ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

DEPARTMENT OF CIVIL ENGINEERING

Correlating Liquidity Index with Vane-Shear Strength of Clays in Addis Ababa

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A Thesis Submitted to the school of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering [Geotechnical Engineering]

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Declaration

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor **Dr-Ing. Asrat Worku** and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have been duly acknowledged.

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Date: _________________

Acknowledgment

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Abstract

Investigation of soil properties is one of the main tasks of Geotechnical Engineer. Soil testing methods don't require the same amount of time and effort; even some of the apparatus are scarce in developing countries. This usually leads to unreliable and impractical test results. That is why it is important to develop empirical equations that fit the local area.

In this thesis, attempts have been made to obtain valid correlations between undrained shear strength and liquidity index. For this purpose, 6 disturbed samples were collected from 5 sites (from depth of 1m to 3m) and for each sample Atterberg limit and standard compaction tests were conducted. Beside that by remolding these samples 12 vane shear tests for each of the samples in total 72 vane shear tests were conducted. From these tests undrained shear strength and liquidity index are determined.

For the analysis, SPSS and EXCEL softwares have been used for the best fit line, correlation, regression analysis and comparative study. Equations were developed for each soil sample with a very strong correlation of coefficient of determination (R^2) values ranging from 0.927 up to 0.991. From the result it has been shown that the use of the commonly assumed 100-fold factor increase in strength from the liquid limit to plastic limit over predicts the measured data of the soil strength. In this study the ratio factor fold in strength is between 30 up to 63. The results are expected to have wide application in the construction sector for soft clays.

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Abbreviations

ASTM	American Society for Testing Materials
CH	Inorganic clay of high plasticity
Cu	Undrained shear strength
D	Diameter
GPS	Global Positioning System
H	Height
KPa	Kilo Pascals
LI	Liquidity Index
LL	Liquid Limit
ρ	Density
ρ_t	Bulk density
ρ_d	Dry density
PL	Plastic Limit
ω	Water content
R^2	Coefficient of determination
SPSS	Statistical package for the Social Sciences
T	Torque reading
USCS	Unified Soil Classification System

1. INTRODUCTION

1.1 GENERAL

It is necessary to determine the in situ Engineering properties of soil for the safety and stability of the structure that is built on it. One of the major tasks as a Geotechnical Engineer is to establish empirical relationship for the soil tests based on the soil condition using sufficient number of tests. However, obtaining the Engineering properties of these tests don't require the same amount of money, time and energy. That is why correlation is necessary. Investigating index properties are much easier than investigating other Engineering properties. Therefore, by obtaining the index property of soils that involves simpler and quicker method of testing, the Engineering properties can be predicted satisfactorily from empirical correlations.

The shear strength of a soil is its maximum resistance to shear stresses just before the failure. For fine grained soils the shear strength generally can be categorized into drained and undrained condition based on the pore water pressure dissipation. In situ soil condition is recorded in undrained condition. The undrained shear strength (Su) of fine-grained soils is one of the key geotechnical parameter. The undrained shear strength can be determined in the laboratory by vane shear test or unconfined compression test. The undrained shear strength of soft clays can be determined in a laboratory by a vane shear test. Unconfined Compression Test is generally performed on intact (non-fissured), saturated clay specimens which can stand without confinement.

The vane shear test is originally developed by British army to measure the cohesion of clay sediments, which is the shear strength in a certain case [4]. The laboratory version of vane shear test provides a rapid determination method for the shear strength of undisturbed and remolded soils. It is recommended for use on soils with undrained shear strength less than 100 kpa [8].

There are various researches that are done to determine the best mathematical relationship between undrained shear strength and liquidity index. Scofield and Wroth (1968) upon examination of vane shear test data from Skempton and Northey (1952) made the observation that liquid limit and plastic limit do correspond approximately to fixed strengths which are the proposed ratio of 1:100. Recently in 2014 Vardanega and Haigh have been analyzed to determine the relationship between undrained shear strength and liquidity index using fall cone test and got the strength factor between liquid limit and plastic limit as 1:35 [7].

1.2 Statement of the problem

Correlations are important to estimate Engineering properties of soil particularly where there is a financial limitation, lack of equipment, limited time and also correlations are used commonly to get preliminary background information of the soil. Many attempts have been made to get the best mathematical relationship between Undrained shear strength and liquidity index for different kinds of soil in different countries [7]. Addis Ababa is a fast growing city in terms of the number and type of construction activities but there are some problems in the construction process. One of the problems islack financial limitation. The second problem is lack of some of the apparatus in the laboratory especially with regard to undrained shear strength test unconfined compression test is found in many laboratories which is ideally suited formeasuring the shear strength of intact, saturated clays, but vane shear test isn't found in most of the laboratories and so that it is difficult to measure the undrained shear strength of soft and sensitive clays.

1.3 Objectives

1.3.1 General objectives

The general objective of this research is to obtain applicable relationship between undrained shear strength and liquidity index for soft clays in Addis Ababa.

1.3.2 Specific objectives

- To develop appropriate empirical correlations between undrained shear strength and Liquidity index.

- To know the factor of ratio between the strength at Liquid limit to that of plastic limit of the soil.
- To examine the validity of the correlations and to draw appropriate conclusions on the relationships of empirical correlations.

1.4 Scope

This research addresses the described objective and provides correlations between the undrained shear strength and liquidity index of clay soils of Addis Ababa. In order to conduct the proposed correlation, 90 laboratory test results are used in this research.

With regard to the regression analysis, depending on the trends of the scattering of test results the correlation is analyzed using a linear and parabolic regression model. The required correlation is carried out with the help of SPSS and Excel softwares. Finally, the scope of the developed correlation is limited to clay soils in Addis Ababa.

1.5 Methodology

The first thing in this research was to study previous works of different researches with regard to undrained shear strength and Atterberg limit. After that 5 sites were selected within Addis Ababa and 6 disturbed samples were collected at a depth ranging from 1.00m to 3.00m.

The liquid limit, plastic limit, standard compaction test and vane shear tests were conducted in order to have satisfactory data for the relationship that is set in the objective. The disturbed samples were remolded for the vane shear test. ASTM procedures were followed while conducting the tests.

Using SPSS software undrained shear strength was correlated with liquidity index and empirical equations are developed. Linear regression is employed in the analysis. The reliability of the correlations were examined and conclusion and recommendation are made.

1.6 Organization of the Thesis

The thesis is organized under seven parts. The first chapter covers the background, statement of the problem, general and specific objective, scope of the study, Methodology and Organization of the thesis. Literature review is undertaken in the second chapter. Chapter three deals with field investigation and soil sampling. Chapter four illustrates the laboratory tests and results. In chapter five the analysis part is presented empirical equations are developed and regression methods have been used to evaluate the validity of the relationships. In chapter six focuses with the discussion of test results. In chapter seven conclusions and recommendation are presented. Detailed test results are presented in the appendices.

2. LITERATURE REVIEW

2.1 Properties of clay soil

The term clay is commonly used to refer to a material composed of a mass of small mineral particles which, in association with certain quantities of water, exhibits the property of plasticity. The behavior of clay soils depends to a large extent on the nature and characteristics of the minerals present. The most significant properties of clay depend upon the type of mineral. Clay minerals are essentially crystalline in nature. The crystalline minerals whose surface activity is high are clay minerals. These clay minerals impart cohesion and plasticity. Clays have less deformation to resistance when they are wet and become hard when they are dry. Clays are virtually impervious and difficult to compact when they are wet. Large expansion and contraction with changes in water content are characteristics of clays. Clay soils swell when wetted and shrink when they dry out. They are also defined as particles smaller than 0.002mm.

The behavior of a soil mass depends upon the behavior of the discrete particles composing the mass and the pattern of particle arrangement. It needs to be well recognized that the presence of clay minerals in a soil aggregate has a great influence on the engineering properties of the soil as a whole. When moisture is present, the engineering behavior of a soil will change greatly as the percentage of clay mineral content increases. The behavior of the soil mass is profoundly influenced by the amount of water present.

Consistency is a term used to indicate the degree of firmness of cohesive soils. The consistency of natural cohesive soil deposits is expressed qualitatively by such terms as very soft, soft, stiff, very stiff, hard. The physical properties of clays greatly differ at different water contents. A soil which is very soft at a higher percentage of water content becomes very hard with a decrease in water content. However it has been found that at the same water content, two samples of clay of different origins may possess different consistency. One clay may be relatively soft while the other maybe hard. Further, a decrease in water content may have little effect on one sample of clay but may transform the other sample from a liquid to a very firm condition.

2.2 Atterberg limit test

A fine-gained soil can exist in any of several states; which state depends on the amount of water in the soil system. When water is added to a dry soil, each particle is covered with a film of adsorbed water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily. The behavior of the soil, therefore, is related to the amount of water in the system. Approximately sixty years ago, A. Atterberg defined the boundaries of four states in terms of "limits" as follows:

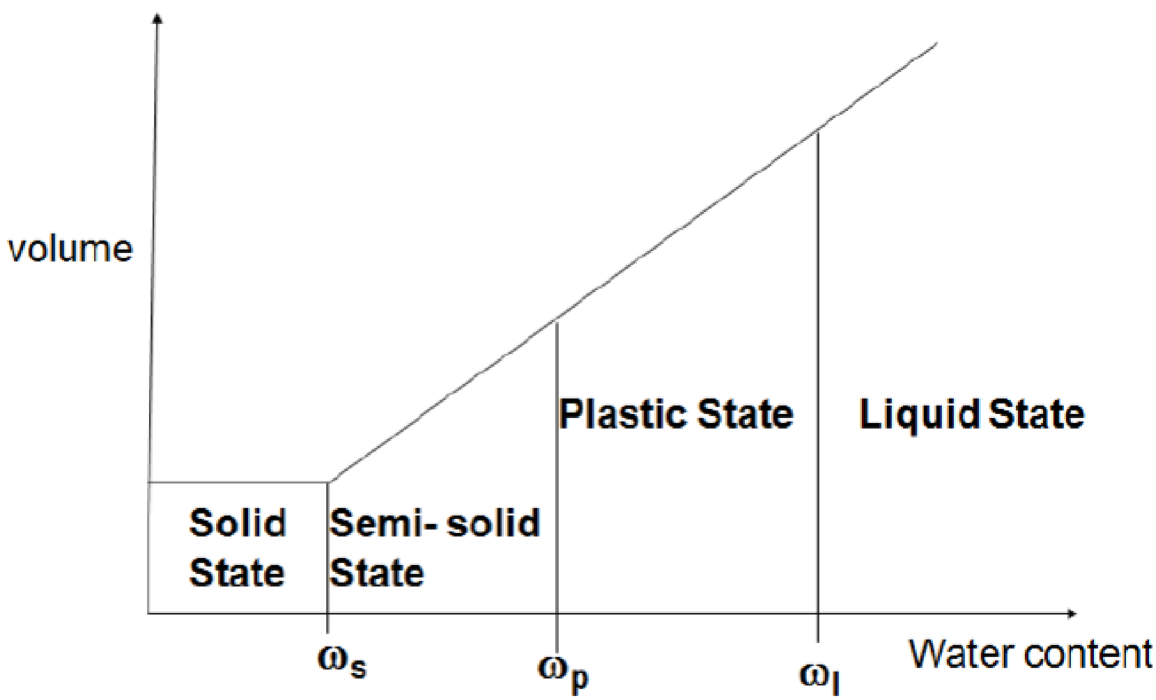


Fig 2.1 Different states of soil

• Liquid limit: The boundary between the liquid and plastic states;

• Plastic limit: The boundary between the plastic and semi-solid states;

• Shrinkage limit: The boundary between the semi-solid and solid states.

These limits have since been more definitely defined by A. Casagrande as the water contents which exist under the following conditions:

2.2.1 Liquid limit

The water content at which the soil has such small shear strength that it flows to close a groove of standard width when jarred in a specified manner. The liquid limit is defined as the moisture content at which soil begins to behave as a liquid material and begins to flow on the application of a very small shearing force. When a soil becomes a viscous fluid, the soil will begin to flow under its own weight and very small amount of energy input. The liquid limit is primarily used by civil and geotechnical engineers as a physical property of a soil. The liquid limit allows engineers to classify soils into their applications.

2.2.2 Plastic limit

The water content at which the soil begins to crumble when rolled into threads of specified ze. The plastic limit of a soil is the water content at the boundary between the plastic and emisolid state. The water content at this boundary is arbitrarily defined as the water content at which soil begins to crumble when rolled into threads of specified size (3.2mm).

- Shrinkage limit: The water content that is just sufficient to fill the pores when the soil is at the minimum volume it will attain by drying.

The amount of water which must be added to change a soil from its plastic limit to its liquid limit is an indication of the plasticity of the soil. The plasticity is measured by the “plasticity index”, which is equal to the liquid limit minus the plastic limit. Although the liquid and plastic limits are necessarily determined on soils which have had their natural structure completely destroyed by kneading or “remolding”, the shrinkage limit can be obtained on soils in either their undisturbed or their remolded states. The difference between the undisturbed and remolded shrinkage limits may be an indication of the amount of natural “structure” a soil possesses. Also the condition of an in situ soil is often partially revealed by its “water-plasticity ratio”, which is the ratio of the difference between the natural water content and the plastic limit to the plasticity index. A high water plasticity ratio, which means that the natural water content is high relative to the liquid limit, indicates a very low remolded strength. For example, if the ratio is greater than 100%, the soil exists at water content greater than the liquid limit, and its remolded strength is thus less than that very small amount which it would possess at the liquid limit. The chemical and mineral composition, size and shape of the soil particles influence the adsorbed water films on the particles. A wide variety of soil engineering properties have been correlated to the liquid and plastic limits, and these Atterberg limits are also used to classify a fine-grained soil according to the Unified Soil Classification system or AASHTO system.

2.2.3 PLASTICITY INDEX

Plasticity index, PI = LL-PL

This indicates:

- The range of water content over which the soil exhibits plasticity.
- The presence of clay in a given soil.
- In general, a high numerical value of plasticity index means a high percentage of clay fractions.

Table 2.1 : Plasticity index

Plasticity Index	Plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium
>17	High Plastic

2.2.4 LIQUIDITY INDEX

The Atterberg limits are found for remolded soil samples. These limits as such do not indicate the consistency of undisturbed soils. The index that is used to indicate the consistency of undisturbed soils is called as the liquidity index or water plasticity ratio. The liquidity index is expressed as

$$LI = (W - WP)/PI$$

The value of LI varies according to the consistency of soils as shown below

Table 2.2 The consistency of the soil

Consistency	Liquidity index
semi solid or solid state	Negative
Very stiff state(ω=ωp)	0
Very soft state(ω=ωl)	1
Liquid state(when disturbed)	>1

2.3 STANDARD PROCTOR TEST

Compaction is the process of densification of soil by reducing air voids. The degree of compaction of a given soil is measured in terms of its dry density. The dry density is maximum at the optimum water content. A curve is drawn between the water content and the dry density to obtain the maximum dry density and the optimum water content.

Soil placed as engineering fill (embankments, foundation pads, road bases) is compacted to a dense state to obtain satisfactory engineering properties such as, shear strength, compressibility, or permeability. Also, foundation soils are often compacted to improve their engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved. The compaction mold has a volume of about 944 cm^3.

Proctor developed this test in connection with the construction of earth fill dams in California in 1933. He gives the standard specifications for conducting the test. A soil at a selected water content is placed in three layers into a mold of 101.6mm diameter, with each layer compacted by 25 blows of a 2.5 Kg hammer dropped from a height of 305 mm.

2.4 undrained shear strength

The shear strength of fine-grained soils generally can be divided into two parts as drained and undrained shear strengths depending on whether the pore water pressure dissipates or not. In situ shear strength of soils is recorded almost in undrained condition. The undrained shear strength (Su) of fine-grained soils that can be measured in situ and in laboratory is one of the key geotechnical parameters. Undrained shear strength of the soil depends upon the prevailing in situ conditions, which can vary with time, the rate of loading, and many other factors. The remolded undrained shear strength (Sur) is of importance in many geotechnical applications including pile design and submarine soil investigations for offshore structures [13]. Undrained shear strength decreases with the increase in water content. Additionally, the undrained shear strength of a clayey soil also depends on the dominant clay mineral present. The undrained shear strength of Kaolinitic soils is a result of the net attractive forces and the mode of particle arrangement as governed by the interparticle forces, whereas that of montmorillonite soils can be attributed to the viscous shear resistance of the double-layer water.

The undrained shear strength of soft clays can be determined in field or laboratory by vane shear test, while that of intact clays can be determined by unconfined compressive strength. The unconfined compression test is a special form of triaxial test in which the confining pressure is zero. The test can be conducted only on clayey soils which can stand without confinement. The test is generally performed on intact (non-fissured), saturated clay specimens. The test is conducted on undisturbed sample or remoulded sample. It is convenient, simple and quick. However, the test cannot be conducted on fissured clays. In this paper the focus of the research is on soft clays for which vane shear test apparatus is easier to use. Besides, if conducted properly, results are more reliable than those obtained from unconfined compressive strength tests, which can be affected by a number of factors.

2.4.1 Vane shear test

The vane shear test is a moderately rapid and economical in-situ method for determining the peak and remolded undrained shear strength of soft to medium stiff clays. The test involves pushing a four-bladed vane into a clay stratum and slowly rotating it while measuring the resisting torque. Generally, the height of the vane is about twice of its width. The vane is pushed into the soil for at least twice its height and then rotated at a constant rate of 0.1 to 0.2 degrees per second until the soil is failed. The maximum torque required to shear soil is then converted to the undrained shear resistance of the cylindrical surface. The VST technique was originally developed by the British Army to measure the cohesion of clay sediments, which is the shear strength in certain special cases [4].

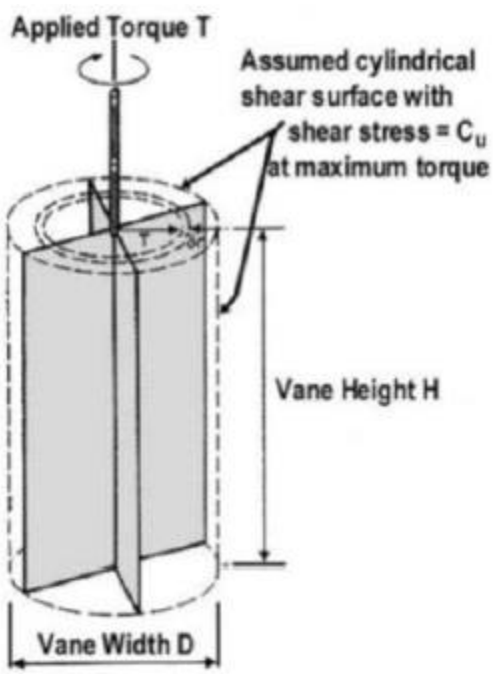


Fig 2.2 Schematic Idealization for the vane shear device

The most common tool used to measure the in situ undrained shear strength of soft to medium stiff cohesive soils is the field vane shear test (VST_F) [3]. It also has considerable application in offshore soil exploration, particularly when used with sample recovery equipment. The test is performed by inserting the vane into the soil and applying a torque after a short time lapse, on the order of 5 to 10 minutes [11]. If the time lapse is less than this, the insertion effects reduce the measured vane shear strength, and if much over this time the soil tends to set up or consolidate, with an increase in vane shear strength. The vane may be inserted into the stratum being tested from the bottom of a borehole or pushed without a hole by using a vane sheath similar to a cone penetration test, with the vane then extended below the sheath for the actual test. The vane test is done at a depth of at least five sheath diameters below the sheath or at least five diameters below the bottom of a drill hole. Vane blades are on the order of 1.5 to 2.5 mm thick, the shaft body is from about 12.7 to 22 mm in diameter. The dimensions are selected to minimize soil disturbance from its insertion, but there is always a small amount (on the order of 15 to 25 percent) of strength loss. The torque is usually applied through a suitable gearing device so that a rate of about 6^{o} of rotation per minute can be achieved. The test is sometimes done using an ordinary torque wrench to apply (and measure) the torque. Commercial supplies can provide the torque equipment, extension rods, and bearings as well as the vane in a package. Somewhat similar to the SPT, the vane test is usually performed every 0.5 to 1m of depth in soft clays and fine silty sands. The test is not well suited for dense, hard, or gravelly deposits. The test can be performed even without drilling a bore hole by direct penetration of the vane from the ground surface if it is provided with a strong shoe to protect it.

A number of assumptions are made in calculating the undrained shear strength from these torque measurements (2), including;

- The soil is completely undrained, i.e. no consolidation takes place during insertion of the vane or during the test.

- No disturbance is caused by the boring operation or installation of the vane.

- There is no progressive failure so that the maximum applied torque overcomes the fully – mobilized shear strength along the cylindrical surface.

- Isotropic strength conditions exist in the soil mass.

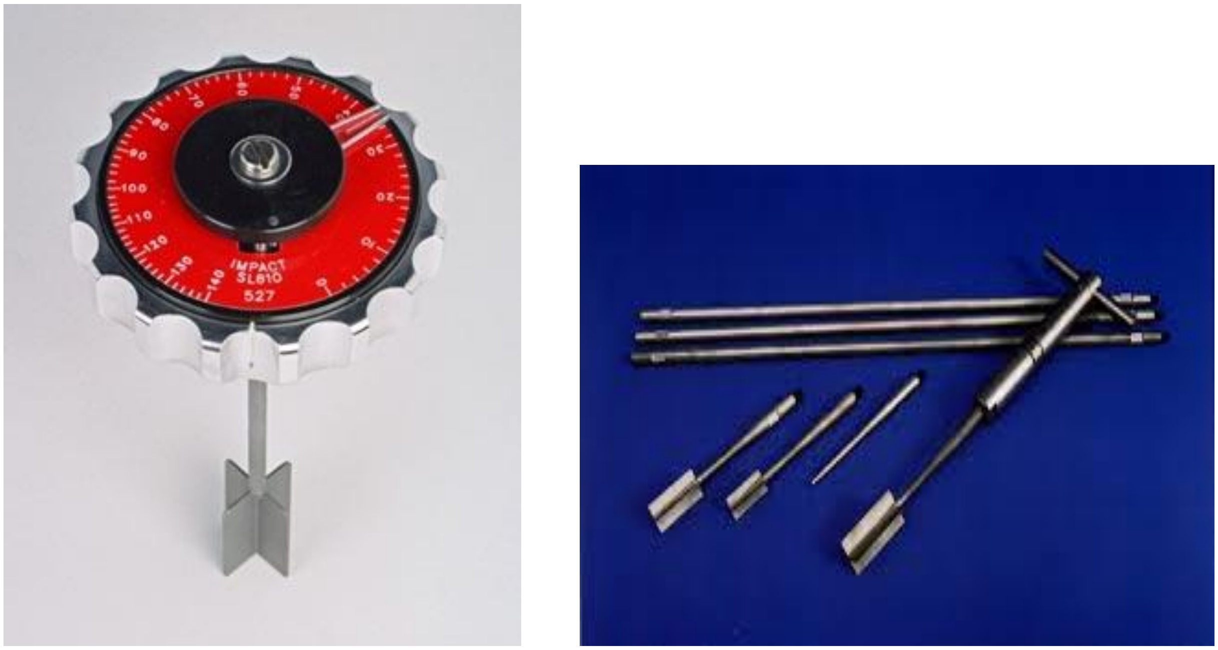


Fig 2.3 Field vane shear test equipment

For conducting the test in the laboratory, a cylindrical specimen of 38mm diameter and 75mm height is taken in a container which is fixed securely to the base. The vane is gradually lowered into the specimen till the top of the vane is at a depth of 10 to 20mm below the top of the specimen. The readings of torque indicator are taken. The torque acting on the specimen is indicated by a pointer fixed to the spring. The torque is continued till the soil fails in shear. Sample disturbance is one of the most important factors influencing the undrained shear strength of fine-grained soils measured by laboratory techniques. The undrained shear strength obtained from VST_L is nearly the same as that from VST_F [12] showed that the vane shear strength is not significantly influenced by the mechanical disturbance caused by sampling, release of overburden pressure, or increase in the confining pressure when the vane is inserted. While the unconfined compression test is highly sensitive to disturbance caused by the sampling process [16].

The formula for vane shear test strength is

$$S = \frac{T}{\pi\left(\frac{D^2H}{2}+\frac{D^3}{6}\right)} \quad \text{......} \quad (2.1)$$

Where T is in N-cm and s in N/cm^2

Eq.2.1 is modified if the top of the vane is above the soil surface and the depth of the vane inside the sample is H_1. In such a case

$$S = \frac{T}{\pi\left(\frac{D^2 H_1}{2} + \frac{D^3}{12}\right)} \qquad (2.2)$$

The shear strength of the soil under undrained conditions is equal to the apparent cohesion Cu.

The test has the following merits

(1)The test is simple and quick.

(2)It is suitable for the determination of the in-situ undrained shear strength of non-fissured, fully saturated clay.

(3)The test can be conveniently used to determine the sensitivity of the soil.

Its major disadvantage is that the test cannot be conducted on fissured clays or clays containing sand or silt laminations.

Table 2.3 Major sources of error in the vane shear test

Cause	Effect	Influence on shear Measurement
Friction between torque rods and soil or casing	Measured torque includes spurious component of resistance	Increases
Poorly calibrated torque measurement	Inaccurate torque	Increases or decreases
Vane rotated too quickly	Soil sheared too rapidly	Increases
Test performed in disturbed sample	Soil structure broken down	Decreases
Damaged vane	Disturbed soil excessively	Decreases peak strength
Unknown sand/silt/shell lenses	Drainage during test	Increases
Isolated gravel/cemented nodules	Measured torque includes spurious component of resistance	Increases

2.5 Existing Correlations

Many attempts have been made to obtain correlations for fine-grained soils between their undrained shear strength and liquidity index. There has been an ongoing debate regarding the undrained shear strength at the Atterberg limits resulting in two distinctive ways. One claims that the undrained shear strengths at the liquid limit and plastic limit are fixed, and the ratio of the former to the latter is 100. Another claims that the range of the variation of the Su at both the liquid and plastic limits is different, and it may not be correct to conclude that the Su has a unique value at either the LL or PL.

The earliest research on the relationship between the undrained shear strength and a consistency limit was done in 1939 by Casagrande [23], who suggested that the Su at LL is 2.65KPa. Skempton and Northey (1952) [25] had studied the relationship between undrained shear strength and Liquidity index and concluded that "it appears that the liquid limit and plastic limit do correspond approximately to fixed strengths which are in the proposed ratio of 1:100". They reported that the Su at the liquid limit ranged from 0.75KPa to 1.75KPa.

Norman (1958) [26] stated that the shear strength at the LL controlled by using an apparatus of ASTM standards, the strength varied from 1.1 to 2.3KN/m^2. Youssef et al. (1965) [17] found the values of the shear strength of the clay ranged from 1.3 to 2.4KN/m^2 with a mean value of 1.7KN/m^2. He suggested that the strength at LL may be a function of density and hence water content. Wroth and Wood (1978) [20] proposed a mean value of 1.7KPa for the Su at the LL and assumption that the Su at the PL is 100 times higher than what it is at the LL. Based on this observation they proposed equation (3)

$$\mathrm{Cu} = 170\mathrm{e}^{-4.6\mathrm{I_L}} = 1.7\mathrm{X}10^{2(1-\mathrm{I_L})}\mathrm{KPa} \quad (2.3)$$

Leroueil et al. (1983) [7] proposed an alternative correlation

$$C_u = \frac{1}{(\mathrm{I_L}-0.21)^2} \; Kpa \qquad 0.5 < \mathrm{I_L} < 2.5 \quad (2.4)$$

Federico (1983) [22] found the shear strength at the liquid limit of soils, falls within limits of 1.7 and 2.8KN/m^2.Locat and Demers (1988) [5] proposed equation for computing strengths at high liquidity indices.

$C_u = \left(\frac{1.167}{I_L}\right)^{2.44}$ (Kpa) $1.5 < I_L < 6$ .... (2.5)

Kayablai and Tufenkci (2010) [13] found that the undrained shear strength at plastic limit in the range (20-320KPa) and they stated that, although the VST_L provides a reasonable undrained shear strength value at the PL, it overestimates the undrained shear strength at the LL. They recommended that care should be taken when the laboratory VST is used to determine the undrained shear strength at water contents near the liquid limit. Nagaraj et al. (2012) [15] stated that published data from various literature sources clearly show that the variation of the undrained shear strength at the liquid limit is observed to be nearly 60 times (from as low as 0.2 KPa to as high as 12KPa) and that at the plastic limit is more than 17 times (from 35KPa to 600KPa), hence no unique value of undrained shear strength can be assigned either at the liquid limit or plastic limit of soils. Haigh et al. (2013) [14] reported ranges (17-530KPa) for undrained shear strength at plastic limit. Brendan C. O'Kelly (2013) [19] found the shear strength at plastic limit of soils, falls within a limit from 34 up to 123 and a mean of 82.

Recently, Dr P. J. Vardanega and Dr S. K. Haigh (2014)[7] had done the research on the ratio between the strength at liquid limit and that extrapolated to plastic limit and suggest that the use of factor to be about 35 is more realistic than 100.

$I_L = 1.150 - 0.283$ ln(Cu) (kpa)..(2.6)

A summary of the proposed S_u Values at the liquid limit and plastic limit is provided in Table 2.4 and Table 2.5 respectively.

Table 2.4 Variation of undrained shear strength at liquid water content [2]

Reference	Range of Cu at LL water content (Kpa)	Range of PL	Test	Remarks
Skempton and Northey (1952)	0.7-1.75	30-97	Vane shear test	Shear strength is lower than 1kPa
Norman (1958)	0.8-1.6	41-72	Miniature Vane shear apparatus	Shear strength of clay soils 25 to 50% higher in ASTM standars
Youseff et al (1965)	1.3-2.7	32-190	Vane shear test	Shear strength more than 1kPa
Wroth and Wood (1978)	More than 1.7	26-190	Vane shear test	Average shear strength is 1.7Kpa
Locat and Demers (1988)	0.2-2.04	27.4-62.8	Viscometer	Shear strength is lower than 1kPa
Kayabali and Tufenkci (2010)	1.2-12	26.4-83.6	Vane shear test	Shear strength is higher than 1kPa

Table 2.5 Undrained shear strengths at plastic limit [13]

Source	S_u range (Kpa)	Average (Kpa)	Remarks
British Standards (BS 1377, 1948)		110	Quoted by Whyte (1982)
Skempton and Northey (1953)	85-125	110	Quoted by whyte (1982)
Dennehy (1978)	30-320	115	Quoted by whyte (1982)
Arrowsmith (1978)	20-220	110	Quoted by Whyte (1982)
Whyte (1982)	25-280	130	
Wroth and Wood (1978)		170	Adopted as the best estimate
Medhat and Whyte (1986)		110	Upon literature review
Sharma and Bora (2003)		170	Cone penetration method

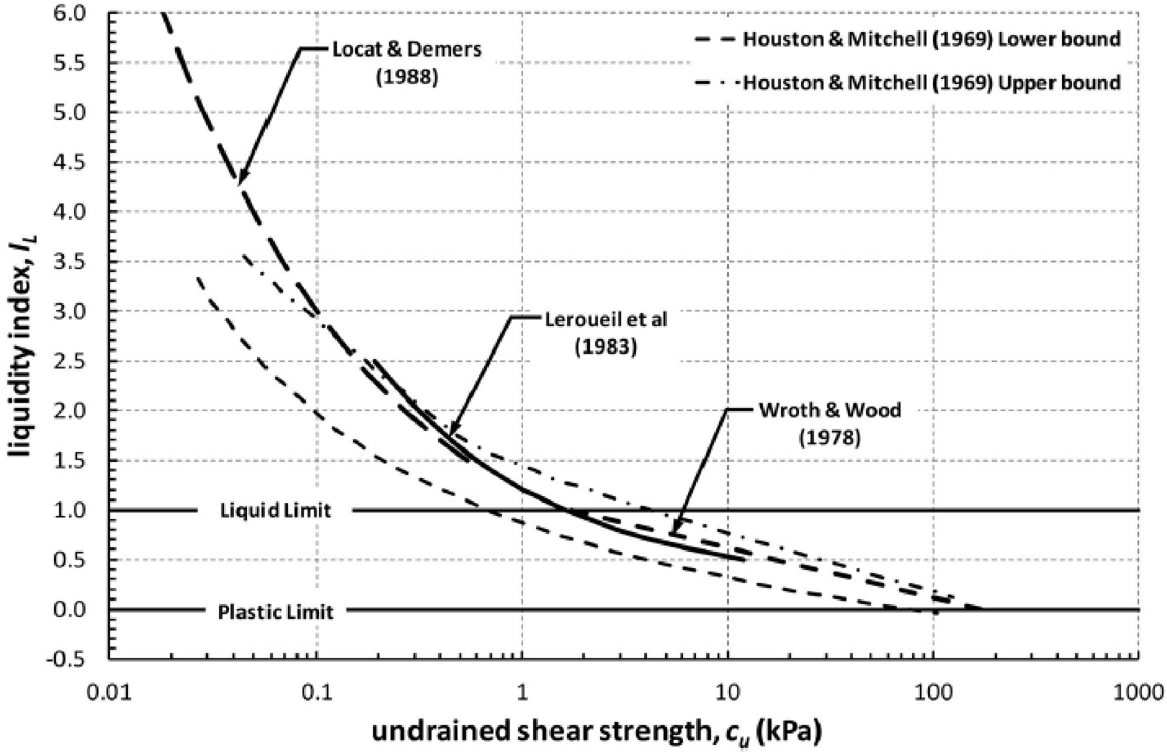


Fig 2.4 some liquidity index versus undrained shear strength relationships [7]

3. FIELD INVESTIGATION AND SOIL SAMPLING

3.1 Area of study and soil sampling

Soil samples are collected from Addis Ababa, which is dominantly covered by red fine grained soils in the northern and western parts and with black expansive soils in the eastern and southern parts. [9]

The locations were made to cover the two main clayey soil categories, i.e., area covered by red clay and black cotton /expansive/ soils. Five sites are selected for sampling within Addis Ababa city and six disturbed samples are collected. These sites are:

(1) AddisuGebeya

(2) Akaki-kality

(3) Hana Mariam

(4) Lamberet

(5) Semit

Table 3.1 Location of the selected sites

S. No.	Loaction	Easting	Northing	Elevation
1	Addisu Gebeya	464479	995045	2445
2	Akaki-Kality	477630	979769	2095
3	Hana Mariam	470648	989820	2414
4	Lamberet	487096	997690	2429
5	Semit	476283	989620	2241

Fig 3.1 Addisu Gebeya Typical location using GPS from Google Earth

The selected sites areal views are presented in the Appendix E.

These samples include four expansive soils were obtained from Akaki-Kality, Hana Mariam and Sumit and the remaining two red clay soils obtained from Addisu Gebeya and Lamberet. The samples were taken from a depth ranging from 1.00m to 3.00m below the ground surface. One test pit was opened at each site in which it was previously non habited empty space and one sample was taken for each test pit at each site except Akaki-Kality. In Akaki-Kality two samples were taken from the test pit from 1.00m and 3.00m in which the color of the soil is black and gray respectively. The soil in the 3.00m depth becomes gray due to lack of oxygen. The samples were transported to AAIT geotechnical laboratory and then the samples were air dried, pulverized and sieved with different sieve sizes depending on the test requirement.

3.2 Visual Identification of Soils in the Field

Field identification of soils was carried out according to the ASTM D-2488 "Standard Practice for Description and Identification of Soils".

The field description and classification of a soil were based on the behavior of fine grained soils. The first step used in describing soil under the visual-manual method was to determine whether the soil is fine-grained or coarse-grained by visually observing the soil sample to be taken. The sample shall be considered to be representative of the stratum from which it was obtained by an appropriate, accepted, or standard procedure.

The descriptive information of the soil was considered. Color is an important property in identifying soils. There was red, gray and black cotton soil. The soil is fine grained if it contains 50% or more fines.

3.3 Sampling Methods and Sample Preservation

Test pits were excavated using hand tools and representative disturbed samples were taken. The disturbed samples had been handled and preserved to prevent contamination by foreign material and to ensure that the in situ soil conditions are preserved. An attempt was made to collect samples that should be representative of the in situ soil at the depth from which the sample was taken. The preserving and transporting of the samples were done according to ASTM D-4220-95 (Standard Practice for Preserving and Transporting of Soil samples).

3.4 Identification Material: To properly identify the origin of the samples all soil samples were properly marked. The name of the sampling area, the depth of the sampling, the sample location and the sampling date were written on each sampling container with waterproof permanent marker.

3.5 Packing of samples and Method of transportation

In general, disturbed samples do not require special transport precautions. However, the sample containers were protected from breakage and exposure to excessive moisture, which may cause deterioration of the labels and the sampling bags. Regarding transportation, the most satisfactory method of sample transportation is vehicle that it can be loaded at the exploration site and driven directly to testing laboratory. Since the sampling area was here in Addis Ababa, sample transportation was not any problem because it requires not more than an hour for transporting samples from the sampling area to the Geotechnical laboratory. Thus, there was no fear of exposing the sample to excessive heat, cold and contaminants.

3.6 Sample Size and Preparations

For fine-grained soils in preparation of the sample to determine the fine content of the soil, the soil sample to be tested was first air dried. Then sample was thoroughly pulverized with fingers and wood hammer. Representative sample of 400gm air dried sample was taken which pass No 40(0.425mm) and soaked in for 24hrs for LL, PL and vane shear tests.

For the compaction characteristics test, the required sample mass for procedure “A” of ASTM D 698 – 91 (Standard Test Method for Laboratory Compaction Characteristics of Soil Using

Standard Effort) is approximately 16kg of dry soil. Therefore, the field sample should have a moist mass of at least 23kg. However about 19kg of representative soil sample was pulverized on 4.75mm (No.4) sieve air-dried soil was prepared for each soil sample. The field soil sample which is collected for compaction characteristics is about 30kg.

4. LABORATORY SOIL TEST RESULTS AND ANALYSIS

4.1 INTRODUCTION

Based on the samples collected from the sites, laboratory tests were conducted in the Geotechnical laboratory of Addis Ababa Institute of Technology. The laboratory testing program includes: Atterberg limit, Standard Compaction test and Vane Shear test. The summaries of laboratory test results conducted on these samples are presented below But the detailed laboratory results are attached in the Appendix.

Table 4.1 Summary of Atterberg limit test results

S. No.	Site	Depth (m)	Liquid limit, %	Plastic limit, %	Plasticity index, %
1	Addisu Gebeya	2	49	17	32
2	Akaki Black	1	95	39	56
3	Akaki Gray	3	102	59	43
4	Hana Mariam	1.5	76	38	38
5	Lamberet	2	64	33	31
6	Semit	1.5	86	40	46

4.2 Vane Shear Test

The undrained shear strength of soft clays can be determined in the laboratory by a vane shear test. The laboratory test is conducted on Wykeham Farrance vane shear apparatus, the diameter and height of the vane is 12 mm. A specimen of the size 38 mm diameter and 76 mm height is put in a container which is fixed securely to the base. The vane is gradually lowered into the specimen till the top of the vane is at a depth of 10 to 20 mm below the top of the specimen. The readings of the torque indicator are taken and changed to torque with their respective spring number.

Since the sample is disturbed soil, it was remolded before the vane shear test was performed. First the LL and PL of each soil sample was obtained according to ASTM 4318-98 and it is presented in Appendix A. After that Standard compaction test of each soil sample was computed according to ASTM D 698 – 91 and it is presented in Appendix B. Twelve water contents from LL up to PL were selected and their respective dry density was known from standard compaction graph. The bulk density of the soil was calculated using equation 4.1.

The mass of the soil which is remolded is known by multiplying bulk density with the volume of the specimen and it is presented in Appendix C. The remolding and the vane shear test starts from the water content around LL and proceeds up to PL. A soil was placed in a specimen in two layers and compacted by 25 blows by the spatula to remold in the specimen. The vane shear test results are presented in Appendix D.

The LI of the soil is calculated using the moisture content, LL and PL of a soil according to equation 4.2

$\rho_t = \rho_d(1 + \omega)$.. (4.1)

$I_L = \frac{W - W_p}{W_L - W_p}$ .... (4.2)

Table 4.2 Typical Vane Shear test results

Liquidty Index		Addisu Gebeya soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
49	100	0.25	2.4525	1.2	1.2	0.864	0.288	3.619115	6.776519
45	87.5	1	9.81	1.2	1.2	0.864	0.288	3.619115	27.10608
42	78.125	1.9	18.639	1.2	1.2	0.864	0.288	3.619115	51.50154
39	68.75	2.5	24.525	1.2	1.2	0.864	0.288	3.619115	67.76519
36	59.375	3.16	30.9996	1.2	1.2	0.864	0.288	3.619115	85.6552
32	46.875	3.71	36.3951	1.2	1.2	0.864	0.288	3.619115	100.5635
29	37.5	4.29	42.0849	1.2	1.2	0.864	0.288	3.619115	116.2851
26	28.125	5.05	49.5405	1.2	1.2	0.864	0.288	3.619115	136.8857
24	21.875	5.75	56.4075	1.2	1.2	0.864	0.288	3.619115	155.8599
21	12.5	6.88	67.4928	1.2	1.2	0.864	0.288	3.619115	186.4898
19	6.25	7.2	70.632	1.2	1.2	0.864	0.288	3.619115	195.1637
17	0	7.45	73.0845	1.2	1.2	0.864	0.288	3.619115	201.9403

Where ω = water content
LI = Liquidity Index
T = Torque reading
D = Diameter of the vane blade
H = Height of the vane blade
$A = (\pi H D^2)/2$
$B = (\pi D^3/6)$
Cu = Undrained shear strength

4.3 ANALYSIS

4.3.1 General

The relationship of two variables can be expressed in a mathematical form by determining an equation connecting the two variables. Before obtaining the relationships between two variables, valid data must be collected from an experiment. In this research 12 test results for each of the 6 samples in total 72 test results were collected. In carrying out the statistical analysis, the statistical software program called a statistical package for social science software (SPSS) is employed to investigate and determine the scatter plot, correlation and regression.

Regression is a statistical technique to determine the relationship between two or more variables. It is concerned with the relationship between independent variable (X) and a dependent variable (Y) and shows us how variation in one variable co-occurs with variation in another. It is very useful in the field of engineering and science in modeling and investigating relationships between two or more variables. The method of regression analysis is used to develop a line or curve which provides the best fit through a set of data points. This basic approach is applicable in situations ranging from single linear regression to more sophisticate nonlinear multiple regressions. The best fit model could be in the form of linear, parabolic or logarithmic trend. A linear relationship is usually practiced in solving different engineering problems because of its simplicity. It is designed to study the relationship between a pair of variables that appear in a data set. Before the application of the analysis methods some important terms are discussed below.

1. **Coefficient:** is the parameter estimate for every increase or decrease in one variable, the change of another variable.

2. Standard Error: if the regression were performed repeatedly on different datasets (that contained the same variables), this would represent the standard deviation of the estimated coefficients. It measures the error of each sample point about the best fit line. Out of all lines the best fit line has the smallest standard error.

3. Confidence interval: 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. This means that we are 95% confident that we could make the right decision and we could be wrong with a probability of 5%.

4. R^2: this statistic represents the proportion of variation in the dependent variable that is explained by the model (the remainder represents the proportion that is present in the error). It is also the square of the correlation coefficient. The correlation coefficient measures how well the best fit line fits the sample data. Value of R=1 or -1 shows there is a perfect correlation. On the other hand R=0 or approaches to zero shows no valid relationship between the variables.

4.3.2 Scatter Plot and Best-Fit Curve

For this analysis, the liquid limit is considered as dependent variable and the undrained shear strength as independent variable. Prior to carrying out the regression analysis, a scatter diagram is generated by applying the SPSS software in order to study the relationships developed between the dependent variable and the independent one so as to determine the model that best suits the results.

The relationship between the dependent and independent variables are examined separately for the sites that the soils had taken and presented in Figs 4.1 to 4.6.

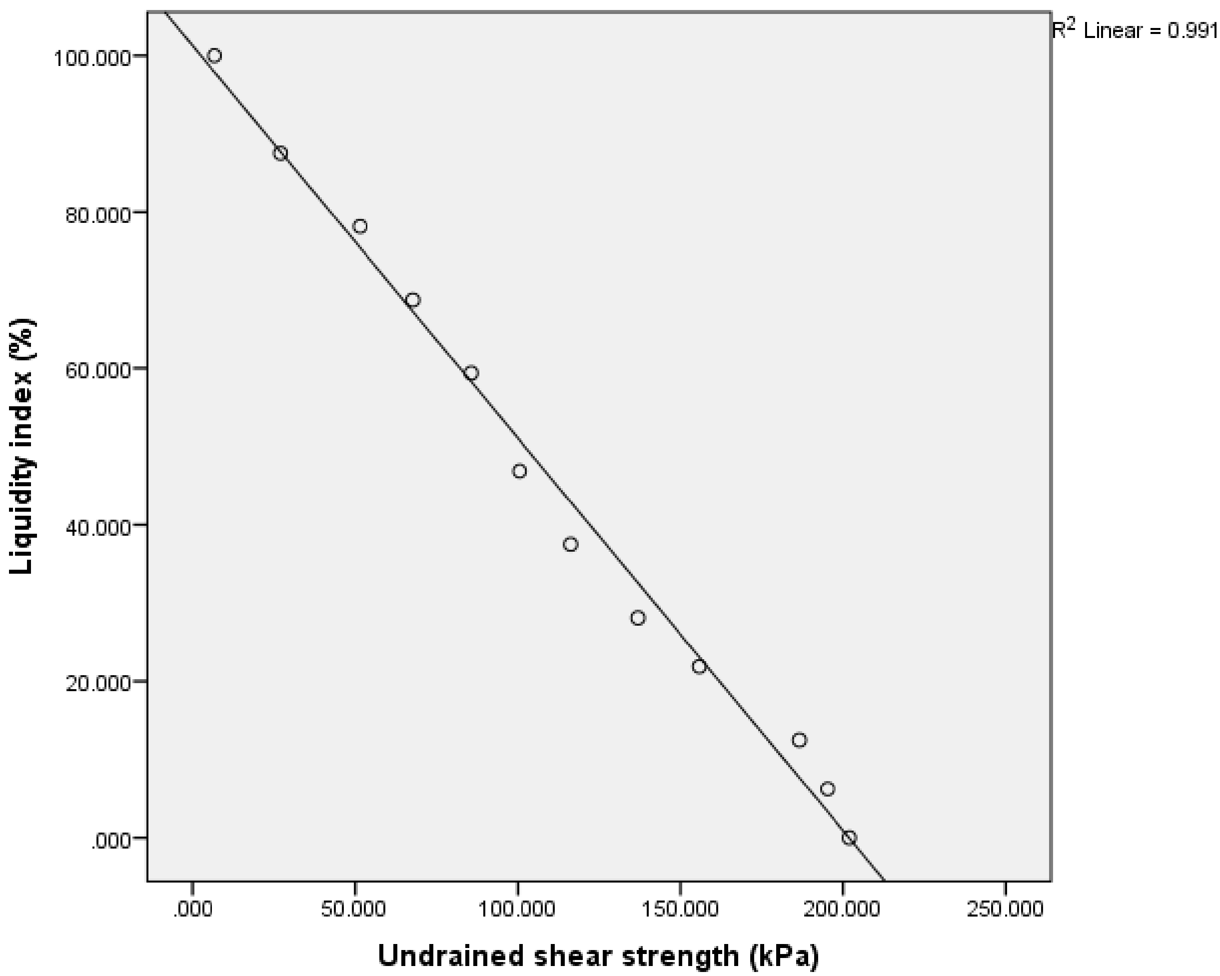


Fig 4.1 Addisu Gebeya soil Scatter plot and best-fit line of LI and Cu

- Pearson correlation coefficient, R = 0.995, R^2 = 0.991
- Standard error of the estimates (SE) = 0.34
- Proposed equation: $LI = 101.280 - 0.502C_u$

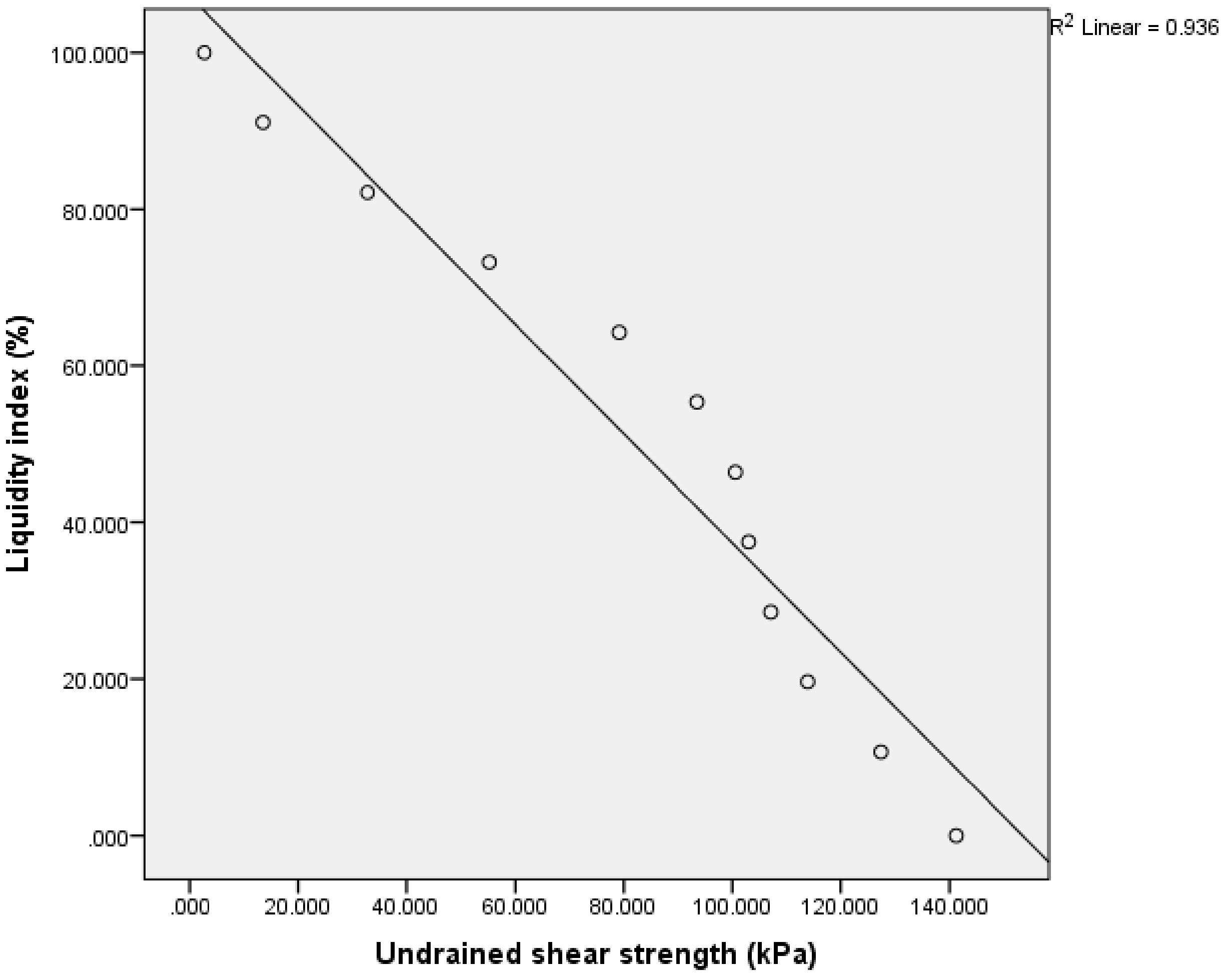


Fig 4.2 Akaki Black soil Scatter plot and best-fit line of LI and C_u

- Pearson correlation coefficient, R = 0.968, R^2 = 0.936
- Standard error of the estimates (SE) = 0.86
- Proposed equation: $LI = 107.193 - 0.698C_u$

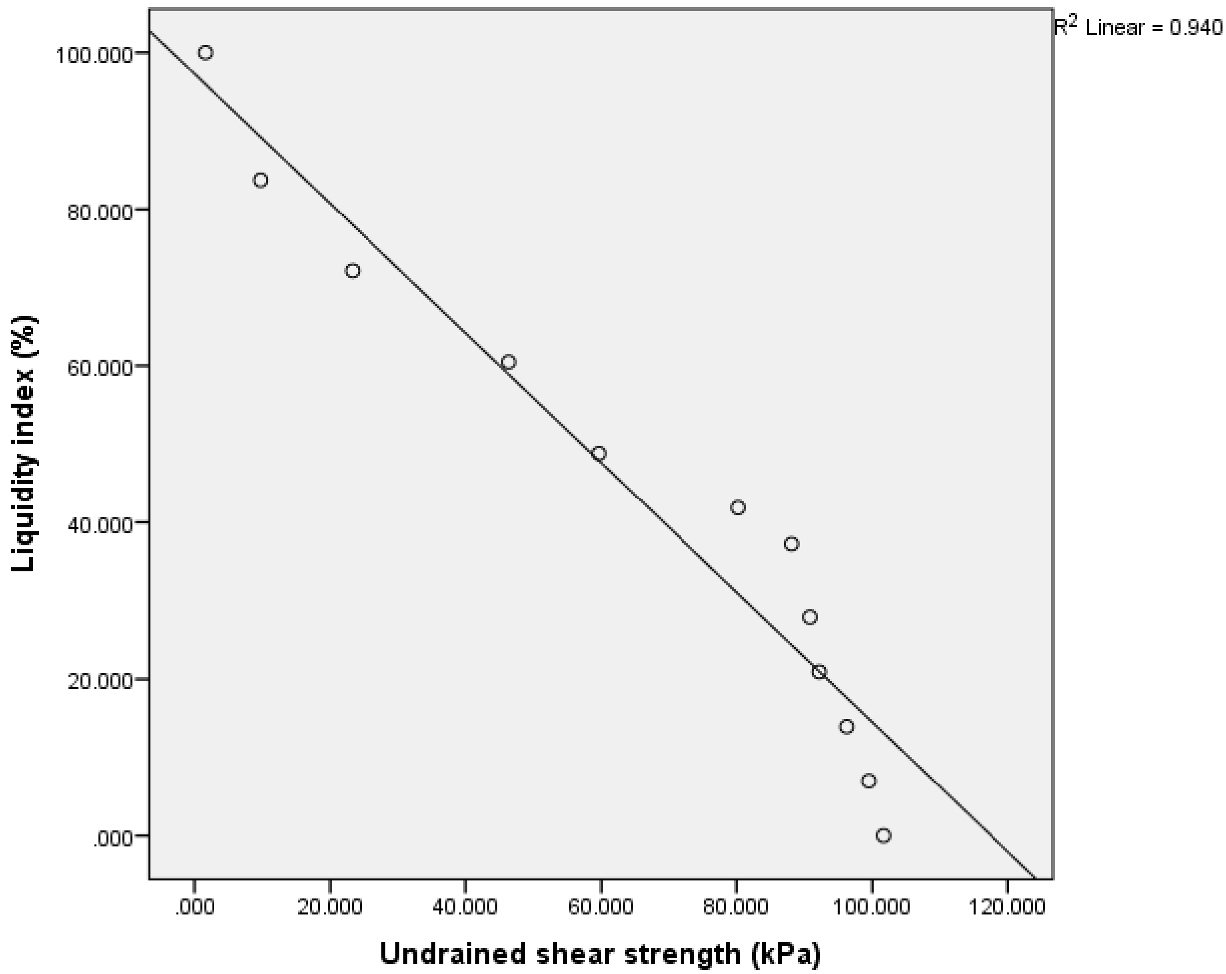


Fig 4.3 Akaki Gray soil Scatter plot and best-fit line of LI and C_u

- Pearson correlation coefficient, R = 0.969, R^2 = 0.940
- Standard error of the estimates (SE) = 0.81
- Proposed equation: LI = 97.287 – 0.828C_u

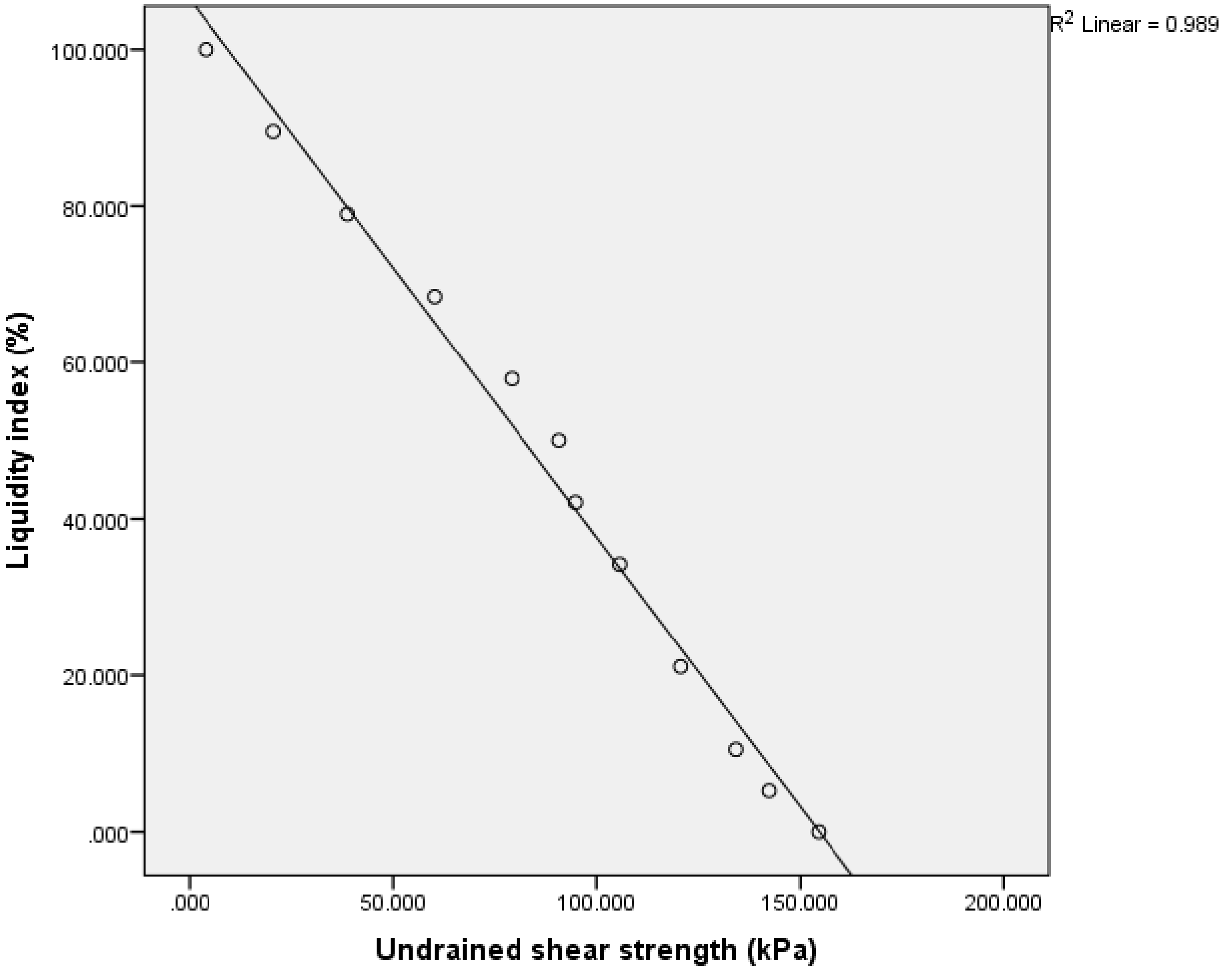


Fig 4.4 Hana Mariam soil Scatter plot and best-fit line of LI and C_u

- Pearson correlation coefficient, R = 0.995, R^2 = 0.989
- Standard error of the estimates (SE) = 0.37
- Proposed equation: $LI = 106.493 - 0.688C_u$

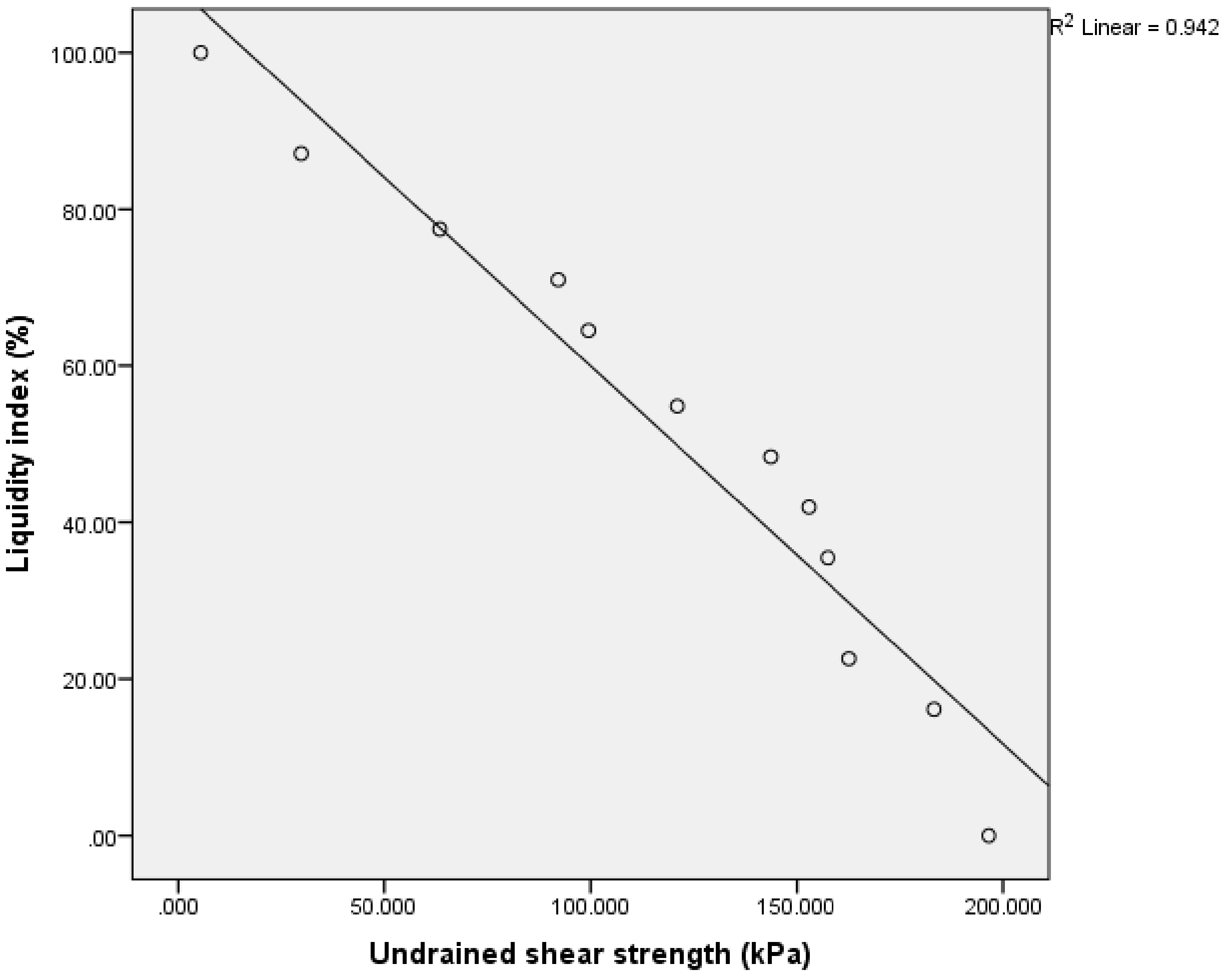


Fig 4.5 Lamberet soil Scatter plot and best-fit line of LI and C_u

- Pearson correlation coefficient, R = 0.971, R^2 = 0.942
- Standard error of the estimates (SE) = 0.76
- Proposed equation: $LI = 108.201 – 0.482C_u$

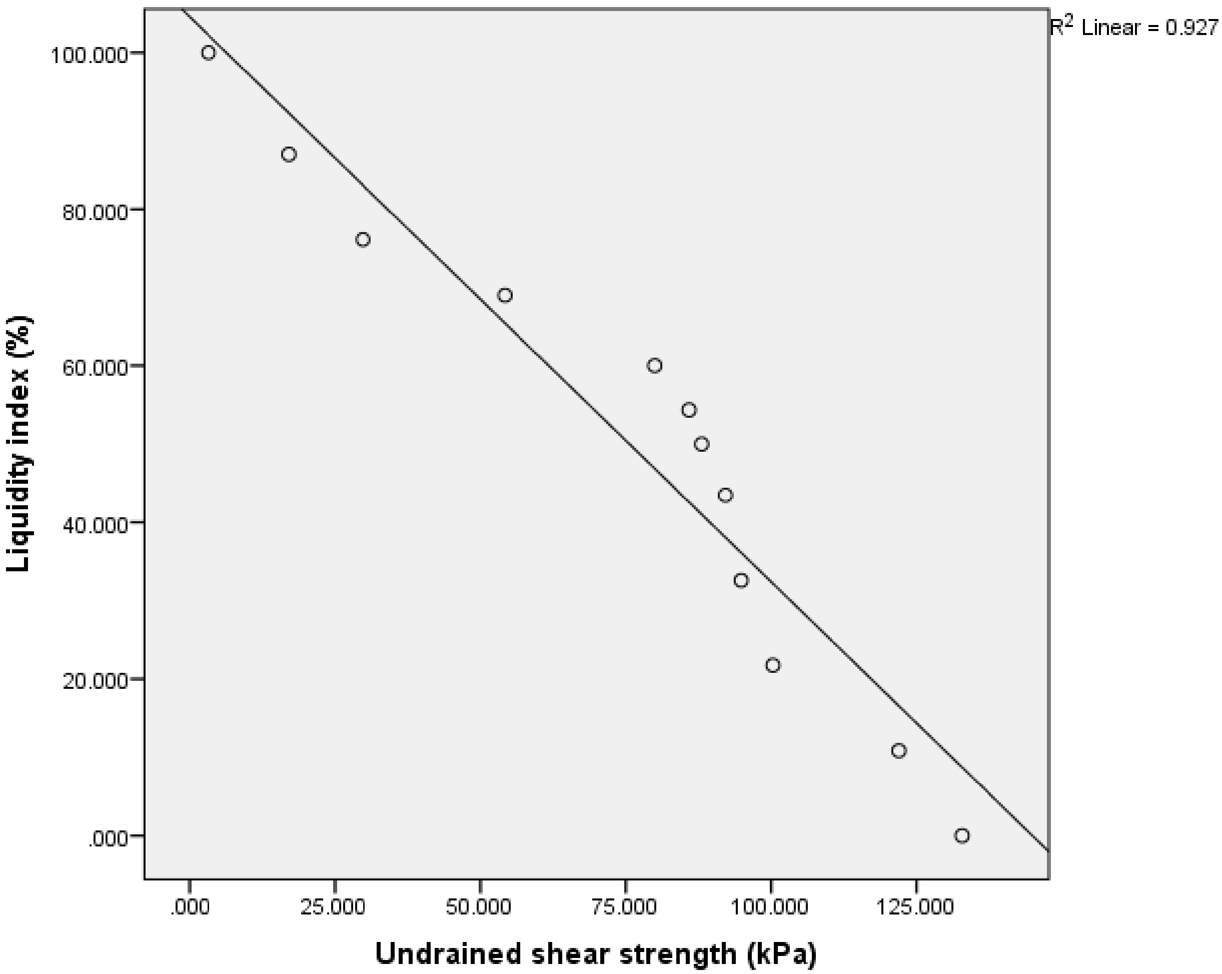


Fig 4.6 Semit soil Scatter plot and best-fit line of LI and Cu

- Pearson correlation coefficient, R = 0.963, R^2 = 0.927
- Standard error of the estimates (SE) = 0.86
- Proposed equation: LI = 104.565 – 0.721C_u

Table 4.3 Summary of the regression analysis

S.No.	Model equation	R^2 (Coefficient of determination)	No. of samples, n
1	LI = 101.280 - 0.502Cu	0.991	12
2	LI = 107.193 - 0.698Cu	0.936	12
3	LI = 97.287 - 0.828Cu	0.94	12
4	LI = 106.493 - 0.688Cu	0.989	12
5	LI = 108.201 - 0.482Cu	0.942	12
6	LI = 104.565 - 0.721Cu	0.927	12

Rearranging the equation

1. $Cu = 200.971 - 1.974LI$
2. $Cu = 148.878 - 1.341LI$
3. $Cu = 114.396 - 1.135LI$
4. $Cu = 153.952 - 1.437LI$
5. $Cu = 218.144 - 1.954LI$
6. $\mathrm{Cu} = 139.835 - 1.285\mathrm{LI}$

4.4 Comparative Graph Analysis with Previous Studies

Comparison of results of this study with the previous studies has been evaluated using EXCEL and presented in the below figures.

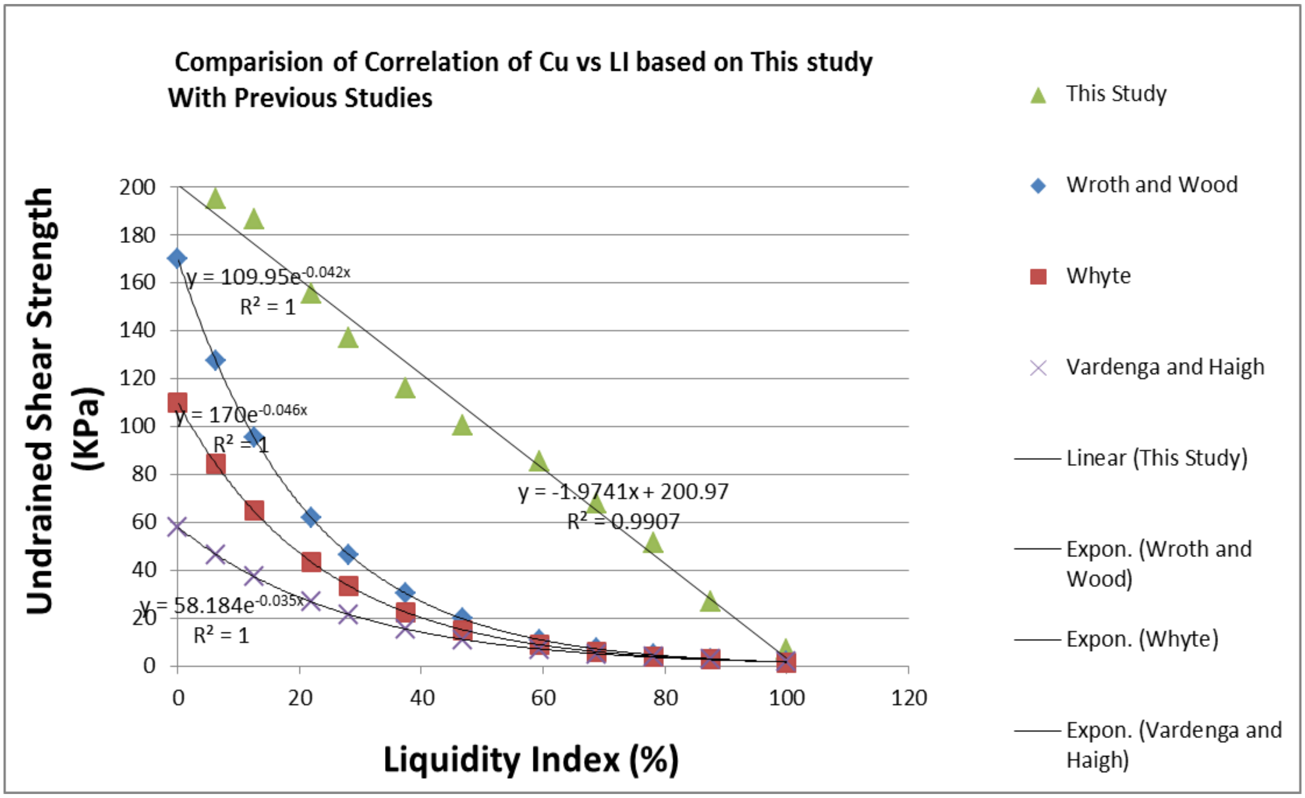


Fig 4.7 Comparison of previous studies with This Analysis for Addisu Gebeya soil

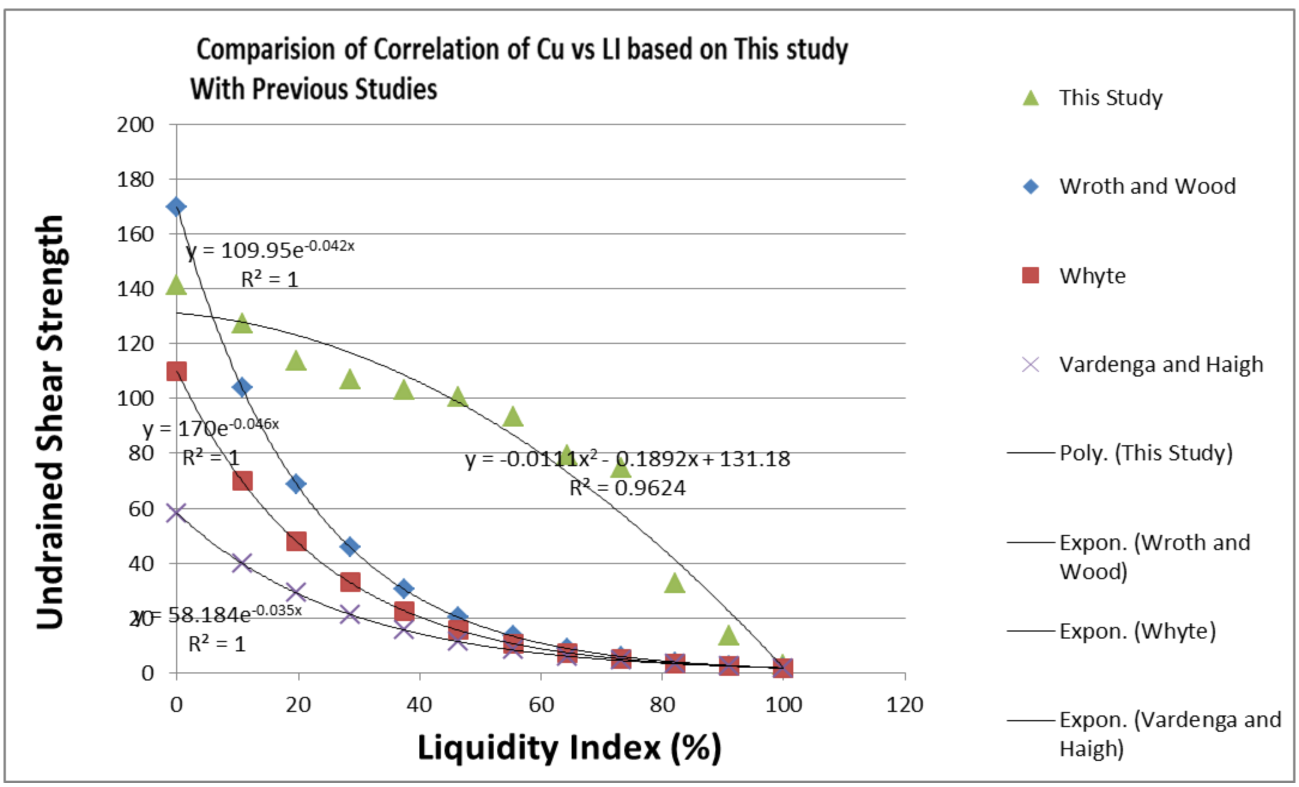


Fig 4.8 Comparison of previous studies with This Analysis for Akaki Black soil

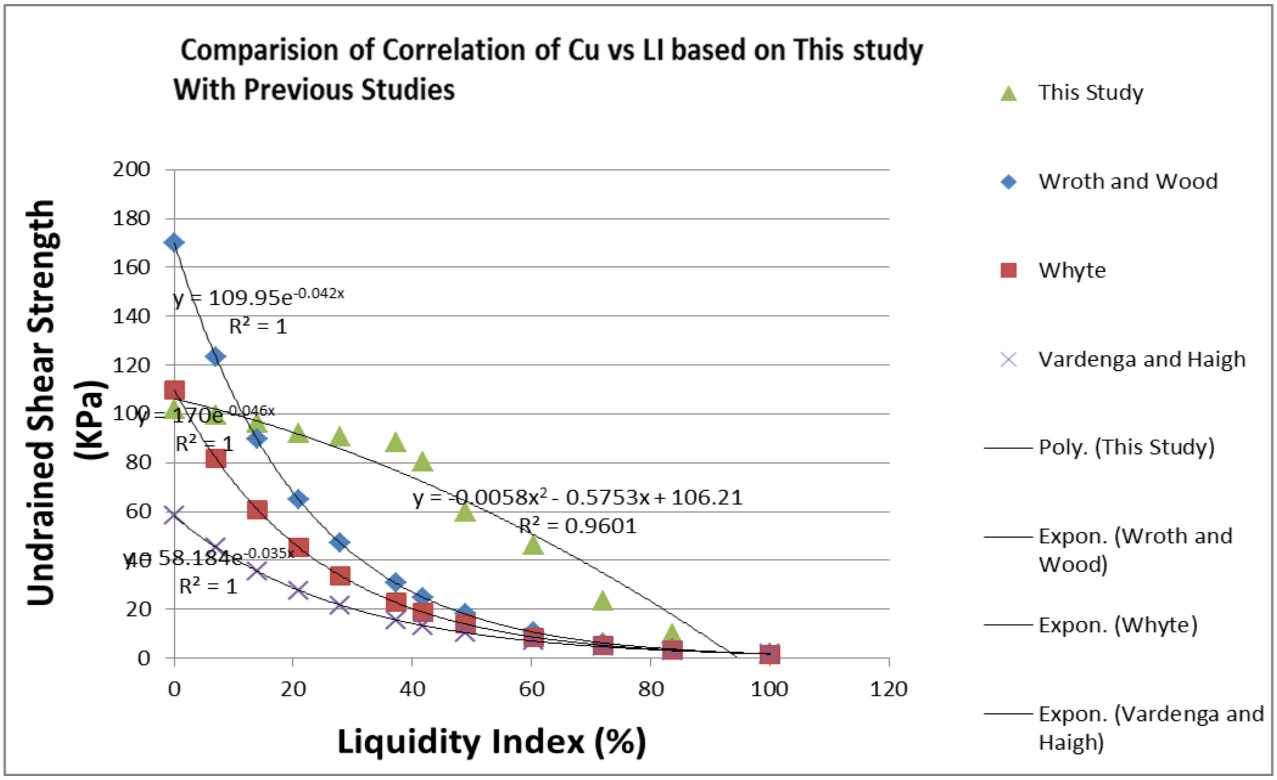


Fig 4.9 Comparison of previous studies with This Analysis for Akaki Gray soil

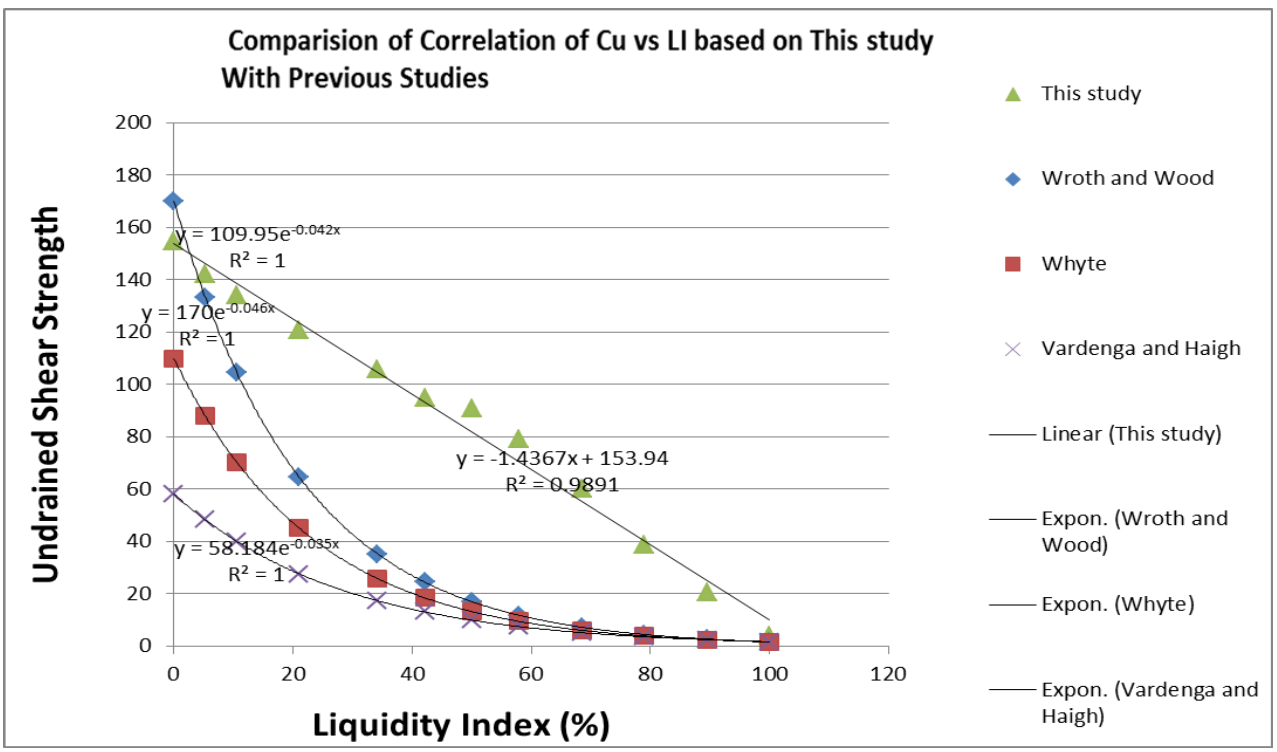


Fig 4.10 Comparison of previous studies with This Analysis for Hana Mariam soil

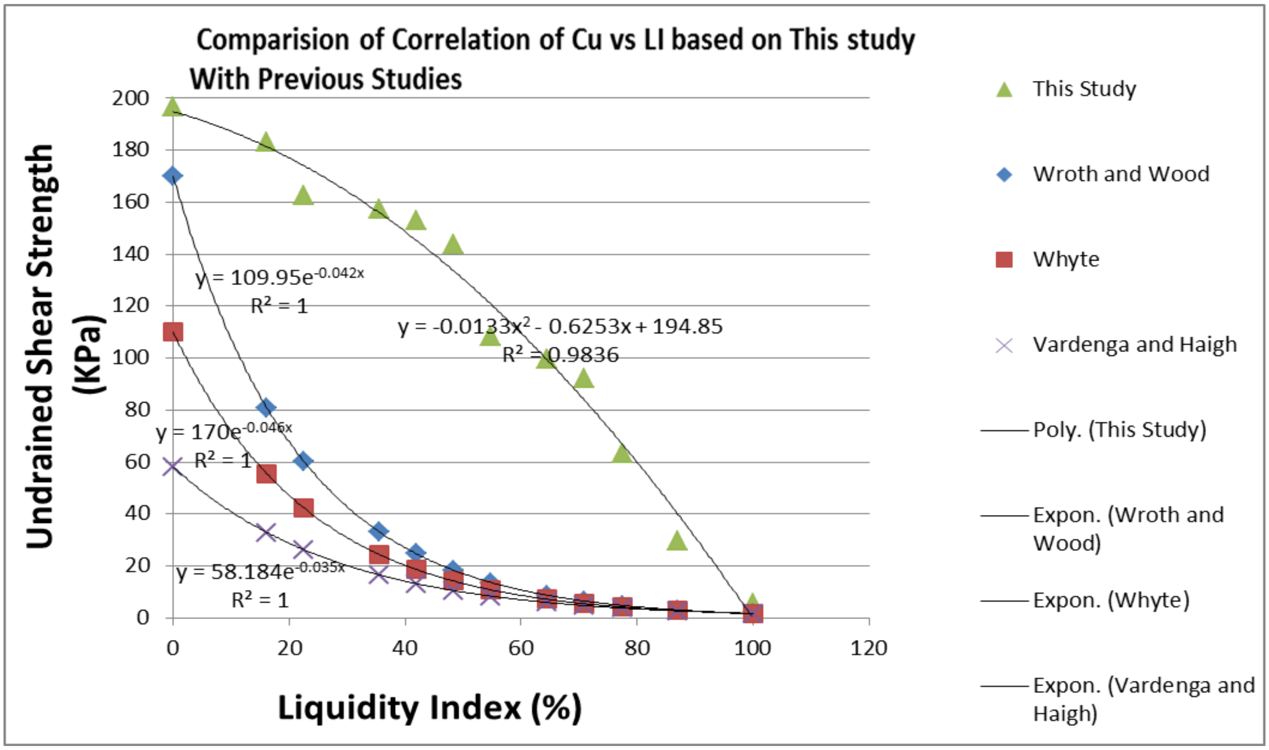


Fig 4.11 Comparison of previous studies with This Analysis for Lamberet soil

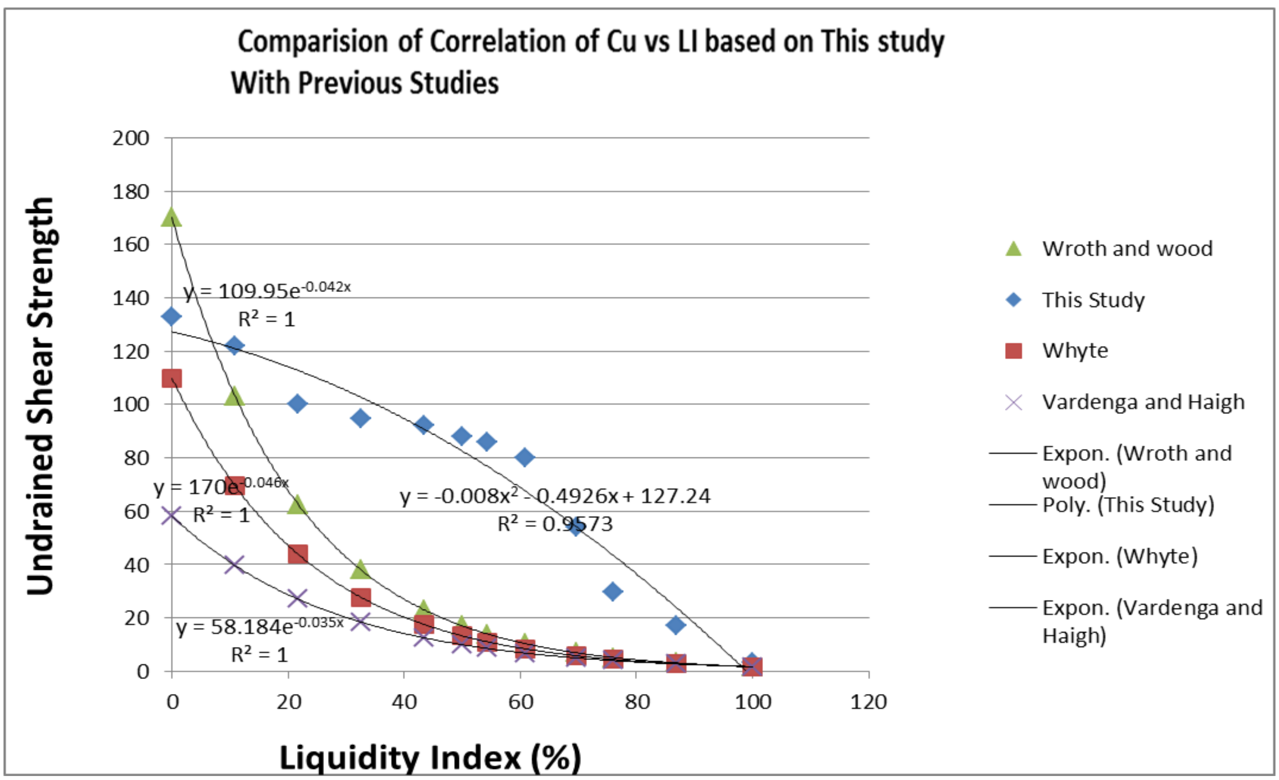


Fig 4.12 Comparison of previous studies with This Analysis for Semit soil

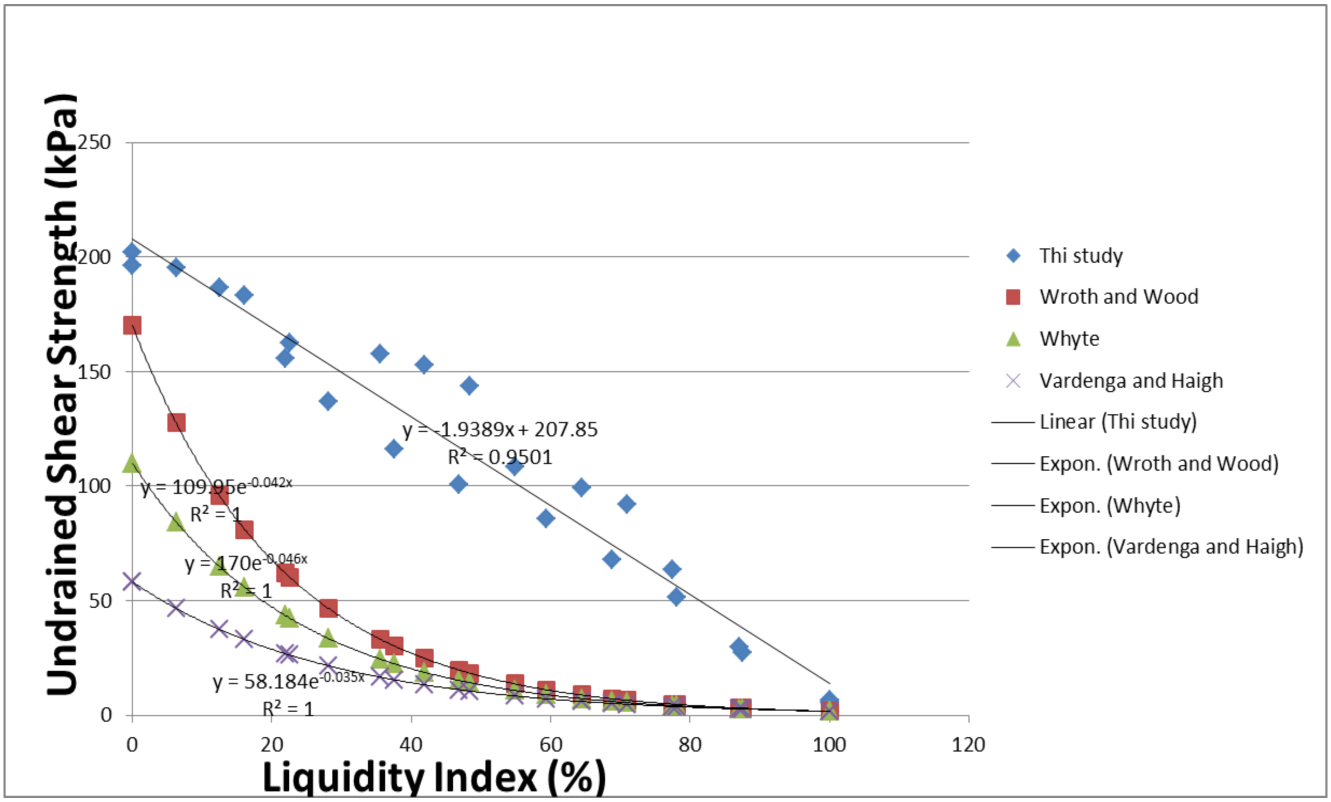


Fig 4.13 Comparison of previous studies with This Analysis for Red clay soil

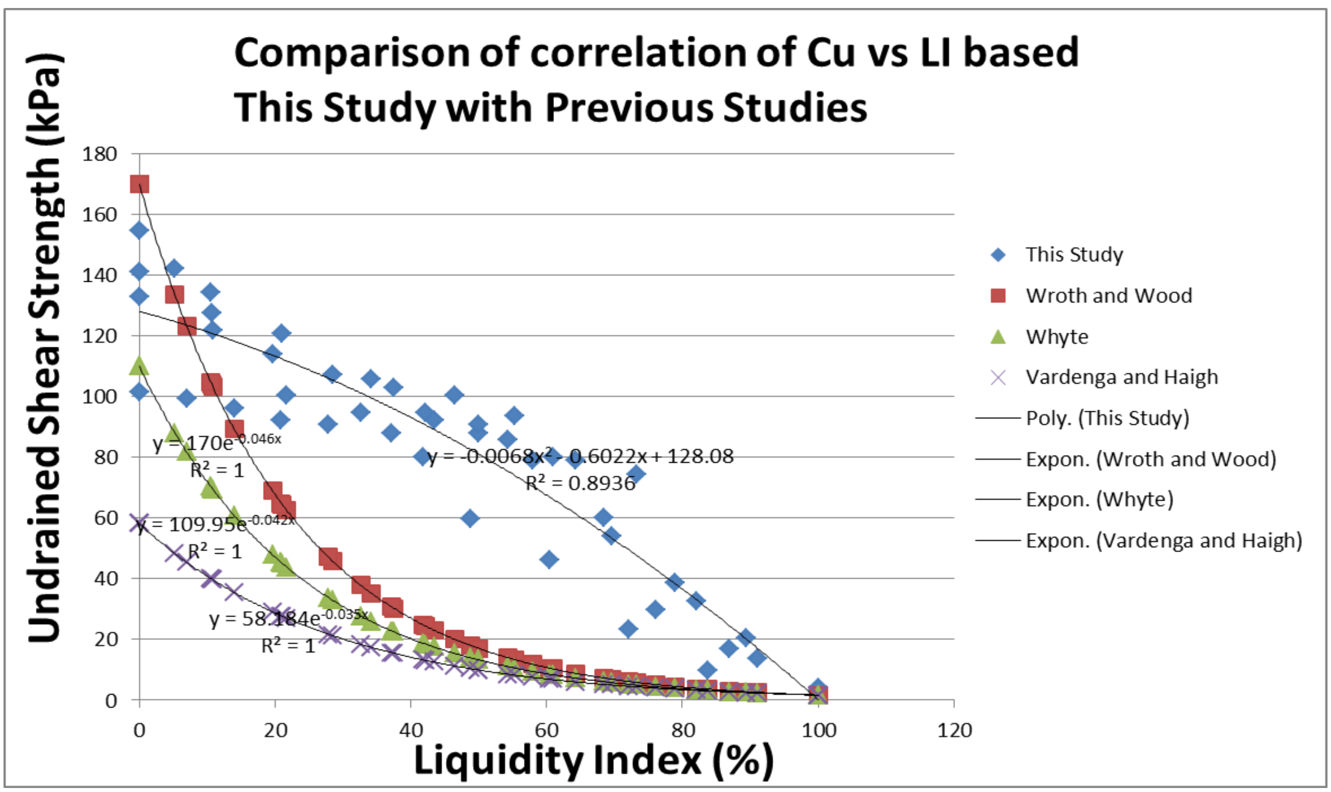


Fig 4.14 Comparison of previous studies with This Analysis for Black cotton soil

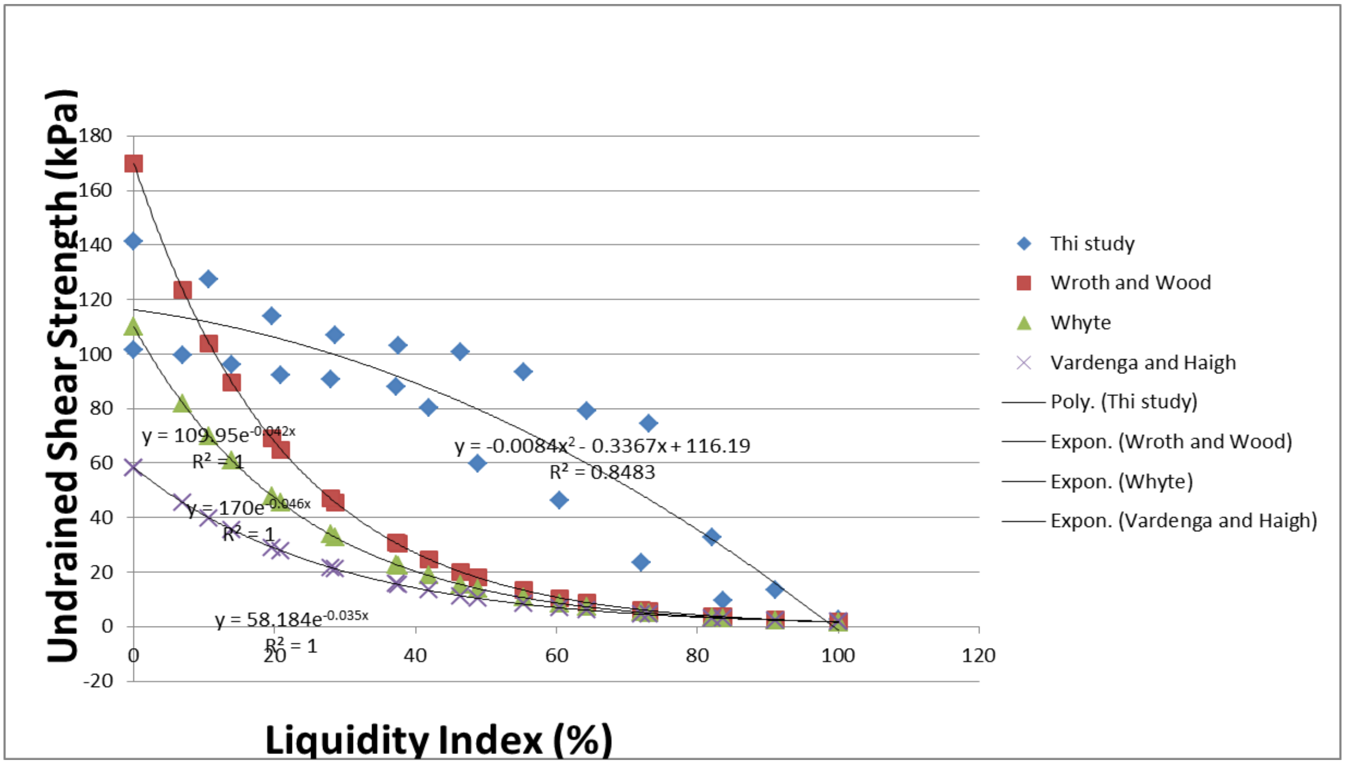


Fig 4.15 Comparison of previous studies with This study for Akaki black and Gray soil

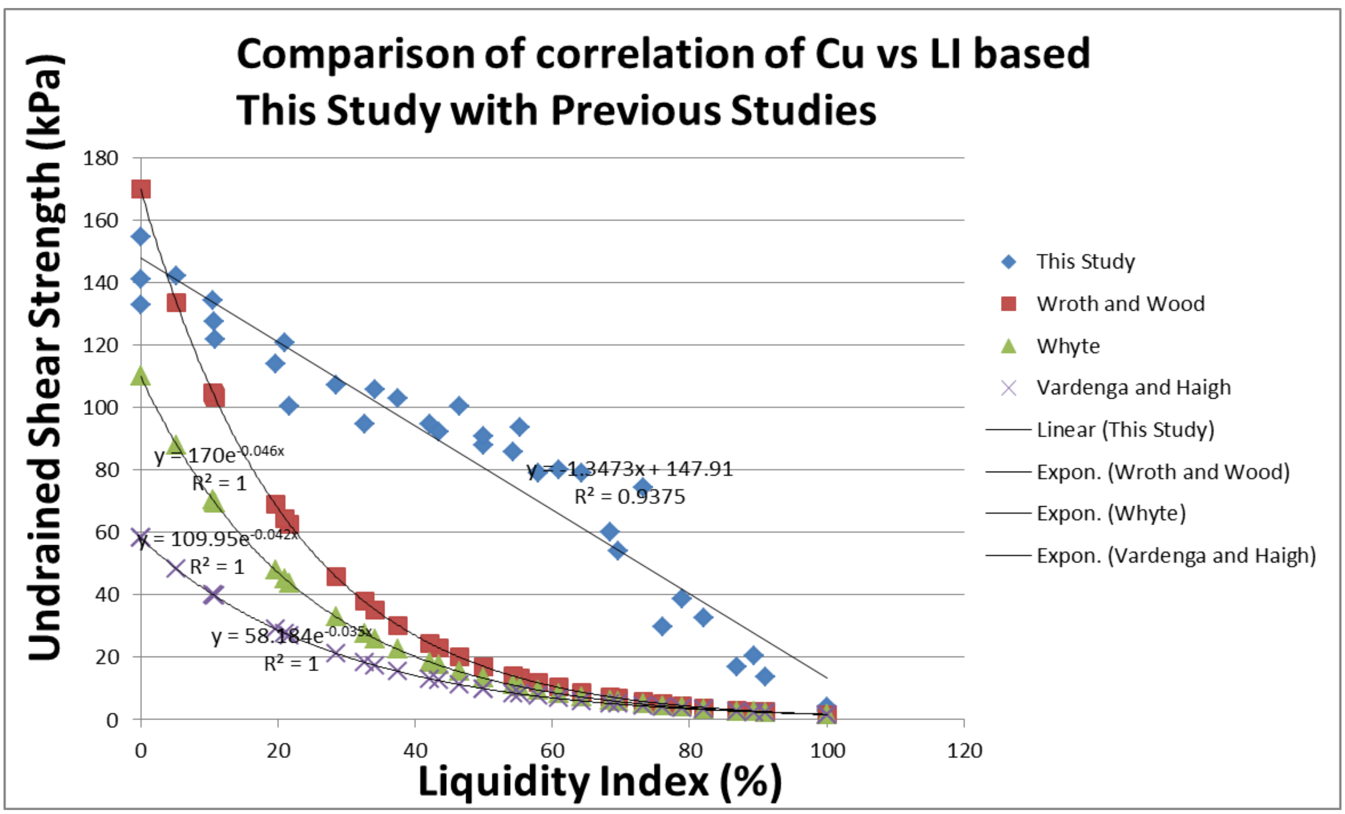


Fig 4.16 Comparison of previous studies with This study For Akaki black, Hana Mariam and Semit soil

5. Discussion of Results

The main objective of this research is to develop an equation that will allow determining the best mathematical relationship linking undrained shear strength with liquidity index. Liquidity index is chosen since the shear strength of the soil is low in the range from LL to PL the vane shear test is applied and also by varying the moisture content between LL and PL different LI can be calculated. From the database, a factor of ratio between the strength at liquid limit and plastic limit has been studied. The tests were done on red and black clay soil.

The results of Atterberg limits tests on 6 samples are presented in Appendix A. It ranges from 49-102 for liquid limits and from17-59 for plastic limits tests. Black clay soils have higher liquid and plastic limits than red clay soils. For each type of soil sample the VSTs are conducted at 12 levels of water content to determine C_u and it is presented in Appendix D. The minimum C_u value is 1.63KPa from Akaki Gray soil and the maximum value is 201.94KPa from Addisu Gebeya soil.

The Standard Compaction test was conducted to determine the 12 dry densities for each soil which correspond the 12 water content levels between liquid limit and plastic limit. In most of the cases the soil was difficult to compact when the water content is around LL, so it is extrapolated. Using the dry density and its respective water content the bulk density was known and by multiplying the bulk density with the volume of the specimen the mass of the soil which is to be remolded was known systematically. It ranges from 89 up to 164 gm. The tables and graphs for the standard compaction test and the mass to be remolded are presented in Appendix B and C respectively.

The vane shear test was conducted 12 times for each soil type on remolded soil. The red clay soils have high Cu values than Black cotton soils. The ratio between the strength at liquid limit and plastic limit ranges from 30 for Addisu Gebeya soil up to 63 for Akaki Gray soil. All of the tests results collected for this research are from experiment (primary data).

Table 5.1 The ratio factor between LL and PL

S No.	Name of locality	Depth (m)	Ratio factor between strength at LL and PL
1	Addisu Gebeya	1.5	30
2	Akaki-Kality	1	52
3	Akaki-Kality	3	63
4	Hana Mariam	1.5	38
5	Lamberet	1.5	36
6	Semit	1.5	41

For the correlation, using SPSS software linear regression equation is used for Analysis of the correlation For Cu vs LI. Significance is the most important parameter in the analysis and should be less than 0.05 for the equation to be accepted. In this analysis all the equations have satisfied this criterion, since their sig. is less than 0.05. The coefficient of determination (R^2) values range from 0.927 for Semit soil up to 0.991 for Addisu Gebeya soil. This is a strong correlation.

Table 5.2 The Equation between LI and Cu

S.No.	Location	Model equation	R^2 (Coefficient of determination)	sig.	No. of samples, n
1	Addisu Gebeya	$C_u = 200.971 - 1.974LI$	0.991	0	12
2	Akaki (1m)	$C_u = 148.878 - 1.341LI$	0.936	0	12
3	Akaki (3m)	$C_u = 114.396 - 1.135LI$	0.94	0	12
4	Hana Mariam	$C_u = 153.952 - 1.437LI$	0.989	0	12
5	Lamberet	$C_u = 218.144 - 1.954LI$	0.942	0	12
6	Semit	$C_u = 139.835 - 1.285LI$	0.927	0	12

For comparative analysis, Excel software is used for comparison of this study with the previous studies. From Fig. 4.7 up to Fig. 4.16 the comparison Analysis graphs are presented and it is clearly shown the equations of this study are statically fit to the previous studies. The previous studies are chosen due to their widely international acceptance and their variety of views on LL/PL relations. The researchers take LI up to 600% but in this research it is up to 100% only and that is the reason the graph is higher than those of the past. The previous studies uses hundreds of tests even using secondary data from different laboratories and the analysis shows an exponential graph but in this analysis the graph shows linear and parabolic form. This may be due to the number of tests between the current research and the previous researches.

6. Conclusions and Recommendations

Based on the analysis of data obtained from laboratory soil testing and the results of the study, the following conclusions and recommendations are drawn.

6.1 Conclusions

1. It was shown that the strength at the plastic limit to the strength at the liquid limit is different for different kinds of soils. The strength ratio between the two increases with the increasing soil plasticity. While there is a 30 fold difference between those strengths at the plastic limit and liquid limit for low plastic soils like Addisu Gebeya, the ratio jumps to 63 for Akaki Gray soil which is highly plastic soil. The 100-fold of strength ratio that is discussed in the literature seems to be very high when compared with this research result.

2. The C_u at the liquid limit ranged from 1.63KPa to 6.78KPa, while the C_u at the plastic limit ranges from 101.65KPa to 201.94KPa.

3. The main objective of this thesis was to obtain valid relationships between liquidity index and undrained shear strength of clays in Addis Ababa. From the developed correlations one would be in a position to determine the Cu from LI for clay soils of Addis Ababa. A very strong correlation between LI and Cu has been obtained.

6.2 Recommendation

•In this research it is observed that there is a relationship between LI and C_u for clay soils in Addis Ababa. For the future research the area coverage should increase to carry out such a study in other parts of Ethiopia.

•To study the impact of salt content of the soil on Undrained shear strength of the soil area in this research

•To extend the Study around the area of Shrinkage limit.

7. Reference

[1] **A. Sridharan, and K. Prakash,. (1999)**. “Mechanisms Controlling the Undrained Shear Strength Behaviour of Clays,” Can. Geotech J, 36(6), pp. 1030-1038.

[2] **Aram M. Raheem, and Mohammad S. Joshaghani., (2016),** “Modeling of shear strength-Water content relationship of ultra-soft clayey soil.” Int. J. of Advanced Research Vol. 4.

[3] **ASTM, 2000.,** “Standard test method for laboratory miniature vane shear test for saturated fine-grained soil.” ASTM Standard D4648-00. American Society for Testing and Materials (ASTM).

[4] **Ashenafi Tamerat., (2013),** “Study on Index properties and Swelling Pressure of Expansive Soils Found in Dukem.” AAU Research Center

[5] **Atsbeha Nerea, (2012),** “Prediction of compaction characteristics from Atterberg limits for fine-grained soils.” AAU Research Center

[6] **Dr K. R. Arora, (2004),** “Soil Mechanics and Foundation Engineering”

[7] **Dr P. J. Vardanega and Dr S. K. Haigh, (2014),** “The undrained strength – liquidity index relationship.” Can. Geotech J. 51:9, 1073-1086.

[8] **Federico, A., (1983),** “Relationships (Cu-W) and (Cu-S) for Remolded Rlayey Soils at High Water Content.” Riv Ital Geotech, 17(1), pp. 38-41

[9] **Flaate K.,(1966),** “Factors Influencing the Results of Vane Tests.” Can. Geotech J., Vol. 3, No. 1, pp 18-31.

[10] **Haigh S. K., Vardanega P. J. and Bolton M. D. (2013),** “The plastic limit of clays.” Geotechnique 63, 435-440.

[11]Joseph E. Bowels, (1988), “Foundation Analysis and Design.” Fifth Edition

[12] **Kamil Kayabali, OzgurAkturk, Mustafa Fener, AylaBulutUStun, OrhanDikmen and FurkanHamzaHarputlugil, (2015)** “Evaluation of Undrained Shear Strength of Fine-Grained soils in Consideration of Soil Plasticity.” Journal of Rock Mechanics and Geotechnical Engineering 7 (2015) 716-721

[13] **Kayabali K. and Tufenkci O. (2010),** "Undrained Shear Strength of Remolded Soils at Consistency Limits." Can. Geotech J., 47(3):b259-266

[14] **Kamil Kayabali, (2015),** "Revisiting the Bjerrum's correction factor: Use of the liquidity index for assessing the effect of soil plasticity on undrained shear strength." Journal of Rock Mechanics and Geotechnical Engineering.

[15] **Kulhawy, F. H., Trautmann, C. H., Beech, J. F., o'Rourke, T. D., McGuire, W., Wood, W. A., and Capano, C., (1983),** "Transmission Line Structure Foundations for Uplift-Compression Loading." Electric Power Research Institute, pp 412.

[16] **Laurence D. Wesley, (2010),** "Fundamentals of soil mechanics for sedimentary and residual soils."

[17] **Locat J. and Demers D. (1988),** "Viscosity, yield stress, remolded strength and liquidity index relationships for sensitive clays." Can. Geotech J., 25(4), 799-806

[18] **Nagaraj H. B., Sridharan A. and Mallikarjuna H. M. (2012),** "Re-examination of undrained strength at atterberg limits water contents." Geotechnical and Geological Engineering, 30, 727-736

[19] **Norman, L. E. J. (1958),** "A comparison of values of Liquid limit determined with apparatus having bases of different hardness." Geotechnique, 8, 79-85.

[20] **O'Kelly, B. C. (2013),** "Atterberg limits and remolded strength-water content relationships."Geotechnical Testing Journal,(36&6), 939-947.

[21] **Sharma, B. and Bora, P. K.,(2003).** "Plastic limit, liquid limit and Undrained Shear Strength of Soil – reappraisal." J. Geotechnical and Geoenvironmental Engineering, ASCE, 129(8), 774-777.

[22] **Skempton A. W. and Northey R. D. (1952),** "The sensitivity of clays." Geotechnique, 3, 30-53.

[23] **Skempton, and Boyce, (1983)., "**Deep sea drilling projects for shear strength measurement of clayey sediment using modified Wykeham Farrance laboratory Vane apparatus." Initial Report of the Deep sea Drilling Project, part v. university of claifornia; pp 1059-1067

[24] **Tanaka H. (1994),** "Vane shear strength of a Japanese marine clay and applicability of Bjerrum's correction factor." Soils and Foundations; 34(3): 39-48

[25] **Tanaka H, Sakakibara M, Goto K, Suzuki K, Fukazawa K. (1992),** "Properties of Japanese normally consolidated marine clays obtained from static piezocone penetration tests." Report of Port and Harbour Research Institute; 31(4):61-92

[26] **Wroth, C. P., and Wood, D. M., (1978).,** "The correlation of index properties with some basic engineering properties of soils." Can Geotech J., 15(2), 137-145.

[27] **Youssef, M. F., El Ramli, A. H., and El Demery M. (1965),** "Relationships between shear strength, consolidation, liquid limit and plastic limit for remolded clays." Proc. 6th Int. Conf. Soil Mech. Found. Eng., Montreal, 1, 126-129.

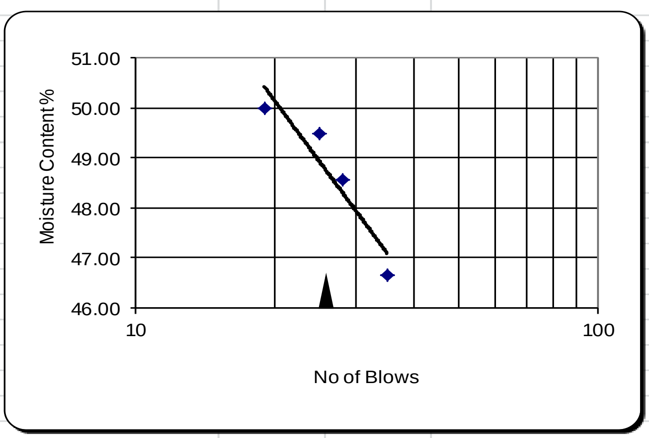
9. LIST OF APPENDIXES

Appendix A: Method of Computation for Atterberg Limits

	Addis Ababa University Institute of Technology	**Atterberg Limit**
	Civil Engineering Department Geotechnical Laboratory	

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Addisu Gebeya		
Sample of	Disturbed		
Depth	1.50 m		
Material			

		LIQUID LIMIT				PLASTIC LIMIT		
		1	2	3	4	1	2	3
No.of blows	N	19	25	28	35			
Tare	n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil	g	29	30.50	25.90	30.20	23.50	22.80	
Wt tare + dry soil	g	24.5	25.60	22.50	25.30	22.40	21.80	
Wt. water	g	4.50	4.90	3.40	4.90	1.10	1.00	
Wt. of tare	g	15.5	15.70	15.50	14.80	16.10	15.70	
Wt. of dry soil	g	9.00	9.90	7.00	10.50	6.30	6.10	
No.of blows	N	19	25	28	35			
Moisture content	%	50.00	49.49	48.57	46.67	17.46	16.39	
						AVERAGE PLASTIC LIMIT		17



LIQUID LIMIT (LL%)	
49	
PLASTIC LIMIT (PL %)	
17	
PLASTICITY INDEX (P.I)	
32	

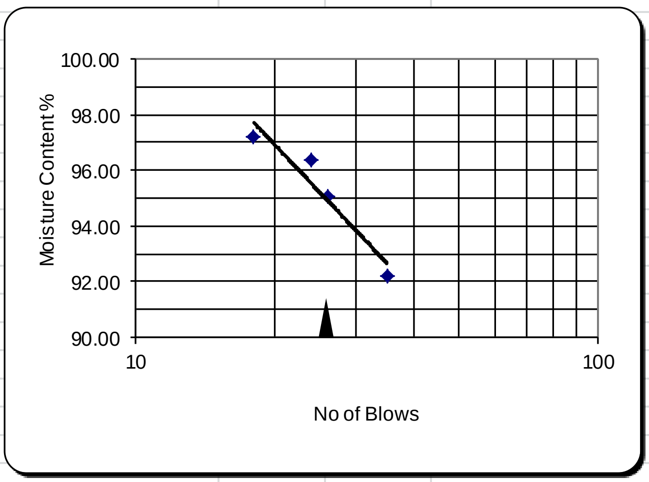
PASSING SIEVE %		

SOIL CLASSIFICATION (AASHTO)

	Addis Ababa University Institute of Technology	**Atterberg Limit**
	Civil Engineering Department Geotechnical Laboratory	

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Akaki		
Sample of	Disturbed		
Depth	1m		
Material			

	LIQUID LIMIT				PLASTIC LIMIT		
	1	2	3	4	1	2	3
No.of blows N	18	24	26	35			
Tare n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil g	29.9	26.40	35.60	32.80	24.70	152.30	
Wt tare + dry soil g	23	21.10	26.00	24.50	22.10	149.50	
Wt. water g	6.90	5.30	9.60	8.30	2.60	2.80	
Wt. of tare g	15.9	15.60	15.90	15.50	15.60	142.30	
Wt. of dry soil g	7.10	5.50	10.10	9.00	6.50	7.20	
No.of blows N	18	24	26	35			
Moisture content %	97.18	96.36	95.05	92.22	40.00	38.89	
					AVERAGE PLASTIC LIMIT		39



LIQUID LIMIT (LL%)
95
PLASTIC LIMIT (PL %)
39
PLASTICITY INDEX (P.I)
56

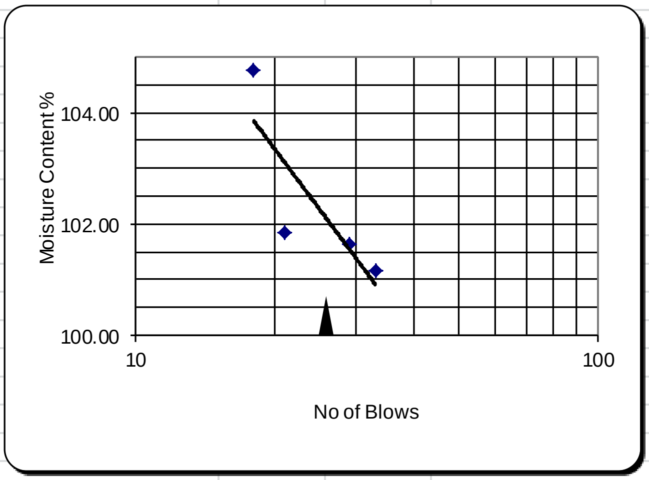
PASSING SIEVE %		

SOIL CLASSIFICATION (AASHTO)

	Addis Ababa University Institute of Technology	**Atterberg Limit**
	Civil Engineering Department Geotechnical Laboratory	

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Akaki		
Sample of	Disturbed		
Depth	3m		
Material			

		LIQUID LIMIT				PLASTIC LIMIT		
		1	2	3	4	1	2	3
No.of blows	N	18	21	29	33			
Tare	n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil	g	28.8	40.00	39.90	33.20	23.30	22.60	
Wt tare + dry soil	g	22.2	34.50	27.60	24.40	20.50	20.20	
Wt. water	g	6.60	5.50	12.30	8.80	2.80	2.40	
Wt. of tare	g	15.9	29.10	15.50	15.70	15.90	16.00	
Wt. of dry soil	g	6.30	5.40	12.10	8.70	4.60	4.20	
No.of blows	N	18	21	29	33			
Moisture content	%	104.76	101.85	101.65	101.15	60.87	57.14	
						AVERAGE PLASTIC LIMIT		59

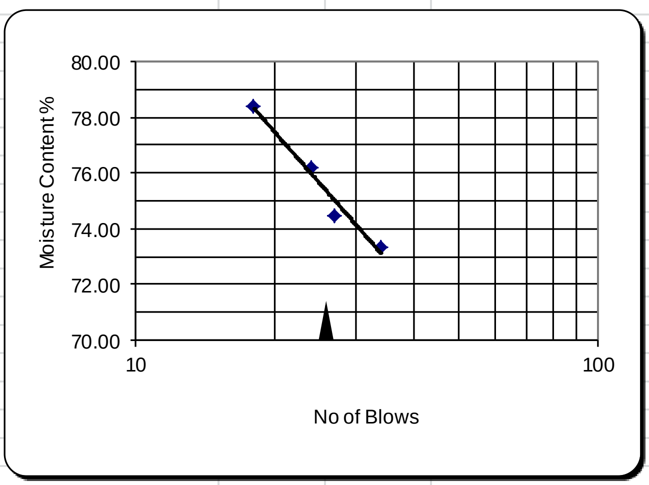


LIQUID LIMIT (LL%)		
102		
PLASTIC LIMIT (PL %)		
59		
PLASTICITY INDEX (P.I)		
43		
PASSING SIEVE %		
SOIL CLASSIFICATION (AASHTO)		

	Addis Ababa University Institute of Technology **Civil Engineering Department Geotechnical Laboratory**	**Atterberg Limit**

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Hana Mariam		
Sample of	Disturbed		
Depth	1.50 m		
Material			

		LIQUID LIMIT				PLASTIC LIMIT		
		1	2	3	4	1	2	3
No.of blows	N	18	24	27	34			
Tare	n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil	g	33.6	27.70	34.69	34.00	22.90	20.00	
Wt tare + dry soil	g	25.69	22.51	26.20	26.30	20.90	18.80	
Wt. water	g	7.91	5.19	8.49	7.70	2.00	1.20	
Wt. of tare	g	15.6	15.70	14.80	15.80	15.70	15.60	
Wt. of dry soil	g	10.09	6.81	11.40	10.50	5.20	3.20	
No.of blows	N	18	24	27	34			
Moisture content	%	78.39	76.21	74.47	73.33	38.46	37.50	
						AVERAGE PLASTIC LIMIT		38



LIQUID LIMIT (LL%)	76
PLASTIC LIMIT (PL %)	38
PLASTICITY INDEX (P.I)	38

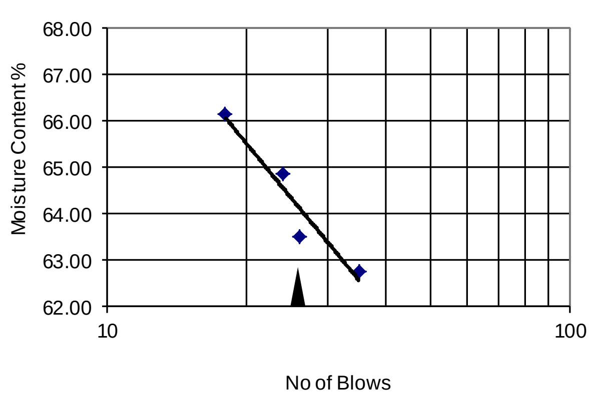
PASSING SIEVE %		

SOIL CLASSIFICATION (AASHTO)

	Addis Ababa University Institute of Technology **Civil Engineering Department Geotechnical Laboratory**	**Atterberg Limit**

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Lamberet		
Sample of	Disturbed		
Depth	1.50 m		
Material			

		LIQUID LIMIT				PLASTIC LIMIT		
		1	2	3	4	1	2	3
No.of blows	N	18	24	26	35			
Tare	n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil	g	26.4	28.00	38.00	30.60	25.40	29.10	
Wt tare + dry soil	g	22.1	23.20	29.30	24.70	22.90	25.80	
Wt. water	g	4.30	4.80	8.70	5.90	2.50	3.30	
Wt. of tare	g	15.6	15.80	15.60	15.30	15.40	15.70	
Wt. of dry soil	g	6.50	7.40	13.70	9.40	7.50	10.10	
No.of blows	N	18	24	26	35			
Moisture content	%	66.15	64.86	63.50	62.77	33.33	32.67	
						AVERAGE PLASTIC LIMIT		33



LIQUID LIMIT (LL%)
64
PLASTIC LIMIT (PL %)
33
PLASTICITY INDEX (P.I)
31

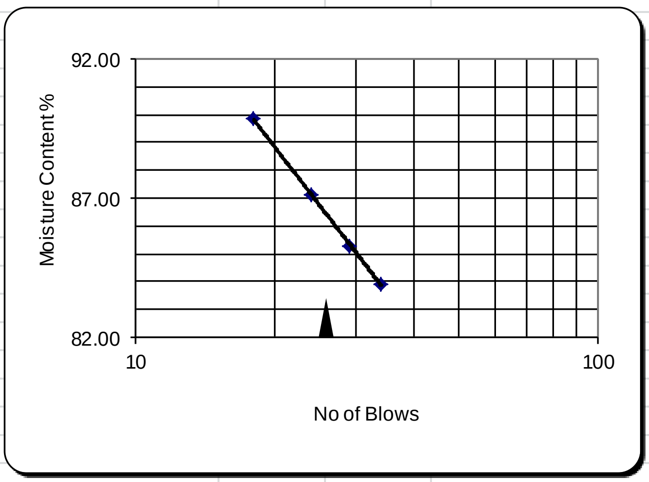
PASSING SIEVE %		

SOIL CLASSIFICATION (AASHTO)

Addis Ababa University Institute of Technology		**Atterberg Limit**
Civil Engineering Department Geotechnical Laboratory		

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Semit		
Sample of	Disturbed		
Depth	1.50 m		
Material			

	LIQUID LIMIT				PLASTIC LIMIT		
	1	2	3	4	1	2	3
No.of blows N	18	24	29	34			
Tare n	C7	B6	A48	A18	A4	B7	
Wt. tare + wet soil g	30.1	24.50	25.00	25.90	20.00	20.90	
Wt tare + dry soil g	23.38	20.03	20.72	21.20	18.74	19.40	
Wt. water g	6.72	4.47	4.28	4.70	1.26	1.50	
Wt. of tare g	15.9	14.90	15.70	15.60	15.60	15.70	
Wt. of dry soil g	7.48	5.13	5.02	5.60	3.14	3.70	
No.of blows N	18	24	29	34			
Moisture content %	89.84	87.13	85.26	83.93	40.13	40.54	
					AVERAGE PLASTIC LIMIT		40



LIQUID LIMIT (LL%)	86
PLASTIC LIMIT (PL %)	40
PLASTICITY INDEX (P.I)	46
PASSING SIEVE %	
SOIL CLASSIFICATION (AASHTO)	

Appendix B: Method of computation for Compaction Characteristics

	Addis Ababa University Institute of Technology Civil Engineering Department Geotechnical Laboratory	Standard Compaction for Moisture Density Relation of Soil

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Addisu Gebeya		
Sample of	Disturbed		
Depth	1.50 m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

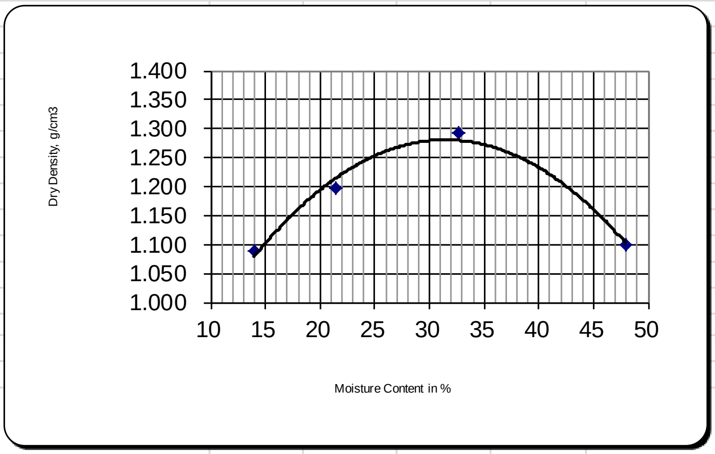
A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4300	4502.3	4748.6	4665	
C	Wt. of Mold	grams	3129	3129	3129	3129	
D	Wt. Wet Soil	grams	1171	1373	1619	1536	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.24	1.45	1.72	1.63	
G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	42.60	28.60	29.80	33.60	
I	Wt. Cont + Dry soil	grams	39.30	26.30	26.30	27.80	
J	Weight of Water	grams	3.30	2.30	3.50	5.80	
K	Weight of Container	grams	15.70	15.60	15.60	15.70	
L	Weight of Dry Soil	grams	23.60	10.70	10.70	12.10	
M	Moisture Content	%	14.0	21.5	32.7	47.9	
N	Dry Density	gr/cu.cm.	1.088	1.197	1.292	1.100	

Maximum Dry Density :

MDD = ________ gm/cc

Optimum Moisture Content :

OMC = ________ %



Addis Ababa University Institute of Technology

Civil Engineering Department Geotechnical Laboratory

Standard Compaction for Moisture Density Relation of Soil

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Akaki		
Sample of	Disturbed		
Depth	1m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

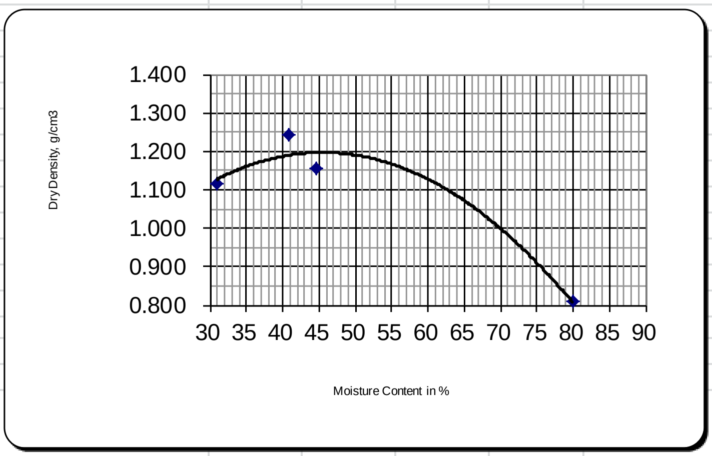
A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	6050	6321	6246.2	6042	
C	Wt. of Mold	grams	4669	4669	4669	4669	
D	Wt. Wet Soil	grams	1381	1652	1577	1373	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.46	1.75	1.67	1.45	
G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	43.10	36.90	42.20	64.30	
I	Wt. Cont + Dry soil	grams	36.60	30.80	34.00	42.80	
J	Weight of Water	grams	6.50	6.10	8.20	21.50	
K	Weight of Container	grams	15.60	15.90	15.60	15.90	
L	Weight of Dry Soil	grams	21.00	14.90	18.40	26.90	
M	Moisture Content	%	31.0	40.9	44.6	79.9	
N	Dry Density	gr/cu.cm.	1.117	1.241	1.155	0.808	

Maximum Dry Density :

MDD = ______ gm/cc

Optimum Moisture Content :

OMC = ______ %



	Addis Ababa University Institute of Technology **Civil Engineering Department Geotechnical Laboratory**	**Standard Compaction for Moisture Density Relation of Soil**

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Akaki		
Sample of	Disturbed		
Depth	3m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4400	4630.7	4748.6	4665	
C	Wt. of Mold	grams	3129	3129	3129	3129	
D	Wt. Wet Soil	grams	1271	1501	1619	1536	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.35	1.59	1.72	1.63	

G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	42.60	29.80	33.60	203.00	
I	Wt. Cont + Dry soil	grams	36.20	25.40	26.90	174.50	
J	Weight of Water	grams	6.40	4.40	6.70	28.50	
K	Weight of Container	grams	15.70	15.60	15.70	142.40	
L	Weight of Dry Soil	grams	20.50	9.80	11.20	32.10	

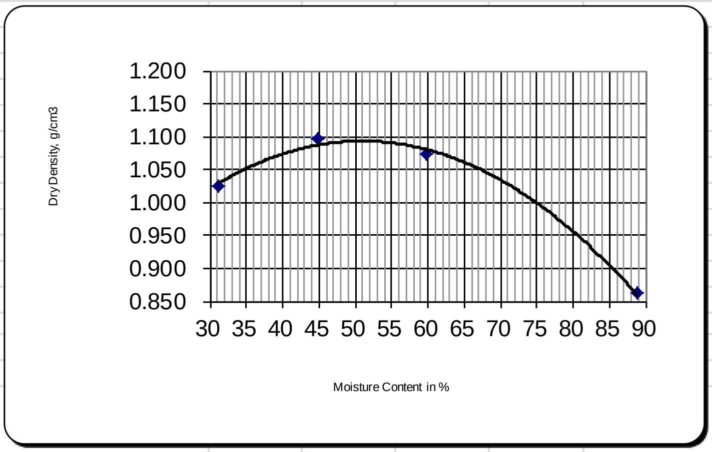
M	Moisture Content	%	31.2	44.9	59.8	88.8	
N	Dry Density	gr/cu.cm.	1.026	1.098	1.073	0.862	

Maximum Dry Density :

MDD = ______ gm/cc

Optimum Moisture Content :

OMC = ______ %



Addis Ababa University Institute of Technology Civil Engineering Department Geotechnical Laboratory	Standard Compaction for Moisture Density Relation of Soil

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Hana Mariam		
Sample of	Disturbed		
Depth	1.50 m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4618.3	4953.5	4779.5	4665	
C	Wt. of Mold	grams	3116	3116	3116	3116	
D	Wt. Wet Soil	grams	1503	1838	1664	1549	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.59	1.95	1.76	1.64	

G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	31.70	55.50	53.90	203.00	
I	Wt. Cont + Dry soil	grams	28.50	44.60	40.00	177.40	
J	Weight of Water	grams	3.20	10.90	13.90	25.60	
K	Weight of Container	grams	15.80	15.60	15.70	142.40	
L	Weight of Dry Soil	grams	12.70	29.00	24.30	35.00	

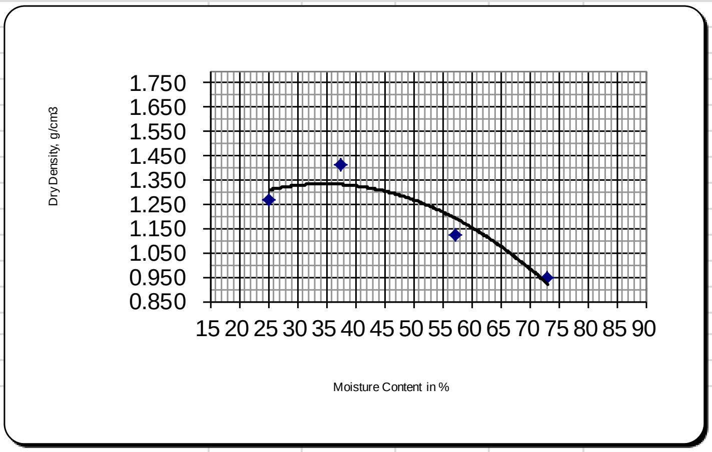
M	Moisture Content	%	25.2	37.6	57.2	73.1	
N	Dry Density	gr/cu.cm.	1.271	1.415	1.121	0.948	

Maximum Dry Density :

MDD = ____ gm/cc

Optimum Moisture Content :

OMC = ____ %



	Addis Ababa University Institute of Technology Civil Engineering Department Geotechnical Laboratory	Standard Compaction for Moisture Density Relation of Soil

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Lamberet		
Sample of	Disturbed		
Depth	1.50 m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4556	4782.3	4730	4620	
C	Wt. of Mold	grams	3116	3116	3116	3116	
D	Wt. Wet Soil	grams	1440	1667	1614	1504	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.53	1.77	1.71	1.59	

G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	35.60	49.10	48.00	26.43	
I	Wt. Cont + Dry soil	grams	32.10	40.80	36.40	22.50	
J	Weight of Water	grams	3.50	8.30	11.60	3.93	
K	Weight of Container	grams	15.70	15.10	11.40	15.60	
L	Weight of Dry Soil	grams	16.40	25.70	25.00	6.90	

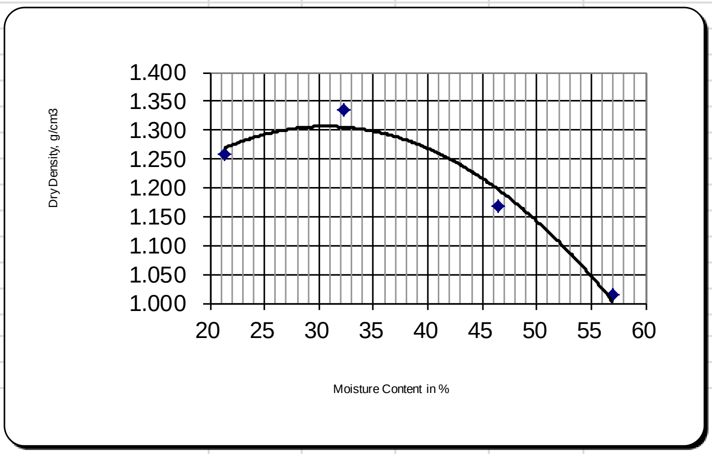
M	Moisture Content	%	21.3	32.3	46.4	57.0	
N	Dry Density	gr/cu.cm.	1.257	1.334	1.168	1.015	

Maximum Dry Density :

MDD = ________ gm/cc

Optimum Moisture Content :

OMC = ________ %



	Addis Ababa University Institute of Technology **Civil Engineering Department Geotechnical Laboratory**	**Standard Compaction for Moisture Density Relation of Soil**

Project:-	Geotechnical Investigation Program	**Date started:-**	20/03/17
Client:-			
Sampled by:-	Jara Mengistu		
Station	Semit		
Sample of	Disturbed		
Depth	1.50 m		
Material			

TEST METHOD : AASHTO T- 99 METHOD A

A	Mold	No.	1	2	3	4	5
B	Wt. of Mold + Wet Soil	grams	4448	4711.2	4760	4703.9	
C	Wt. of Mold	grams	3128	3128	3128	3128	
D	Wt. Wet Soil	grams	1320	1583	1632	1576	
E	Volume of Mold	cu.cm.	944	944	944	944	
F	Wet Density	gr/cu.cm.	1.40	1.68	1.73	1.67	

G	Container	No.	G8	F2	B9		
H	Wt. Cont + Wet soil	grams	50.80	160.70	135.80	38.60	
I	Wt. Cont + Dry soil	grams	44.30	155.50	128.90	28.90	
J	Weight of Water	grams	6.50	5.20	6.90	9.70	
K	Weight of Container	grams	16.50	142.40	115.50	15.00	
L	Weight of Dry Soil	grams	27.80	13.10	13.40	13.90	

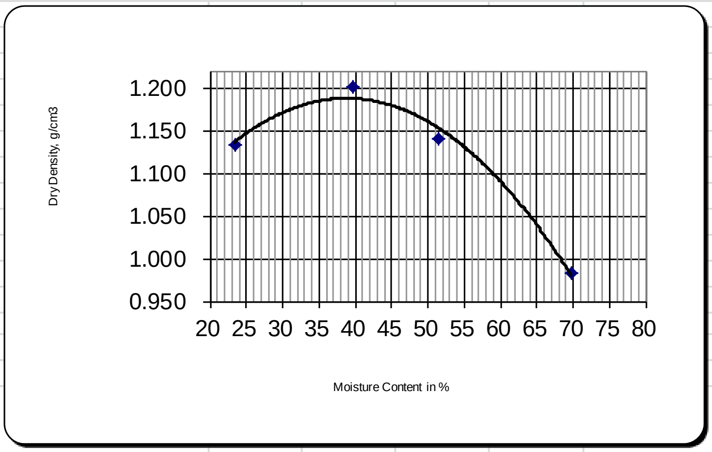
M	Moisture Content	%	23.4	39.7	51.5	69.8	
N	Dry Density	gr/cu.cm.	1.133	1.200	1.141	0.983	

Maximum Dry Density :

MDD = ________ gm/cc

Optimum Moisture Content :

OMC = ________ %



Appendix C: Method of determining the mass of the soil for remolding in the specimen

	Addisu Gebeya			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
17	1.145	1.33965	86.192736	115.4681
19	1.18	1.4042	86.192736	121.0318
21	1.205	1.45805	86.192736	125.6733
24	1.245	1.5438	86.192736	133.0643
26	1.26	1.5876	86.192736	136.8396
29	1.275	1.64475	86.192736	141.7655
32	1.28	1.6896	86.192736	145.6312
36	1.26	1.7136	86.192736	147.6999
39	1.245	1.73055	86.192736	149.1608
42	1.21	1.7182	86.192736	148.0964
45	1.16	1.682	86.192736	144.9762
49	1.1	1.639	86.192736	141.2699

	Akaki black cotton soil			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
39	1.19	1.6541	86.192736	142.5714
45	1.2	1.74	86.192736	149.9754
50	1.195	1.7925	86.192736	154.5005
55	1.16	1.798	86.192736	154.9745
60	1.13	1.808	86.192736	155.8365
65	1.07	1.7655	86.192736	152.1733
70	1	1.7	86.192736	146.5277
75	0.9	1.575	86.192736	135.7536
80	0.8	1.44	86.192736	124.1175
85	0.71	1.3135	86.192736	113.2142
90	0.62	1.178	86.192736	101.535
95	0.53	1.0335	86.192736	89.08019

	Akaki Gray soil			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
59	1.08	1.7172	86.192736	148.0102
62	1.07	1.7334	86.192736	149.4065
65	1.06	1.749	86.192736	150.7511
68	1.05	1.764	86.192736	152.044
71	1.03	1.7613	86.192736	151.8113
75	1	1.75	86.192736	150.8373
77	0.985	1.74345	86.192736	150.2727
80	0.95	1.71	86.192736	147.3896
85	0.9	1.665	86.192736	143.5109
90	0.85	1.615	86.192736	139.2013
95	0.81	1.5795	86.192736	136.1414
102	0.75	1.515	86.192736	130.582

	Hana Mariam			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
38	1.33	1.8354	86.192736	158.1981
40	1.32	1.848	86.192736	159.2842
42	1.31	1.8602	86.192736	160.3357
46	1.3	1.898	86.192736	163.5938
51	1.26	1.9026	86.192736	163.9903
54	1.225	1.8865	86.192736	162.6026
57	1.2	1.884	86.192736	162.3871
60	1.15	1.84	86.192736	158.5946
64	1.1	1.804	86.192736	155.4917
68	1.025	1.722	86.192736	148.4239
72	0.95	1.634	86.192736	140.8389
76	0.89	1.5664	86.192736	135.0123

	Lamberet soil			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
33	1.3	1.729	86.192736	149.0272
38	1.28	1.7664	86.192736	152.2508
40	1.265	1.771	86.192736	152.6473
44	1.225	1.764	86.192736	152.044
46	1.2	1.752	86.192736	151.0097
48	1.17	1.7316	86.192736	149.2513
50	1.145	1.7175	86.192736	148.036
53	1.09	1.6677	86.192736	143.7436
55	1.05	1.6275	86.192736	140.2787
57	1	1.57	86.192736	135.3226
60	0.96	1.536	86.192736	132.392
64	0.88	1.4432	86.192736	124.3934

	Semit soil			
	Mass of soil to be remolded			
ω	ρ_d	ρ	Volume	Mass
40	1.19	1.666	86.192736	143.5971
45	1.175	1.70375	86.192736	146.8509
50	1.16	1.74	86.192736	149.9754
55	1.135	1.75925	86.192736	151.6346
60	1.095	1.752	86.192736	151.0097
63	1.06	1.7278	86.192736	148.9238
65	1.04	1.716	86.192736	147.9067
68	1	1.68	86.192736	144.8038
72	0.924	1.58928	86.192736	136.9844
75	0.88	1.54	86.192736	132.7368
80	0.81	1.458	86.192736	125.669
86	0.73	1.3578	86.192736	117.0325

Appendix D: Method of determining the laboratory Vane Shear Test

Liquidty Index		Addisu Gebeya soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
49	100	0.25	2.4525	1.2	1.2	0.864	0.288	3.619115	6.776519
45	87.5	1	9.81	1.2	1.2	0.864	0.288	3.619115	27.10608
42	78.125	1.9	18.639	1.2	1.2	0.864	0.288	3.619115	51.50154
39	68.75	2.5	24.525	1.2	1.2	0.864	0.288	3.619115	67.76519
36	59.375	3.16	30.9996	1.2	1.2	0.864	0.288	3.619115	85.6552
32	46.875	3.71	36.3951	1.2	1.2	0.864	0.288	3.619115	100.5635
29	37.5	4.29	42.0849	1.2	1.2	0.864	0.288	3.619115	116.2851
26	28.125	5.05	49.5405	1.2	1.2	0.864	0.288	3.619115	136.8857
24	21.875	5.75	56.4075	1.2	1.2	0.864	0.288	3.619115	155.8599
21	12.5	6.88	67.4928	1.2	1.2	0.864	0.288	3.619115	186.4898
19	6.25	7.2	70.632	1.2	1.2	0.864	0.288	3.619115	195.1637
17	0	7.45	73.0845	1.2	1.2	0.864	0.288	3.619115	201.9403

Liqidity Index		Akaki Black soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
95	100	0.1	0.981	1.2	1.2	0.864	0.288	3.619115	2.710608
90	91.07143	0.5	4.905	1.2	1.2	0.864	0.288	3.619115	13.55304
85	82.14286	1.21	11.8701	1.2	1.2	0.864	0.288	3.619115	32.79835
80	73.21429	2.75	26.9775	1.2	1.2	0.864	0.288	3.619115	74.54171
75	64.28571	2.92	28.6452	1.2	1.2	0.864	0.288	3.619115	79.14974
70	55.35714	3.45	33.8445	1.2	1.2	0.864	0.288	3.619115	93.51596
65	46.42857	3.71	36.3951	1.2	1.2	0.864	0.288	3.619115	100.5635
60	37.5	3.8	37.278	1.2	1.2	0.864	0.288	3.619115	103.0031
55	28.57143	3.95	38.7495	1.2	1.2	0.864	0.288	3.619115	107.069
50	19.64286	4.2	41.202	1.2	1.2	0.864	0.288	3.619115	113.8455
45	10.71429	4.7	46.107	1.2	1.2	0.864	0.288	3.619115	127.3986
39	0	5.21	51.1101	1.2	1.2	0.864	0.288	3.619115	141.2227

Liquidity Index		Akaki Gray soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
102	100	0.06	0.5886	1.2	1.2	0.864	0.288	3.619115	1.626365
95	83.72093	0.36	3.5316	1.2	1.2	0.864	0.288	3.619115	9.758187
90	72.09302	0.86	8.4366	1.2	1.2	0.864	0.288	3.619115	23.31123
85	60.46512	1.71	16.7751	1.2	1.2	0.864	0.288	3.619115	46.35139
80	48.83721	2.2	21.582	1.2	1.2	0.864	0.288	3.619115	59.63337
77	41.86047	2.96	29.0376	1.2	1.2	0.864	0.288	3.619115	80.23399
75	37.2093	3.25	31.8825	1.2	1.2	0.864	0.288	3.619115	88.09475
71	27.90698	3.35	32.8635	1.2	1.2	0.864	0.288	3.619115	90.80536
68	20.93023	3.4	33.354	1.2	1.2	0.864	0.288	3.619115	92.16066
65	13.95349	3.55	34.8255	1.2	1.2	0.864	0.288	3.619115	96.22657
62	6.976744	3.67	36.0027	1.2	1.2	0.864	0.288	3.619115	99.4793
59	0	3.75	36.7875	1.2	1.2	0.864	0.288	3.619115	101.6478

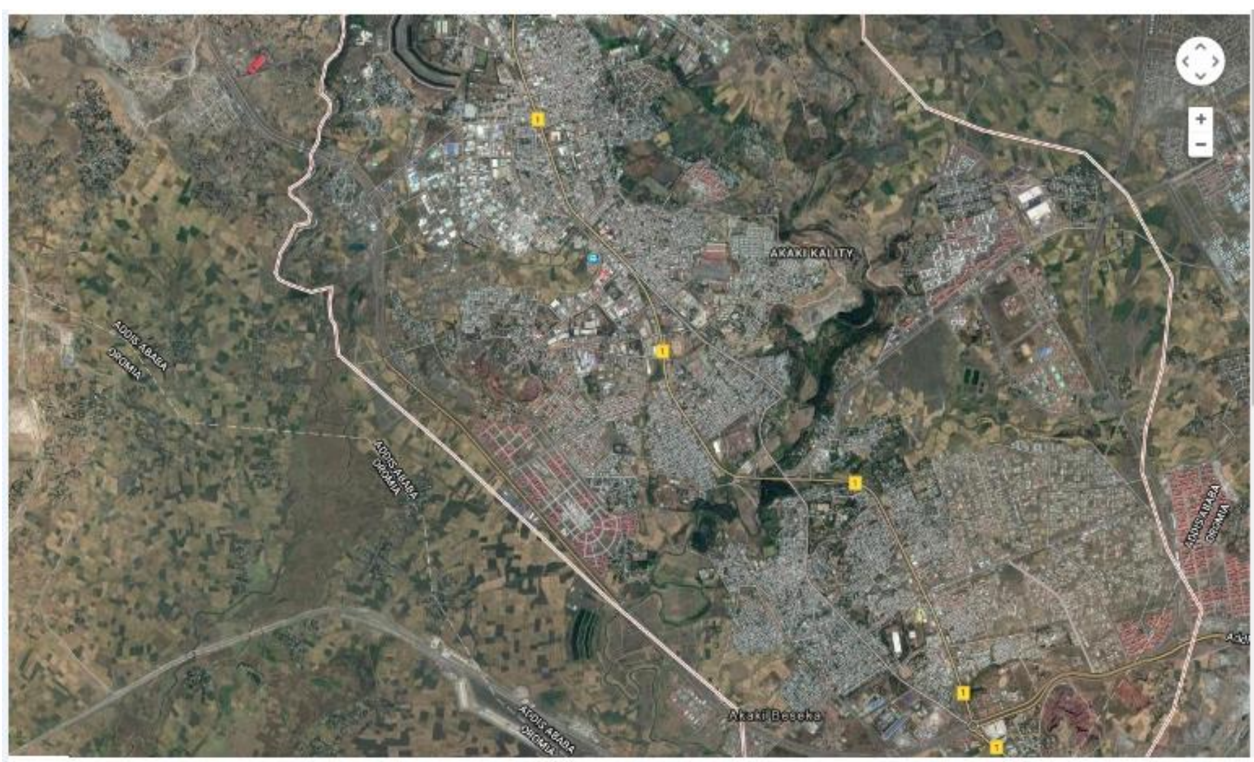
Liquidity Index		Hana Mariam soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
76	100	0.15	1.4715	1.2	1.2	0.864	0.288	3.619115	4.065911
72	89.47368	0.76	7.4556	1.2	1.2	0.864	0.288	3.619115	20.60062
68	78.94737	1.43	14.0283	1.2	1.2	0.864	0.288	3.619115	38.76169
64	68.42105	2.22	21.7782	1.2	1.2	0.864	0.288	3.619115	60.17549
60	57.89474	2.92	28.6452	1.2	1.2	0.864	0.288	3.619115	79.14974
57	50	3.35	32.8635	1.2	1.2	0.864	0.288	3.619115	90.80536
54	42.10526	3.5	34.335	1.2	1.2	0.864	0.288	3.619115	94.87127
51	34.21053	3.9	38.259	1.2	1.2	0.864	0.288	3.619115	105.7137
46	21.05263	4.45	43.6545	1.2	1.2	0.864	0.288	3.619115	120.622
42	10.52632	4.95	48.5595	1.2	1.2	0.864	0.288	3.619115	134.1751
40	5.263158	5.25	51.5025	1.2	1.2	0.864	0.288	3.619115	142.3069
38	0	5.7	55.917	1.2	1.2	0.864	0.288	3.619115	154.5046

Liquidity Index		Lamberet soil Undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
64	100	0.2	1.962	1.2	1.2	0.864	0.288	3.619115	5.421215
60	87.09677	1.1	10.791	1.2	1.2	0.864	0.288	3.619115	29.81668
57	77.41935	2.34	22.9554	1.2	1.2	0.864	0.288	3.619115	63.42822
55	70.96774	3.4	33.354	1.2	1.2	0.864	0.288	3.619115	92.16066
53	64.51613	3.67	36.0027	1.2	1.2	0.864	0.288	3.619115	99.4793
50	54.83871	4	39.24	1.2	1.2	0.864	0.288	3.619115	108.4243
48	48.3871	5.3	51.993	1.2	1.2	0.864	0.288	3.619115	143.6622
46	41.93548	5.64	55.3284	1.2	1.2	0.864	0.288	3.619115	152.8783
44	35.48387	5.81	56.9961	1.2	1.2	0.864	0.288	3.619115	157.4863
40	22.58065	6	58.86	1.2	1.2	0.864	0.288	3.619115	162.6365
38	16.12903	6.76	66.3156	1.2	1.2	0.864	0.288	3.619115	183.2371
33	0	7.25	71.1225	1.2	1.2	0.864	0.288	3.619115	196.5191

Liquidity Index		Semit soil undrained shear strength using Vane Shear Test							
ω	LI	T	Torque	D	H	A	B	A+B	Cu (kpa)
86	100	0.12	1.1772	1.2	1.2	0.864	0.288	3.619115	3.252729
80	86.95652	0.63	6.1803	1.2	1.2	0.864	0.288	3.619115	17.07683
75	76.08696	1.1	10.791	1.2	1.2	0.864	0.288	3.619115	29.81668
72	69.56522	2	19.62	1.2	1.2	0.864	0.288	3.619115	54.21215
68	60.86957	2.95	28.9395	1.2	1.2	0.864	0.288	3.619115	79.96292
65	54.34783	3.17	31.0977	1.2	1.2	0.864	0.288	3.619115	85.92626
63	50	3.25	31.8825	1.2	1.2	0.864	0.288	3.619115	88.09475
60	43.47826	3.4	33.354	1.2	1.2	0.864	0.288	3.619115	92.16066
55	32.6087	3.5	34.335	1.2	1.2	0.864	0.288	3.619115	94.87127
50	21.73913	3.7	36.297	1.2	1.2	0.864	0.288	3.619115	100.2925
45	10.86957	4.5	44.145	1.2	1.2	0.864	0.288	3.619115	121.9773
40	0	4.9	48.069	1.2	1.2	0.864	0.288	3.619115	132.8198

Appendix E: Aerial photographs

Addisu Gebeya



Akaki Kality

Hana Mariam

Lamberet


St. Gabriel Hospital
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CMC
GERJI
Summit EPS Factory
Summit
YERER
Flintstone Homes
Goro
Yerer Hospital
Summit condo
SEFERA

Semit