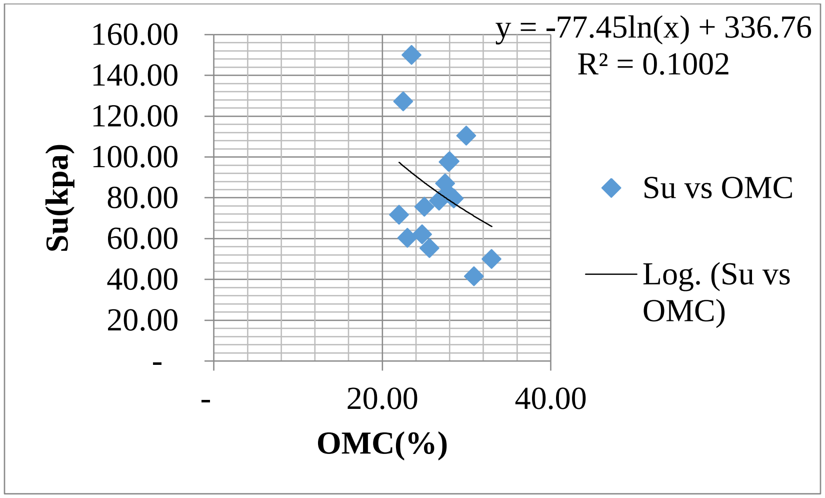


Figure 5.4 Scatter plot and best-fit curve of Optimum moisture content and Su

F. Relation between MDD and Su

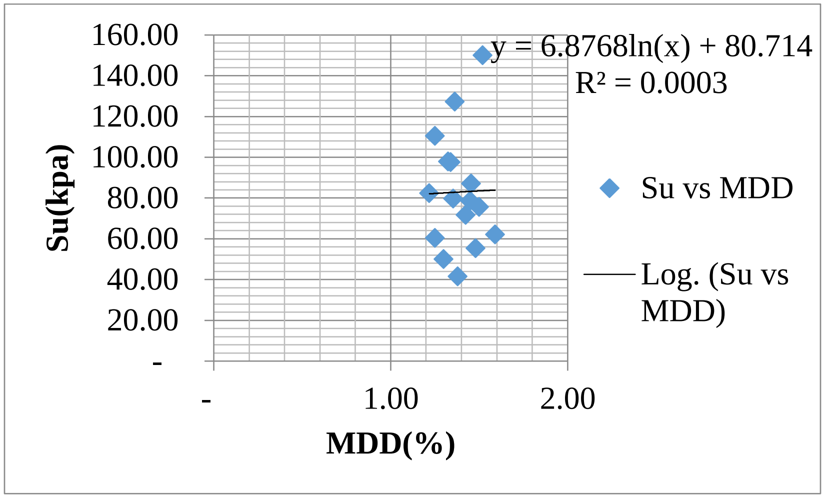


Figure 5.5 Scatter plot and best-fit curve of Maximum Dry Density and Su

G. Relation between Su vs w

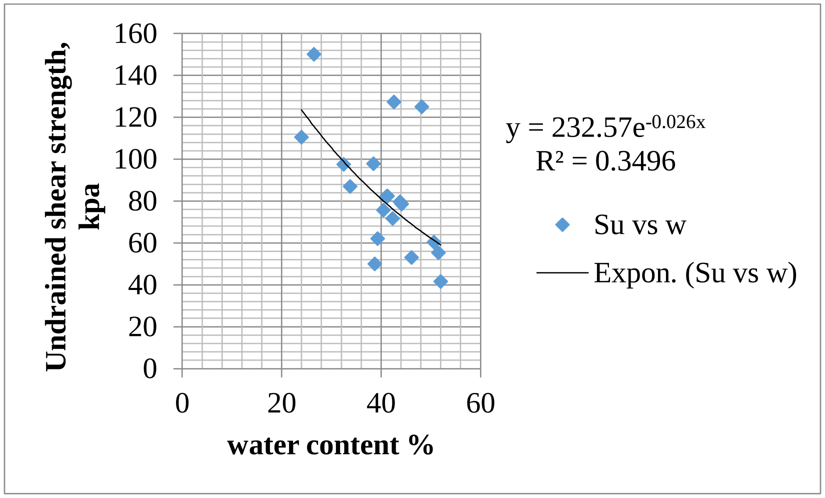


Figure 5.6 Scatter plot and best-fit curve of undrained shear strength vs water content

5.4 Correlation and Regression analysis

Correlation and regression are statistical methods that are commonly used in the applied science to compare two or more variables. But they are quite different.

5.4.1 Correlation

Correlation measures the association between two variables and quantities the strength of their relationship. Correlation evaluates only the existing data. Correlation provides a numerical measure of the linear relationship between two continuous variables X and Y. The resulting correlation coefficient, “r value” is known as the Pearson product moment correlation coefficient after the mathematician who first described it. X is known as the independent (explanatory) variable while Y is known as the dependent (response) variable. A significant advantage of the correlation coefficient is that it does not depend on the units of X and Y and can therefore be used to compare any two variables regardless of their units**. [12]**

The calculation of the correlation coefficient is mathematically complex, but readily performed by most computer statistics programs which is called Statistical Package for Social Science (SPSS).

Correlation and correlation coefficient of the laboratory experimented samples are present on tables 5.2 as shown below.

Table 5.2 Correlation coefficients (R) between variables for primary data

Correlations								
		Su	LL	PL	PI	OMC	MDD	LI
Su	Pearson Correlation	1	-.394	.117	-.584*	-.325	.014	-.762**
	Sig. (2-tailed)		.131	.665	.017	.220	.958	.001
	N	16	16	16	16	16	16	16
LL	Pearson Correlation	-.394	1	.603*	.788**	-.253	.246	.041
	Sig. (2-tailed)	.131		.013	.000	.345	.358	.881
	N	16	16	16	16	16	16	16
PL	Pearson Correlation	.117	.603*	1	-.017	-.389	-.109	-.289
	Sig. (2-tailed)	.665	.013		.951	.137	.687	.277
	N	16	16	16	16	16	16	16
PI	Pearson Correlation	-.584*	.788**	-.017	1	-.017	.393	.274
	Sig. (2-tailed)	.017	.000	.951		.951	.132	.304
	N	16	16	16	16	16	16	16
OMC	Pearson Correlation	-.325	-.253	-.389	-.017	1	-.406	.191
	Sig. (2-tailed)	.220	.345	.137	.951		.119	.479
	N	16	16	16	16	16	16	16
MDD	Pearson Correlation	.014	.246	-.109	.393	-.406	1	-.092
	Sig. (2-tailed)	.958	.358	.687	.132	.119		.734
	N	16	16	16	16	16	16	16
LI	Pearson Correlation	-.762**	.041	-.289	.274	.191	-.092	1
	Sig. (2-tailed)	.001	.881	.277	.304	.479	.734	
	N	16	16	16	16	16	16	16
*. Correlation is significant at the 0.05 level (2-tailed).								
**. Correlation is significant at the 0.01 level (2-tailed).								

The result of ‘r’ value is influenced by different statically methods, such as the mean and standard deviation as well as markedly influence by a single outlying value. The square of ‘r’ value known as the coefficient of determination or R^2. It describes the proportion of change in the dependent variable Su which is explained by each independent variable LL, PL, PI, OMC, MDD and LI. For example, as it seen on the above table the value of r is 0.584 between the dependent Su and independent variable PI. Therefore, coefficient of determination R^2 is 0.341.This means about 34.1% of the change in Su can be explained by a change in PI. And this is the same for all variables shown on the table. The larger the correlation coefficient, the larger the coefficient of determination, and the more influence changes in the independent variable have on the dependent variable.

5.4.2 Regression analysis

Regression uses the existing data to define a mathematical equation which can be used to predict the value of one variable based on the value of one or more other variables. The regression equation can therefore be used to predict the outcome of observations not previously seen or tested. [15] Regression analysis mathematically describes the dependence of the Y variable on the X variable and constructs an equation which can be used to predict any value of Y for any value of X. Regression analysis can be two types. These are Simple regression and multiple regression analysis.

A. Simple Regression

Simple linear regression considers a single independent or predictor x and a dependent or response variable Y. the dependent can be described by the model:

$$Y = b + aX \text{ for linear} \qquad (6.1)$$

Where Y is the dependent variable, X is the single independent variable, a is the Y-intercept of the regression line, and b is the slope of the line (also known as the regression coefficient).[16]

B. Multiple Regression analysis

A multiple regression is a model that contains more than one independent variable to describe the dependent one. The dependent can be described by the model:

$Y = a_0 + a_1x_1 + a_2x_2 + a_3x_3$…………………….. for linear (6.1)

$Y = a_0 + a_1x_1 + a_2x_2 + a_3x_3 + a_4x_1^2 + a_5x_2^2 + a_6x_3^2 + \ldots + a_{n-2}x_1^n + a_n + x_3^n$ for nonlinear (6.2)

Where, $a_1, a_2, a_3 \ldots\ldots x_n$.= coefficients of the independent variables

$x_1, x_2, x_3 \ldots\ldots x_n$= the independent variables

Y= the response (dependent variable)

In addition to deriving the regression equation, regression analysis also draws a line of best fit through the data points of the scatter gram. These "regression lines" may be linear, in which case the relationship between the variables fits a straight line, or nonlinear, in which case a polynomial equation is used to describe the relationship.

5.4.2.1 Regression analysis between Su and LL

Table 5.3 summary model of Regression analysis between Su and LL

Model Summary				
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.394[a]	.155	.095	27.34321
a. Predictors: (Constant), LL				

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	159.798	48.401		3.302	.005
	LL	-1.229	.766	-.394	-1.604	.131

a. Dependent Variable: Su

The model equation is then:

Su= 159.798 -1.229LL with $R^2 = 0.155$ (5.4)

Where Su in Kpa and LL in (%)

5.4.2.2 Regression analysis between Su and pL

Table 5.4 summary model of Regression analysis between Su and PL

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.117[a]	.014	-.057	29.54358

a. Predictors: (Constant), PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	68.153	34.238		1.991	.066
	PL	.594	1.343	.117	.443	.665

a. Dependent Variable: Su

The model equation is then:

Su= 68.153 -0.594PL with $R^2 = 0.014$ **(5.5)**

Where Su in Kpa and pL in (%)

5.4.2.3 Regression analysis between Su and pI

Table 5.5 summary model of Regression analysis between Su and PL

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.584[a]	.341	.294	24.14201

a. Predictors: (Constant), PI

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	168.943	32.484		5.201	.000
	PI	-2.283	.848	-.584	-2.694	.017

a. Dependent Variable: Su

The model equation is then:

Su= 168.943 -2.283PI with R^2 = 0.341 **(5.5)**

Where Su in Kpa and pI in (%)

5.4.2.4 Regression analysis between Su and OMC

Table 5.6 summary model of Regression analysis between Su and OMC

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.325[a]	.105	.041	28.13813

a. Predictors: (Constant), OMC

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	162.197	62.108		2.612	.021
	OMC	-2.971	2.314	-.325	-1.284	.220

a. Dependent Variable: Su

Su= 162.197 -2.971OMC with R^2 = 0.105 **(5.5)**

Where Su in Kpa and OMC in (%)

5.4.2.5 Regression analysis between Su and MDD

Table 5.7 summary model of Regression analysis between Su and MDD

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.014[a]	.000	-.071	29.74646

a. Predictors: (Constant), MDD

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	77.674	99.423		.781	.448
	MDD	3.800	71.456	.014	.053	.958

a. Dependent Variable: Su

Su= 77.674+ 3.8MDD with R^2 = 0.000 **(5.5)**

Where Su in Kpa and MDD in (g/cc)

5.4.2.6 Regression analysis between Su and LI

Table 5.8 summary model of Regression analysis between Su and LI

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.762[a]	.581	.551	19.26094

a. Predictors: (Constant), LI

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	132.297	12.196		10.848	.000
	LI	-124.152	28.188	-.762	-4.404	.001

a. Dependent Variable: Su

Su= 132.297-124.152LI with R^2 = 0.581 **(5.5)**

Where Su in Kpa and LI in (decimal)

5.4.2.7 Regression analysis between Su and w

Table 5.9 summary model of Regression analysis between Su and w

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.706[a]	.498	.463	21.06993

a. Predictors: (Constant), w

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	6175.242	1	6175.242	13.910	.002[b]
	Residual	6215.185	14	443.942		
	Total	12390.427	15			

a. Dependent Variable: Su
b. Predictors: (Constant), w

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	184.062	27.618		6.664	.000
	w	-2.518	.675	-.706	-3.730	.002

a. Dependent Variable: Su

5.4.2.8 Multiple Regression analysis between Su and all parameters.

To examine the combined effect of these parameters on Su a multiple regression analysis is conducted. The basic form of the equation is as follows:

$$y = \beta_o + \beta_1 x_1 + \beta_2 x_2 + \dots\dots\dots\dots + \beta_n x_n + \varepsilon \;\dots\dots\dots\dots\dots\dots\dots\dots\; (5.1)$$

The developed equations for multiple regressions were consisted of three models:

Model 1: Regression between Su and other independent variables (w, LI, PI, LL, OMC, MDD, and LI)

Model 2: Regression between Su and other independent variables (w, LI, PI, LL, OMC, and LI)

Model 3: Regression between Su and other independent variables (w, LI, PI, LL, and LI)

Table 5.10 summary model of Regression analysis between model 1, 2 and 3

Model 1 Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.900[a]	.810	.683	16.19210

a. Predictors: (Constant), LI, LL, MDD, OMC, PL, w

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	10030.771	6	1671.795	6.376	.007[b]
	Residual	2359.656	9	262.184		
	Total	12390.427	15			

a. Dependent Variable: Su

b. Predictors: (Constant), LI, LL, MDD, OMC, PL, w

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	311.266	118.842		2.619	.028
	w	2.066	5.090	.579	.406	.694
	LL	-2.228	1.762	-.714	-1.264	.238
	PL	-.635	3.224	-.125	-.197	.848
	OMC	-2.571	1.725	-.281	-1.491	.170
	MDD	-10.374	52.026	-.039	-.199	.846
	LI	-184.057	192.304	-1.130	-.957	.364

a. Dependent Variable: Su

From this model 1 Result Eventhough the R-square is good result but the model is insignificant since significance is > 0.005 which is 0.007.

Model 1 Equation:

Su= 311.266 + 2.066w -2.228LL -0.635PL -2.571OMC -10.374MDD -184.057LI ……… (5.2)

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.869[a]	.755	.666	16.61358

a. Predictors: (Constant), LI, LL, PI, w

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	9354.305	4	2338.576	8.473	.002[b]
	Residual	3036.122	11	276.011		
	Total	12390.427	15			

a. Dependent Variable: Su

b. Predictors: (Constant), LI, LL, PI, w

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
	(Constant)	236.278	57.455		4.112	.002
	w	4.123	4.995	1.156	.825	.427
1	LL	-4.191	4.625	-1.344	-.906	.384
	PI	1.272	3.091	.326	.412	.689
	LI	-263.294	187.246	-1.616	-1.406	.187

a. Dependent Variable: Su

From this model 2 Result is insignificant since significance is < 0.005 which is 0.002.

Model 2 Equation:

Su= 236.278 + 4.123w -4.191LL +1.272PI -263.294LI …………………………………… (5.3)
With R^2 =0.755

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.899[a]	.809	.713	15.39507

a. Predictors: (Constant), OMC, PI, LI, w, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
	Regression	10020.346	5	2004.069	8.456	.002[b]
1	Residual	2370.081	10	237.008		
	Total	12390.427	15			

a. Dependent Variable: Su

b. Predictors: (Constant), OMC, PI, LI, w, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	291.541	62.621		4.656	.001
	w	1.949	4.807	.546	.405	.694
	LL	-2.687	4.378	-.862	-.614	.553
	PI	.431	2.908	.110	.148	.885
	LI	-178.302	180.768	-1.095	-.986	.347
	OMC	-2.405	1.434	-.263	-1.676	.125

a. Dependent Variable: Su

From this model 3 Result is significant since significance is < 0.005 which is 0.002.

Model 3 Equation:

Su= 291.541 + 1.949w -2.687LL +0.431PI -178.302LI -2.405OMC……………………… (5.4)
With R^2 =0.809

It can be observe that the value of R square is greater on model 3 compared to other models. As well as it is significant. The above equation is therefore the final prediction equation of Su. This means the equation model is able to express **80.9%** of the variation of Su. On other hand about **19.1%** Su is not explained by the developed equation.

Table 5.11 Summary of regression equations and their corresponding R^2 value in predicting Su.

M/N	Coefficients of predictor							Output Equations	R^2
	LL	PL	PI	w	OMC	MDD	LI		
1	-1.229							**Su=159.798-1.229LL**	0.155
2		-0.594						**Su=68.153-0.594PL**	0.014
3			-2.283					**Su=168.943-2.283PI**	0.341
4					-2.971			**Su=162.197-2.971OMC**	0.105
5						3.8		**Su=77.674+3.8MDD**	- 0.071
6				-2.518				**Su=184.062-2.518w**	0.498
7							-124.152	**Su=132.297-124.152LI**	0.581
		Multiple Regression							R^2
2	4.191		1.272	4.123			263.94	**Su= 236.278 + 4.123w - 4.191LL +1.272PI - 263.294LI**	0.755
3	2.687		0.431	1.949	-2.405		178.302	**Su= 291.541 + 1.949w - 2.687LL +0.431PI - 178.302LI -2.405OMC**	0.809

Where LL, PL, PI, OMC, LI and Su in Kpa and MDD in g/cc

Note: M/N=model number, LL=liquid limit, PL=plastic limit, PI=plastic limit LI= Liquidity index, Su= Undrained Shear strength, OMC=Optimum moisture content and MDD=maximum dry density

5.5 comparisons between the actual Su and developed equation

Eventhough I haven't get secondary data for the study area the validation of the developed correlation is conducted by using two known test results which follows similar testing procedures with the subject research. Depending on the relative significance order, **Model 3 Su= 291.541 + 1.949w -2.687LL +0.431PI -178.302LI -2.405OMC** is preferably selected among the different alternative correlations for further verifications.

Subsequently, using the control test results and the developed correlation equation, the predicted Su is determined so as to compare it with the actual Su value as shown in Table 5.12:

Table 5.12**:** Validation of the Developed Correlation

Sample No.	**Control Test Point**							**Actual Su (Kpa)**	**Developed Su (Kpa)**
	W (%)	**LL (%)**	**PL (%)**	**PI (%)**	**OMC (%)**	**MDD(g/cc)**	**LI(Decimal)**		
17	48.24	62.95	28.34	34.61	26.80	1.25	0.57	**125.00**	**65.249**
18	46.15	62.35	25.88	36.47	25.00	1.50	0.56	**53.03**	**69.697**

As shown in table 5.12 the value of Actual undrained shear strength and developed undrained shear strength have good relation for sample no.18 which is about 76.09% near to the actual Su and for sample no. 17 the validation of developed Su is about 52.2% near to the actual one. By investigating the accuracy of the predicted Su values, the developed correlation applied for preliminary characterization of the undrained shear strength of the Red clay soil in the study area.

5.6 Discussion on Correlation Results

As the aim of the thesis is to formulate a correlation between the variables of the stated parameters, which is expected to predict the value of **Su**, an attempt is made to obtain which one of the predictors can be strongly related with dependent variables.

There are two options to predict the Su. The first is Correlation table .this can be noted that Su can be strongly related with LI from other atterberg limits and PI the second.

The correlation table also helps to show that the positive and negative (the increase or decrease) effect of each variable on the dependent variable Su as well as level of significant and number of samples used.

The other method is using regression analysis. First each independent variable is analyzed its effect and prediction capacity one by one. The analysis is then expressed using equations. From these equations it can be observed that Su has relatively strong correlation with LI and PI which have R^2=0.581 and R^2=0.341 respectively.

After having these equations and the corresponding coefficient of determination values, multiple regressions is used by enter method. The result of this analysis gave 2 models with their corresponding coefficient of determination values. According to the analysis the model with large 'r 'value is the more that explains or predicates Su. Therefore, model number 3 with R^2=0.809 is chosen as final equation to predict Su. This means the Su value is more influenced by w, LL, PI, LI and OMC for the study area Red clay soil.

5.7 Summary of Laboratory Test Results

In order to analyze the intended correlation, the test results were compiled and summarized in a way that the SPSS Software inputs the data as shown in Table 5.13

Table 5.13 Summary of Laboratory Test Result

S/N	Area of Sampling	Designation And Depth	Water content (%)	Specific Gravity (Gs)	Grain Size Distribution					Atterberg Limit				Soil Classification		Compaction		UU-Triaxial test
					Gravel (%)	Sand (%)	Silt (%)	Clay (%)	FC (%)	LL (%)	PL (%)	PI (%)	LI	AASHTO	UCS	OMC (%)	MDD (g/cc)	Cu(Kpa)
1	B/Hora Teachers college	TP-1@2.5m	41.17	2.66	0	9.60	19.00	71.40	90.4	59.00	21.36	37.64	0.53	A-7-5	CH	27.80	1.22	82.28
1*	B/Hora Teachers college	TP-1@3m	41.35	2.67	0	9.70	18.67	71.63	90.3	58.55	20.21	38.34	0.55	A-7-5	CH	27.60	1.23	82.33
2	Kidiste Mariam school	TP-2@3m	32.52	2.85	0	5.80	19.96	74.20	94.16	52.80	24.50	28.30	0.28	A-7-6	CH	27.90	1.34	97.54
3	Abayi junior school	TP-3@3m	39.32	2.71	0	2.50	14.50	83.00	97.5	59.80	16.97	42.83	0.52	A-7-6	CH	24.75	1.59	62.10
4	Mekane Iyesus church	TP-4@2.5m	52.00	2.79	0	1.98	22.02	76.00	98.02	62.80	25.66	37.14	0.71	A-7-6	CH	30.90	1.38	41.52
5	B/Hora University*	TP-5@2m	43.86	2.78	0	3.30	22.30	74.40	96.7	69.00	24.50	44.50	0.44	A-7-6	CH	28.50	1.35	79.61
6	preparatory school	TP-6@2.5m	24.00	2.7	0	11.20	32.53	56.27	88.80	42.00	18.75	23.25	0.23	A-7-6	CL	30.00	1.25	110.45
7	Robi Gebeya	TP-7@2.75m	42.36	2.68	0	18.80	17.84	63.36	81.2	64.90	21.60	43.30	0.48	A-7-6	CH	22.00	1.45	71.60
8	Around Madiya	TP-8@2.5m	33.82	2.65	0	4.50	32.48	63.02	95.5	59.05	25.09	33.96	0.26	A-7-6	CH	27.50	1.46	87.00
9	Arsema	TP-9@2.5m	42.64	2.67	0	8.30	25.57	66.13	91.7	72.53	31.05	41.45	0.28	A-7-6	CH	22.50	1.36	127.30
10	Agricultural Beruae	TP-10@2.5m	38.50	2.69	0	14.60	20.94	64.46	85.4	54.42	23.97	30.45	0.48	A-7-6	CH	28.00	1.32	97.84
11	Algi church	TP-11@3m	38.79	2.66	0	5.50	24.03	70.47	94.5	65.89	19.98	45.91	0.41	A-7-6	CH	33.00	1.30	49.99
12	Around Bule Hora Hotel	TP-12@2.5m	40.50	2.72	0	16.20	18.80	65.00	83.8	79.84	31.27	48.57	0.19	A-7-5	CH	25.00	1.50	75.65
13	B/Hora University**	TP-13@2.5m	50.66	2.77	0	4.00	6.80	89.20	96	66.34	33.19	33.15	0.53	A-7-5	CH	23.00	1.25	60.39
14	Tsahay Hora Hotel	TP-14@3m	26.50	2.76	0	4.10	31.30	64.00	95.3	53.23	20.97	32.26	0.17	A-7-6	CH	23.50	1.52	150.00
15	B/Hora Woreda Finance office	TP-15@3m	51.57	2.83	0	4.00	39.80	56.20	96.6	68.70	22.06	46.64	0.63	A-7-6	CH	25.64	1.48	55.37
16	Around Hospital	TP-16@3m	44.16	2.68	0	3.40	14.10	82.50	96.6	70.43	30.29	40.14	0.35	A-7-5	MH	26.75	1.45	78.51
17	Zone administration office	TP-17@2.75m	48.24	2.79	0	5.00	15.40	79.60	95	62.95	28.34	34.61	0.57	A-7-5	CH	26.80	1.25	125.00
18	B/Hora Woreda Court office	TP-18@2.5m	46.15	2.71	0	16.10	18.90	65.00	83.9	62.35	25.88	36.47	0.56	A-7-6	CH	25.00	1.50	53.03

CHAPTER 6

CONCLUSION AND RECOMMENDATION

The main focus of this research is on the outcome of predicting of Su from index properties of Red clay soils. Considering this all the necessary data and statistical methods are used in order to achieve the goal. Based on these a number of conclusions and recommendations are made.

6.1 Conclusion

- The prediction of Su value by MDD, PL, OMC and LL is very weak with R^2=-0.071, R^2= 0.014, R^2= 0.105, R^2= 0.155, respectively which can be usually neglect and the model equation is nonlinear.
- Liquidity index highly affects the value of Su with R^2=0.581, Plasticity index moderately affects the value of Su with R^2=0.341. Water content also highly affects Su with R^2= 0.498 this means any increment in PI will decrease the value of Su by 34.1% since they have inversely proportional relation.

Generally, the larger the correlation coefficient, the larger the coefficient of determination, and the more influence changes in the independent variable have on the dependent variable.

- Liquidity index, Plasticity index and water content of the Red clay soil affects the value of undrained shear strength of Red clay soil.
- From the developed equations, the combination of w, LL, PI, LI and OMC gives relatively strong correlation and can be able to express the Su value about 80.9% with R^2=0.809
- The comparison between the developed equation and the actual Su value is good. The small percent difference comes because of that the developed equation can be express about 80.9%. In other word it is not a perfect relation about 19.1% a change in Su cannot be explain by the predictors.

6.2 Recommendation

The exposure encountered in trying to conduct the current research has revealed areas where further efforts may be proved in the future. Following are some of the recommendations in relation to the subject study:

i. The sample sizes which were used as a control point may affect the required result. Therefore, more number of sample sizes with accurate laboratory test may increase the prediction result.
ii. It is advisable to conduct comparative correlations between consolidated undrained shear strength tests with soil index properties for unsaturated condition.
iii. It is also recommended to carry out such a study on fine grained soil on other parts of Ethiopia.
iv. In order to get the perfect relation among the variables and to develop the extent of applicability further works are required.

REFERENCES

[1] Obasi and Anyaegbunam., “correlation of the undrained shear strength and plasticity index of tropical clays” Nigerian Journal of Technology, Vol. 24, No. 2, September

[2] 2005V.N.S. Murthy, Principles and Practices of Soil Mechanics and Foundation Engineering

[3] Raymond, G.P. (1997). Geotechnical Engineering. Unified Soil Classification.

[4] Fundamentals of Geotechnical Engineering, Third Edition by Braja M. Das

[5] Budhu, M. Soil mechanics and foundations / Muni Budhu.—3rd ed.

[6] Arora, K.R., Soil Mechanics and Foundation Engineering, Re-print Standard Publishers Distributer, Nai Sarak, Delhi, 2004.

[7] Fredrics, M., Standard Hand Book for Civil Engineers, McGraw-Hill Book Company, New York, 1983.

[8] Carter, M. and Bentley, S.P., "*Correlation of Soil Properties*, "Pentech Press, London, 1991.

[9] Mittal, S. and Shukla, J.P., Soil Testing for Engineers, Romesh Chander Khanna Publishers Delhi (India), 2000.

[10] Nagaraj H.B. (2000). Prediction of engineering properties of Fine-Grained Soils from their index properties. PhD Thesis, Indian institute of Science, India.

[11] Anteneh Getachew (2012).Correlating Dynamic Cone Penetration Index (DCPI) with Undrained Shear Strength for Clayey Soils. MSC Thesis, Addis Ababa University, Addis Ababa.

[12] Geotechnical investigation report for Bule Hora university project (2015), Betterway Consulting Engineers PLC.

[13] ASTM (1998). Annual Book of ASTM Standards on Soils and Rocks.

[14] Zelalem Worku (2010). Prediction of California Bearing Ratio (CBR) Value from Index Properties of Soil. MSC Thesis, Addis Ababa University, Addis Ababa.

[15] American Society for Testing and Materials, D 2487–00, Standard Practice for classification of Soils for Engineering Purposes (Unified Soil Classification System), Annual Book of ASTM Standards, Volume 04.08, West Conshohocken, Pennsylvania, 2000.

APPENDIX I

Laboratory experimented tests

A. Specific gravity

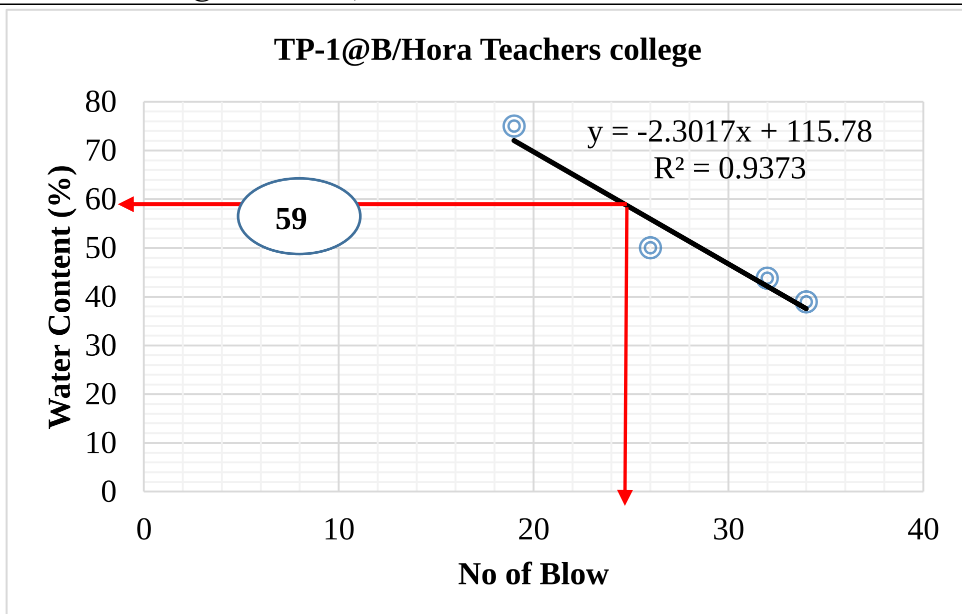
Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Test Pit -1 Bule Hora Teachers college,@2.5m	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B1	B7
Mass empty Pycnometer, Mp (g)	44	44
Mass of soil, Ms (g)	10	11.3
Testing Temperature, T (^{0}C)	19	19.5
Mass of pycnometer + water, Mpw (g)	153	153
Mass of pycnometer + water + Soil, Mpws (g)	159.21	160.10
(Mpw – Mpws + Ms) (g)	3.79	4.20
GS(at Testing Temprature)	2.64	2.69
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.64	2.69
GS (at 20^{0}c) Avg.		2.66

B. Moisture content

Sample number :pit1 @2.5m in B/Hora Teachers College		
specimen number	1	2
Mc=mass of empty,clean+llid(g)	61	62
Mcms=mass of can,lid,and moist soil(g)	120	123
Mcds=mass of can,lid and dry soil (g)	103	105
Ms =mass of soil solids (g) =Mcds-Mc	42	43
Mw= mass of pore water (g) =Mcms-Mcds	17	18
w=water content ,w%= (mw/ms)*100	40.4762	41.86
Average	41.17	

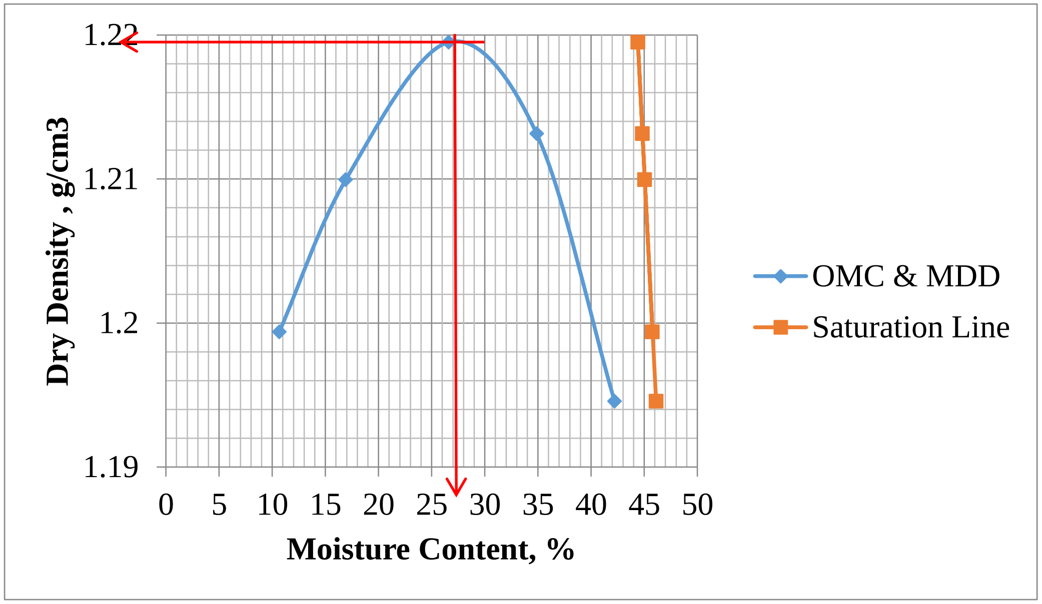
C. Atterberg Limit

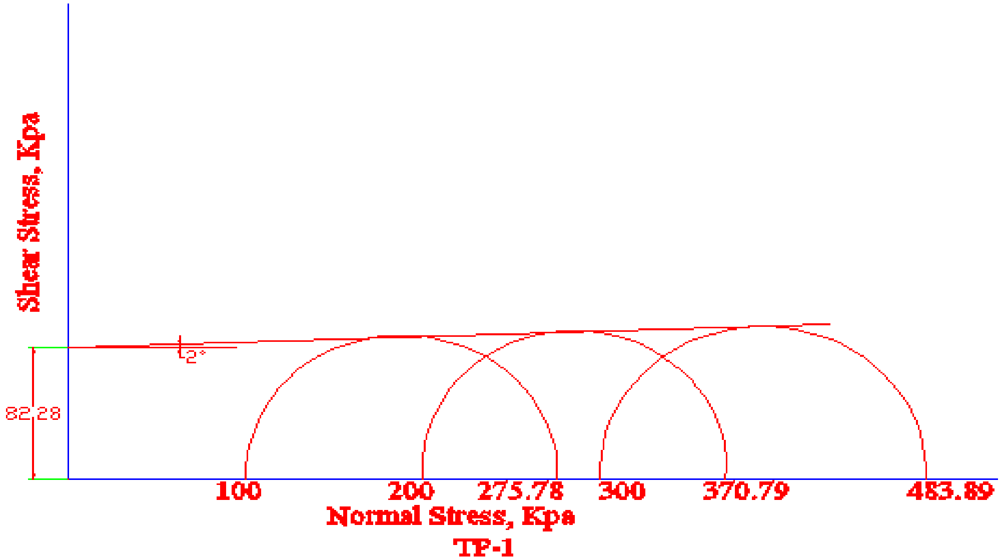
sample location	pit 1@ B/Hora Teachers college ,Depth @2.5 m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	60	60	60	60	61	61
MCMS=mass of Can,lid and moist soil (grams)	81	75	85	83	73	74.5
MCDS=mass of can,lid and dry soils(grams)	72	70	78	76	71	72
MS=mass of soil solids(g)= MCDS-MC	12	10	18	16	10	11
MW= mass of pore water (g)=MCMS-MCDS	9	5	7	7	2	2.5
W=water content,w%=(MW/MS)*100	75	50	38.8889	43.75	20	22.7273
No. of drops (N)	19	26	34	32	Avg.	21.3636
LL=59@ Blow =25, PI=LL-PL=59-21.3636 =37.6364						



D. Compaction

water content determination for TP 1:@2.5m B/Hora Teachers college						
sample description	Red clay soil					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	
MC = Mass of empty, clean can + lid (grams)	62	62	96	93	97	
MCMS = Mass of can, lid, and moist soil (grams)	90	121	157	154	167	
MCDS = Mass of can, lid, and dry soil (grams)	86	110	142	136.3	143.6	
MS = Mass of soil solids (grams)	24	48	46	43.3	46.6	
MW = Mass of pore water (grams)	4	11	15	17.7	23.4	
W = Water content, w%	16.6667	22.9167	32.6087	40.8776	50.2146	
density determination	volume of mold=944cm3					
Compacted Soil - Sample no.	1	2	3	4	5	
w = Assumed water content, w%	8	12	16	20	24	
Actual average water content, w%	10.66667	16.91667	26.6087	34.8776	42.21459	
Mass of compacted soil and mold (grams)	5658	5739	5861	5948	6007	
Mass of mold (grams)	4405	4405	4405	4405	4405	
Wet mass of soil in mold (grams)	1253	1334	1456	1543	1602	
Wet density, ρ , (g/cm3)	1.327331	1.414634	1.544008	1.636267	1.698834	
Dry density, ρd , (g/cm3)	1.199395	1.209951	1.219512	1.21315	1.194556	
dry unit weight ɤd (KN/m3)	11.99395	12.09951	12.19512	12.1315	11.94556	
for drawing saturation line(zero air void line S=100%	Wsat= (ɤw/ɤd-1/Gs)100				Gs=2.66	
ɤd	11.99395	12.09951	12.19512	12.1315	11.94556	
water content(%)	45.78138	45.054	44.40602	44.83607	46.1191	
		MDD=1.219		OMC=27.8		


1.22
1.21
1.2
1.19
Dry Density , g/cm3
0
5
10
15
20
25
30
35
40
45
50
Moisture Content, %
OMC & MDD
Saturation Line


Shear Stress, Kpa
82.28
12°
100
200
275.78
300
370.79
483.89
Normal Stress, Kpa
TP-1
B/Hora Teachers College

2. Sample 2

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Kidiste Mariam school, depth3m	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B2	B8
Mass empty Pycnometer, Mp (g)	45.6	54.2
Mass of soil, Ms (g)	25	25
Testing Temperature, T (^{0}C)	18	19
Mass of pycnometer + water, Mpw (g)	150.88	153.35
Mass of pycnometer + water + Soil, Mpws (g)	167.50	169.20
(Mpw – Mpws + Ms) (g)	8.38	9.15
GS(at Testing Temprature)	2.98	2.73
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.98	2.73
GS (at 20^{0}c) Avg.		2.85

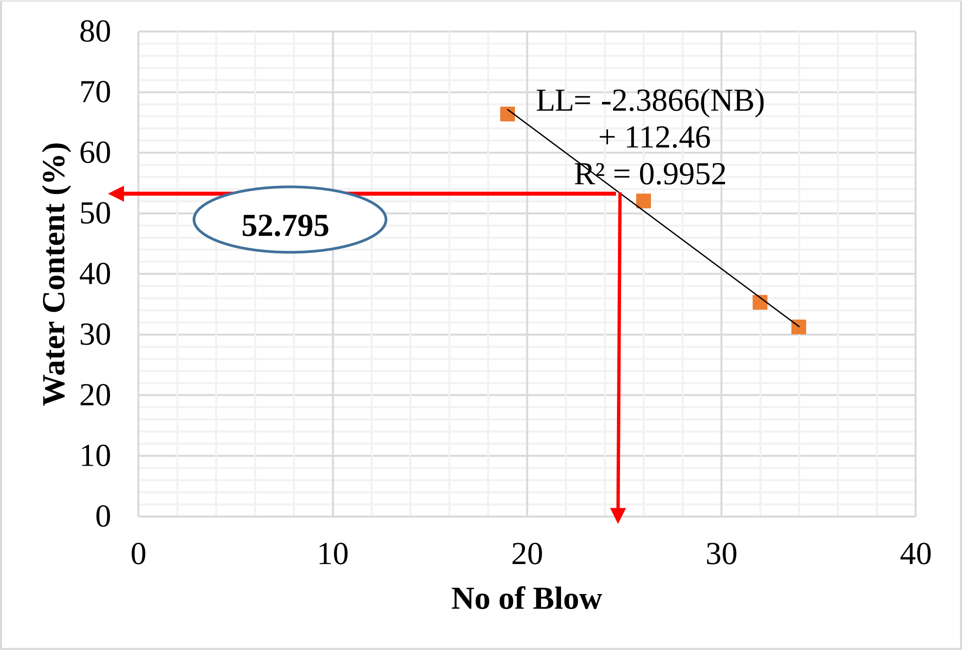
B) Moisture content

Sample number :pit2

specimen number	1	2
Mc=mass of empty,clean+llid(g)	61	61
Mcms=mass of can,lid,and moist soil(g)	114	116
Mcds=mass of can,lid and dry soil (g)	101	102.5
Ms =mass of soil solids (g) =Mcds-Mc	40	41.5
Mw= mass of pore water (g) =Mcms-Mcds	13	13.5
w=water content ,w%= (mw/ms)*100	32.5	32.53
average	32.52	

C) Atterberg Limit

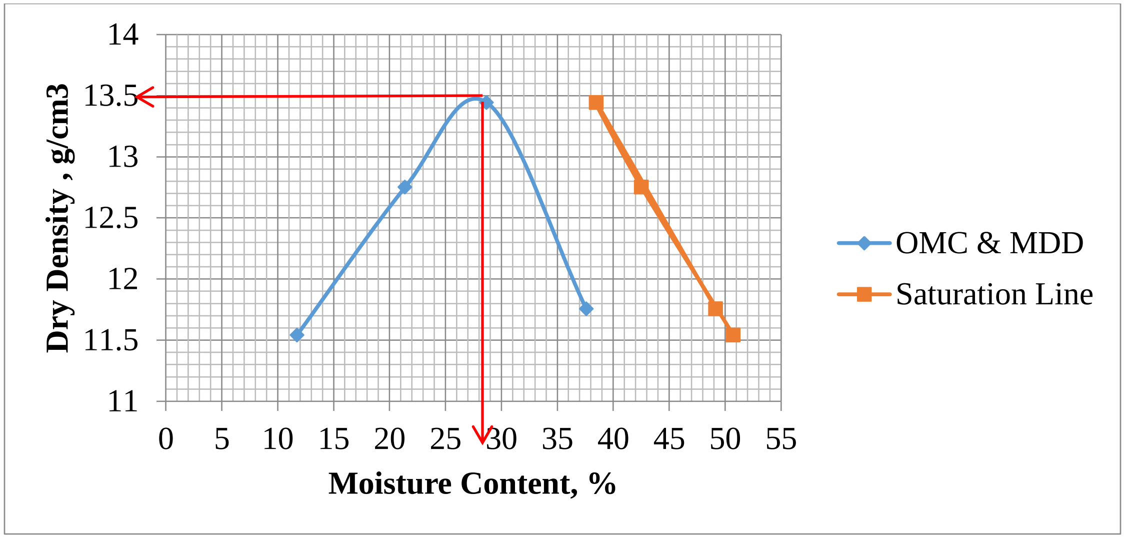
sample location	pit 2@ Kidiste Mariam school,Depth @3 m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	60	60	60	60	60	60
MCMS=mass of Can,lid and moist soil (grams)	78.3	75.2	81	83	70	75.5
MCDS=mass of can,lid and dry soils(grams)	71	70	76	77	68	72.5
MS=mass of soil solids(g)= MCDS-MC	11	10	16	17	8	12.5
MW= mass of pore water (g)=MCMS-MCDS	7.3	5.2	5	6	2	3
W=water content,w%=(MW/MS)*100	66.3636	52	31.25	35.2941	25	24
No. of drops (N)	19	26	34	32	Avg.	24.5
LL=52.795@ Blow =25, PI=LL-PL=52.795-24.5 =28.295						



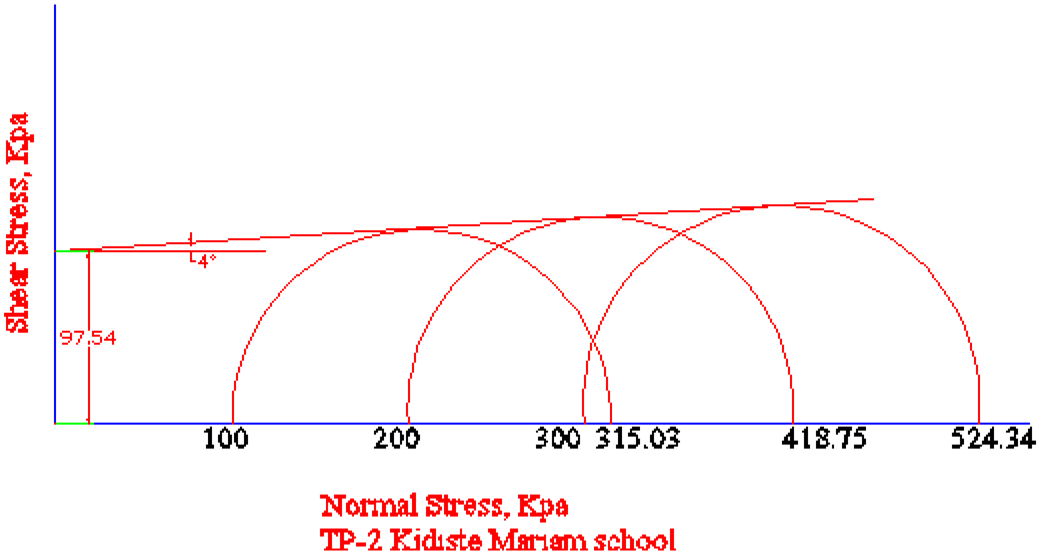
80
70
60
50
40
30
20
10
0
0
10
20
30
40
Water Content (%)
No of Blow
LL= -2.3866(NB) + 112.46
R^2 = 0.9952
52.795

D. Compaction

water content determination for TP 2:@3m Kidiste Mariam school						
sample location	TP 2:@3m Kidiste Mariam school					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4		
Water content - Sample no.						
Moisture can number - Lid number						
MC = Mass of empty, clean can + lid (grams)	32.9	33.7	32.9	33		
MCMS = Mass of can, lid, and moist soil (grams)	158.4	152.3	152.5	150.6		
MCDS = Mass of can, lid, and dry soil (grams)	139.5	126.8	121.7	114.9		
MS = Mass of soil solids (grams)	106.6	93.1	88.8	81.9		
MW = Mass of pore water (grams)	18.9	25.5	30.8	35.7		
W = Water content, w%	17.7298	27.3899	34.6847	43.5897		
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4		
w = Assumed water content, w%	8	16	24	32		
Actual average water content, w%	11.72983	21.3899	28.68468	37.58974		
Mass of compacted soil and mold (grams)	5861.4	6120.8	6303.9	6190.8		
Mass of mold (grams)	4566.1	4566.1	4566.1	4566.1		
Wet mass of soil in mold (grams)	1295.3	1554.7	1737.8	1624.7		
Wet density, ρ , (g/cm3)	1.289561	1.547812	1.730101	1.617502		
Dry density, ρd , (g/cm3)	1.154178	1.275075	1.34445	1.175598		
dry unit weight ɤd (KN/m3)	11.54178	12.75075	13.4445	11.75598		
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.786**	
ɤd	11.54178	12.75075	13.4445	11.75598		
water content(%)	50.74797	42.53301	38.48611	49.16935		
		MDD=13.4445		OMC=27.9		



E. UU-Triaxial test



3. Sample 3

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Abayi junior school	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B2	B8
Mass empty Pycnometer, Mp (g)	45.6	54.2
Mass of soil, Ms (g)	25	25
Testing Temperature, T (^{0}C)	18	19
Mass of pycnometer + water, Mpw (g)	151.86	153.35
Mass of pycnometer + water + Soil, Mpws (g)	167.50	169.20
(Mpw – Mpws + Ms) (g)	9.36	9.15
GS(at Testing Temprature)	2.67	2.73
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.67	2.73
GS (at 20^{0}c) Avg.		2.71

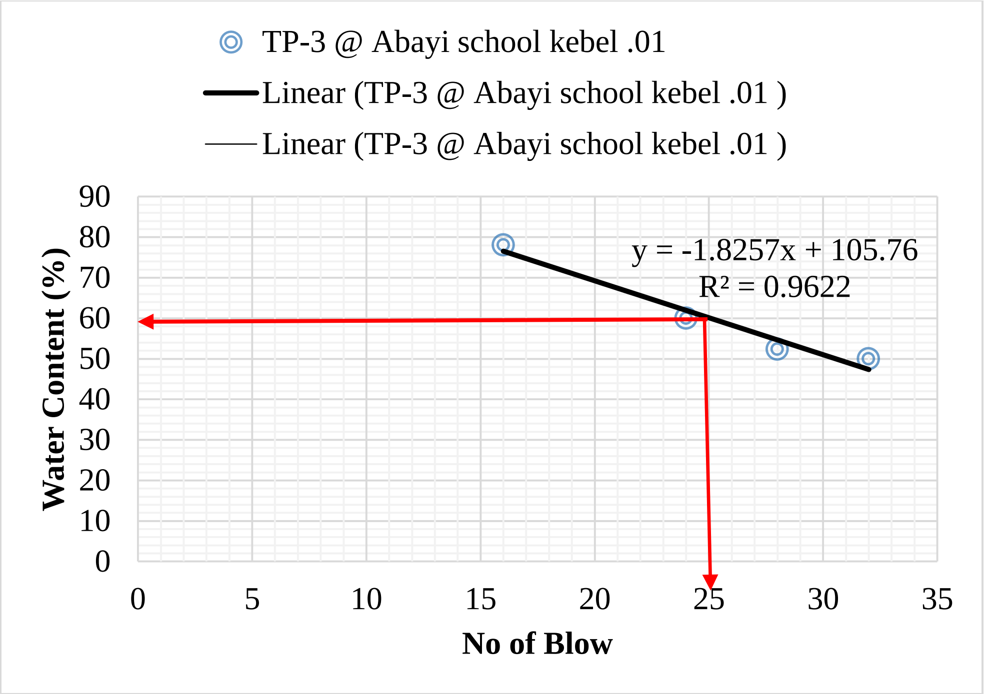
B) Moisture content

Sample no.:pit3

specimen number	1	2
Mc=mass of empty,clean+llid(g)	61	61
Mcms=mass of can,lid,and moist soil(g)	122	124
Mcds=mass of can,lid and dry soil (g)	105	106
Ms =mass of soil solids (g) =Mcds-Mc	44	45
Mw= mass of pore water (g) =Mcms-Mcds	17	18
w=water content ,w%= (mw/ms)*100	38.6364	40.00
average	39.44	

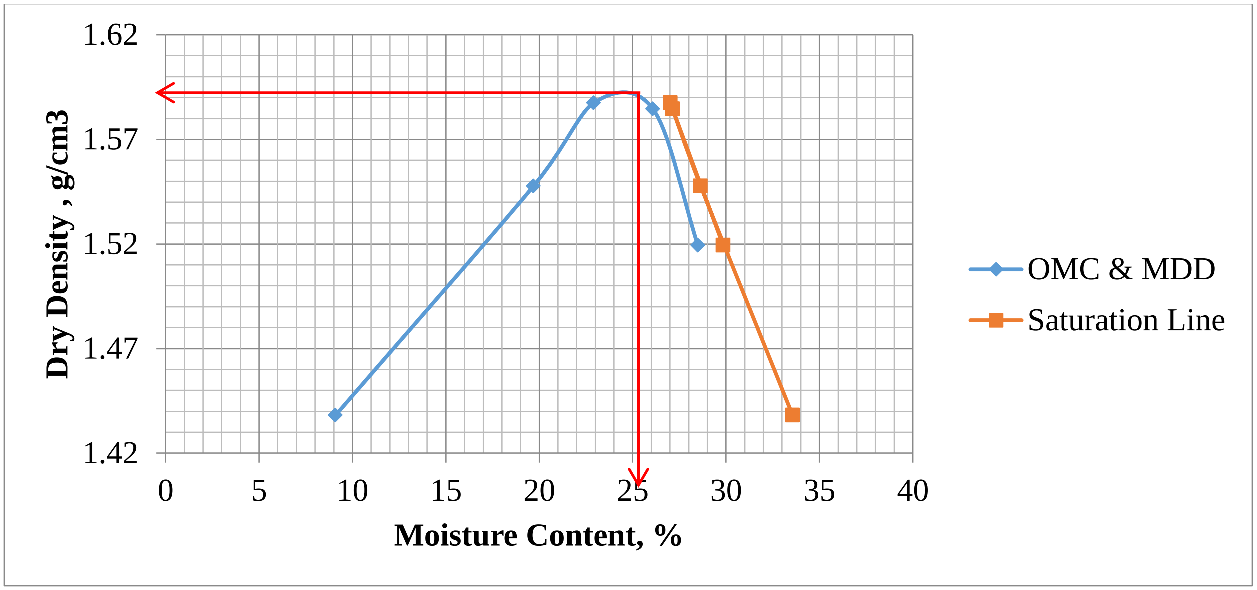
C) Atterberg Limit

sample location	Pit -3 Abayi school kebel.01,Depth @3 m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	7	7	7	7	61	61
MCMS=mass of Can,lid and moist soil (grams)	20	23	23	22	75	73.9
MCDS=mass of can,lid and dry soils(grams)	14.3	17	17.5	17	73	72
MS=mass of soil solids(g)= MCDS-MC	7.3	10	10.5	10	12	11
MW= mass of pore water (g)=MCMS-MCDS	5.7	6	5.5	5	2	1.9
W=water content,w%=(MW/MS)*100	78.0822	60	52.381	50	16.6667	17.2727
No. of drops (N)	16	24	28	32	Avg.	16.9697
LL=59.8@ Blow =25, PI=LL-PL=59.8-16.97 = 42.83						

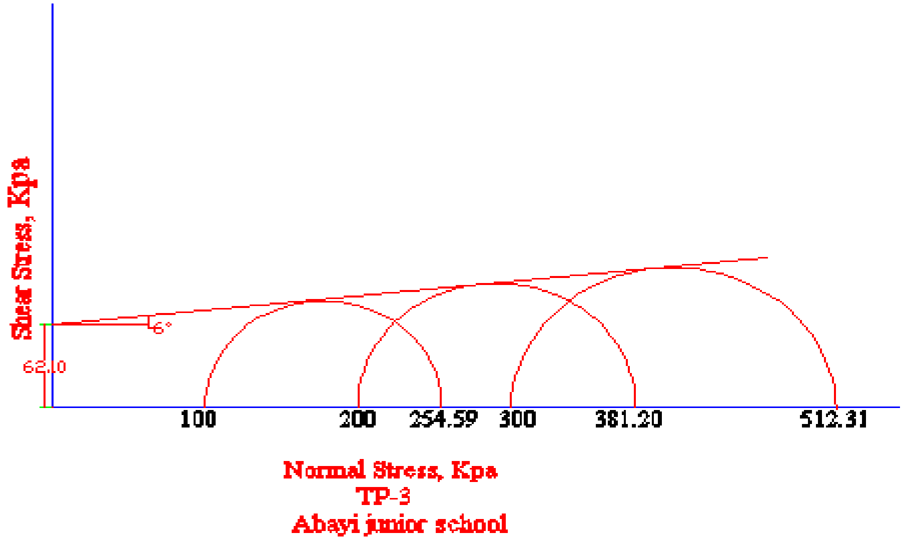

TP-3 @ Abayi school kebel .01
Linear (TP-3 @ Abayi school kebel .01)
Linear (TP-3 @ Abayi school kebel .01)
y = -1.8257x + 105.76
R² = 0.9622
Water Content (%)
No of Blow
0
10
20
30
40
50
60
70
80
90
0
5
10
15
20
25
30
35

D) Compaction

water content determination for TP 3:@3m B/Hora Abayi school						
sample location	TP-3@ B/Hora Abayi school 01 Keb., depth 3m					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	
MC = Mass of empty, clean can + lid (grams)	94	95	60	61	61	
MCMS = Mass of can, lid, and moist soil (grams)	178	174	119	119	120.75	
MCDS = Mass of can, lid, and dry soil (grams)	171	161	108	107	107.5	
MS = Mass of soil solids (grams)	77	66	48	46	46.5	
MW = Mass of pore water (grams)	7	13	11	12	13.25	
W = Water content, w%	9.09091	19.697	22.9167	26.087	28.4946	
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	
w = Assumed water content, w%	8	16	24	32	36	
Actual average water content, w%	9.090909	19.69697	22.91667	26.08696	28.49462	
Mass of compacted soil and mold (grams)	5886	6152	6245	6289	6246	
Mass of mold (grams)	4405	4405	4405	4405	4405	
Wet mass of soil in mold (grams)	1481	1747	1840	1884	1841	
Wet density, ρ , (g/cm3)	1.568856	1.852598	1.95122	1.997879	1.95228	
Dry density, ρd , (g/cm3)	1.438118	1.54774	1.587433	1.584525	1.519348	
dry unit weight ɤd (KN/m3)	14.38118	15.4774	15.87433	15.84525	15.19348	
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.78**	
ɤd	1.438118	1.54774	1.587433	1.584525	1.519348	
water content(%)	33.5641	28.6391	27.02357	27.13918	29.8465	
		MDD=15.9		OMC=24.8		



E) UU-Triaxial test



4. Sample 4

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	TP-4 Mekane iyesus church @2.5m	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
	Test Trials	
Description	1	2
Pycnometer bottle number	B2	B8
Mass empty Pycnometer, Mp (g)	45.6	54.2
Mass of soil, Ms (g)	25	25
Testing Temperature, T (^{0}C)	17	17.4
Mass of pycnometer + water, Mpw (g)	145.31	153.92
Mass of pycnometer + water + Soil, Mpws (g)	161.40	169.90
(Mpw – Mpws + Ms) (g)	8.91	9.01
GS(at Testing Temprature)	2.80	2.77
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.80	2.77
GS (at 20^{0}c) Avg.		2.79

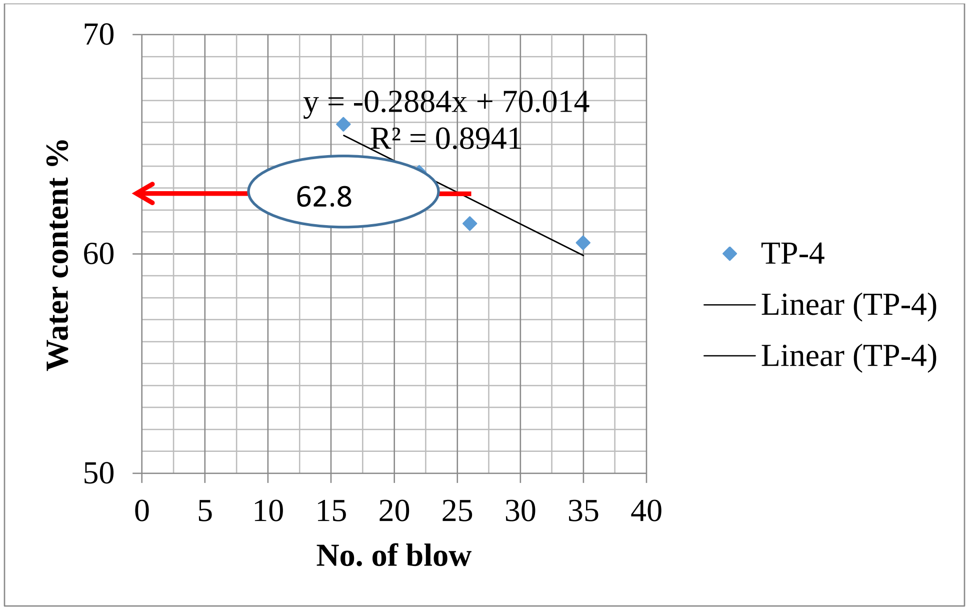
B) Moisture content

Sample no:pit4

specimen number	1	2
Mc=mass of empty,clean+llid(g)	62	62
Mcms=mass of can,lid,and moist soil(g)	144	141
Mcds=mass of can,lid and dry soil (g)	116	114
Ms =mass of soil solids (g) =Mcds-Mc	54	52
Mw= mass of pore water (g) =Mcms-Mcds	28	27
w=water content ,w%= (mw/ms)*100	51.8519	51.92
average	52.00	

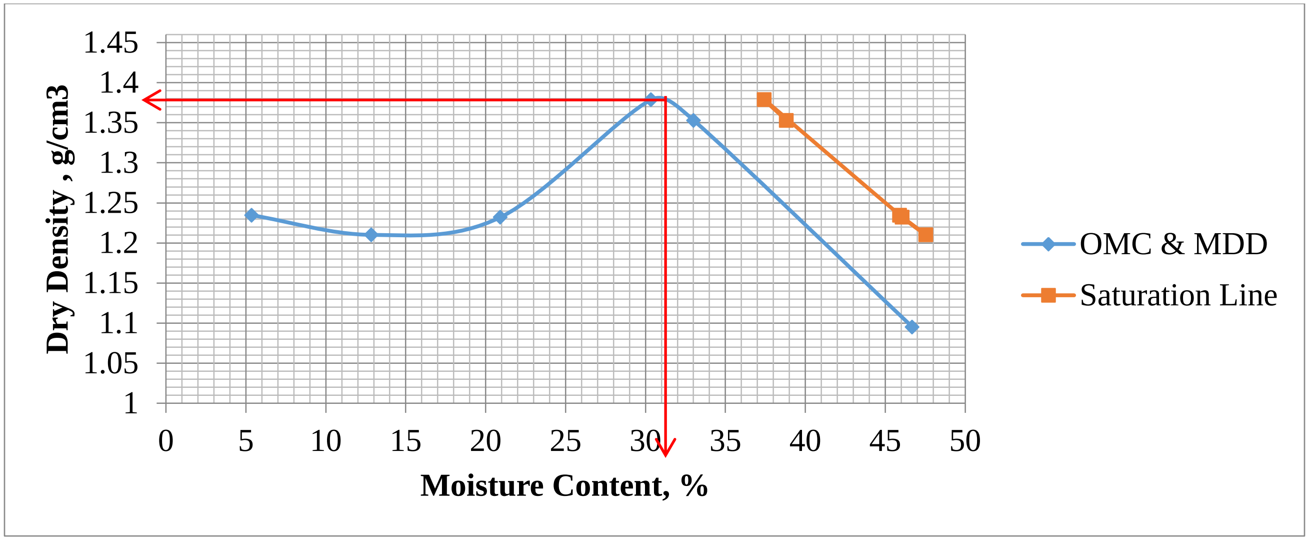
C) Atterberg Limit

sample location	Pit -4Mekane iyesus church, Depth @2.5m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	33.5	33.2	33.4	33.5	33.3	33.5
MCMS=mass of Can,lid and moist soil (grams)	52.6	49.5	53.7	48.1	40.5	40
MCDS=mass of can,lid and dry soils(grams)	45.4	43.3	45.8	42.3	39	38.7
MS=mass of soil solids(g)= MCDS-MC	11.9	10.1	12.4	8.8	5.7	5.2
MW= mass of pore water (g)=MCMS-MCDS	7.2	6.2	7.9	5.8	1.5	1.3
W=water content,w%=(MW/MS)*100	60.5042	61.3861	63.7097	65.9091	26.3158	25
No. of drops (N)	35	26	22	16	Avg.	25.6579
LL=62.8@ Blow =25, PI=LL-PL=62.8-37.049 = 25.75						

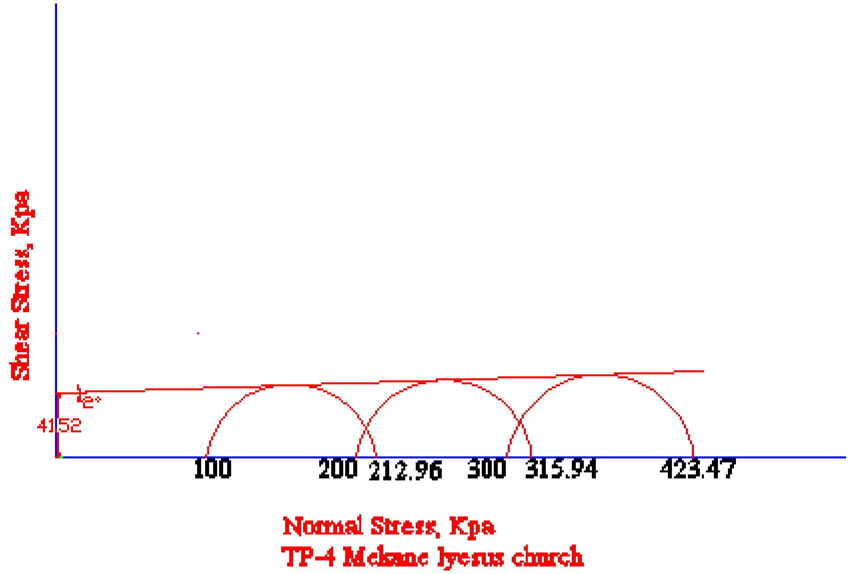

y = -0.2884x + 70.014
R² = 0.8941
62.8
Water content %
No. of blow
70
60
50
0
5
10
15
20
25
30
35
40
TP-4
Linear (TP-4)
Linear (TP-4)

D) Compaction

water content determination for TP 4:@2.5m Mekane Iyesus church						
sample location	Mekane Iyesus church					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	6
MC = Mass of empty, clean can + lid (grams)	60	62	61	61	61	61
MCMS = Mass of can, lid, and moist soil (grams)	109	125	94	106	118	117.5
MCDS = Mass of can, lid, and dry soil (grams)	104	115	87	94	102	98
MS = Mass of soil solids (grams)	44	53	26	33	41	37
MW = Mass of pore water (grams)	5	10	7	12	16	19.5
W = Water content, w%	11.3636	18.8679	26.9231	36.3636	39.0244	52.7027
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%	8	16	24	32	40	48
Actual average water content, w%	5.363636	12.86792	20.92308	30.36364	33.02439	46.7027
Mass of compacted soil and mold (grams)	5633	5693	5810	6100	6102	5920
Mass of mold (grams)	4405	4405	4405	4405	4405	4405
Wet mass of soil in mold (grams)	1228	1288	1405	1695	1697	1515
Wet density, ρ , (g/cm3)	1.300847	1.365854	1.489926	1.797455	1.799576	1.606575
Dry density, ρd , (g/cm3)	1.234627	1.210134	1.232127	1.378801	1.352816	1.095123
dry unit weight ɤd (KN/m3)	12.34627	12.10134	12.32127	13.78801	13.52816	10.95123
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.85**	
ɤd	12.34627	12.10134	12.32127	13.78801	13.52816	
water content(%)	45.90843	47.54773	46.07275	37.43907	38.83214	
		MDD=13.8		OMC=30.9		



E) UU-Triaxial test



5. Sample 5

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	TP-5 Bule Hora University @2m	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	A1	B9
Mass empty Pycnometer, Mp (g)	47.60	50.80
Mass of soil, Ms (g)	25.00	25.00
Testing Temperature, T (^{0}C)	17.60	17.60
Mass of pycnometer + water, Mpw (g)	147.41	150.61
Mass of pycnometer + water + Soil, Mpws (g)	163.40	166.60
(Mpw – Mpws + Ms) (g)	9.01	9.01
GS(at Testing Temprature)	2.77	2.78
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.78	2.78
GS (at 20^{0}c) Avg.		2.78

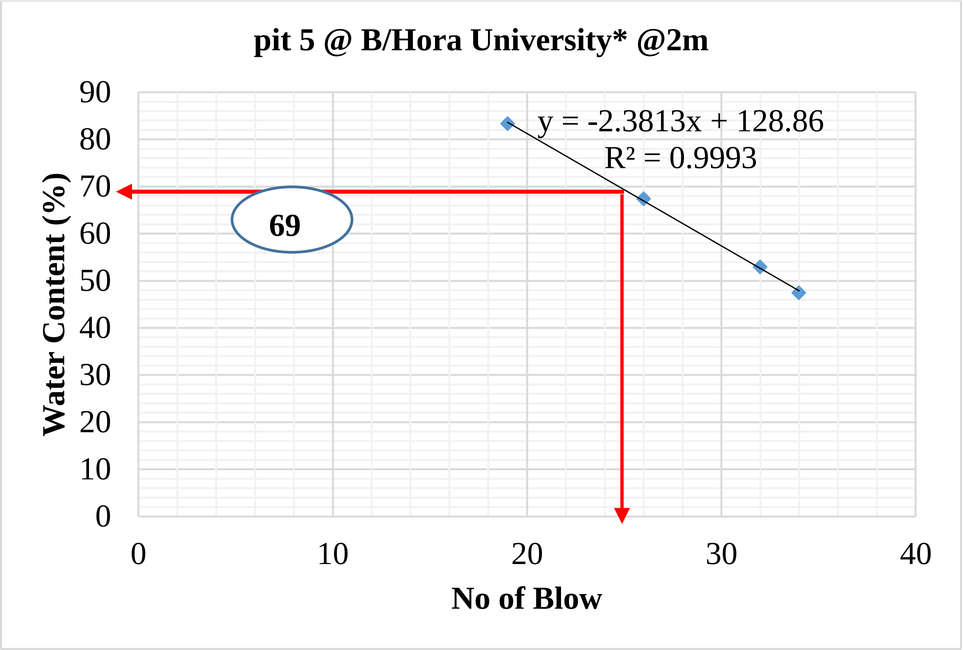
B) Moisture content

Sample no:pit5

specimen number	1	2
Mc=mass of empty,clean+llid(g)	97	96
Mcms=mass of can,lid,and moist soil(g)	203	197
Mcds=mass of can,lid and dry soil (g)	171	166
Ms =mass of soil solids (g) =Mcds-Mc	74	70
Mw= mass of pore water (g) =Mcms-Mcds	32	31
w=water content ,w%= (mw/ms)*100	43.2432	44.29
average	43.86	

C) Atterberg Limit

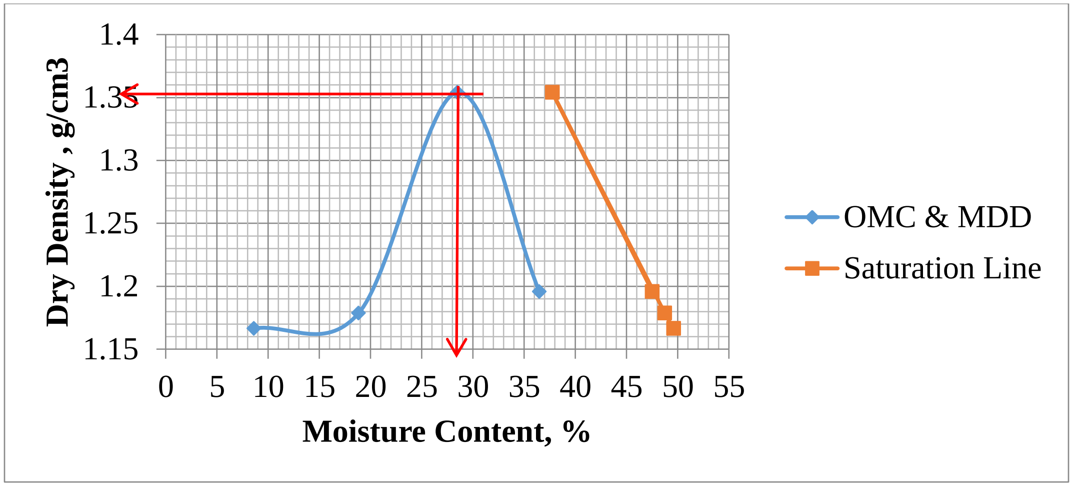
sample location	pit 5@ B/Hora Woreda Court, Depth @2m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	60	60	60	60	60	60
MCMS=mass of Can,lid and moist soil (grams)	82	79	83	86	70	75.5
MCDS=mass of can,lid and dry soils(grams)	72	71.35	75.6	77	68	72.5
MS=mass of soil solids(g)= MCDS-MC	12	11.35	15.6	17	8	12.5
MW= mass of pore water (g)=MCMS-MCDS	10	7.65	7.4	9	2	3
W=water content,w%=(MW/MS)*100	83.3333	67.4009	47.4359	52.9412	25	24
No. of drops (N)	19	26	34	32	Avg.	24.5
LL=69@ Blow =25, PI=LL-PL=69-24.5 =44.5						



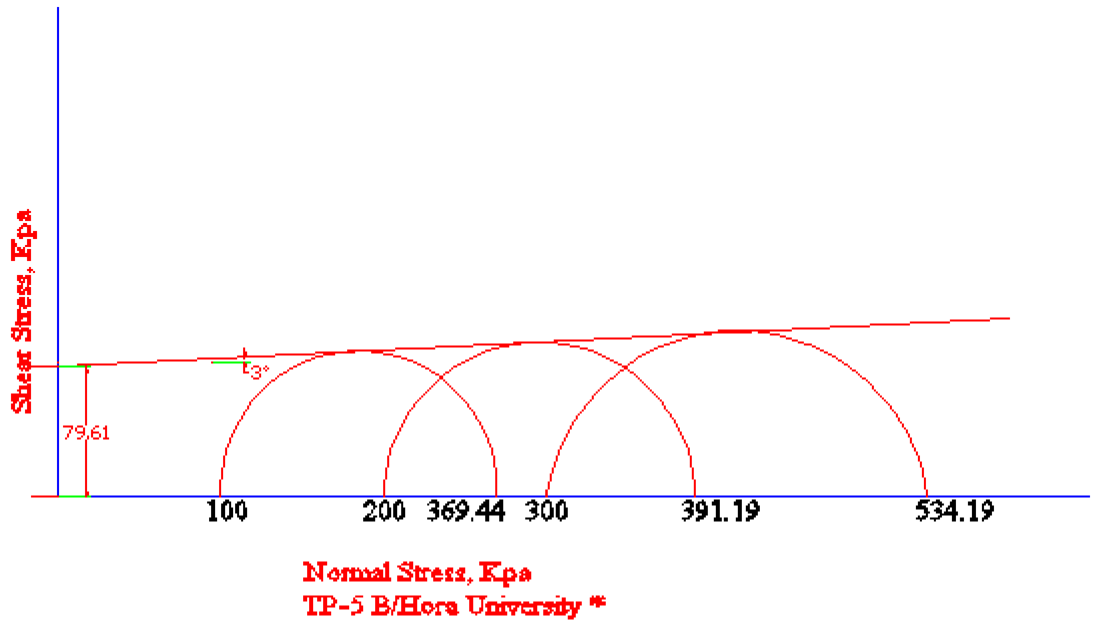
pit 5 @ B/Hora University* @2m
y = -2.3813x + 128.86
R² = 0.9993
69
Water Content (%)
No of Blow
0
10
20
30
40
50
60
70
80
90

D) Compaction

water content determination for pit 5:@2m B/HoraUniversity						
sample location	TP-5@ B/Hora University*, depth 2m					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4		
MC = Mass of empty, clean can + lid (grams)	33.7	32.9	33	33.2		
MCMS = Mass of can, lid, and moist soil (grams)	177.2	165.6	150.4	151.6		
MCDS = Mass of can, lid, and dry soil (grams)	158.9	139.2	120.3	116.3		
MS = Mass of soil solids (grams)	125.2	106.3	87.3	83.1		
MW = Mass of pore water (grams)	18.3	26.4	30.1	35.3		
W = Water content, w%	14.6166	24.8354	34.4788	42.4789		
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4		
w = Assumed water content, w%	8	16	24	32		
Actual average water content, w%	8.616613	18.83537	28.47881	36.47894		
Mass of compacted soil and mold (grams)	5838.9	5973	6313.6	6205.5		
Mass of mold (grams)	4566.1	4566.1	4566.1	4566.1		
Wet mass of soil in mold (grams)	1272.8	1406.9	1747.5	1639.4		
Wet density, ρ , (g/cm3)	1.267161	1.400667	1.739758	1.632137		
Dry density, ρd , (g/cm3)	1.166637	1.178662	1.354121	1.195889		
dry unit weight ɤd (KN/m3)	11.66637	11.78662	13.54121	11.95889		
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.77**	
ɤd	11.66637	11.78662	13.54121	11.95889		
water content(%)	49.61541	48.7409	37.74758	47.5187		
		MDD=1.354		OMC=28.5		



E) UU-Triaxial test



6. Sample 6

A) Specific gravity

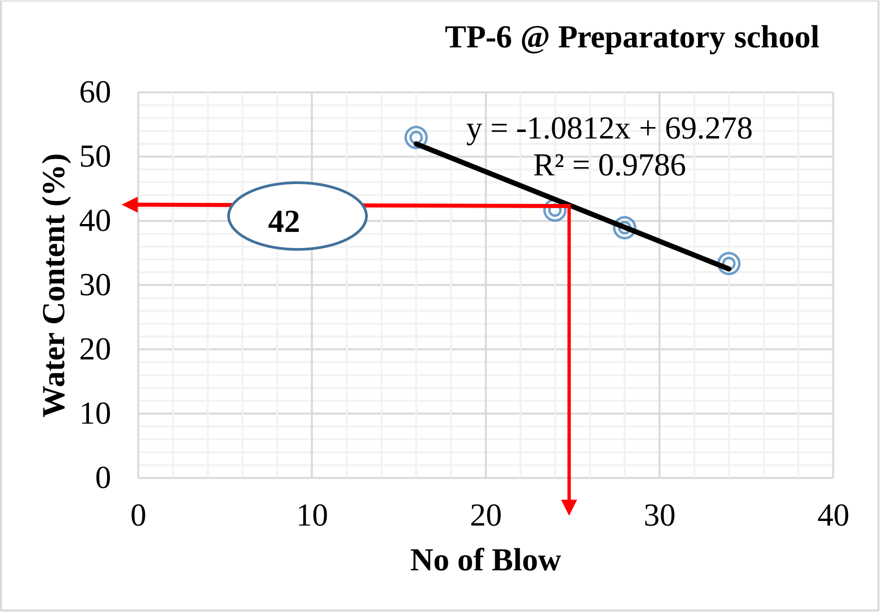
Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Preparatory School	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B2	B8
Mass empty Pycnometer, Mp (g)	45.6	54.2
Mass of soil, Ms (g)	25	25
Testing Temperature, T (^{0}C)	18	19
Mass of pycnometer + water, Mpw (g)	151.86	153.35
Mass of pycnometer + water + Soil, Mpws (g)	167.50	169.20
(Mpw – Mpws + Ms) (g)	9.36	9.15
GS(at Testing Temprature)	2.67	2.73
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.67	2.73
GS (at 20^{0}c) Avg.		2.70

B) Moisture content

specimen number	1	2
Mc=mass of empty,clean+llid(g)	97	96
Mcms=mass of can,lid,and moist soil(g)	194	191
Mcds=mass of can,lid and dry soil (g)	176	172
Ms =mass of soil solids (g) =Mcds-Mc	79	76
Mw= mass of pore water (g) =Mcms-Mcds	18	19
w=water content ,w%= (mw/ms)*100	23.00	25.00
average	24.00	

C) Atterberg Limit

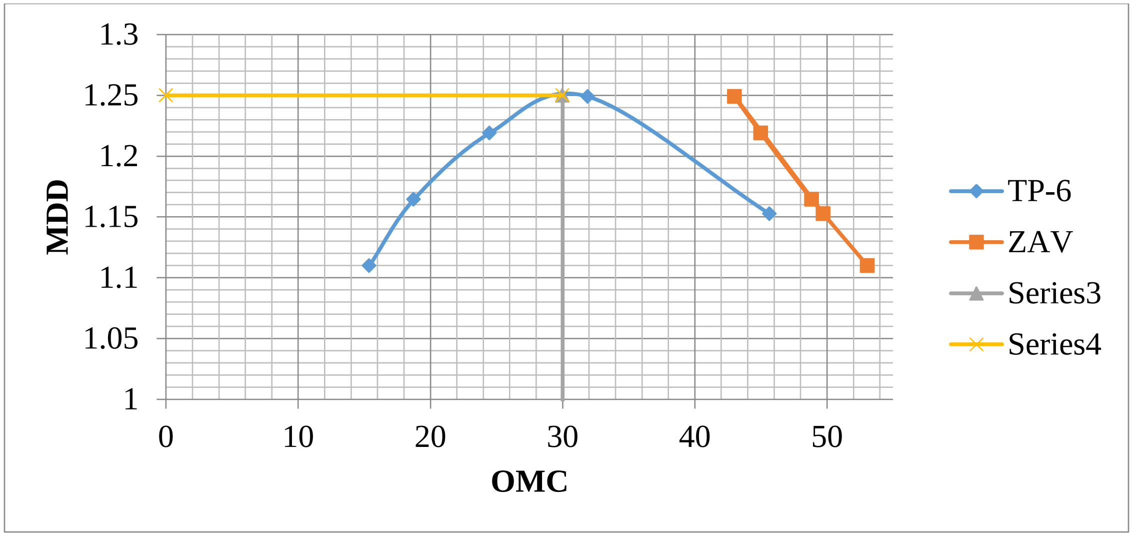
sample location	Pit -6 preparatory school ,Depth @2.5 m					
sample description	Reddish brown soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	61	61	61	61	61	61
MCMS=mass of Can,lid and moist soil (grams)	74	78	89	86	73	72.75
MCDS=mass of can,lid and dry soils(grams)	69.5	73	82	79	71	71
MS=mass of soil solids(g)= MCDS-MC	8.5	12	21	18	10	10
MW= mass of pore water (g)=MCMS-MCDS	4.5	5	7	7	2	1.75
W=water content,w%=(MW/MS)*100	52.9412	41.6667	33.3333	38.8889	20	17.5
No. of drops (N)	16	24	34	28	Avg.	18.75
LL=42@ Blow =25, PI=LL-PL=42-18.75 = 23.25						


TP-6 @ Preparatory school
y = -1.0812x + 69.278
R² = 0.9786
42
Water Content (%)
No of Blow
0
10
20
30
40
50
60
0
10
20
30
40

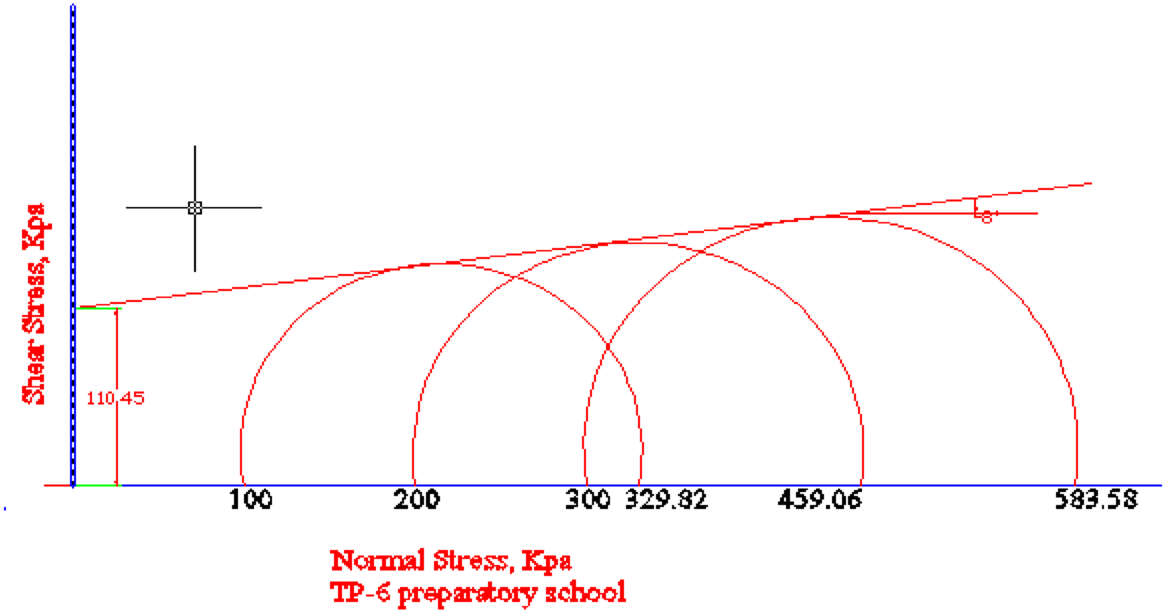
D) Compaction

water content determination for pit 6:@2.5m

sample location	Preparatory school @2.5m					
sample description	Reddish clay soil					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	
MC = Mass of empty, clean can + lid (grams)	60	60	61	60	60	
MCMS = Mass of can, lid, and moist soil (grams)	90	117	122	122	127	
MCDS = Mass of can, lid, and dry soil (grams)	86	108	110	107	106	
MS = Mass of soil solids (grams)	26	48	49	47	46	
MW = Mass of pore water (grams)	4	9	12	15	21	
W = Water content, w%	15.38	18.75	24.49	31.91	45.65	
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	
w = Assumed water content, w%	8	12	16	20	24	
Actual average water content, w%	9.38462	12.75	18.4898	25.9149	37.6522	
Mass of compacted soil and mold (grams)	5551	5643	5767	5888	5901	
Mass of mold (grams)	4405	4405	4405	4405	4405	
Wet mass of soil in mold (grams)	1146	1238	1362	1483	1496	
Wet density, ρ , (g/cm3)	1.21	1.31	1.44	1.57	1.59	
Dry density, ρd , (g/cm3)	1.11	1.16	1.22	1.25	1.15	
dry unit weight ɤd (KN/m3)	11.10	11.64	12.19	12.49	11.52	
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.7**	
ɤd	11.10	11.64	12.19	12.49	11.52	
water content(%)	53.07	48.85	45.00	43.03	49.73	
		MDD=12.5		OMC=30		



E) UU-Triaxial test



7. Sample 7

A) Specific gravity

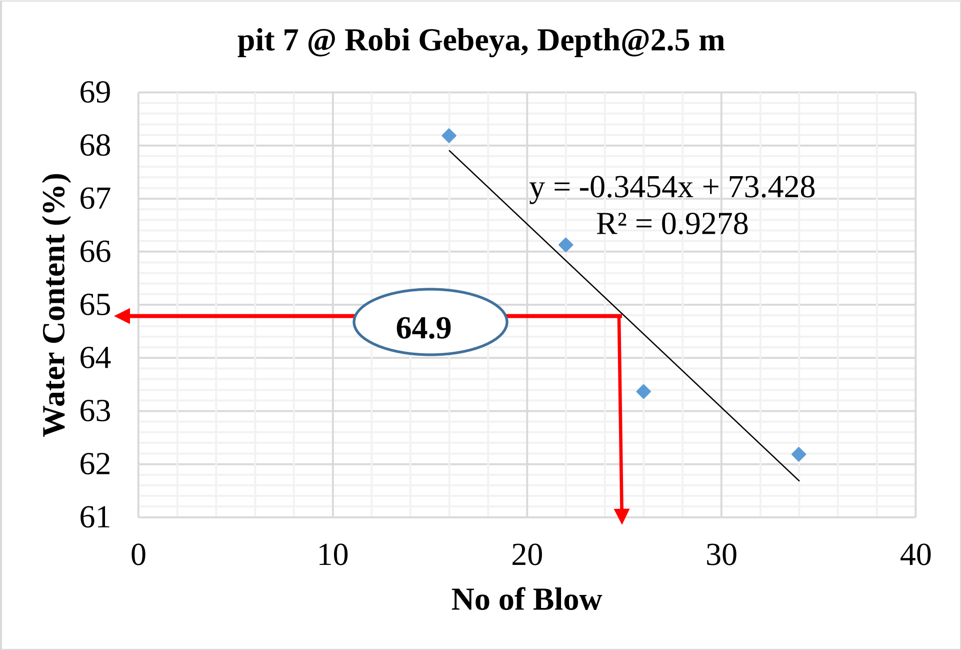
Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Robi Gebeya	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B1	B7
Mass empty Pycnometer, Mp (g)	44	44
Mass of soil, Ms (g)	10.3	10
Testing Temperature, T (^{0}C)	21	21
Mass of pycnometer + water, Mpw (g)	154	153
Mass of pycnometer + water + Soil, Mpws (g)	160	159.63
(Mpw – Mpws + Ms) (g)	4.3	3.37
GS(at Testing Temprature)	2.40	2.97
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.39	2.97
GS (at 20^{0}c) Avg.		2.68

B) Moisture content

specimen number	1	2
Mc=mass of empty,clean+llid(g)	96	96
Mcms=mass of can,lid,and moist soil(g)	200	197
Mcds=mass of can,lid and dry soil (g)	169	167
Ms =mass of soil solids (g) =Mcds-Mc	73	71
Mw= mass of pore water (g) =Mcms-Mcds	31	30
w=water content ,w%= (mw/ms)*100	42.4658	42.25
average	42.36	

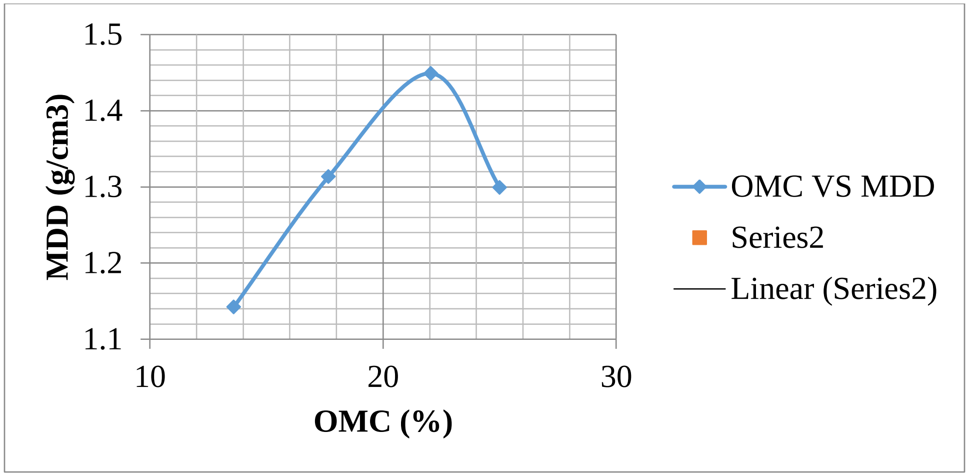
C) Atterberg Limit

sample location	Pit -7 Robi Gebeya ,Depth @2.75m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	33.5	33.2	33.4	33.5	33.3	33.3
MCMS=mass of Can,lid and moist soil (grams)	52.8	49.7	54	48.3	39	40.25
MCDS=mass of can,lid and dry soils(grams)	45.4	43.3	45.8	42.3	38	39
MS=mass of soil solids(g)= MCDS-MC	11.9	10.1	12.4	8.8	4.7	5.7
MW= mass of pore water (g)=MCMS-MCDS	7.4	6.4	8.2	6	1	1.25
W=water content,w%=(MW/MS)*100	62.1849	63.3663	66.129	68.1818	21.2766	21.9298
No. of drops (N)	34	26	22	16	Avg.	21.6032
LL=64.9@ Blow =25, PI=LL-PL=64.9-21.6 = 43.3						

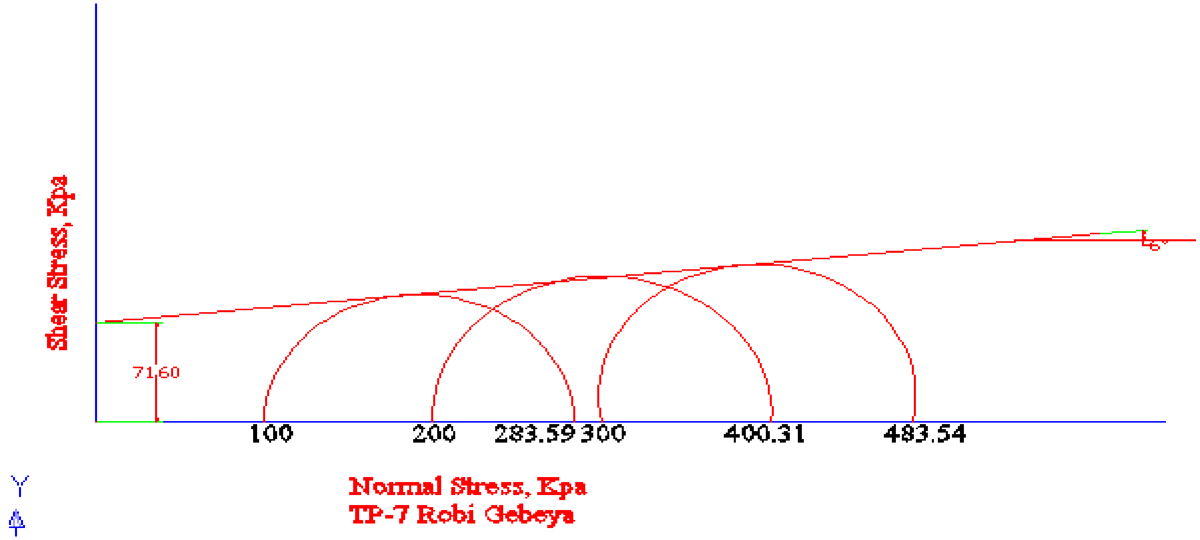

pit 7 @ Robi Gebeya, Depth@2.5 m
Water Content (%)
69
68
67
66
65
64
63
62
61
y = -0.3454x + 73.428
R² = 0.9278
64.9
0
10
20
30
40
No of Blow

D) Compaction

water content determination for pit 7:@2.75m Robi Gebeya						
sample location	Robi Gebeya					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4		
MC = Mass of empty, clean can + lid (grams)	58.1	57	58.2	56		
MCMS = Mass of can, lid, and moist soil (grams)	385.7	390	418	406		
MCDS = Mass of can, lid, and dry soil (grams)	341.5	340	353	336		
MS = Mass of soil solids (grams)	283.4	283	294.8	280		
MW = Mass of pore water (grams)	44.2	50	65	70		
W = Water content, w%	**15.5963**	**17.6678**	**22.0488**	**25**		
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4		
w = Assumed water content, w%	8	16	24	32		
Actual average water content, w%	13.59633	17.66784	22.04885	25		
Mass of compacted soil and mold (grams)	5869.4	6118.7	6342.5	6197.4		
Mass of mold (grams)	4566.1	4566.1	4566.1	4566.1		
Wet mass of soil in mold (grams)	1303.3	1552.6	1776.4	1631.3		
Wet density, ρ , (g/cm3)	1.297526	1.545722	1.76853	1.624073		
Dry density, ρd , (g/cm3)	1.142225	1.313631	1.449035	1.299258		
dry unit weight ɤd (KN/m3)	11.42225	13.13631	14.49035	12.99258		
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.68**	
ɤd	11.42225	13.13631	14.49035	12.99258		
water content(%)	51.05205	39.62852	32.51511	40.47064		
		MDD=14.5		OMC=22		



E) UU-Triaxial test



8. Sample 8

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Around Madiya	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
Description	**Test Trials**	
	1	2
Pycnometer bottle number	B1	B7
Mass empty Pycnometer, Mp (g)	44	44
Mass of soil, Ms (g)	10	10
Testing Temperature, T (^{0}C)	21	21
Mass of pycnometer + water, Mpw (g)	153	153
Mass of pycnometer + water + Soil, Mpws (g)	159.25	159.2
(Mpw – Mpws + Ms) (g)	3.75	3.8
GS(at Testing Temprature)	2.67	2.63
Correction Factor	0.99	0.99979
GS (at 20^{0}c)	2.666	2.631
GS (at 20^{0}c) Avg.		2.65

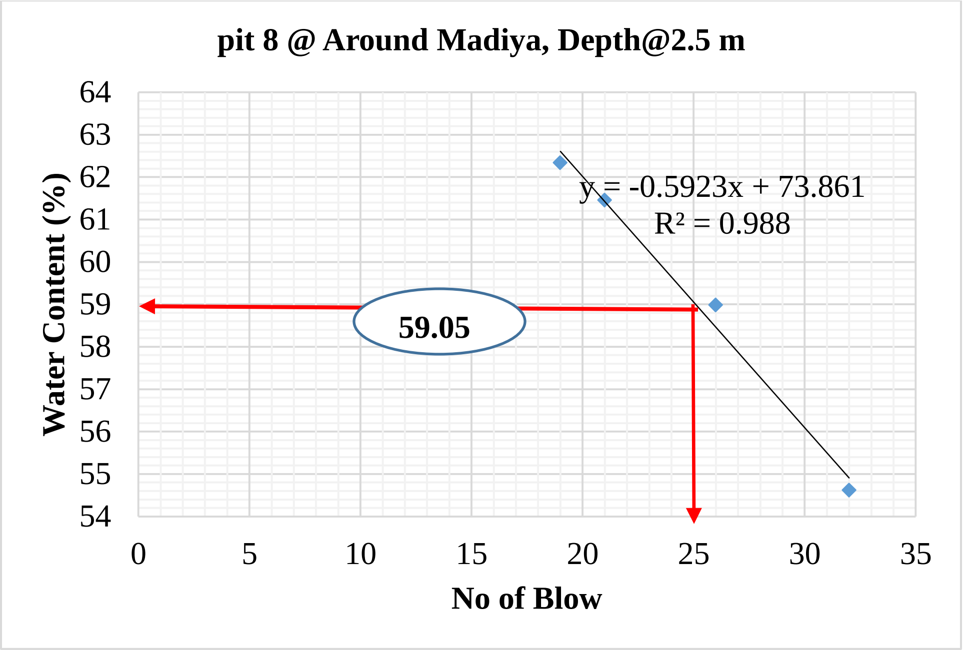
B) Moisture content

Pit8:

specimen number	1	2
Mc=mass of empty,clean+llid(g)	96	96
Mcms=mass of can,lid,and moist soil(g)	183	191
Mcds=mass of can,lid and dry soil (g)	161	167
Ms =mass of soil solids (g) =Mcds-Mc	65	71
Mw= mass of pore water (g) =Mcms-Mcds	22	24
w=water content ,w%= (mw/ms)*100	33.8462	33.80
average	33.82	

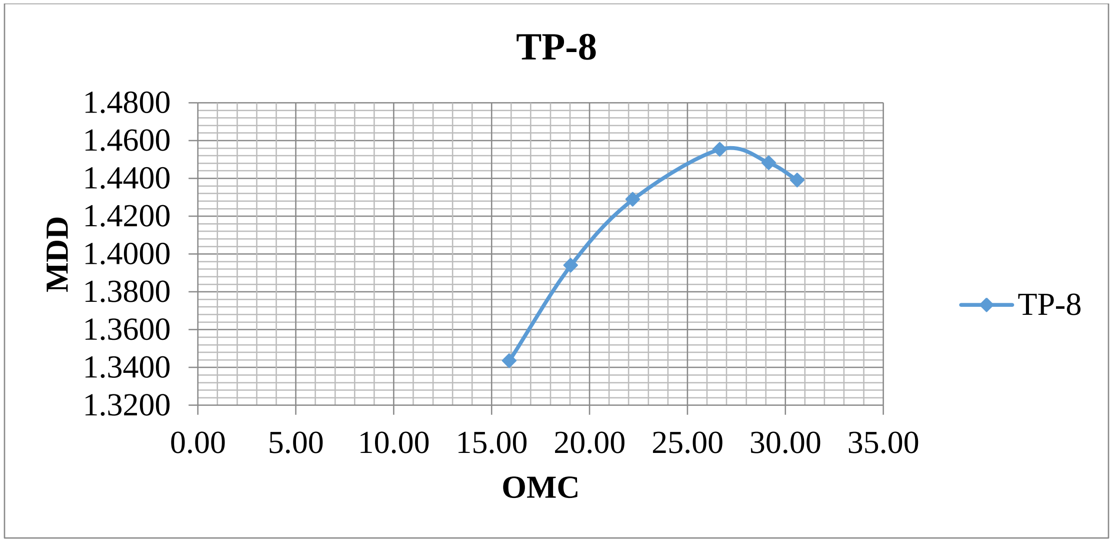
C) Atterberg Limit

sample location	Pit -8 Around madiya ,Depth @2.5m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	33.5	33.2	33.4	33.5	33.3	33.3
MCMS=mass of Can,lid and moist soil (grams)	51.9	49	52	46	46	47
MCDS=mass of can,lid and dry soils(grams)	45.4	43.138	44.92	41.2	43.5	44.2
MS=mass of soil solids(g)= MCDS-MC	11.9	9.938	11.52	7.7	10.2	10.9
MW= mass of pore water (g)=MCMS-MCDS	6.5	5.862	7.08	4.8	2.5	2.8
W=water content,w%=(MW/MS)*100	54.6218	58.9857	61.4583	62.3377	24.5098	25.6881
No. of drops (N)	32	26	21	19	Avg.	25.0989
LL=59.05@ Blow =25, PI=LL-PL=59.05-25.099 = 33.955						

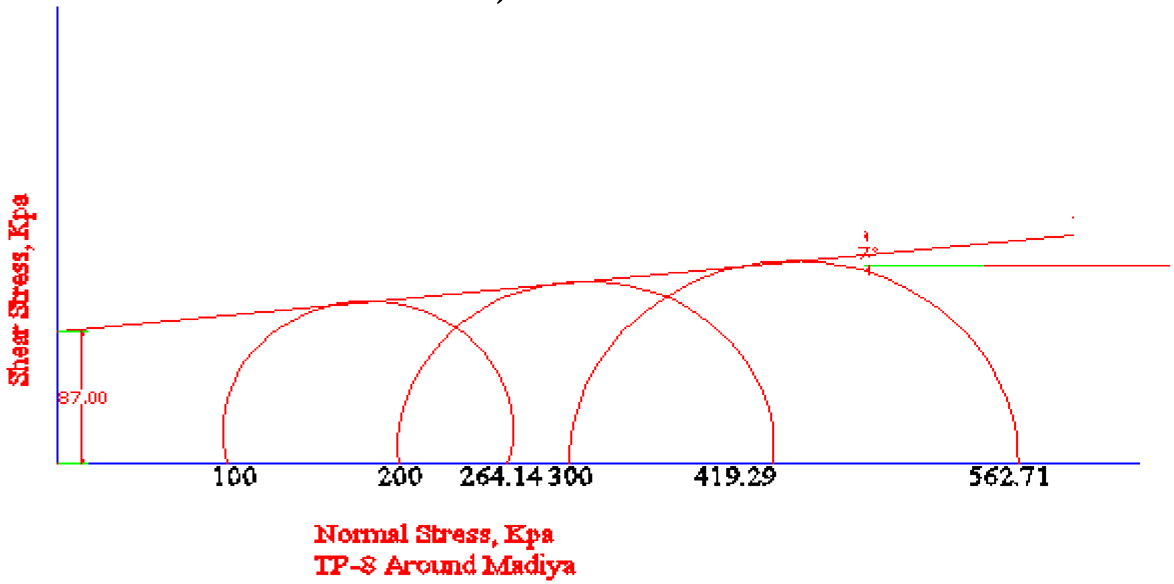

pit 8 @ Around Madiya, Depth@2.5 m
Water Content (%)
No of Blow
y = -0.5923x + 73.861
R² = 0.988
59.05
64
63
62
61
60
59
58
57
56
55
54
0
5
10
15
20
25
30
35

D) Compaction

water content determination for pit 8:@2.5m						
sample location	**Around Madiya**					
sample description	**Reddish colour**					
method of test used	**ASTM D 698**					
compacted soil- sample no	1	2	3	4	5	6
MC = Mass of empty, clean can + lid (grams)	8	8	8	9	7	8
MCMS = Mass of can, lid, and moist soil (grams)	59	58	52	66	69	72
MCDS = Mass of can, lid, and dry soil (grams)	52	50	44	54	55	57
MS = Mass of soil solids (grams)	44	42	36	45	48	49
MW = Mass of pore water (grams)	7	8	8	12	14	15
W = Water content, w%	15.91	19.05	22.22	26.67	29.17	30.61
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%	8	16	24	32	40	48
Actual average water content, w%	9.90909	13.0476	16.2222	20.6667	23.1667	24.6122
Mass of compacted soil and mold (grams)	5799	5891	5971	6061	6087	6096
Mass of mold (grams)	4405	4405	4405	4405	4405	4405
Wet mass of soil in mold (grams)	1394	1486	1566	1656	1682	1691
Wet density, ρ , (g/cm3)	1.47669	1.57582	1.66066	1.7561	1.78367	1.79321
Dry density, ρd , (g/cm3)	1.3436	1.3939	1.4289	1.4553	1.4482	1.4390
dry unit weight ɤd (KN/m3)	13.4356	13.9395	14.2886	14.5533	14.4818	14.3903
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.65**	
ɤd	13.4356	13.9395	14.2886	14.5533	14.4818	14.3903
water content(%)	36.6933	34.003	32.2498	30.9771	31.3166	31.7552
		MDD=14.56		OMC=27.5		



E) UU-Triaxial test



9. Sample 9

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Arsema	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
	Test Trials	
Description	1	2
Pycnometer bottle number	B1	B7
Mass empty Pycnometer, Mp (g)	44	44
Mass of soil, Ms (g)	10	10
Testing Temperature, T (0C)	22	22
Mass of pycnometer + water, Mpw (g)	154	155
Mass of pycnometer + water + Soil, Mpws (g)	160.55	160.9
(Mpw – Mpws + Ms) (g)	3.45	4.1
GS(at Testing Temprature)	2.898550725	2.43902439
Correction Factor	0.99957	0.99957
GS (at 20^0c)	2.897304348	2.43797561
GS (at 20^0c) Avg.		2.67

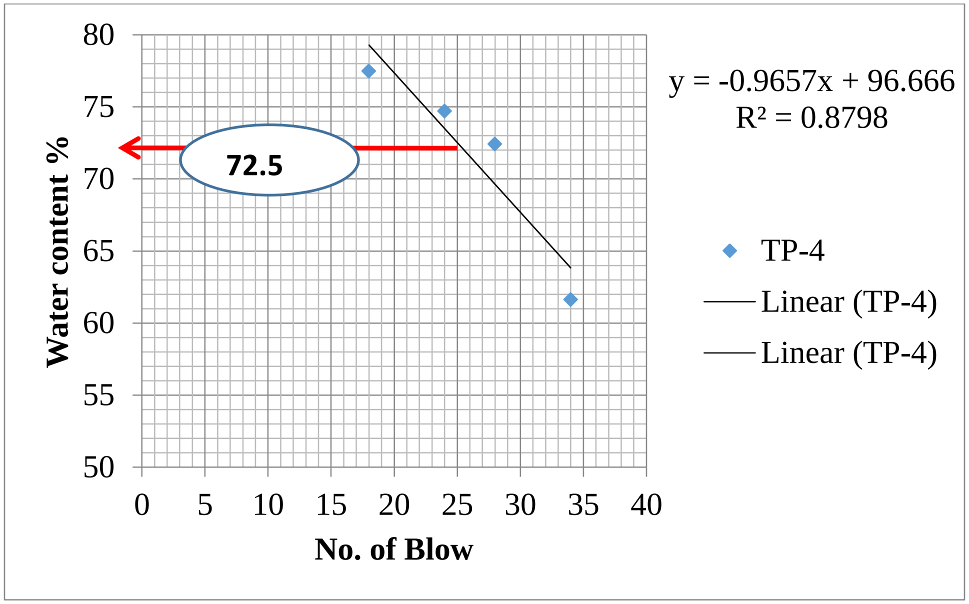
B) Moisture content

pit 9:

specimen number	1	2
Mc=mass of empty,clean+llid(g)	96	96
Mcms=mass of can,lid,and moist soil(g)	190	186
Mcds=mass of can,lid and dry soil (g)	162	159
Ms =mass of soil solids (g) =Mcds-Mc	66	63
Mw= mass of pore water (g) =Mcms-Mcds	28	27
w=water content ,w%= (mw/ms)*100	42.4242	42.86
average	42.64	

C) Atterberg Limit

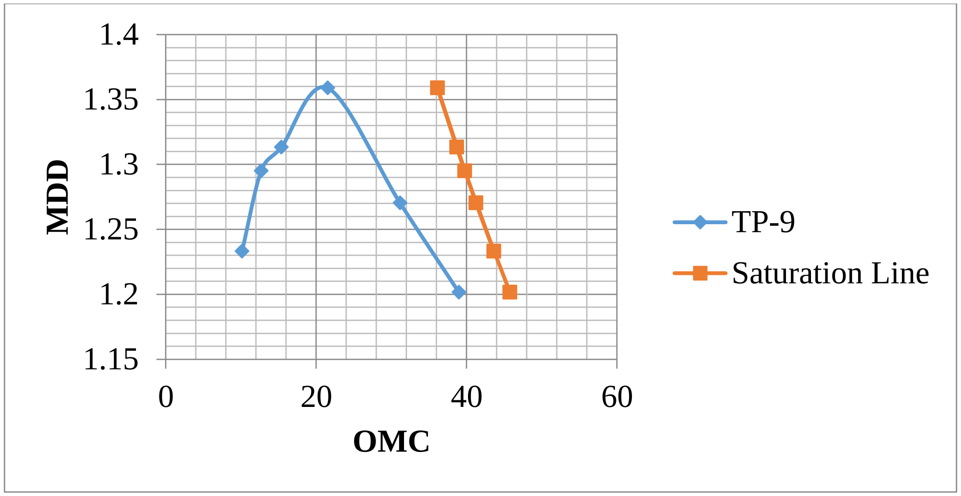
sample location	Pit -9 Arsema ,Depth @2.5m					
sample description	Red clay soil					
method of testing used	ASTM D 4318					
	LL				**PL**	
sample no.	A	B	C	D	1	2
MC=mass of empty,clean can+lid(grams)	33.5	33.2	33.4	33.5	33.3	33.3
MCMS=mass of Can,lid and moist soil (grams)	59.2	52.2	54.19	48	44	44.35
MCDS=mass of can,lid and dry soils(grams)	49.4	44.22	45.3	41.67	41.4	41.8
MS=mass of soil solids(g)= MCDS-MC	15.9	11.02	11.9	8.17	8.1	8.5
MW= mass of pore water (g)=MCMS-MCDS	9.8	7.98	8.89	6.33	2.6	2.55
W=water content,w%=(MW/MS)*100	61.6352	72.4138	74.7059	77.4786	32.0988	30
No. of drops (N)	34	28	24	18	Avg.	31.05
LL=72.5@ Blow =25, PI=LL-PL=72.5-31.05 = 42.481						


80
75
70
65
60
55
50
Water content %
72.5
0
5
10
15
20
25
30
35
40
No. of Blow
y = -0.9657x + 96.666
R² = 0.8798
TP-4
Linear (TP-4)
Linear (TP-4)

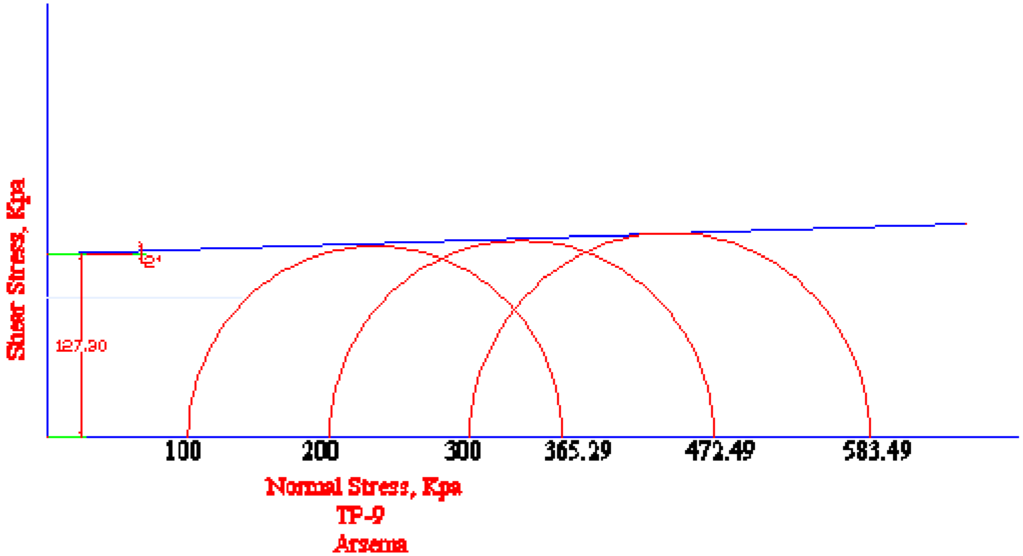
D) Compaction

water content determination for pit 9:@2.5m

sample location	Arsema					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	6
MC = Mass of empty, clean can + lid (grams)	60	61	61	60	60	61
MCMS = Mass of can, lid, and moist soil (grams)	136	132	136	139	140	143
MCDS = Mass of can, lid, and dry soil (grams)	129	124	126	125	121	120
MS = Mass of soil solids (grams)	69	63	65	65	61	59
MW = Mass of pore water (grams)	7	8	10	14	19	23
W = Water content, w%	10.1449	12.6984	15.3846	21.5385	31.1475	38.9831
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%	8	16	20	24	28	32
Actual average water content, w%	4.14493	6.69841	9.38462	15.5385	25.1475	32.9831
Mass of compacted soil and mold (grams)	5695	5793	5848	5982	6002	6010
Mass of mold (grams)	4405	4405	4405	4405	4405	4405
Wet mass of soil in mold (grams)	1290	1388	1443	1577	1597	1605
Wet density, ρ , (g/cm3)	1.28428	1.38185	1.43661	1.57001	1.58992	1.59789
Dry density, ρd , (g/cm3)	1.23317	1.2951	1.31335	1.35887	1.27044	1.20157
dry unit weight ɤd (KN/m3)	12.3317	12.951	13.1335	13.5887	12.7044	12.0157
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.67**	
ɤd	12.3317	12.951	13.1335	13.5887	12.7044	12.0157
water content(%)	43.6386	39.761	38.6878	36.1376	41.2597	45.771
		MDD=13.62		OMC=28		



E) UU-Triaxial test



10. Sample 10

A) Specific gravity

Specific Gravity of soil solids by water pycnometer		
Location and sample depth	Agricultural Beruae	
Sample Description	Dark Red, Fine Grained soils	
Method of Test Procedures	ASTM D854-98	
	Test Trials	
Description	1	2
Pycnometer bottle number	B1	B7
Mass empty Pycnometer, Mp (g)	44.00	44.00
Mass of soil, Ms (g)	10.00	10.00
Testing Temperature, T (^{0}C)	21.00	21.00
Mass of pycnometer + water, Mpw (g)	153.00	153.00
Mass of pycnometer + water + Soil, Mpws (g)	159.15	159.40
(Mpw – Mpws + Ms) (g)	3.85	3.60
GS(at Testing Temprature)	2.60	2.78
Correction Factor	1.00	1.00
GS (at 20^{0}c)	2.60	2.78
GS (at 20^{0}c) Avg.		2.69

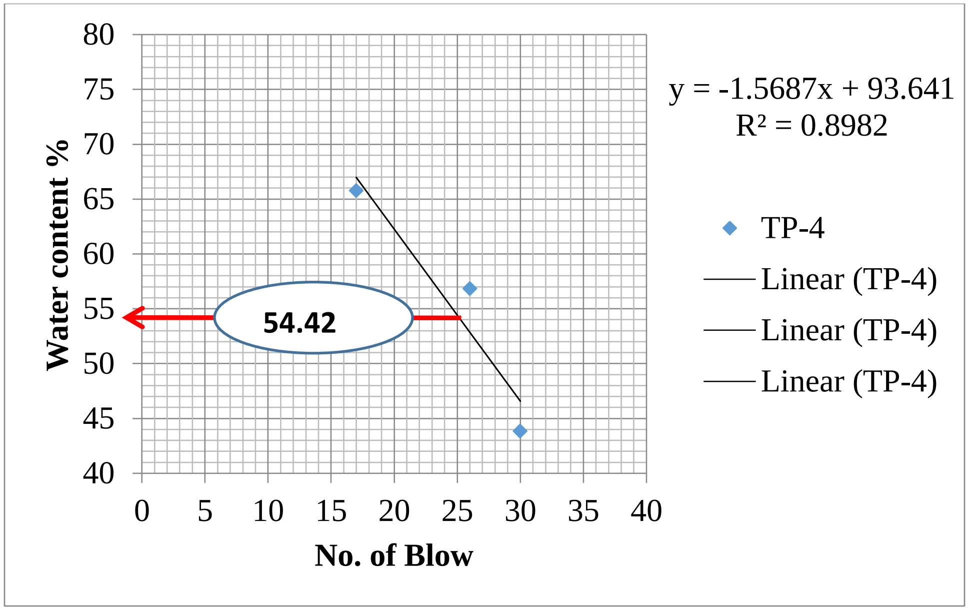
B) Moisture content

pit10

specimen number	1	2
Mc=mass of empty,clean+llid(g)	62	61
Mcms=mass of can,lid,and moist soil(g)	162	167
Mcds=mass of can,lid and dry soil (g)	130	132
Ms =mass of soil solids (g) =Mcds-Mc	68	71
Mw= mass of pore water (g) =Mcms-Mcds	32	35
w=water content ,w%= (mw/ms)*100	47.0588	49.30
average	48.18	

C) Atterberg Limit

sample location	Pit -10 Agricultural Beruae, Depth @2.5m				
sample description	Red clay soil				
method of testing used	ASTM D 4318				
	LL			**PL**	
sample no.	A	B	C	1	2
MC=mass of empty,clean can+lid(grams)	55.4	60	58.2	113.05	80.49
MCMS=mass of Can,lid and moist soil (grams)	76.4	80.7	82.4	118.4	86.5
MCDS=mass of can,lid and dry soils(grams)	70	73.2	72.8	117.4	85.3
MS=mass of soil solids(g)= MCDS-MC	14.6	13.2	14.6	4.35	4.81
MW= mass of pore water (g)=MCMS-MCDS	6.4	7.5	9.6	1	1.2
W=water content,w%=(MW/MS)*100	43.8356	56.8182	65.7534	22.9885	24.948
No. of drops (N)	30	26	17	Avg.	23.9683
LL=54.4235 @ Blow =25, PI=LL-PL=54.4235-23.97 = 30.46					

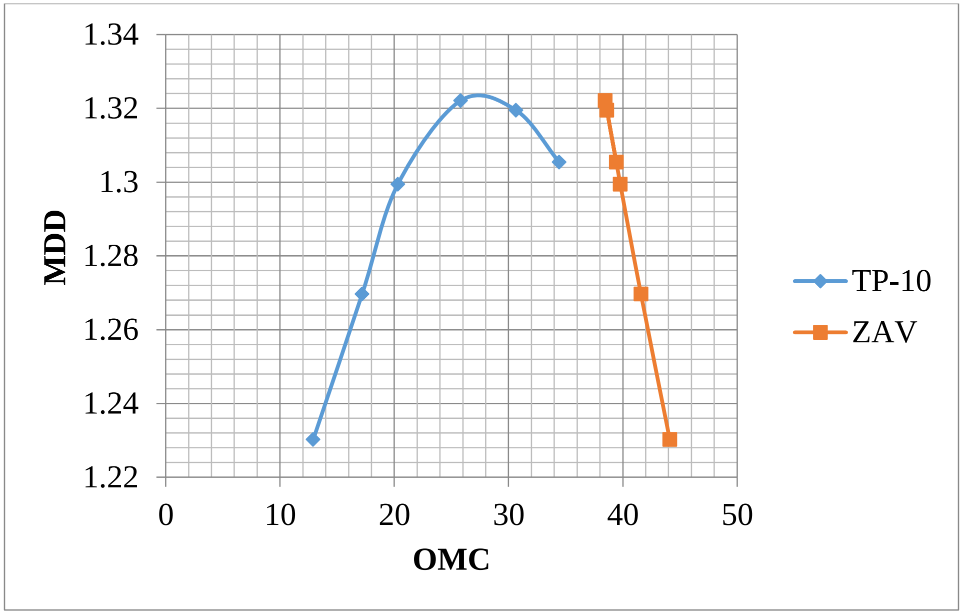


80
75
70
65
60
55
50
45
40
Water content %
0
5
10
15
20
25
30
35
40
No. of Blow
54.42
y = -1.5687x + 93.641
R² = 0.8982
TP-4
Linear (TP-4)
Linear (TP-4)
Linear (TP-4)

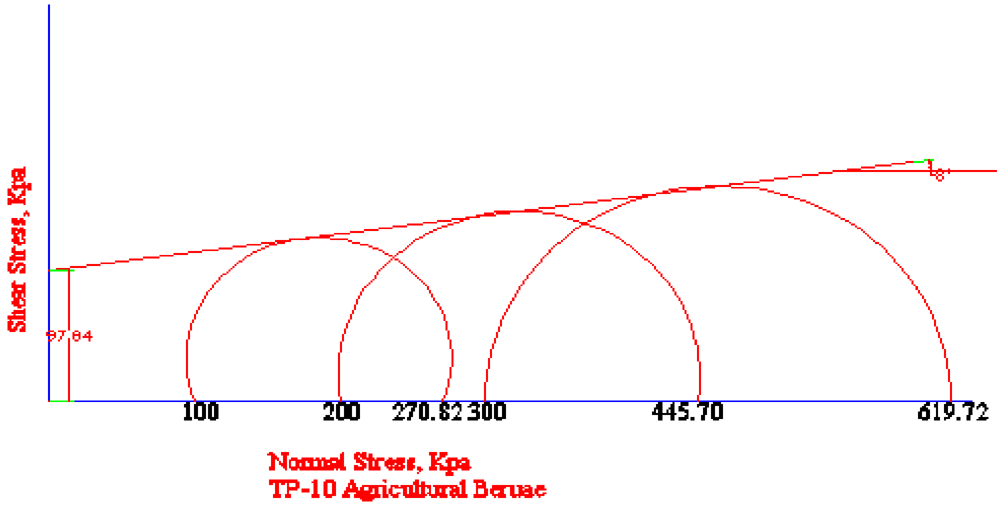
D) Compaction

water content determination for pit 10:@2.5m

sample location	TP-10@2.5m					
sample description	Reddish colour					
method of test used	ASTM D 698					
compacted soil- sample no	1	2	3	4	5	6
MC = Mass of empty, clean can + lid (grams)	60	59	59	60	59	59
MCMS = Mass of can, lid, and moist soil (grams)	130	134	136	138	140	141
MCDS = Mass of can, lid, and dry soil (grams)	122	123	123	122	121	120
MS = Mass of soil solids (grams)	62	64	64	62	62	61
MW = Mass of pore water (grams)	8	11	13	16	19	21
W = Water content, w%	12.9032	17.1875	20.3125	25.8065	30.6452	34.4262
density determination	**volume of mold=944cm3**					
Compacted Soil - Sample no.	1	2	3	4	5	6
w = Assumed water content, w%	8	16	20	24	28	32
Actual average water content, w%	6.90323	11.1875	14.3125	19.8065	24.6452	28.4262
Mass of compacted soil and mold (grams)	5726	5823	5897	5996	6057	6089
Mass of mold (grams)	4405	4405	4405	4405	4405	4405
Wet mass of soil in mold (grams)	1321	1418	1492	1591	1652	1684
Wet density, ρ , (g/cm3)	1.31515	1.41172	1.48539	1.58395	1.64468	1.67654
Dry density, ρd , (g/cm3)	1.23022	1.26967	1.29941	1.32209	1.31949	1.30545
dry unit weight ɤd (KN/m3)	12.3022	12.6967	12.9941	13.2209	13.1949	13.0545
for drawing saturation line(zero air void line S=100%	**Wsat= (ɤw/ɤd-1/Gs)100**				**Gs=2.69**	
ɤd	12.3022	12.6967	12.9941	13.2209	13.1949	13.0545
water content(%)	44.1114	41.5857	39.7832	38.463	38.6121	39.4273
		MDD=13.24		OMC=28		



E) UU-Triaxial test



APPENDIX II

I. During conducting Laboratory test

Wet sieve

Hydrometer Analysis

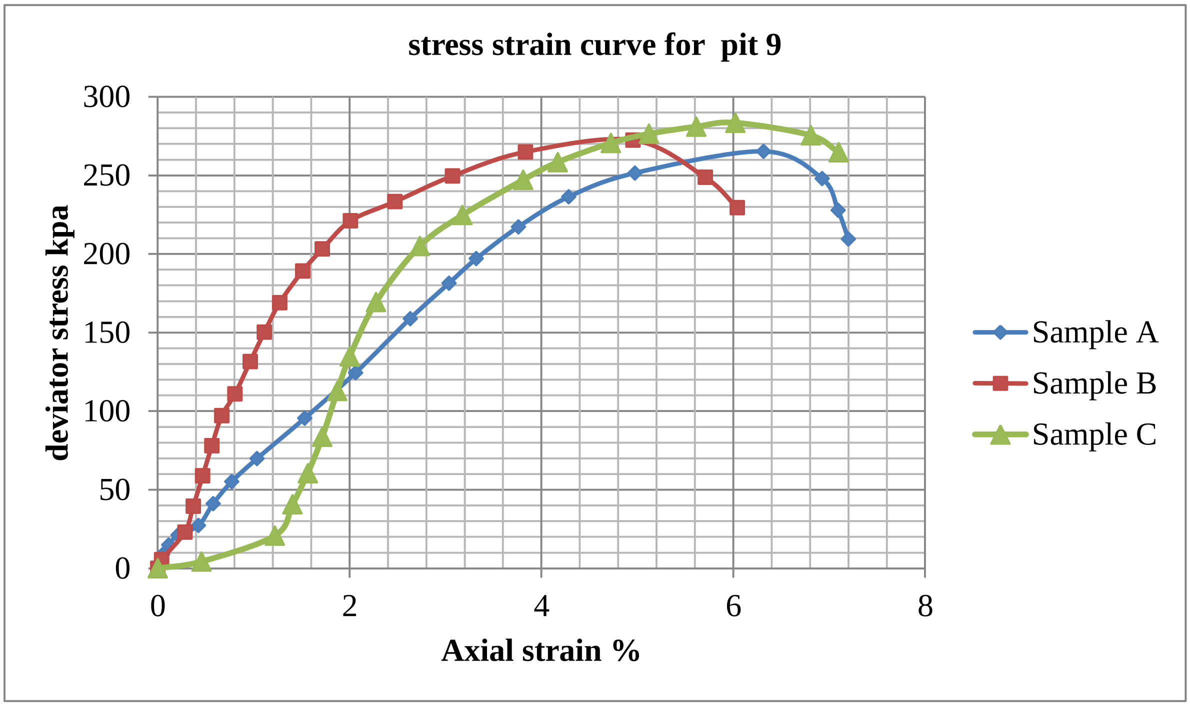
LL test

Unconsolidated Undrained Triaxial Test

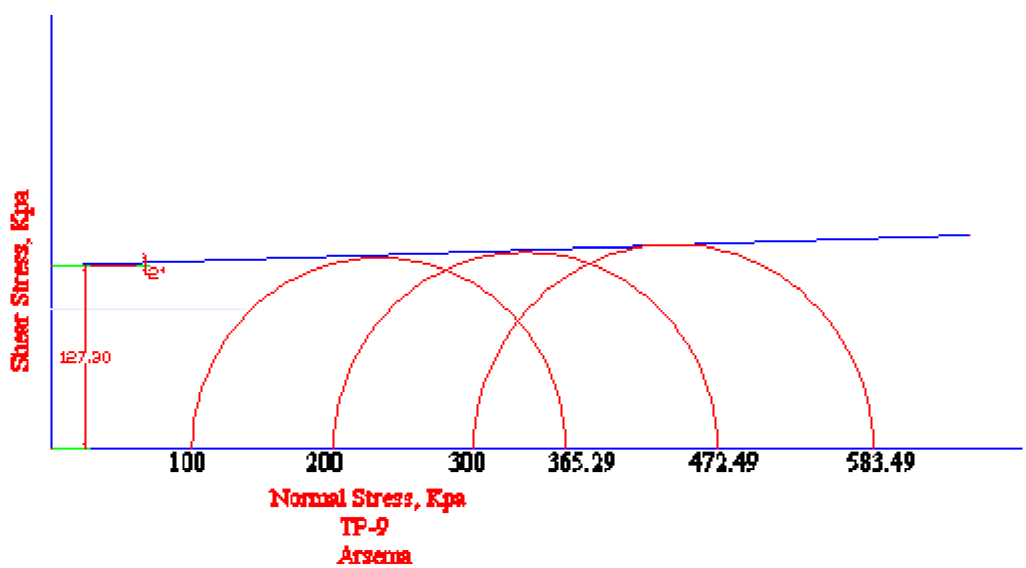
Samples after shear failure

II. Typical Steps for analysis of Unconsolidated Undrained Tri-Axial Shear Test

laboratory results for triaxial test Location: pit 9 (Arsema): Depth 2.5m														
test details														
sample A					sample B					sample C				
axial deformation (ΔL) mm	Axial load P N	axial strain (ε_a)=ΔL/L0	corrected area (A)=A0/(1-εa) (mm2)	Deviator stress (σ_d)=P/A kN/m2)	axial deformation (ΔL) mm	Axial load P N	axial strain (ε_a)=ΔL/L0	corrected area (A)=A0/(1-εa)	Deviator stress (σ_d)=P/A	axial deformation (ΔL) mm	Axial load P N	axial strain (ε_a)=ΔL/L0	corrected area (A)=A0/(1-εa)	Deviator stress (σ_d)=P/A
0	0	0	1133.54	0	0	0	0	1133.54	0	0	0	0	1133.54	0
0.039	10	0.051315789	1134.121984	8.817394	0.04	46	0.05263158	1134.136914	5.559477	0.458	5	0.602632	1140.412	4.384379
0.088	17	0.115789474	1134.854042	14.9799	0.283	66	0.37236842	1137.776721	23.00787	1.223	24	1.609211	1152.079	20.8319
0.16	24	0.210526316	1135.931435	21.12804	0.371	85	0.48815789	1139.10061	39.62027	1.406	47	1.85	1154.906	40.69596
0.323	31	0.425	1138.378107	27.23173	0.469	107	0.61710526	1140.57857	58.81204	1.564	70	2.057895	1157.357	60.48262
0.44	47	0.578947368	1140.140815	41.22298	0.564	129	0.74210526	1142.014953	77.95824	1.717	97	2.259211	1159.741	83.63936
0.586	63	0.771052632	1142.348105	55.14956	0.668	151	0.87894737	1143.591568	97.04015	1.872	131	2.463158	1162.166	112.7206
0.787	80	1.035526316	1145.400928	69.84454	0.806	173	1.06052632	1145.690348	111.0007	2.003	157	2.635526	1164.223	134.8538
1.165	110	1.532894737	1151.186477	95.55359	0.965	197	1.26973684	1148.118078	131.5851	2.277	198	2.996053	1168.55	169.4407
1.567	144	2.061842105	1157.40384	124.4164	1.113	219	1.46447368	1150.387117	150.3707	2.732	241	3.594737	1175.807	204.9656
2	185	2.631578947	1164.176216	158.9107	1.272	241	1.67368421	1152.834814	169.0499	3.178	266	4.181579	1183.008	224.8505
2.307	212	3.035526316	1169.026095	181.3475	1.511	265	1.98815789	1156.533716	189.133	3.811	295	5.014474	1193.382	247.1967
2.524	231	3.321052632	1172.478632	197.0185	1.716	282	2.25789474	1159.725378	203.161	4.172	310	5.489474	1199.38	258.467
2.859	256	3.761842105	1177.848813	217.3454	2.01	304	2.64473684	1164.333559	221.0936	4.723	327	6.214474	1208.651	270.5495
3.257	280	4.285526316	1184.293197	236.4279	2.472	326	3.25263158	1171.64944	233.2402	5.12	336	6.736842	1215.421	276.4474
3.78	300	4.973684211	1192.869565	251.4944	3.073	348	4.04342105	1181.305141	249.5894	5.616	344	7.389474	1223.986	281.0489
4.801	321	6.317105263	1209.975421	265.2946	3.834	370	5.04473684	1193.762159	264.9445	6.023	349	7.925	1231.105	283.4851
5.264	302	6.926315789	1217.895273	247.9688	4.956	385	6.52105263	1212.615281	272.4956	6.808	343	8.957895	1245.072	275.486
5.39	278	7.092105263	1220.068546	227.8561	5.707	360	7.50921053	1225.570683	248.7407	7.098	331	9.339474	1250.313	264.7338
5.472	256	7.2	1221.487069	209.5806	6.041	338	7.94868421	1231.421833	229.4795	7.374	309	9.702632	1255.341	246.1482
σf=265.295kpa		σ3=100kpa	σ1f=365.295kpa		σf=272.5kpa		σ3=200kpa	σ1f=472.5kpa		σf=283.49kpa		σ3=300kpa	σ1f=583.49kpa	



Deviator Stress Vs Strain at 2.50 m depth at TP9



Summary of Regression Data

S/N	Su	w	LL	PL	PI	OMC	MDD	LI=(w-PL)/PI	Gs	Depth
1	82.28	41.17	59.00	21.36	37.64	27.80	1.22	0.53	2.66	2.50
2	97.54	32.52	52.80	24.50	28.30	27.90	1.34	0.28	2.85	3.00
3	62.10	39.44	59.80	16.97	42.83	24.75	1.59	0.52	2.71	3.00
4	41.52	52.00	62.80	25.66	37.14	30.90	1.38	0.71	2.79	2.50
5	79.61	43.86	69.00	24.50	44.50	28.50	1.35	0.44	2.78	2.00
6	110.45	24.00	42.00	18.75	23.25	30.00	1.25	0.23	2.70	2.50
7	71.60	42.36	64.90	21.60	43.30	22.00	1.43	0.48	2.68	2.75
8	87.00	33.82	59.05	25.09	33.96	27.50	1.46	0.26	2.65	2.50
9	127.30	42.64	72.53	31.05	41.48	22.50	1.36	0.28	2.67	2.50
10	97.84	38.50	54.42	23.97	30.45	28.00	1.32	0.48	2.69	2.50
11	49.99	38.79	65.89	19.98	45.91	33.00	1.30	0.41	2.66	3.00
12	75.65	40.50	79.84	31.27	48.57	25.00	1.50	0.19	2.72	2.50
13	60.39	50.66	66.34	33.19	33.15	23.00	1.25	0.53	2.77	2.50
14	150.00	26.50	53.23	20.97	32.26	23.50	1.52	0.17	2.76	3.00
15	55.37	51.57	68.70	22.06	46.64	25.64	1.48	0.63	2.83	3.00
16	78.51	44.16	70.43	30.29	40.14	26.75	1.45	0.35	2.68	3.00
17	125.00	48.24	62.95	28.34	34.61	26.80	1.25	0.57	2.79	2.75
18	53.03	46.15	62.35	25.88	36.47	25.00	1.50	0.65	2.71	2.50

DECLARATION

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

Name: Yohannis Anole

Signature: _______________

Date: _______________

Advisor: Dr. Argaw Ashaw