ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE

EMPIRICAL MODEL FOR ESTIMATING COMPRESSION INDEX FROM INDEX PROPERTIES OF FINE GRAINED SOILS

(CASE OF MODJO TOWN)

A Thesis Submitted to the Graduate School of Civil Engineering and Architecture of Adama Science and Technology University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Geotechnical Engineering)

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LIST OF SYMBOLS

A	Percent Passing Sieve No. 10 (2mm Sieve Size);
AASHTO	American Association of State Highway and Transportation Officials;
Adj. R2	Adjusted R-square (Adjusted Coefficient of Determination);
ASTM	American Society for Testing and Materials;
c	Cohesion of Soil;
C	Percentage of colloids smaller than 0.002m;
LL	Liquid Limit;
NMC	Moisture Content;
PI	Plasticity Index;
R	Pearson Product Moment Correlation Coefficient;
R2	Coefficient of Determination;
R-Value	Resistance Value;
α	Standard Significant Error;
γ	Bulk density of Soil;
$\alpha 1, \alpha 2, \alpha 3, \alpha n$	Coefficients of the Multiple Linear Regression Equation;
$\beta 1, \beta 2, \beta 3, \beta n$	Coefficients of the Single Linear Regression Equation;
σn	Normal Stress;
$\sigma 2$	Statistical Variance;
ϕ	Internal Friction Angle;
ε	Statistical Random Error;
Yd	Dry density;
ao,a1	Coefficients of the single linear regression equation;
bo, b1,b2,bn	Coefficients of the multiple linear regression equation;

LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials;
ASTM	American Society for Testing and Materials;
Cc	Compression index;
CH	High Plasticity Clay;
CL	Inorganic Silts of Low to Medium Plasticity;
EMC	Existing moisture content of soil after it is air dried;
FDD	Field Dry Density;
GC	Clayey Gravels;
GI	Group Index;
GM	Silty Gravels;
GP	Poorly Graded Gravels;
GW	Well Graded Gravel;
LI	Liquidity Index;
LL	Liquid Limit;
MDD	Maximum Dry Density;
ML	Inorganic Silts of Low Plasticity;
MH	Inorganic Silts of High Plasticity;
NCHRP	National Cooperative Highway Research Program of United States of America;
NMC	Natural moisture content of soil;
OMC	Optimum Moisture Content;
PI	Plasticity Index;
PL	Plasticity Limit;
SPSS	Statistical Package for Social Science Software;
SI	Suitability Index of de-Graft Johnson Equation;
USCS	Unified Soil Classification System;
CEC	Cation exchange capacity;
CES	Classification of expansive soils;
CH	Inorganic clay of high compressibility;

CL	Inorganic clay of low compressibility;
CL	Mean confidence level;
CM	Inorganic clay of medium compressibility;
EBC	Exchangeable base capacity;
EC	Electrical conductivity;
ESCA	Expansive Soil Classification Based on Atterberg Limits;
FS	Free swell;

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ABSTRACT

Compressibility of soil is an important engineering property and is related to settlement of the foundation or structures. Compression index is one of the soil parameter that is required for analysis of foundation settlement, however, the determination of compression index (Cc) is expensive and time consuming. In order to mitigate these complexities, equations have been developed by several researchers to predict compression index using index properties of soil which are easier to conduct in the laboratory.

This thesis illustrates prediction of compression index value from index properties of Mojo fine grained soils. After collecting 25 representative samples, laboratory index property tests consists of natural moisture content, Atterberg limit, free swell and one-dimensional consolidation tests were conducted. Accordingly the index properties results obtained were correlated with compression index using SPSS software.

From the single and multiple regression analysis it was found that the multiple regression analysis made strong correlation (C_c = -0.253+0.004LL+0.004PL+0.003NMC) with coefficient of determination of R-square (R^2) = 0.917 for a sample of 25.

After validating the developed correlation with control test result, the correlation between the compression index and the remaining parameters on the other hand yield relatively weaker relationship compared with that of the LL, PL & NMC. Result of the validity of the newly developed correlation with control test results shows that, correlation of compression index value with soil index properties is valid only to estimate settlement for preliminary design purposes.

KEYWORDS: *Compression index, Regression analysis, One-dimensional consolidation.*

1 INTRODUCTION

1.1 Background of the Study

Soil is a non-homogeneous porous material, in which the pores are filled with fluid and air. The pores within the soil structure render compressibility characteristics to the soil under the application of stress and change in moisture content. The vertical settlement or consolidation of soil under foundation supporting various structures and embankment is attributed to the compressibility nature of the soils underlying them [9].

Compressibility characteristics of fine-grained soils are often the most important parameters, in the design of foundation earth retaining structure and embankments, for settlement evaluation. In any construction civil engineers always encounter difficulties in obtaining representative compression index (C_c) value for preliminary design and rough assessment of the area.

Laboratory one dimensional consolidation test requires relatively large effort to conduct the test and it is time consuming. The alternate method could be to correlate C_c with simpler test results such as soil index properties. These tests are much economical and rapid than one dimensional consolidation test. This thesis gives an overview to obtain a correlation between C_c values with soil index properties that is suited for Mojo fine grained soil.

1.2 Objective of the Study

1.2.1 General Objective

The main objective of this thesis is to give more attention to find correlation between Compression indexes with soil index properties for representative Fine grained soil samples recovered from different localities of Mojo town.

1.2.2 Specific Objectives

- ✓ To establish relevant relationship between compression index values and index properties of Mojo Fine grained soils.
- ✓ Examining validity of the correlations, and to draw appropriate conclusions on the relationships of each empirical equation.

1.3 Scope of the Study

The thesis is conducted only on 25 samples size at 3m depth. 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 equation and multiple linear regression equations with the aid of Excel software. Furthermore, the scope of the developed correlation is limited to the test procedures followed in the research work.

1.4 Description of the Study Area

Modjo or Mojo is a town in central Ethiopia at 73km from Addis Ababa, named after the nearby Modjo River. Modjo is located in the East Shewa zone of the Oromia region and has a latitude and longitude of 8°39′N 39°5′E with an elevation between 1788 and 1825 meters above sea level. It is the administrative center of Lome woreda.It is not only accessible by road but has been the location of train station of the Addis Ababa-Djibouti railway since the line was extended from Dire Dawa to Akaki in 1915. Based on figures from the Central Statistical Agency in 2005, Modjo has an estimated total population 2015 of 39,316 of which 19,278 were males and 20,038 were females.

From geographical evidences it is located in the great Rift Valley zone of Ethiopia which is prone to an earthquake activity and classified as seismic zone four [37]. Moreover, it is the center of distribution for the south eastern and central parts of Ethiopia, which exposed for further civil engineering works like high-rise buildings, residential complexes, Factories, large industrial and commercial centers etc. Modjo town hosts Modjo dry port, Tanneries and Textile Factories and others.

In such high-risk area, it is necessary to have knowledge of the Properties of soils subjected to loads. In light of this, the output of the proposed correlation will provide preliminary background information on the value of C_c, for a localized soil, from soil index properties with a benefit of time saving and without incurring any additional cost for carrying out laboratory Consolidation test.

1.5 Organization of the Study

The thesis is organized and presented in six chapters. After introducing the main and detail objectives in chapter one, and in chapter two literature review regarding the soil compressibility and previously established correlation equations. In Chapter three, discussions on sample collection and on test results were made. In Chapter four, correlation and regression analyses were conducted and Chapter five focuses on validating and evaluating the obtained correlation. 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 follows:

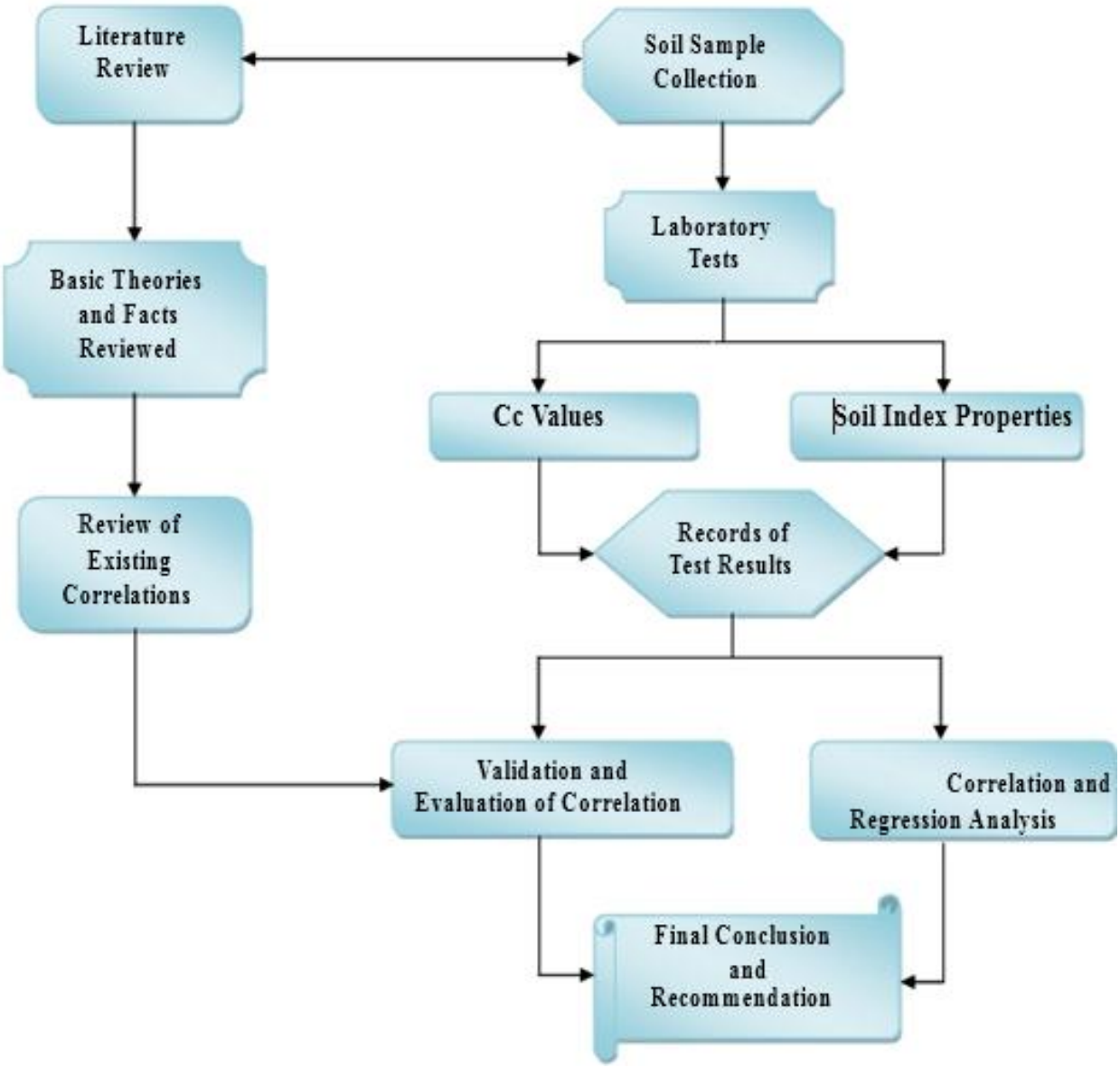


Figure 1.1 Design flow chart for research methodology

2. LITERATURE REVIEW

2.1 General

Soil, a natural property of earth which is formed from small pieces of mineral particles and it may contain water, air and organic materials. It supports every construction which is designed by human such as buildings, retaining walls and embankments. There are many types of soil.The main soil groups are granular soil, fine-grained soil and organic soil. the behaviour of soil is an important element which always concerned in civil engineering[27].

The properties such as the plasticity, compressibility, or strength of soil always affect the design in construction. Geotechnical properties and behaviours of soil can be determined by several tests such as odometer test and Atterberg limits test [11].

Expansive soil, which is recognized by its considerable volume change upon exposure to moisture variation, has caused a number of problems in most structures constructed in Mojo town This problems are either due to mis understanding of the behaviour of the soil or lack of information on the engineering properties of the soil.

The six major natural hazards are earthquakes, landslides, expansive soils, hurricane, tornado and flood. A study points out that expansive soils tie with hurricane wind/storm surge for second place among America's most destructive natural hazards in terms of dollar losses to buildings. According to the study, it was projected that by the year 2000, losses due to expansive soil would exceed 4.5 billion dollars annually [7].

Expansive soils owe their expansive character mainly to the constituent clay mineral. The most important clay mineral, which is the cause for nature is montmorillonite has an octahedral sheet sandwiched between two silica sheets. When this mineral is exposed to moisture, water is absorbed between interlayering lattice structures and exert an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with expansive soils.Even though, expansive soils are found in different parts of the world, the engineering properties of the soil are not similar. The behaviour of expansive soils varies from place to place depending upon the type of parent material, climate and topography[27].Therefore, due to the fact that the engineering properties of expansive soil of Ethiopia are different from the same soil in other locality, researches on correlation between compression index and index properties of expansive soils of Mojo town has been done.

2.2 Mineralogical structure

Lattice structure of clays can have fundamental molecular structure of alumina octahedron and the silica tetrahedron as basic units.The alumina octahedron consists of an aluminium atom surrounded octahedrally by six oxygen ions. When each oxygen atom is shared by two tetrahedra, a plate-shaped layer is formed. The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions.The tetrahedrons are arranged so that tips of all of them point in the same direction, and the bases of all tetrahedrons are in the same plane[32].

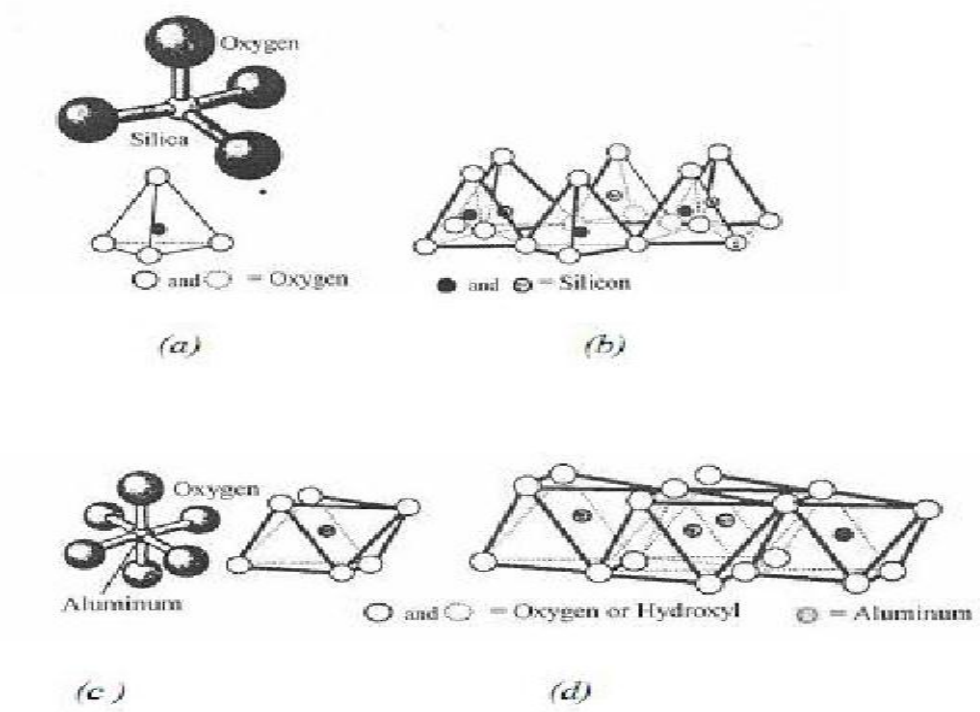


Figure 2. 1 (a) A slica tetrahedron , (b) Slica Sheets , (C) An Aluminium Octahedron, and (d) Aluminium Sheets[24].

The above mineralogical structures lead to the formation of different types of clay mineral, namely kaolinite, halloysite, montmorillonite, illite, etc (Table 2-1). But the most common minerals found in clay soils are grouped as kaolinite, illite and montmorillonite [22].

2.2.1 Kaolinite

Kaolinite has a structure that consists of one silica sheet and one alumna sheet bonded together in to a layer about 0.72mm thick and stacked repeatedly. The layers are held together by

hydrogen bonds. Kaolinite has a few or no exchangeable cation, and the interlayer bonds are relatively strong preventing any hydration between layers and allowing many layers to build up. Kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently Kaolinite shows little swelling on wetting. Kaolinites are found in soils that have undergone considerable weathering in warm, moist climates. They have low liquid limit and a low activity [12].

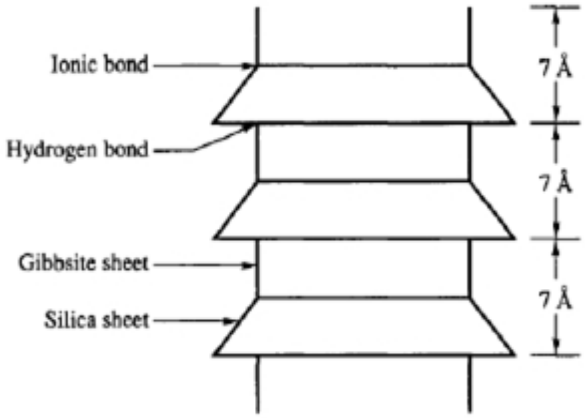


Figure 2.2 Structure of kaolinite layer [3]

2.2.2 Montmorillonite

Montmorillonites are made up of sheet like unit comprising an alumna octahedral sheet between two silica tetrahedral sheets. As the electrons rotate around the nucleus of an atom there will be times when there are more electrons on one side of the atom than the other, giving rise to a weak instantaneous dipole. Weak Vander Waals forces hold layers together and the bonding of these sheets is rather weak, resulting in a rather unstable mineral, especially when wet. In fact,montmorillonite display a significant affinity for water, with subsequent swelling and expansion. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements [12].

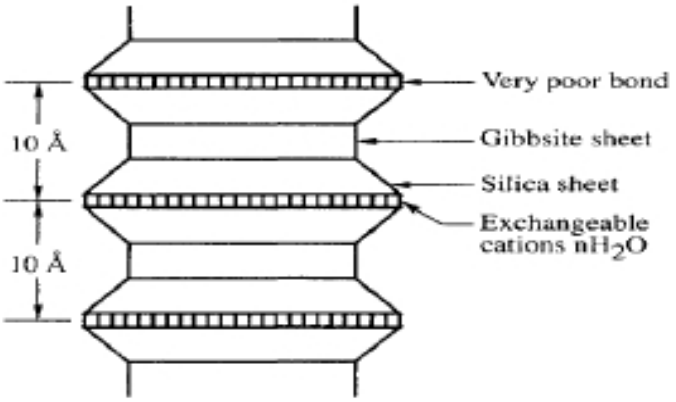


Figure 2.3 Structure of montmorillonite layer [3]

2.2.3 Illite

The illites are some what similar to montmorillonites in the structural units, but are different in their chemical composition. In illite, the layers are separated by potassium ion, where as in montmorillonite the layers are separated by loosely held water and exchangeable metallic ions. Unlike montmorillonite particles, which are extremely small and have a great affinity for water, the illite particles will normally aggregate and there by develop less affinity for water than montmorillonites. Correspondingly, their expansion properties are less. The cation exchange capacity of illite is less than that of montmorillonite[12].

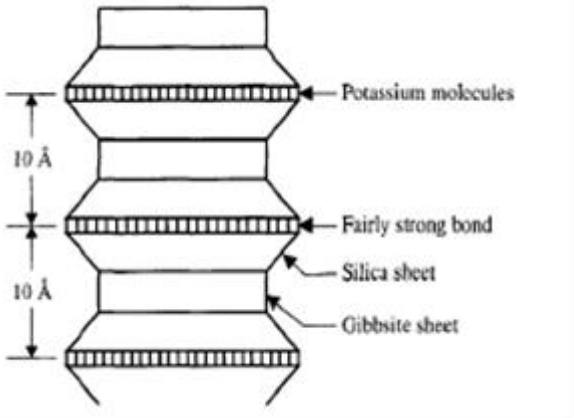


Figure 2.4 Structure of illite layer [3]

2.3 Identification and Classification of Expansive Soils

2.3.1 Field Identification

It is evident that expansive soil can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include [11].

- ✓ Usually have a color of black or gray
- ✓ Wide or deep shrinkage cracks.
- ✓ High dry strength and low wet strength.
- ✓ Stickiness and low trafficability when wet.
- ✓ Cut surfaces have a shiny appearance
- ✓ Appearance of cracks in nearby structures.
- ✓ Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature

2.3.2 Laboratory Identification

There are three different methods of identifying potential expansive soil in laboratory.
These are:

- ✓ Mineralogical methods of identification
- ✓ Direct measurement
- ✓ Indirect methods of identification

2.3.2.1 Mineralogical Methods of Identification

Type of clay mineral is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques, some of the common types of mineralogical tests are:

- ✓ X-ray Diffraction
- ✓ Differential Thermal Analysis (DTA)
- ✓ Electron Microscopy

2.3.2.2 Direct Measurement

These methods offer the most useful data by direct measurement and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of

samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer [11].

2.3.2.3 Indirect measurement

These methods include the index property, Potential Volume Change (require special PVC apparatus) and activity methods, which are valuable tools in evaluating the swelling property. However, none of the indirect methods should be used independently.

2.4 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 [7]. 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 [8].

2.4.1 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 [4], [[5].

2.4.2 Atterberg Limits

Based on their mode of formation and mineralogical composition different soils respond differently for the same moisture content. Albert Atterberg, [23] 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].

Liquid Limit: The liquid limit (LL) is the water content, expressed in percent, at which the soil changes from a liquid state to a plastic. The conventional liquid limit test is carried out in accordance of test procedures of AASHTO T 89 or ASTM D 4318 [4], [5].

Table 2.1 Soil expansiveness predications by liquid limit [32].

Degree of expansion	Liquid limit, WL [%]	
	Chen	IS 1498
Low	<30	20-35
Medium	30-40	30-50
High	40-60	50-70
Very High	>90	70-90

*Plastic Limit***:** The plastic limit (PL) is the water content, expressed in percentage, below which the soil stops behaving as a plastic material and it begin to crumble when rolled into a thread of soil of 3.0mm diameter. The conventional plastic limit test is carried out as per the procedure of AASHTO T 90 or ASTM D 4318 [4], [5].

Table 2.2 Soil expansiveness predication by plastic index [14].

Plasticity Index %	Description
0	Non-plastic
1-5	Slightly Plastic
5-10	Low Plastic
10-20	Medium Plastic
20-40	High Plastic
>40	Very high plastic

*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 [4], [5].Plasticity Index, PI is a measure of the plasticity of soil. The plasticity index is the size of the range of water content where the soil exhibits plastic properties. The PI is the difference between the liquid limit and plastic limit.

Table2.3 Classification of soil by plasticity index [14].

Plasticity Index %	Description
0	Non-plastic
1-5	Slightly Plastic
5-10	Low Plastic
10-20	Medium Plastic
20-40	High Plastic
>40	Very high plastic

Liquidity Index, LI is the numerical relationship of the moisture content to the liquid limit and plastic limit [18].

Table 2.4 Description of the soil strength based on liquidity index [29]

Value LI	Description of soil strength
LI<0	Semisolid state-high strength, brittle(sudden) fracture is expected
0<LI<1	Plastic state-intermediate strength, soil deforms like aplastic material
LI>1	Liquid state-low strength, soil deforms like a viscous fluid

Activity of clay: Is soil plasticity index, PI divided by the percentage of clay fraction

On the basis of activity, clay can be classified into the four groups shown in table [2, 5].

Table2.5 Classification of soil based on the activity of clay [9].

Description	Activity
Inactive	<0.75
Normal	0.75-1.25
Active clay	1.25-2
High active clay	>2

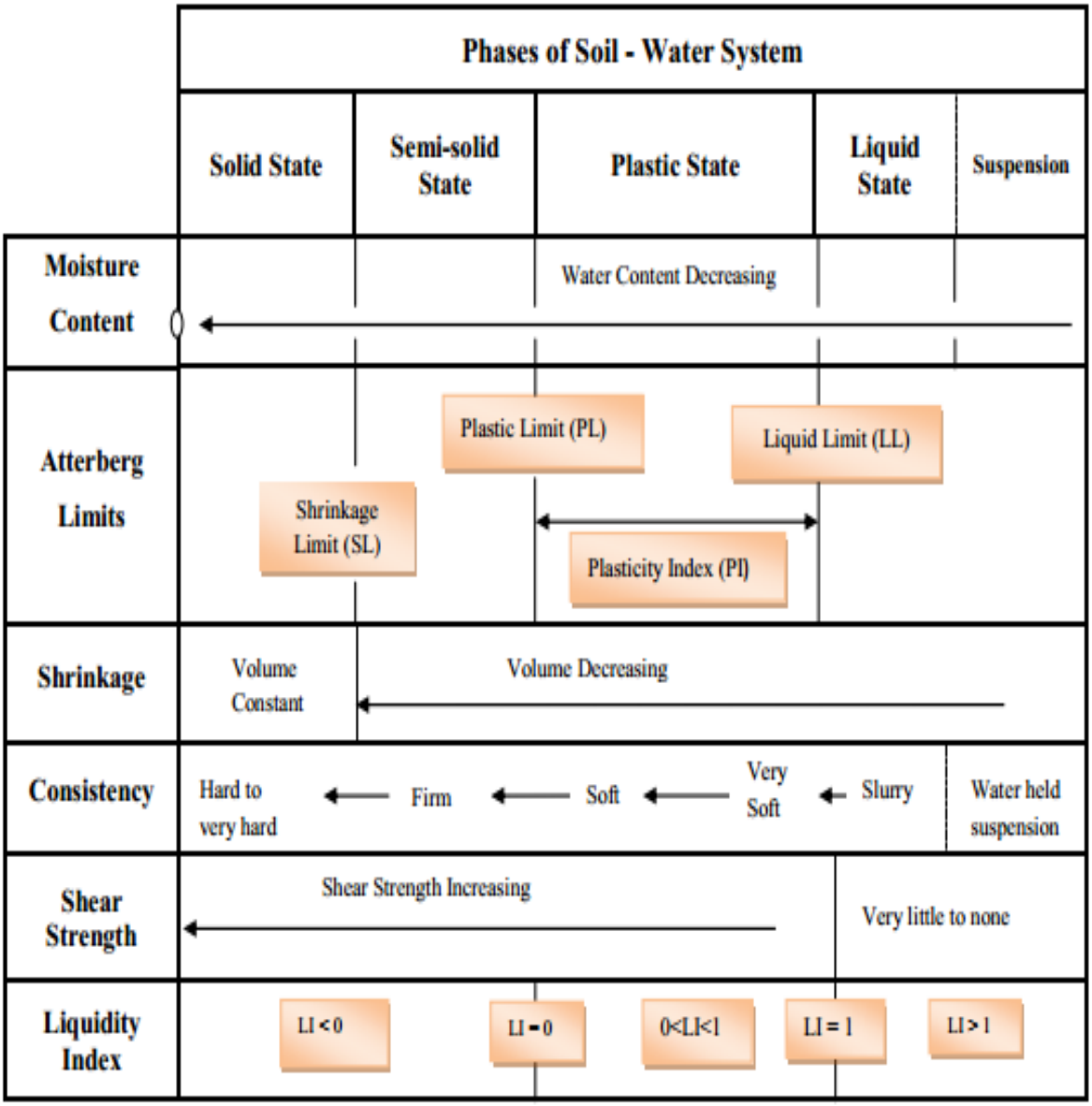


Figure 2.5 Descriptions of Phases of Soil-Water System [14].

Free Swell: test may be considered as a measurement of volume change in clay upon Saturation

$$\text{Free swell [\%]} = \frac{\text{Final volume - Initial volume}}{\text{Initial volume}} X100 \text{----------[2.1]}$$

Table 2.6 Degree of expansiveness and free swell [10]

Degree of expansiveness	DFS%
Low	<20
Moderate	20-35
High	35-50
Very high	>50

Table 2.7 Index property of some city of Ethiopia expansive clay by different researchers

Item	Addis Ababa [24]	Bahir Dar [19]	Mekelle [23]	Assela [37]
Gravel,%				
Sand,%	2-16	0.7-1.53	3.79-13.52	1.3-11.5
Silt,%	9-46.4	17.6-24.90	37.5-69.46	15-46
Clay,%	30-70	75-78.6	14.3-57.6	48-87
Liquid limit,%	63-108	60.5-64.65	52.08-89.66	72-129.2
plastic limit ,%	45-76.8	25.28-31.38	15.45-70.58	25.6-76.9
shrinkage,%	4-9	15.2-17.8	4.5-17.22	9-22.2
Specific Gravity,%	2.77-2.88	2.75-2.83		2.6-2.74
Free swell,%	51.80-118	80-120	12-127	47-147.5

2.5 Classification of Expansive Soils

Once a soil is identified as expansive, it has to be classified according to the property it exhibits. The different classification systems are categorized into two:

General classification systems: - which have evolved over many years and are based largely on correlation with actual performance

Classification specific to expansive soils: -Theses systems are based on in direct and direct prediction of swell potential, as well as combinations, to arrive at a rating. Shrink and swell in expansive soil can be induced by different factors. Generally, these factors are categorized into three groups namely the soil properties, the environment factors and the stress condition. These factors are summarized below [9]:

2.5.1 Soil Properties

The swell potential of an expansive soil may be affected by either the soil properties influencing the nature of the internal force field, the environmental factors those may change the internal force system or the state of stress present on the soil. Some physical factor such is initial water content, initilal dry density, amount and type of compaction also influence the swell potential and swell parameters of soils[9].

2.5.2 Factors influencing shrink- swelling characteristics of expansive soils

- ✓ Clay Mineralogy
- ✓ Soil Water Chemistry
- ✓ Soil Suction
- ✓ Plasticity
- ✓ Soil Structure and Fabric
- ✓ Dry Density

2.6 Theory of One-Dimensional Consolidation

The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi [12]. The underlying assumptions in the derivation of the mathematical equation are the following:

(i) The soil is homogeneous and isotropic

(ii) The soil is fully saturated

(iii) The soil particles and the water in the voids are incompressible. The consolidation occurs due to expulsion of water from the voids.

(iv) Darcy's law is valid throughout the consolidation process

(v) Soil is laterally confined and the consolidation takes place only in the axial direction. Drainage of water also occurs only in the vertical direction [21].

The assumptions made by Terzaghi are not fully satisfied in actual field conditions. The results obtained from the use of the theory to practical problem are approximate.

However, considering complexity of the problem, the theory gives reasonably accurate estimate of the time rate of settlement of a structure built on the soil. The standard one- dimensional consolidation test is usually carried out on saturated specimen using an Odometer. The schematic diagram for conducting one consolidation test is shown in figure below.

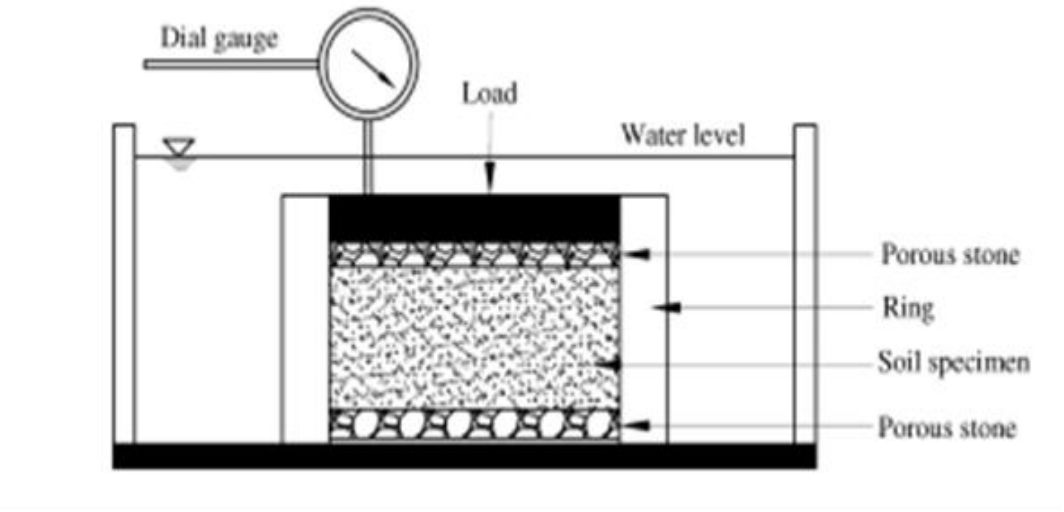


Figure 2.6 Schematic diagram of oedometer [9].

2.6.1 Oedometer Methods for Measuring Compression Properties

The most common measurement of stress strain relationship involves the use of the one dimensional

consolidation apparatus or oedometer. A compression curve obtained from an oedometer test for a saturated soil is a relationship between the amount of volume change in the soil and the normal stress[28].

Any structure built on the ground cause pressures on the underlying soil layers. The soil layers are unable to spread laterally as surrounding soil strata confines them. Hence there must be adjustment to the new pressure by vertical deformation. The compression of the soil mass leads to decrease the volume of soil mass, which result in the settlement of the structure, built on the mass. The vertical compression of soil mass under increased pressure is thus made up of the following components.

(i) Deformation of soil grain

(ii) Compression of water and air with in the void

(iii) An escape of water and air from the void

Generally, the volume change in a soil deposit can be divided into three stages [29].

i. Initial consolidation or compression

When a load is applied to a partially saturated soil decrease in volume occurs due to explusion of water and compression of air in the void. A small decrease in volume also occurs due compression of solid particles .The reduction in volume of thesoil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particle.

ii. Primary consolidation

After initial consolidation, further reduction in volume occurs due to expulsion of water from void. When a saturated soil is subjected to a pressure, initially all the applied Pressure is taken up by water as excess, as water is almost in compressible as a hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the

permeability of the soil and therefore, time dependant. The reduction in volume is called primary consolidation.

In fine grained soils, the primary consolidation occurs over long time.On other hand, in coarse grained soils, the primary consolidation occurs quickly due to high permeability. As water escapes from the soil, the pressure is gradually transferred from the water in the void to solid particles[12]. The figure below shows the model for compression of soil due to expulsion of water and air when pressure is applied on the soil.

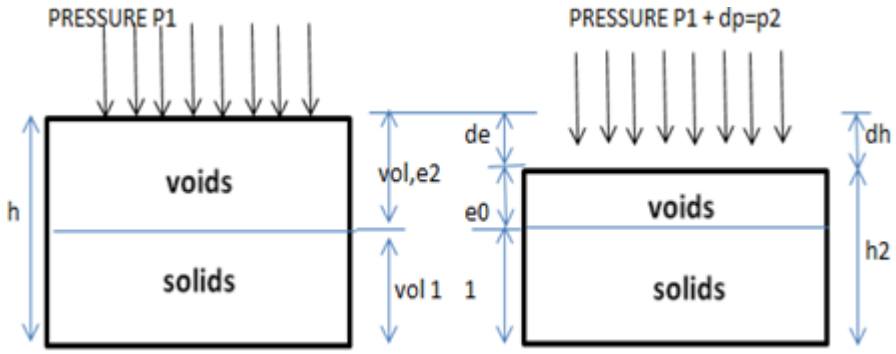


Figure 2.7 Compression of a model sample[19].

Where;

P1 = Initial Pressure

P2 =Total Pressure after Loading

e= Void Ratio

dp =Pressure Increment

iii. Secondary consolidation

The reduction in volume continues at very slow rate even after excess pore water pressure developed by applied pressure is fully dissipated and the primary consolidation is completed. This additional reduction in volume is called secondary consolidation. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the absorbed water to the new system.

Factors affecting the consolidation properties of the fine grained soils

The consolidation behaviour of clay soil in its natural state is highly dependent on stress history and permeability. The effects of those factors are explained below.

2.6.1.1 Stress history

The maximum stress towhich the soil is subjected in the past influence the consolidation properties of the soil in its insitu condition. In remolded soils, because it has lost its structure characteristics as compared with its normal condition, it is inferred that a remolded soil is unstuible for evaluating its stress history [30].As to the stress history, the insitu soil can be grouped in two categories.

(i) Normal consolidation clay soil

A normaly consolidated soil is one whose present effective overburden pressure on the in situ prototype soil deposit is the maximum pressure to which the soil has ever been subjected at any time in the past history. In other words, the normal consolidated soil is one whose preconsolidation pressure is equal to its present effective overburden pressure[25].

(i) Over consolidated clay soil

Over consolidated clay is one which has been completely consolidated under a large overburden pressure in the past that is larger than the present overburden pressure. The response of over consolidated clay soils applied a load is such that at earlyloading the soil shows relatively the soil shows relatively small decrease of void ratio with load up to maximum effective stress to which the soil subject in the past. If the effective stress on the soil sample is increased further, the decrease of the void ratio with stress level will be larger[27].

Permeability: The expulsion of water from the void of saturated clay soil by externally applied load in the consolidation process and the change in volume associated with such a process are essentially a hydraulic problem. Specifically, it is a problem of permeability of the soil .Therefore, the rate of consolidation depends on the permeability of the soil .The permeability of the soil by itself is Function of the soil type, size and shape of soil particles (round, angular, or flaky), and thus up on the size and geometry of void. Also resistance is a function of temperature of water (viscosity and surface tension effect) [30].

2.7 Compression Index

The compression index of fine grained soils is one of the most widely used enginnering properties of such types of soils. The compression index is usually obtained by conducting one-dimensional consolidation tests in the laboratory and same is used to determine the primary settlement of fine grained soils.The compression index C_c is the slope of the void ratio versus pressure curve which is plotted on logarithmic scale as indicated in Figure2.8 below.

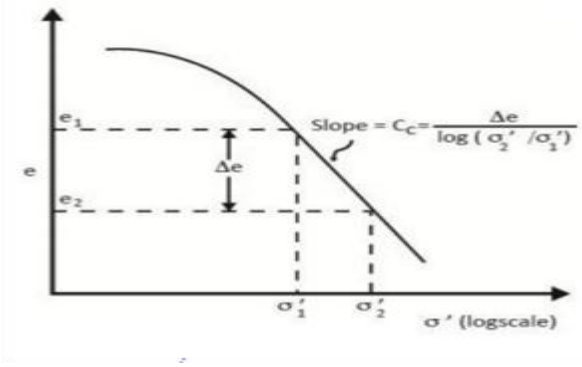


Figure 2.8 Compression index [14]

$$Cc = \frac{e_o - e}{\log[\frac{p}{p_o}]} \text{----------[2.2]}$$

Where Po, is the pressure corresponding to the initial void ratio, eo

P, is the pressure corresponding to the void ratio, e

2.7.1 Application of Consolidation Test

As it is discussed clearly in the above section the purpose of undertaking one- dimensional consolidation is to study the compressibility behaviour of soils. The parameters determined through this laboratory test will be used to estimate the settlement of structure founded on consolidating soil, like clay soils. In other words ,the data obtained from the consolidation test are used to determine the magnitude and rate of differential and total settlement of ,a structure or earth fill [36].

2.7.1.1 Settlement

In order to evaluate the suitability of a foundation or earth structure , it is necessary to design civil structure against both bearing capacity failure and excessive settlement. From the conslidometer, the compressibility of the soil for one-dimensional consolidation is determined. Settlement due to one-dimensional compression results from decrease in the volume of voids. The data obtained from the test are used to determine the relationship between the effective stresses and void ratio or strains [40].

The total settlement is the sum of immediate settlement, primary consolidation settlement and secondary consolidation settlement [23].

Immediate settlement; is predominant in all coarse grained and dry or partially saturated fine grained soils. It is due to the elastic deformationof soil with out any change in the soil water content.

Primary consolidation settlement ; is time dependent. This is because it takes place due to pore water dissipation. It is a common behavior of soil with low coefficient of permeability. Secondary compression settlement ; is a form of soil creep. after complete dissipation of excess pore water pressure- some deformation of the sample is caused by plastic readjustment [20].

$$\rho_{max} = s_i + s_c + s_s \quad \text{----------[2.3]}$$

Where p_{max} = maximum settlement over the life of the structure

s_i = immediate settlement

s_c = primary consolidation settlement

s_s = secondary compression settlement

$$s_t = \frac{c_c}{1+e_o} H \log \frac{p_{o+}\Delta p}{p_o} \quad \text{----------[2.4]}$$

The thickness of the clay stratum, H

The initial void ratio, e_0

The consolidation pressure P_o

Change in Pressure Δp

Compression Index, C_c

2.8 Previously Established Compression Index Correlation Equations

2.8.1 Correlation

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 equipment's and limited time [16]. Many attempts have been made to obtain the correlation equations for fine-grained soils that relate Cc with index properties the development of empirical equations for estimating compression index considered valuable exercise since it essentially serve as a substitute for conducting consolidation tests. It is often desirable to obtain approximate values using other soil indices which are more easily determined than the commonly known standard consolidation test. Over the past fifty years, numerous efforts have been exerted by different researcher in order to develop empirical equations that relate compression index to soil index properties. To this effect, currently, there are a number of correlation methods that exist to estimate the compression index of a soil, based on soil; classification, other index test values, and/or physical property measurements of the soil. The majority of available empirical formulas relate compression index to liquid limit, natural water content, plasticity index, shrinkage limit, initial void ratio and etc. Several authors have listed a number of such empirical equations in terms of soil index properties as shown in the table below.

Table 2.8 Existing Compression index equations ([9], [10], [2], and [7])

Equation	Applicability	Reference
$Cc = 0.006(wL - 38)$	Remoulded clays	Solomon D,2012
Cc=0.004W+0.006FSI+0.001LL-0.353	Normally consolidated clays	Abadi L,2014
$Cc = 0.007(wL - 10)$	Remoulded clays	Skempton 1944
$Cc = 0.0046(wL - 9)$	Brazilian clays	Cozzolino 1961
$Cc = 0.009(wL - 10)$	Normally consolidated clays	Terzaghi and Peck 1 967
$Cc = 0.006(wL - 9)$	All clays with $wL < 100\%$	Azzouz et al. 1976
$Cc10 = 0.009(wL - 8)$	Osaka Bay clay	Tsuchida 1991
$Cc = 0.01(wn - 5)$	All clays	Azzouz et al. 1976
$Cc = 0.01wn$	All clays	Koppula 1981
$Cc = 0.01(wn - 7.549)$	All clays	Herrero 1983
$Cc = 0.0115wn$	Organic silt and clays	Bowles 1989
$Cc = 1.15(e - eo)$	All clays	Nishida 1956
$Cc = 0.29(eo - 0.27)$	Inorganic soils	Hough 1957
$Cc = 0.35(eo - 0.5)$	Organic soils	Hough 1957
$Cc = 0.246 + 0.43(eo - 0.25)$	Motley clays from Sao Paulo,Brazil	Cozzolino 1961
$Cc = 0.75(eo - 0.50)$	Soils with low plasticity	Sowers 1970
$Cc = 0.156eo + 0.17$	All clays	Bowles 1989
$Cc = 0.5IPGs$	All remoulded, normally consolidated clays-	Wroth and Wood
$Cc = 0.329[0.027(w - wp) + 0.0133Ip(1.192 + ACT-1)]$	All remoulded, normally consolidated clays-	Carrier 1985
$Cc = 0.2237eL$	All remoulded, normally consolidated clays-	Nagaraj and Srinivas a Murthy 1983
$Cc = 0.2343eL$	All remoulded, normally consolidated clays-	Nagaraj and Srinivas a Murthy

2.8.2 Relationship between Compression Index and Atterberg Limits

The index tests namely the atteberg limits, form the most important inferential soil test with very wide universal acceptance since these tests are relatively simple to perform and provide a basis for explaining most engineering properties of soils in geotechnical practice

According to Gopal Ranjan and A.S.Rao [13] the natural structure of a clay soil has a marked influence on its engineering properties. However, except for shrinkage limit, which can be determined by both undisturbed and disturbed soil conditions. Atterberg limit are found to correlate well with the engineering properties of soils because both engineering properties and Atterberg limit are influenced by the same set of factors such as the clay minerals, the ions in pore water, the stress history of the soil deposit, etc.

Table 2.9 General Relationship of Atterberg Limit and engineering properties [13].

Characteristics	Comparing soil properties at equal LL with Pl increasing	Comparing soil properties at equal PI with LL increasing
Strength	Increase	Decrease
Toughness plastic limit	Increase	Decrease
Compressibility	About the same	Increase
Permeability	Decrease	Increase
Rate of Volume change	Decrease	Increase

A.I.Johnson and R.P.Moston [3] in their study of the subsiding sediments in the California, have made a detailed investigation as to the validity of the equations developed by Terzaghi and Peck [9], which was a continuation of work begun by Skempton [9] and stated that the compression index for clays in a remolded state (Cc) increase consistently with increasing liquid limit (LL). Using data from different parts of the world and representing both ordinary and extra-sensitive clays, Terzaghi and Peck [9] state that the data on compression indices and liquid limits for these clays plot on a graph within=+/-30 percent of a line representing the equation [2].

$$c_c = 0.007 \text{ (LL-10 Percent)} \text{ ----------} [2.5]$$

They state further that for an ordinary clay of medium or low sensitivity tested in the undisturbed state, the value of Cc, corresponding to field consolidation is approximately equal to 1.3 Cc thus,

$$c_c = 0.009 (LL-10 \text{ Percent}) \text{ ----------[2.6]}$$

Hence, these authors conclude that for normally loaded clays with low or moderate sensitivity, the compression index Cc, can be estimated approximately from knowledge of the liquid limit and use of Equation (2.6), However, Terzaghi and Peck [9] do caution that this approximate method of compression may furnish merely a lower limiting values for the compression index of extra sensitive clay. Later papers by Nishida [5], and Roberts and Darragh[11], showed exception to the compression index liquid limit relationships described by Terzaghi and Peck [9] and indicated that there was a wide scattering of data.

In conclusion, A.I. Jonhnson and R.P.Moston have to note that their study show that the equations presented by Terzaghi and Peck [9] (Equation 2.5 and 2.6) do not apply to the relationship between compression index and liquid limit for sedimentary deposits tested from subsidence areas in the San Joaquin or Santa Clara Valleys. Furthermore, the data show that no single equation applies to the relationship for all areas studied, with the following equations being obtained for the two valleys [2].

$$c_c = 0.014 (LL-22) \text{ ----------[2.7]}$$

Some researchers reported that though there are a number of empirical formulas developed to correlate the compression index properties, the compressibility behavior has not been considered together in any single study to examine which of the index property of the soil correlate better with the compressibility, behavior, especially with the same set of test results. Despite this fact, has been shown by different authors that correlating the compression index with liquid limit has received a higher preference and many empirical formulas derived so far tried to make use of the same.

On the other hand, Amir Al-Khafajii [36] has made a thorough investigation on the available empirical correlations in the light of comprehending the applicability of the existing formulae. In accordance with the paper presented by Amir Al-khafaji, [36] almost all empirical equations,

developed so far, do not seem to have theoretical basis to justify their development. Furthermore the author reported that the developers of these empirical equations do not explicitly specify whether their equations are for undisturbed samples or remolded samples except their mentioning that the equation are for all clays soils. In this respect the author is of the view that there is a need for the development of objectives and rational procedures which could provide a magnitude measure of validity and applicability regression equation [34] In view of the points outlined above, the investigator has made an attempt to identify and group the available empirical equation in to equations developed for undisturbed samples and remolded ones. However, due to the lack of clear distinction between formulas developed for two samples types by different authors, as mentioned in the preceding paragraph, it was not possible to categorize the equation in to remolded and undisturbed [36].

3. RESEARCH METHODOLGY AND LABORATORY TEST RESULTS

3.1 Methodology

Primarily,in order to address the intended objectives of the study, basic theories and descriptions of compression index in general and in relation to soil index property of soil is reviewed. Subsequently, previous works of different researchers with regard to prediction of C_C value from basic soil index properties were assessed.

In order to have satisfactory data for utilizing the correlations, laboratory tests were conducted by the researcher on samples collected from different localities of Mojo, so as to get records of test results of C_c values along with the associated soil indices particularly the grain size analysis, Atterberg limits, Void ratio relationships.

Then, discussions on sample collection and summary of laboratory test results were presented. Statistical regression analyses of test results were carried out and correlations were developed and also analyzed to fit the test results. Under the discussions of the obtained results the suitability of the developed correlations were examined. Finally, a generalized conclusion and recommendation were made.

3.2 Collection of Soil Samples

In order to have sufficient and reliable data for the target analysis, laboratory tests conducted on soil samples obtained from different location of Mojo town in addition to this an attempt has been made to collect data from consulting offices that are involving in soil practice but no relevant test data could be obtained, Nevertheless, three consolidation test data for fine grained soil of Mojo with the corresponding index properties have been obtained from past thesis by Yohannis H [37]. The data obtained from the above referred thesis works are used as validity test pit.

The location of collected soil samples site is shown with the aid of map in (Figure 3.1).

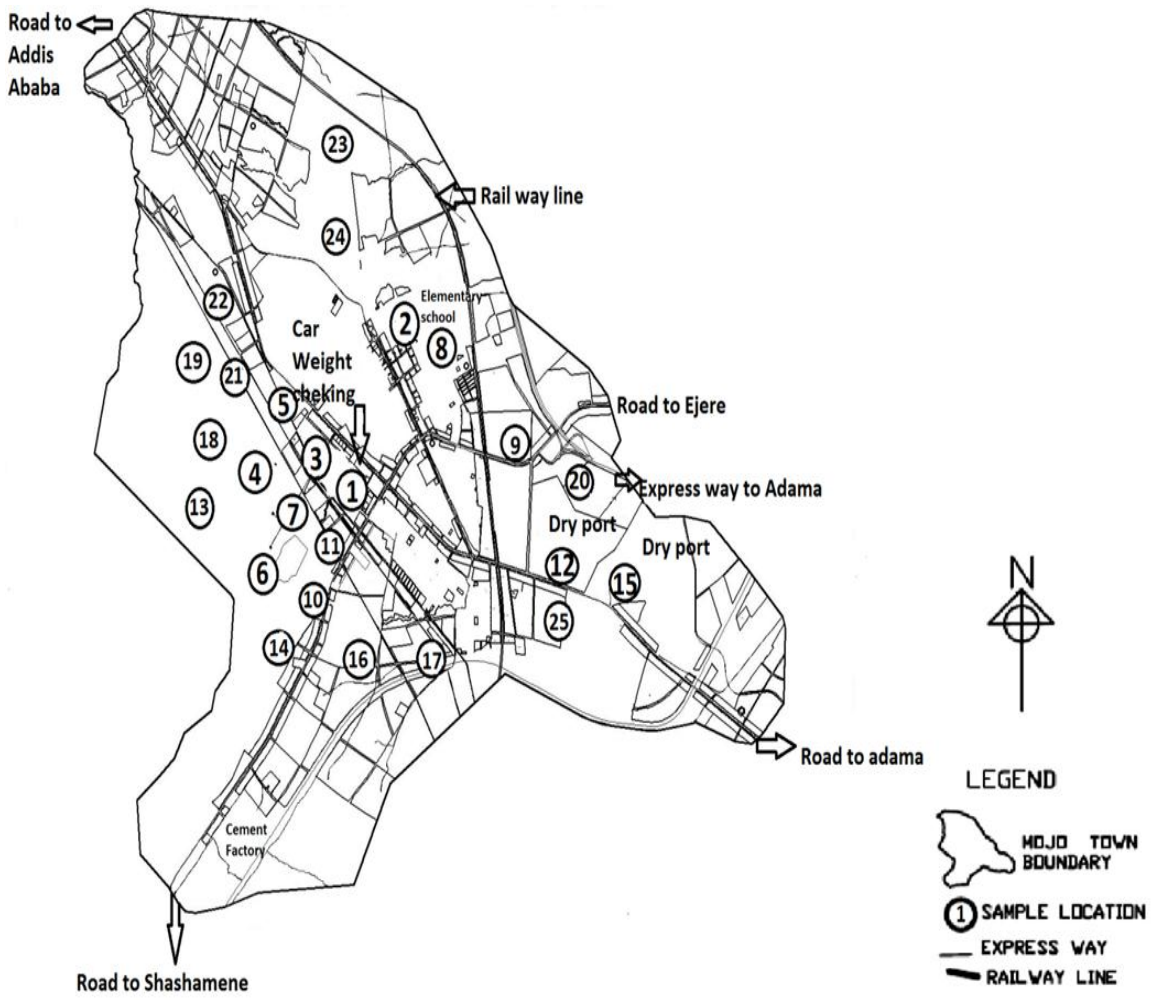


Figure 3.1 Sample Location on map of Mojo Town

3.3 Laboratory Tests and Results

3.3.1 Discussion on Laboratory Tests

Laboratory tests are carried out in accordance with the ASTM standard testing methods. The actual test results are presented in the Appendices.

Total of twenty five undisturbed and disturbed samples were gathered as it will represent mojo town soil based on the samples retrieved from the sites, laboratory tests on the twenty five samples were conducted in the geotechnical laboratories of Addis Ababa Institute of

Technology and at Building Design Share Company laboratories. In addition, by staying about one month at mojo dry port laboratories to undertake moisture content & Atterberg limit tests.

Accordingly, the following different kinds of tests have been performed

- ✓ Particle size distribution (ASTM D422)
- ✓ Specific gravity of soil solid (Gs) (ASTM D854)
- ✓ Natural moisture content (ASTM D2216)
- ✓ Dry density (ASTM D2937)
- ✓ Atterberg limits (ASTM D4318)
- ✓ Shrinkage Limit (ASTM D427)
- ✓ AASHTO classification of a soil(ASTM D3282)
- ✓ USCS classification of a soil(ASTM D2487)
- ✓ Consolidation (ASTM D-2435)

3.4 Laboratory Test Results

The soil profile of Mojo town varies from place to place within the town. Generally, the soil profiles could be treated into two major groups, namely expansive and non-expansive soils. For the laboratory works of this thesis, fine grained soil samples were taken from different sites in town. In addition to the laboratory tests, the presence of distinct cracks on light weight buildings and surface cracks have been used as identifications of the presence of expansive soils in the sampling field.

3.4.1 **Natural Moisture Content**

The natural moisture content tests were carried out in laboratory as per the procedures of ASTM D 2216 - Standard Test Method [4].The laboratory test results show that, moisture contents of the study area falls in the range of (29.4-40.22) %.

Table 3.1 Natural moisture content test results of mojo fine grained soil

Test pit No	Location	Test Pits	Depth(M)	Natural Moisture Content (%)
1	Mizzen Gerber	TP 1	3	32.66
2	Elementary School	TP 2	3	38.28
3	Mizzen Gerber	TP 3	3	35.30
4	Mizzen Gerber	TP 4	3	34.33
5	Mizzen Gerber	TP 5	3	38.11
6	Mizzen Gerber	TP 6	3	31.00
7	Mizzen Gerber	TP 7	3	35.80
8	Elementary School	TP 8	3	31.02
9	Road to Jeer	TP 9	3	30.14
10	Mizzen Gerber	TP 10	3	29.77
11	Mizzen Gerber	TP 11	3	30.17
12	Dry port	TP 12	3	37.84
13	Debora	TP 13	3	30.38
14	Mizzen Gerber	TP 14	3	30.29
15	Dry port	TP 15	3	30.87
16	Near NOC	TP 16	3	40.11
17	Near NOC	TP 17	3	33.31
18	Debora	TP 18	3	29.52
19	Debora	TP 19	3	37.94
20	Road to Ejare	TP 20	3	35.52
21	Mizzen Gerber	TP 21	3	27.23
22	Debora	TP 22	3	36.74
23	Dibandiba	TP 23	3	33.00
24	Dibandiba	TP 24	3	40.22
25	Miizan Gerba	TP 25	3	47.85

3.4.2 **Dry Density Test**

ASTM D 2937-00 – Standard Test Method was laboratory test procedure to carry out these tests. The laboratory test results show that, dry density of the study area falls in the range of 1.2 gm/cm^3-1.88 gm/cm^3 (Table 3.2).

Table 3.2 Dry Density Test Results of Mojo Fine Grained soil

Test pit No	Location	Test Pits	Depth (M)	Bulk Density, (gm/cm^3)	Dry Density, (gm/cm^3)
1	Miizan Gerba	TP 1	3	1.2	0.90
2	Elementary School	TP 2	3	1.77	1.38
3	Miizan Gerba	TP 3	3	1.77	1.42
4	Miizan Gerba	TP 4	3	1.61	1.20
5	Miizan Gerba	TP 5	3	1.4	1.09
6	Miizan Gerba	TP 6	3	1.54	1.18
7	Miizan Gerba	TP 7	3	1.44	1.14
8	Elementary School	TP 8	3	1.67	1.27
9	Road to Ejere	TP 9	3	1.57	1.22
10	Miizan Gerba	TP 10	3	1.83	1.30
11	Miizan Gerba	TP 11	3	1.47	1.07
12	Dry port	TP 12	3	1.6	1.32
13	Dibora	TP 13	3	1.28	1.01
14	Miizan Gerba	TP 14	3	1.6	1.32
15	Dry port	TP 15	3	1.55	1.10
16	Near NOC	TP 16	3	1.62	1.25
17	Near NOC	TP 17	3	1.63	1.26
18	Dibora	TP 18	3	1.59	1.24
19	Dibora	TP 19	3	1.88	1.48
20	Road to Ejere	TP 20	3	1.53	1.17
21	Miizan Gerba	TP 21	3	1.41	1.31
22	Dibora	TP 22	3	1.33	1.11
23	Dibandiba	TP 23	3	1.43	1.22
24	Dibandiba	TP 24	3	1.51	1.35
25	Miizan Gerba	TP 25	3	1.61	1.46

3.4.3 Specific Gravity test

These tests were carried out in laboratory as per the procedures of ASTM D 854-00 Standard Test Method. The laboratory test results show that, specific gravity of the study area falls in the range of 2.6-2.8 (Table 3.3).

Table 3.3 Specific gravity test results of mojo fine grained soil

Test pit No	Location	Test Pits	Depth(M)	Specific Gravity
1	Miizan Gerba	TP 1	3	2.72
2	Elementary School	TP 2	3	2.64
3	Miizan Gerba	TP 3	3	2.68
4	Miizan Gerba	TP 4	3	2.73
5	Miizan Gerba	TP 5	3	2.77
6	Miizan Gerba	TP 6	3	2.65
7	Miizan Gerba	TP 7	3	2.63
8	Elementary School	TP 8	3	2.63
9	Road to Ejere	TP 9	3	2.64
10	Miizan Gerba	TP 10	3	2.69
11	Miizan Gerba	TP 11	3	2.80
12	Dry port	TP 12	3	2.74
13	Dibora	TP 13	3	2.66
14	Miizan Gerba	TP 14	3	2.73
15	Dry port	TP 15	3	2.69
16	Near NOC	TP 16	3	2.63
17	Near NOC	TP 17	3	2.71
18	Dibora	TP 18	3	2.70
19	Dibora	TP 19	3	2.60
20	Road to Ejere	TP 20	3	2.77
21	Miizan Gerba	TP 21	3	2.63
22	Dibora	TP 22	3	2.70
23	Dibandiba	TP 23	3	2.74
24	Dibandiba	TP 24	3	2.71
25	Miizan Gerba	TP 25	3	2.73

3.4.4 Atterberg limit test

ASTM D 4318 - Standard Test Method was laboratory test procedure to carry out these tests. The detail summary of laboratory test results of the Atterberg Limits and Indices of the study area falls in the ranges of:-Liquid Limit (61-101) %; Plastic Limit (27-38) %; Plastic Index (37-65) % and Liquidity Index(-14.43-39.37) % (Table 3.4).

Table 3.4 Atterberg Limits Test Results of Mojo Fine Grained Soil

Test pit No	Location	Test Pits	Depth(M)	Liquid limit(LL) %	Plastic limit(PL) %	Plastic Index(PI) %	Moisture Content(NMC) %	Liquidity Index(LI) %
1	Miizan Gerba	TP 1	3	75	28	47	32.66	9.91
2	Elementary School	TP 2	3	65	28	37	38.28	27.78
3	Miizan Gerba	TP 3	3	83	31	52	35.3	8.27
4	Miizan Gerba	TP 4	3	75	27	48	34.33	15.27
5	Miizan Gerba	TP 5	3	62	26	36	38.11	33.64
6	Miizan Gerba	TP 6	3	75	33	42	31	-4.76
7	Miizan Gerba	TP 7	3	67	26	41	35.8	23.90
8	Elementary School	TP 8	3	78	30	48	31.02	2.13
9	Road to Ejere	TP 9	3	70	22	48	30.14	16.96
10	Miizan Gerba	TP 10	3	71	27	44	29.77	6.30
11	Miizan Gerba	TP 11	3	85	31	54	30.17	-1.54
12	Dry port	TP 12	3	87	35	52	37.84	5.46
13	Dibora	TP 13	3	66	28	38	30.38	6.26
14	Miizan Gerba	TP 14	3	83	28	55	30.29	4.16
15	Dry port	TP 15	3	84	30	54	30.87	1.61
16	Near NOC	TP 16	3	68	22	46	40.11	39.37
17	Near NOC	TP 17	3	86	36	50	33.31	-5.38
18	Dibora	TP 18	3	79	27	52	29.52	4.85
19	Dibora	TP 19	3	87	36	51	37.94	3.80
20	Road to Ejere	TP 20	3	72	24	48	35.52	24.00
21	Miizan Gerba	TP 21	3	73	33	40	27.23	-14.43
22	Dibora	TP 22	3	61	27	34	36.74	28.65
23	Dibandiba	TP 23	3	97	32	65	33	1.54
24	Dibandiba	TP 24	3	93	38	55	40.22	4.04
25	Miizan Gerba	TP 25	3	101	36	65	47.85	18.23

3.4.5 Grain Size Analysis

Grain size analysis tests were carried out in accordance to ASTM D 422 - Standard Test Method. The detail summary of laboratory test results of the grain size analysis of the study area is shown in Table 3.5.

Table 3.5 Grain size analysis of Mojo fine grained soil

Test Pits	Color	Depth(M)	Grain size Analysis			
			Gravel,%	Sand,%	Silt,%	Clay,%
TP 1	Black	3	1.38	12.25	39.11	47.26
TP 2	Grey	3	6.3	15.44	26.56	51.7
TP 3	Light Grey	3	4.95	24.93	14.44	55.68
TP 4	Light Grey	3	1.69	24.27	38.73	35.31
TP 5	Grey	3	0.1	11.69	37.48	50.73
TP 6	Light Grey	3	0.3	15.4	27.47	56.83
TP 7	Brown	3	0.6	12.4	29	58
TP 8	Dark Grey	3	2.45	21.55	7.68	68.32
TP 9	Light Grey	3	9.89	14.14	13.7	62.27
TP 10	Light Grey	3	5.65	10.26	40.03	44.06
TP 11	Dark Grey	3	2.25	11.74	20.85	65.16
TP 12	Light Grey	3	1.73	23.08	20.85	54.34
TP 13	Light Grey	3	0.15	22.62	49.27	27.96
TP 14	Light Grey	3	3.19	10.77	11.05	74.99
TP 15	Dark Grey	3	1.8	10.8	11.06	76.34
TP 16	Light Grey	3	0.88	14.87	24.91	59.34
TP 17	Light Grey	3	6.31	16.16	26.45	51.08
TP 18	Light Grey	3	12.22	14.15	7.78	65.85
TP 19	Black	3	6.31	10.86	14.63	68.2
TP 20	Black	3	6.31	10.71	14.63	68.35
TP 21	Light Red	3	1.09	9.22	46.29	43.4
TP 22	Dark Grey	3	0.45	2.91	57.37	39.27
TP 23	Black	3	0	2.51	24.24	73.25
TP 24	Dark Grey	3	0	11.02	42.79	46.19
TP 25	Black	3	3.51	14.19	30.75	52.55

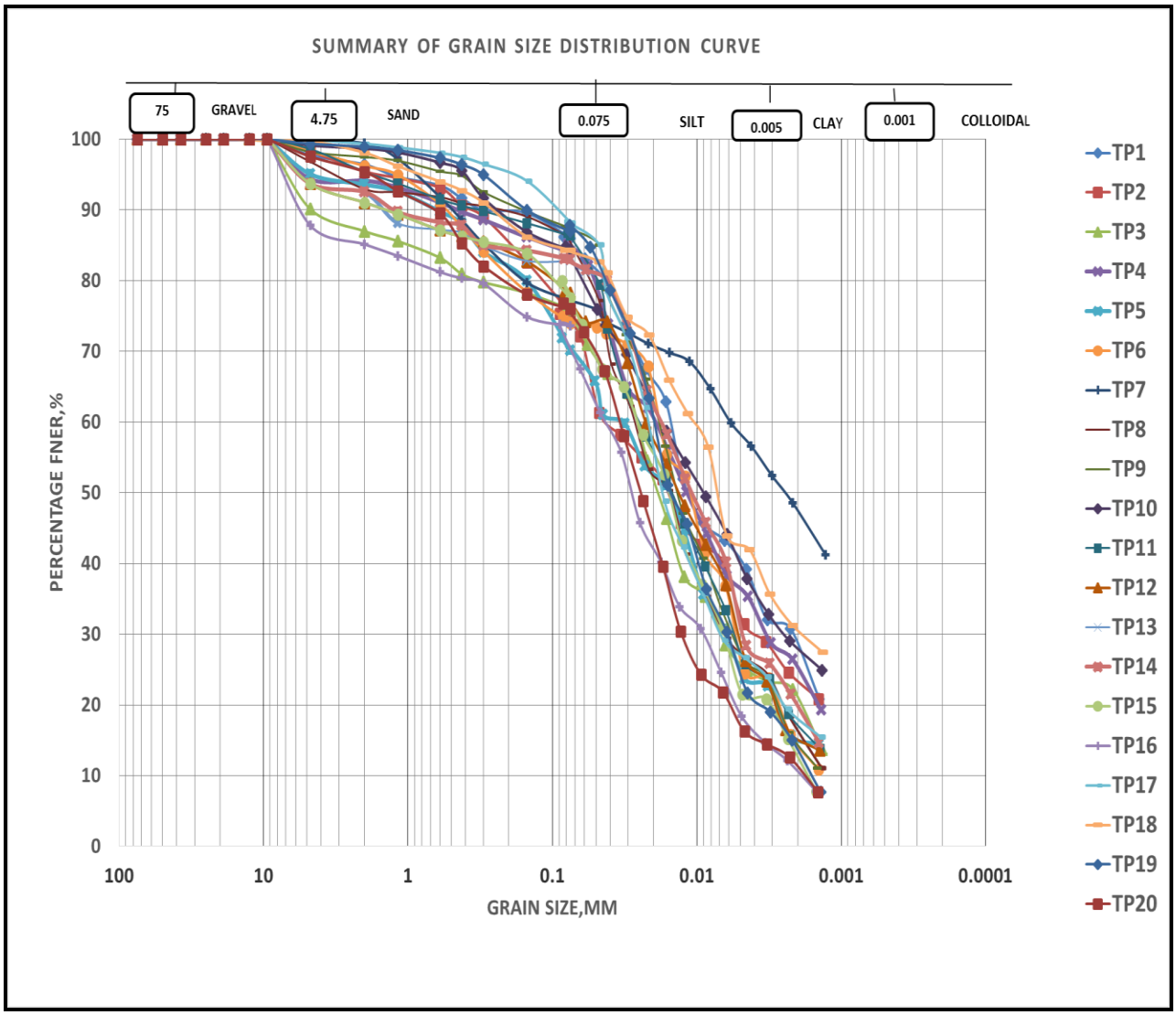


Figure 3.2 Combined sieve-hydrometer analysis graph of the study area

3.4.6 Results for the Free Swell Test

The free swell test is one of the most commonly used simple tests for estimating soil swelling potential. Results of the free swell tests of the study area are shown in Table 4.6. The detailed free swell test results are attached in Appendix A.

Table 3.6 Free swell of the study area

Test Pits	Depth (m)	Free swell (%)
TP 1	3	93
TP 2	3	90
TP 3	3	90
TP 4	3	90
TP 5	3	70
TP 6	3	72
TP 7	3	63
TP 8	3	80
TP 9	3	93
TP 10	3	110
TP 11	3	85
TP 12	3	103
TP 13	3	77
TP 14	3	58
TP 15	3	98
TP 16	3	57
TP 17	3	55
TP 18	3	130
TP 19	3	97
TP 20	3	92
TP 21	3	115
TP 22	3	86
TP 23	3	140
TP 24	3	106
TP 25	3	110

3.4.7 USCS and AASHTO Classification

I) USCS Classification

According to USCS classification scheme the soil of the study area falls in CH & MH region

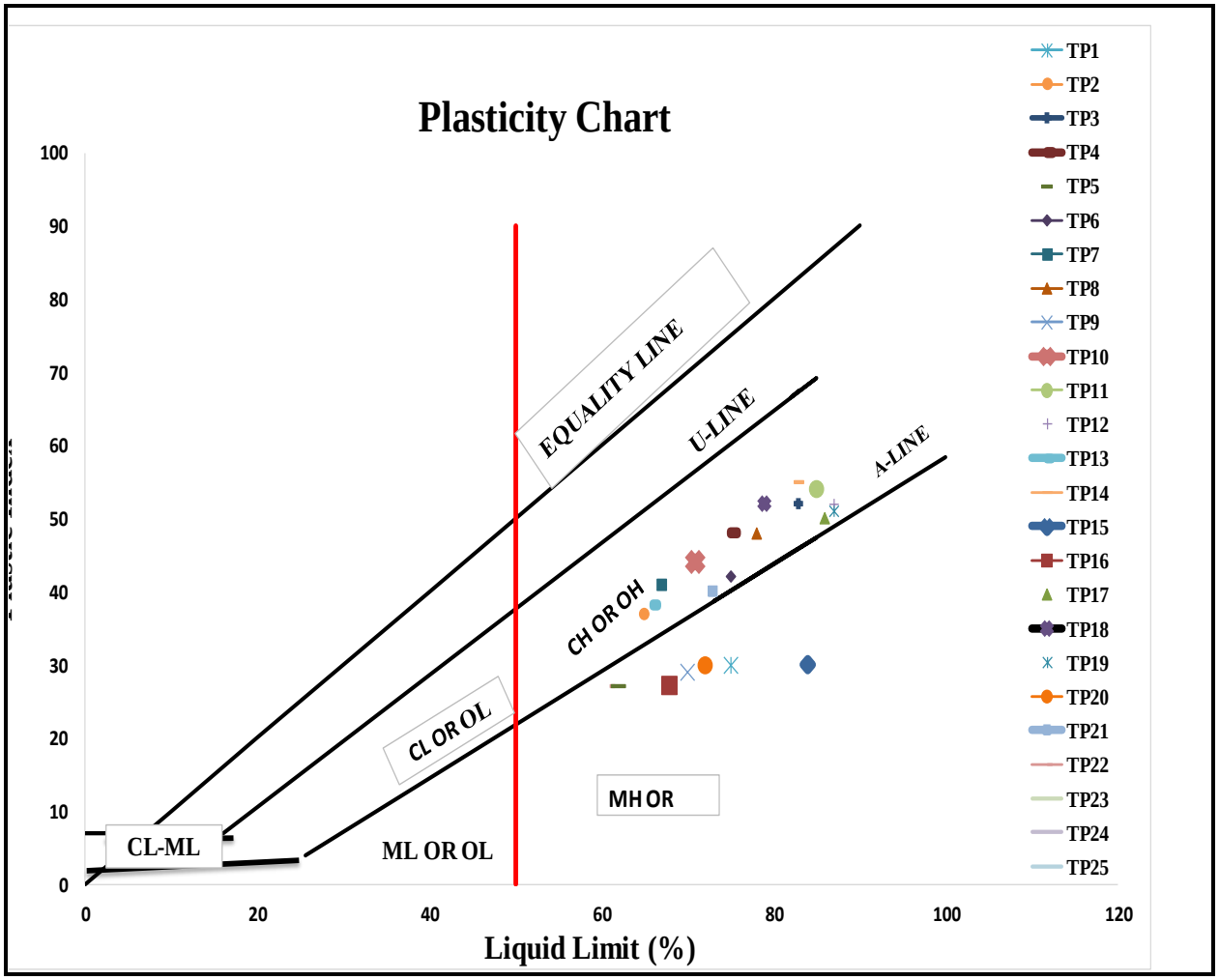


Figure 3.3 Plasticity chart of the soil of the study area, according to Unified Soil Classification System (USCS)

II) AASHTO Classification

According to this system the soil of the study area falls in the region of A-2-7 and A-7-5

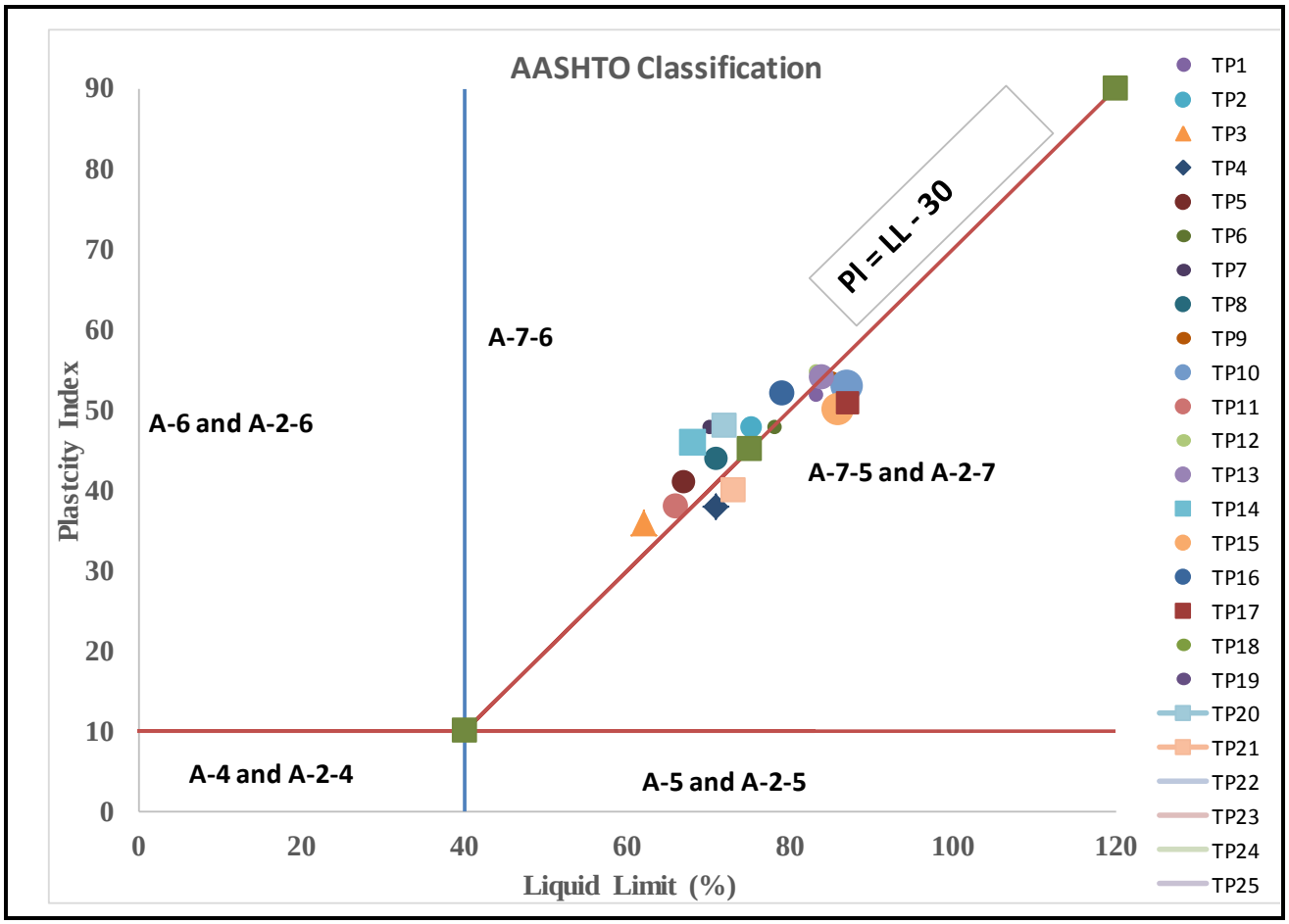


Figure 3.4 Classification of the Soil of the Study Area According To AASHTO System

3.4.8 Other Classifications

(ii) The V. D.Merwe Method [27].

Potential expansiveness (P.E.) of Mojo expansive soil ranges from high to very high for black soils and medium to very high for the gray sample. The PE ratings have been given heave values by V.D.Merwe [27] so as can be Utilized for heave estimation in the field.

The activity defined as:

$$AC = \frac{PI}{C}$$

Where, Ac= activity number

C = percentage of clay size finer than 0.002mm and

PI = plasticity index.

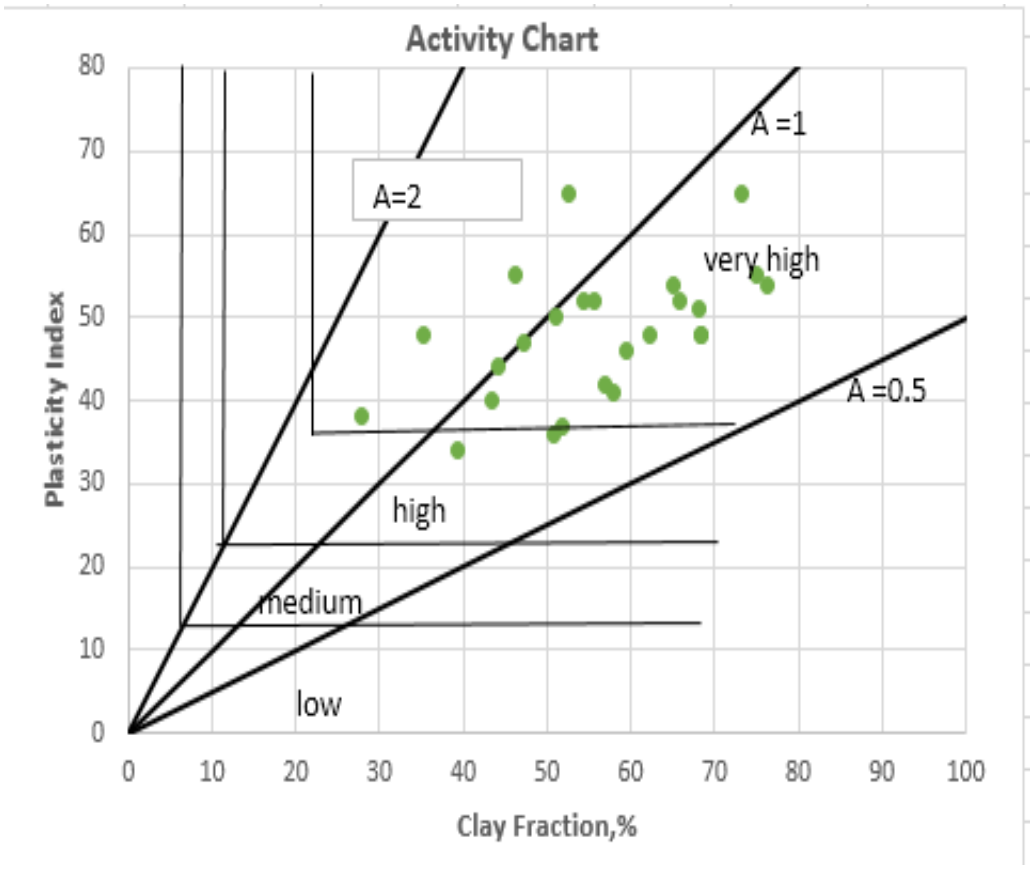


Figure 3.5 Activity chart of Mojo fine grained soils using method of V.D. Merwe [27]

3.4.9 Consolidation Test

In order to achieve the intended objectives, a consolidation- compression index test is carried out on 25 soil samples obtained from 25 test pits in different parts Mojo town where expansive soil is dominant. The ASTM procedures D2435-test methods for one-dimensional consolidation properties of soil, is followed. The standard one dimensional consolidation test is usually carried out on saturated specimen using an Odometer [5]. In this test a small representative sample of soil carefully trimmed and fitted into a rigid metal ring. The soil sample is mounted on a porous stone base and a similar stone is placed on top to permit water, which is squeezed out of the sample to escape freely at the top and bottom. Prior to loading, the height of thesample should be accurately measured. Also, a micrometer dial is mounted in such a manner that the vertical strain in the sample can be measured as loads are applied. The consolidation test apparatus is designed to permit the sample to be submerged in water during the test to

simulate the position below a water table of the prototype soil sample from which the test sample was taken.

Loads are applied in steps in such a way that the successive load intensity, P, is twice the preceding one; the load intensities commonly used being ¼, ½, 1, 2, 4,8,16 kg/m Each load is allowed to stand until primary consolidation is practically ceased. The dial readings are taken at elapsed time of 24 hours. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitude of plastic or permanent deformation.The consolidation characteristics (or parameter) of a soil which is the compression index, Cc is determined from the test. The compression index relates to how much consolidation or settlement will take place [15]. The results of the odometer test are usually presented in the form of an e-log P graph (Figure 3.6)

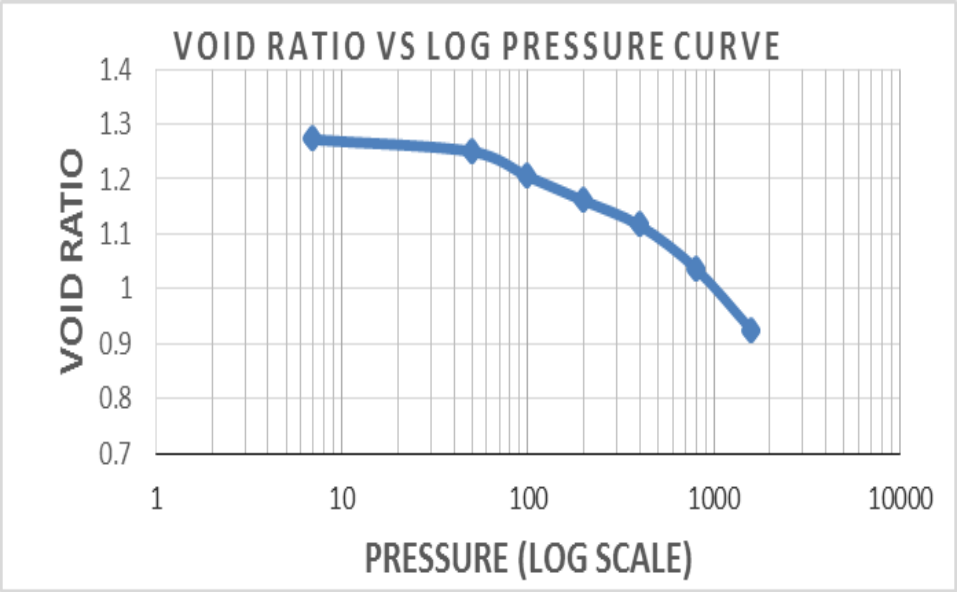


Figure 3.6 Plot of Void Ratio Versus Log-Pressure for Sample Mizan gerba (Pt-5)

Table 3.6 Results of consolidation of the study areas.

Location	Test Pits	Depth(M)	Initial void ratio	Compression Index,Cc
Miizan Gerba	TP 1	3	1.02	0.209
Elementary School	TP 2	3	1.07	0.211
Miizan Gerba	TP 3	3	1.24	0.265
Miizan Gerba	TP 4	3	1.14	0.251
Miizan Gerba	TP 5	3	0.84	0.206
Miizan Gerba	TP 6	3	1.08	0.245
Miizan Gerba	TP 7	3	0.83	0.178
Elementary School	TP 8	3	1.34	0.241
Road to Ejere	TP 9	3	1.33	0.18
Miizan Gerba	TP 10	3	1.47	0.205
Miizan Gerba	TP 11	3	1.32	0.245
Dry port	TP 12	3	1.22	0.298
Dibora	TP 13	3	1.24	0.166
Miizan Gerba	TP 14	3	1.25	0.278
Dry port	TP 15	3	1.14	0.299
Near NOC	TP 16	3	1.22	0.199
Near NOC	TP 17	3	1.09	0.308
Dibora	TP 18	3	1.02	0.243
Dibora	TP 19	3	1.41	0.297
Road to Ejere	TP 20	3	1.24	0.197
Miizan Gerba	TP 21	3	1.75	0.255
Dibora	TP 22	3	1.27	0.178
Dibandiba	TP 23	3	1.15	0.327
Dibandiba	TP 24	3	1.22	0.335
Miizan Gerba	TP 25	3	1.33	0.399

3.5 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 3.7.

Table 3.7 Summary of Laboratory Test Results

It. No.	Sample Code	Soil Classification									Atterberg Limits				Consolidation	
		bulk density	Dry density	NMC	specific gravity	AASHTO Class	USCS Class	Clay fraction	Activity of clay	Free swell	LL	PL	PI	LI	e_O	C_C
1	TP1	1.2	0.9	32.66	2.72	A-7-6	MH	47.26	0.99	93	75	28	47	0.10	1.02	0.209
2	TP2	1.77	1.38	38.28	2.64	A-7-5	CH	51.7	0.72	90	65	28	37	0.28	1.07	0.211
3	TP3	1.77	1.42	35.3	2.68	A-7-6	CH	55.68	0.93	90	83	31	52	0.08	1.24	0.265
4	TP4	1.61	1.2	34.33	2.73	A-7-6	CH	35.31	1.36	90	75	27	48	0.15	1.14	0.251
5	TP5	1.4	1.09	38.11	2.77	A-7-6	MH	50.73	0.71	70	62	26	36	0.34	0.84	0.206
6	TP6	1.54	1.18	31	2.65	A-7-6	CH	56.83	0.74	72	75	33	42	-0.05	1.08	0.245
7	TP7	1.44	1.14	35.8	2.63	A-7-6	CH	58	0.71	63	67	26	41	0.24	0.83	0.178
8	TP8	1.67	1.27	31.02	2.63	A-7-5	CH	68.32	0.70	80	78	30	48	0.02	1.34	0.241
9	TP9	1.57	1.22	30.14	2.64	A-7-5	CH	62.27	0.77	93	70	22	48	0.17	1.33	0.18
10	TP10	1.83	1.3	29.77	2.69	A-7-6	CH	44.06	1.00	110	71	27	44	0.06	1.47	0.205
11	TP11	1.47	1.07	30.17	2.8	A-7-6	CH	65.16	0.83	85	85	31	54	-0.02	1.32	0.245
12	TP12	1.6	1.32	37.84	2.74	A-7-6	CH	54.34	0.96	103	87	35	52	0.05	1.22	0.298
13	TP13	1.28	1.01	30.38	2.66	A-7-6	CH	27.96	1.36	77	66	28	38	0.06	1.24	0.166
14	TP14	1.6	1.32	30.29	2.73	A-7-6	CH	74.99	0.73	58	83	28	55	0.04	1.25	0.278
15	TP15	1.55	1.1	30.87	2.69	A-7-5	MH	76.34	0.71	98	84	30	54	0.02	1.14	0.299
16	TP16	1.62	1.25	40.11	2.63	A-7-6	CH	59.34	0.78	57	68	22	46	0.39	1.22	0.199
17	TP17	1.63	1.26	33.31	2.71	A-7-6	CH	51.08	0.98	55	86	36	50	-0.05	1.09	0.308
18	TP18	1.59	1.24	29.52	2.7	A-7-6	CH	65.85	0.79	130	79	27	52	0.05	1.02	0.243
19	TP19	1.88	1.48	37.94	2.6	A-7-6	CH	68.2	0.75	97	87	36	51	0.04	1.41	0.297
20	TP20	1.53	1.17	35.52	2.77	A-7-5	MH	68.35	0.70	92	72	24	48	0.24	1.24	0.197
21	TP21	1.41	1.31	27.23	2.63	A-7-6	CH	43.4	0.92	115	73	33	40	-0.14	1.75	0.225
22	TP22	1.33	1.11	36.74	2.7	A-7-5	MH	39.27	0.87	86	61	27	34	0.29	1.27	0.178
23	TP23	1.43	1.22	33	2.74	A-7-5	CH	73.25	0.89	140	97	32	65	0.02	1.15	0.327
24	TP24	1.51	1.35	40.22	2.71	A-7-5	CH	46.19	1.19	106	93	38	55	0.04	1.22	0.335
25	TP25	1.61	1.46	47.85	2.73	A-7-5	CH	52.55	1.24	110	101	36	65	0.18	1.33	0.399

4. REGRESSION ANALYSIS AND CORRELATIONS

4.1 Introduction

Regression analysis is a statistical technique that is 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 the line or curve which provides the best of it through a set of data points. This basic approach is applicable in situations ranging from single linear regression to more sophisticated nonlinear, parabolic or logarithmic trend. A linear relationship is usually practiced in solving different engineering problems because of its simplicity.

Fitting a regression model requires several assumptions. The method of least squares is used in order to choose the best fitting line for a set of data. Estimation of the model parameters requires the assumption that, the residuals (actual values less estimated values) corresponding to different observations are uncorrelated random variables with zero mean and constant variance. In most practical situation, the variance of the random error will be unknown and must be estimated from the sample data [17]. The standard error of an estimate gives some idea about the precision of an estimate. During modeling, a variable that shows the least standard error of estimates is the one to be chosen.

A convenient way of measuring how well the regression model performs as a predictor of the dependent variable is to compute the reduction in the sum of squares of deviations that can be attributed to regressor variables and this quantity termed the coefficient of determination, R^2.The value of R^2 is always between 0 and 1, because R is between 1 and +1, where by a negative value of R indicates inversely relationship and positive value implies direct relationship. Many problems in engineering require that we decide whether to accept or reject a statement about some correlations. A number of techniques can be used to judge the adequacy of a regression model some of which are standard error (α), R-squared value (R^2), R-adjusted and the t-test [18].

4.2 Scatter Plot

In this study, the Compression index is taken as the dependent variable whereas the liquid limit, plastic limit, plasticity index, Natural Moisture content, Void Ratio Specific gravity, Liquidity index and Clay fraction are independent variables. Prior to carrying out the regression analysis using the twenty five test results, a scatter diagram generated by applying the Excel Spreadsheet, in order to study the relationships developed between the dependent variable and the regression variables so as to determine the model that best suits the test results.

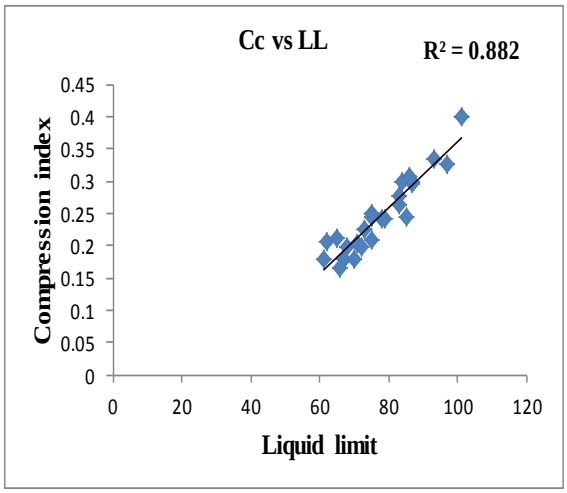


Figure 4.1 Scattered diagram of Cc vs. LL

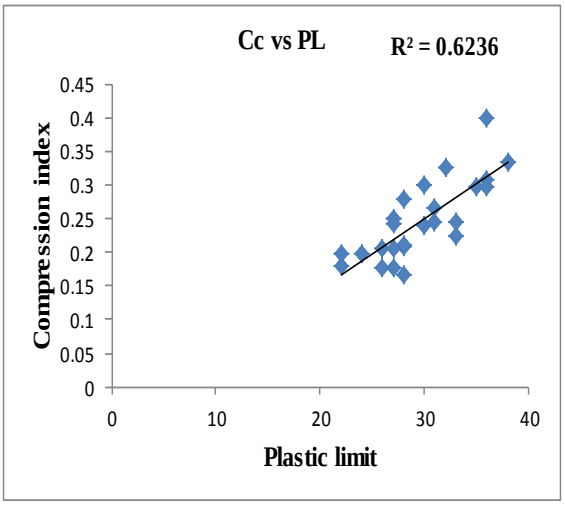


Figure 4.2 Scattered diagram of Cc vs. PL

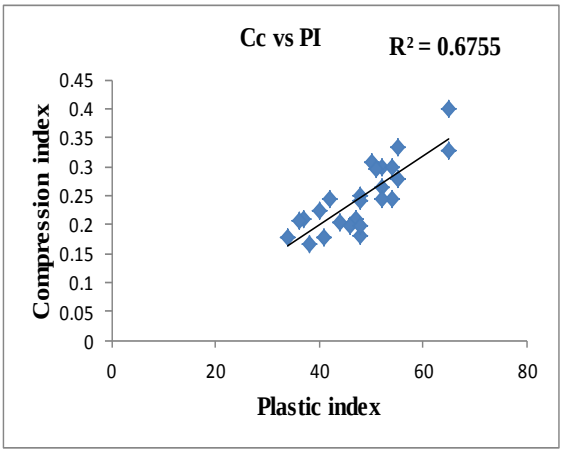


Figure 4.3 Scattered diagram of Cc vs. PI

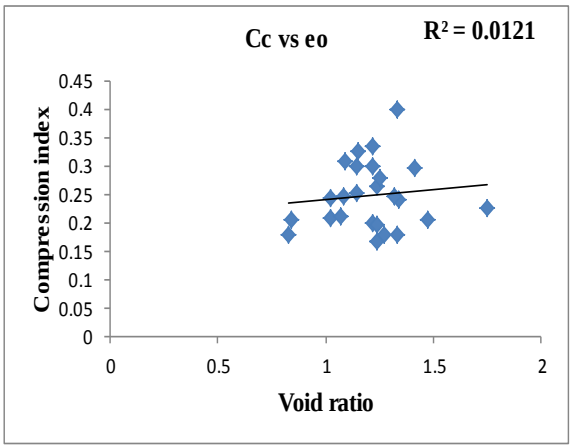
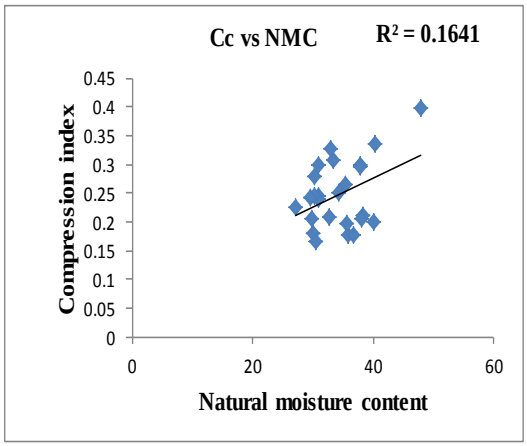


Figure 4.4 Scattered diagram of Cc vs.e_o



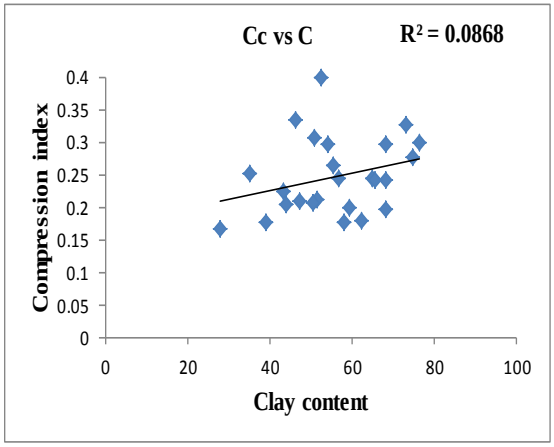


Figure 4.5 Scattered diagram of Cc vs. NMC Figure 4.6 Scattered diagram of Cc Vs.C

The above scatter diagrams provide a visual method of displaying a relationship between variables as plotted in a two dimensional coordinate system. Inspection of the scatter diagrams indicate that, although no simple curve will pass exactly through all the points, there is a reasonable indication that the points lie scattered randomly around a straight line, particularly for the liquid limit, plasticity index and plastic limit While the remaining independent variables such as moisture content, Void Ratio results outliers away from the possible straight line. Relatively the above scatter plot shows a linear response and hence, a linear regression model expresses the association between the subject parameters.

4.3 Regression Analysis

In this research work, an attempt is made to apply single linear regression model and multiple linear regression models to characterize compressibility of soil from soil index parameters using a statistical approach. The general representation of a probabilistic single and multiple linear regression models are presented in the following forms:

$$Y = \beta o + \beta 1 x + \varepsilon \text{----------[4.1]}$$

$$Y = \alpha o + \alpha 1 x1 + \alpha 2 x2 ..... + \alpha n xn + \varepsilon \text{----------[4.2]}$$

Where, the slope (β1) and intercept (β0) of the single linear regression model are called regression coefficients. Similarly, coefficients α0, α1, α2 and αn are termed multiple

regression coefficients. The appropriate way to generalize this to a probabilistic linear model is to assume that the actual value of Y is determined by the mean value function (the linear model) plus the random error term, ε [17]. The basic assumption to estimate the regression coefficients of the single and multiple regression models is based on the least square method.

Specific to this research, a statistical package for social science software (SPSS) is employed to investigate the significance of individual regressor variables. Accordingly, the twenty five laboratory test results of the independent and dependent variables are used in the following

4.4 Development of New Empirical Equations

In developing new empirical equations for prediction of the compression behavior of the study area, several functions composed of different parameters in different combinations were tested using the SPSS 20 software (Appendix B). Out of these, equations with higher correlation coefficient were selected and using these equations the compression behavior of the soil of the study area were calculated. Then a graph is plotted which shows the measured value against the predicted or calculated value. Finally equations 17 are selected which predicted the measured value better than the others. This is done using a multiple regression analysis. Refer (Appendix -B).

4.4.1 Single Linear Regression Analysis

EQN1: Correlation between compression index (C_c) and liquid limit (LL)

The resulting equation from the regression analysis made between compression index and liquid limit is presented as follows;

Cc= -0.151+0.005LL, With **R^2 =0.815** and sample number = 25

EQN2: Correlation between compression index (C_c) and plastic limit (PL)

The resulting equation from the regression analysis made between compression index and plastic limit is presented as follows

C_c= -0.0064+0.010PL With **R^2 =0.6241** and sample number = 25

EQN3: Correlation between compression index (C_c) and Natural moisture content (NMC)

The resulting equation from the regression analysis made between compression index and Natural moisture content is presented as follows

C_c= 0.072+0.005NMC With **R^2 =0.164** and sample number = 25

EQN.7: Correlation between compressions index (C_c) and void ratio (e_o)

The resulting equation from the regression analysis made between compression index and Void ratio is presented as follows

C_c= 0.207+0.034e_o With R^2 = 0.012 and sample number = 25

4.4.2 Multiple Linear Regression Analysis

EQN.9: Correlation between compression index (C_c) and liquid limit &Natural moisture content.

The resulting equation from the regression analysis is presented as follows;

C_c= -0.237+0.005LL+0.003NMC With R^2 = 0.873 and sample number = 25

EQN.16: Correlation between compression index (C_c) and liquid limit, Plastic limit &initial void ratio.

The resulting equation from the regression analysis is presented as follows.

C_c = -0.155+0.00LL+0.005PL+0.003e_0 with R^2=0.88 and sample number = 25

EQN.17: Correlation between compression index (C_c) and liquid limit, Plastic limit &Natural moisture content.

The resulting equation from the regression analysis is presented as follows

C_c= -0.253+0.004LL+0.004PL+0.003NMC With R^2 = 0.917 and sample number = 25

Table 4.1 Summary of the Developed correlations

TYPES OF REGRESSION	R^2	DEVELOPED EQUATION	EQUATIONS
SINGLE	0.815	Cc = -0.151+0.005LL	Equation 1
SINGLE	0.624	Cc = -0.064+0.010PL	Equation 2
SINGLE	0.164	Cc = 0.072+0.005NMC	Equation 3
SINGLE	0.12	Cc = 0.163+0.001Fs	Equation 4
SINGLE	0.087	Cc = 0.171+0.001C	Equation 5
SINGLE	0.068	Cc = -0.534+0.290Gs	Equation 6
SINGLE	0.012	Cc = 0.207+0.034eo	Equation 7
MULTIPLE	0.819	Cc = -0.132+0.005LL-0.020eo	Equation 8
MULTIPLE	0.873	Cc = -0.237+0.005LL+0.003NMC	Equation 9
MULTIPLE	0.83	Cc = -0.142+0.006LL-0.001C	Equation 10
MULTIPLE	0.815	Cc = -0.171+0.005LL+0.008Gs	Equation 11
MULTIPLE	0.722	Cc = -0.148+0.011PL+0.001C	Equation 12
MULTIPLE	0.675	Cc = -0.142+0.010PL+0.003NMC	Equation 13
MULTIPLE	0.671	Cc = -0.708+0.01PL+0.241GS	Equation 14
MULTIPLE	0.629	Cc = -0.042+0.011PL-0.023eo	Equation 15
MULTIPLE	0.884	Cc = -0.155+0.004LL+0.005PL-0.031eo	Equation 16
MULTIPLE	0.917	Cc = -0.253+0.004LL+0.004PL+0.003NMC	Equation 17
MULTIPLE	0.878	Cc = -0.348+0.004LL+0.005PL+0.064Gs	Equation 18
MULTIPLE	0.791	Cc = -0.248+0.010PL+0.002C+0.003NMC	Equation 19
MULTIPLE	0.762	Cc = -0.740+0.010PL+0.001C+0.223GS	Equation 20
MULTIPLE	0.725	Cc = -0.132+0.011PL+0.001C-0.017eo	Equation 21
MULTIPLE	0.675	Cc = -0.137+0.010PL-0.004eo+0.003NMC	Equation 22

R^2 is coefficient of correlation

Using the above new equations the compression index of the study area is calculated. The results of the analysis are given in (Table 4.2).

Table 4.2 Comparison of measured value with calculated value

Measured Value,(Cc)	Eqn 1	Eqn 8	Eqn 9	Eqn 10	Eqn 11	Eqn 16	Eqn 17	Eqn 18	Eqn 19	Eqn 20
0.209	0.257	0.223	0.236	0.261	0.226	0.253	0.224	0.266	0.225	0.194
0.211	0.234	0.172	0.203	0.196	0.175	0.212	0.174	0.221	0.250	0.180
0.265	0.309	0.258	0.284	0.300	0.265	0.294	0.264	0.311	0.279	0.223
0.251	0.258	0.220	0.241	0.273	0.226	0.245	0.224	0.262	0.196	0.174
0.206	0.213	0.161	0.187	0.179	0.161	0.197	0.159	0.207	0.228	0.188
0.245	0.272	0.221	0.231	0.251	0.225	0.277	0.224	0.287	0.289	0.238
0.178	0.226	0.186	0.205	0.202	0.185	0.217	0.184	0.218	0.235	0.164
0.241	0.272	0.231	0.246	0.258	0.240	0.265	0.239	0.282	0.282	0.215
0.18	0.205	0.191	0.203	0.216	0.200	0.194	0.199	0.211	0.187	0.131
0.205	0.228	0.194	0.207	0.240	0.206	0.218	0.204	0.243	0.199	0.174
0.245	0.302	0.267	0.279	0.303	0.276	0.299	0.274	0.326	0.283	0.260
0.298	0.349	0.279	0.312	0.326	0.286	0.330	0.284	0.350	0.324	0.275
0.166	0.214	0.173	0.184	0.226	0.180	0.211	0.179	0.226	0.179	0.161
0.278	0.282	0.258	0.269	0.281	0.266	0.278	0.264	0.299	0.273	0.224
0.299	0.296	0.265	0.276	0.286	0.271	0.296	0.269	0.310	0.297	0.236
0.199	0.227	0.184	0.223	0.207	0.190	0.189	0.189	0.202	0.211	0.126
0.308	0.335	0.276	0.293	0.323	0.281	0.335	0.279	0.349	0.314	0.275
0.243	0.260	0.243	0.247	0.266	0.246	0.264	0.244	0.276	0.242	0.198
0.297	0.353	0.275	0.312	0.312	0.285	0.329	0.284	0.346	0.362	0.268
0.197	0.238	0.203	0.230	0.222	0.211	0.215	0.209	0.237	0.235	0.186
0.225	0.253	0.198	0.210	0.253	0.215	0.248	0.214	0.277	0.250	0.220
0.178	0.209	0.148	0.178	0.185	0.156	0.185	0.154	0.204	0.211	0.171
0.327	0.362	0.330	0.347	0.367	0.336	0.357	0.334	0.375	0.318	0.264
0.335	0.392	0.309	0.349	0.370	0.316	0.369	0.314	0.387	0.345	0.291
0.399	0.439	0.346	0.412	0.411	0.356	0.388	0.354	0.411	0.361	0.281

4.5 Graphical Representation of the Measured and Calculated Values

The following graphs are plotted to investigate the approximation accuracy of the newly developed formulas. The measured and calculated values are plotted (Fig. 4.7 to Fig.4.8) trend lines are drawn to observe the gap between the measured and the calculated values.

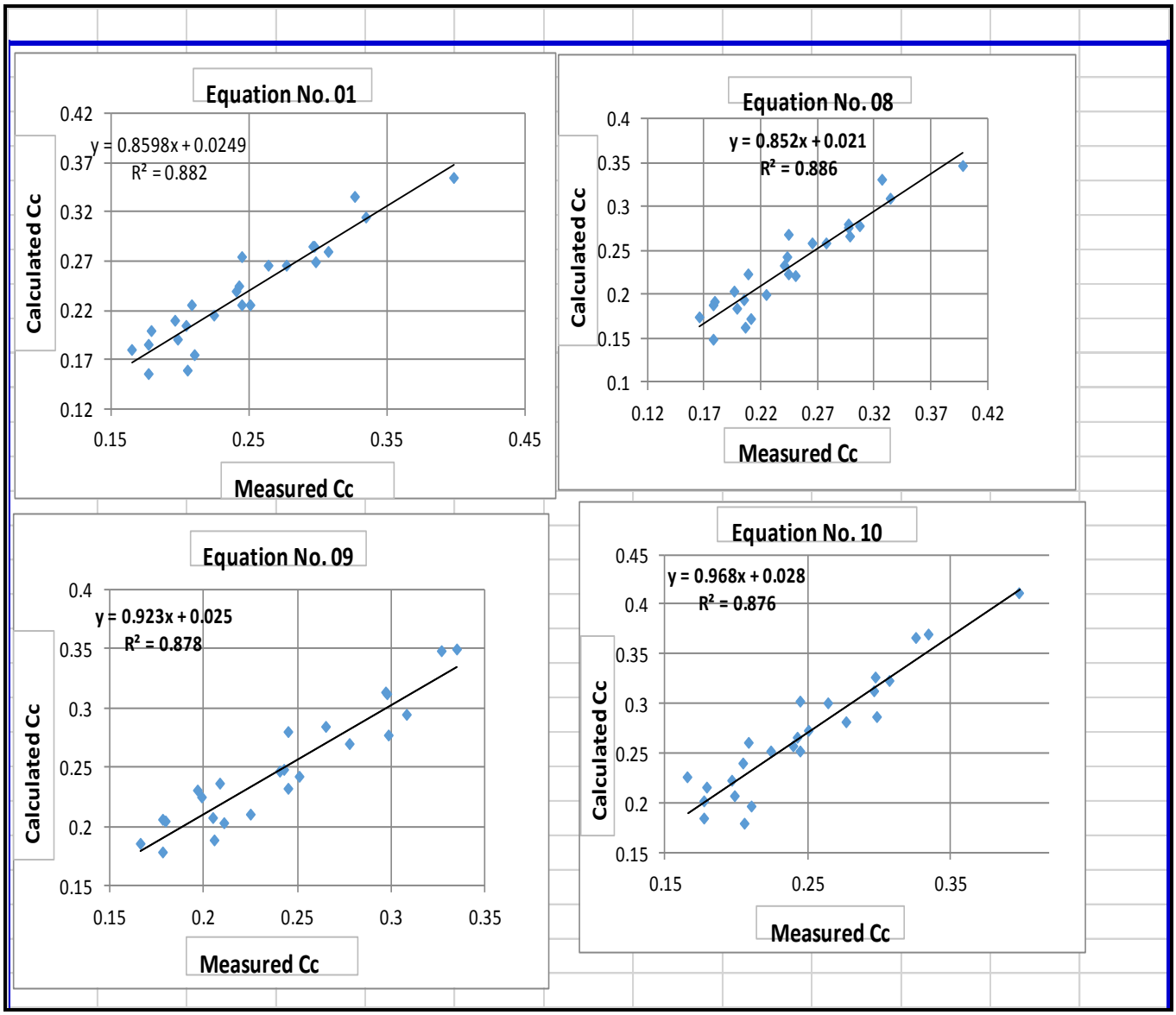


Figure 4.7 Plot of Equetion-1, 8, 9, & 10 vs. measured value

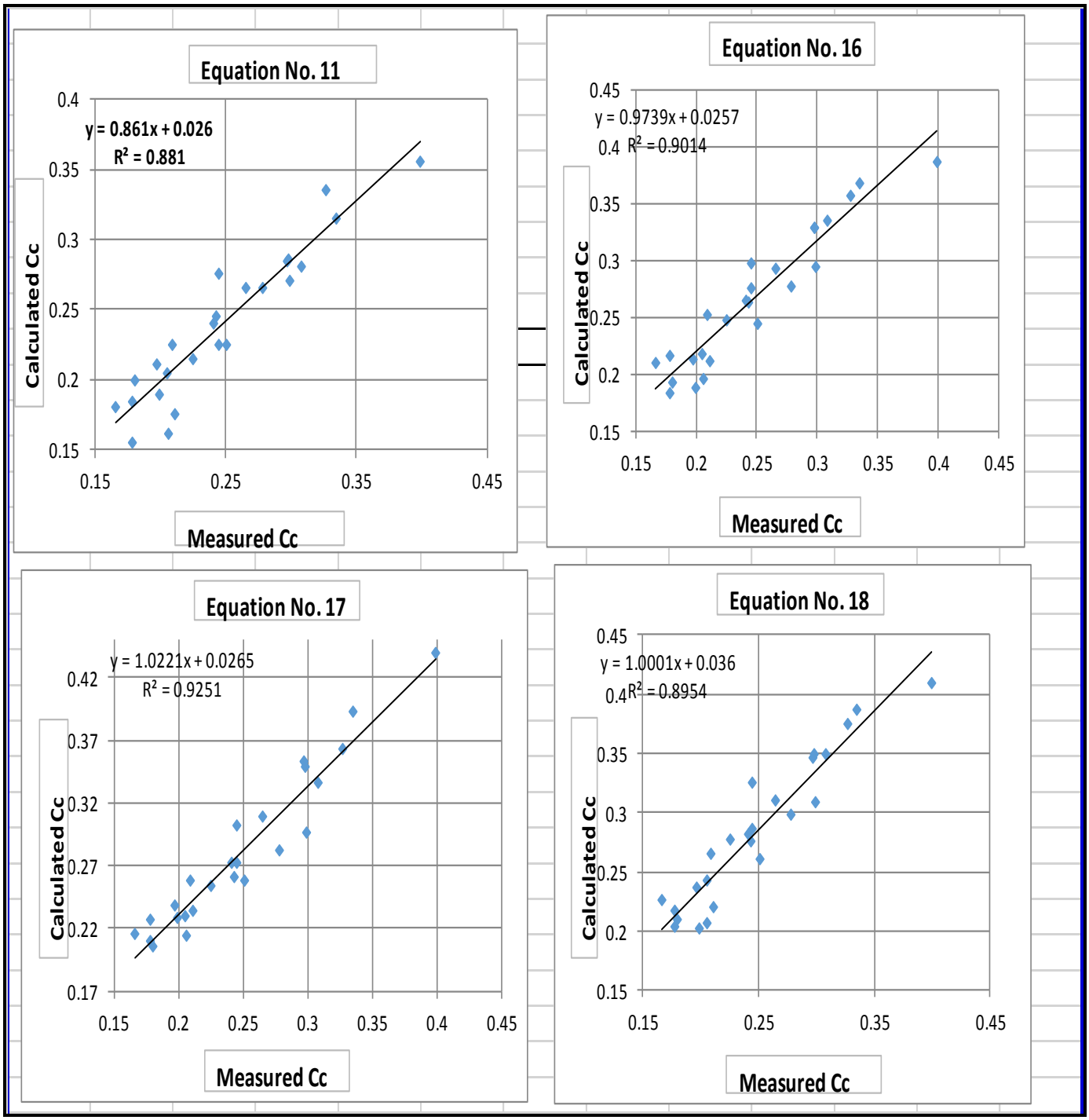


Figure 4.8 Plot Equetion-11, 16, 17, & 18 vs. measured value

5 DISCUSSION ON CORRELATION RESULTS

5.1 The Developed Correlation

The validation of the developed correlation is conducted by using three known test results which follows similar testing procedures with the subject research. These control test results were obtained from thesis conducted by Yohannis H. [37] [Table 5.1]. Depending on the relative significance order Eqn.17, C_c=-0.253+0.004LL+0.004PL+0.003NMC, is preferably selected among the different alternative correlations for further verifications.

Table 5.1 Validation of the developed correlation from previously conducted thesis, Yohannis H. [37].

Test Pit ID No	Independent Variable									Actual Cc Value	Developed Cc Value	Variation ,%
	LL,%	PL,%	PI, %	eo	NMC	FS	Activity	C,%	Gs			
MD	65	28	37	1.3	28.8	45	2.40	15	2.63	0.314	0.263	16.21
TK	87	28	59	1.3	28.4	30	1.54	38	2.67	0.249	0.264	-5.97
SH	64	31	33	1.3	17.4	40	1.44	23	2.62	0.383	0.346	9.69

5.2 Comparison Between Developed and Existing Correlations Equations

The existing empirical equations and the newly developed equations are utilized to calculate the compression index for some the index properties considered in the study in a view to analyze the significance for some of the index properties considered in the study in a view to analyses the significance of both formulae to the Mojo fine grained soils. Most of papers reviewed indicate that the liquid limit, plastic limit in-situ water content & void ratio are considered for the evaluation purposes (Table 5.2).

Table 5.2 Developed and existing correlation

Test pit ID No	Measured value of, Cc	Calculated Cc Values using Previously developed eqn									
		Calculated CC, value from developed Eqn	Difference from Actual,Cc value	Solomon Desta,2012	Difference from Actual,Cc value	Legesse Abadi,2014	Difference from Actual,Cc value	skempton(1944)	Difference from Actual,Cc value	Worth & Wood, 1978	Difference from Actual,Cc value
TP1	0.209	0.26	22.96	0.106	-49.5	0.385	84.44	0.476	127.751	0.639	-36.08
TP2	0.211	0.23	10.82	0.049	-77.0	0.359	69.95	0.406	92.417	0.488	-51.16
TP3	0.265	0.31	16.57	0.151	-43.0	0.369	39.11	0.532	100.755	0.697	-30.32
TP4	0.251	0.26	2.78	0.106	-58.0	0.385	53.54	0.476	89.641	0.655	-34.48
TP5	0.206	0.21	3.56	0.031	-84.8	0.229	10.94	0.385	86.893	0.499	-50.14
TP6	0.245	0.27	11.02	0.106	-56.9	0.270	10.34	0.476	94.286	0.557	-44.35
TP7	0.178	0.23	27.19	0.060	-66.3	0.172	-3.37	0.420	135.955	0.539	-46.09
TP8	0.241	0.27	12.89	0.123	-49.1	0.277	15.09	0.497	106.224	0.631	-36.88
TP9	0.18	0.21	14.12	0.077	-57.2	0.319	77.48	0.441	145.000	0.634	-36.64
TP10	0.205	0.23	11.37	0.083	-59.7	0.488	137.99	0.448	118.537	0.592	-40.82
TP11	0.245	0.30	23.07	0.163	-33.7	0.320	30.48	0.546	122.857	0.756	-24.40
TP12	0.298	0.35	16.95	0.174	-41.6	0.473	58.83	0.560	87.919	0.712	-28.76
TP13	0.166	0.21	29.00	0.054	-67.3	0.328	97.64	0.413	148.795	0.505	-49.46
TP14	0.278	0.28	1.39	0.151	-45.6	0.120	-56.83	0.532	91.367	0.751	-24.93
TP15	0.299	0.30	-1.13	0.157	-47.6	0.369	23.52	0.539	80.268	0.726	-27.37
TP16	0.199	0.23	14.24	0.066	-67.0	0.109	-45.07	0.427	114.573	0.605	-39.51
TP17	0.308	0.33	8.74	0.168	-45.4	0.198	-35.77	0.553	79.545	0.678	-32.25
TP18	0.243	0.26	6.81	0.128	-47.2	0.564	132.22	0.504	107.407	0.702	-29.80
TP19	0.297	0.35	18.79	0.174	-41.4	0.416	39.93	0.560	88.552	0.663	-33.70
TP20	0.197	0.24	20.59	0.088	-55.1	0.313	59.04	0.455	130.964	0.665	-33.52
TP21	0.225	0.25	12.31	0.094	-58.2	0.555	146.76	0.462	105.333	0.526	-47.40
TP22	0.178	0.21	17.54	0.026	-85.6	0.353	98.57	0.378	112.360	0.459	-54.10
TP23	0.327	0.36	10.70	0.231	-29.4	0.640	95.57	0.630	92.661	0.891	-10.95
TP24	0.335	0.39	16.91	0.208	-37.9	0.526	56.90	0.602	79.701	0.745	-25.48
TP25	0.399	0.44	9.91	0.254	-36.4	0.525	31.55	0.658	64.912	0.887	-11.28

It can be clearly observed from above table the newly developed relation is betters when compared to the existing equations. The existing equation and the newly developed equations yield compression index values with a maximum variability of about 85.9% from the actual compression index values and a maximum variability of about 22% from actual values respectively. This is may be due to the difference in test procedures and also the unique properties of the geological material where this correlation was developed. In light of the above, it is worth to note that the test results obtained from the subject study area are not suited by the above existing correlations.

6. CONCLUSION AND RECOMMENDATION

6.1 CONCLUSIONS

The research was conducted to find a localized correlation between C_c value and soil index properties within the scope of the study. Accordingly, the required laboratory tests were conducted on samples retrieved from different geographical area of Mojo town.

Using the obtained twenty five test results a single and multiple linear regressions were analyzed and a relationship was deveoped that predict C_c value in terms of LL,PL,PI,NMC,Gs,LI,Clay fraction,FSI,and e_o.

In accordance with the statistical computations made, the relationship that has been made between LL ,PL, & NMC and Cc is found to be with the highest coefficient of determination(R^2) Value, which equals to 91.7% and the correlation equation for the same is given by:

$$\mathbf{Cc=-0.253+0.004LL+0.004PL+0.003NMC}$$

In view of the above points and the analysis carried out to relate compression index to the various index properties, the following conclusions are made.

- ✓ The liquid limit, Plastic limit, & Natural moisture content generate a more reliable or stronger relation with compression index than the other parameters considered in the regression analysis.
- ✓ The comparisons made between the newly developed equations and the existing equations reveal that the developed equation approximate compression index value of Mojo fine grained soils in a better way.
- ✓ For preliminary design purpose the above correlation might use, if the predicted C_c value is with minimum variation otherwise, a detailed laboratory test should be carried out to obtain the actual C_c value.

6.2 RECOMMENDATIONS

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 recommndations in relation to the subject study.

- ✓ In order to enforce the conclusion drawn herein above and determine the extent of applicability of the newly developed empirical equations, it is suggested that additional works are required to be executed.
- ✓ The equations developed may be further improved by increasing numbers of samples collected from different locations of Mojo town especially from areas cannot covered in this paper.
- ✓ It is also recommended to carry out such a study in other parts of Ethiopia especially in regions where clay soils are abundantly to be found.

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APPENDIX A: DETAILS OF THE LABORATORY TEST RESULTS

Table I-1 Laboratory test results of Bulk density and Dry density For Test Pit 1, 2, 15&16

Bulk density & dry density determination		
Sample location	TP-1	
Sample depth(m)	3	
Trial	1	2
Internal diameter of core cutter in CM	7.5	7.5
Internal height of core cutter in CM	12.5	12.5
Volume of core cutter in(V) in CM	551.95	551.95
Weight of core cutter(W1) in gram	510	510
Weght of core cutter + soil(W2) in gram	1158	1182
Weight of soil(W2-W1) IN GRAM	648	672
Bulk density of soil(Pb)=(W2-W1)/V in gram/cm3	1.17	1.22
Average value in g/cm3	1.2	
moisture content(W)in %	32.52	32.8
Dry density of soil in g/cm3=100pb/100+w	0.89	0.92
Average value	0.905	

Bulk density & dry density determination		
Sample location	TP-2	
Sample depth(m)	3	
Trial	1	2
Internal diameter of core cutter in CM	7.5	7.5
Internal height of core cutter in CM	12.5	12.5
Volume of core cutter in(V) in CM	551.95	551.95
Weight of core cutter(W1) in gram	510	510
Weght of core cutter + soil(W2) in gram	1482	1494
Weight of soil(W2-W1) IN GRAM	972	984
Bulk density of soil(Pb)=(W2-W1)/V in gram/cm3	1.76	1.78
Average value in g/cm3	1.77	
moisture content(W)in %	28.18	28.39
Dry density of soil in g/cm3=100pb/100+w	1.37	1.39
Average value	1.380	

Bulk density & dry density determination		
Sample location	TP-15	
Sample depth(m)	3	
Trial	1	2
Internal diameter of core cutter in CM	7.5	7.5
Internal height of core cutter in CM	12.5	12.5
Volume of core cutter in(V) in CM	551.95	551.95
Weight of core cutter(W1) in gram	510	510
Weght of core cutter + soil(W2) in gram	1524	1511
Weight of soil(W2-W1) IN GRAM	1014	1001
Bulk density of soil(Pb)=(W2-W1)/V in gram/cm3	1.84	1.81
Average value in g/cm3	1.83	
moisture content(W)in %	28.18	40.52
Dry density of soil in g/cm3=100pb/100+w	1.32	1.29
Average value	1.305	

Bulk density & dry density determination		
Sample location	TP-16	
Sample depth(m)	2.7	
Trial	1	2
Internal diameter of core cutter in CM	7.5	7.5
Internal height of core cutter in CM	12.5	12.5
Volume of core cutter in(V) in CM	551.95	551.95
Weight of core cutter(W1) in gram	510	510
Weght of core cutter + soil(W2) in gram	1442	1453
Weight of soil(W2-W1) IN GRAM	932	943
Bulk density of soil(Pb)=(W2-W1)/V in gram/cm3	1.69	1.71
Average value in g/cm3	1.7	
moisture content(W)in %	33.08	33.55
Dry density of soil in g/cm3=100pb/100+w	1.27	1.28
Average value	1.275	

Table I-2 Laboratory test results of Natural Moisture content For Test Pit 3,4,13&14

LABORATORY TEST RESULTS OF NATURAL MOISTURE CONTENT

NATURAL MOISTURE CONTENT

LOCATION	TP-3			
DEPTH	3M			
no of trial	A28	14	O2	H2
weight of can	17.2	18.85	17.2	17.91
mass of wet soil + can	120.23	126.78	120.7	124
mass of dry soil + can	99.53	105.08	99.9	103
mass of dry soil	82.33	86.23	82.7	85.09
mass of water	20.7	21.7	20.8	21
moisture content	25.14	25.17	25.15	24.68
	25.15		24.92	
AVE NMC	25.03			

NATURAL MOISTURE CONTENT

LOCATION	TP-12			
DEPTH	3M			
no of trial	2	B	D	B1
weight of can	18.21	25.2	25.25	18.05
mass of wet soil + can	148.47	139.15	138.3	150.01
mass of dry soil + can	115.58	110.88	109.7	119
mass of dry soil	97.37	85.68	84.45	100.95
mass of water	32.89	28.27	28.6	31.01
moisture content	33.78	32.99	33.87	30.72
	33.39		32.29	
AVE NMC	32.84			

LOCATION	TP-4			
DEPTH	3M			
no of trial	1	2	3	N12
weight of can	25.25	18.21	25.25	15.15
mass of wet soil + can	143.06	130.08	144	136.8
mass of dry soil + can	113.32	100.56	114.1	105.91
mass of dry soil	88.07	82.35	88.85	90.76
mass of water	29.74	29.52	29.9	30.89
moisture content	33.77	35.85	33.65	34.03
	34.81		33.84	
AVE NMC	34.33			

LOCATION	TP-13			
DEPTH	3M			
no of trial	A2	N	H2	12
weight of can	18.46	17.91	18.2	18.56
mass of wet soil + can	123.65	131.87	125.5	124.33
mass of dry soil + can	99.13	105.23	100.4	99.88
mass of dry soil	80.67	87.32	82.2	81.32
mass of water	24.52	26.64	25.1	24.45
moisture content	30.40	30.51	30.54	30.07
	30.45		30.30	
AVE NMC	30.38			

Table I-3 Laboratory test results of Specific Gravity for Test pit 1, 2, 5& 6

Specific Gravity Determination				
LOCATION	TP-5	DEPTH	3M	
Pycnometer No.			P4	P3	P5
Weight of dry, clean pycnometer, w_p (g)			63.4	59.9	67.1
Weight of pycnometer + water, w_{pw} (g)			162.7	158.3	165.3
Observed temperature of water, T_i (oc)			20	17.9	18
Determination No.			1	2	3
Pycnometer No.			P4	P3	P5
Weight of pycnometer + soil + water, W_{pws} (g)			178.6	174.6	181.5
Temperature, T_x(^{0}c)			20	18	18.5
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)			162.70	158.30	165.30
Weight of dry soil , w_s (gm)			25	25	25
Conversion factor , K			1.00	1.00	1.00
Specific gravity of soil at 20°c.			2.75	2.87	2.84
Average specific gravity of soil .			2.82		

Specific Gravity Determination				
LOCATION	TP-6	DEPTH	3M	
Pycnometer No.			P5	P2	P2
Weight of dry, clean pycnometer, wp (g)			66.9	59.4	59.4
Weight of pycnometer + water, wpw (g)			166.1	158.1	158.2
Observed temperature of water, Ti (oc)			22.7	22.5	23.3
Determination No.			1	2	3
Pycnometer No.			P5	P2	P2
Weight of pycnometer + soil + water, Wpws (g)			181.90	173.60	173.80
Temperature, Tx(oc)			23.00	23.50	23.00
Wpw(atTx) (g)			166.10	158.08	158.20
Weight of dry soil , ws (gm)			25.00	25.00	25.00
Conversion factor , K			1.00	1.00	1.00
Specific gravity of soil at 20°c.			2.72	2.64	2.66
Average specific gravity of soil .			2.67		

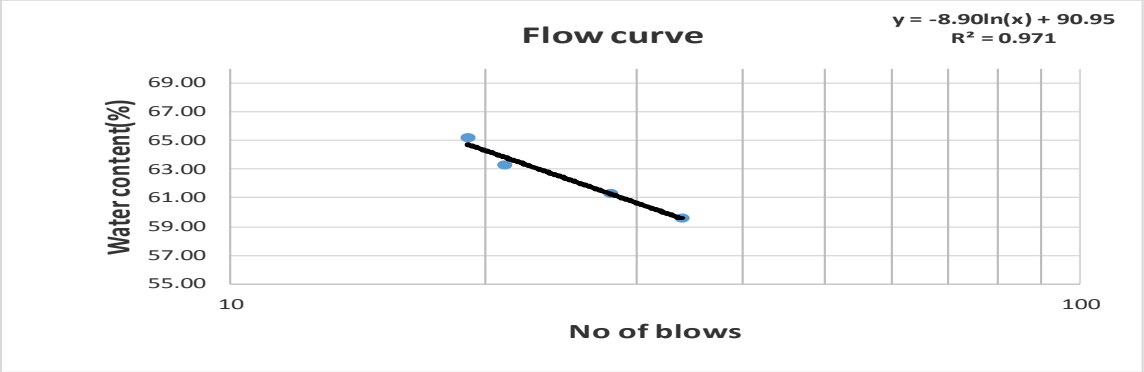
LOCATION	TP-1	DEPTH	3M	
Pycnometer No.			P5	P2	P3
Weight of dry, clean pycnometer, w_p (g)			66.9	59.4	59.9
Weight of pycnometer + water, w_{pw} (g)			163	158.9	159
Observed temperature of water, T_i (oc)			22.7	22.5	23.3
Determination No.			1	2	3
Pycnometer No.			P5	P2	P3
Weight of pycnometer + soil + water, W_{pws} (g)			178.6	174.7	175
Temperature, T_x(^{0}c)			23	23.5	23
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)			163.00	158.88	159.00
Weight of dry soil , w_s (gm)			25	25	25
Conversion factor , K			1.00	1.00	1.00
Specific gravity of soil at 20°c.			2.66	2.72	2.78
Average specific gravity of soil .			2.72		

LOCATION	TP-2	DEPTH	3M	
Pycnometer No.			P5	P3	P3
Weight of dry, clean pycnometer, w_p (g)			67	59.9	59.8
Weight of pycnometer + water, w_{pw} (g)			165.1	158.1	158.1
Observed temperature of water, T_i (oc)			19.5	19.5	18
Determination No.			1	2	3
Pycnometer No.			P5	P3	P3
Weight of pycnometer + soil + water, W_{pws} (g)			180.8	173.7	173.6
Temperature, T_x(^{0}c)			20	20	18.5
Weight of pycnometer + water at T_x , W_{pw}(atT_x) (g)			165.17	158.15	158.19
Weight of dry soil , w_s (gm)			25	25	25
Conversion factor , K			1.00	1.00	1.00
Specific gravity of soil at 20°c.			2.67	2.64	2.61
Average specific gravity of soil .			2.64		

Table I-4 Laboratory test results of Liquid Limit and Plastic Limit

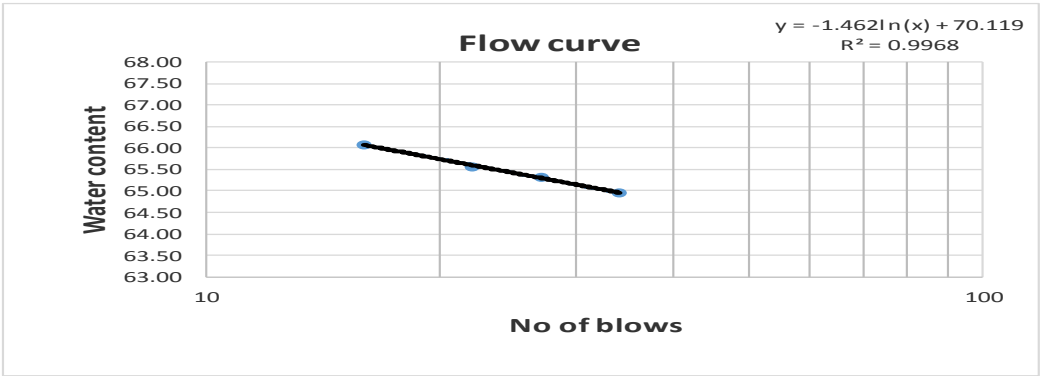
Test Pit 1	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	4	4.7	4.8	4.07	4.08	4.07
Mass of container + Wet soil, g	29.86	25.64	27.63	27.41	20.07	20.2
Mass of container + Dry soil, g	20.2	17.68	18.78	18.2	16.5	16.7
Mass of water, g	9.66	7.96	8.85	9.21	3.57	3.50
Mass of dry soil, g	16.20	12.98	13.98	14.13	12.42	12.63
Water content, %	59.63	61.33	63.30	65.18	28.74	27.71
No of blows	34	28	21	19	28.23	

Liquid Limit, % = 62 Plastic Limit = 28 PI= 34



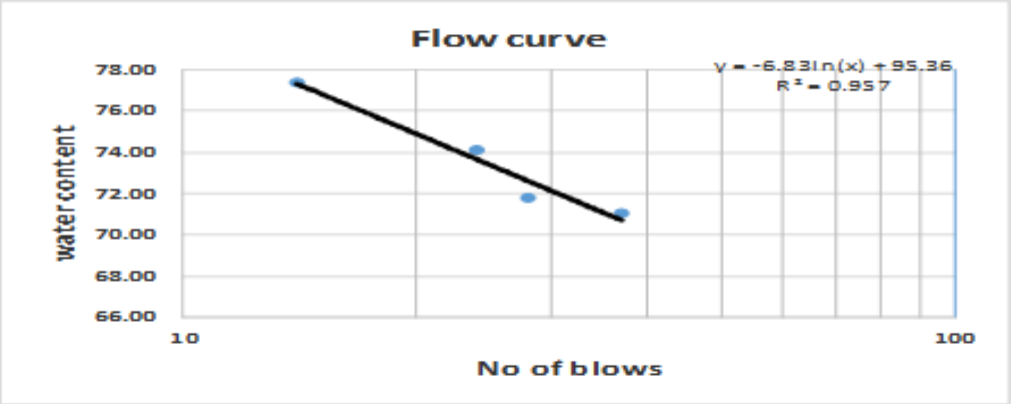
Test Pit 2	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	DD	DF
Mass of container, g	4.6	4	4.96	4.44	22.15	22.22
Mass of container + Wet soil, g	46.5	49.96	41.45	41.11	37.11	34.69
Mass of container + Dry soil, g	30	31.8	27	26.52	33.12	31.33
Mass of water, g	16.50	18.16	14.45	14.59	3.99	3.36
Mass of dry soil, g	25.40	27.80	22.04	22.08	10.97	9.11
Water content, %	64.96	65.32	65.56	66.08	36.37	36.88
No of blows	34	27	22	16	36.63	

Liquid Limit, % = 65 Plastic Limit, % = 37 PI, %= 29



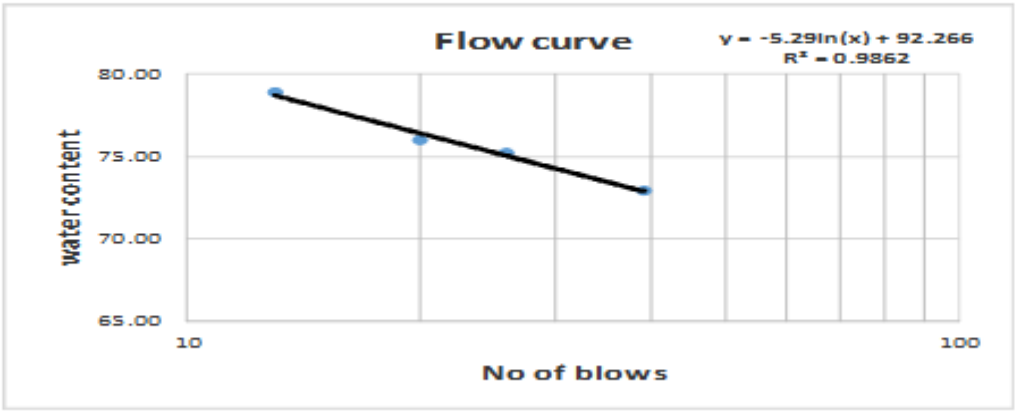
Test Pit 3	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C1	C2	C3	C4	2	7
Mass of container, g	16	15.8	15.7	15.6	22.85	22.85
Mass of container + Wet soil, g	29	30.4	30.5	30.4	37.98	37.62
Mass of container + Dry soil, g	23.6	24.3	24.2	23.9	34.5	34.1
Mass of water, g	5.40	6.10	6.30	6.50	3.48	3.52
Mass of dry soil, g	7.60	8.50	8.50	8.40	11.65	11.25
Water content, %	71.05	71.76	74.12	77.38	29.87	31.29
No of blows	37	28	24	14	30.58	

Liquid Limit, % 73 Plastic Limit, % 31 PI, % 43



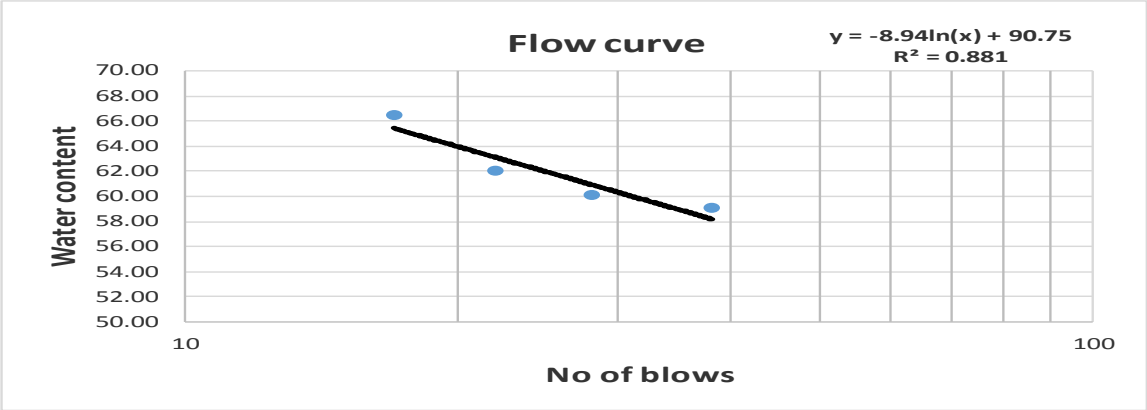
Test Pit 4	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E5	E6	E7	E8	2	7
Mass of container, g	16	15.6	15.7	15.3	4.6	4.07
Mass of container + Wet soil, g	32.6	34	33.3	28	16.99	16.6
Mass of container + Dry soil, g	25.6	26.1	25.7	22.4	14.4	13.89
Mass of water, g	7.00	7.90	7.60	5.60	2.59	2.71
Mass of dry soil, g	9.60	10.50	10.00	7.10	9.80	9.82
Water content, %	72.92	75.24	76.00	78.87	26.43	27.60
No of blows	39	26	20	13	27.01	

Liquid Limit, % = 75 Plastic Limit, % = 27 PI, %= 48



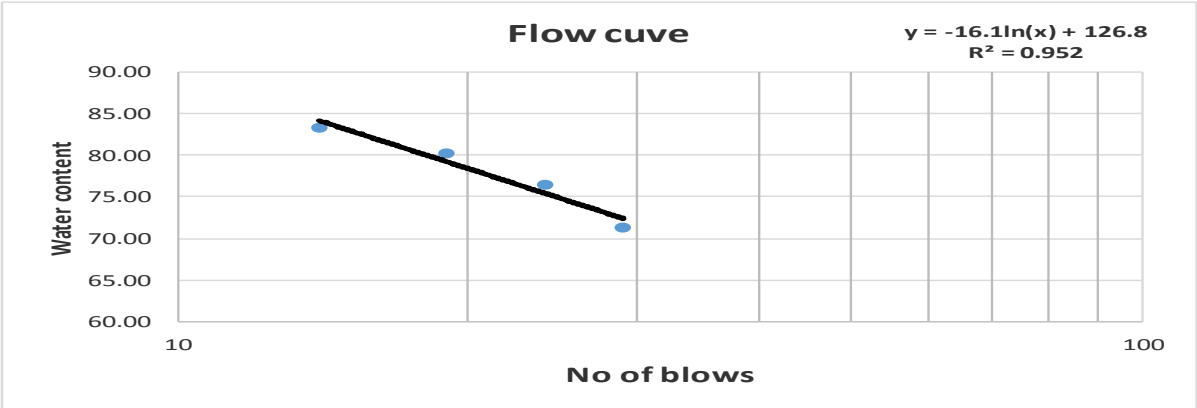
Test Pit 5	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	I	E	C	34	E	G
Mass of container, g	22.23	22.15	22.22	22.22	4.54	4.07
Mass of container + Wet soil, g	56.23	58.9	39.77	41.33	19.72	17.19
Mass of container + Dry soil, g	43.6	45.11	33.05	33.7	16.5	14.5
Mass of water, g	12.63	13.79	6.72	7.63	3.22	2.69
Mass of dry soil, g	21.37	22.96	10.83	11.48	11.96	10.43
Water content, %	59.10	60.06	62.05	66.46	26.92	25.79
No of blows	38	28	22	17	26.36	

Liquid Limit, % = 62 Plastic Limit, % = 26 PI, %= 36



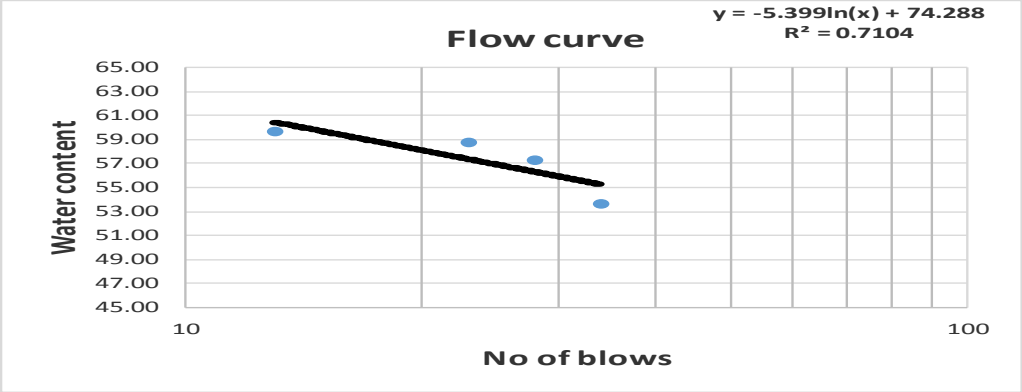
Test Pit 6	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	14.4	15.1	16	15.7	4.7	4
Mass of container + Wet soil, g	30.5	27.8	31.5	27.8	16.13	16.48
Mass of container + Dry soil, g	23.8	22.3	24.6	22.3	13.3	13.4
Mass of water, g	6.70	5.50	6.90	5.50	2.83	3.08
Mass of dry soil, g	9.40	7.20	8.60	6.60	8.60	9.40
Water content, %	71.28	76.39	80.23	83.33	32.91	32.77
No of blows	29	24	19	14	32.84	

Liquid Limit, % = 75 Plastic Limit, % = 33 PI, %= 42



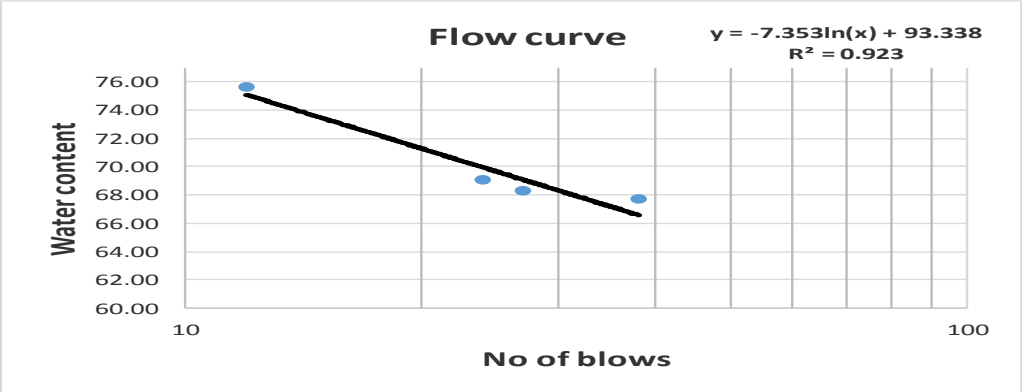
Test Pit 7	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	16	15.9	16.3	16.5	22.23	26.94
Mass of container + Wet soil, g	26.6	28.8	26.3	26.4	36.6	40.99
Mass of container + Dry soil, g	22.9	24.1	22.6	22.7	33.7	38.1
Mass of water, g	3.70	4.70	3.70	3.70	2.90	2.89
Mass of dry soil, g	6.90	8.20	6.30	6.20	11.47	11.16
Water content, %	53.62	57.32	58.73	59.68	25.28	25.90
No of blows	34	28	23	13	25.59	

Liquid Limit, % = 57 Plastic Limit, % = 26 PI, %= 31



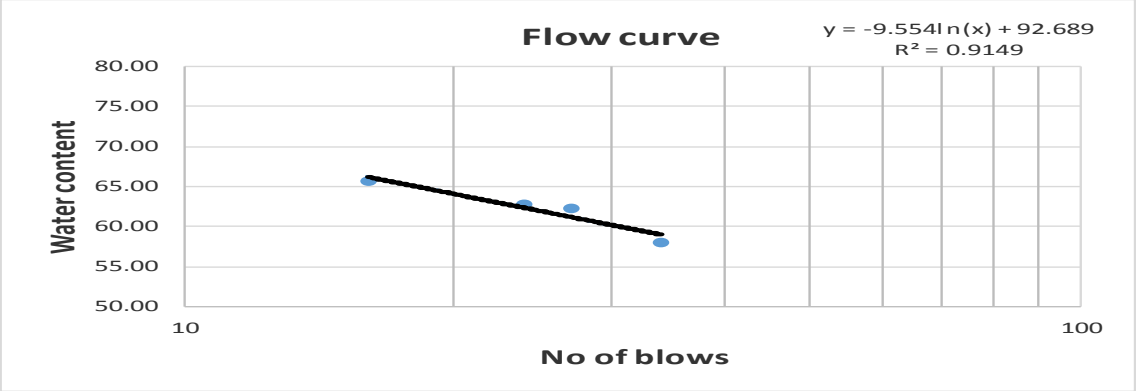
Test Pit 8	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	AA	BB	DC	GC	C	L
Mass of container, g	16.2	15.2	15.6	15.5	4.96	4.7
Mass of container + Wet soil, g	25.76	25.6	22.7	28.5	16.4	16.83
Mass of container + Dry soil, g	21.9	21.38	19.8	22.9	13.5	13.8
Mass of water, g	3.86	4.22	2.90	5.60	2.90	3.03
Mass of dry soil, g	5.70	6.18	4.20	7.40	8.54	9.10
Water content, %	67.72	68.28	69.05	75.68	33.96	33.30
No of blows	38	27	24	12	33.63	

Liquid Limit, % = 70 Plastic Limit, % = 34 PI, %= 36



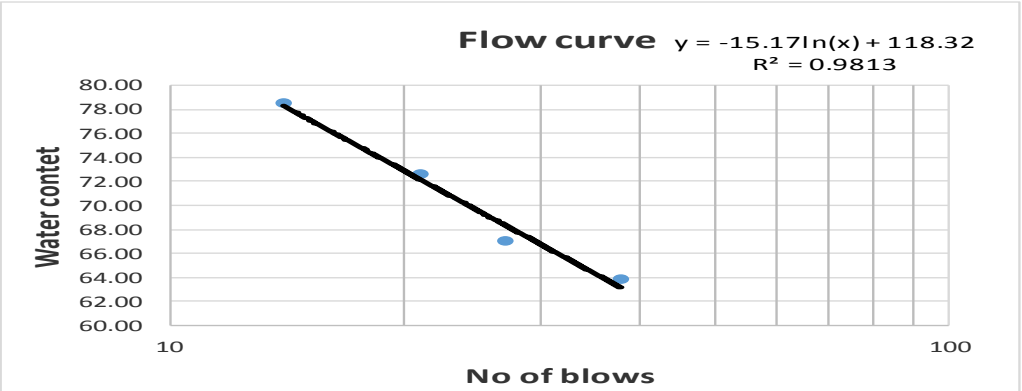
Test Pit 9	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	A2	A22
Mass of container, g	15.4	15.9	17.2	15.8	4.1	4.07
Mass of container + Wet soil, g	28.5	23.2	29.9	28.39	16.56	16.97
Mass of container + Dry soil, g	23.69	20.4	25	23.4	13.23	13.6
Mass of water, g	4.81	2.80	4.90	4.99	3.33	3.37
Mass of dry soil, g	8.29	4.50	7.80	7.60	9.13	9.53
Water content, %	58.02	62.22	62.82	65.66	36.47	35.36
No of blows	34	27	24	16	35.92	

Liquid Limit, % = 62 Plastic Limit, % = 36 PI, %= 26



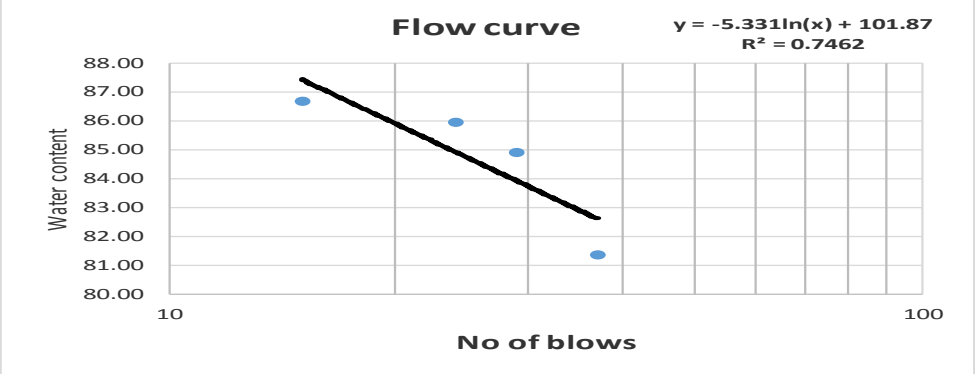
Test Pit 10	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	A2	A23	A22	A24	M	K
Mass of container, g	15.8	16	15.9	16.3	4.1	4.96
Mass of container + Wet soil, g	33	30.7	28.5	27.9	18.57	18.12
Mass of container + Dry soil, g	26.3	24.8	23.2	22.8	15.48	15.29
Mass of water, g	6.70	5.90	5.30	5.10	3.09	2.83
Mass of dry soil, g	10.50	8.80	7.30	6.50	11.38	10.33
Water content, %	63.81	67.05	72.60	78.46	27.15	27.40
No of blows	38	27	21	14	27.27	

Liquid Limit, % = 71 Plastic Limit, % = 27 PI, %= 44



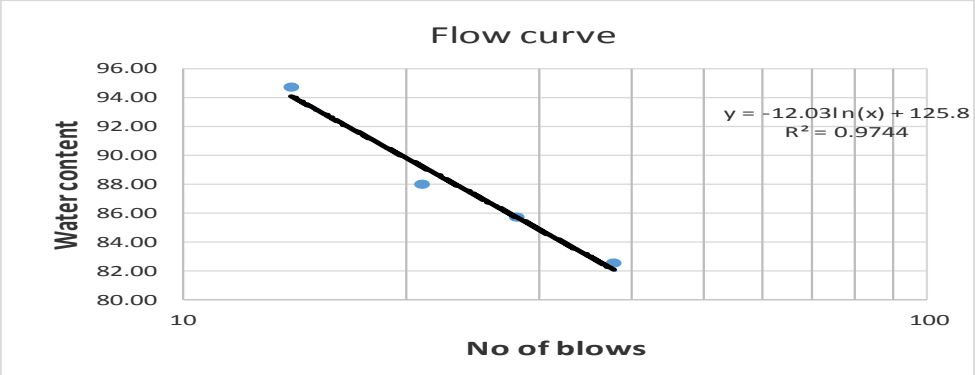
Test Pit 11	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	AB	BB	C2	D64	C	L
Mass of container, g	15.9	15.4	15.6	15.5	4.1	4.15
Mass of container + Wet soil, g	26.6	25.2	26.2	26.7	19	17.43
Mass of container + Dry soil, g	21.8	20.7	21.3	21.5	14.8	13.99
Mass of water, g	4.80	4.50	4.90	5.20	4.20	3.44
Mass of dry soil, g	5.90	5.30	5.70	6.00	10.70	9.84
Water content, %	81.36	84.91	85.96	86.67	39.25	34.96
No of blows	37	29	24	15	37.106	

Liquid Limit, % = 85 Plastic Limit, % = 37 PI, %= 48



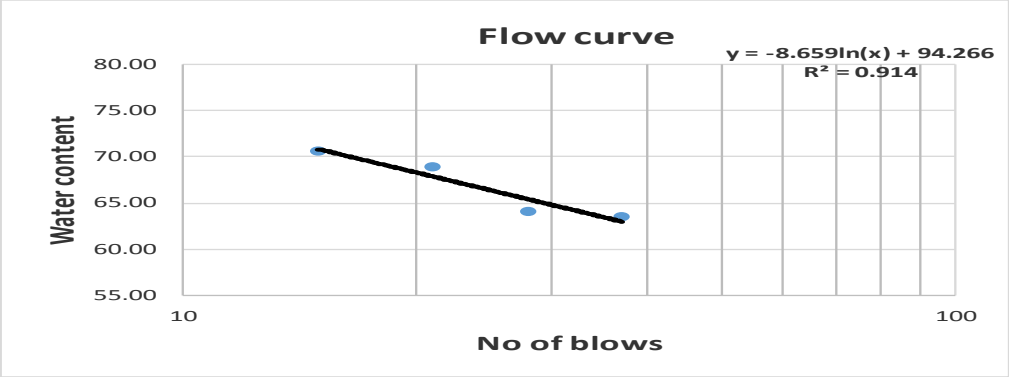
Test Pit 12	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	D1	D2	D3	D4	A12	A32
Mass of container, g	15.6	16	16	15.7	22.15	22.22
Mass of container + Wet soil, g	30.2	31.6	30.1	30.5	37.17	34.92
Mass of container + Dry soil, g	23.6	24.4	23.5	23.3	32.5	30.8
Mass of water, g	6.60	7.20	6.60	7.20	4.67	4.12
Mass of dry soil, g	8.00	8.40	7.50	7.60	10.35	8.58
Water content, %	82.50	85.71	88.00	94.74	45.12	48.02
No of blows	38	28	21	14	46.57	

Liquid Limit, % = 87 Plastic Limit, % = 47 PI, %= 41



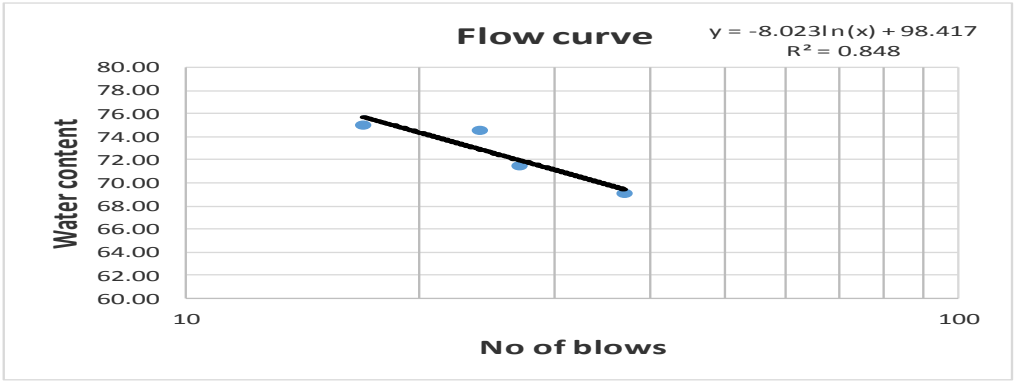
Test Pit 13	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	16	14.2	15.9	16	27.54	19.71
Mass of container + Wet soil, g	31.7	31.1	32.1	30.5	41.99	32.42
Mass of container + Dry soil, g	25.6	24.5	25.49	24.5	38.8	29.6
Mass of water, g	6.10	6.60	6.61	6.00	3.19	2.82
Mass of dry soil, g	9.60	10.30	9.59	8.50	11.26	9.89
Water content, %	63.54	64.08	68.93	70.59	28.33	28.51
No of blows	37	28	21	15	28.42	

Liquid Limit, % = 66 Plastic Limit, % = 28 PI, %= 38



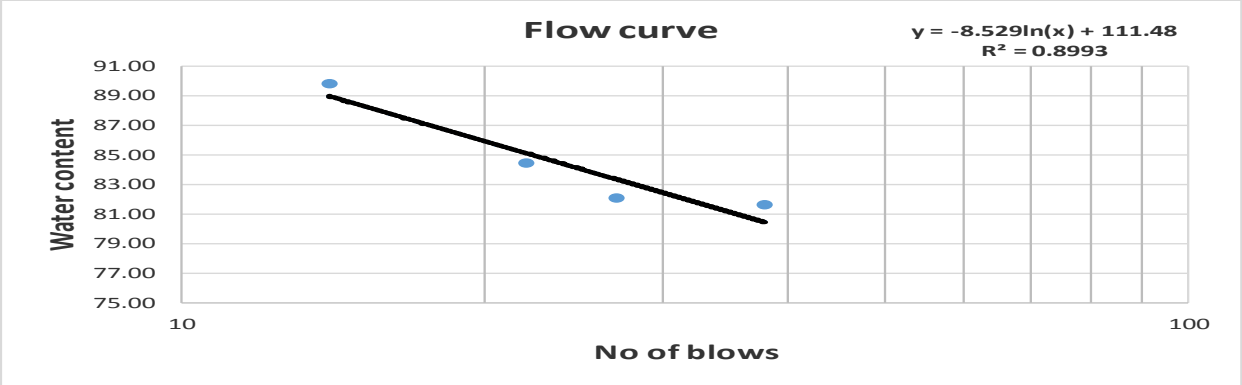
Test Pit 14	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	15.6	15	14.5	15.8	17.21	19.8
Mass of container + Wet soil, g	22.7	24.6	24.1	25.6	38.96	38.17
Mass of container + Dry soil, g	19.8	20.6	20	21.4	34.1	34.17
Mass of water, g	2.90	4.00	4.10	4.20	4.86	4.00
Mass of dry soil, g	4.20	5.60	5.50	5.60	16.89	14.37
Water content, %	69.05	71.43	74.55	75.00	28.77	27.84
No of blows	37	27	24	17	28.31	

Liquid Limit, % = 73 Plastic Limit, % = 28 PI, %= 44



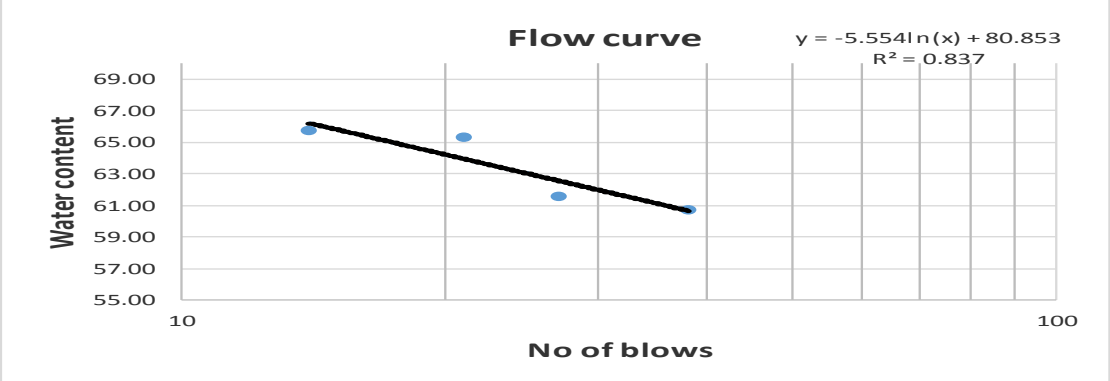
Test Pit 15	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	15.4	15.7	15.8	15.8	26.6	21.77
Mass of container + Wet soil, g	24.3	22.8	24.1	25.1	37	32.82
Mass of container + Dry soil, g	20.3	19.6	20.3	20.7	34.38	30.07
Mass of water, g	4.00	3.20	3.80	4.40	2.62	2.75
Mass of dry soil, g	4.90	3.90	4.50	4.90	7.78	8.30
Water content, %	81.63	82.05	84.44	89.80	33.68	33.13
No of blows	38	27	22	14	33.40	

Liquid Limit, % = 84 Plastic Limit, % = 33 PI, %= 51



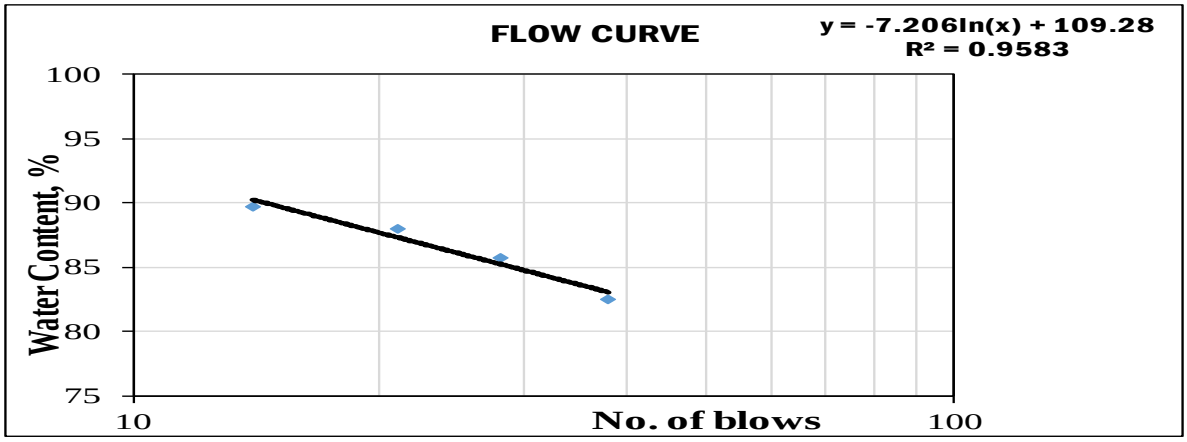
Test Pit 16	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	15.8	16.1	16.1	16.3	22.23	26.87
Mass of container + Wet soil, g	33	30.8	28.5	27.9	37.56	42.24
Mass of container + Dry soil, g	26.5	25.2	23.6	23.3	33.97	38.74
Mass of water, g	6.50	5.60	4.90	4.60	3.59	3.50
Mass of dry soil, g	10.70	9.10	7.50	7.00	11.74	11.87
Water content, %	60.75	61.54	65.33	65.71	30.58	29.49
No of blows	38	27	21	14	30.03	

Liquid Limit, % = 63 Plastic Limit, % = 30 PI, %= 33



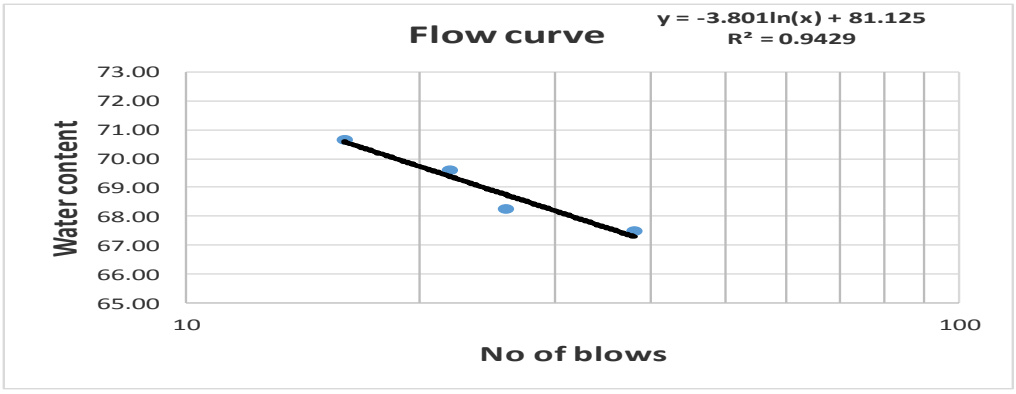
Test Pit 17	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	BB	A12
Mass of container, g	15.6	16	16	15.7	4.08	22.23
Mass of container + Wet soil, g	30.2	31.6	30.1	30.5	18.14	32.82
Mass of container + Dry soil, g	23.6	24.4	23.5	23.5	14.21	30.07
Mass of water, g	6.60	7.20	6.60	7.00	3.93	2.75
Mass of dry soil, g	8.00	8.40	7.50	7.80	10.13	7.84
Water content, %	82.50	85.71	88.00	89.74	38.80	35.08
No of blows	38	28	21	14	36.94	

Liquid Limit, % = 86 Plastic Limit, % = 37 PI, %= 49



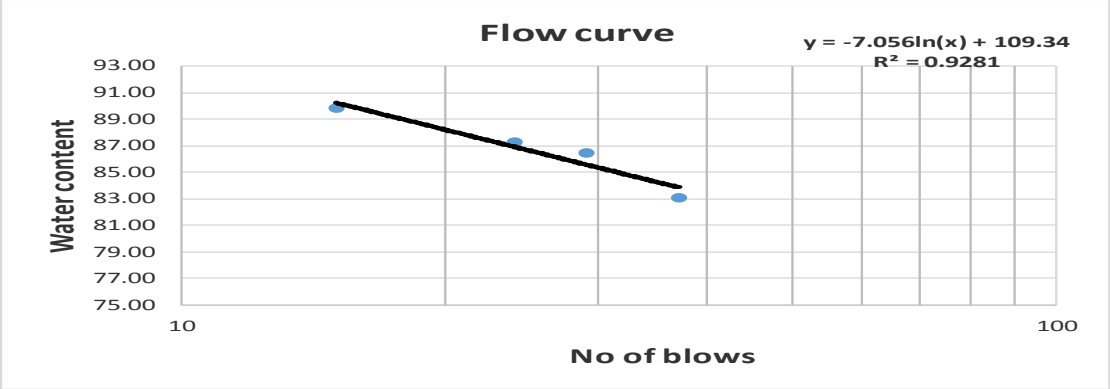
Test Pit 18	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	4.15	4	4.7	4.07	4.85	4.54
Mass of container + Wet soil, g	48.32	47.76	37.43	46.96	19.91	19.68
Mass of container + Dry soil, g	30.52	30.01	24	29.2	16.7	16.4
Mass of water, g	17.80	17.75	13.43	17.76	3.21	3.28
Mass of dry soil, g	26.37	26.01	19.30	25.13	11.85	11.86
Water content, %	67.50	68.24	69.59	70.67	27.09	27.66
No of blows	38	26	22	16	27.37	

Liquid Limit, % = 69 Plastic Limit, % = 27 PI, %= 42



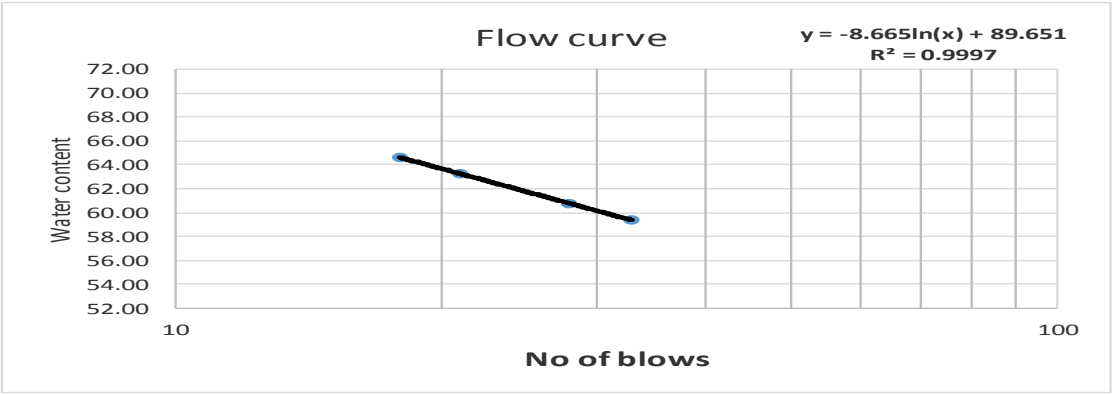
Test Pit 19	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	C	L
Mass of container, g	15.9	15.4	15.6	15.5	4.1	4.15
Mass of container + Wet soil, g	26.7	25.3	26.2	26.7	18.78	17.43
Mass of container + Dry soil, g	21.8	20.71	21.26	21.4	14.8	13.7
Mass of water, g	4.90	4.59	4.94	5.30	3.98	3.73
Mass of dry soil, g	5.90	5.31	5.66	5.90	10.70	9.55
Water content, %	83.05	86.44	87.28	89.83	37.20	39.06
No of blows	37	29	24	15	38.127	

Liquid Limit, % = 87 Plastic Limit, % = 38 PI, %= 49



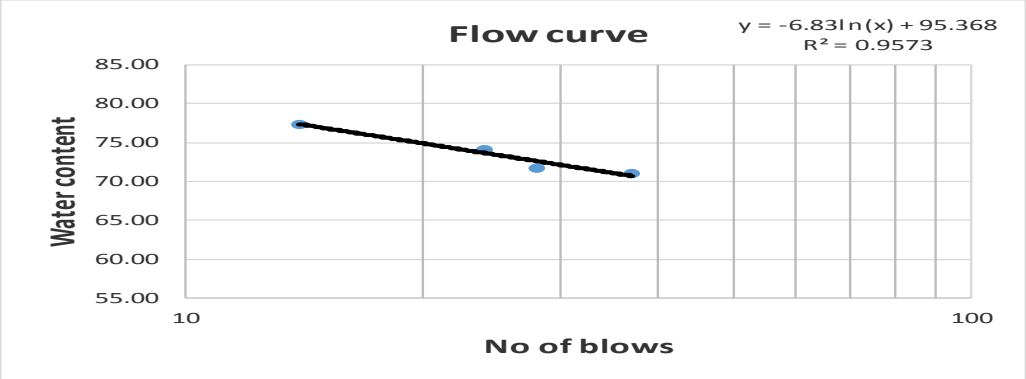
Test Pit 20	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	A12	B22
Mass of container, g	4.85	4.1	4.96	4.07	4.08	4.85
Mass of container + Wet soil, g	35.37	31.27	26.08	29.64	16.5	16.9
Mass of container + Dry soil, g	24	21	17.9	19.6	13.4	13.9
Mass of water, g	11.37	10.27	8.18	10.04	3.10	3.00
Mass of dry soil, g	19.15	16.90	12.94	15.53	9.32	9.05
Water content, %	59.37	60.77	63.21	64.65	33.26	33.15
No of blows	33	28	21	18	33.21	

Liquid Limit, % = 62 Plastic Limit, % = 33 PI, %= 29



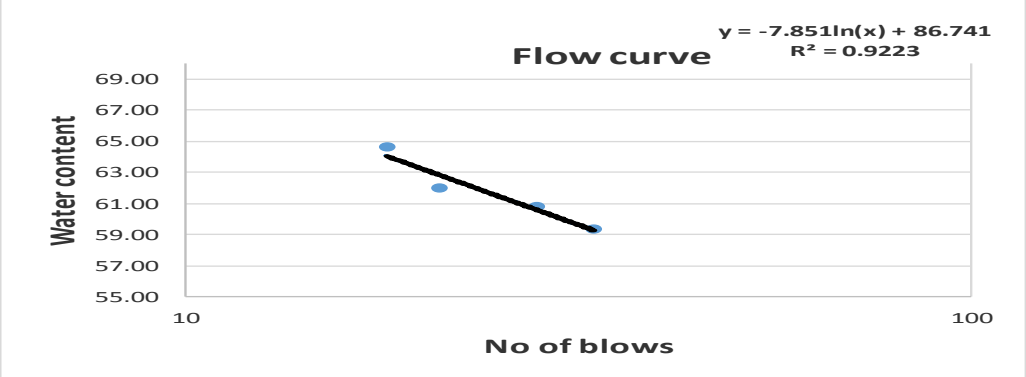
Test Pit 21	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	C1	C2	C3	C4	C	L
Mass of container, g	16	15.8	15.7	15.6	26.6	21.77
Mass of container + Wet soil, g	29	30.4	30.5	30.4	37	32.82
Mass of container + Dry soil, g	23.6	24.3	24.2	23.9	34.38	30.07
Mass of water, g	5.40	6.10	6.30	6.50	2.62	2.75
Mass of dry soil, g	7.60	8.50	8.50	8.40	7.78	8.30
Water content, %	71.05	71.76	74.12	77.38	33.68	33.13
No of blows	37	28	24	14	33.40	

Liquid Limit, % = 73 Plastic Limit, % = 33 PI, %= 40



Test Pit 22	Sample Depth				3M	
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	E1	E2	E3	E4	B2	B1
Mass of container, g	4.85	4.1	4.96	4.07	4.08	4.85
Mass of container + Wet soil, g	35.37	31.27	26.08	29.64	16.5	16.9
Mass of container + Dry soil, g	24	21	18	19.6	13.4	14
Mass of water, g	11.37	10.27	8.08	10.04	3.10	2.90
Mass of dry soil, g	19.15	16.90	13.04	15.53	9.32	9.15
Water content, %	59.37	60.77	61.96	64.65	33.26	31.69
No of blows	33	28	21	18	32.48	

Liquid Limit, % = 61 Plastic Limit, % = 32 PI, %= 29



ONE DIMENTIONAL CONSOLIDATION TEST RESULT

Table I-5 Laboratory test results of One Dimensional Consolidation

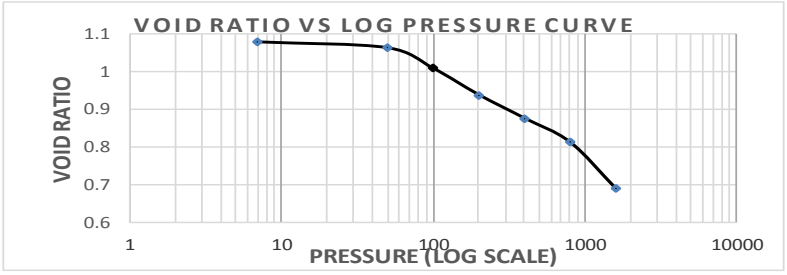
One Dimension Consolidation Test

Test Pit 1							
		50	100	200	400	800	1600
Time(min.)	√time	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]
0	0.00	3.452	3.60	4.12	4.80	5.40	6.014
0.1	0.32	3.256	3.664	4.304	5.000	5.610	6.124
0.25	0.50	3.274	3.668	4.316	5.024	5.620	6.246
0.5	0.71	3.302	3.684	4.330	5.050	5.644	6.286
1	1.00	3.340	3.704	4.352	5.084	5.684	6.486
2	1.41	3.372	3.756	4.380	5.122	5.712	6.488
4	2.00	3.406	3.790	4.430	5.160	5.714	6.520
8	2.83	3.434	3.792	4.472	5.220	5.726	6.540
15	3.87	3.454	3.832	4.496	5.240	5.728	6.600
30	5.48	3.482	3.858	4.526	5.260	5.746	6.680
60	7.75	3.520	3.898	4.592	5.284	5.758	6.730
120	10.95	3.540	3.912	4.642	5.320	5.786	6.742
240	15.49	3.562	3.977	4.682	5.342	5.846	6.754
480	21.91	3.574	4.088	4.752	5.346	5.866	6.898
1440	37.95	3.600	4.120	4.800	5.400	6.014	7.100

Pit no	1	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Black

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	101.67
Ring Area,cm^2	44.16	Final Moisture Content,%	34.56
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.63
Seating Load,kpa	7	Final Void Ratio,e_f	0.69
Initial Void Ratio,e_o	1.08	Specific gravity	2.72
initial moisture content,%	32.66	Dry Density,g/cm^3	1.151
wet density,g/cm^3	1.20	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.08
50	1.06
100	1.01
200	0.94
400	0.87
800	0.81
1600	0.69



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.209

e_A =	0.936	P_A =	200
e_B =	0.81	P_B =	800

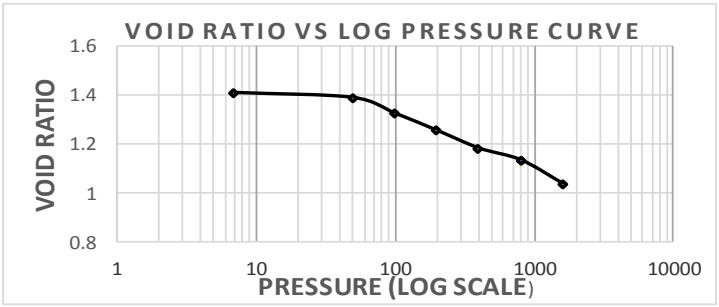
One Dimension Consolidation Test

Test Pit 2

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.290	4.410	4.741	5.200	5.800	6.300
0.1	0.32	4.326	4.540	4.940	5.320	5.810	6.446
0.25	0.50	4.330	4.550	4.946	5.334	5.824	6.468
0.5	0.71	4.336	4.560	4.964	5.346	5.838	6.472
1	1.00	4.338	4.660	4.982	5.486	5.842	6.488
2	1.41	4.346	4.582	5.002	5.520	5.874	6.514
4	2.00	4.354	4.600	5.022	5.546	5.904	6.546
8	2.83	4.356	4.618	5.040	5.584	5.936	6.584
15	3.87	4.358	4.634	5.066	5.622	5.996	6.624
30	5.48	4.364	4.660	5.102	5.746	6.112	6.676
60	7.75	4.366	4.674	5.114	5.748	6.246	6.748
120	10.95	4.370	4.684	5.124	5.752	6.266	6.812
240	15.49	4.386	4.688	5.145	5.774	6.278	6.824
480	21.91	4.400	4.696	5.176	5.778	6.284	6.846
1440	37.95	4.410	4.741	5.200	5.800	6.300	6.900

Pit no	2	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Grey
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	105.49
Ring Area,cm^2	44.16	Final Moisture Content,%	31.04
Diameter of Ring,mm	75	Height of Solid (Hs),mm	8.91
Seating Load,kpa	7	Final Void Ratio,e_f	0.95
Initial Void Ratio,e_o	1.24	Specific gravity	2.64
initial moisture content,%	25.03	Dry Density,g/cm^3	1.19
wet density,g/cm^3	1.77	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.41
50	1.39
100	1.33
200	1.26
400	1.18
800	1.14
1600	1.04



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.211

e_A =	1.14	P_A =	200
e_B =	0.95	P_B =	1600

One Dimension Consolidation Test

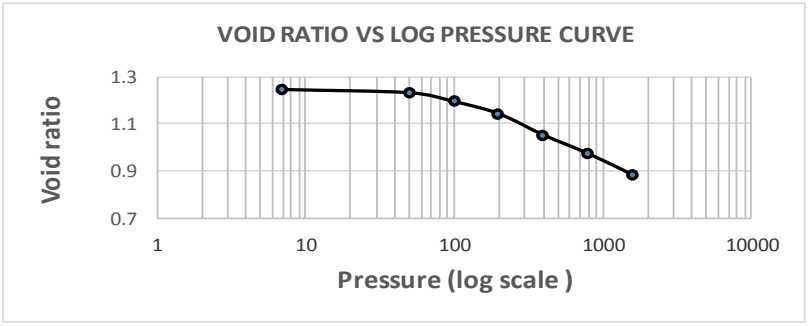
Test Pit 3

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	2.120	2.280	2.600	3.000	3.600	4.500
0.1	0.32	2.128	2.324	2.724	3.224	3.730	4.660
0.25	0.50	2.138	2.380	2.728	3.238	3.740	4.708
0.5	0.71	2.140	2.390	2.730	3.246	3.842	4.724
1	1.00	2.142	2.394	2.748	3.268	3.960	4.742
2	1.41	2.152	2.404	2.768	3.292	4.010	4.762
4	2.00	2.168	2.414	2.790	3.320	4.138	4.786
8	2.83	2.180	2.426	2.812	3.356	4.152	4.820
15	3.87	2.192	2.434	2.834	3.386	4.252	4.866
30	5.48	2.210	2.448	2.860	3.428	4.336	4.948
60	7.75	2.228	2.468	2.870	3.486	4.372	5.124
120	10.95	2.246	2.484	2.886	3.532	4.412	5.248
240	15.49	2.268	2.558	2.904	3.541	4.446	5.382
480	21.91	2.272	2.562	2.916	3.574	4.464	5.484
1440	37.95	2.280	2.600	3.000	3.600	4.500	5.600

Pit no	3	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	43
Ring Area,cm^2	19.63	Final Moisture Content,%	20.09
Diameter of Ring,mm	50	Height of Solid (Hs),mm	8.3
Seating Load,kpa	7	Final Void Ratio,e_f	0.9
Initial Void Ratio,e_o	1.24	Specific gravity	2.68
initial moisture content,%	28.28	Dry Density,g/cm^3	1.09
wet density,g/cm^3	1.77	Volume of Ring,Cm^3	39.29

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.24
50	1.23
100	1.19
200	1.14
400	1.05
800	0.97
1600	0.90



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.265

e_A =	1.35	P_A =	100
e_B =	1.11	P_B =	800

One Dimension Consolidation Test

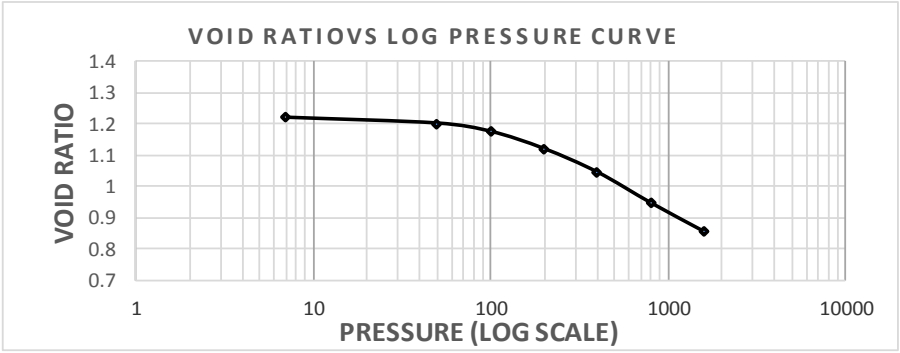
Test Pit 4

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	2.128	2.294	2.530	3.024	3.698	4.576
0.1	0.32	2.138	2.324	2.724	3.224	3.830	4.660
0.25	0.50	2.148	2.380	2.728	3.238	3.878	4.708
0.5	0.71	2.154	2.390	2.730	3.246	3.942	4.724
1	1.00	2.156	2.394	2.748	3.268	3.960	4.742
2	1.41	2.158	2.404	2.768	3.292	4.010	4.762
4	2.00	2.168	2.414	2.790	3.320	4.038	4.786
8	2.83	2.180	2.426	2.812	3.356	4.052	4.820
15	3.87	2.192	2.434	2.834	3.386	4.082	4.866
30	5.48	2.210	2.448	2.860	3.428	4.136	4.948
60	7.75	2.228	2.468	2.890	3.486	4.212	5.024
120	10.95	2.246	2.484	2.926	3.532	4.312	5.148
240	15.49	2.268	2.510	2.964	3.610	4.446	5.182
480	21.91	2.274	2.530	3.024	3.648	4.484	5.202
1440	37.95	2.294	2.534	3.024	3.698	4.576	5.420

Pit no	4	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	108.67
Ring Area,cm^2	44.16	Final Moisture Content,%	27.03
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.01
Seating Load,kpa	7	Final Void Ratio,e_f	0.86
Initial Void Ratio,e_o	1.22	Specific gravity	2.73
initial moisture content,%	34.33	Dry Density,g/cm^3	1.23
wet density,g/cm^3	1.61	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.22
50	1.20
100	1.17
200	1.12
400	1.04
800	0.95
1600	0.86



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.251

e_A =	1.17	P_A =	100
e_B =	0.95	P_B =	800

One Dimension Consolidation Test

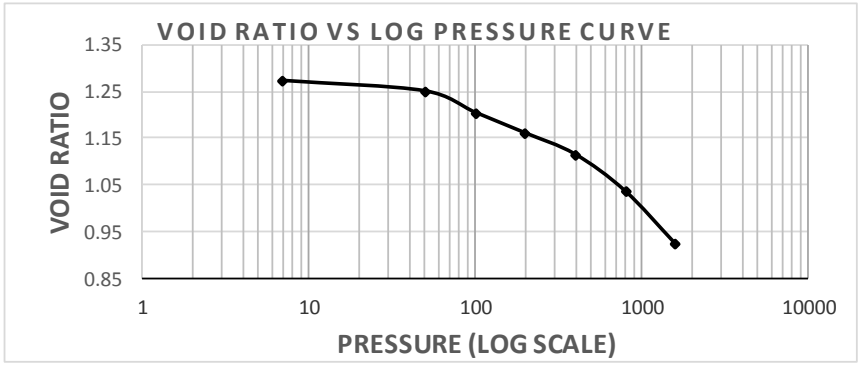
Test Pit 5

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.818	4.012	4.412	4.800	5.200	5.890
0.1	0.32	3.85	4.172	4.520	4.844	5.482	6.428
0.25	0.50	3.870	4.184	4.584	4.854	5.492	6.442
0.5	0.71	3.872	4.188	4.626	4.856	5.502	6.462
1	1.00	3.884	4.190	4.628	4.858	5.504	6.490
2	1.41	3.894	4.192	4.630	4.860	5.506	6.524
4	2.00	3.91	4.202	4.634	4.862	5.508	6.564
8	2.83	3.922	4.265	4.636	4.866	5.512	6.612
15	3.87	3.930	4.288	4.678	4.870	5.546	6.656
30	5.48	3.944	4.312	4.712	4.892	5.574	6.706
60	7.75	3.96	4.322	4.742	4.946	5.626	6.754
120	10.95	3.974	4.342	4.744	4.978	5.686	6.796
240	15.49	4.002	4.356	4.746	5.000	5.773	6.832
480	21.91	4.010	4.366	4.768	5.124	5.808	6.862
1440	37.95	4.012	4.412	4.800	5.200	5.890	6.880

Pit no	5	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	48.9
Ring Area,cm^2	19.63	Final Moisture Content,%	31.65
Diameter of Ring,mm	50	Height of Solid (Hs),mm	8.8
Seating Load,kpa	7	Final Void Ratio,e_f	0.92
Initial Void Ratio,e_o	1.27	Specific gravity	2.77
initial moisture content,%	28.11	Dry Density,g/cm^3	1.25
wet density,g/cm^3	1.4	Volume of ring, Cm^3	39.26

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.27
50	1.25
100	1.20
200	1.16
400	1.12
800	1.04
1600	0.92



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.206

e_A =	1.16	P_A =	200
e_B =	1.04	P_B =	800

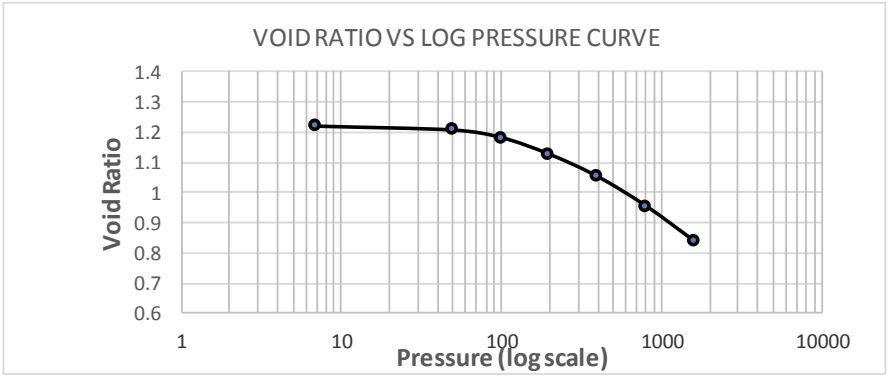
One Dimension Consolidation Test

Test Pit 6

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1.192	1.308	1.534	2.034	2.700	3.577
0.1	0.32	1.118	1.324	1.724	2.224	2.930	3.660
0.25	0.50	1.128	1.380	1.728	2.238	2.940	3.708
0.5	0.71	1.134	1.390	1.730	2.246	2.942	3.724
1	1.00	1.142	1.394	1.748	2.268	2.960	3.742
2	1.41	1.152	1.404	1.768	2.292	3.010	3.762
4	2.00	1.168	1.414	1.790	2.320	3.038	3.786
8	2.83	1.180	1.426	1.812	2.356	3.052	3.820
15	3.87	1.192	1.434	1.834	2.386	3.082	3.866
30	5.48	1.210	1.448	1.860	2.428	3.136	3.948
60	7.75	1.228	1.468	1.890	2.486	3.212	4.024
120	10.95	1.246	1.484	1.926	2.532	3.312	4.148
240	15.49	1.268	1.510	1.964	2.610	3.446	4.282
480	21.91	1.282	1.522	2.018	2.684	3.514	4.384
1440	37.95	1.308	1.534	2.034	2.700	3.577	4.620

Pit no	6	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	105.49
Ring Area,cm^2	44.16	Final Moisture Content,%	28.69
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.01
Seating Load,kpa	7	Final Void Ratio,e_f	0.84
Initial Void Ratio,e_o	1.22	Specific gravity	2.65
initial moisture content,%	31	Dry Density,g/cm^3	1.195
wet density,g/cm^3	1.54	Volume of Ring ,Cm^3	88.3

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.22
50	1.21
100	1.18
200	1.13
400	1.05
800	0.95
1600	0.84



$Cc=(e_A-e_B)/\log(P_B-P_A)$ **Cc= 0.245**

e_A =	1.18	P_A =	200
e_B =	1.05	P_B =	400

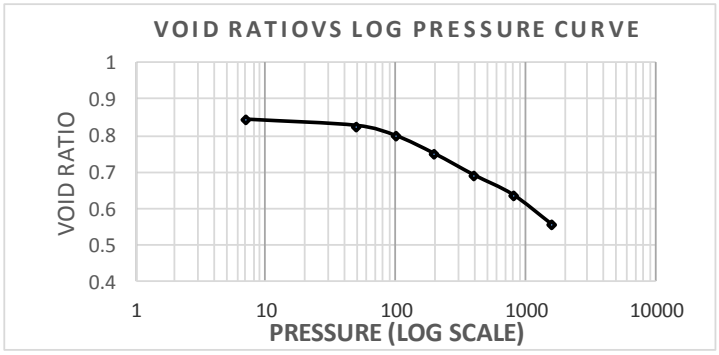
One Dimension Consolidation Test

Test Pit 7

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.898	4.082	4.380	4.916	5.540	6.120
0.1	0.32	3.800	4.142	4.460	5.144	5.736	6.450
0.25	0.50	3.810	4.146	4.500	5.168	5.744	6.464
0.5	0.71	3.820	4.156	4.598	5.182	5.762	6.490
1	1.00	3.838	4.170	4.600	5.208	5.786	6.500
2	1.41	3.856	4.186	4.624	5.238	5.810	6.520
4	2.00	3.720	4.202	4.644	5.272	5.838	6.570
8	2.83	3.904	4.226	4.672	5.302	5.872	6.612
15	3.87	3.988	4.246	4.704	5.334	5.900	6.648
30	5.48	3.990	4.268	4.744	5.364	5.936	6.686
60	7.75	4.000	4.286	4.790	5.412	5.978	6.772
120	10.95	4.004	4.310	4.836	5.448	6.022	6.868
240	15.49	4.016	4.338	4.863	5.482	6.064	6.938
480	21.91	4.024	4.366	4.890	5.500	6.099	6.994
1440	37.95	4.082	4.380	4.916	5.540	6.120	7.000

Pit no	7	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Brown
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	115.97
Ring Area,cm^2	44.16	Final Moisture Content,%	26
Diameter of Ring,mm	75	Height of Solid (Hs),mm	10.85
Seating Load,kpa	7	Final Void Ratio,e_f	0.56
Initial Void Ratio,e_o	0.84	Specific gravity	2.63
initial moisture content,%	25.8	Dry Density,g/cm^3	1.31
wet density,g/cm^3	1.44	Volume of Ring ,Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	0.84
50	0.83
100	0.80
200	0.75
400	0.69
800	0.64
1600	0.56



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.178

e_A =	0.8	P_A =	100
e_B =	0.64	P_B =	800

One Dimension Consolidation Test

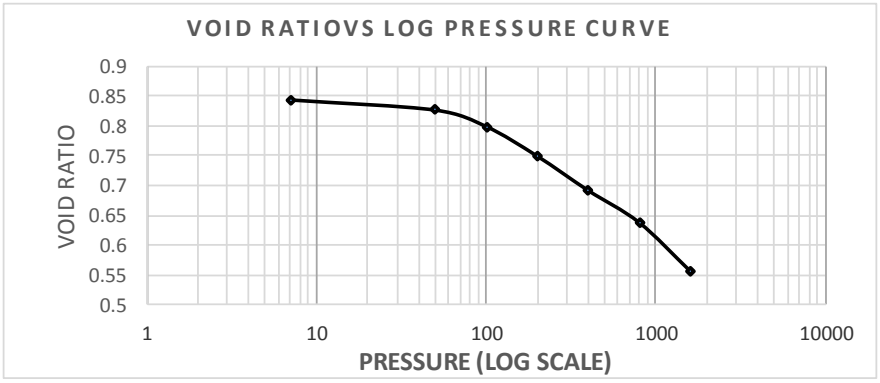
Test Pit 8

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.128	3.288	3.526	4.020	4.596	5.566
0.1	0.32	3.118	3.324	3.724	4.224	4.630	5.660
0.25	0.50	3.128	3.380	3.728	4.238	4.670	5.708
0.5	0.71	3.134	3.390	3.730	4.246	4.742	5.724
1	1.00	3.142	3.394	3.748	4.268	4.760	5.742
2	1.41	3.152	3.404	3.768	4.292	4.812	5.762
4	2.00	3.168	3.414	3.790	4.324	4.856	5.786
8	2.83	3.180	3.426	3.812	4.356	4.952	5.820
15	3.87	3.192	3.434	3.834	4.386	5.082	5.866
30	5.48	3.210	3.448	3.860	4.428	5.136	5.948
60	7.75	3.228	3.468	3.890	4.486	5.212	6.024
120	10.95	3.246	3.484	3.926	4.532	5.312	6.146
240	15.49	3.268	3.510	3.964	4.614	5.446	6.184
480	21.91	3.282	3.522	4.018	4.682	5.514	6.286
1440	37.95	3.288	3.526	4.020	4.596	5.566	6.394

Pit no	8	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	108.67
Ring Area,cm^2	44.16	Final Moisture Content,%	27.03
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.36
Seating Load,kpa	7	Final Void Ratio,e_f	0.79
Initial Void Ratio,e_o	1.14	Specific gravity	2.63
initial moisture content,%	31.02	Dry Density,g/cm^3	1.23
wet density,g/cm^3	1.67	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.14
50	1.12
100	1.09
200	1.04
400	0.98
800	0.88
1600	0.79



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.241

e_A =	1.09	P_A =	100
e_B =	0.88	P_B =	800

One Dimension Consolidation Test

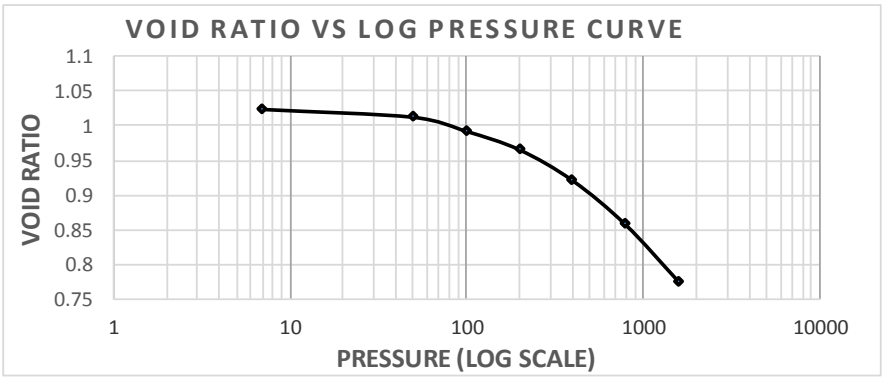
Test Pit 9

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.000	4.604	4.796	4.916	5.180	5.620	6.252
0.1	0.320	4.68	4.830	4.990	5.290	5.740	6.400
0.25	0.500	4.684	4.832	4.998	5.302	5.760	6.420
0.5	0.707	4.692	4.834	5.000	5.316	5.780	6.436
1	1.000	4.700	4.838	5.006	5.332	5.808	6.460
2	1.414	4.704	4.840	5.020	5.350	5.840	6.496
4	2.000	4.72	4.844	5.030	5.372	5.850	6.530
8	2.828	4.722	4.852	5.044	5.396	5.902	6.582
15	3.873	4.726	4.860	5.060	5.420	5.952	6.678
30	5.477	4.734	4.868	5.078	5.448	6.010	6.750
60	7.746	4.742	4.878	5.094	5.484	6.014	6.824
120	10.954	4.772	4.886	5.112	5.516	6.126	6.910
240	15.492	4.78	4.894	5.130	5.548	6.176	6.984
480	21.909	4.786	4.908	5.148	5.570	6.220	7.028
1440	37.947	4.796	4.916	5.180	5.620	6.252	7.068

Pit no	9	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	51.2
Ring Area,cm^2	19.63	Final Moisture Content,%	36.72
Diameter of Ring,mm	50	Height of Solid (Hs),mm	9.88
Seating Load,kpa	7	Final Void Ratio,e_f	0.77
Initial Void Ratio,e_o	1.02	Specific gravity	2.64
initial moisture content,%	29.14	Dry Density,g/cm^3	1.304
wet density,g/cm^3	1.57	Volume of ring, Cm^3	39.26

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.02
50	1.01
100	0.99
200	0.97
400	0.92
800	0.86
1600	0.77



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.180

e_A =	0.97	P_A =	200
e_B =	0.86	P_B =	400

One Dimension Consolidation Test

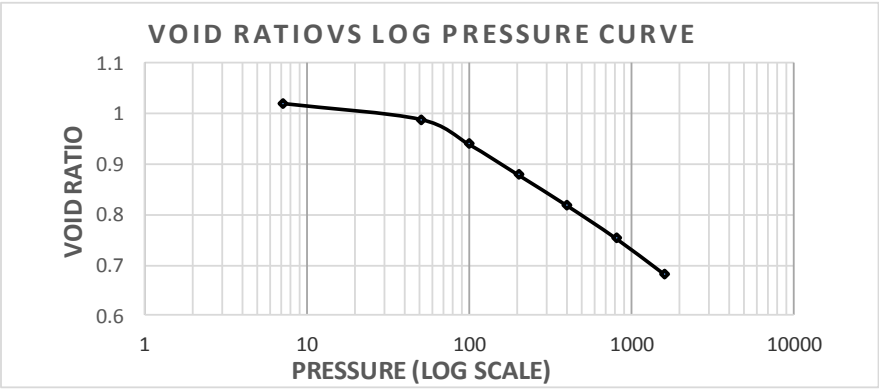
Test Pit 10

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	0.604	0.916	1.407	2.000	2.602	3.240
0.1	0.32	0.830	1.023	1.490	2.214	2.698	3.400
0.25	0.50	0.832	1.156	1.498	2.302	2.742	3.420
0.5	0.71	0.834	1.200	1.500	2.316	2.764	3.436
1	1.00	0.840	1.208	1.506	2.332	2.786	3.460
2	1.41	0.844	1.224	1.620	2.350	2.808	3.490
4	2.00	0.852	1.236	1.030	2.372	2.842	3.530
8	2.83	0.860	1.238	1.044	2.396	2.864	3.586
15	3.87	0.868	1.246	1.060	2.420	2.968	3.678
30	5.48	0.878	1.300	1.128	2.448	3.010	3.670
60	7.75	0.886	1.336	1.134	2.484	3.014	3.824
120	10.95	0.894	1.388	1.142	2.516	3.126	3.910
240	15.49	0.898	1.392	1.150	2.548	3.170	3.924
480	21.91	0.908	1.398	1.178	2.588	3.210	3.930
1440	37.95	0.916	1.407	2.000	2.602	3.240	3.986

Pit no	10	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	51.2
Ring Area,cm^2	19.63	Final Moisture Content,%	28.48
Diameter of Ring,mm	50	Height of Solid (Hs),mm	9.92
Seating Load,kpa	7	Final Void Ratio,e_f	0.68
Initial Void Ratio,e_o	1.02	Specific gravity	2.69
initial moisture content,%	29.77	Dry Density,g/cm^3	1.303
wet density,g/cm^3	1.83	Volume of ring, Cm^3	39.28

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.02
50	0.99
100	0.94
200	0.88
400	0.82
800	0.75
1600	0.68



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.205

e_A =	0.94	P_A =	100
e_B =	0.75	P_B =	800

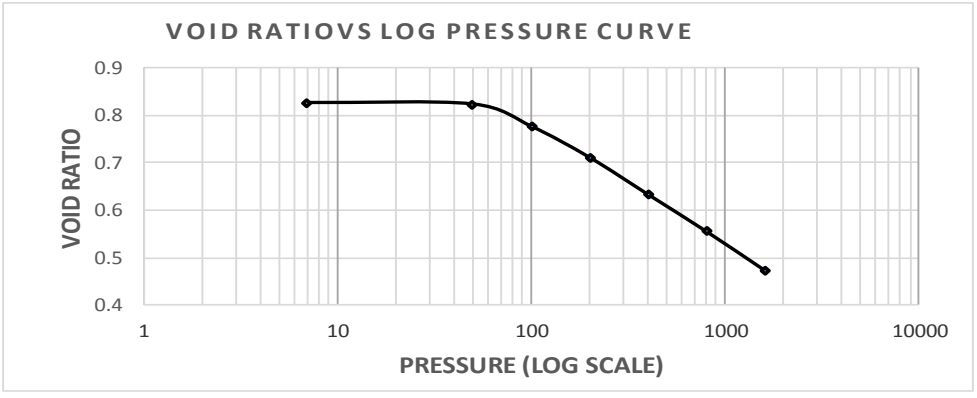
One Dimension Consolidation Test

Test Pit 11

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1.160	1.192	1.704	2.422	3.274	4.126
0.1	0.32	1.162	1.335	1.764	2.782	3.612	4.438
0.25	0.50	1.163	1.340	2.008	2.794	3.632	4.450
0.5	0.71	1.164	1.740	2.030	2.816	3.660	4.472
1	1.00	1.165	1.798	2.070	2.848	3.692	4.498
2	1.41	1.166	1.430	2.102	2.884	3.724	4.528
4	2.00	1.167	1.470	2.144	2.924	3.758	4.562
8	2.83	1.168	1.512	2.194	2.974	3.802	4.606
15	3.87	1.072	1.542	2.236	3.020	3.842	4.656
30	5.48	1.084	1.570	2.280	3.076	3.898	4.724
60	7.75	1.100	1.572	2.318	3.138	3.956	4.796
120	10.95	1.118	1.632	2.348	3.178	4.018	4.874
240	15.49	1.140	1.662	2.374	3.220	4.072	4.884
480	21.91	1.152	1.684	2.404	3.232	4.104	4.888
1440	37.95	1.192	1.704	2.422	3.274	4.126	5.024

Pit no	11	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	131.07
Ring Area,cm^2	44.16	Final Moisture Content,%	37.68
Diameter of Ring,mm	75	Height of Solid (Hs),mm	10.95
Seating Load,kpa	7	Final Void Ratio,e_f	0.47
Initial Void Ratio,e_o	0.83	Specific gravity	2.8
initial moisture content,%	32.84	Dry Density,g/cm^3	1.48
wet density,g/cm^3	1.47	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	0.83
50	0.82
100	0.78
200	0.71
400	0.63
800	0.56
1600	0.47



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.245

e_A =	0.78	P_A =	100
e_B =	0.56	P_B =	800

One Dimension Consolidation Test

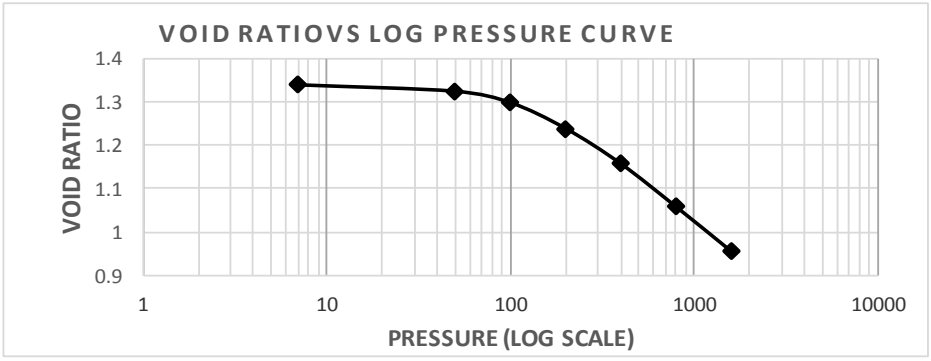
Test Pit 12

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	0.192	0.318	0.538	1.044	1.734	2.577
0.1	0.32	0.118	0.324	0.724	1.224	1.930	2.660
0.25	0.50	0.128	0.380	0.728	1.238	1.940	2.708
0.5	0.71	0.134	0.390	0.730	1.246	1.942	2.724
1	1.00	0.142	0.394	0.748	1.268	1.960	2.742
2	1.41	0.152	0.404	0.768	1.292	2.010	2.762
4	2.00	0.168	0.414	0.790	1.320	2.038	2.786
8	2.83	0.180	0.426	0.812	1.356	2.052	2.820
15	3.87	0.192	0.434	0.834	1.386	2.082	2.866
30	5.48	0.210	0.448	0.860	1.428	2.136	2.948
60	7.75	0.228	0.468	0.890	1.486	2.212	3.024
120	10.95	0.246	0.484	0.926	1.532	2.312	3.148
240	15.49	0.268	0.510	0.964	1.610	2.446	3.820
480	21.91	0.282	0.522	1.010	1.684	2.514	3.400
1440	37.95	0.318	0.538	1.044	1.734	2.577	3.452

Pit no	12	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	103.5
Ring Area,cm^2	44.16	Final Moisture Content,%	36.91
Diameter of Ring,mm	75	Height of Solid (Hs),mm	8.56
Seating Load,kpa	7	Final Void Ratio,e_f	0.96
Initial Void Ratio,e_o	1.34	Specific gravity	2.74
initial moisture content,%	30.38	Dry Density,g/cm^3	1.172
wet density,g/cm^3	1.6	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.34
50	1.32
100	1.30
200	1.24
400	1.16
800	1.06
1600	0.96



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.298

e_A =	1.24	P_A =	200
e_B =	1.06	P_B =	800

One Dimension Consolidation Test

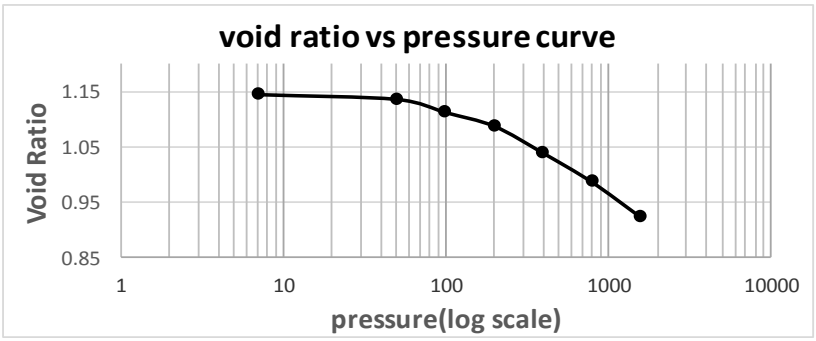
Test Pit 13

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.480	4.560	4.780	5.010	5.559	5.944
0.1	0.32	4.458	4.660	4.880	5.160	5.726	5.986
0.25	0.50	4.460	4.670	4.900	5.282	5.730	6.020
0.5	0.71	4.464	4.674	4.910	5.296	5.746	6.260
1	1.00	4.470	4.680	4.918	5.306	5.760	6.275
2	1.41	4.478	4.688	4.924	5.318	5.772	6.280
4	2.00	4.484	4.960	4.932	5.330	5.774	6.310
8	2.83	4.492	4.704	4.420	5.340	5.810	6.356
15	3.87	4.500	4.712	4.950	5.358	5.842	6.370
30	5.48	4.508	4.720	4.960	5.376	5.855	6.400
60	7.75	4.518	4.730	4.988	5.382	5.872	6.432
120	10.95	4.528	4.744	4.990	5.404	5.894	6.458
240	15.49	4.538	4.760	4.990	5.422	5.914	6.488
480	21.91	4.554	4.778	5.002	5.428	5.932	6.520
1440	37.95	4.560	4.780	5.010	5.459	5.944	6.531

Pit no	13	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	112.49
Ring Area,cm^2	44.16	Final Moisture Content,%	33.19
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.33
Seating Load,kpa	7	Final Void Ratio,e_f	0.92
Initial Void Ratio,e_o	1.14	Specific gravity	2.66
initial moisture content,%	29.29	Dry Density,g/cm^3	1.274
wet density,g/cm^3	1.28	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.14
50	1.13
100	1.11
200	1.09
400	1.04
800	0.99
1600	0.92



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.166

e_A =	1.09	P_A =	200
e_B =	0.99	P_B =	800

One Dimension Consolidation Test

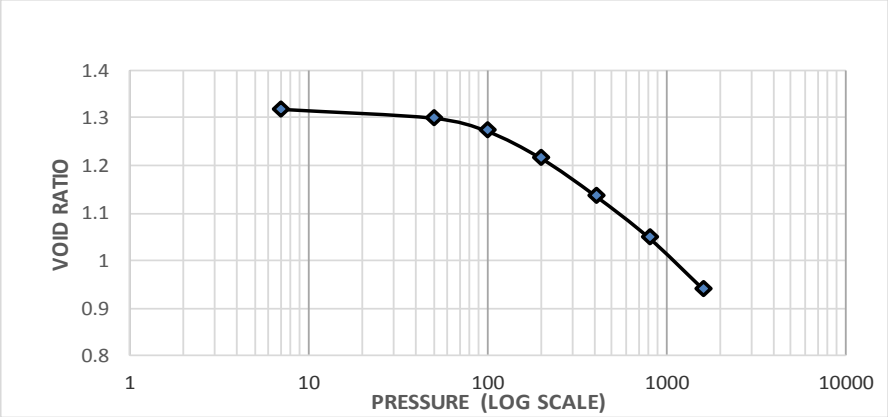
Test Pit 14

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	6.128	6.290	6.528	7.022	7.898	8.466
0.1	0.32	6.118	6.324	6.724	7.224	7.930	8.660
0.25	0.50	6.128	6.380	6.728	7.238	7.940	8.708
0.5	0.71	6.134	6.390	6.730	7.246	7.942	8.724
1	1.00	6.142	6.394	6.748	7.268	7.960	8.742
2	1.41	6.152	6.404	6.768	7.292	8.010	8.762
4	2.00	6.168	6.414	6.790	7.320	8.038	8.786
8	2.83	6.180	6.426	6.812	7.356	8.052	8.820
15	3.87	6.192	6.434	6.834	7.386	8.082	8.866
30	5.48	6.210	6.448	6.860	7.428	8.136	8.948
60	7.75	6.228	6.468	6.890	7.486	8.212	9.024
120	10.95	6.246	6.484	6.926	7.532	8.312	9.148
240	15.49	6.268	6.510	6.964	7.610	8.446	9.182
480	21.91	6.282	6.522	7.018	7.684	8.454	9.284
1440	37.95	6.290	6.528	7.022	7.698	8.466	9.396

Pit no	14	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	105.49
Ring Area,cm^2	44.16	Final Moisture Content,%	18.73
Diameter of Ring,mm	75	Height of Solid (Hs),mm	0.94
Seating Load,kpa	7	Final Void Ratio,e_f	8.62
Initial Void Ratio,e_o	1.32	Specific gravity	2.73
initial moisture content,%	20.87	Dry Density,g/cm^3	1.194
wet density,g/cm^3	1.6	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.32
50	1.30
100	1.27
200	1.22
400	1.14
800	1.05
1600	0.94



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.178

e_A =	1.22	P_A =	100
e_B =	1.05	P_B =	800

One Dimension Consolidation Test

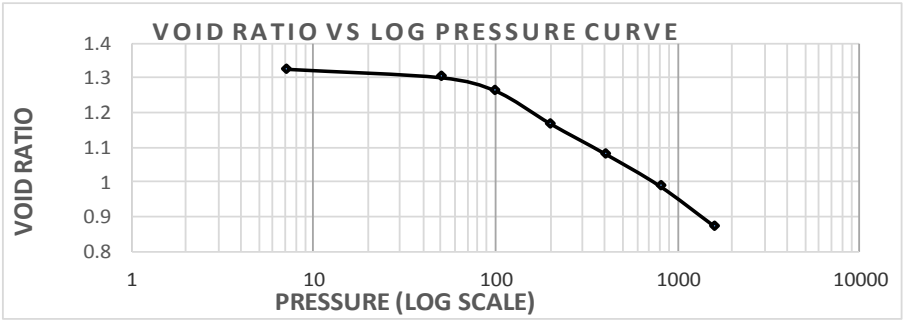
Test Pit 15

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	3.300	3.510	3.842	4.654	5.400	6.200
0.1	0.32	3.35	3.572	4.220	4.706	5.482	6.428
0.25	0.50	3.870	3.584	4.284	4.754	5.492	6.542
0.5	0.71	3.872	3.588	4.346	4.756	5.502	6.662
1	1.00	3.884	3.590	4.368	4.798	5.554	6.749
2	1.41	3.894	3.592	4.384	4.816	5.596	6.784
4	2.00	3.91	3.602	4.434	4.852	5.608	6.844
8	2.83	3.922	3.665	4.436	4.866	5.642	6.882
15	3.87	3.930	3.688	4.478	4.870	5.676	6.756
30	5.48	3.944	3.712	4.512	4.892	5.724	6.786
60	7.75	3.96	3.722	4.542	4.946	5.776	6.854
120	10.95	3.408	3.742	4.578	4.978	5.806	6.896
240	15.49	3.412	3.756	4.606	5.156	5.873	6.932
480	21.91	3.466	3.766	4.486	5.286	5.908	7.122
1440	37.95	3.510	3.842	4.654	5.400	6.200	7.200

Pit no	15	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	45.4
Ring Area,cm^2	19.63	Final Moisture Content,%	45.37
Diameter of Ring,mm	50	Height of Solid (Hs),mm	8.6
Seating Load,kpa	7	Final Void Ratio,e_f	0.87
Initial Void Ratio,e_o	1.33	Specific gravity	2.69
initial moisture content,%	40.01	Dry Density,g/cm^3	1.156
wet density,g/cm^3	1.55	Volume of ring, Cm^3	39.26

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.33
50	1.30
100	1.26
200	1.17
400	1.08
800	0.99
1600	0.87



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.278

e_A =	1.17	P_A =	200
e_B =	0.99	P_B =	800

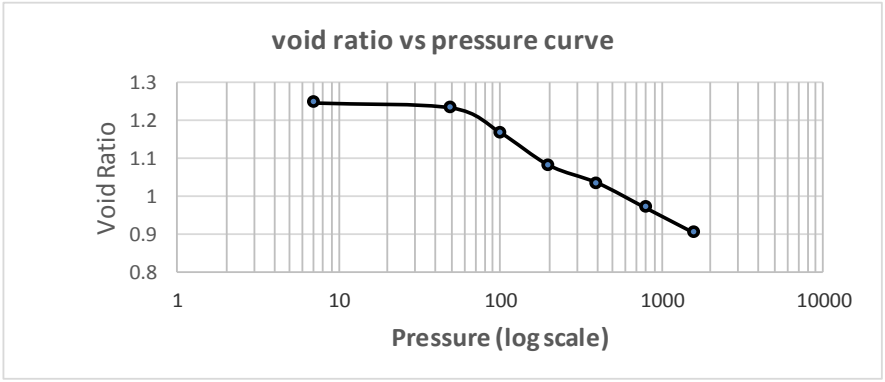
One Dimension Consolidation Test

Test Pit 16

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.200	4.410	5.000	5.778	6.200	6.800
0.1	0.32	4.226	4.540	5.344	5.852	6.258	6.846
0.25	0.50	4.250	4.550	5.416	5.884	6.300	6.868
0.5	0.71	4.256	4.560	5.464	5.946	6.320	7.020
1	1.00	4.258	4.660	5.482	5.986	6.352	7.128
2	1.41	4.280	4.582	5.502	6.052	6.384	7.214
4	2.00	4.300	4.600	5.522	6.146	6.420	7.246
8	2.83	4.326	4.618	5.540	6.154	6.458	7.260
15	3.87	4.340	4.634	5.566	6.162	6.480	7.284
30	5.48	4.344	4.660	5.602	6.186	6.520	7.296
60	7.75	4.366	4.694	5.614	6.188	6.580	7.308
120	10.95	4.370	4.754	5.664	6.192	6.728	7.318
240	15.49	4.386	4.842	5.715	6.194	6.758	7.354
480	21.91	4.400	4.932	5.746	6.196	6.778	7.376
1440	37.95	4.410	5.000	5.778	6.200	6.800	7.400

Pit no	16	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	105.49
Ring Area,cm^2	44.16	Final Moisture Content,%	35.69
Diameter of Ring,mm	75	Height of Solid (Hs),mm	8.78
Seating Load,kpa	7	Final Void Ratio,e_f	0.9
Initial Void Ratio,e_o	1.24	Specific gravity	2.63
initial moisture content,%	33.31	Dry Density,g/cm^3	1.194
wet density,g/cm^3	1.62	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.24
50	1.23
100	1.17
200	1.08
400	1.03
800	0.97
1600	0.90



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.199

e_A =	1.08	P_A =	200
e_B =	0.9	P_B =	1600

One Dimension Consolidation Test

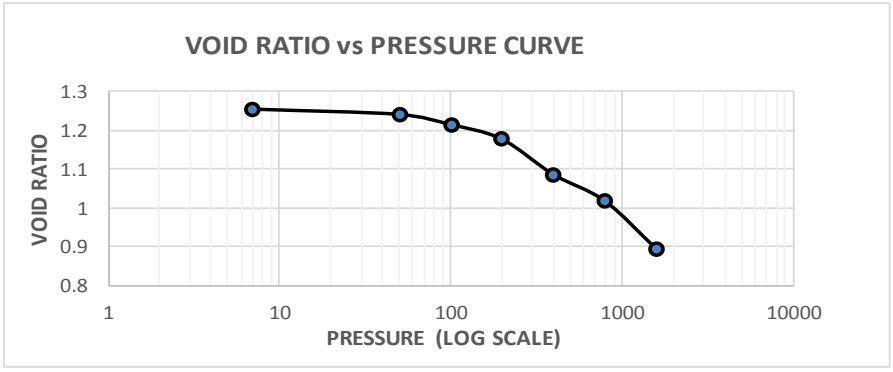
Test Pit 17

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.290	4.410	4.642	4.976	5.788	6.398
0.1	0.32	4.340	4.482	4.704	5.008	5.854	6.546
0.25	0.50	4.350	4.486	4.722	5.016	5.868	6.842
0.5	0.71	4.364	4.406	4.752	5.242	5.892	6.954
1	1.00	4.380	4.442	4.754	5.244	5.924	7.122
2	1.41	4.398	4.462	4.756	5.248	5.972	7.228
4	2.00	4.410	4.484	4.766	5.252	6.128	7.336
8	2.83	4.426	4.504	4.768	5.258	6.194	7.346
15	3.87	4.436	4.522	4.774	5.262	6.224	7.384
30	5.48	4.480	4.544	4.778	5.274	6.266	7.432
60	7.75	4.462	4.580	4.802	5.278	6.322	7.466
120	10.95	4.478	4.600	4.824	5.288	6.338	7.466
240	15.49	4.490	4.622	4.832	5.346	6.346	7.472
480	21.91	4.498	4.632	4.884	5.564	6.364	7.482
1440	37.95	4.410	4.642	4.976	5.788	6.398	7.500

Pit no	17	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	104.2
Ring Area,cm^2	44.16	Final Moisture Content,%	28.69
Diameter of Ring,mm	75	Height of Solid (Hs),mm	8.81
Seating Load,kpa	7	Final Void Ratio,e_f	0.89
Initial Void Ratio,e_o	1.25	Specific gravity	2.71
initial moisture content,%	27.94	Dry Density,g/cm^3	1.180
wet density,g/cm^3	1.63	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.25
50	1.24
100	1.22
200	1.18
400	1.09
800	1.02
1600	0.89



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.209

e_A =	1.22	P_A =	100
e_B =	0.94	P_B =	800

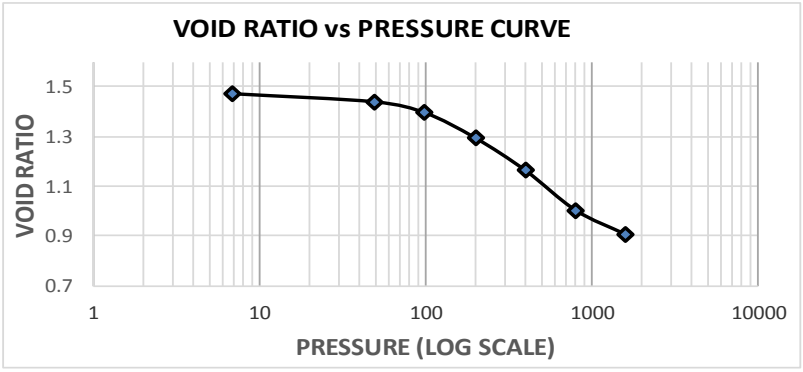
One Dimension Consolidation Test

Test Pit 18

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1.000	1.268	1.630	2.452	3.520	4.824
0.1	0.32	1.194	1.338	1.740	2.800	3.226	4.844
0.25	0.50	1.196	1.380	1.789	2.812	3.228	4.856
0.5	0.71	1.198	1.390	1.846	2.814	3.230	4.864
1	1.00	1.200	1.394	1.869	2.816	3.366	4.866
2	1.41	1.202	1.404	1.945	2.818	3.368	4.872
4	2.00	1.204	1.414	2.034	2.820	3.468	4.968
8	2.83	1.206	1.426	2.134	2.822	3.678	4.976
15	3.87	1.208	1.634	2.256	2.824	3.689	4.986
30	5.48	1.212	1.648	2.274	2.826	3.724	5.102
60	7.75	1.214	1.668	2.288	2.834	3.744	5.244
120	10.95	1.246	1.680	2.312	2.894	3.956	5.366
240	15.49	1.248	1.681	2.368	2.904	4.426	5.420
480	21.91	1.264	1.622	2.402	3.224	4.666	5.448
1440	37.95	1.268	1.630	2.452	3.520	4.824	5.616

Pit no	18	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Light Grey
In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	43
Ring Area,cm^2	19.63	Final Moisture Content,%	33
Diameter of Ring,mm	50	Height of Solid (Hs),mm	8.08
Seating Load,kpa	7	Final Void Ratio,e_f	0.9
Initial Void Ratio,e_o	1.47	Specific gravity	2.7
initial moisture content,%	29.52	Dry Density,g/cm^3	1.095
wet density,g/cm^3	1.47	Volume of ring, Cm^3	39.26

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.47
50	1.44
100	1.40
200	1.29
400	1.16
800	1.00
1600	0.90



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.265

e_A =	1.44	P_A =	50
e_B =	1.29	P_B =	200

One Dimension Consolidation Test

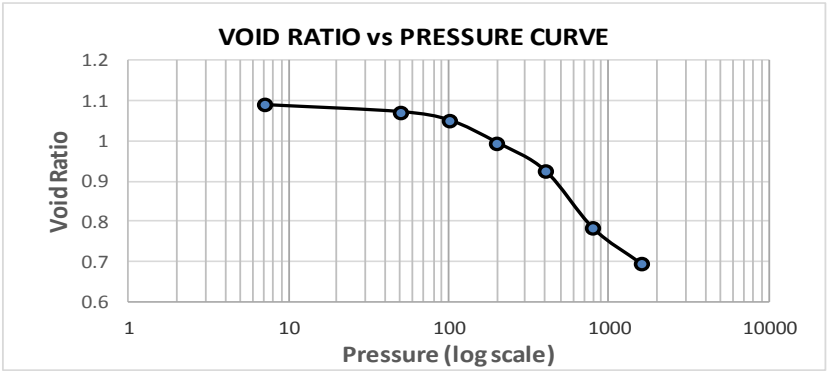
Test Pit 19

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1.128	1.294	1.488	2.034	2.688	4.054
0.1	0.32	1.138	1.324	1.624	2.194	2.792	4.266
0.25	0.50	1.148	1.360	1.688	2.218	2.840	4.308
0.5	0.71	1.150	1.376	1.730	2.246	2.912	4.324
1	1.00	1.158	1.384	1.748	2.268	2.960	4.342
2	1.41	1.162	1.386	1.768	2.292	3.010	4.462
4	2.00	1.168	1.414	1.790	2.320	3.038	4.586
8	2.83	1.180	1.426	1.812	2.356	3.052	4.620
15	3.87	1.192	1.434	1.834	2.386	3.082	4.666
30	5.48	1.210	1.448	1.860	2.428	3.636	4.648
60	7.75	1.228	1.458	1.890	2.486	3.712	4.724
120	10.95	1.246	1.460	1.926	2.532	3.812	4.748
240	15.49	1.268	1.466	1.964	2.610	3.846	4.782
480	21.91	1.282	1.476	2.012	2.654	3.914	4.784
1440	37.95	1.294	1.488	2.024	2.688	4.054	4.888

Pit no	19	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Black cotton

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	112.49
Ring Area,cm^2	44.16	Final Moisture Content,%	21.69
Diameter of Ring,mm	75	Height of Solid (Hs),mm	9.58
Seating Load,kpa	7	Final Void Ratio,e_f	0.7
Initial Void Ratio,e_o	1.09	Specific gravity	2.66
initial moisture content,%	25.52	Dry Density,g/cm^3	1.274
wet density,g/cm^3	1.88	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.09
50	1.07
100	1.05
200	0.99
400	0.93
800	0.78
1600	0.70



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.297

e_A =	1.05	P_A =	100
e_B =	0.78	P_B =	800

One Dimension Consolidation Test

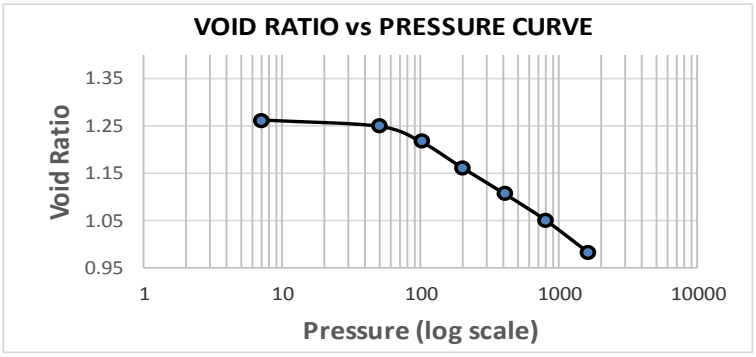
Test Pit 20

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	4.200	4.410	4.700	5.200	5.688	6.200
0.1	0.32	4.226	4.540	4.940	5.320	5.700	6.246
0.25	0.50	4.250	4.550	4.946	5.234	5.746	6.368
0.5	0.71	4.256	4.560	4.964	5.346	5.788	6.372
1	1.00	4.258	4.660	4.982	5.486	5.812	6.388
2	1.41	4.280	4.582	5.002	5.520	5.874	6.414
4	2.00	4.300	4.600	5.022	5.546	5.904	6.446
8	2.83	4.326	4.618	5.040	5.584	5.936	6.484
15	3.87	4.340	4.634	5.066	5.622	5.996	6.524
30	5.48	4.344	4.660	5.102	5.646	6.012	6.576
60	7.75	4.366	4.674	5.114	5.648	6.146	6.648
120	10.95	4.370	4.684	5.124	5.652	6.166	6.712
240	15.49	4.386	4.688	5.145	5.674	6.178	6.724
480	21.91	4.400	4.696	5.176	5.678	6.184	6.746
1440	37.95	4.410	4.700	5.200	5.688	6.200	6.800

Pit no	20	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Black Cotton

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	103.45
Ring Area,cm^2	44.16	Final Moisture Content,%	20.69
Diameter of Ring,mm	75	Height of Solid (Hs),mm	8.87
Seating Load,kpa	7	Final Void Ratio,e_f	0.97
Initial Void Ratio,e_o	1.24	Specific gravity	2.71
initial moisture content,%	33.31	Dry Density,g/cm^3	1.171
wet density,g/cm^3	1.24	Volume of ring, Cm^3	88.32

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.24
50	1.23
100	1.20
200	1.14
400	1.09
800	1.03
1600	0.97



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.197

e_A =	1.14	P_A =	200
e_B =	0.97	P_B =	1600

One Dimension Consolidation Test

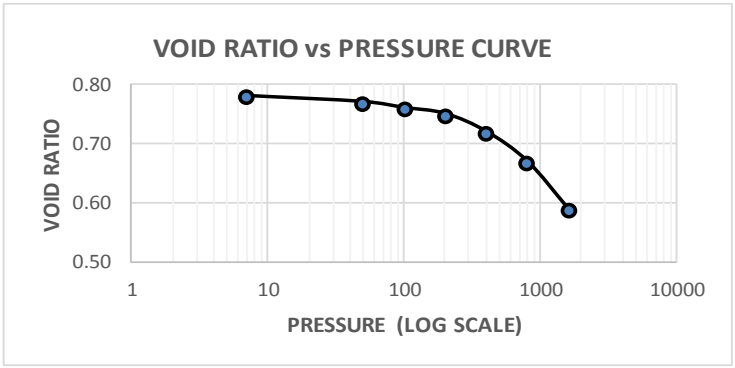
Test Pit 21

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0.00	1	1.097	1.182	1.329	1.57	2.109
0.1	0.32	1.035	1.135	1.25	1.424	1.758	2.308
0.25	0.50	1.054	1.139	1.255	1.433	1.773	2.322
0.5	0.71	1.056	1.141	1.259	1.44	1.788	2.336
1	1.00	1.058	1.142	1.264	1.457	1.808	2.353
2	1.41	1.063	1.144	1.268	1.468	1.832	2.376
4	2.00	1.067	1.146	1.272	1.476	1.854	2.407
8	2.83	1.068	1.115	1.276	1.485	1.884	2.451
15	3.87	1.071	1.152	1.282	1.498	1.916	2.5
30	5.48	1.074	1.156	1.287	1.512	1.955	2.57
60	7.75	1.082	1.16	1.296	1.528	1.996	2.645
120	10.95	1.092	1.165	1.303	1.541	2.037	2.770
240	15.49	1.093	1.172	1.312	1.544	2.067	2.862
480	21.91	1.095	1.176	1.320	1.554	2.084	2.920
1440	37.95	1.097	1.182	1.329	1.570	2.109	3.019

Pit no	21	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Brown to light Red

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	55.21
Ring Area,cm^2	19.63	Final Moisture Content,%	29.96
Diameter of Ring,mm	50	Height of Solid (Hs),mm	9
Seating Load,kpa	7	Final Void Ratio,e_f	0.59
Initial Void Ratio,e_o	0.78	Specific gravity	2.63
initial moisture content,%	27.62	Dry Density,g/cm^3	1.132
wet density,g/cm^3	1.34	Volume of ring, Cm^3	39.27

pressure P (kPa)	Ratio, e e = Hv/Hs
7	0.78
50	0.77
100	0.76
200	0.75
400	0.72
800	0.67
1600	0.59



$Cc=(e_A-e_B)/\log(P_B-P_A)$ **Cc= 0.225**

e_A = 0.72	P_A = 400
e_B = 0.59	P_B = 1600

One Dimension Consolidation Test

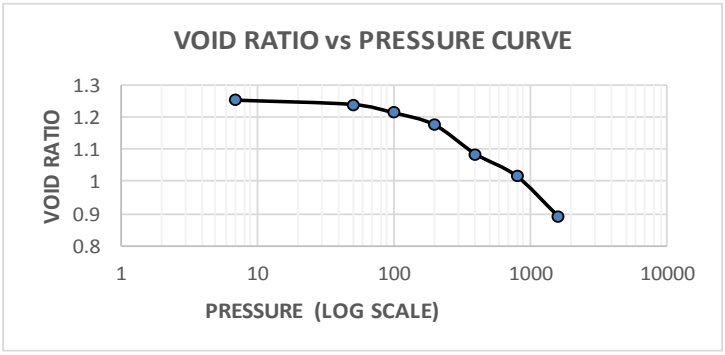
Test Pit 22

Time(min.)	√time	50 [kPa]	100 [kPa]	200 [kPa]	400 [kPa]	800 [kPa]	1600 [kPa]
0	0	1.564	1.916	2.253	2.972	3.93	4.995
0.1	0.32	1.744	2.008	2.53	3.34	4.314	5.2
0.25	0.5	1.768	2.026	2.57	3.39	4.342	5.242
0.5	0.71	1.778	2.038	2.601	3.434	4.37	5.29
1	1	1.79	2.054	2.642	3.474	4.403	5.321
2	1.41	1.802	2.07	2.678	3.521	4.443	5.36
4	2	1.8135	2.089	2.718	3.572	4.493	5.42
8	2.83	1.824	2.112	2.754	3.628	4.558	5.472
15	3.87	1.833	2.116	2.788	3.679	4.626	5.562
30	5.48	1.845	2.156	2.825	3.735	4.709	5.68
60	7.75	1.856	2.175	2.859	3.786	4.794	5.784
120	10.95	1.87	2.198	2.898	3.846	4.862	5.878
240	1.88	2.213	2.926	3.8685	4.9301	5.9435	7.354
480	1.893	2.234	2.945	3.885	4.963	6	7.376
1440	1.916	2.253	2.972	3.93	4.995	6.018	7.400

Pit no	22	**Test method**	**ASTM D 2435-96 Standard Test Method for one Dimensional Consolidation Properties of soil**
Depth	3m	**Color of sample**	Dark Grey

In The Begning of Test		**In The End of The Test**	
Height of Ring,mm	20	Dry Specimen Weight,gm	46.12
Ring Area,cm^2	39.27	Final Moisture Content,%	36
Diameter of Ring,mm	50	Height of Solid (Hs),mm	8.81
Seating Load,kpa	7	Final Void Ratio,e_f	0.78
Initial Void Ratio,e_o	1.27	Specific gravity	2.67
initial moisture content,%	37.5	Dry Density,g/cm^3	1.174
wet density,g/cm^3	1.47	volume of ring ,cm^3	39.27

pressure P (kPa)	Ratio, e e = Hv/Hs
7	1.27
50	1.23
100	1.19
200	1.11
400	1.01
800	0.89
1600	0.78



$Cc=(e_A-e_B)/\log(P_B-P_A)$ Cc= 0.178

e_A =	0.854	P_A =	50
e_B =	0.775	P_B =	1600

APPENDIX B: LINEAR REGRESSION ANALYSIS

Table B.1 Regression analysis output for Equation 1

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.903[a]	.815	.807	.025703

a. Predictors: (Constant), LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.067	1	.067	101.386	.000[b]
	Residual	.015	23	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.151	.040		-3.778	.001
	LL	.005	.001	.903	10.069	.000

a. Dependent Variable: Cc

Table B.2 Regression analysis output for Equation 2

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	PL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.790[a]	.624	.607	.036670

a. Predictors: (Constant), PL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.051	1	.051	38.110	.000[b]
	Residual	.031	23	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.064	.051		-1.248	.225
	PL	.010	.002	.790	6.173	.000

a. Dependent Variable: Cc

Table B.3 Regression analysis output for Equation 3

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	NMC[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.405[a]	.164	.128	.054649

a. Predictors: (Constant), NMC

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.013	1	.013	4.516	.045[b]
	Residual	.069	23	.003		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), NMC

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.072	.083		.869	.394
	NMC	.005	.002	.405	2.125	.045

a. Dependent Variable: Cc

Table B.4 Regression analysis output for Equation 4

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	Fs[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.346[a]	.120	.081	.056085

a. Predictors: (Constant), Fs

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.010	1	.010	3.124	.090[b]
	Residual	.072	23	.003		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), Fs

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.163	.049		3.303	.003
	Fs	.001	.001	.346	1.768	.090

a. Dependent Variable: Cc

Table B.5 Regression analysis output for Equation 5

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	C[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.295[a]	.087	.047	.057121

a. Predictors: (Constant), C

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.007	1	.007	2.185	.153[b]
	Residual	.075	23	.003		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), C

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.171	.053		3.232	.004
	C	.001	.001	.295	1.478	.153

a. Dependent Variable: Cc

Table B.6 Regression analysis output for Equation 6

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	Gs[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.261[a]	.068	.028	.057702

a. Predictors: (Constant), Gs

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.006	1	.006	1.681	.208[b]
	Residual	.077	23	.003		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), Gs

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.534	.603		-.886	.385
	Gs	.290	.224	.261	1.296	.208

a. Dependent Variable: Cc

Table B.7 Regression analysis output for Equation 7

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	eo[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.110[a]	.012	-.031	.059410

a. Predictors: (Constant), eo

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.001	1	.001	.282	.600[b]
	Residual	.081	23	.004		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), eo

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.207	.077		2.673	.014
	eo	.034	.063	.110	.531	.600

a. Dependent Variable: Cc

Table B.8 Regression analysis output for Equation 8

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	NMC, LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.935[a]	.873	.862	.021741

a. Predictors: (Constant), NMC, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.072	2	.036	75.925	.000[b]
	Residual	.010	22	.000		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), NMC, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.237	.043		-5.478	.000
	LL	.005	.000	.857	11.105	.000
	NMC	.003	.001	.246	3.185	.004

a. Dependent Variable: Cc

Table B.9 Regression analysis output for Equation 9

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	eo, LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.905[a]	.819	.803	.025992

a. Predictors: (Constant), eo, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.067	2	.034	49.819	.000[b]
	Residual	.015	22	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), eo, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.132	.048		-2.746	.012
	LL	.005	.001	.915	9.908	.000
	eo	-.020	.028	-.065	-.701	.490

a. Dependent Variable: Cc

Table B.10 Regression analysis output for Equation 10

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	C, LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.911[a]	.830	.814	.025211

a. Predictors: (Constant), C, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.068	2	.034	53.645	.000[b]
	Residual	.014	22	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), C, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.142	.040		-3.592	.002
	LL	.006	.001	.963	9.802	.000
	C	-.001	.000	-.136	-1.381	.181

a. Dependent Variable: Cc

Table B.11 Regression analysis output for Equation 11

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	Gs, LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.903[a]	.815	.798	.026277

a. Predictors: (Constant), Gs, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.067	2	.033	48.505	.000[b]
	Residual	.015	22	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), Gs, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.171	.277		-.619	.542
	LL	.005	.001	.901	9.429	.000
	Gs	.008	.106	.007	.076	.940

a. Dependent Variable: Cc

Table B.16 Regression analysis output for Equation 16

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	eo, LL, PL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.940[a]	.884	.868	.021268

a. Predictors: (Constant), eo, LL, PL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.073	3	.024	53.557	.000[b]
	Residual	.009	21	.000		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), eo, LL, PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.155	.040		-3.885	.001
	LL	.004	.001	.686	6.809	.000
	PL	.005	.001	.350	3.444	.002
	eo	-.031	.023	-.102	-1.335	.196

a. Dependent Variable: Cc

Table B.17 Regression analysis output for Equation 17

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	NMC, LL, PL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.958[a]	.917	.906	.017969

a. Predictors: (Constant), NMC, LL, PL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.075	3	.025	77.832	.000[b]
	Residual	.007	21	.000		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), NMC, LL, PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.253	.036		-7.016	.000
	LL	.004	.000	.669	7.861	.000
	PL	.004	.001	.288	3.347	.003
	NMC	.003	.001	.213	3.304	.003

a. Dependent Variable: Cc

Table B.18 Regression analysis output for Equation 18

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	Gs, PL, LL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.937[a]	.878	.860	.021894

a. Predictors: (Constant), Gs, PL, LL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.072	3	.024	50.147	.000[b]
	Residual	.010	21	.000		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), Gs, PL, LL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.348	.237		-1.468	.157
	LL	.004	.001	.654	5.957	.000
	PL	.005	.001	.345	3.270	.004
	Gs	.064	.090	.057	.707	.487

a. Dependent Variable: Cc

Table B.19 Regression analysis output for Equation 19

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	NMC, C, PL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.889[a]	.791	.761	.0286187

a. Predictors: (Constant), NMC, C, PL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.065	3	.022	26.444	.000[b]
	Residual	.017	21	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), NMC, C, PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.248	.061		-4.047	.001
	PL	.010	.001	.734	7.149	.000
	C	.002	.000	.343	3.412	.003
	NMC	.003	.001	.271	2.624	.016

a. Dependent Variable: Cc

Table B.20 Regression analysis output for Equation 20

Variables Entered/Removed[a]

Model	Variables Entered	Variables Removed	Method
1	Gs, C, PL[b]	.	Enter

a. Dependent Variable: Cc

b. All requested variables entered.

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.873[a]	.762	.728	.0305174

a. Predictors: (Constant), Gs, C, PL

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.063	3	.021	22.412	.000[b]
	Residual	.020	21	.001		
	Total	.082	24			

a. Dependent Variable: Cc

b. Predictors: (Constant), Gs, C, PL

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-.740	.320		-2.315	.031
	PL	.010	.001	.786	7.366	.000
	C	.001	.000	.303	2.841	.010
	Gs	.223	.119	.200	1.877	.074

a. Dependent Variable: Cc