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Statistical Model to Predict Compression Index from Index Properties of Fine-Grained Soils, Case in Azezo Tseda Sub City, Gondar.

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SCHOOL OF RESEARCH AND GRADUATE STUDIES

FACULTY OF CIVIL AND WATER RESOURCES ENGINEERING

MASTER'S PROGRAM IN GEOTECHNICAL ENGINEERING

MSc Thesis on:

Statistical Model to Predict Compression Index from Index Properties of
Fine-Grained Soils, Case in Azezo Tseda Sub City, Gondar.

By:

Birhanu Bieza Weldmariam

March, 2021

Bahir Dar, Ethiopia



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Statistical Model to Predict Compression Index from Index Properties of
Fine-Grained Soils, Case in Azezo Tseda Sub City, Gondar.

By

Birhanu Bieza Weldmariam

A Thesis Submitted

In Partial Fulfillment of the Requirements for the Degree of Master of Science
in Geotechnical Engineering

Advisor: Addiszemen Teklay (Ph.D)

March, 2021

Bahir Dar, Ethiopia

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Declaration

This is to certify that the thesis entitled Statistical Model to Predict Compression Index from Index Properties of Fine-Grained Soils, Case in Azezo Tseda Sub City, Gondar.

Submitted in partial fulfillment of the requirements for the degree of Master of Science in Geotechnical Engineering under Faculty of Civil and Water Resources Engineering, Bahir Dar Institute of Technology, is a record of original work carried out by me and has never been submitted to this or any other institution to get any other degree or certificates. The assistance and help I received during the course of this investigation have been duly acknowledged.

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Approval of thesis for defense result

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Dedication

I dedicate this work to my mother, wife and daughter

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First, I would like to thank the almighty God for his unspeakable gift, help and protection during my research work. Second, I want to address my appreciable gratitude to my advisor Dr.AddisezemenTeklay for his precious assistance in this entire works. I am thankful for his valuable encouragement, direction, advice, and helpful contribution during the preparation of this work. Finally, yet importantly, I want to express my lovely appreciation to my family and my friends for their endless support in every aspect during the preparation of this work. Thank you all.

ABSTRACT

Subgrade material deformation is one of the most important factors considering for design. For saturated subgrade soils, the compression mainly comes from the extrusion of water from the voids and is shown as the vertical and lateral deformation. Generally, vertical deformation is the main part that must be satisfied within the required limit to make sure the normal use and safety of structures. The subgrade compressibility index is the most important parameter used for calculating subgrade settlement. The main intention that leads this research was the Coefficient of compression (C_c) is a major geotechnical soil parameter required quite frequently for determining settlement of fine-grained soil layers subjected to loads such as that of buildings, vehicles, and infrastructures. If C_c is not known accurately, soil settlement is under- or overestimated, which both cases are not desirable from technical prospective. Thus, determining of C_c is significance importance in settlement calculations. However, measuring C_c using the traditional Oedometer test method is time-consuming, that demands skilled technicians, and requires special laboratory equipment. These bring out that, estimation of C_c from index property data of fine-grained soils are a justified cause. To achieve the goals of the major objective set in this research study, that is to develop a statistical model to predict compression index from index properties of fine grained soils in Azezo Tseda Sub City, Gondar, several steps/phases of studies were made under the different chapters. These studies include: undertaking detail literature review related to this research, conducting index property tests and classification studies, conducting one dimensional consolidation tests and determining compression index parameter; perform multiple regression analysis with subset selection and interaction routines, developing and validation of the newly developed prediction model using primary and secondary data and lastly performing comparative studies of the newly developed prediction model with other similar models.

The results from the study suggest, there are different types of soils in the sub city, however, the fine grained soils studied in the Azezo Tseda *sub city* mainly the Silty Clay and Clay with High and low plasticity is characterized in this study. The main intention was to make the developed models more specific to the corresponding soil types. Based on different index property tests made, the fine grained soils in the region are not expansive since it displays a Liquid Limit (%) range between 45-64% and the %Passing #200 Sieve size are greater than 50%. The Compression Index value calculated from one dimensional consolidation/ Oedometer test outputs determined from e-log p curve for these fine grained soils studied suggest that, the recommended range for settlement computation is between 0.19 - 0.56 with a mean C_c value of 0.37. Here in this study, based on the range of the C_c values, it is concluded that the fine grained soils in the study area are having a medium compressible nature. Moreover, three new simplified statistical models that can predict Compression Index (C_c) value from index property test data more accurately and reasonably are developed, validated and calibrated. These models are one, two and three parameter models respectively for Model 1, 2 and 3, and it is concluded that Compression Index (C_c) value can be predicted more reasonably only from the Atterberg limit test data.

The Multiple Regression Analysis showed that there is a relationship between index properties and compression characteristics of the fine grained soils. Especially Plasticity Index for Model 1, Plasticity Index and Liquidity Index for Model 2 and Plasticity Index, Plastic Limit and Liquidity Index for Model 3 are found a significant/powerful variable for prediction of compression index of the fine grained soils found around Azezo-Tseda sub city area. The three Cc predictive models satisfy all the required statistical goodness of fitting measures and the different multiple regression assumptions, however comparison of the new models each other suggest that Model 3 has the highest significance recommended for practice. Comparative studies suggest, the newly developed models are the best fit to the data measured for the study area than when the data fitted with model developed from other regions.

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NOMENCLATURE

CH	=	Inorganic clays of high plasticity
CL	=	Inorganic clays of low to medium plasticity
FC	=	Fine content (percentage of soil passing No. 200 sieve)
LL	=	Liquid Limit
ML	=	Inorganic silts of low plasticity
MH	=	Inorganic silts of high plasticity
N	=	Number of sample
OH	=	Organic clays of medium of high plasticity.
OL	=	Organic silts of low plasticity
OMC	=	Optimum Moisture Content
PI	=	Plastic Index
PL	=	Plastic Limit
e_o	=	Void Ratio
W_n	=	Water Content
G_s	=	Specific Gravity
C_c	=	Compression Index
R^2	=	Correlation Coefficient
SE	=	Standard error of the estimate
USCS	=	Unified Soil Classification System
σ_1	=	stress 1
σ_2	=	stress 2
#200p	=	particles that pass sieve no #200

CHAPTER ONE

INTRODUCTION

1.1. BACKGROUND OF THE STUDY

The stability of any structure resting on the fine grained soils is one major concern of geotechnical engineers that demands several phases of detail investigation. The need for engineering properties of soils for use in planning, site selection, design, construction, and maintenance of transportation facilities is recognized by civil engineers. The geotechnical and structural stability requirements are significantly affected by the properties of the soil found below the structure that however needs to be properly identified, tested and characterized. These will help to detail the behavior of soil. The properties of soil such as permeability, compressibility and strength of soil always affect the design and constructions. However, obtaining these engineering properties of soils requires extensive laboratory and field testing, and these approaches relatively consume more time and money. Moreover, appropriate estimation of soil settlement is of significant importance since it directly influences the performance of building and infrastructures that are built on soil. However, most of the Engineering properties of soils depend up on the simple basic/index properties. In other word, considering test time, money, and effort, laboratory determination of the index properties of a soil is much easier than investigating the engineering properties. Therefore, by obtaining the index property of soils that involves simpler and quicker method of testing, the properties can be statistically predicted satisfactorily from empirical correlations.

The soil consolidation will occur as a result of expulsion of water from the soil pores. This process has occurred continuously until all the excess pore water pressure caused by total stress change is totally dissipated. The designing process, is really necessary to takes total consolidation that will occur in the structure and avoid the differential settlement which led to crack in building structure. Settlements in deep deposits in soft clay constitute an engineering challenge in construction of buildings, road embankments, flood control levees, dykes and dams. For improvement of soft soil or the construction of building foundations or embankments, it is essential to know the rate of compression of this soil. This can be done by conducting routine one-dimensional consolidation tests from

which the values of compression index C_c and coefficient of consolidation c_v can be evaluated. The compression index is one of the important soil parameters that are essential for geotechnical designs. Because laboratory and in-situ tests for determining the compression index (C_c) value are laborious, time consuming and costly, empirical formulas based on soil parameters are commonly used. This process is time consuming and there is a need to predict long term settlement with a high degree of accuracy in short duration of time by correlating these long process for determination of consolidation test of soil with some other simple properties tests. Due to time constraints, emergency of the project as well as for a better comparison, several correlations are widely used while preparing geotechnical investigation report. Several empirical correlations were developed by other researchers. Some correlation based from Liquid Limit value has been developed. However, this empirical formula is only correlations of soil in the particular country. Therefore, a new empirical correlation is needed which can more generally applied in all type of clay soil.

Compression Index is used widely for estimation of settlement during design of foundations. In particular, it needs tests, limited laboratory facilities and preliminary evaluations, various equations are available in the literature to simply estimate the compression index using some soil index properties. However, applicability of these equations is often limited for the interested site due to their establishment from different databases. Thus, it is more realistic way to employ the correlations developed using local data of region, provided that data is available. So, this study aims to develop correlations for the compression index dependent upon the soil index parameters of plastic index for fine grained soil at Azezo-Tseda sub city. The behavior of soil is an important element which always concerned in civil engineering. The properties of soil such as plasticity, compressibility or strength of soil always affect the design in constructions. Lack of understanding of the properties of the soil can lead to construction errors that are costly in effort and material. The suitability of a soil for a particular use should be determined based on its engineering characteristics and not on visual inspection or apparent similarity to other soils. The aim of this research is to know the relations PI between physical properties and mechanical properties of the soil. Physical properties refer to the Atterberg Limit Test to determine Plasticity Index (PI). The Atterberg Limit test is simple and rapid. Meanwhile mechanical properties refer to One Dimensional Consolidation test (Oedometer test) to

determine Compression Index (C_c). The Oedometer test is a complex, time consuming and expensive test if compared to other soil tests. The main objective of this thesis is to develop the correlation between index properties and compression index soils widely found around Azezo Tseda.

1.2. STUDY AREA

Gondar is one of the historical cities of Ethiopia which is located in the Central Gondar Zone of the Amara Region, Gondar is north of Lake Tana on the Lesser Angereb River and southwest of the Simien Mountains. Its latitude and longitude of $12^{\circ}36'N$ $37^{\circ}28'E$ with an elevation of 2133 meters above sea level. It has six (6) sub cities in the town and this study focuses on Azezo Tseda sub city due to the distribution soils in the area.

1.3. STATEMENT OF THE PROBLEM

Laboratory tests are typically classified in the following main categories: (a). Tests for index properties (e.g., water content, unit weight, particle size, Atterberg limits); (b). Tests for deformation properties (e.g., consolidation, triaxial, simple shear, resonant column); (c). Tests for strength properties (e.g., direct shear, unconfined compression, triaxial, lab vane); (d). Tests for flow properties (e.g., constant head permeameter, falling head permeameter, erosion tests). Determination of Compression Index from consolidation tests is expensive, cumbersome and time consuming since it takes a maximum of three (3) weeks to complete a typical consolidation test. Due to this, Compression Index will be correlated with index properties of a soil which are relatively easier to determine and take lesser time to compare in the laboratory test. Correlations have been developed in the literature relating the compression index C_c of soft soils to simple index properties. But, these empirical correlations are specific to soils of a certain geographic region or geological origin and may not be applicable in other region. Therefore, the correlation relationship PI between compression ratio C_c and the plastic index (PI) is the most fruitful preliminary predictions of compression index to be made.

1.4. THE OBJECTIVE OF THE STUDY

1.4.1. General Objective

The objective of the research study is to develop a statistical model to predict compression index from index properties of soil in Azezo Tseda Sub City, Gondar

1.4.2. Specific Objectives

The specific objectives of this study are.

- a) To undertake detail literature review related to this research
- b) To conduct Index Property Tests and classify the soils found in Azezo Tseda sub city.
- c) To conduct one dimensional consolidations tests and determine compression index parameters of the soils in Azezo Tseda sub city. This is for the corresponding index properties and each category of the soils classified for each soil sample
- d) To develop a new simplified statistical model helpful to predict compression index from index properties of soil in Azezo Tseda Sub City, Gondar using multiple regression analysis with subset selection and interaction routines.
- e) To validate the newly developed prediction model using primary and secondary data
- f) To compare the newly developed prediction model with other similar models

1.5. SCOPE

This study is restricted to predict compression index from index properties of fine-grained soils at Azezo Tseda sub city Gondar. The depth of ground investigation is limited to 2.5m-3meters. In order to conduct the proposed predictive models or correlation, only twenty laboratory test results are used in this study. With regard to the regression analysis, depending on the trends of the scattering of test results the correlation is analyzed using multiple regression models within the aid of excel and JMP-13 SAS software.

1.6. SIGNIFICANCE OF THE RESEARCH

This study is important to estimate one of the fundamental engineering properties of soils particularly useful for settlement computation for the project where there is a financial limitation, lack of test equipment or limited time, and statistical predictive models are commonly used in the preliminary stage of any project. The attempts have been made to obtain correlation between Compression Index (cc) and index properties of fine grained soils of Azezo-Tseda sub city.

CHAPTER TWO

LITERATURE REVIEW

2.1. GENERAL FOCUS OF THE REVIEW

The vertical, downward movement of a structure occurs when the volume of a soil on which the structure is built decreases. Unless the settlement is not kept within a tolerable limit, the desired use of the structure may be impaired and the design life of the structure may be reduced. The safety of structures in fine grained/clay soils demand greater attention. The consolidation characteristic of saturated soft/compressible geo-materials is very useful for forecasting the magnitude and time of the settlement of the structure. One responsibility of the Geotechnical Engineers in this regard is to calculate the extent of the possible settlement of the structure, therefore the study focusing consolidation characterization of clay soil is very important. Precise estimation of this property is critical for calculating settlement of soil layers. Due to their low permeability, the case has become more critical for fine-grained soils, resulting in Compression Index (C_c) to be the most accepted parameter to date to represent soil compressibility.

As explained in chapter 1, the objective of this study is to develop a statistical model to predict Compression Index (C_c) from conventional index test parameters of fine-grained soils. In order to achieve such an objectives, it is necessary to perform a review of the relevant literature, which has been organized according to the problems listed in the previous chapter. In this literature review, therefore, the following points are focused and summarized: fine grained soils: - index property testing and characterization classification of fine grained soils. Identification of clay soils: - methods based on physical properties, methods based on consistency limits; consolidation characterization of clay soil, consolidation properties: one dimensional consolidation tests and existing fine grained soil compressibility index prediction models.

2.2. FINE GRAINED/CLAY SOILS

2.2.1. Characteristics of Fine Grained/Clay Soils

Plastic Fine Grained soils are those that plot above the 'A' line on the plasticity chart (Bowles, 1992; Roy.whitlow, 1995). Clays are virtually impervious, difficult to

compact when wet, and impossible to drain by ordinary means. Clays are large expansion and contraction with changes in water content. The higher the liquid limit of a clay, the more compressible it will be, and so, the liquid limit is used to distinguish between clays of high compressibility (H) and low compressibility (L) and cohesive clays have higher liquid limit and plasticity index (Raymond, 1997). Clays are defined as the particles of size less than 2microns. But all particles less than 2 microns do not influence the engineering properties in the same way. Clay mineral plays a dominant role in controlling the physical and engineering properties of fine grained soils. Clay minerals occur small in size that physio-chemical interactions with each other and with the water-electricity phase of a soil could be significant (Sridharan, 2014).

Consolidation is the volume reduction due to the increase in existing confining pressures taking place due to the drainage of pore water. This phenomenon is important mainly for cohesive, fine-grained soil and it is a time-dependent process since the rate of drainage of pore water is a function of the permeability of the soil layer. Consolidation tests are expensive and time-consuming. Special sampling and testing techniques are required. It needs conduct these tests with the utmost accuracy and to adopt realistic and suitable procedures to evaluate and interpret the results obtained. However, the groups of tests performed to obtain index soil properties are relatively inexpensive and simple (kok shien Ng,Yee.et al,2018). These do not require many time or any sophisticated testing systems Soil compressibility is considered to be the volume reduction under load of pore water drainage. The estimation of this property is critical for calculating the settlement of soil layers (Tiwari, B Ajmera B,2011). This problem has become more critical for fine-grained soils due to their low permeability, resulting in the Compression Index (Cc) being the most accepted parameter to date to represent soil compressibility (Carter, M and Bentley, S.P, 1991).

2.2.2. Index Property Testing of Fine Grained Soils

To classify a soil, tests are performed according to the American Society for Testing and Materials (ASTM) standards, and the results of these tests are used in a classification system recommended by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The tests are the sieve analysis, the hydrometer analysis, and the Atterberg limits.

2.2.3. Classification of Fine Grained Soils

The classification system is called the Unified Soil Classification System (USCS). The Unified Soil Classification System, or USCS, is the system used internationally to classify soils. Most commonly, it employs a two-letter symbol. The first letter indicates whether the soil is Gravel (G), sand (S), Silt (M), or Clay (C). The letter for silt could not be S, as that letter was already used for sand, so the letter M was chosen. The second letter gives additional information on the soil. For coarse-grained soils, the second letter can be M or C, indicating that the gravel or sand has a significant amount of silt or clay particles in it. For fine-grained soils, the second letter can be H, meaning high plasticity (high liquid limit and high PI), or L for low plasticity (low liquid limit and low PI). Flowchart to classify a fine grained soil by the USCS is shown in Figure 2. 1.

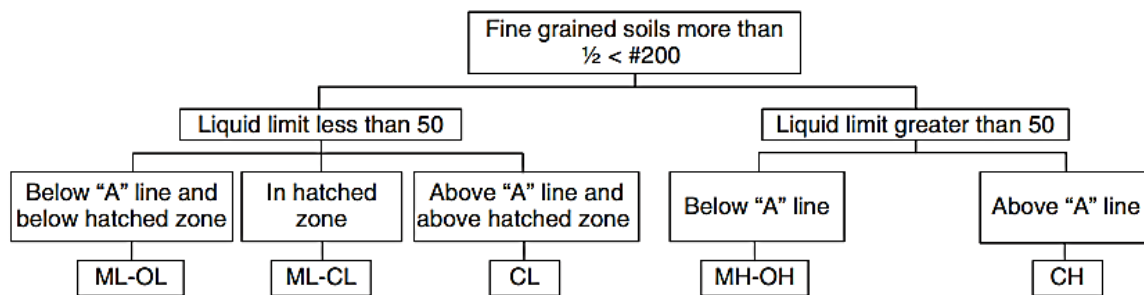


Figure 2. 1. Flowchart to classify a soil by the USCS.

2.2.4. Engineering Significance of Classification Parameters and Plasticity Chart

The plasticity index PI is definitely the index most used in practice, with the liquidity index a distant second. The others are rarely used. The PI essentially relates to how small the clay particles are in the soil: the higher PI is, the smaller the clay particles are. Very high PI values (60 or more) are associated with a predominance of very small clay particles, such as in montmorillonite; low PI values (20 or less) are associated with a predominance of larger clay particles, such as in kaolinite. Thus, the PI value gives an indication of some important properties of a soil. For example, a high PI value indicates a soil that will be very difficult to compact, has a high shrink-swell potential, and has low permeability. A low PI value is often required for fill material when good drainage is important, such as for pavement layers and retaining walls backfill. From the point of view of soil strength, the friction between particles decreases with increasing PI. Also, a

comparison between the natural water content and the limits can give an indication of possible soil behavior.

For example, if the natural water content is higher than the liquid limit, the soil is likely to be sensitive (it may lose significant strength when remolded). If the soil has a water content close to the shrinkage limit and a high shrink-swell index, beware of swelling problems if the soil can get wet (Bowles, 1992; engineering properties of expansive clays, 1956; Briaud, 2013). The plasticity chart was developed by Arthur Casagrande, an Austrian-born American civil engineer, around 1932. The plasticity chart is a plot of the plasticity index versus the liquid limit of a soil (Figure 2.2), and is used for the purpose of classifying fine-grained soils according to their plasticity.

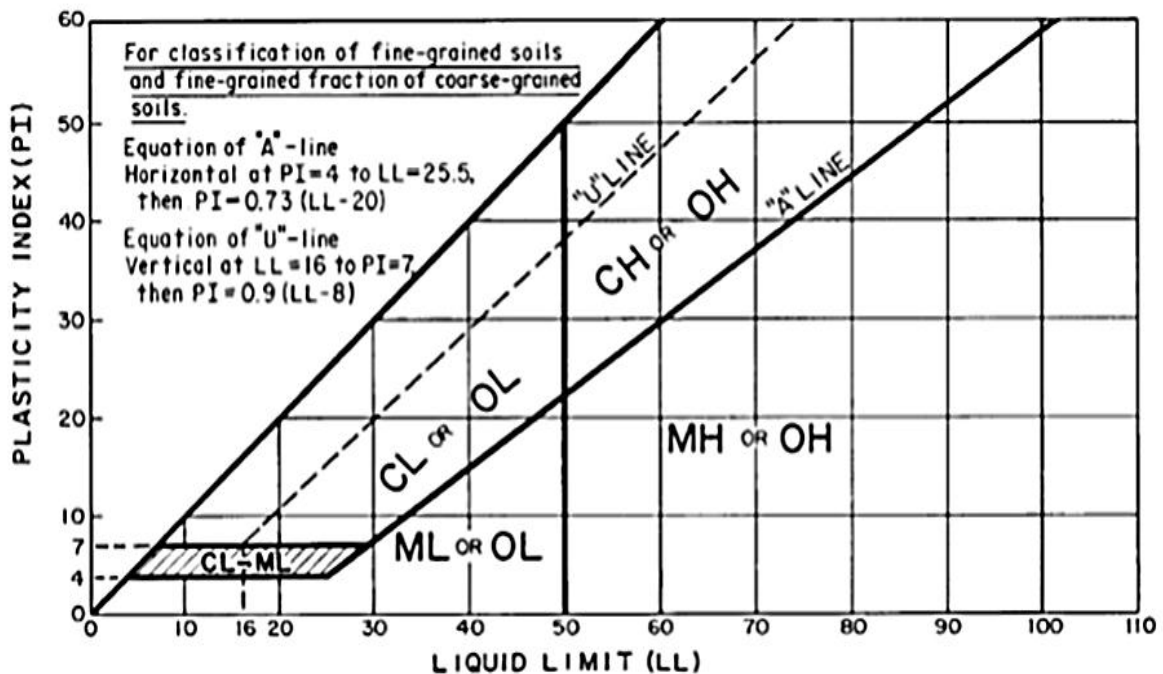


Figure 2.2. Plasticity Chart (Notes of the compressibility of clays, 1944)/

The A-line is an empirically chosen line that splits the chart between clays above the A line and silts below the A line. The vertical line, corresponding to a liquid limit equal to 50%, separates high-plasticity Fine-Grained Soils ($w_L > 50$) from low-plasticity fine-grained soils ($w_L < 50$). To classify a soil, the plasticity index and liquid limit of that soil are plotted on the chart; the region in which the point falls indicates what type of fine-grained soil it is or what kind of fines are encountered in a coarse-grained soil. The plasticity chart is the basis for the classification of fine-grained soils and of the fines fraction of coarse-grained soils.

2.3. IDENTIFICATION OF CLAY SOILS

The soil can be identifying whether it has the potential to be compressible or not. Such identification is important during the reconnaissance and preliminary stages of a site investigation. It suggests an appropriate sampling and testing methods to be used. It helps to identify the compressible potential of soils and can be measured by physical properties of soils, such as Atterberg limits, free swell, hydrometer analysis, and potential volume change.

2.3.1. Methods Based on Physical Properties

2.3.2. Methods Based on Plasticity

Atterberg limits are used in some methods to identify expansive soils. Two indices defined on the basis of the Atterberg limits are the plasticity index, PI, and the liquidity index, LI. Many expansive soil identification methods make use of one or both of these indices. As shown in Figure 2.2, the more expansive minerals tend to exhibit higher plasticity. The correlation is significantly improved by correlating expansion potential with the product of plasticity index and percentage passing the No.200 (75µm) sieve. The plasticity of a soil may be an indicator of expansive minerals that in itself is not definitive identification of an expansive soil. Atterberg limits and clay content can be combined into a parameter called activity, Ac. This term was defined by Skempton (1953) as:

$$\diamond Ac = \left(\frac{PI}{\% \text{ finer than } 2} \right) \quad (2-1)$$

2.3.3. Atterberg Limits and Other Limits

The term "clay" refers to a naturally occurring material composed primarily of fine-grained minerals, which is generally plastic at appropriate water contents and will harden when dried or fired. Clay usually contains phyllosilicates, it may contain other materials that impart plasticity and harden when dried or fired. Associated phases in clay may include materials that not impart plasticity and organic matter (Thair Al Ani and olli sarapaa, 2008). Particle size is not the main factor controlling the behavior of silts and clays. Instead, the behavior and therefore the classification are based on the ability of the soil to be deformed and stay together (consistency) (Briaud, 2013). This is measured by the Atterberg limits. Albert Atterberg was a Swedish chemist who worked in the field of

agricultural science; he came up with what is now known as the Atterberg limits around 1910 as a means of classifying fine-grained soils. The Atterberg limits are water contents of remolded fine-grained soil. The limit tests are performed on remolded samples of silts or clays or more generally on the portion of a sample finer than sieve #40 (0.425 mm opening). These limits indicate the points at which the consistency of a fine-grained soil (Figure 2.3) changes from a liquid state to a plastic state (liquid limit), from a plastic state to a semisolid state (plastic limit), and from a semisolid state to a solid state (shrinkage limit). The liquid limit w_L has a precise ASTM definition (ASTM D4318; ASTM 2004a). In short, it is the water content at which the remolded soil behaves like a soft paste (toothpaste consistency). This particular water content varies significantly depending on how fine the particles are. For example, very fine clay particles can have liquid limits approaching 100%, whereas silt particles may have liquid limits of around 30%. More precisely (Figure 2.3), the liquid limit is defined as the water content at which the two sides of a small amount of soil placed in a standard cup and grooved by a standard tool will flow together over a distance of 12.5mm when hit by 25 blows in a standard liquid limit apparatus.

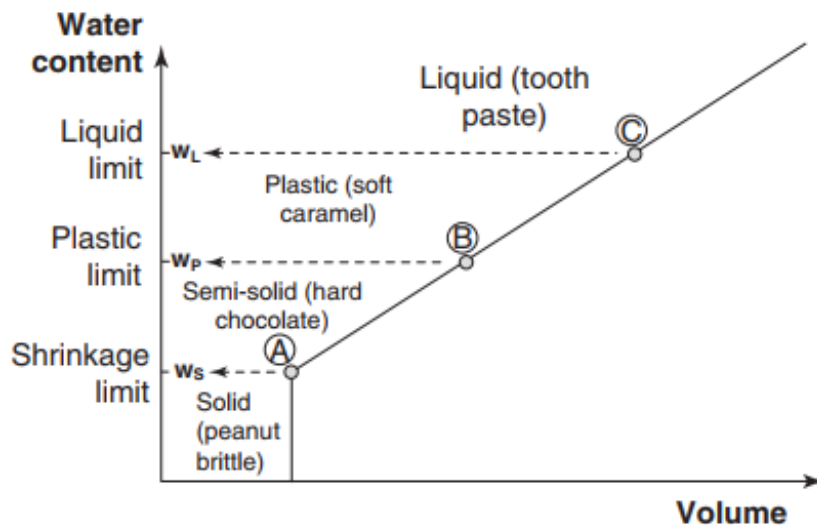


Figure 2.3. States of consistency and Atterberg limits.

The plastic limit w_p also has a precise ASTM definition (ASTM D4318; ASTM 2004a). In short, it is the water content at which the remolded soil behaves like a hard paste (soft caramel). More precisely, the plastic limit is the water content at which a soil will begin to crumble when rolled into a thread 3.2 mm in diameter. The difference between the liquid limit and the plastic limit is the plasticity index or I_p . The plasticity index has been

found to be related to a number of useful soil properties. The shrinkage limit w_l is defined in ASTM D4943 (ASTM 2004b). The shrinkage limit is the water content corresponding to the amount of water necessary to fill all the voids of the dry soil after shrinkage. It is close to the lowest water content at which the remolded soil is still saturated during a drying process; any further drying leads to a degree of saturation less than 100%.

2.4. CONSOLIDATION CHARACTERISTICS OF CLAY SOIL

2.4.1. Consolidation Properties of soils

Consolidation is a process by which soils decrease in volume by squeezing out pore water on account of gradual dissipation of excess hydrostatic pressure induced by an imposed total stress. According to Karl Terzaghi "consolidation is the process which involves decrease in water content of a saturated soil without replacement of water by air." Terzaghi shown the consolidation process in a clay soil subjected to loading is analogous to the behavior of the spring-piston model. Springs represent soil skeleton. Spring surrounded by water represents saturated soil. Perforations in the pistons are analogous to the voids that impart permeability to soil. Compressibility characteristics of soils forms one of the most important parameters required in the design of foundations. The Oedometer test is used to determine the compressibility characteristics of fine grained soils and the slope of the straight-line portion of the void ratio (e) versus logarithm of effective pressure ($\log p$) curve, defined as compression index (C_c) is widely used world over to estimate the consolidation settlement of foundations on clays. But, estimation of compression index from laboratory Oedometer test requires considerable time and effort. In view of the above and also due to the cost involved in obtaining undisturbed samples, it is highly desirable, at least for a preliminary assessment of settlement of structures, that an empirical correlation for compression index is developed. When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index. The soil which had its load removed is considered to be over consolidated. The highest stress that it has been subjected to is termed the reconsolidation stress. The over consolidation ratio or OCR is defined as the highest stress experienced divided by the current stress. A soil which is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. A soil could be considered under-

consolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate.

2.4.2. One Dimensional Consolidation Tests/Oedometer test

The consolidation test dates back to the early 1900s, and it may be appropriate to attribute its early development to Terzaghi, around 1925, with Casagrande and Taylor making significant contributions as well. The consolidation test (ASTM D2435) is used mostly for determining the compressibility of saturated fine-grained soils. It consists of placing a disk of soil approximately 25 mm high and 75 mm in diameter in a steel ring of the same diameter and applying a vertical load on the sample while recording the decrease in thickness of the sample (Figure 2.4). Filter stones are placed at the top and bottom of the sample to allow the water squeezed out of the sample to drain at both ends. There are several loading procedures: incremental loading, constant rate of strain, and constant gradient the incremental loading procedure is the most popular and consists of placing a load on the sample for 24 hours while recording the decrease in sample thickness. The load creates a constant total normal stress σ on the surface of the sample. When σ is applied, the water stress u_w goes up because the water has difficulty escaping from the small soil pores quickly enough. It takes some time for the water stress u_w to decrease and come back to its original value (Briaud, 2013). This decrease in u_w is associated with a corresponding increase in effective stress ($\sigma' = \sigma - u_w$ in this case, because the soil is saturated) and a settlement of the soil; this is the process of consolidation. The 24-hour loading step is considered to be sufficient in general for the water stress u_w to decrease back to zero.

Therefore, it is assumed that at the end of each 24-hour loading step, the water stress is back to zero and the total normal stress σ is equal to the effective normal stress σ' . The loads and associated pressures are applied in a sequence where the load is doubled each time. A typical sequence is 12, 25, 50, 100, 200 kPa for σ . The last point at the end of the 24-hour loading step curve (displacement vs. time) gives one point (vertical effective stress σ' and vertical strain ϵ) on the consolidation test stress-strain curve (stress vs. strain. The upward curvature of this stress-strain curve and the lack of maximum stress or failure stress or strength is due to the steel ring that confines the soil sample. The more load that is applied to the sample, the more the steel ring contributes to the resistance. Note that this curve is often presented as void ratio e versus the decimal logarithm of the effective stress

$\log \sigma$ (Compressibility characteristic of soil for Geotechnical and geological engineers, 2003).

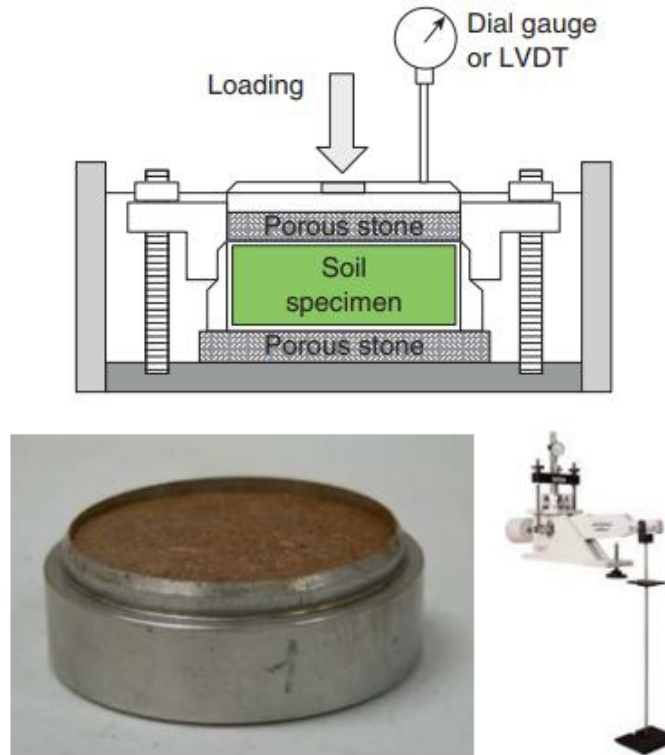


Figure 2.4. Consolidation test and equipment: (a) Principle. (b) Sample in ring. (c) Complete setup. (Briaud, 2013).

The compression index C_c is defined as the slope of the linear portion of the $e - \log \sigma$ curve past the initial rounded part of that curve and computed as:

$$C_c = \frac{\Delta e}{\Delta \log \sigma} = \frac{e_1 - e_2}{\log \sigma_1 - \log \sigma_2} \quad (2-2)$$

The compression index C_c , usually determined by Oedometer testing, is an important one-dimensional compressibility parameter with particular relevance to primary settlement calculations for normally consolidated or lightly over consolidated natural soils. In geotechnical practice, the extent of Oedometer testing specified for these softer soils is often insufficient in light of their inherent variability. The reliability of Oedometer derived parameters (including C_c) is heavily dependent upon the appropriateness and quality of the sampling method used. Unfortunately, ideal sample retrieval standards are not always delivered, owing to the associated cost and a lack of appreciation of the impact of sample disturbance on laboratory derived parameters. Value of Compression Index (C_c) is different between each type of soil. Usually for the coarse-grained soil have a high permeability flow

occurs very rapidly, while for finer grained soil that has low permeability, the flow process takes a longer time (Bujang,K.Huat,Ahmed,Jusoh and Shukri maail, 1991).

2.4.3. Normally Consolidated and Over consolidated Clays

A clay is said to be normally consolidated if the present effective overburden pressure σ_o is the maximum pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is said to be over consolidated if the layer was subjected at one time in its history to a greater effective overburden pressure, σ_c , than the present

pressure, σ_o . The ratio σ_c / σ_o is called the over consolidation ratio (OCR). Over

consolidation of a clay stratum may have been caused due to some of the following factors:

(a) Weight of an overburden of soil which has eroded, (b) Weight of a continental ice sheet that melted and (c) Desiccation of layers close to the surface. Experience indicates that the natural moisture content, is commonly close to the liquid limit, (d) for normally consolidated clay soil whereas for the over consolidated clay, w_n is close to plastic limit.

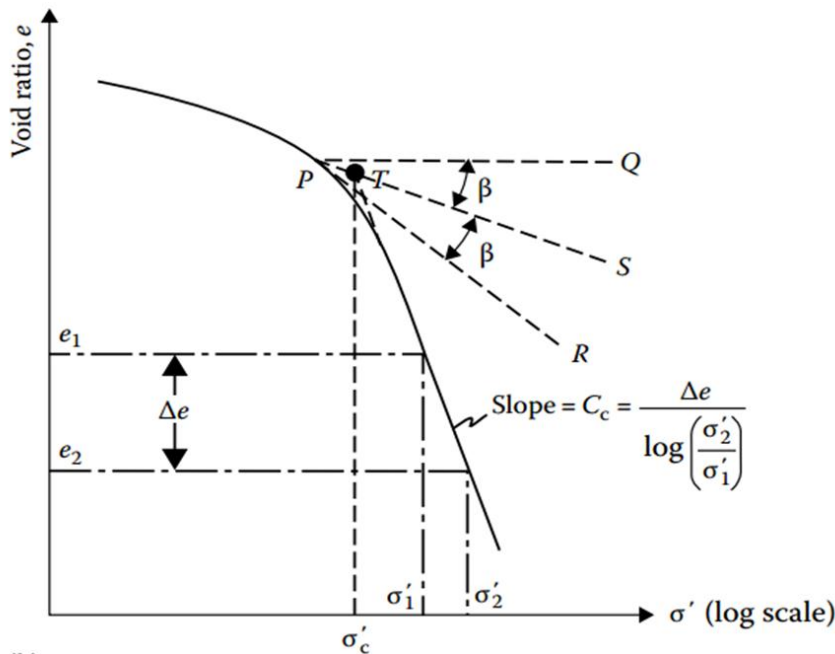


Figure 2.5. One Dimensional Compression pressure (Briaud J.L.2013).

The pre consolidation pressure σ_c is another important soil parameter that can be obtained from the consolidation test. It is the effective vertical stress before which the deformation of the soil is small and after which the deformation of the soil increases more rapidly. It can be thought of as a vertical yield stress, although failure does not necessarily

happen at σ_p . This effective stress corresponds to the highest long-term effective stress that the soil has been subjected to. It is recommended to obtain σ_p from the consolidation test (Figure 2.5).

2.5. EXISTING FINE GRAINED SOIL COMPRESSIBILITY PREDICTION MODELS

Several Linear and nonlinear empirical equations have been published that relate compression index (C_c) to soil index properties. Some equations are supposed to reflect C_c of all soils while others are limited to specific soil types and/or geography (Peck, R.B. and Reed W.C., 1954). The virgin compression behavior of clays is influenced by the several factors. For reconstituted clays the compression behavior is controlled primarily by the inherent characteristics of the clay, such as, the particle size-distribution, mineralogical composition, pore-water chemistry and particle arrangement (Terzaghi, K. Peck, R.B., 1967). The same factors influence the index parameters. Hence several attempts have been made in the literature to find suitable correlations between simple index parameters and the compressibility of clays under one-dimensional compression as quantified.

The statistical evaluation of soil properties has always attracted the interest of geotechnical engineers. The reason for this enthusiasm is the wish to establish correlations between various soil parameters in order to estimate the behavior of soil layers approximately without going through detailed testing and evaluation stages. It is useful to review some of the similar relationships proposed by (Brief note on the compression index of the soil mechanics and foundation division, 1956; regression analysis of soil compressibility in soil and foundation, 1976; Notes of the compressibility of clays, 1944; Terzaghi, K. Peck, R.B., 1967; Peck, R.B. and Reed W.C., 1954; The Genesis, Mineralogy and Identification of Expansive Soils, 1984; Danial Mohammadzadeh, 2019; Determination of Plasticity Index and Compression Index of Soil at Perlis, 2012) to evaluate the compression index in terms of index properties. In most cases, three basic index soil properties, namely, void ratio, water content, and liquid limit, are used to establish expressions to obtain the compression index. As expected, these proposed relationships differ from each other, as can be seen in Table 1.1, since they are based on different databases. In engineering practice, engineers often rely on empirical equations to estimate the C_c in terms of soil index properties because they are less expensive than consolidation tests and provide meaningful initial estimates for settlement of foundations. Such estimates

are only valid when dealing with normally consolidated fine-grained soils. The focus of this paper is on published linear empirical equations used to estimate C_c in terms of the index properties of the soil (A,AL,K,K,M,and Robert Jacobs, 2017).

Table 1.1. Compression index equations (Sridharan, et al., 2000).

Equation	Applicability	Reference
$C_c = 0.007(w_L - 10)$	Remolded clays	Skempton 1944
$C_c = 0.0046(w_L - 9)$	Brazilian clays	Cozzolino 1961
$C_c = 0.009(w_L - 10)$	Normally consolidated clays	Terzaghi and Peck 1967
$C_c = 0.006(w_L - 9)$	All clays with $w_L < 100\%$	Azzouz et al. 1976
$C_{c10} = 0.009(w_L - 8)$	Osaka Bay clay	Tsuchida 1991
$C_c = 0.01(w_n - 5)$	All clays	Azzouz et al. 1976
$C_c = 0.01w_n$	All clays	Koppula 1981
$C_c = 0.01(w_n - 7.549)$	All clays	Herrero 1983
$C_c = 0.0115w_n$	Organic silt and clays	Bowles 1989
$C_c = 1.15(e - e_o)$	All clays	Nishida 1956
$C_c = 0.29(e_o - 0.27)$	Inorganic soils	Hough 1957
$C_c = 0.246 + 0.43(e_o - 0.25)$	Mostley clays from Sao Paulo, Brazil	Cozzolino 1961
$C_c = 0.75(e_o - 0.50)$	Soils with low plasticity	Sowers 1970
$C_c = 0.156e_o + 0.0107$	All clays	Bowles 1989
$C_c = 0.5(\gamma_w / \gamma_d)^{2.4}$	All soil types	Herrero 1980
$C_c = 0.185[G_s(\gamma_w / \gamma_d)^2 - 0.144]$	All soil types	Herrero 1983
$C_c = 0.5.IP.G_s$	All remolded, normally consolidated clays	Wroth and Wood 1978
$C_c = 0.329[0.027(w - w_p) + 0.0133Ip(1.192 + ACT^{-1})]$	All remolded, normally consolidated clays	Carrier 1985
$C_c = 0.2343e_L$	All remolded, normally consolidated clays	Nagaraj and Srinivasa Murthy 1986
Note: ACT, activity; C_c , compression index; C_{c10} , compression index when consolidation pressure $p = 10 \text{ kg/cm}^2$; e , void ratio at a specific pressure; e_L , void ratio at liquid limit; e_o , initial or in situ void ratio; G_s , specific gravity; IP , plasticity index; w_L , liquid limit; w_n , natural water content; w , water content of the remolded soil considered; w_p , plastic limit; γ_d , dry unit weight of soil at which C_c is required; γ_w , unit weight of water.		

CHAPTER THREE

MATERIALS, METHODS AND PROCEDURES FOR LABORATORY TESTING, RESULTS AND DISCUSSION

3.1. INTRODUCTION

This chapter mainly provides the materials, methods, and procedures that are required for laboratory testing of the soils to generate the primary sources of data used for model development. In order to accomplish the objective of this thesis different field and laboratory works were conducted. All laboratory tests were conducted according to ASTM standards. Description of the study area, selection of sampling sites, sample collection; preparation of soil samples for different laboratory tests, and final summary of the test data will be focused in this part.

3.2. MATERIAL CHARACTERIZATION: MATERIALS, METHODS AND PROCEDURES

3.2.1. Description of the Study Area

Gondar Town, former capital of North-Western Ethiopia during the reign of Emperor Fasilidas (1632-1667), is located in the northwestern part of Ethiopia at a distance of 737 km from Addis Ababa, the national capital, 180 km north of Bahir Dar, the regional capital, and 250 km from Gedarif, the Sudanese border town. The city has a latitude and longitude of 12°35'N and 37°27'E respectively. The town is linked to a neighboring country Sudan via Metema and as a result expected as a promising center for transit of goods and services with Sudan. The total coverage area of the town is approximately 51.27 square kilometers. The town is endowed with many historical sites registered by UNESCO at international level and it provides good stimulus to the economy by attracting tourists to the area. Population Characteristics of Gondar town according to the 2007 National Census Report, which was compiled in the year 2008 the total population of the Town is 206,987 (CSA, 2007) and the average annual growth rate is 4.69%. The total population of the town in 2018 is 342,690 (Stotaw, 2019). Rainfall of Gondar is characterized as mono-modal type. The annual rainfall varies from 711.8 to 1822.42 mm with a mean annual value of 1200mm. Long-term distribution of rainfall data indicates that most of the rain occurs in July followed by August. The rainfall in May and June is also quite significant.

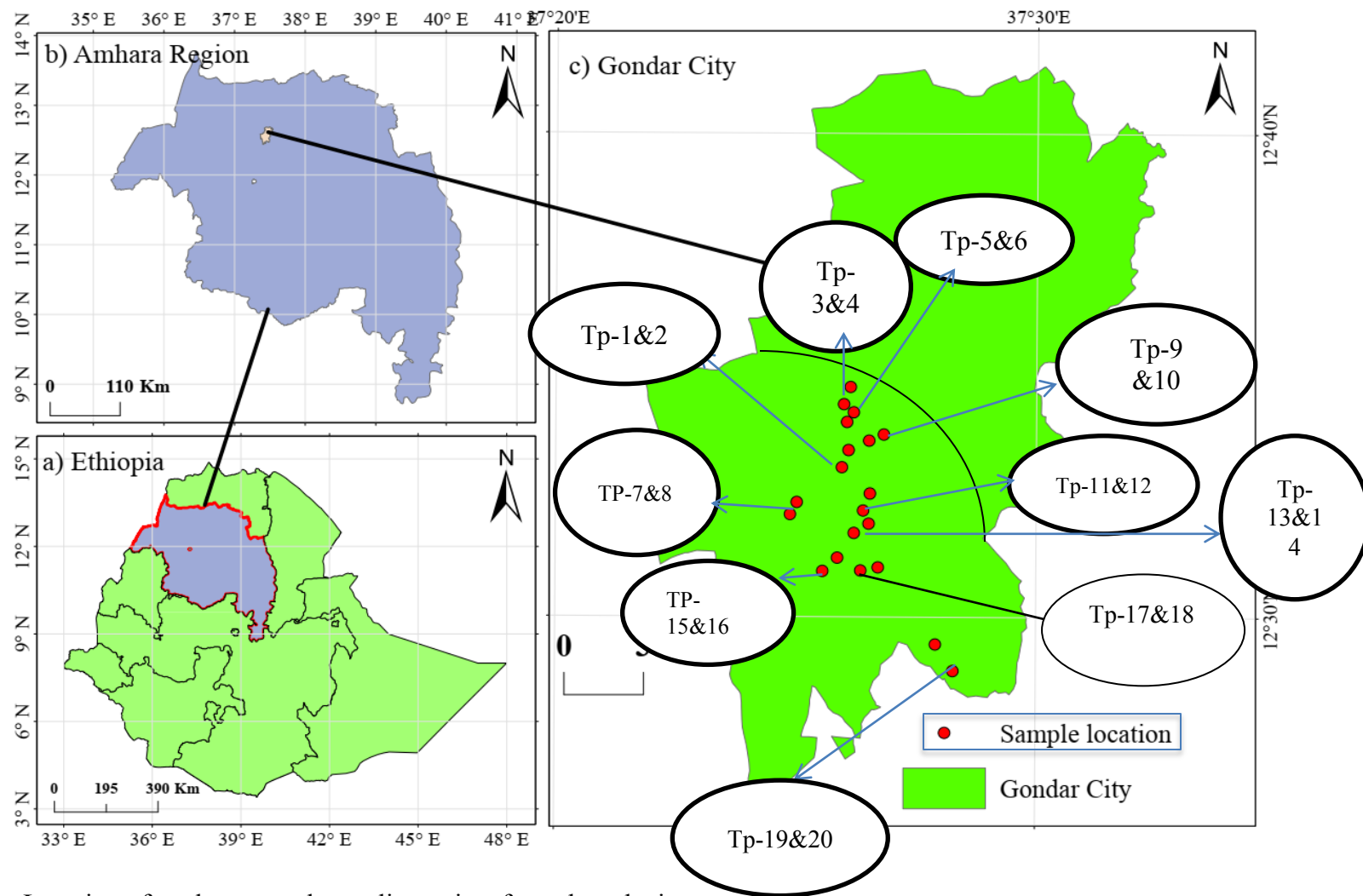


Figure 3.1. Location of study area and sampling points from the sub city

The mean annual temperature in Gondar Town varies between 16°C and 20°C, which makes it in WeinaDega zone. Maximum temperature occurs in March and April and minimum temperatures are at their lowest in November to February (GTWSSS, 2017). (Tiru, 2018).The location of collected soil samples site is shown with the aid of map in Figure 3.1.

3.2.2. Soil Sampling and Collection

3.2.2.1. Selection of Sampling Sites/Location

When selecting possible sampling sites, the major factor considered extremely important was that the site to be studied should be represented properly. To ensure samples covered a sufficient area of this formation, sites were identified by visual investigation during field visit of the soils in the study area. In order to have sufficient and reliable data for the analysis, representative soils are sampled from each test pit area. The representative samples are collected from Azezo Industry Zone (i.e., 1, 2, 3, and 4) and Steda, Loza-Mariam, Aba samuale Keble, Dashen-Brewery, Ayera and Azezo Condominium. A total of 20 disturbed and undisturbed samples were collected within a reasonable sampling interval at about 2.5-3m depth location. Typical onsite soil sampling site photo is indicated in Figure 3.2.

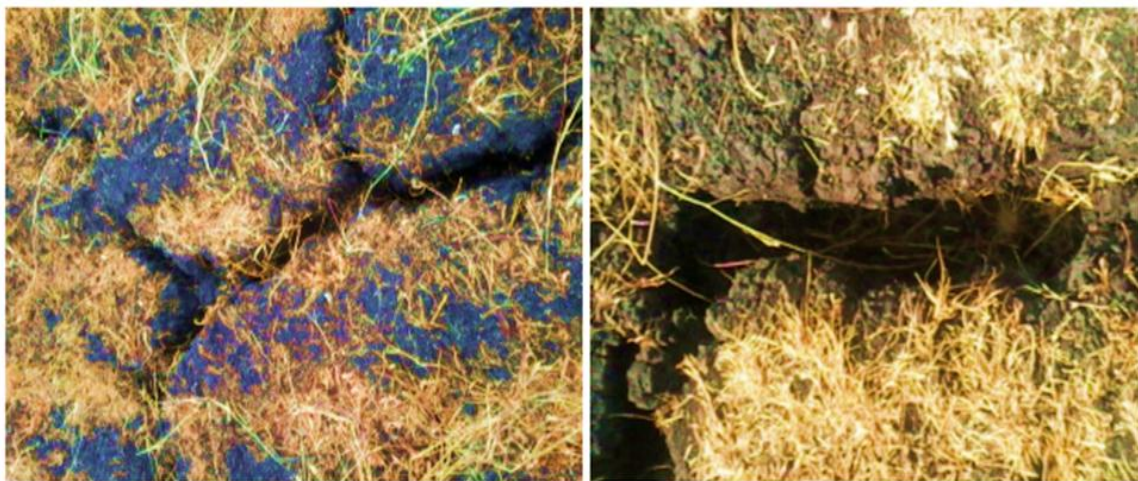


Figure 3.2. Typical photo of soil sampling points from the *Subcity*.

3.2.2.2. Disturbed Soil Sample Collection

The disturbed soil (N=20) was placed in tight plastic bag with a reference tag to describe the location of the sample and the depth (2.5-3m) taken. Materials required are: soil corer or spade; Buckets; New plastic bag or sample container (supplied in soil kit); Labels if more than one sample is collected, Record sheet to record sample site and sample. Remember a soil test is only as good as the care taken in sampling. Tools and equipment should be clean prior to sampling.

3.2.2.3. Undisturbed Soil Sample Collection

Undisturbed soil samples (N=20) for this research were recovered by applying static force using hydraulic jack and pressing a Shelby tube. The sampling tube was then removed from the hole and the ends of the Shelby were immediately wax sealed and covered with plastic bags and labeled with necessary information for subsequent laboratory testing. All samples that were collected, whether they comprised disturbed or undisturbed samples, were labeled appropriate and given a sample number prior to any preparation work on it.

3.2.3. Preparation of Disturbed Soil Samples

3.2.3.1. Types of Laboratory tests and Test Standards Used

The disturbed samples were used to determine the Atterberg Limits, Specific Gravity and Particle Size Distribution using both sieve and hydrometer methods. The undisturbed soil samples were used to determine the natural water content and the consolidation tests. First the soil sample was dried and reduces the clods in the sample to pass through a 10mm sieve. To achieve the required size, the clods were initially chopped up with a shovel and then made to crumble by hammer. During this process of chopping and crumbling extreme care was taken to ensure that individual particles were not crushed. In this thesis all laboratory testing was undertaken according to America Standard for Material Testing (ASTM) standard.

3.2.3.2. Preparation of soil for Liquid Limit, Plastic Limit

The method adopted for both of these tests was the dry preparation method using a mechanical device. The sample that has been rubbed down is then sieved through a 425 μ m sieve. After the entire sample has been sieved, the material which has passed through this sieve take a mass of at least 200grams so both tests can be performed.

3.2.3.3. Preparation of soil for Specific Gravity Test

The sample that has been rubbed down is then sieved through a 2mm sieve. After the entire sample has been sieved, the material which has passed through this sieve is oven dried and then weighs 25 grams of sample from oven dried for test.

3.2.3.4. Preparation of the soil for Particle Size Distribution test using wet sieves

For fine-grained soils that may contain high fraction of clay soils, it was necessary to soak the soil prior to washing in order to keep the fine material from adhering to larger particles. The representative sample was measured before soaking and washing through the 75 μ m sieve. After washing, the material which has retained on 75 μ m sieve is oven dried and used for mechanical sieve.

3.2.3.5. Preparation of the soil for Hydrometer Test

The sample that has been rubbed down is then sieved through a 75 μ m sieve. After the entire sample had been sieved, 50gm of the material which has passed through 75 μ m sieve is used

3.2.3.6. Preparation of Soil Oedometer Test

The one-dimensional consolidation test is performed by placing a cylindrical specimen of undisturbed fine-grained soil in a consolidation cell. With regard to specimen dimensions, (ASTM-D2435, 1993) specifies that 1) the minimum height and diameter is 0.5 in. and 2.00 in., respectively, 2) the height must exceed 10 times the maximum particle size, and 3) the diameter: height ratio must exceed 2.5. For consolidation testing, it is generally desirable that the applied pressure at any loading stage be double than that at the preceding stage. The test may, therefore, be continued using a loading sequence which would successively apply stress of 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, etc., kgf/cm² on the soil specimen (IS 2720, 2016). For each loading increment, after application of load, readings of the dial gauge shall be taken using a time sequence such as 0, 0.25, 1, 2.25, 4, 6.25, 9,

12.25, 16, 20.25, 25, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225, etc., min, up to 24 hour(s) or 0, 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60 min, and 2, 4, 8, 24 hour(s). These time sequences facilitate plotting of thickness or change of thickness of specimen against square root of time or against logarithm of time (ASTM, 1997).

3.3. LABORATORY TEST: RESULTS AND DISCUSSION

3.3.1. Description of Study Site and Sampling Locations

The location name, the test pits designation, colors of the soil and the depth where the samples were taken is shown in Table 3.1. The field visualization of soils indicate that the soils are reddish to grayish in color. The depth where all the soils are sampled are between 2.5-3m.

3.3.2. Index Properties of Soils Studied

Index property is a property, which helps in distinguishing the characteristics of a soils. Soil grain property and soil aggregate property are two main categories under this term. Soil grain property is based on the individual grains and depends on size, shape and mineralogical characteristics. Soil aggregate property, on the other hand is based on the property of the soil mass as a whole. Atterberg limit test, grain size analysis, specific gravity and hydrometer are among the tests. Engineering properties, identification, and classification of natural soil are presented in this section

3.3.2.1. Results of Water Content Test of the soils

The laboratory determinations are made on the recovered soil samples to obtain the natural water Content (w). The different tests that demand a water content test include Liquid (LL) and plastic (PL) tests that are commonly made on cohesive soils. The results of water content test are presented in Table 3.2. The water content test results suggest that the minimum of 43.53%, a maximum of 62.12%, range of 18.59%, mean of 51.28% with the standard deviation of 1.2. These suggest that the soil water holding capacity of the soil is large indicating that the soil in the study area is Silty/clayey.

Table 3.1. Description of test pits location.

S.No.	Location	Test Pit Id	Color	Depth(m)	Easting	Northing
1	Ayera	TP-1	Red	2.5-3	330745	1389062
2	Ayera	TP-2	Yellowish red	2.5-3	331299	1389298
3	Dashen	TP-3	Brownish gray	2.5-3	329798	1390454
4	Dashen	TP-4	Brownish gray	2.5-3	330052	1391108
5	Abasamual	TP-5	Yellowish gray	2.5-3	329913	1389779
6	Abasamual	TP-6	Reddish gray	2.5-3	330167	1390144
7	Loza	TP-7	Reddish	2.5-3	327759	1386266
8	Loza	TP-8	Reddish gray	2.5-3	328013	1386726
9	Condominium	TP-9	Whitish gray	2.5-3	329715	1388052
10	Condominium	TP-10	Whitish gray	2.5-3	329969	1388704
11	Industry area -1	TP-11	Dark gray	2.5-3	330514	1386397
12	Industry area -1	TP-12	Dark gray	2.5-3	330768	1387049
13	Industry area -3	TP-13	Dark gray	2.5-3	330164	1385546
14	Industry area -3	TP-14	Dark gray	2.5-3	330718	1385898
15	Industry area -4	TP-15	Yellowish gray	2.5-3	328976	1384098
16	Industry area -4	TP-16	Yellowish gray	2.5-3	329530	1384603
17	Industry area -2	TP-17	Yellowish gray	2.5-3	330408	1384113
18	Industry area -2	TP-18	Yellowish gray	2.5-3	331062	1384231
19	Tseda	TP-19	Yellowish red	2.5-3	333873	1380278
20	Tseda	TP-20	Yellowish red	2.5-3	333219	1381290

Table 3.2. Results of Water content test.

<i>Test Pit</i>	<i>Depth (m)</i>	<i>Natural Water Content (%)</i>
TP-1	2.5 -3	56.13
TP-2	2.5 -3	58.00
TP-3	2.5 -3	62.12
TP-4	2.5 -3	56.00
TP-5	2.5 -3	48.68
TP-6	2.5 -3	53.00
TP-7	2.5 -3	61.87
TP-8	2.5 -3	55.00
TP-9	2.5 -3	45.56
TP-10	2.5 -3	50.00
TP-11	2.5 -3	49.81
TP-12	2.5 -3	51.00
TP-13	2.5 -3	43.53
TP-14	2.5 -3	48.67
TP-15	2.5 -3	45.33
TP-16	2.5 -3	46.43
TP-17	2.5 -3	48.37
TP-18	2.5 -3	50.00
TP-19	2.5 -3	46.1
TP-20	2.5 -3	50.00

3.3.2.2. Results for the Atterberg Limit Test

The results of Atterberg limit test is given in Table 3.3. ASTM D-4318 is followed for the testing procedure. Liquidity Index (LI): The Atterberg limits are found using disturbed soil samples. These limits as such do not indicate the consistency of undisturbed soils. The index that is used to indicate the consistency of undisturbed soils is called as the liquidity index or water plasticity ratio as computed Table 3.3. Table 3.4 indicates the descriptive statistics summary of the results of the Atterberg Limit tests. The results of Atterberg limit test suggest that (on average), the soils in the study area exhibits a liquid limit of 53.23%, a plastic limit of 29.8% and a plasticity index of 25%. Low PI values (25 or less) are associated with a predominance of larger clay particles, such as in kaolinite (Briaud, 2013). Liquid limit less than 55% indicates the soil is having an intermediate plasticity (Roy.whitlow, 1995). Hence, these values indicate that the soil is

Clay having an Intermediate Plasticity. Moreover based on the mean liquidity index value of less than 1, the soil is in liquid state. The detailed Atterberg limit test results are attached in Appendix A. Table 3.3. Results of the Atterberg Limit Tests

<i>Test Pit</i>	<i>Depth (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>	<i>Liquidity Index</i>
TP-1	2.5 -3	64	37	27	0.71
TP-2	2.5 -3	48	27	21	1.48
TP-3	2.5 -3	58	33	25	1.16
TP-4	2.5 -3	59	30	29	0.90
TP-5	2.5 -3	52	28	24	0.86
TP-6	2.5 -3	55	29	26	0.92
TP-7	2.5 -3	54	27	27	1.29
TP-8	2.5 -3	53	27	26	1.08
TP-9	2.5 -3	63	34	29	0.40
TP-10	2.5 -3	57	31	26	0.73
TP-11	2.5 -3	52	31	21	0.90
TP-12	2.5 -3	61	32	29	0.66
TP-13	2.5 -3	50	27	23	0.72
TP-14	2.5 -3	55	29	26	0.76
TP-15	2.5 -3	45	25	20	1.02
TP-16	2.5 -3	51	26	25	0.82
TP-17	2.5 -3	59	33	26	0.59
TP-18	2.5 -3	52	28	24	0.92
TP-19	2.5 -3	64	36	28	0.36
TP-20	2.5 -3	54	27	27	0.85

Table 3.4. The descriptive statics summary of the results of the Atterberg Limit Tests

<i>Statistics</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>	<i>Liquidity Index</i>
Min	45	25	20	0.36
Max	64	37	29	1.48
Range	19	12	9	1.12
Mean	55.3	29.85	25.45	0.8565
Std Err	1.1790	0.7618	0.5915	0.0605
Median	54.5	29	26	0.855

3.3.2.3. Results for the Specific Gravity Test

Specific gravity of soil is the ratio of the unit weight of solids in the soil to the unit weight of water. The laboratory determinations were made on the recovered soil samples to obtain the Specific Gravity value. The specific gravity value is used to compute particle size tests. The results of specific gravity test are presented in Table 3.5. The Specific Gravity test results suggest that a mean G_s value of 2.69 with the standard deviation of 0.07. Since the mineralogy of clay is more variable, G_s for clay is more variable. As it is indicated in this study, is often to be somewhere between 2.70 and 2.80 depending on mineralogy.

Table 3.5. Results of Specific Gravity (G_s) test

<i>Test Pit</i>	<i>Depth (m)</i>	<i>Specific Gravity (G_s)</i>
TP-1	2.5 -3	2.80
TP-2	2.5 -3	2.76
TP-3	2.5 -3	2.70
TP-4	2.5 -3	2.71
TP-5	2.5 -3	2.56
TP-6	2.5 -3	2.70
TP-7	2.5 -3	2.70
TP-8	2.5 -3	2.69
TP-9	2.5 -3	2.81
TP-10	2.5 -3	2.72
TP-11	2.5 -3	2.65
TP-12	2.5 -3	2.68
TP-13	2.5 -3	2.57
TP-14	2.5 -3	2.70
TP-15	2.5 -3	2.48
TP-16	2.5 -3	2.65
TP-17	2.5 -3	2.72
TP-18	2.5 -3	2.70
TP-19	2.5 -3	2.65
TP-20	2.5 -3	2.72

3.3.2.4. Results for the Grain Size Analysis Test (Sieve + Hydrometer Analysis)

As mentioned earlier, particle size is not the main factor controlling the behavior of silts and clays. Instead, the behavior and therefore the classification are based on the ability of the soil to be deformed and stay together (consistency). The purpose of these tests is to determine the grain size distribution (i.e.; grain size versus percent by weight) of soil, and

to determine the percentage of fines (i.e.; material passing the No.200 sieve) in soil. This information is used to classify the soil in accordance with the Unified Soil Classification System (USCS). In some instances, the gradation curve cannot be reliably quantified at smaller grain sizes (less than a millimeter) using sieves because the smaller clay particles in soil form clods and cannot pass through the screens individually. However, this portion of the gradation curve can be defined using a hydrometer analysis. The material that passed the #40 sieve during the mechanical sieve analysis is used to perform the hydrometer analysis. As shown in Table 3.6 the %Passing #200 Sieve Size on average is 71.4% and the minimum % passing through No. 200 sieve is 59%, this suggests the soil is fine grained according to USCS.

Table 3.6. Results of Grain/Particle Size Analysis Test

<i>Test Pit</i>	<i>Depth (m)</i>	<i>%Passing #200 Sieve Size</i>
TP-1	2.5 -3	81
TP-2	2.5 -3	85
TP-3	2.5 -3	69
TP-4	2.5 -3	75
TP-5	2.5 -3	59
TP-6	2.5 -3	67
TP-7	2.5 -3	67
TP-8	2.5 -3	68
TP-9	2.5 -3	83
TP-10	2.5 -3	75
TP-11	2.5 -3	83
TP-12	2.5 -3	89
TP-13	2.5 -3	68
TP-14	2.5 -3	72
TP-15	2.5 -3	70
TP-16	2.5 -3	72
TP-17	2.5 -3	70
TP-18	2.5 -3	70
TP-19	2.5 -3	70
TP-20	2.5 -3	67

3.3.2.5. USCS Classification of Soils

ASTM D2487 standard practice for classification of soils for engineering purposes (Unified Soil Classification System) is used. Soil is classified by geotechnical engineers for engineering purposes in accordance with the Unified Soil Classification System (USCS).

Soils sharing a common USCS classification possess similar engineering properties, including strength, permeability, and compressibility, so the USCS is useful for specifying soil types to achieve a desired performance. To use the USCS, information regarding the liquid and plastic limits and gradation of the soil is required. Using the USCS, each soil is assigned a two-letter group symbol and a group name. Silts and Clays are symbolized as: ML, CL, CH, MH, OH, and OL. Under the USCS, there is no direct distinction between silts and clays, although clay particles are smaller than silt particles and are mineralogical different than silt particles. Silts and clays are indirectly distinguished in the USCS through the use of liquid and plastic limits as described later. Table 3.7 indicates the data used for USCS classification plasticity chart of the fine grained soils Figure 3.3 indicates the USCS Classification Plasticity Chart of the Fine Grained soils studied. The USCS indicates the soils are Clay with high and low plasticity and some of the soils are elastic Silt.

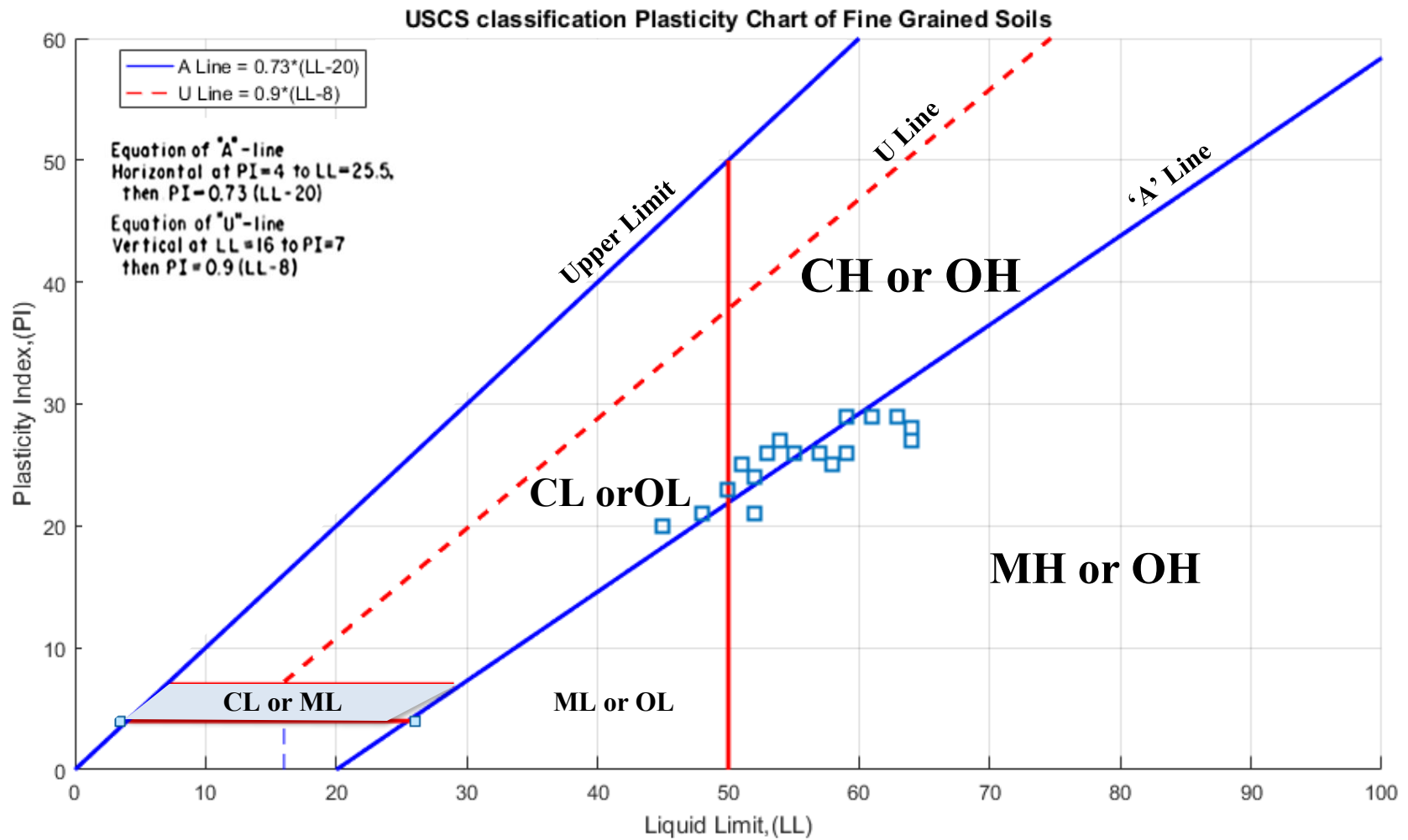


Figure 3.3. USCS Classification Plasticity Chart of Fine Grained Soils/Results of this study.

Table 3.7. Data used for USCS Classification Plasticity Chart of the Fine Grained Soils

<i>Test Pit</i>	<i>Depth (m)</i>	<i>Liquid Limit (%)</i>	<i>Plastic Limit (%)</i>	<i>Plasticity Index (%)</i>	<i>Liquidity Index</i>	<i>%Passing #200 Sieve Size</i>	<i>Remark</i>	<i>USCS</i>
TP-1	2.5 -3	64	37	27	0.71	81	Liquid	MH
TP-2	2.5 -3	48	27	21	1.48	85	Plastic	CL
TP-3	2.5 -3	58	33	25	1.16	69	Plastic	MH
TP-4	2.5 -3	59	30	29	0.9	75	Plastic	CH
TP-5	2.5 -3	52	28	24	0.86	59	Liquid	CH
TP-6	2.5 -3	55	29	26	0.92	67	Plastic	CH
TP-7	2.5 -3	54	27	27	1.29	67	Plastic	CH
TP-8	2.5 -3	53	27	26	1.08	68	Plastic	CH
TP-9	2.5 -3	63	34	29	0.4	83	Liquid	MH
TP-10	2.5 -3	57	31	26	0.73	75	Liquid	MH
TP-11	2.5 -3	52	31	21	0.9	83	Liquid	MH
TP-12	2.5 -3	61	32	29	0.66	89	Liquid	CH
TP-13	2.5 -3	50	27	23	0.72	68	Liquid	CL
TP-14	2.5 -3	55	29	26	0.76	72	Liquid	CH
TP-15	2.5 -3	45	25	20	1.02	70	Plastic	CL
TP-16	2.5 -3	51	26	25	0.82	72	Liquid	CH
TP-17	2.5 -3	59	33	26	0.59	70	Liquid	MH
TP-18	2.5 -3	52	28	24	0.92	70	Liquid	CH
TP-19	2.5 -3	64	36	28	0.36	70	Liquid	MH
TP-20	2.5 -3	54	27	27	0.85	67	Liquid	CH

3.3.3. One Dimensional Consolidation Test Result

When a layer of fine-grained soil, including ML, CL and CH, is subjected to an increase in effective stress through an increase overburden stress, the soil undergoes a long-term reduction in void ratio, e , which is accompanied by settlement of the soil layer. To quantify both the ultimate amount of settlement and the time rate of settlement in the soil layer, a one-dimensional consolidation test is performed in the laboratory. Using laboratory-derived parameters, field settlement behavior of the soil layer can be predicted. The one-dimensional consolidation test is performed by placing a cylindrical specimen of undisturbed fine-grained soil in a consolidation cell. Table 3.8 shows the One Dimensional Consolidation test result of the fine grained soils studied.

With regard to specimen dimensions, ASTM D2435 specifies that 1) the minimum height and diameter is 0.5 in. and 2.00 in., respectively, 2) the height must exceed 10 times the maximum particle size, and 3) the diameter: height ratio must exceed 2.5. After conducting the consolidation tests, the e - σ' data pairs are then used to plot the e - $\log \sigma'$ curve. The e - $\log \sigma'$ curve is bilinear, with the flatter portion corresponding to recompression at lower stresses, and the steeper portion corresponding to virgin consolidation at higher stresses. The e - $\log \sigma'$ curve is used to determine the compression index, C_c . The compression index which is used to estimate ultimate settlement. The compression and recompression indices are the slopes of the two portions of the curve. To calculate these parameters, two points are selected along a linear section of each portion the curve. The two points possess void ratios e_1 and e_2 , and stresses σ_1 and σ_2 respectively, and are selected. Compression index are then expressed as:

$$C_c = \frac{\Delta e}{\Delta \log \sigma'}, \quad (3.1)$$

Consolidation occurs in soil due to long-term increases in effective stress. Increasing the overburden is one mechanism that increases effective stress by increasing total stress. However, lowering the water table also increases effective stress by reducing pore water pressure. In some areas where groundwater pumping has been extensive, large urban areas have undergone significant settlement due to an increase in effective stress. For large construction projects in congested downtown areas where the water table is near the ground surface and groundwater must be pumped to drain the basement for excavation, settlement of adjacent structures may be significant, and mitigating measures may be warranted. Several sources may affect the values such as disturbance.

Table 3.8. One dimensional consolidation test result of the fine grained soils studied

<i>Test Pit</i>	<i>Depth (m)</i>	<i>Natural Water Content (ω) (%)</i>	<i>Compression Index (C_c)</i>
TP-1	2.5 -3	56.13	0.422
TP-2	2.5 -3	58.00	0.26
TP-3	2.5 -3	62.12	0.463
TP-4	2.5 -3	56.00	0.52
TP-5	2.5 -3	48.68	0.331
TP-6	2.5 -3	53.00	0.42
TP-7	2.5 -3	61.87	0.56
TP-8	2.5 -3	55.00	0.47
TP-9	2.5 -3	45.56	0.288
TP-10	2.5 -3	50.00	0.37
TP-11	2.5 -3	49.81	0.254
TP-12	2.5 -3	51.00	0.40
TP-13	2.5 -3	43.53	0.269
TP-14	2.5 -3	48.67	0.41
TP-15	2.5 -3	45.33	0.191
TP-16	2.5 -3	46.43	0.34
TP-17	2.5 -3	48.37	0.391
TP-18	2.5 -3	50.00	0.34
TP-19	2.5 -3	46.1	0.293
TP-20	2.5 -3	50.00	0.41

The results One Dimensional Consolidation test is used to determine the compression index of the fine grained soils and this result indicated on Table 3.8. The compression index results on average suggest 0.51 with a standard error of 0.01. Figure 3. 4 shows typical Consolidation test results of e-log σ curve.

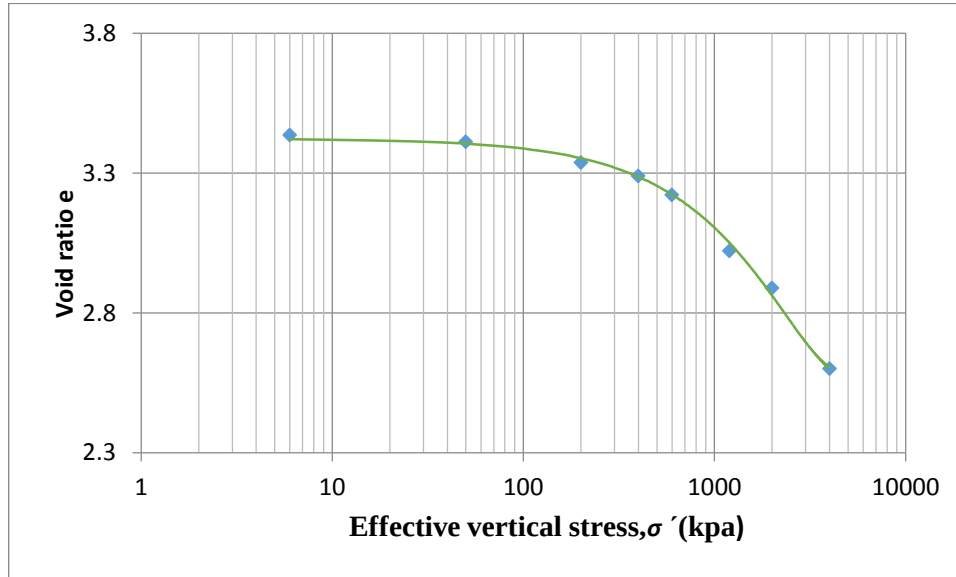


Figure 3.4. Typical consolidation test results e-log σ curve

Table 3.9. Descriptive statistics summary of the laboratory test data.

Parameter	Cc	Wn (%)	Gs	LL (%)	PL (%)	PI (%)	%P₂₀₀	LI
Mean	0.37	51.28	2.6835	55.3	29.85	25.45	73	0.8565
StdDev	0.095	5.3565	0.0776	5.2726	3.4070	2.6453	7.5394	0.2703
Min	0.191	43.53	2.48	45	25	20	59	0.36
Max	0.56	62.12	2.81	64	37	29	89	1.48
Range	0.369	18.59	0.33	19	12	9	30	1.12
Std Err	0.021	1.1977	0.0174	1.1790	0.7618	0.5915	1.6859	0.0605
Median	0.381	50	2.7	54.5	29	26	70	0.855

Table 3.10. Summary of the laboratory test result data

<i>Test Pit</i>	<i>Cc</i>	<i>Wn(%)</i>	<i>Gs</i>	<i>LL (%)</i>	<i>PL (%)</i>	<i>PI (%)</i>	<i>%P200</i>	<i>LI</i>	<i>Remark</i>	<i>USCS</i>
TP-1	0.422	56.13	2.80	64	37	27	81	0.71	Liquid	MH
TP-2	0.26	58.00	2.76	48	27	21	85	1.48	Plastic	CL
TP-3	0.463	62.12	2.70	58	33	25	69	1.16	Plastic	MH
TP-4	0.52	56.00	2.71	59	30	29	75	0.90	Plastic	CH
TP-5	0.331	48.68	2.56	52	28	24	59	0.86	Liquid	CH
TP-6	0.42	53.00	2.70	55	29	26	67	0.92	Plastic	CH
TP-7	0.56	61.87	2.70	54	27	27	67	1.29	Plastic	CH
TP-8	0.47	55.00	2.69	53	27	26	68	1.08	Plastic	CH
TP-9	0.288	45.56	2.81	63	34	29	83	0.40	Liquid	MH
TP-10	0.37	50.00	2.72	57	31	26	75	0.73	Liquid	MH
TP-11	0.254	49.81	2.65	52	31	21	83	0.90	Liquid	MH
TP-12	0.40	51.00	2.68	61	32	29	89	0.66	Liquid	CH
TP-13	0.269	43.53	2.57	50	27	23	68	0.72	Liquid	CL
TP-14	0.41	48.67	2.70	55	29	26	72	0.76	Liquid	CH
TP-15	0.191	45.33	2.48	45	25	20	70	1.02	Plastic	CL
TP-16	0.34	46.43	2.65	51	26	25	72	0.82	Liquid	CH
TP-17	0.391	48.37	2.72	59	33	26	70	0.59	Liquid	MH
TP-18	0.34	50.00	2.70	52	28	24	70	0.92	Liquid	CH
TP-19	0.293	46.1	2.65	64	36	28	70	0.36	Liquid	MH
TP-20	0.41	50.00	2.72	54	27	27	67	0.85	Liquid	CH

CHAPTER FOUR

STATISTICAL MODEL DEVELOPMENT SETUP AND DATA ANALYSIS APPROACHES

4.1. INTRODUCTION

4.1.1. Primary and Secondary Sources of Data

All the required laboratory tests of 20 soil samples excavated from the study site were first carried out. These tests include water-content, Specific Gravity, the Atterberg limits, Particle size distribution, and consolidation characteristics. In addition, secondary fine-grained soil test data containing similar property of the different tests conducted in this thesis are carefully selected from literature are used for validation and calibration of the newly developed model. The stepwise subset selection multiple regression routines of *JMP-13* statistical software developed by SAS Institute Inc. in 2016 was used to undertake all the required statistical analysis. The stepwise regression control panel is used to limit regress or effect probabilities, determine the method of selecting effects, begin or stop the selection process, and run a model. However, prior to the predictive model development using *JMP-13* statistical software, some statistical pre-analysis and model setup is required in this part of the thesis. Moreover, in this chapter, the results of the summery of laboratory tests and the classification of natural soils tests are presented and the discussion related to the result are presented.

4.1.2. Approaches used for Model Development

Multiple Regression analysis is a statistical technique that is very useful in the field of engineering and science in modeling and finding relationships between two or more variables. Multiple regressions with subset selection and interaction analysis routines of *JMP-13* statistical software was used in the analysis. The method of subset selection multiple regression analysis is used to develop the line or curve which provides the best fit through a set of data points. A Regression Model that contains more than one predictor variable is called Multiple Regression Models. The multiple regression equations can take two forms: these are model with intercept and model without intercept.

In *JMP*, stepwise regression is a section of the Fit Model platform. The Stepwise feature computes estimates that are the same as those of other least squares platforms, but it

facilitates searching and selecting among many models. Using the Fit Model platform, one can specify complex models tests efficiently. The task is simplified by Macros, Attributes, and transformations. Fit Model is the gateway to fitting a broad variety of models and effect structures. These include: simple and multiple linear regression, analysis of variance and covariance and response screening, for studying a large number of responses.

The Fit Model platform gives an efficient way to specify models that have complex effect structures. These effect structures are linear in the predictor variables. Once the model specified, one can select the appropriate fitting technique from a number of fitting personalities. Once you choose the option, the Fit Model window provides choices that are relevant for the chosen model. Fit Model can be used to specify a wide variety of models that can be fit using various methods. Standard Model Types lists some typical models that can be defined using Fit Model. Eqn.(4.1) and model without intercept in Eqn (4.2):

$$y = b_o + b_1x_1 + b_2x_2 + L L + b_nx_n + c \quad (4.1)$$

$$y = b_1x_1 + b_2x_2 + L L + b_nx_n + c \quad (4.2)$$

Where the ‘b’s are the regression coefficients, representing the amount the dependent variable changes when the independent changes 1 unit. The variable c is a constant, where the regression line intercepts the y axis; representing the amount the dependent y will be c when all the independent variables are fitting linear relationship.

A linear relationship is usually practiced in solving different engineering problems because of its simplicity. However, the reality for soils is totally different. To develop the model, two groups of variables are set out. The compression index test results obtained from the corresponding test approaches are categorized as a dependent variable and all other parameters from index property test results are categorized under independent variables. These dependent and the independent variables have to be checked first for the normality of the corresponding data before the regression analysis is carried out to consider as a variable in the model. Therefore, first the geotechnical parameter/data obtained from each test sample need to be checked for its normality by using JMP-13 software.

If the test data of the corresponding tests shows normal distribution or the residuals of the fit model obtained are normally distributed, then, regression will be accepted. If the residuals of the fit model obtained are not normally distributed, data transformation is performed until the corresponding data became normally distributed. Details of checking normality of the data will be discussed in the data analysis and model development section of this chapter.

4.2. STATISTICAL REGRESSION ANALYSIS

4.2.1. Model Variables

4.2.1.1.Independent Variable

In this study, the intention is to develop a compression index statistical predicative model from index property test data of fine-grained soils. The index properties of soils show that, all the soils used for developing the models are Fine-Grained Soils. Therefore, the independent variables used include: water-content, Specific Gravity, the Atterberg limits, Liquidity Index and Particle size distribution.

4.2.1.2.Dependent Variables Cc

The compression index values obtained from tests are used dependent variable.

4.2.2. Summary of Data used for Model Development

Primary data for both the tests are analyzed for 20 samples. All soil samples were collected from Gondar town in Azezo Tseda sub city. All the laboratory tests of the fine grained soils were performed according to ASTM testing standard.

The laboratory test results have been used for regression analysis and validation of the modeled equation. The summery of all primary data is shown in Table 3.10. Moreover, all other data are indicated in Appendix A.

4.2.3. Model Setup

4.2.3.1. Assumptions: Used in the Model Development

Estimation of the model parameters and finding the best predictor variable that can estimate the desired parameter (in this case compression Index) with the highest quality requires several assumptions.

The normality of the data used in the model development and the normality of the residual obtained from the model fit is one. Moreover, the numbers of variables to be included in the model need to be minimum in number; in this study it is limited to a maximum of three. Moreover, these parameters included in the model need to be easily measurable index properties. The other assumption required is that, the residuals (i.e. 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 (ϵ) will be unknown and must be estimated from the sample data. Plot of residuals versus fitting values, if the model is correct and if the assumptions are satisfied, the residuals should be structure less; in particular, they should be unrelated to any other variable including the predicted response. A simple check is to plot the residuals versus the fitted values. Plots of residuals versus other variables: if data have been collected on any other variables that might possibly affect the response, the residuals should be plotted against these variables. Patterns in such residual plots imply that the variable affects the response. This suggests that the variable should be either controlled more carefully in future experiments or included in the analysis. In general, Multiple linear regression is based on the following assumptions: A linear relationship between the dependent and independent variables; the independent variables are not very highly correlated with each other; the variance of the residuals is constant; independence of observation (the model assumes that the observations should be independent of one another. Simply put, the model assumes that the values of residuals are independent. To test for this assumption, we use the Durbin Watson statistic); Multivariate normality, Multivariate normality occurs when residuals are normally distributed. To test this assumption, look at how the values of residuals are distributed. It can also be tested using two main methods, i.e., a histogram with a superimposed normal curve or the Normal Probability Plot method. Other statistical goodness of fit measures needs to be with the

required range, such as: RMSE, R^2 , Adj R^2 , Coefficient of Variation, Mean Square Error BIC and AIC.

4.2.3.2. One, Two and Three Parameter Models

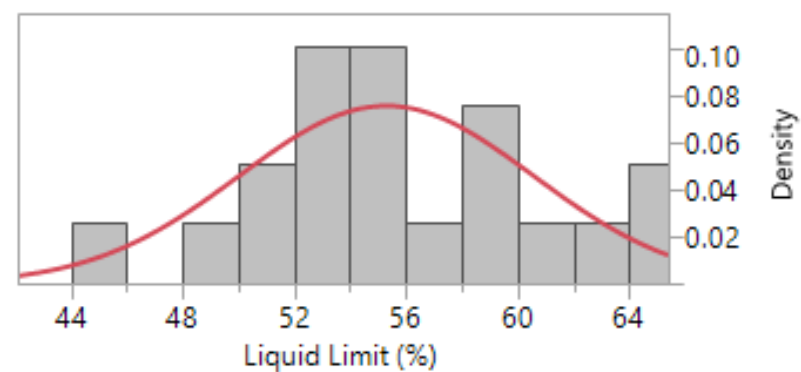
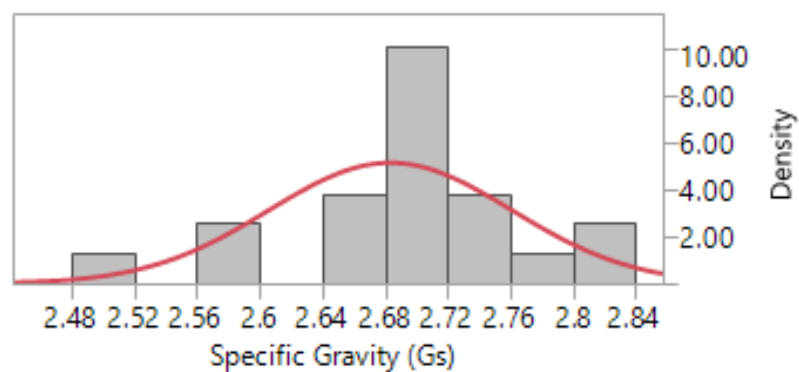
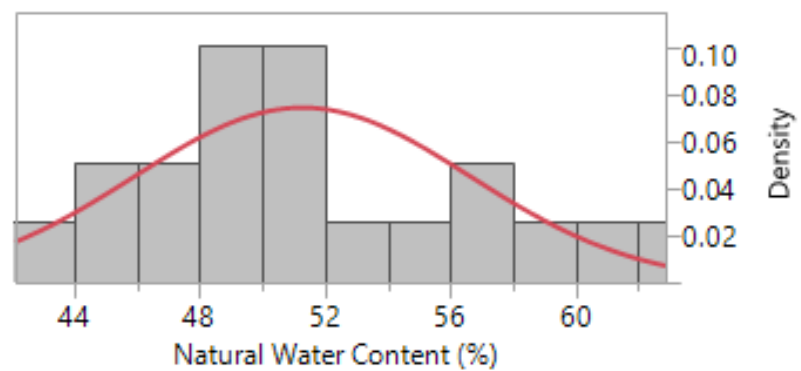
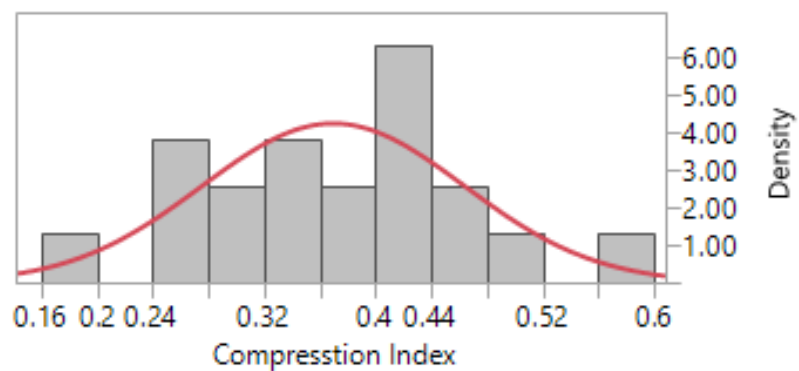
In order to correlate the Compression Index (Cc) with its index properties, the dependent and independent variables must be specified. Hence, the Compression Index (Cc) was set as a dependent variable (y_i) and the index properties were set as an independent variable (x_i) for any trial. Having specified the dependent variable and the independent variable, the normality of the data was checked. For data that were not normal, data transformation will be carried. The transformation generates a new data. Therefore, in the model developed, the number of parameters included in the model can be one, two or three. However increasing the number of parameters in the model even though significant is not advised as it will lead to increasing the cost.

4.2.3.3. Normality Tests/Data and Residual Distribution

A check of the normality of each the data could be made by plotting a histogram of the residuals. If the assumptions on the errors is satisfied, this plot should look like a sample from a normal distribution centered at zero. Gross deviations from normality are potentially serious and require further analysis. A very useful procedure is to construct a normal probability plot of the residuals. In this thesis a normal probability plot of the raw data is used to check the assumption of normality. In the analysis of variance, it is usually more effective (and straightforward) to do this with the residuals. If the underlying error distribution is normal, this plot will resemble a straight line. *Shapiro-Wilk W* tests is commonly used ones that were adopted in this thesis. The following typical reports in Table 4 -1 and Figure 4-1 shows, the normality test of all the laboratory test results. Atypical analysis indicates that Compression Index data are normality distributed.

Table 4 - 1. The normality test of all the laboratory test data results

<i>Shapiro-Wilk W Test</i>	<i>W</i>	<i>Prob<W</i>	<i>Remark</i>
Compression Index (Cc)	0.984916	0.981	Normally Distribution
Natural Water Content (%)	0.928952	0.1474	Normally Distribution
Specific Gravity (Gs)	0.903081	0.0471	<i>Not Normally Distribution</i>
Liquid Limit (%)	0.968217	0.7169	Normally Distribution
Plastic Limit (%)	0.929966	0.1542	Normally Distribution
Plasticity Index (%)	0.927067	0.1356	Normally Distribution
Liquidity Index	0.971535	0.7869	Normally Distribution
%Passing #200 Sieve Size	0.911723	0.0687	Normally Distribution



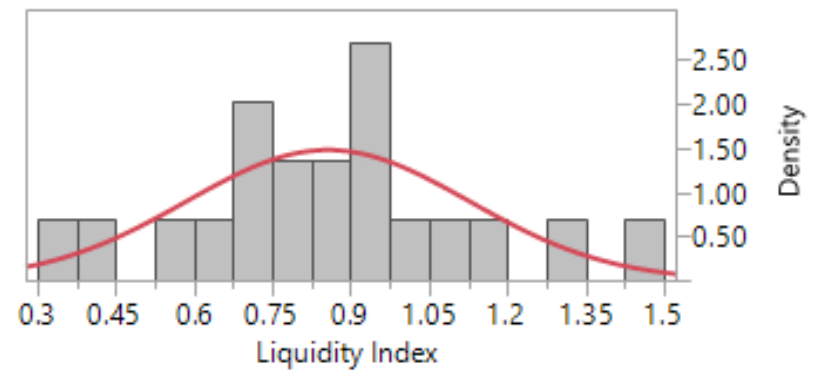
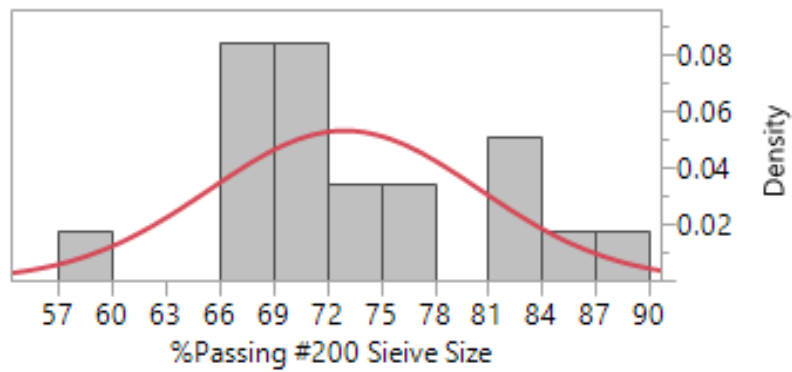
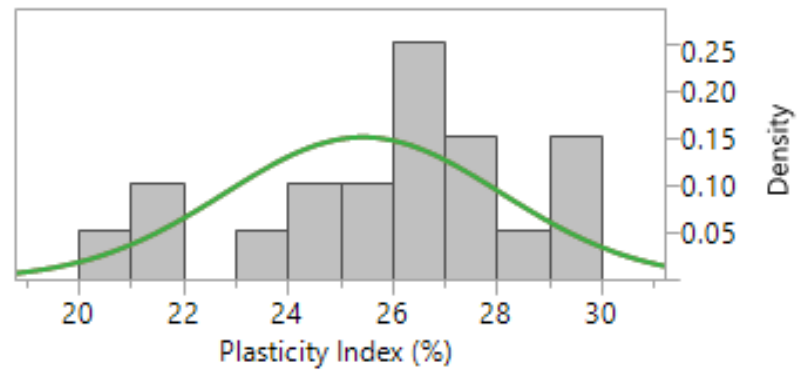
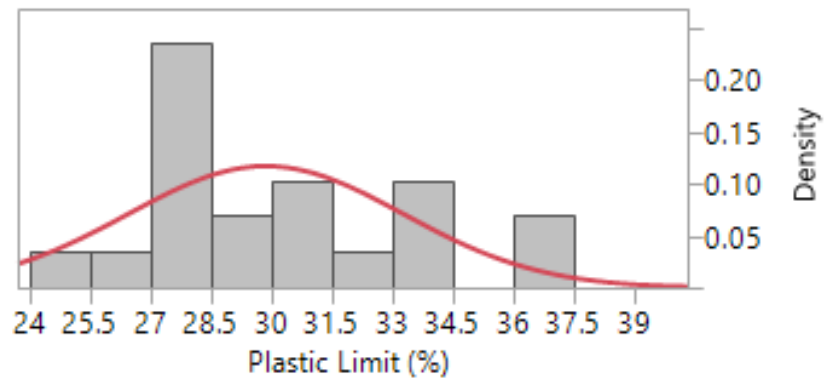


Figure 4. 1. Histogram and the Normal Distribution fit of the labourtory test data.

4.2.3.4. Multivariate Scatterplot Matrix/correlation Analysis

The Correlations table, which is a matrix of correlation coefficients that summarizes the strength of the linear relationships between each pair of response (y) variables. Correlation probability shows the correlation probability report, which is a matrix of p-values. Each p-value corresponds to a test of the null hypothesis that the true correlation between the variables is zero. This is a test of no linear relationship between the two response variables. Usually the Pearson product-moment correlation coefficient (r) is reported as it measures the strength of the linear relationship between two variables. If there is an exact linear relationship between two variables, the correlation is 1 or -1, depending on whether the variables are positively or negatively related. If there is no linear relationship, the correlation tends toward zero.

Table 4 - 2. Multivariate correlation and correlation probability analysis of all primary data used in model development.

Correlations	Cc	Wn(%)	Gs	LL(%)	PL(%)	PI(%)	%P200	LI
Cc	1							
Wn(%)	0.62	1						
Gs	0.431	0.4331	1					
LL(%)	0.3749	0.1057	0.6005	1				
PL(%)	0.0918	0.0642	0.5056	0.9021	1			
PI(%)	0.6925	0.1279	0.5456	0.8313	0.5101	1		
%P200	-0.2108	0.0656	0.4677	0.3005	0.3914	0.095	1	
LI	0.2194	0.6924	-0.0691	-0.6362	-0.5932	-0.504	-0.1041	1

Correl Probability	Cc	Wn(%)	Gs	LL(%)	PL(%)	PI(%)	%P200	LI
Cc	<.0001	0.0007	0.0578	0.1033	0.7003	0.003	0.3724	0.3526
Wn(%)	0.0007	<.0001	0.0565	0.6575	0.7879	0.591	0.7833	0.0007
Gs	0.0578	0.0565	<.0001	0.0051	0.0229	0.0128	0.0376	0.7722
LL(%)	0.1033	0.6575	0.0051	<.0001	<.0001	<.0001	0.1979	0.0026
PL(%)	0.7003	0.7879	0.0229	<.0001	<.0001	0.0216	0.0879	0.0058
PI(%)	0.0003	0.591	0.0128	<.0001	0.0216	<.0001	0.6903	0.0235
%P200	0.3724	0.7833	0.0376	0.1979	0.0879	0.6903	<.0001	0.6624
LI	0.3526	0.0007	0.7722	0.0026	0.0058	0.0235	0.6624	<.0001

The Pearson Product-Moment Correlation coefficient and the correlation probability analysis of all primary data used in model development as shown above suggest cc have a correlation with Wn(%) and PI(%).

4.2.3.5.Data Transformation for Normality

If any of the distribution of the dependent and the independent variable is not normally distributed, possible transformation model were applied till the data and the residuals from the model fit becomes normally distributed. Transformations generate new data. These variables were transformed using Logarithmic, Exponential or other derivatives of original data. The results of the normality checks of the different index property test data are shown in Table 4.1, almost all are normally distributed data. However, to apply transformation it will be checked latter ether it provide a uniformly distributed data distribution of the residuals in the model fit in the results and discussion part.

4.2.4. Subset Selection Routines

Subset selection refers to the task of finding a small subset of the available independent variables that does a good job of predicting the dependent variable during multiple regressions. Exhaustive searches are possible for regressions with up to 15 IV's. However, when more than 15 IV's are available, algorithms that add or remove a variable at each step must be used. Two such searching algorithms are available in this module and used in this study: forward selection and forward selection with switching.

4.2.5. Measuring How Well the Regression Model Performs

Finding an appropriate Mathematical Model that expresses the relationship between a dependent variable and several independent variables and estimating the values of its parameters using multiple linear regression is the main task of these section. A convenient way of measuring how well the regression model performs as a predictor of the dependent variable is to compute the following parameters such as: RMSE, R^2 , Adj R^2 , Coefficient of Variation, Mean Square Error BIC and AIC.

4.2.6. Model Validation and Comparison

After conducting the regression and once the model is developed, statistical data fitting parameters are obtained. In addition, performing the statistical analysis and investigating whether the underlying assumptions are satisfied or not, the experimenter is ready to draw practical conclusions about the problem under consideration. This might be done somewhat informally, perhaps by inspection of graphical displays such as the box plots scatter diagram, Standard error, root mean square and R^2 value.

CHAPTER FIVE

RESULTS AND DISCUSSIONS

5.1. INTRODUCTION

5.1.1. General Background

The evaluation of compression behavior of a soil using undisturbed samples is a difficult and expensive process. Therefore there is a need for simple routine tests to be performed on disturbed samples to achieve the same purpose. To facilitate this, empirical models had been put forwarded by different researchers to predict compression behavior of a soil. The formation and development of soil structure has very erratic nature and the compression potential is dependent on the geology, environmental factors, soil characteristics and many other factors which vary from place to place. Therefore equations developed for soils in one place may not work at all if tested on soils of other place of the same region. Hence specific predictive models have to be developed for specific areas in order to give fair evaluations. These models comprise different soil parameters indifferent combinations. Index properties are the widely used parameters in these models because these properties have significance in indicating the compression behavior of a soil. In general, previously developed empirical equations and equations to be developed in the future are not to be expected to determine compression index precisely and accurately for all soils.

Stepwise subset selection multiple regression analysis is used to produce an equation that predicts a dependent variable using one or more independent variables. Regression analysis is statistical tool that is used to define the analytical – mathematical models, i.e. functions between the independent (x) and dependent (y) variables. Strength of dependence between variables is determined using the correlation analysis (Đoković, K2013). A statistical analysis was used to investigate the relationship between two or more variables. A variable whose value is predicted is called dependent variable or response. A variable used to predict the value of dependent variable is termed independent or predictor variable. A regression model that contains more than one predictor variable is called multiple regression models. Alternatively, multiple regression model containing one independent variable is termed as single regression model. MLR was implemented to develop soil compressibility prediction models for soils studied.

The statistical significance of the model is defined using p - value or significance level p . If $p < 0.05$, $p < 0.01$ or $p < 0.1$ is acceptable to the model and the safety of $P > 95\%$, $P > 99\%$, or $P > 90\%$. In the case where $p > 0.05$ correlation was not significant, and then regardless of the value coefficient of correlation, a model should not be accepted and interpreted. The results in this regard are presented in this chapter.

5.1.2. Data used for Model Development

The soil studied in Azezo Tseda sub city based on USCS is CH, CL and MH, these indicates that there is a high settlement potential for these soils. A random sample from the primary data developed in laboratory testing, equal in size to 75% ($n=16$) of the original dataset, was taken and used for model development. The remainder of the primary dataset i.e., 25% ($n=4$) plus the secondary data collected was used for testing the models. All the developed models were employed in this test set and the corresponding outcomes were recorded. Refer chapter 3 for the complete data set developed from modeling and validation.

5.2. CORRELATION/SCATTER PLOT AND LINEAR BEST-FIT MODEL

In this work, the value of compression index (cc) was considered as the dependent variable whereas liquid limit (LL), plasticity index (PI), plastic limit (PL), specific gravity, liquidity index, %P200 and moisture content (ω) are the independent (Predictor) variables. The correlation of Compression Index (Cc) with other index property test data are first investigated to assess whether any one parameter significantly impacting the Cc parameter. As shown in Table 4-2, the correlation analysis of all primary data used in model development with cc shows, the Compression Index (Cc) versus Plastic Limit (%). Compression Index (Cc) versus %Passing #200 Sieve Size and the Compression Index (Cc) versus LI are not correlated significantly well. All other correlations of Compression Index (Cc) are having a good correlation even though not as such significant. Linear fit of Compression Index (Cc) with other index property test data are plotted as shown in Figure 5.1. Therefore, here we can conclude that either multiple factors and/or transformed single parameter may affect the Cc predictive model as it will be shown later or outliers need to be removed.

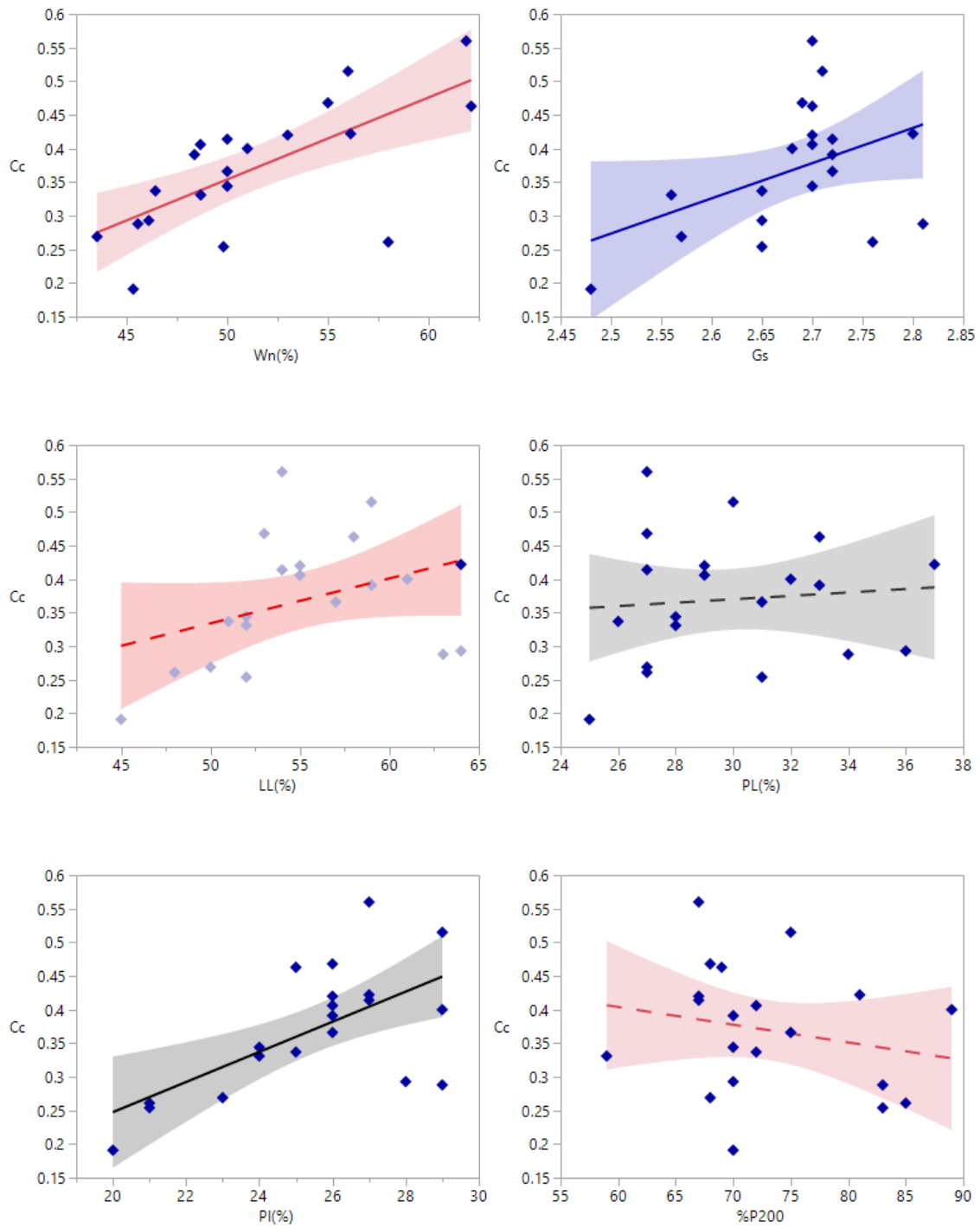


Figure 5.1. Linear fit analysis of compression index (C_c) with other index property test data.

5.3. STATISTICAL PREDICTION MODEL FOR COMPRESSION INDEX ,CC

5.3.1. Multi-Variate Regression Models/Focus

Here one, two and three parameter model using multiple linear regression will be considered. The forward selection stepwise regression procedure was used to minimize the term-trusted “goodness of fit” measure Bayesian Information Criterion (BIC). The variable selection procedure used during the forward selection stepwise regression takes into consideration the correlation coefficients of the participating variables to minimize multicollinearity. At the first step of the process, the initial regression model for every group of samples will contain no variables. At each iteration, the present independent variable in the equation will be help fixed and only the variable that is the most highly correlated with the response variable (i.e., Cc) enters the regression model. This procedure leads to the most parsimonious model while trying to eliminate multicollinearity. The multicollinearity refers to the problems that arise in regression models when some of the predictors are highly correlated with each other. Although multicollinearity by itself does not affect the predictive performance of the model, it may negatively influence the determination of which factors are the most significant. Multicollinearity essentially obstructs the effective choosing of the right predictors to include in the predictive model, making it hard to obtain a parsimonious model. Parsimony is an essential property of every effective prediction model. The forward selection stepwise regression procedure that was used in this study leads to models that are based only on the most influential parameters.

5.3.2. Newly Developed Compression Index (Cc) Predictive Models

5.3.2.1. One Parameter Model: Model-1/ $C_c = f(PI)$

The first predictive model developed contains only one significantly affecting predictor variable and satisfies all the statistical goodness of fit measures and assumption required, here after called Model - 1, are presented in Eq(5.1) as follows:

$$C_c = -0.434210568 + 0.0318839071 \times PI \quad (5.1)$$

Where: Cc is Compression Index, dimensionless and PI is Plasticity Index in %

To evaluate the correlation between the actual and predicted values of Cc, the linear regression plot analysis is necessary. Based on this analysis report, the statically significance of the model such as R^{Squared} , adjusted R^2 and RMSE are tabulated below in

Table 5.1. This indicates that the actual versus predicted compression index (Cc) plot using Model - 1. Residual analysis plots for Model - 1 fit predicted compression index (Cc) is shown in Figure 5.3. The shows the actual versus predicted compression index (Cc) plot indicates; Model - 1 is powerful in predicting Cc. As shown in Table 5.1 the statistical goodness of fit measures of the model with significant predictor variable PI are: R^{Square} of 0.945; RMSE of 0.023; AIC^c of -62.18 and BIC of -62.67.

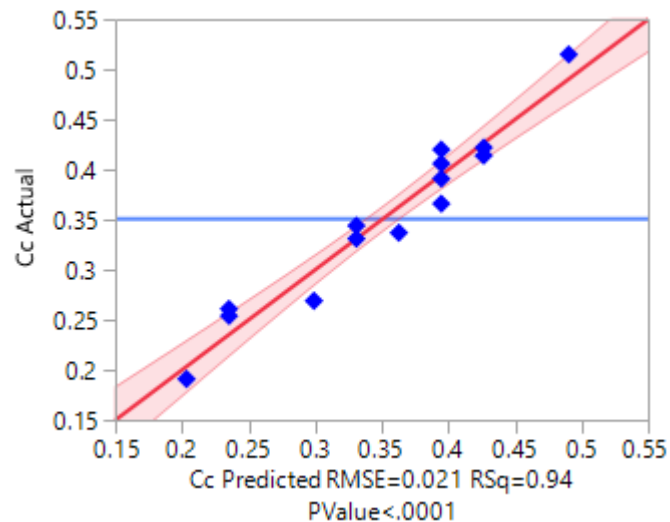


Figure 5.2. Comparson of actual vesrus pridicted compression index (Cc) using Model - 1 before validation and calibration.

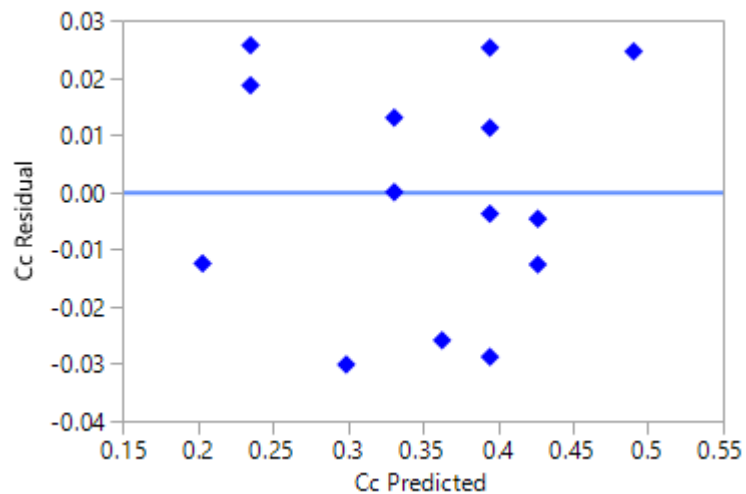


Figure 5.3. Residual analysis plot using Model - 1 fit pridicted Compression Index (Cc) before validation and calibration.

These shows the model is highly significant in predicting Cc. The *Durbin-Watson test* shows no significant autocorrelation of residuals exist, moreover as shown in Figure 5.3, the residual analysis plot using Model-1 fit predicted compression index (Cc) suggest

the residuals are normally distributed with constant variance at mean of zero; which is a basic requirement set by MLRⁱ analysis for a very good model. Once more, the *Shapiro-Wilks test* shows the distribution residual computed using Model-1 are normally distributed. However this model needs to be tested using validation data and need to be calibrated later.

Table 5.1. Summary of statistical data fitting goodness measures for One Parameter Model (Model -1) before validation and calibration.

Goodness of Fitting Measures	Significance of Model-1 Before Validation/Calibration
No of Parameter	1
Significant predictor variable	PI(%)
R ^{Square}	0.94481
R ^{SquareAdj}	0.9402
RMSE	0.0210
Mean of Response	0.3515
Number of Data	14
AIC ^c	-62.18
BIC	-62.67
F Ratio	205.45
Durbin-Watson	2.5638
Autocorrelation	-0.2991
Prob<DW	0.8893
W	0.916
Prob<W	0.1913
Remark	Normal

5.3.2.2. Two Parameter Cc Predictive Model: Model-2/ $Cc = f(LI, PI)$

The second predictive model developed which contains only two significantly affecting predictor variable which satisfies all the statistical goodness of fit measures and assumption, here after called Model 2, are presented in Eq(5.2) as follows. In Model 2, LI and PI are found significantly correlated with Cc.

$$C_c = -0.677400781 + 0.0333066554 \cdot PI(\%) + 0.2492772334 \cdot LI + (PI - 25.421052652) \cdot ((LI - 0.8705263158) \cdot 0.0490790017) \quad (5.2)$$

Where: Cc is Compression Index, dimensionless, LI is the liquidity index and PI is plasticity Index in %

ⁱ MLR – Multiple Linear Regression

Summary of statistical goodness of data fitting measures in the multiple regression subset selection report for two parameter model before validation and calibration are tabulated as shown in Table 5.2. Figure 5.4 indicates the actual versus predicted compression index (Cc) using Model 2. Residual analysis plot using Model 2 fit predicted compression index (Cc) are also shown in Figure 5.5.

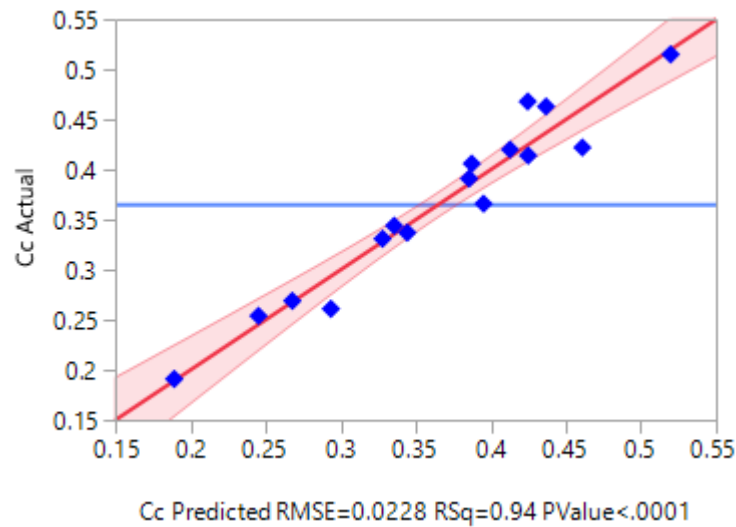


Figure 5.4. Comparison of actual versus predicted Compression Index (Cc) using Model 2, before validation and calibration.

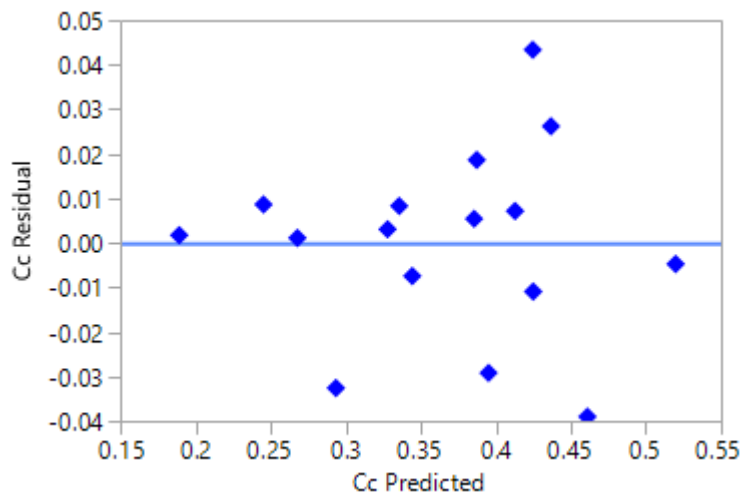


Figure 5.5. Residual analysis plot using Model 2 fit predicted compression index (Cc), before validation and calibration.

Table 5.2. Summary of statistical data fitting goodness measures for Model-2, a two parameter model before validation and calibration.

Goodness of fitting measures	Significance of Model-3 Before Validation/Calibration
No of Parameter	2
Significant predictor variable	PI and LI
RSquare	0.966
RSquareAdj	0.959
RMSE	0.020
Mean of Response	0.369
Number of data	19
AICc	-85.2763
BIC	-85.1695
F Ratio	299.05
Durbin-Watson	2.32476
Autocorrelation	-0.1628
Prob<DW	0.7562
W	0.936973
Prob<W	0.1137
Remark	Normal

5.3.2.3. Three Parameter Model: Model-3/ $C_c = f(PI, PL, LI)$

The third MLR predictive model developed which contains only three significantly affecting predictor variable which satisfies all the statistical goodness of fit measures and assumption, here after called Model 3, are presented in Eq(5.3) as follows.

$$\begin{aligned}
 C_c = & - 0.815278327 + 0.005720225 \cdot PL(\%) \\
 & + 0.030987914 \cdot PI(\%) \\
 & + 0.2793471824 \cdot (LI) \\
 & + (PI(\%) - 25.3888889) \cdot ((LI - 0.8766667) \cdot 0.0541578097)
 \end{aligned}
 \tag{5.3}$$

Where:

C_c is Compression Index, dimensionless, LI is liquidity index (dimensionless), PI is a Plasticity Index in %, and PL is a Plastic limit in %.

In Model-3, PL, LI and PI are found significantly correlated with cc . The prediction capabilities of Model-3 are demonstrated using different plots as shown in Figure 5.6 and

Figure 5.7. The statistical goodness of fit measures of the model with significant predictor variable PI, PL, LI are: R^{Square} of 0.99; RMSE of 0.007; AICc of -112.12 and BIC of -114.41. These shows the model is highly significant in predicting Cc. The maximum F Ratio (772.96) is another indicator of how very good is Model - 3 in predicting Cc.

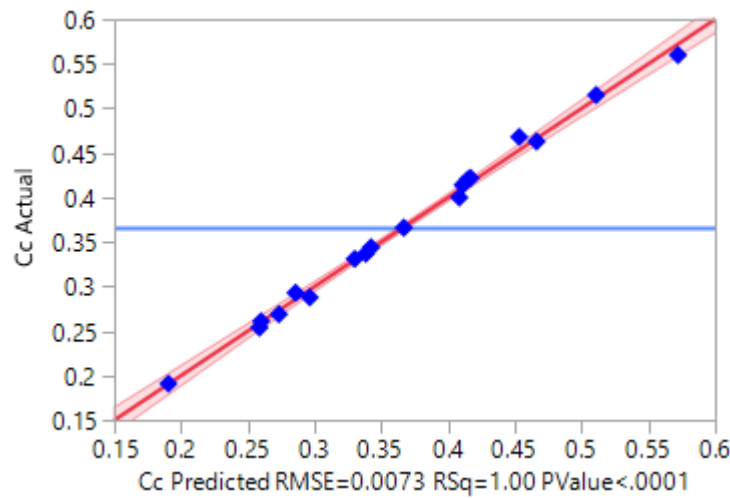


Figure 5.6. Comparson of the actual vesrus prdicted compression index (Cc) using Model - 3before validation and calibration.

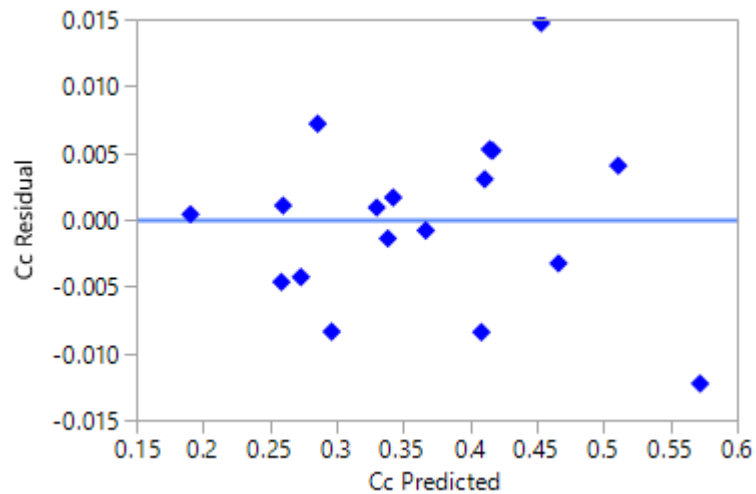


Figure 5.7. Residual analysis plot using Model-3 fit prdicted Compression Index (Cc) before validation and calibration.

The *Durbin-Watson test* shows no significant correlation of residuals, moreover as shown in Figure 5.7, the residual analysis plot using Model-3 fit predicted compression index (Cc) suggest the residuals are normally distributed with constant variance at mean of

zero. This is a requirement set by MLR analysis for a very good model. Once more, the *Shapiro Wilk test* shows the model residual are normally distributed.

Table 5.3. Summary of statistical data fitting goodness measures for Model-3, a three parameter model before validation and calibration.

Data Goodness of Fitting Measures	Significance of Model-3 Before Validation/Calibration
No of Parameter	3
Significant predictor variable	PI, PL and LI
R^{Square}	1
$\text{Adj}R^{\text{Square}}$	0.99
RMSE	0.007
Mean of Response	0.366
Number of Parameter	17
AICc	-112.12
BIC	-114.41
F Ratio	772.96
Durbin-Watson	2.683
Autocorrelation	-0.3672
Prob<DW	0.918
W	0.981
Prob<W	0.9587
Remark	Normal

5.3.3. Validation and Calibration of the Newly Developed Models

The three newly developed models were validated and calibrated using similar data used for model development (training data) and obtained from literature, these data is obtained from literatures. The validation data that were deducted from analysis which is not used in the model development and contents primary and secondary data's were used to validate and calibrate the new models. The results of the validation and the calibrated models are presented as follows.

5.3.3.1. Validated and Calibrated One Parameter Model - 1, $Cc = f(PI)$

Equation (5.4) is calibrated model validated using the training and the validated data sets is an improved model from Eq.(5.1).

$$Cc = - 0.398501119 + 0.0303434004' PI(\%) \quad (5.4)$$

Figure 5. 8 show the summary of the predicted values versus the measured Cc values using the validation data sets. The newly developed one parameter Cc predictive

model for the calibrated model is indicated in Eq.(5.4). The cross validation of the model using training and validation data set results are shown in Table 5.4 and this fit shows a very good (R^2 of 0.86) agreement against the validation data set used and the model is now changed/calibrated.

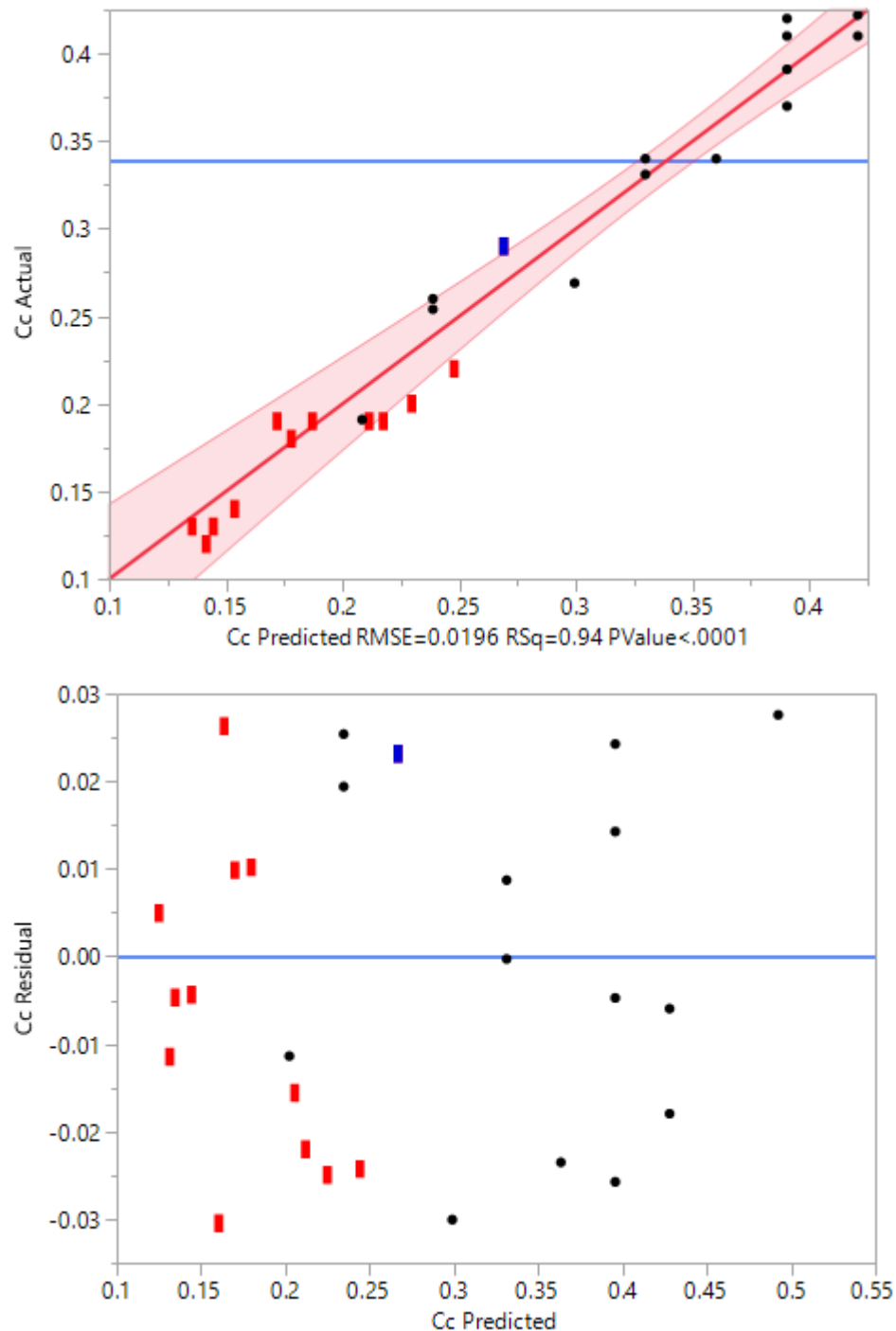


Figure 5. 8. Comparstion of the actual vesrus pridicted Compression Index (Cc) using the calbrated Model - 1 and the residual analysis plot using the calibrated Model - 1 fit versus the pridicted Compression Index (Cc).

Table 5.4. Crossvalidation analysis and summary of statistical data fitting goodness measures for Model - 1.

Source	R ^{Square}	RASE
Training Set	0.9376	0.01801
Validation Set	0.8607	0.01887

Table 5.5. Validation and calibration of Model -1 using data from literature and the training data set (Primery data).

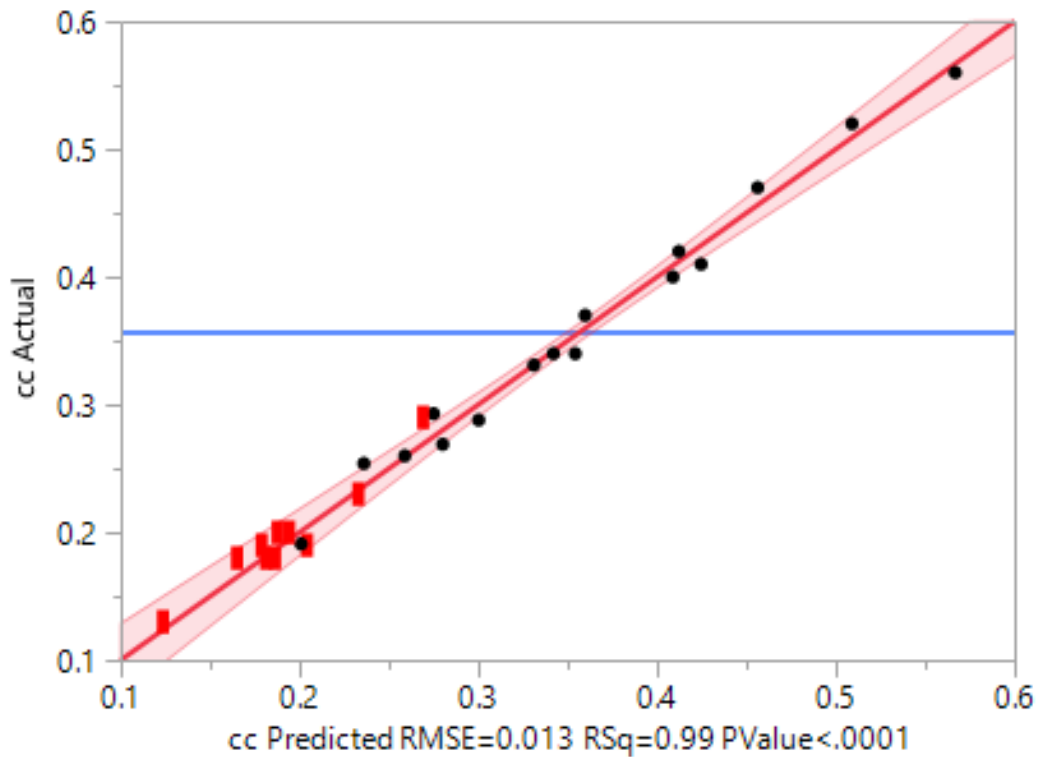
Soil	Literature	Model - 1 (Before)		Model - 1 (Validated)		Data
ID	Cc	Predicted Cc	Residual	Predicted Cc	Residual	usage
TP-2	0.26	0.235	0.025	0.24	0.0213	Training
TP-5	0.33	0.331	0.000	0.33	0.0013	Training
TP-6	0.42	0.395	0.025	0.39	0.0296	Training
TP-10	0.37	0.395	-0.025	0.39	-0.0204	Training
TP-11	0.25	0.235	0.019	0.24	0.0153	Training
TP-13	0.27	0.299	-0.030	0.30	-0.0304	Training
TP-15	0.19	0.203	-0.012	0.21	-0.0174	Training
TP-16	0.34	0.363	-0.023	0.36	-0.0201	Training
TP-18	0.34	0.331	0.009	0.33	0.0103	Training
TP-20	0.41	0.427	-0.017	0.42	-0.0108	Training
TP-1	0.42	0.427	-0.005	0.42	0.0012	Training
TP-14	0.41	0.395	0.015	0.39	0.0196	Training
TP-17	0.39	0.395	-0.004	0.39	0.0006	Training
R52	0.29	0.267	0.023	0.27	0.0209	Validation
R50	0.20	0.226	-0.026	0.23	-0.0296	Validation
R52	0.29	0.267	0.023	0.27	0.0209	Validation
R49	0.19	0.207	-0.017	0.21	-0.0214	Validation
R5	0.12	0.133	-0.013	0.14	-0.0216	Validation
R8	0.13	0.137	-0.007	0.14	-0.0146	Validation
R18	0.13	0.127	0.003	0.14	-0.0055	Validation
R19	0.14	0.146	-0.006	0.15	-0.0137	Validation
R24	0.19	0.213	-0.023	0.22	-0.0275	Validation
R28	0.18	0.172	0.008	0.18	0.0020	Validation
R29	0.19	0.165	0.025	0.17	0.0180	Validation
R73	0.18	0.172	0.008	0.18	0.0020	Validation
R77	0.19	0.181	0.009	0.19	0.0029	Validation
R78	0.22	0.245	-0.025	0.25	-0.0278	Validation
Using all data		Sum of Resid.	-0.0398199	Sum of Resid.	-0.0950760	
Using Training data		Sum of Resid.	-0.0226	Sum of Resid.	0.0000	

5.3.3.2. Validated and Calibrated Two Parameter Model 2, $Cc = f(PI \text{ and } LI)$

Equation (5.5) is the calibrated model validated using the training and the validated data sets and is an improved model from Eq. (5.2).

$$\begin{aligned} Cc = & - 0.293720953 + 0.0197882672 \times PI(\%) \\ & + 0.1712838669 \times LI \\ & + (PI - 23.788461538) \times ((LI - 0.5965384615) \times 0.0472774524) \end{aligned} \quad (5.5)$$

Figure 5.9 shows the summary of the predicted values versus the measured Cc values using the validation data sets. The newly developed two parameter Cc predictive model for the calibrated model is indicated in Eq. (5.5). The cross validation of the model using training and validation data set results are shown in Table 5.4 and this fit shows a very good (R^2 of 0.92) agreement against the validation data set used and the model is now changed/calibrated.



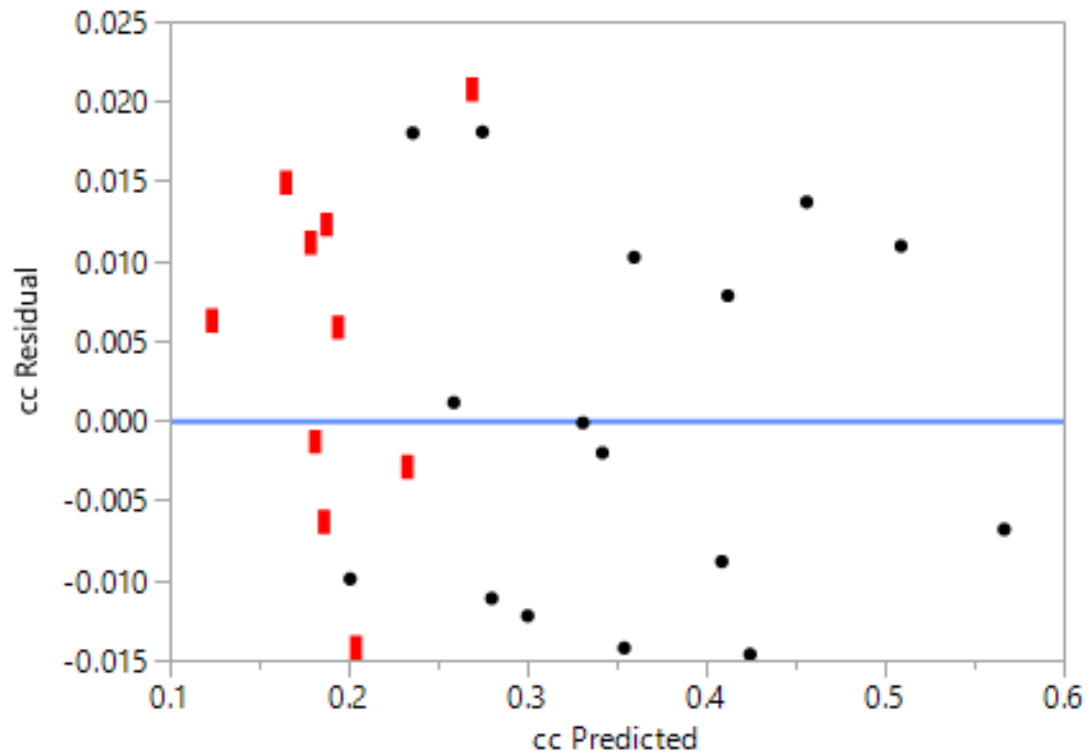


Figure 5. 9. Comparison of the actual versus predicted Compression Index (Cc) using the calibrated Model - 2 and the residual analysis plot using the calibrated Model - 2 fit versus the predicted Compression Index (Cc).

Table 5.6. Crossvalidation analysis and summary of statistical data fitting goodness measures for Model - 2.

Source	R ^{Square}	RASE
Training Set	0.9868	0.01130
Validation Set	0.9176	0.01120

Table 5.7. Validation and calibration of Model -2 using data from literature and the training data set (Primery data).

Soil ID	Literature	Model - 2 (original)		Model - 2 (Validated)		Data usage
	Cc	Predicted Cc	Residual	Predicted Cc	Residual	
TP-2	0.26	0.259	0.00	0.260	0.259	Training
TP-4	0.52	0.518	0.00	0.520	0.509	Training
TP-5	0.33	0.337	-0.01	0.331	0.331	Training
TP-6	0.42	0.419	0.00	0.420	0.412	Training
TP-7	0.56	0.576	-0.02	0.560	0.567	Training
TP-8	0.47	0.464	0.01	0.470	0.456	Training
TP-9	0.29	0.306	-0.02	0.288	0.300	Training
TP-10	0.37	0.367	0.00	0.370	0.360	Training
TP-11	0.25	0.240	0.01	0.254	0.236	Training
TP-12	0.40	0.416	-0.02	0.400	0.409	Training
TP-13	0.27	0.286	-0.02	0.269	0.280	Training
TP-15	0.19	0.203	-0.01	0.191	0.201	Training
TP-16	0.34	0.361	-0.02	0.340	0.354	Training
TP-18	0.34	0.348	-0.01	0.340	0.342	Training
TP-19	0.29	0.280	0.01	0.293	0.275	Training
TP-20	0.41	0.432	-0.02	0.410	0.425	Training
R15	0.13	0.134	0.00	0.130	0.124	Validation
R22	0.18	0.192	-0.01	0.180	0.186	Validation
R31	0.20	0.197	0.00	0.200	0.188	Validation
R42	0.19	0.186	0.00	0.190	0.179	Validation
R43	0.23	0.247	-0.02	0.230	0.233	Validation
R47	0.20	0.206	-0.01	0.200	0.194	Validation
R49	0.19	0.205	-0.01	0.190	0.204	Validation
R52	0.29	0.274	0.02	0.290	0.269	Validation
R72	0.18	0.174	0.01	0.180	0.165	Validation
R74	0.18	0.194	-0.01	0.180	0.181	Validation
Using all data		Sum of Resid.	-0.13	Sum of Resid.	-0.05	
Using Training data		Sum of Resid.	-0.1	Sum of Resid.	0.0000	

5.3.3.3. Validated and Calibrated Three Parameter Model 3, $C_c = f(\text{PI, PL and LI})$

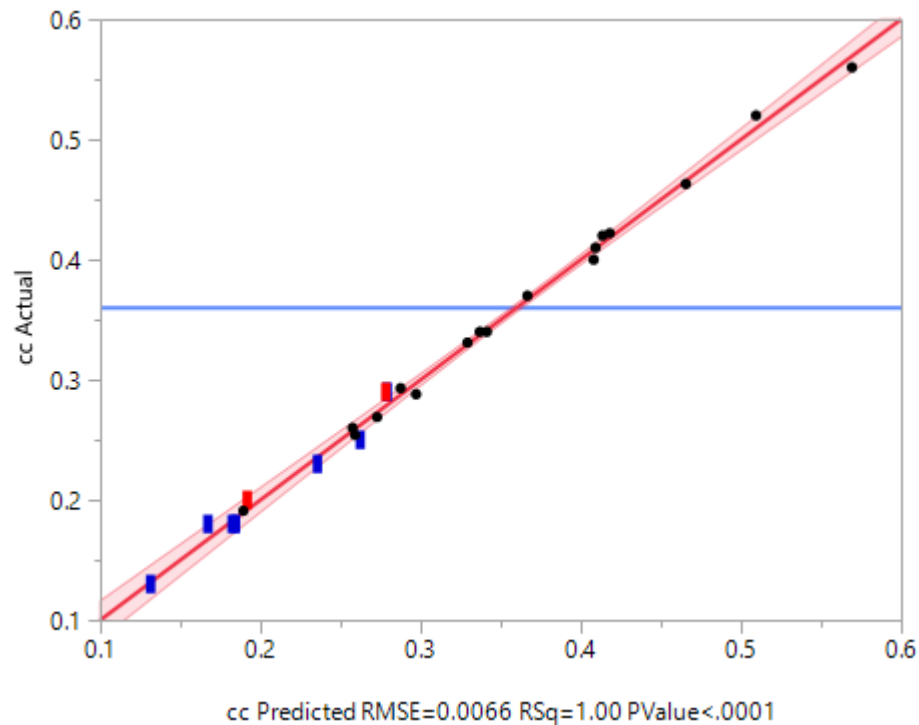
Equation (5.6) is the calibrated model validated using the training and the validated data sets and is an improved model form of Eq.(5.3).

$$\begin{aligned}
C_c = & - 0.487769935 + 0.005971569' \text{ PL} \\
& + 0.0200864602' \text{ PI} + 0.1932388363' \text{ LI} \\
& + (\text{PI} - 23.853846154)' ((\text{LI} - 0.679615384)' 0.0542180915)
\end{aligned}
\tag{5.6}$$

Figure 5.10 shows the summary of the predicted values versus the measured C_c values using the validation data sets. The newly developed two parameter C_c predictive model for the calibrated model is indicated in Eq.(5.5). The cross validation of the model using training and validation data set results are shown in Table 5.6 and this fit shows a very good (R^2 of 0.97) agreement against the validation data set used and the model is now changed/calibrated.

Table 5.8. Crossvalidation analysis and summary of statistical data fitting goodness measures for Model - 2.

Source	R ^{Square}	RASE
Training Set	0.9967	0.00552
Validation Set	0.9707	0.00881



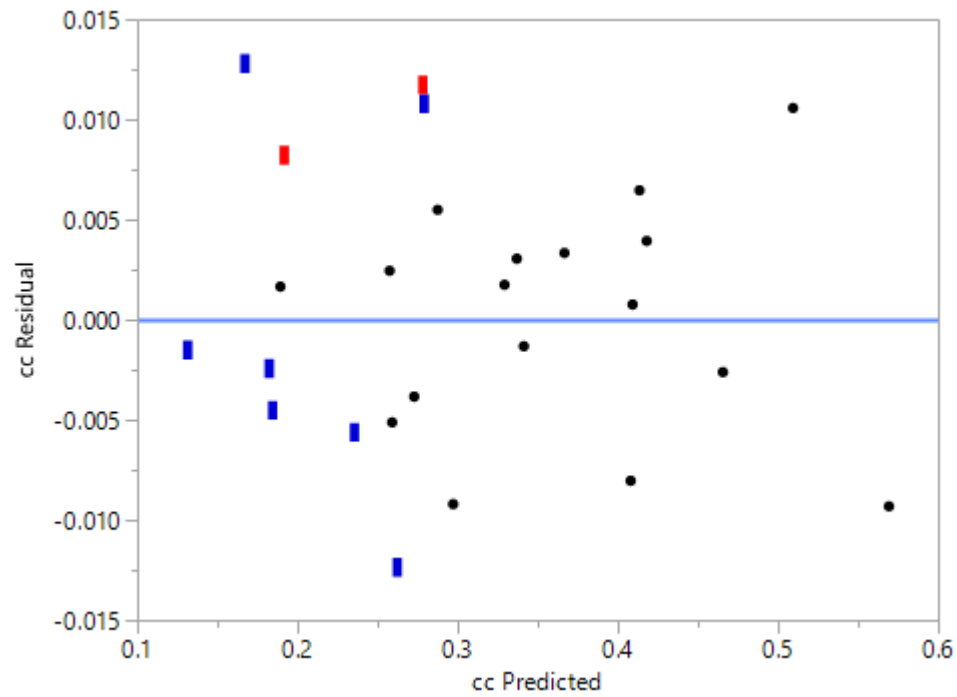


Figure 5. 10. Comparstion of the actual vesrus pridicted Compression Index (Cc) using the calbrated Model - 3 and the residual analysis plot using the calibrated Model – 3 fit versus the pridicted Compression Index (Cc).

Table 5.9. Validation and calibration of Model -3 using data from literature and the training data set (Primery data).

Soil ID	Literature	Model - 3 (original)		Model - 3 (Validated)		Data usage
	Cc	Predicted Cc	Residual	Predicted Cc	Residual	
TP-2	0.260	0.260	0.000	0.260	0.257	Training
TP-4	0.520	0.511	0.009	0.520	0.509	Training
TP-5	0.331	0.330	0.001	0.331	0.329	Training
TP-6	0.420	0.415	0.005	0.420	0.413	Training
TP-7	0.560	0.572	-0.012	0.560	0.569	Training
TP-9	0.288	0.296	-0.008	0.288	0.297	Training
TP-10	0.370	0.367	0.003	0.370	0.367	Training
TP-11	0.254	0.259	-0.005	0.254	0.259	Training
TP-12	0.400	0.408	-0.008	0.400	0.408	Training
TP-13	0.269	0.273	-0.004	0.269	0.273	Training
TP-15	0.191	0.191	0.000	0.191	0.189	Training
TP-16	0.340	0.338	0.002	0.340	0.337	Training
TP-18	0.340	0.342	-0.002	0.340	0.341	Training
TP-19	0.293	0.286	0.007	0.293	0.287	Training
TP-20	0.410	0.411	-0.001	0.410	0.409	Training
TP-1	0.422	0.417	0.005	0.422	0.418	Training
TP-3	0.463	0.466	-0.003	0.463	0.465	Training
TP-8	0.470	0.453	0.017	0.470	0.451	Training
R52	0.290	0.279	0.011	0.290	0.278	Validation
R50	0.200	0.195	0.005	0.200	0.192	Validation
R22	0.130	0.132	-0.002	0.130	0.131	Validation
R31	0.180	0.166	0.014	0.180	0.167	Validation
R43	0.230	0.233	-0.003	0.230	0.236	Validation
R47	0.180	0.180	0.000	0.180	0.182	Validation
R52	0.290	0.279	0.011	0.290	0.279	Validation
R72	0.250	0.257	-0.007	0.250	0.262	Validation
Using all data		Sum of Resid.	0.03	Sum of Resid.	-0.04	
Using Training data		Sum of Resid.	0.00	Sum of Resid.	-0.002	

5.3.3.4. Analysis of Variance Details for all Cc Predictive Models: before and after validation and calibration.

The Analysis of variance report partitions the total variation of a sample into two components. The ratio of the two mean squares forms the F ratio. If the probability associated with the F ratio is small, then the model is a better fit statistically than the overall response mean. The detail analysis of variance results for all models developed are tabulated in Table 5.8, and this shows the validated Model 3 is a better fit to the data calibrated. If the analysis of variance model results in a significant reduction of variation from the total, the F ratio is higher than expected. Probability of obtaining (by chance alone) an F value greater than the one calculated if, in reality, there is no difference in the population group means. Observed significance probabilities of 0.05 or less are often considered evidence that there are differences in the group means. Therefore, the model is best when the probability level is minimum and the F-ratio is relatively maximum (The calibrated Model 3 selected best).

Table 5.10. Analysis of variance detail summery and statistical goodness of data fitting measures for one, two and three parameter models developed.

Model	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model - 1	Model	1	0.0907	0.0907	205.4325	<.0001
	Error	12	0.0053	0.0004		
	C. Total	13	0.0960			

Model	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model – 1 Calibrated	Model	1	0.0633	0.0633	165.2181	<.0001
	Error	11	0.0042	0.0004		
	C. Total	12	0.0675			

Model	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model - 2	Model	2	0.1120	0.0560	107.2462	<.0001
	Error	13	0.0068	0.0005		
	C. Total	15	0.1187			

Model New	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model – 2 Calibrated	Model	3	0.15272817	0.050909	299.0515	<.0001
	Error	12	0.00204283	0.00017		
	C. Total	15	0.154771			

Model	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model - 3	Model	4	0.16602637	0.041507	772.9603	<.0001
	Error	13	0.00069808	0.000054		
	C. Total	17	0.16672444			

Model New	Source	DF	Sum of Squares	Mean Square	F Ratio	Prob> F
Model - 3 Calibrated	Model	4	0.15672022	0.03918	908.2452	<.0001
	Error	12	0.00051766	0.000043		
	C. Total	16	0.15723788			

5.3.4. Comparison of Newly developed Predictive Models

All the models developed satisfy the entire required multiple linear regression statistical analysis requirements, as a result all the developed models are suggested best. The number of parameters significantly affecting the value of Cc and selected through MLR analysis for each model are different. Notice that as the number of parameters increases the performance of the model increases. Here, the maximum numbers of significantly affecting parameters found are three. However, in this particular case the parameters that are selected affecting the model significantly are all simple to measure in laboratory and one can't consider the number of parameter a primary variable to compare each of the models. These significantly affecting parameters for Model -1, 2 and 3 are PI; PI and LI; and PI, PL, and LI respectively.

The %passing #200 sieve size and the specific Gravity of the soils are found not significant as a Cc predictor in either of the models developed and as such not required. In the case considered, to compare the significance of the newly developed models having different number of variables, the AIC and BIC are the best statistical measures. Therefore in addition to the other statistical goodness of fitting measures, the model having the lowest BIC and AIC values are the best predictive model. Figure 5.12 shows the comparison of statistical goodness of prediction measures for all Cc predictive Models. In this regard, even though all of them perform very well, Model-3 is found the best model for predicting Cc.

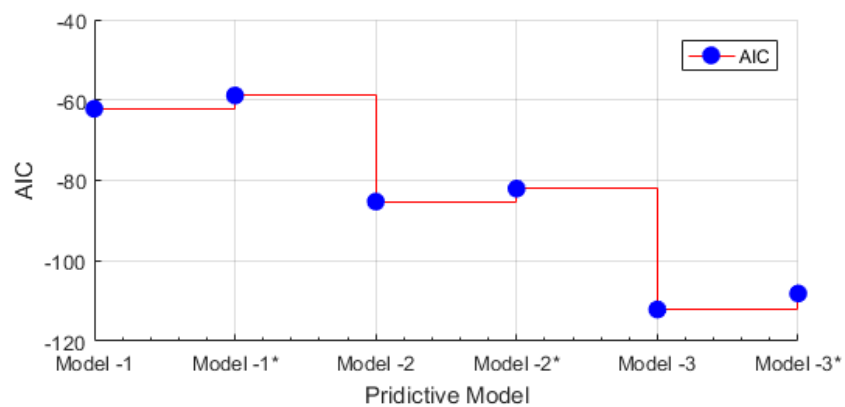
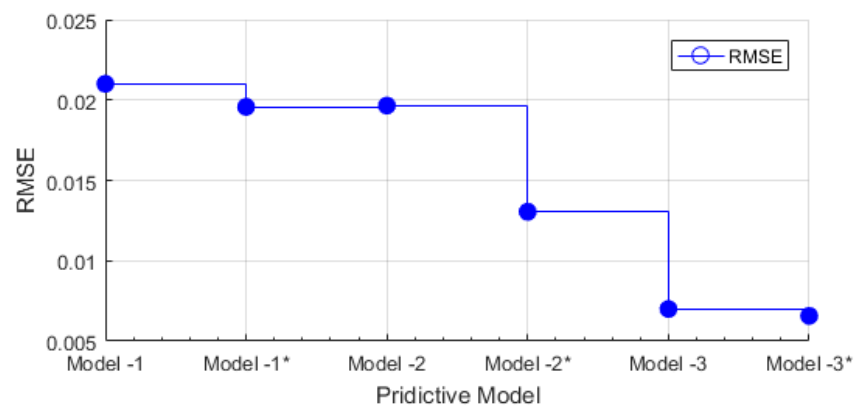
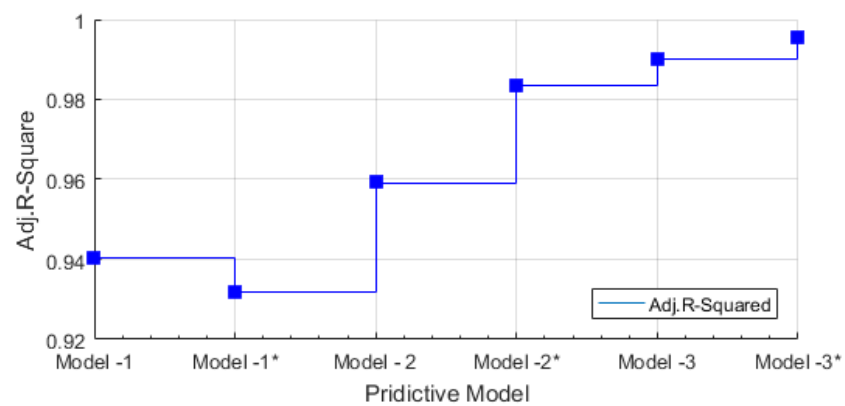
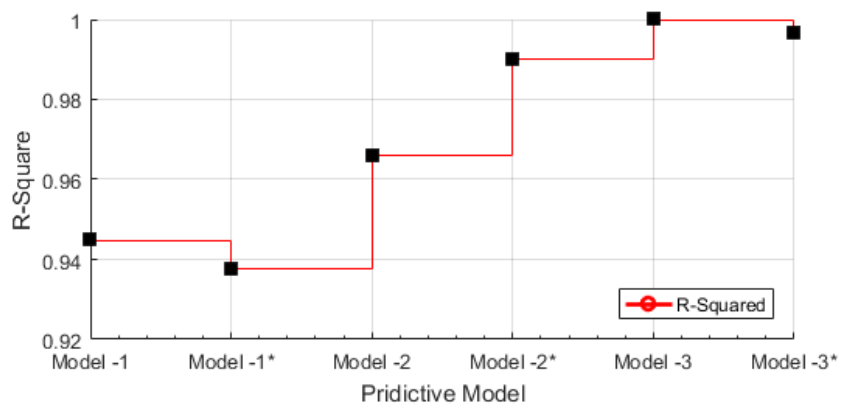
The *Durbin-Watson test* shows/gives a statistic to test whether the residuals have first-order autocorrelation. The value reports the Significance P Value obtained in the fit. This option computes and displays Prob,<DW, the exact probability associated with the statistic Autocorrelation of residuals is checked using the Durbin-Watson test in this model

development process, and the value indicates whether there is the high probability of autocorrelation in between residuals. Therefore the result indicates for all the developed models that there is no significant autocorrelation of residuals from the fit of the new models developed. Moreover, the Shapiro-Wilk Goodness of Fit option computes the goodness of fit test for the fitted distribution. For Normal distributions, the Shapiro-Wilk test for normality is used to check whether the residuals are normally distributed or not, and it was found that the residuals obtained from the fit of all data using the models are normally distributed. The corrected Akaike's Information Criterion (AICc) and the Bayesian Information Criterion (BIC) are information-based criteria that assess model fit performance and model comparison.

This value can be used to compare various models for the same data set to determine the best-fitting model. The model having the smallest value is usually the preferred model. In general, BIC penalizes models with more parameters more than AICc does. For this reason, it leads to choosing more parsimonious models, that is, models with fewer parameters, than does AICc. Therefore, the lower BIC value is the best model that can be selected. In this case, the best performing models, based on BIC and AIC values computed, are arranged as follows: Model - 3 > Model - 2 > Model - 1. Meaning Model - 3 is the best. Comparison of the statistical goodness of prediction measures for all Cc predictive models after validation and calibration are plotted in Figure 5.5. Summary of statistical goodness of data fitting measures for one, two and three parameter models developed are indicated in Table 5.7 and the analysis of variance in Table 5.8. All the comparison shows that the calibrated and validated Model - 3 is the best.

Table 5.11. Comparison of developed models after and before validation and calibration:- summary of statistical goodness of data fitting measures for one,two and three parameter models developed.

Goodness of Fitting Measures	Before Validation	Validated Calibrated	Before Validation	Validated/ Calibrated	Before Validation	Validated Calibrated
	Model-1	Model-1 New	Model-2	Model-2 New	Model-3	Model-3 New
No of Parameter	1	1	2	2	3	3
Significant Predictor	PI(%)	PI(%)	PI, and LI	PI, and LI	PI, PL and LI	PI, PI, and LI
R ^{Square}	0.9448	0.9376	0.9660	0.9900	1.0000	0.9967
R ^{Square} Adj	0.9402	0.9319	0.9592	0.9835	0.9900	0.9956
RMSE	0.0210	0.0196	0.0197	0.0130	0.0070	0.0066
Mean of Response	0.3515	0.3391	0.3690	0.3573	0.3660	0.3606
AICc	-62.18	-58.8818	-85.2763	-82.0501	-112.12	-108.146
BIC	-62.67	-59.8537	-85.1695	-84.1871	-114.41	-111.547
F Ratio	205.45	165.00	299.05	299.05	772.96	908.2452
Durbin-Watson	2.564	2.233	2.325	2.325	2.683	2.099
Autocorrelation	-0.2991	-0.1834	-0.1628	-0.1628	-0.3672	-0.0605
Prob<DW	0.8893	0.7022	0.7562	0.7562	0.918	0.5384
W	0.916	0.929519	0.936973	0.936973	0.981	0.975263
Prob<W	0.191	0.06	0.1137	0.1137	0.9587	0.761
Remark	Normal	Normal	Normal	Normal	Normal	Normal



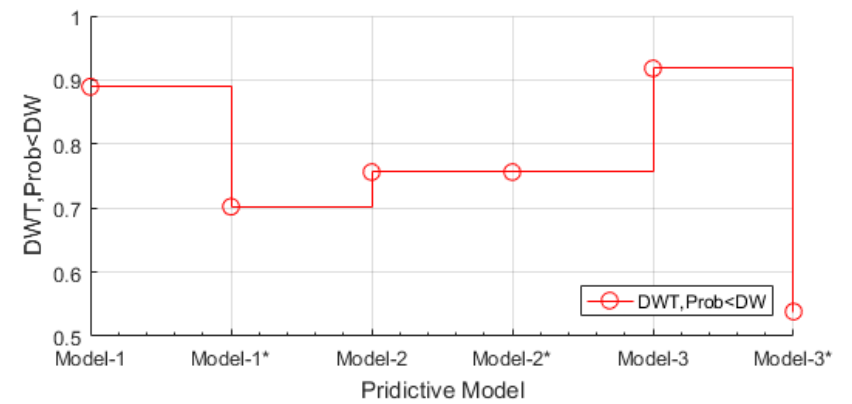
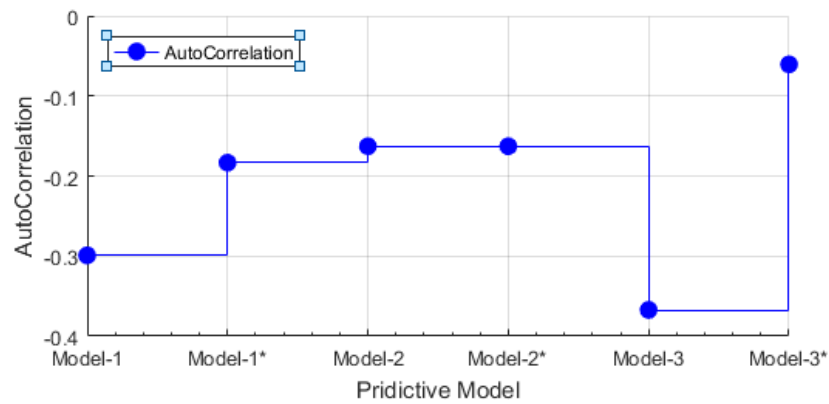
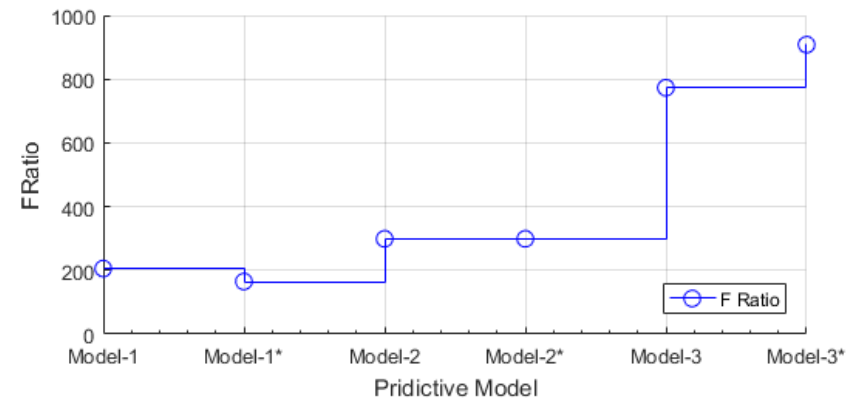
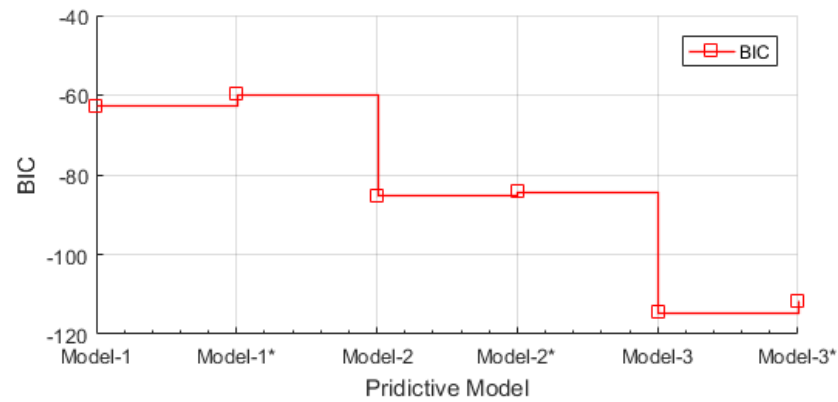


Figure 5.11. Comparison of statistical data fitting goodness measures for all the newly developed Models, before and after validation and calibration.

5.4. COMPARISON OF THE NEWLY DEVELOPED C_c PREDICTIVE MODELS WITH OTHER SIMILAR MODELS

The comparison of the new models developed in this study with other similar models is also another approach to assess the predictive capability of the new models. However to compare models each other at least the number and type of parameters included in the models must be the same. Therefore a limited success particularly for model 2 and model 3 types are achieved as there is no previous model developed containing a LI as a significant predictor variable/

However the new Model 1 is compared with the following model (Eq(5.7)) by Nakase et al (1988) that uses PI as a C_c predictor:

$$C_c = 0.046 + 0.0104I_p$$

$$\text{Best for } I_p < 50\% \quad \text{Nakase et al. (1988)} \quad (5.7)$$

Similar comparison were made for Model 2 with the model proposed by Koppula, (1981) as shown in eq 5.8

$$C_c = 0.009w + 0.005LL \quad (5.8)$$

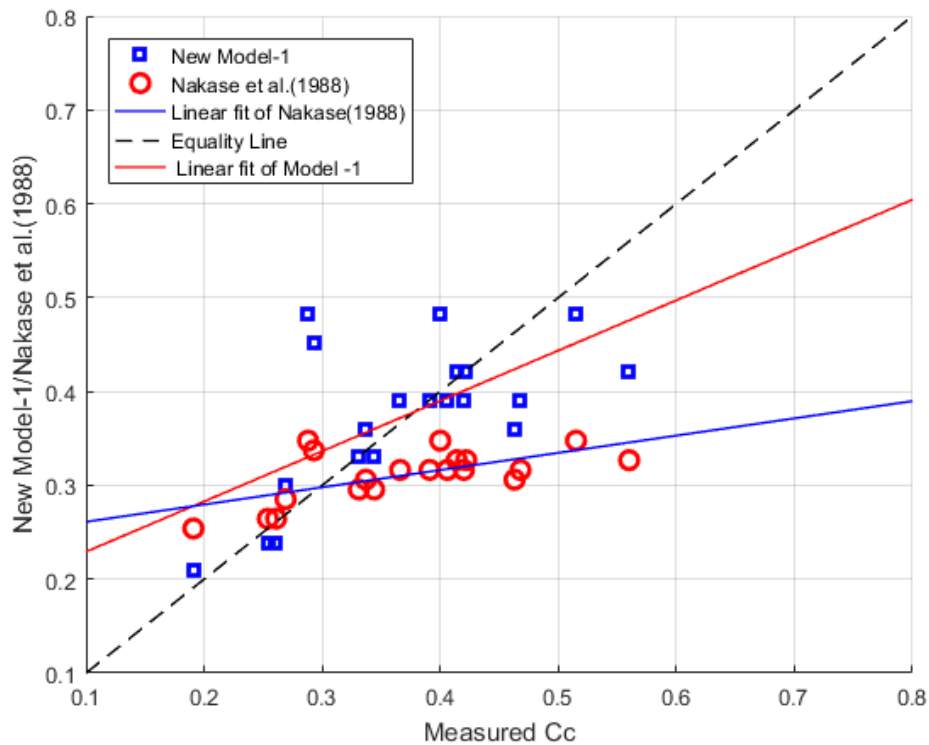


Figure 5.12. Comparison of one parameter models.

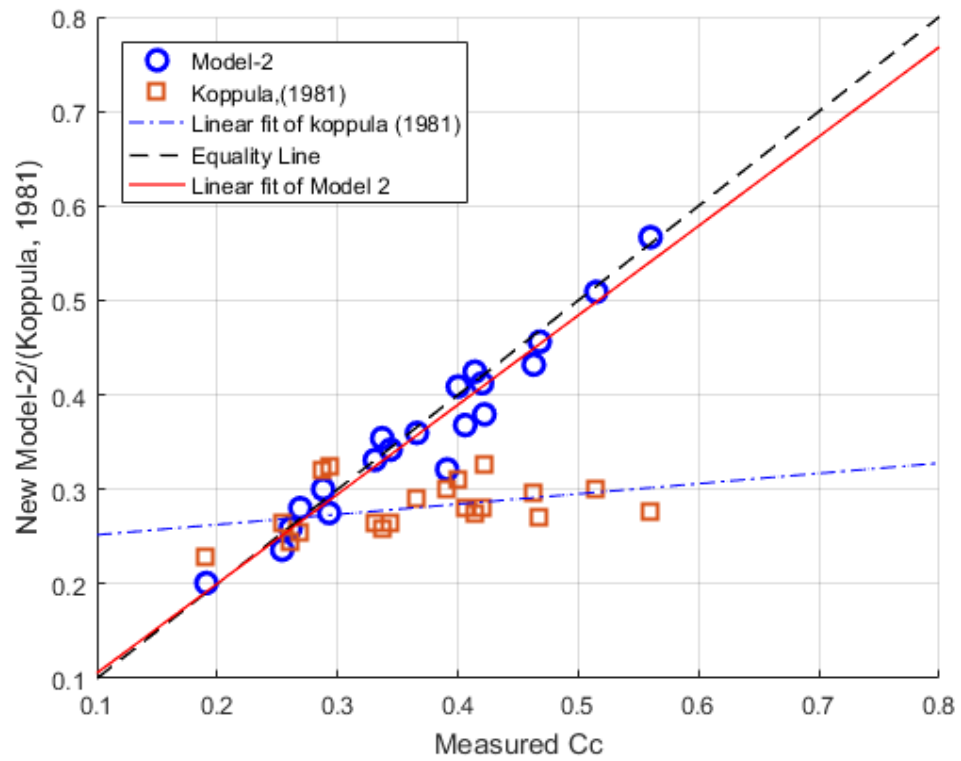


Figure 5.13. Comparison of two parameter models.

Table 5.12. In general, the comparison of model 1 is the following .

model 1			Birhanu			
				Nakase .et al.(1988)	Vinod and Bindu (2010)	Sridharan and Nagaraj (2000)
Test pit		measured	$cc = -0.398501119 + 0.0303434004 \cdot PI$	$cc = 0.046 + 0.0104PI$	$Cc = 0.0086(PI + 24.2674)$	$Cc = 0.014(PI + 3.6)$
	pi	cc	New model			
TP-1	27	0.422	0.420721699	0.3268	0.44089964	0.4284
TP-2	21	0.26	0.238661297	0.2644	0.38929964	0.3444
TP-3	25	0.463	0.360034898	0.306	0.42369964	0.4004
TP-4	29	0.52	0.4814085	0.3476	0.45809964	0.4564
TP-5	24	0.331	0.329691498	0.2956	0.41509964	0.3864
TP-6	26	0.42	0.390378299	0.3164	0.43229964	0.4144
TP-7	27	0.56	0.420721699	0.3268	0.44089964	0.4284
TP-8	26	0.47	0.390378299	0.3164	0.43229964	0.4144
TP-9	29	0.288	0.4814085	0.3476	0.45809964	0.4564
TP-10	26	0.37	0.390378299	0.3164	0.43229964	0.4144
TP-11	21	0.254	0.238661297	0.2644	0.38929964	0.3444
TP-12	29	0.4	0.4814085	0.3476	0.45809964	0.4564
TP-13	23	0.269	0.299348097	0.2852	0.40649964	0.3724
TP-14	26	0.41	0.390378299	0.3164	0.43229964	0.4144
TP-15	20	0.191	0.208317896	0.254	0.38069964	0.3304
TP-16	25	0.34	0.360034898	0.306	0.42369964	0.4004
TP-17	26	0.391	0.390378299	0.3164	0.43229964	0.4144
TP-18	24	0.34	0.329691498	0.2956	0.41509964	0.3864
TP-19	28	0.293	0.451065099	0.3372	0.44949964	0.4424
TP-20	27	0.41	0.420721699	0.3268	0.44089964	0.4284

Table 5.13.In general ,the comparison of model 2 is the following .

cc=- 0.293720953+0.0197882672*PI+0.1712838669*LI+(PI- 23.788461538)*((LI-0.5965384615)*0.0472774524).							Kumar et al. (2016) . Black cotton soil. red soil and yellow soil, Bhopal, India	Azzouz et al. (1976) Clays	Koppula 1981
Test pit					lab	birhanu	Cc=0.0128(LL)–0.008(PI)–0.1423	Cc = 0.009w + 0.002LL – 0.01	Cc=0.009Wn+0.005LL
id no	LI	LL	pi	wn	cc	New model			
TP-1	0.71	64	27	56.13	0.422	0.379401053	0.4609	0.62317	0.82517
TP-2	1.48	48	21	58	0.26	0.258864847	0.3041	0.608	0.762
TP-3	1.16	58	25	62.12	0.463	0.431949217	0.4001	0.66508	0.84908
TP-4	0.9	59	29	56	0.52	0.509063637	0.3809	0.612	0.799
TP-5	0.86	52	24	48.68	0.331	0.331136464	0.3313	0.53212	0.69812
TP-6	0.92	55	26	53	0.42	0.412174965	0.3537	0.577	0.752
TP-7	1.29	54	27	61.87	0.56	0.566809043	0.3329	0.65483	0.82683
TP-8	1.08	53	26	55	0.47	0.456309329	0.3281	0.591	0.76
TP-9	0.4	63	29	45.56	0.288	0.300227573	0.4321	0.52604	0.72504
TP-10	0.73	57	26	50	0.37	0.359765409	0.3793	0.554	0.735
TP-11	0.9	52	21	49.81	0.254	0.235982392	0.3553	0.54229	0.70829
TP-12	0.66	61	29	51	0.4	0.408822326	0.4065	0.571	0.764
TP-13	0.72	50	23	43.53	0.269	0.280131369	0.3137	0.48177	0.64177
TP-14	0.76	55	26	48.67	0.41	0.368040602	0.3537	0.53803	0.71303
TP-15	1.02	45	20	45.33	0.191	0.200908243	0.2737	0.48797	0.63297
TP-16	0.82	51	25	46.43	0.34	0.354238029	0.3105	0.50987	0.67287
TP-17	0.59	59	26	48.37	0.391	0.321147841	0.4049	0.54333	0.73033
TP-18	0.92	52	24	50	0.34	0.342013556	0.3313	0.544	0.71
TP-19	0.36	64	28	46.1	0.293	0.274915356	0.4529	0.5329	0.7349
TP-20	0.85	54	27	50	0.41	0.424637464	0.3329	0.548	0.72

Table 5.14..In general ,the comparison of model 3 is as the following.

Cc=- 0.487769935+0.005971569*PL+0.0200864602*PI+0.19323883 63*LI+ (PI-23.853846154)*(LI-0.679615384)*0.0542180915).										Cc=- 1.4045+0.0155* w+0.0005*LL+0 .398*Gs- 0.00003*[(w- 68.837)*(LL- 105.48)]- 0.00002*[(w- 68.837)*(w- 68.837)].	Cc = - 1.6912 + 0.0118 * w + 0.5919 * Gs.
id no	LI	Gs	PL	LL	#p2 00	pi	wn	measur ed			
								cc	New model		
TP-1	0.71	2.8	37	64	81	27	56.1 3	0.422	0.4178950 78	0.592873052	2.150534
TP-2	1.48	2.76	27	48	85	21	58	0.26	0.2574279	0.595943866	2.148924
TP-3	1.16	2.7	33	58	69	25	62.1 2	0.463	0.4654625	0.651489943	2.162026
TP-4	0.9	2.71	30	59	75	29	56	0.52	0.5092899	0.550384316	2.095729
TP-5	0.86	2.56	28	52	59	24	48.6 8	0.331	0.3291238 42	0.354454016	1.920568
TP-6	0.92	2.7	29	55	67	26	53	0.42	0.4134045 02	0.490100236	2.05441
TP-7	1.29	2.7	27	54	67	27	61.8 7	0.56	0.5691934 18	0.644354383	2.159076
TP-8	1.08	2.69	27	53	68	26	55	0.47	0.4509972 37	0.519505776	2.072091
TP-9	0.4	2.81	34	63	83	29	45.5 6	0.288	0.2970495 06	0.411059417	2.031727
TP-10	0.73	2.72	31	57	75	26	50	0.37	0.3665237 92	0.447066816	2.030848
TP-11	0.9	2.65	31	52	83	21	49.8 1	0.254	0.2589791 89	0.410487547	1.987173
TP-12	0.66	2.68	32	61	89	29	51	0.4	0.4078922 71	0.452975136	2.018972
TP-13	0.72	2.57	27	50	68	23	43.5 3	0.269	0.2727134 13	0.263145144	1.865717
TP-14	0.76	2.7	29	55	72	26	48.6 7	0.41	0.3638686 3	0.413309937	2.003316
TP-15	1.02	2.48	25	45	70	20	45.3 3	0.191	0.1892293 6	0.253952318	1.833686
TP-16	0.82	2.65	26	51	72	25	46.4 3	0.34	0.3368320 29	0.348701526	1.947289
TP-17	0.59	2.72	33	59	70	26	48.3 7	0.391	0.3351230 42	0.420377853	2.011614
TP-18	0.92	2.7	28	52	70	24	50	0.34	0.3411936 23	0.433781266	2.01901
TP-19	0.36	2.65	36	64	70	28	46.1	0.293	0.2873449 8	0.358116654	1.943395
TP-20	0.85	2.72	27	54	67	27	50	0.41	0.4091138 09	0.443871486	2.030848

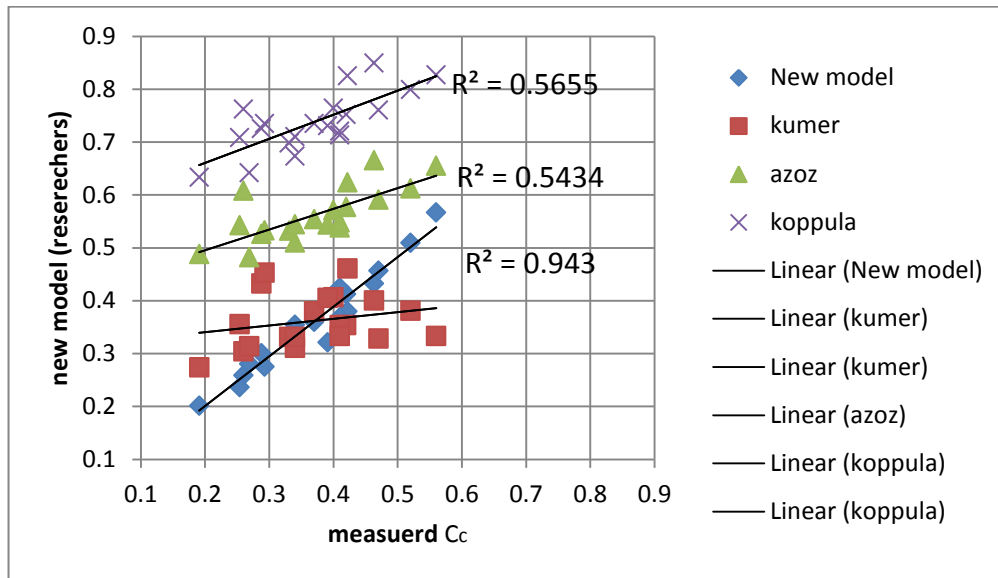


Figure 5.14.comprasion of model (more than 2)

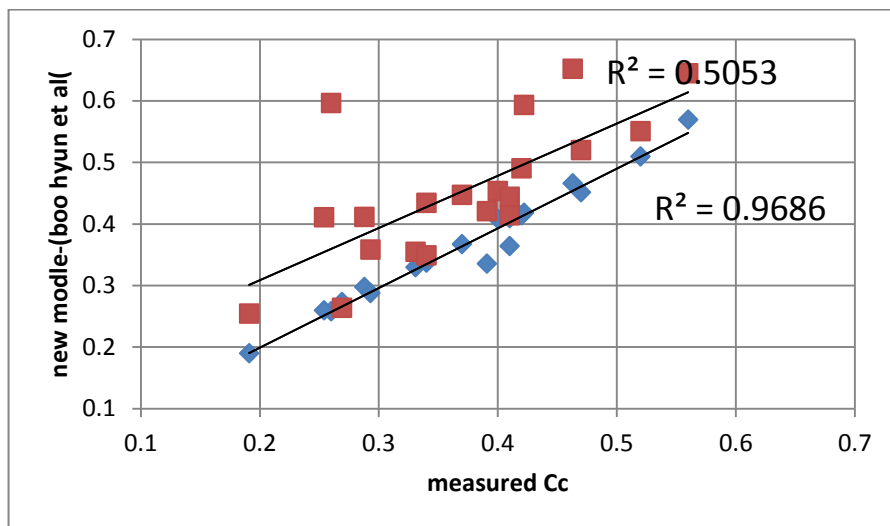


Figure.5.15.comparision of model 3

comparison and their R square values		
researcher	model2	model3
New model, 2021.	0.943	0.9686
nakase.et al., 1998.		
Vinod and Bindu, 2010.		
Sridharan and Nagaraj, 2000.		
Kumar et al, 2016.	0.00551	
Azzouz et al, 1976.	0.5434	
Koppula, 1981.	0.5655	
Boo Hyun et al.2019.		0.5053

Therefore, in the above cases (Figure 5.12, 5.13, 5.14 and 5.15) and their R² values, uses the newly developed models are the best fit to the data measured for the study area than when the data fitted with model developed from other regions.

CHAPTER SIX

CONCLUSION AND RECOMMENDATION

6.1. CONCLUSION

In order to make sure the normal use and safety of medium to super high-rise buildings, the design of most foundation must satisfy certain settlement requirements. Subgrade deformation is one of the most important factors of high-rise building design. For saturated subgrade, its compression mainly comes from the extrusion of water from the voids. The compression of subgrade is shown as the vertical deformation and lateral deformation. Generally, vertical deformation is the main part, so lots of research including this one puts emphasis on it. The subgrade compressibility index is the most important one used for calculating subgrade settlement. In common projects, the laboratory compression test with lateral deformation (lateral confined) is adopted to test compressibility index of subgrade sample. As a result, apart from the research on subgrade stress variation behavior, the compression behavior must also be researched as it is focused in this thesis.

Therefore, the conclusions and recommendation made from this study are summarized and presented with respect to the findings from two major studies conducted, these are: (a) the geotechnical engineering properties of fine grained soils studied and (b) the statistical C_c predictive model developed to predict Compression Index from index properties of soil in Azezo Tseda Sub City, Gondar. To achieve the goals of the major objective set in this research study, that is to develop a statistical model to predict compression index from index properties of fine grained soils in Azezo Tseda Sub City, Gondar. In this regard several steps/phases of studies were made under the different chapters. These studies include: undertaking detail literature review related to this research, conducting index property tests and classification of the fine grained soils, conducting one dimensional consolidation tests and determining compression index parameter; developing a new simplified statistical model helpful to predict compression index from index properties of the fine grained soil using multiple regression analysis with subset selection and interaction routines, undertaking validation of the newly developed prediction model using primary and secondary data and lastly performing comparative studies of the newly developed prediction model with other similar models.

The main intention that leads this research was the Coefficient of compression (C_c) is a major geotechnical soil parameter required quite frequently for determining settlement

of fine-grained soil layers subjected to loads such as that of buildings, vehicles, and infrastructures. If C_c is not known accurately, soil settlement is under- or overestimated, which both cases are not desirable from technical prospective. Thus, determining C_c is of significance importance in settlement calculations. However, measuring C_c using the traditional Oedometer test method is time-consuming, that demands skilled technicians, and requires special laboratory equipment. These bring out that, estimation of C_c using index parameters of fine-grained soils are a justified cause. Therefore, the conclusions and recommendation pointed out from this study are presented as follows.

6.1.1. Conclusions from the Geotechnical Engineering Properties of Fine Grained Soils Studied

Even though there are different types of soils in the sub city, the fine grained soils studied in the Azezo Tseda sub city mainly Silty Clay and Clay with High and low plasticity is characterized in this study. The main intention was to make the developed models more specific to the corresponding soil types. Therefore, the following points are vital to conclude from the study, these are:

Based on different index property tests made, the fine grained soils studied in the region are having a natural water content (%) value range from 43.53-62.12%, Specific Gravity (G_s) value range from 2.5- 2.8, Liquid Limit (%) range between 45-64% and the %Passing #200 Sieve size are greater than 50%.

- (a) . The fine grained soils studied in the Azezo Tseda sub city focused in this research is mainly Silty Clay and Clay with High and Low plasticity, the classification and identification tests made suggest the fine grained soil studied is non-expansive nature.
- (b) . The Compression Index value calculated from one dimensional consolidation/ Oedometer test outputs determined from e-log p curve for these Silty Clay and Clay with High and Low plasticity fine grained soils studied in Azezo Tseda sub city suggest that, the recommended range for settlement computation is between 0.19 - 0.56 with a mean C_c value of 0.37.
- (c) . Here in this study, based on the range of the C_c values, it is concluded that the soils in the study area are having a medium compressible nature. The bigger the compressibility index is, the higher the subgrade compressibility is. C_c value is generally smaller than 0.2 for the slightly compressible subgrade; while C_c value is bigger than 0.4 for the highly compressible subgrade.

6.1.2. Conclusions from the C_c Predictive Model Development and Validation

In these part, the study was made: - to develop a new simplified statistical model helpful to predict compression index from index properties of the fine grained soil using multiple regression analysis software with subset selection and interaction routines (JMP-

13), to undertake validation and calibration of the newly developed predictive model using primary and secondary data and finally performing comparative studies of the newly developed predictive model with other similar models. Therefore the following conclusions are made from this study:

- (a) . Here in this part of the study, three new simplified statistical models that can predict Compression Index (C_c) value from index property test data more accurately and reasonably are developed, validated and calibrated. These models are one, two and three parameter models respectively for Model 1, 2 and 3.
- (b) . By referring to the three newly simplified statistical models developed, it is concluded that Compression Index (C_c) value can be predicted more reasonably only from the Atterberg limit test data.
- (c) . The Multiple Regression Analysis showed that there is a relationship between index properties and compression characteristics of the fine grained soils of Azezo-Tseda area. Especially Plasticity Index for Model 1, Plasticity Index and Liquidity Index for Model 2 and Plasticity Index, Plastic Limit and Liquidity Index for Model 3 are found a significant/powerful variable for prediction of compression index of the fine grained soils found around Azezo-Tseda sub city area.
- (d) . The three C_c predictive models satisfy all the required statistical goodness of fitting measures and the different multiple regression assumptions, however comparison of the new models each other suggest that Model 3 has the highest significance recommended for practice.
- (e) . Comparative studies suggest, the newly developed models are the best fit to the data measured for the study area than when the data fitted with model developed from other regions.
- (f) . Such predictive models will bring a lot of conveniences to the engineers working in the geotechnical field whenever the need arises to perform quick and simple identification of the compressibility characteristics of the fine grained soils for the specific study area. It will also be helpful to the civil engineers during planning the construction of an appropriate structure with the soil conditions without having to make all tests for determining soil properties.

6.2. RECOMMENDATIONS

The following are recommended for further development of the work

- (a) . The equation developed may be further improved by increasing the number of samples and tests performed on a number of undisturbed samples during the driest season of the year.
- (b) . The prediction of compression index by using empirical relationships cannot be expected to yield accurate results. Therefore, for detail investigation compression index should be determined from Oedometer tests on a sample that have an expected initial condition that could yield maximum compression index. For small projects and for preliminary design purpose of any size of building the equations developed can be used to predict the compression index.

- (c) . Field validation of the developed soil compressibility models is recommended.
- (d) . These models are intended to supplement but not eliminate a geotechnical sampling and testing program.

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APPENDIX- A

Laboratory Test Result

Different Test Results

Appendex.A1.Sample Representative of Moisture Content Test Results

Formula

$$W = \frac{M2-M3}{(M3-M1)} * 100\%$$

Table A.1 determination of moisture content for test pit tp 1

Sampl 1		
depth	2.5-3m	
container no	A9	45G
mass of container(M1),gm	36.67	37.26
mass of container wet soil (M2),gm	61.16	60.56
mass of container+drysiol (M3),gm	52.53	52.04
mass of water (M2-M3), gm	8.63	8.55
mass of dry soil (M3-M1),kg	15.86	14.78
moistur content(w) %	54.41	57.84
average moistur content (w) %	56.13	

Table A.2. determination of moisture content for test pit tp 4

sampl 4		
depth	2.5-3m	
contianer no	11	564
mass of container(M1),gm	40	38.1
mass of container+wet soil (M2),gm	65	66.5
mass of container+drysiol (M3),gm	56.15	56.16
mass of water (M2-M3), gm	8.85	10.34
mass of dry soil (M3-M1),GM	16.15	18.06
moistur content(w) %	54.79	57.25
average moistur content (w) %	56.00	

Table A.3 determination of moisture content for test pit tp 5

sample 5		
depth	2.5-3m	
contianer no	19	94
mass of container(M1),gm	37.1	35.5
mass of container+wet soil (M2),gm	64.7	66.6
mass of container+drysiol (M3),gm	55.6	56.49
mass of water (M2-M3), gm	9.1	10.11
mass of dry soil (M3-M1),GM	18.5	20.99
moistur content(w) %	49.19	48.17
average moistur content (w) %	48.68	

Table A.4.determination of moisture content for test pit tp 8

Sampl 8		
depth	2.5-3m	
contianer no	49	34
mass of container(M1),gm	37	36.5
mass of container+wet soil (M2),gm	71.4	64
mass of container+drysiol (M3),gm	58.8	54.55
mass of water (M2-M3), gm	12.6	9.45
mass of dry soil (M3-M1),GM	21.8	18.05
moistur content(w) %	57.79	52.35
average moistur content (w) %	55.00	

Table A.5 determination of moisture content for test pit tp 9

Sampl 9		
depth	2.5-3m	
contianer no	T35	P20
mass of container(M1),gm	36.2	38.1
mass of container+wet soil (M2),gm	61.12	65.4
mass of container+drysiol (M3),gm	52.9	57.4
mass of water (M2-M3), gm	8.22	8
mass of dry soil (M3-M1),kg	16.7	19.3
moistur content(w) %	49.22	41.45
average moistur content (w) %	45.33	

Table A.6.determination of moisture content for test pit tp 12

Sampl 12		
depth	2.5-3m	
contianer no	5D	L10
mass of container(M1),gm	38.3	37.6
mass of container+wet soil (M2),gm	76.6	71.8
mass of container+drysiol (M3),gm	64	59.95
mass of water (M2-M3), gm	12.6	11.85
mass of dry soil (M3-M1),GM	25.7	22.35
moistur content(w) %	49.02	53.02
average moistur content (w) %	51.00	

Table A.7. Determination of moisture content for test pit tp 13

Sampl 13		
depth	2.5-3m	
contianer no	2B	L23
mass of container(M1),gm	37.12	37.38
mass of container+wet soil (M2),gm	75.5	70.9
mass of container+drysiol (M3),gm	64.5	60.2
mass of water (M2-M3), gm	11	10.7
mass of dry soil (M3-M1),GM	27.38	22.82
moistur content(w) %	40.18	46.89
average moistur content (w) %	43.53	

Table A.8.determination of moisture content for test pit tp 16

sampl 16		
depth	2.5-3m	
contianer no	R2	K24
mass of container(M1),gm	37.3	37.1
mass of container+wet soil (M2),gm	60.1	66.9
mass of container+drysiol (M3),gm	52.8	57.52
mass of water (M2-M3), gm	7.3	9.38
mass of dry soil (M3-M1),kg	15.5	20.42
moistur content(w) %	47.09	45.94
average moistur content (w) %	46.52	

Table A.9. Determination of moisture content for test pit tp 17

sampl 17		
depth	2.5-3m	
contianer no	F10	M2
mass of container(M1),gm	37.1	36.53
mass of container+wet soil (M2),gm	60.1	66.85
mass of container+drysiol (M3),gm	52.5	57.1
mass of water (M2-M3), gm	7.6	9.75
mass of dry soil (M3-M1),kg	15.4	20.57
moistur content(w) %	49.35	47.39
average moistur content (w) %	48.37	

Table A.10. Determination of moisture content for test pit tp 20

SAMPL 20		
depth	2.5-3m	
contianer no	A12	T10
mass of container(M1),gm	38	37.96
mass of container+wet soil (M2),gm	59.99	68.11
mass of container+drysiol (M3),gm	52.62	57.98
mass of water (M2-M3), gm	7.37	10.13
mass of dry soil (M3-M1),kg	14.62	20.02
moisture content(w) %	50.41	50.59
average moisture content (w) %	50.50	

Appendix B.1. Sample Representative of Specific gravity test results

Formula used

$$GS = K \left[\frac{(m_2 - m_1)}{(m_2 - m_1) - (M_3 - M_4)} \right]$$

Table A.2.0: Temperature Correction Factors for Gs. Temperatur 20 0 c.
Relative density of water at different temperature and its correction factor

Temp.oc	Relative density of water	Temp.corre.factor.K
16	0.99895	1.0007
17	0.99878	1.0006
18	0.99862	1.0004
19	0.99843	1.0002
20	0.99823	1.0000
21	0.99802	0.9998
22	0.99780	0.9996
23	0.99757	0.9993
24	0.99733	0.9991
25	0.99708	0.9988
26	0.99682	0.9986
27	0.99645	0.9983
28	0.99626	0.9980
29	0.99598	0.9977
30	0.99568	0.9974

Table B.2. Determination of specific gravity for test pit TP_1 by using density bottle method

	Test pit 1	
Density no	1	2
Mass of empty density bottle (M1)	42.85	47.6
Mass of bottle dry soil(M2)	67.85	72.65
Mass of bottle soil and water(M3)	117	114.4
Mass of bottle filled with water((M4)	100.9	98.3
M2-M1	25	25.05
M3-M4	16.1	16.1
(M2-M1)-(M3-M4)	8.9	8.95
Gs	2.81	2.79
Average Gs	2.80	

Table B.3.Determination of specific gravity for test pit TP_4

	Test pit 4	
Density no	1	2
Mass of empty density bottle (M1)	42.75	47.65
Mass of bottle dry soil(M2)	67.7	73.4
Mass of bottle soil and water(M3)	116	115.6
Mass of bottle filled with water((M4)	100.25	99.3
M2-M1	24.95	25.75
M3-M4	15.75	16.3
(M2-M1)-(M3-M4)	9.2	9.45
Gs	2.71	2.72
Average Gs	2.71	

Table B.4. Determination of specific gravity for test pit TP_5

	Test pit 5	
Density no	1	2
Mass of empty density bottle (M1)	43.46	47.4
Mass of bottle dry soil(M2)	68.6	73.12
Mass of bottle soil and water(M3)	116.11	113.2
Mass of bottle filled with water((M4)	100.47	97.9
M2-M1	25.14	25.72
M3-M4	15.64	15.3
(M2-M1)-(M3-M4)	9.5	10.42
Gs	2.64	2.46
Average Gs	2.56	

Table B.5.Determination of specific gravity for test pit TP_8

	Test pit 8	
Density no	1	2
Mass of empty density bottle (M1)	48.4	47.5
Mass of bottle dry soil(M2)	68.5	69.5
Mass of bottle soil and water(M3)	110.35	109
Mass of bottle filled with water((M4)	97.9	95
M2-M1	20.1	22
M3-M4	12.45	14
(M2-M1)-(M3-M4)	7.65	8
Gs	2.63	2.75
Average Gs	2.69	

Table B.6. Determination of specific gravity for test pit TP_9

	Test pit 9	
Density no	1	2
Mass of empty density bottle (M1)	46.1	46.4
Mass of bottle dry soil(M2)	68.8	69.5
Mass of bottle soil and water(M3)	113	112.5
Mass of bottle filled with water((M4)	98.2	97.8
M2-M1	22.7	23.1
M3-M4	14.8	14.7
(M2-M1)-(M3-M4)	7.9	8.4
Gs	2.87	2.75
Average Gs	2.81	

Table B.7. Determination of specific gravity for test pit TP_12

	Test pit 12	
Density no	1	2
Mass of empty density bottle (M1)	45.8	43.1
Mass of bottle dry soil(M2)	68.9	66.9
Mass of bottle soil and water(M3)	112	108.9
Mass of bottle filled with water((M4)	97.5	94
M2-M1	23.1	23.8
M3-M4	14.5	14.9
(M2-M1)-(M3-M4)	8.6	8.9
Gs	2.68	2.67
Average Gs	2.68	

Table B.8. Determination of specific gravity for test pit TP_13

	Test pit 13	
Density no	1	2
Mass of empty density bottle (M1)	45.5	47.7
Mass of bottle dry soil(M2)	69.3	71.2
Mass of bottle soil and water(M3)	113.2	111.8
Mass of bottle filled with water((M4)	98.6	97.5
M2-M1	23.8	23.5
M3-M4	14.6	14.3
(M2-M1)-(M3-M4)	9.2	9.2
Gs	2.58	2.55
Average Gs	2.57	

Table B.9. Determination of specific gravity for test pit TP_16

	Test pit 16	
Density no	1	2
Mass of empty density bottle (M1)	45.3	42.4
Mass of bottle dry soil(M2)	68.8	65.8
Mass of bottle soil and water(M3)	112.3	103.4
Mass of bottle filled with water((M4)	97.8	88.7
M2-M1	23.5	23.4
M3-M4	14.5	14.7
(M2-M1)-(M3-M4)	9	8.7
Gs	2.61	2.69
Average Gs	2.65	

Table .B.9. Determination of specific gravity for test pit TP_17

	Test pit 17	
Density no	1	2
Mass of empty density bottle (M1)	46.8	47.4
Mass of bottle dry soil(M2)	71.5	65.2
Mass of bottle soil and water(M3)	105.2	99.8
Mass of bottle filled with water((M4)	89.5	88.6
M2-M1	24.7	17.8
M3-M4	15.7	11.2
(M2-M1)-(M3-M4)	9	6.6
Gs	2.74	2.69
Average Gs	2.72	

Table.B.10. Determination of specific gravity for test pit TP_20

	Test pit 20	
Density no	1	2
Mass of empty density bottle (M1)	43.9	47.2
Mass of bottle dry soil(M2)	69.6	71.5
Mass of bottle soil and water(M3)	112.4	110.5
Mass of bottle filled with water((M4)	96.2	95.1
M2-M1	25.7	24.3
M3-M4	16.2	15.4
(M2-M1)-(M3-M4)	9.5	8.9
Gs	2.71	2.73
Average Gs	2.72	

Appendec.C.1. sample Representative Liquid Limit and Plastic Limit test results

Table C.1. Liquid limit and plastic limit by using con penetration method for test pit TP_1

Trial no	1	2	3	4	1	2
Container no	B9	G30	456	13	18	A13
Mass of container(gm)	35.1	35.6	35.8	36.2	37	37.1
Mass of cont.+wet soil gm	61.2	62.1	63	63	39.7	39.5
Mass of cont.+dry soil gm	51.6	52	52.1	52.1	39.02	38.84
Water content (w) (%)	58.2	61.2	66.9	68.6	36.2	37.6
Con penetration mm	15	18.5	21.5	25		
Average limit LL (%)	64				Plastic limit PL (%) = 37	

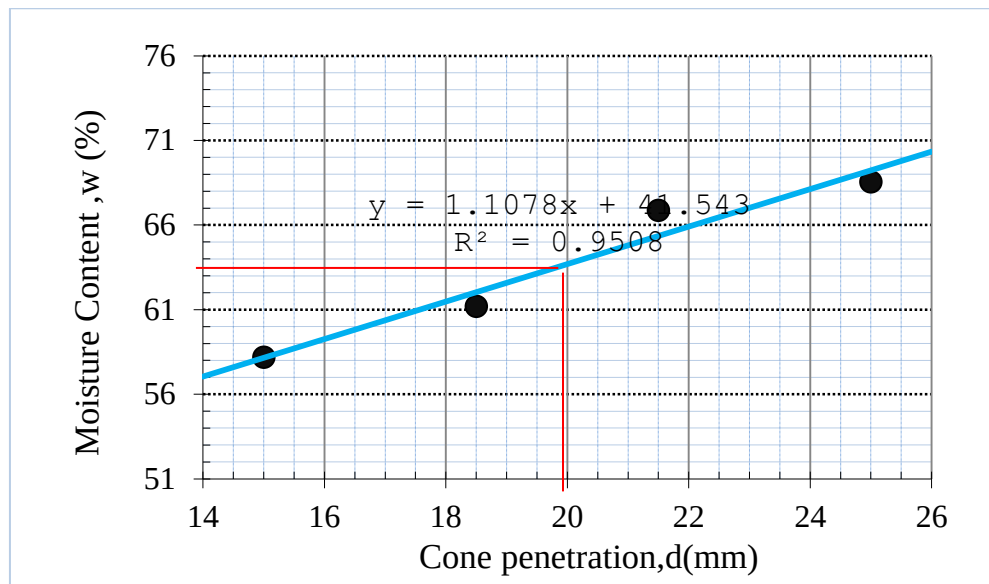


Figure C.1 Liquid limit for pt -1

Table C.2. Liquid limit and plastic limit test pit TP_4

Trial no	1	2	3	4	1	2
Container no	3A	P2	R	C2	577	923
Mass of container(gm)	37.4	36.8	36.5	37.2	37.6	37.0
Mass of cont.+wet soil gm	76.2	72.3	73.5	88.1	38.7	40.0
Mass of cont.+dry soil gm	63.1	59.3	59.2	67.8	38.45	39.29
Water content(w) (%)	50.6	57.8	62.6	66.7	29.4	31.0
Con penetration mm	15	18.5	22	25		
Average limit LL (%)	59				Plastic limit PL (%) = 30	

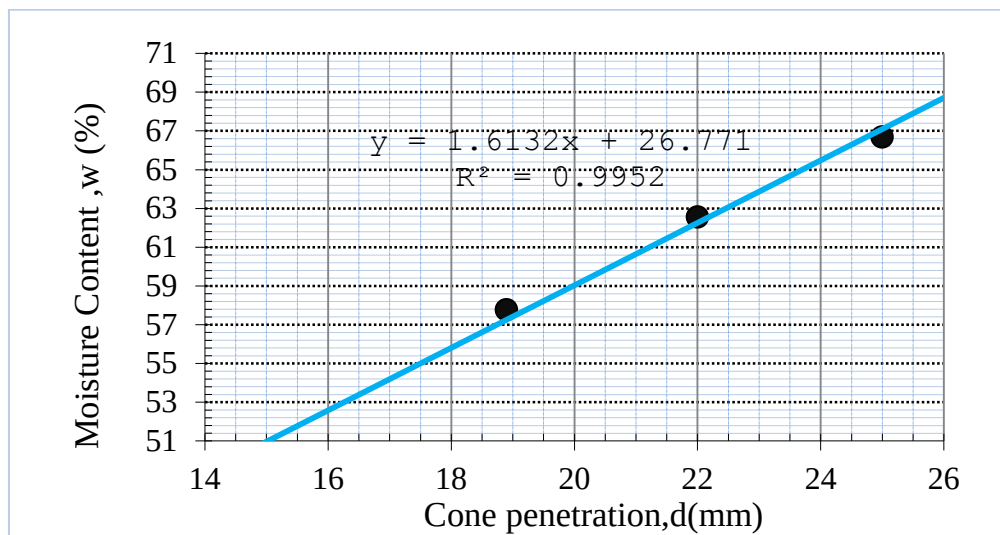


Figure C.2 Liquid limit for pt -4

Table C.3 Liquid limit and plastic limit test pit TP_5

Trial no	1	2	3	4	1	2
Container no	D5	D1	45	18	12	A11
Mass of container(gm)	34.5	35.6	34.5	37.5	37.3	37.0
Mass of cont.+wetsoil gm	60	62	62.5	62.5	40.1	39.3
Mass of cont.+dry soil gm	52.5	53.1	52.5	53.2	39.5	38.8
Water content(w) (%)	41.7	50.9	55.6	59.2	21.3	27.8
Con penetration mm	15	18.5	21.5	25		
Average limit LL (%)	52				Plastic limit PL (%) = 37	

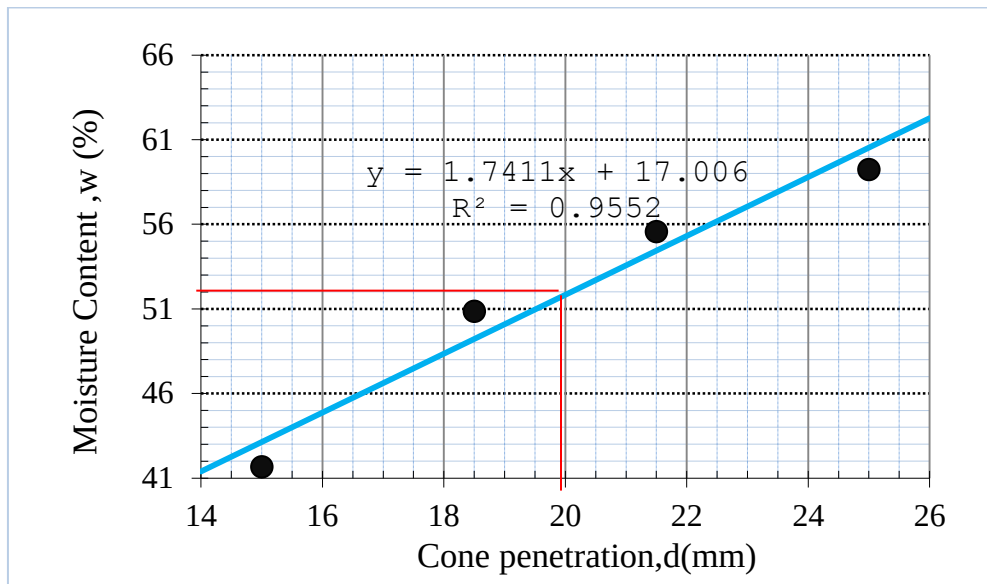


Figure C.3 Liquid limit for pt -5

Table C.4. Liquid limit and plastic limit test pit TP_8

Trial no	1	2	3	4	1	2
Container no	3A	B4	2B	C2	531	919
Mass of container(gm)	37.2	35.9	36.5	37	37.5	37
Mass of cont.+wet soil gm	76.5	71.2	72.5	87	38.7	39.9
Mass of cont.+dry soil gm	63.9	59.7	59.5	68	38.45	39.29
Water content(w) (%)	47.2	48.3	56.5	61.3	26.90%	26.60%
Con penetration mm	15	18.9	22	25		
Average limit LL (%)	53				Plastic limit PL (%) =27%	

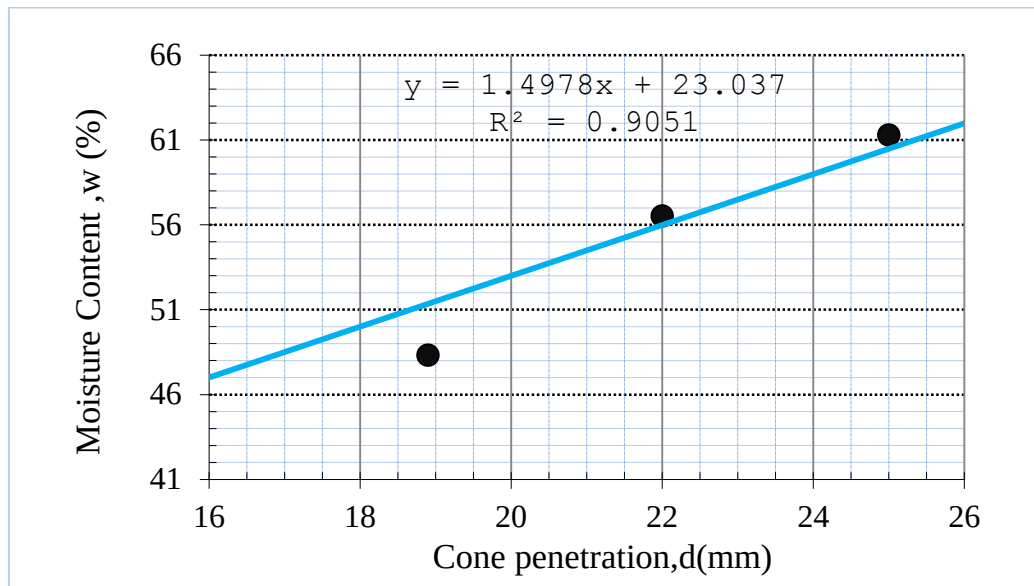


Figure C.4. Liquid limit for pt -8

Table C.5. Liquid limit and plastic limit test pit TP_9

Trial no	1	2	3	4	1	2
Container no	11	578	A4	G1	27	16
Mass of container(gm)	37.1	35.2	36.5	36.9	34.1	35.0
Mass of cont.+wet soil gm	65	67.9	72.5	69	39.4	38.5
Mass of cont.+dry soil gm	56	57.9	57.2	54.8	38.1	37.6
Water content(w) (%)	47.6	44.1	73.9	79.3	32.5	34.6
Con penetration mm	15	17.8	21	24.6		
Average limit LL (%)	63				Plastic limit PL (%) =34%	

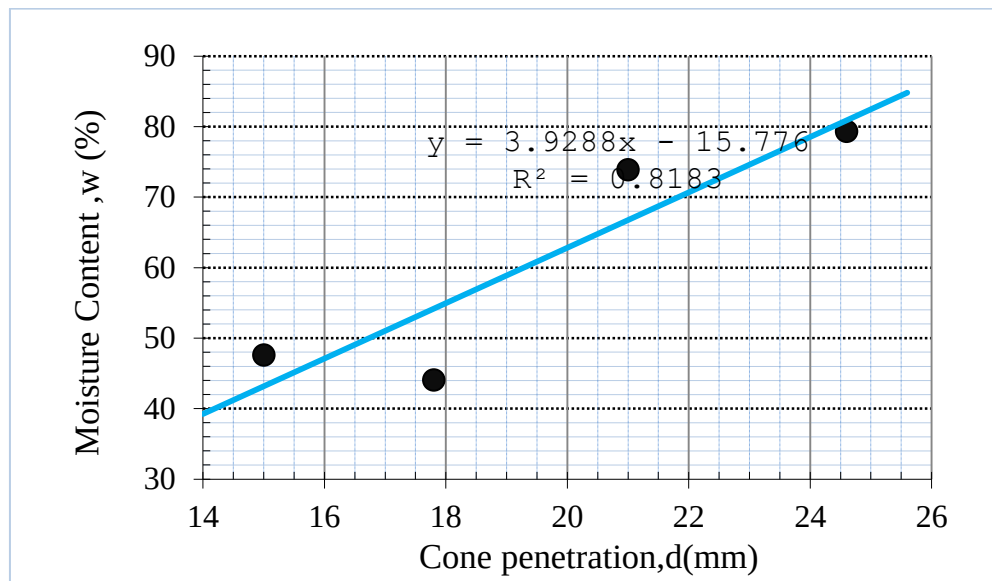


Figure C.5. Liquid limit for pt -9

Table C.6. Liquid limit and plastic limit test pit TP 12

Trial no	1	2	3	4	1	2
Container no	F21	H10	428	13	754	41
Mass of container(gm)	37.3	37.2	36.5	36.7	37.2	36.5
Mass of cont.+wet soil gm	83.9	88.8	82.5	84.5	40.1	41.0
Mass of cont.+dry soil gm	69.2	70.2	63.3	65	39.4	39.9
Water content(w) (%)	45.9	56.4	72	68.9	31.8	32.4
Con penetration mm	15	18	21.9	24.8		
Average limit LL (%)	61				Plastic limit PL (%) PL=32%	

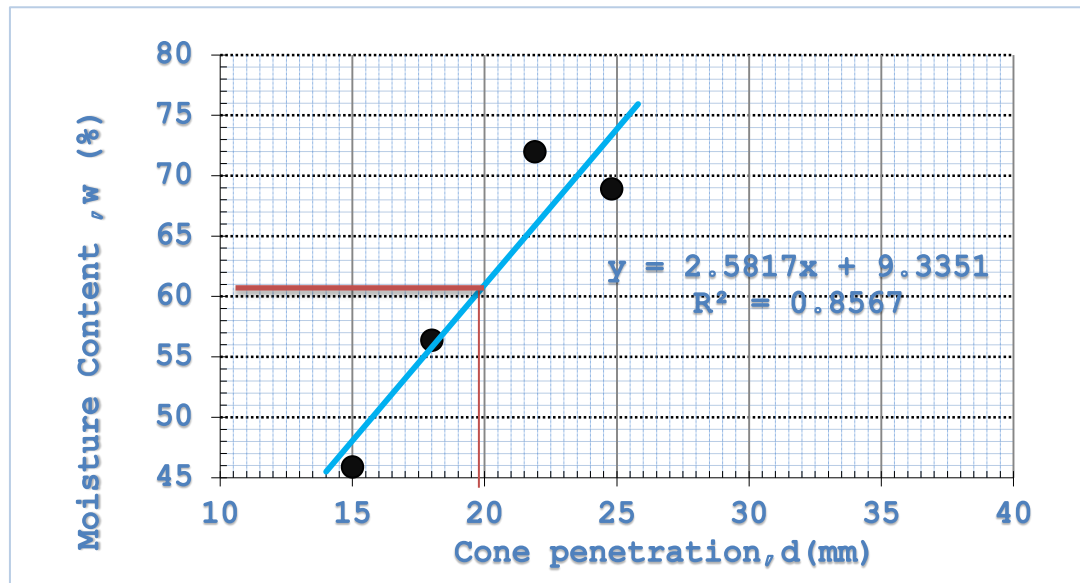


Figure C.6. Liquid limit for pt -12

Table C.7. Liquid limit and plastic limit test pit TP 13

Trial no	1	2	3	4	1	2
Container no	A17	B10	A3	D6	389	83
Mass of container(gm)	35.9	35.4	36.5	37.2	34.5	35.1
Mass of cont.+wet soil gm	78.3	77.8	77.1	82.3	41.5	41.3
Mass of cont.+dry soil gm	65.2	65.5	63.2	65.5	40.0	39.9
Water content(w) (%)	44.7	40.8	52.1	59.2	25.5	29.2
Con penetration mm	15.5	16	21.5	25		
Average limit LL (%)	50				Plastic limit PL (%) =27%	

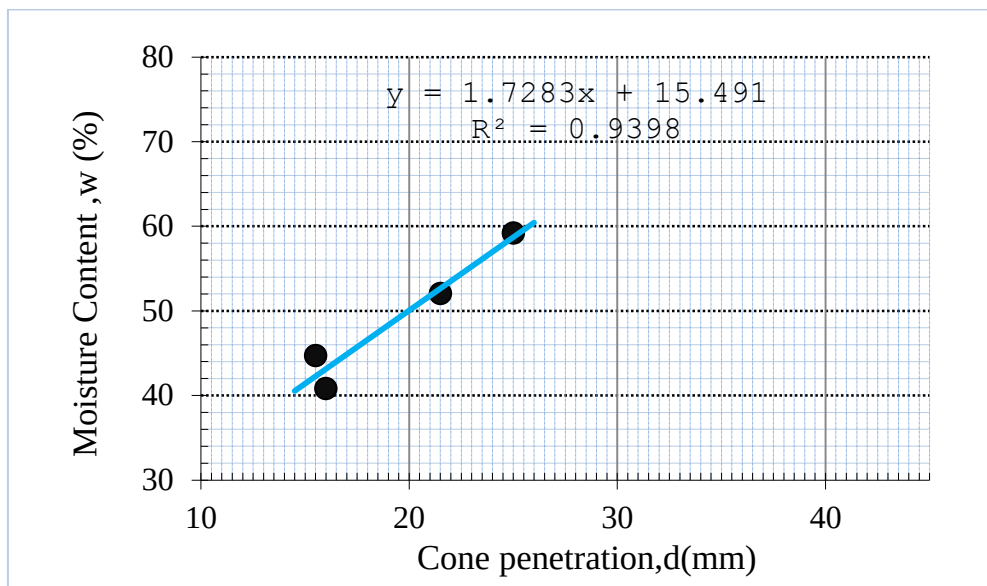


Figure C.7. Liquid limit for pt -13

Table C.8. Liquid limit and plastic limit test pit TP 16

Trial no	1	2	3	4	1	2
Container no	2k	4G	5A	12B	25	36
Mass of container(gm)	36.3	36.2	35.5	37.8	35.7	34.6
Mass of cont.+wet soil gm	85.4	84.6	84.3	84	41.2	40.4
Mass of cont.+dry soil gm	70	69	68	67	40.1	39.2
Water content(w) (%)	45.7	47.6	50.2	58.2	25	26.1
Con penetration mm	15	17.5	22	24.9		
Average limit LL (%)	51				Plastic limit PL (%) = 26	

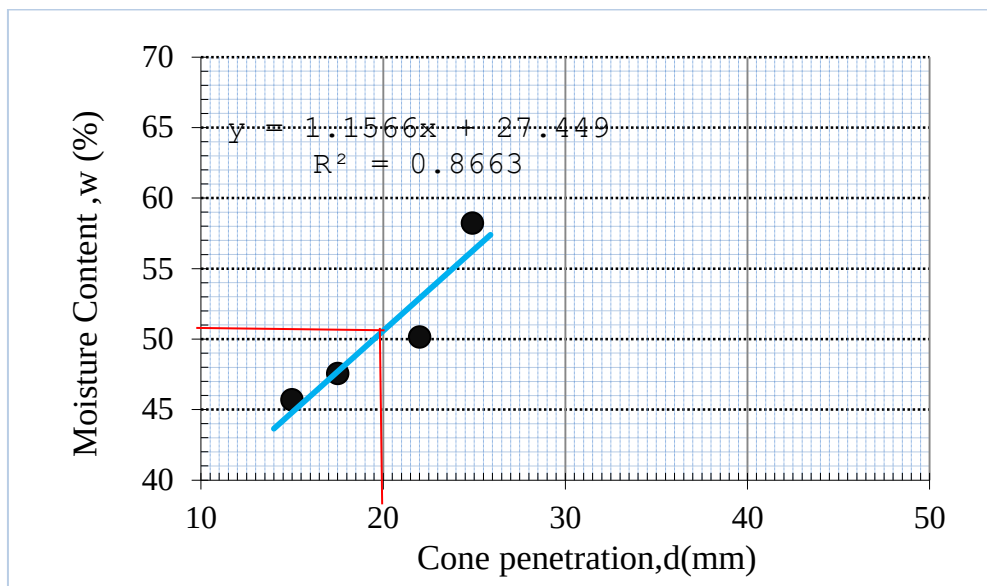


Figure C.8. Liquid limit for pt -16

Table C.9. Liquid limit and plastic limit test pit TP 17

Trial no	1	2	3	4	1	2
Container no	831	792	11A	2R	63	36
Mass of container(gm)	36.5	37.8	37.2	35.2	34.5	35.4
Mass of cont.+wet soil gm	83.1	84	86.5	83.5	43.7	41.5
Mass of cont.+dry soil gm	69	67.5	66.7	64	41.5	40
Water content(w) (%)	43.4	51.6	67.1	67.7	33.3	32.6
Con penetration mm	15	17.5	22	24.9		
Average limit LL (%)	59				Plastic limit PL (%) =33%	

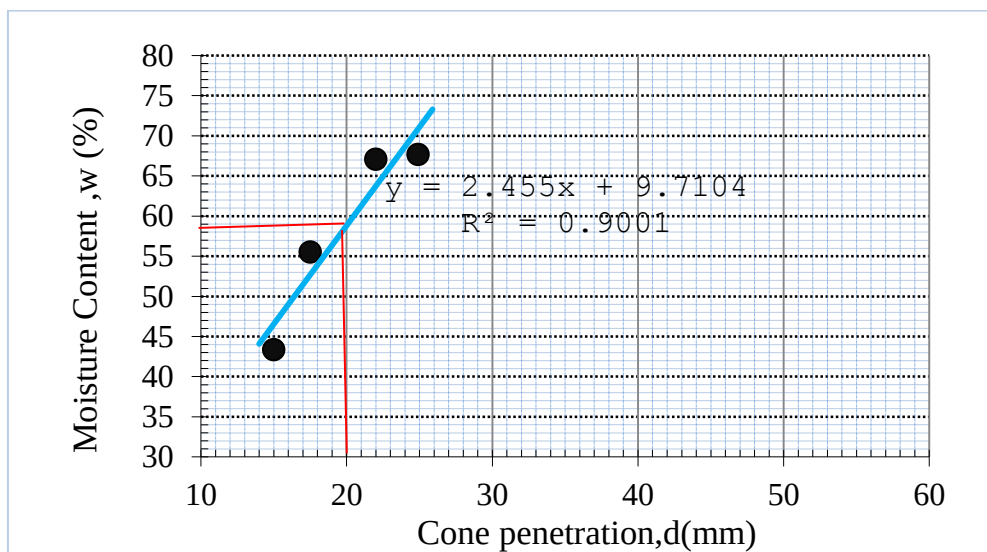


Figure C.9. Liquid limit for pt -17

Table C.10. Liquid limits and plastic limit test pit TP 20

Trial no	1	2	3	4	1	2
Container no	A0	B0	C0	D0	56	24
Mass of container(gm)	37.8	36.8	37	36.5	35.5	36.8
Mass of cont.+wet soil gm	82.1	87.1	78.8	82.5	40.5	41.3
Mass of cont.+dry soil gm	69	69.5	63.2	65	39.4	40.36
Water content(w) (%)	42	53.8	59.5	61.4	28.2	26.4
Con penetration mm	15	18	21.9	24.8		
Average limit LL (%)	54				Plastic limit PL (%) =27%	

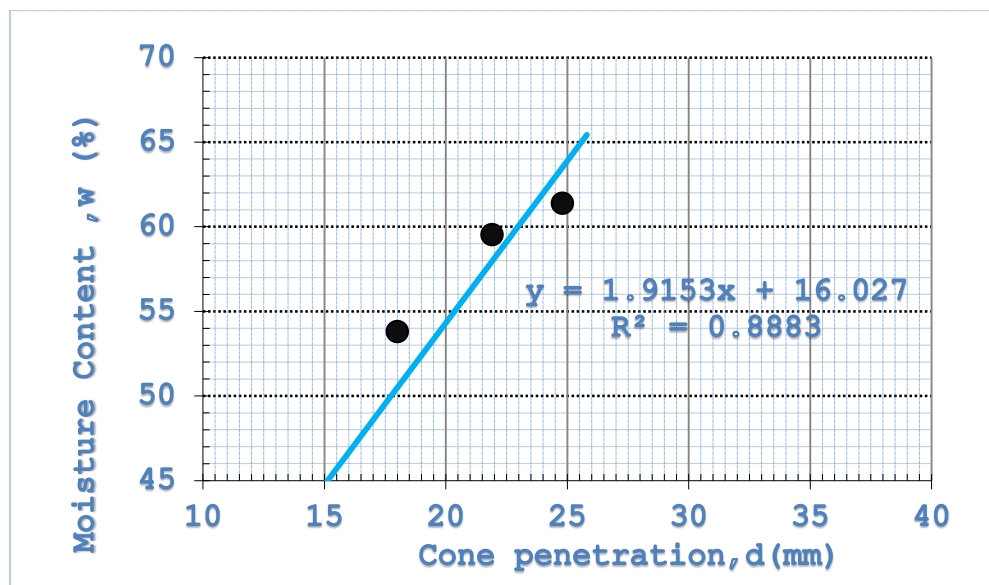


Figure C.10. Liquid limit for pt -20

Appendix D.1. Sample Representative Grain Size Analysis test results

Formula used

For Sieve Analysis

Mass of retained = Mass of sieve + retained - Mass of sieve

Percentage retained = (Mass of retained / Total mass used in sieve analysis)*100%

Cumulative percentage retained = Summation of Percentage retained at each sieve

Percentage finer = 100 - Cumulative percentage retained

For Hydrometer Analysis

Corrected Hydrometer reading (RC) = Actual Hydrometer reading (RA) - Composition correction

Effective depth (L), cm = from the ASTM 151H table

$$\text{Grain size (D), mm} = K * \sqrt{\frac{L}{T}}$$

Where T= Elapse time, min

K= Constant from table

$$\text{Percentage finer(P)} = \left\{ \left[\frac{G_s}{G_s - G_1} \right] * \frac{V}{M_s} \right\} * (R_c - G_1) * 100\%$$

Where G_s=specific gravity of soil used

G₁= specific gravity of water=1

V= volume of suspension=1000ml

M_s= Dry mass of soil =50gm

$$\text{Combined Percentage finer(Pa)} = P * \frac{W_1}{W}$$

Where W₁ = weight soil finer than No 200 sieve

W = Total soil mass used in combined analysis

Hydrometer analysis is based on the principle of sedimentation of soil grains in water. When a soil specimen is dispersed in water, the particles settle at different velocities, depending on their

Shape, size, and weight, and the viscosity of the water.

Table D.0.1. Values of k for Use in Equation for Computing Diameter of Particle in Hydrometer analysis.

Temp.,c	Specific Gravity of Soil Particles								
	2.45	2.5	2.55	2.6	2.65	2.7	2.75	2.8	2.85
16	0.0151	0.01505	0.01481	0.01457	0.01435	0.01414	0.01394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.01323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01225	0.01208	0.01191	0.01175
29	0.01312	0.0129	0.01269	0.01269	0.0123	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

Table D.1. Sieve analysis for test pit TP_1

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.9	336.9	0	0.0	0.0	100.0
1.18	280.6	284.3	3.72	2.5	2.5	97.5
0.600	265.6	278.0	12.43	8.3	10.8	89.2
0.425	379.1	380.3	1.25	0.8	11.6	88.4
0.300	262.4	264.9	2.5	1.7	13.3	86.7
0.150	266.3	269.4	3.1	2.1	15.3	84.7
0.075	253.5	259.0	5.54	3.7	19.0	81.0
pan	268.0	389.0	121.0	80.9	100.0	0.0

Table D.1 Hydrometer Analysis pt1

	G1=1.00	Gs=2.8	Temp. °C = 20		K =0.01307				
Elapsed Time (min)	Actual Hyd.Reading	Hyd.Reading taken at suspension	Temp. °C	Composite correction	Corr. Hyd.Reading (R)	L(cm)	D (mm)	P(%)	Adjusted P(%)
2	1.025	1.001	20	0.001	1.024	12.9	0.03319	74.67	60.44
5	1.024	1.001	20	0.001	1.0230	13.1	0.02116	71.56	57.92
15	1.022	1.001	20	0.001	1.021	13.4	0.01235	65.33	52.89
30	1.018	1.001	20	0.001	1.017	13.7	0.00883	52.89	42.81
60	1.014	1.001	20	0.001	1.013	13.9	0.00629	40.44	32.74
250	1.011	1.001	20	0.001	1.010	14.2	0.00311	31.11	25.18
1440	1.010	1.001	20	0.001	1.009	14.4	0.00131	28.00	22.67

Table D.2. sieve analysis for PT- 4

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	288.3	7.7	2.5	2.5	97.5
0.600	265.6	287.9	22.33	7.2	9.7	90.3
0.425	379.0	386.6	7.6	2.5	12.2	87.8
0.300	262.5	281.4	18.9	6.1	18.3	81.7
0.150	266.6	279.8	13.2	4.3	22.6	77.4
0.08	254.5	261.8	7.3	2.4	25.0	75.0
Pan	289.6	521.0	231.4	75.0	100.0	0.0

Table D.2.Hydrometer Analysis pt -4

	G1=1.00	Gs=2.71	Temp. °c =20	K= 0.013402					
Elaps ed Time (min)	Actual Hyd.Rea ding	Hyd.Rea ding taken at suspensi on	Temp. °c	Comp osite correct ion	Corr. Hyd.R eading (R)	L(c m)	D (mm)	P(%)	Adjus ted P(%)
2	1.025	1.001	20	0.001	1.024	9.4	0.0290 5	76.07	57.07
5	1.024	1.001	20	0.001	1.0230	9.7	0.0186 7	72.90	54.69
15	1.023	1.001	20	0.001	1.022	10.2	0.0110 5	69.73	52.31
30	1.021	1.001	20	0.001	1.020	10.5	0.0079 3	63.39	47.56
60	1.019	1.001	20	0.001	1.018	11.3	0.0058 2	57.05	42.80
250	1.017	1.001	20	0.001	1.016	11.8	0.0029 1	50.71	38.05
1440	1.015	1.001	20	0.001	1.014	12.3	0.0012 4	44.37	33.29

Table D.3. sieve analysis for PT- 5

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	296.8	16.2	4.1	4.1	95.9
0.600	265.6	298.4	32.83	8.4	12.5	87.5
0.425	379.1	396.1	17.03	4.3	16.8	83.2
0.300	262.4	290.5	28.06	7.2	24.0	76.0
0.150	266.3	298.2	31.86	8.1	32.1	67.9
0.075	253.5	288.2	34.74	8.9	41.0	59.0
Pan	289.6	521.0	231.4	59.0	100.0	0.0

Table D.3.Hydrometer Analysis pt5

	G1=1.00	Gs=2.56	Tem p. °c =20	K =0.014036					
Elaps ed Time (min)	Actual Hyd.Rea ding	Hyd.Re ading taken at suspens ion	Temp . °c	Composite correction	Corr. Hyd. Read ing (R)	L(c m)	D (mm)	P(%)	Adjus ted P(%)
2	1.026	1.001	20	0.001	1.025	9.4	0.0304 3	82.05	48.42
5	1.025	1.001	20	0.001	1.0240	9.7	0.0195 5	78.77	46.48
15	1.024	1.001	20	0.001	1.023	10.2	0.0115 7	75.49	44.54
30	1.023	1.001	20	0.001	1.022	10.5	0.0083 0	72.21	42.61
60	1.021	1.001	20	0.001	1.020	11.3	0.0060 9	65.64	38.73
250	1.019	1.001	20	0.001	1.018	11.8	0.0030 5	59.08	34.86
1440	1.018	1.001	20	0.001	1.017	12.3	0.0013 0	55.79	32.92

Table D.4.sieve analysis for pt-8

Sieve size(m m)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	292.6	12	3.6	3.6	96.4
0.600	265.6	295.1	29.53	8.9	12.6	87.4
0.425	379.1	398.9	19.83	6.0	18.6	81.4
0.300	265.2	276.8	11.6	3.5	22.1	77.9
0.150	265.5	282.8	17.3	5.2	27.3	72.7
0.075	252.5	268.2	15.7	4.7	32.0	68.0
pan	289.6	514.3	224.7	68.0	100.0	0.0

Table D.4. Hydrometer Analysis pt-8

	G1=1.00	Gs=2.69	Temp. °C =20	K =0.01348					
Elapsed Time (min)	Actual Hyd.Reading	Hyd.Reading taken at suspension	Temp. °C	Composite correction	Corr. Hyd.Reading (R)	L(cm)	D (mm)	P(%)	Adjusted P(%)
2	1.028	1.002	20	0.002	1.026	9.2	0.02891	82.77	56.24
5	1.027	1.002	20	0.002	1.0250	9.4	0.01848	79.59	54.08
15	1.025	1.002	20	0.002	1.023	9.7	0.01084	73.22	49.75
30	1.023	1.002	20	0.002	1.021	10.7	0.00805	66.85	45.43
60	1.022	1.002	20	0.002	1.020	11.5	0.00590	63.67	43.26
250	1.018	1.002	20	0.002	1.016	12.1	0.00297	50.93	34.61
1440	1.017	1.002	20	0.002	1.015	12.6	0.00126	47.75	32.45

Table D.5.sieve analysis for pt-9

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	285.9	5.3	2.0	2.0	98.0
0.600	265.6	269.1	3.53	1.3	3.3	96.7
0.425	379.1	389.0	9.93	3.7	6.9	93.1
0.300	262.4	277.1	14.66	5.4	12.4	87.6
0.150	266.3	267.0	0.66	0.2	12.6	87.4
0.075	253.5	265.3	11.84	4.4	17.0	83.0
Pan	289.6	514.3	224.7	83.0	100.0	0.0

Table D.5.Hydrometer Analysis pt 9

	G1=1.00	Gs=2.81	Temp. °C =20	K =0.013034					
Elaps ed Time (min)	Actual Hyd.Read ing	Hyd.Read ing taken at suspensio n	Temp. °C	Composit e correctio n	Corr. Hyd.Re ading (R)	L(c m)	D (mm)	P(%)	Adjus ted P(%)
2	1.028	1.003	20	0.003	1.025	9.2	0.02795	77.62	64.45
5	1.026	1.003	20	0.003	1.0230	9.4	0.01787	71.41	59.30
15	1.025	1.003	20	0.003	1.022	9.7	0.01048	68.31	56.72
30	1.021	1.003	20	0.003	1.018	10.7	0.00778	55.89	46.40
60	1.020	1.003	20	0.003	1.017	11.5	0.00571	52.78	43.83
250	1.018	1.003	20	0.003	1.015	12.1	0.00287	46.57	38.67
1440	1.015	1.003	20	0.003	1.012	12.6	0.00122	37.26	30.94

Table D.6.sieve analysis for pt-12

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	285.9	5.3	2.1	2.1	97.9
0.600	265.6	269.4	3.83	1.5	3.6	96.4
0.425	379.1	381.2	2.13	0.8	4.5	95.5
0.300	263.5	278.0	14.5	5.7	10.2	89.8
0.150	266.5	266.7	0.2	0.1	10.3	89.7
0.075	255.3	257.0	1.7	0.7	11.0	89.0
pan	289.6	514.3	224.7	89.0	100.0	0.0

Table D.6.Hydrometer Analysis pt12

	G1=1.00	Gs=2.68	Tem p. °c =20	K =0.013524					
Elap sed Time (min)	Actual Hyd.Reading	Hyd.Reading taken at suspension	Tem p. °c	Com posit e corr ectio n	Corr. Hyd.Reading (R)	L(cm)	D (mm)	P(%)	Adjus ted P(%)
2	1.026	1.002	20	0.002	1.024	9.2	0.02901	76.57	68.18
5	1.025	1.002	20	0.002	1.0230	9.4	0.01854	73.38	65.34
15	1.024	1.002	20	0.002	1.022	9.7	0.01088	70.19	62.50
30	1.023	1.002	20	0.002	1.021	10.7	0.00808	67.00	59.66
60	1.020	1.002	20	0.002	1.018	11.5	0.00592	57.43	51.13
250	1.019	1.002	20	0.002	1.017	12.1	0.00298	54.24	48.29
1440	1.016	1.002	20	0.002	1.014	12.6	0.00127	44.67	39.77

TableD.7. sieve analysis for pt-13

Sieve size(mm)	Mass of sieve(g)	Mass of sieve+soil (g)	Mass of soil retained on each sieve(g)	Percentage of mass retained on each sieve	Cum. percent retained	Percent finer
2.36	334.6	334.6	0	0.0	0.0	100.0
2.00	336.4	336.4	0	0.0	0.0	100.0
1.18	280.6	292.6	12	3.6	3.6	96.4
0.600	265.6	295.1	29.53	8.9	12.6	87.4
0.425	379.1	398.9	19.83	6.0	18.6	81.4
0.300	262.4	276.8	14.36	4.3	22.9	77.1
0.150	266.3	282.6	16.26	4.9	27.9	72.1
0.08	253.5	267.0	13.54	4.1	32.0	68.0
pan	289.6	514.3	224.7	68.0	100.0	0.0

Table D .7.Hydrometer Analysis pt-13

	G1=1.00	Gs=2.57	Temp. °C =20	K =0.013991					
Elapsed Time (min)	Actual Hyd. Reading	Hyd. Reading taken at suspension	Temp. °C	Composite correction	Corr. Hyd. Reading (R)	L(cm)	D (mm)	P(%)	Adjusted P(%)
2	1.027	1.002	20	0.002	1.025	9.2	0.03001	81.85	55.69
5	1.025	1.002	20	0.002	1.0230	9.4	0.01918	75.30	51.24
15	1.024	1.002	20	0.002	1.022	9.7	0.01125	72.03	49.01
30	1.021	1.002	20	0.002	1.019	10.7	0.00836	62.20	42.33
60	1.019	1.002	20	0.002	1.017	11.5	0.00613	55.66	37.87
250	1.016	1.002	20	0.002	1.014	12.1	0.00308	45.83	31.19
1440	1.015	1.002	20	0.002	1.013	12.6	0.00131	42.56	28.96

	initial dial reading =0.04mm						
pressre	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
Kpa							
6	-0.04	-0.04	20.04	16.49	4.64507042	-0.01127	4.725554
50	0.14	0.1	19.9	16.35	4.6056338	0.039437	4.674849
100	0.67	0.77	19.23	15.68	4.41690141	0.188732	4.525554
200	0.31	1.08	18.92	15.37	4.32957746	0.087324	4.626962
400	0.29	1.37	18.63	15.08	4.24788732	0.08169	4.632596
800	0.78	2.15	17.85	14.3	4.02816901	0.219718	4.494568

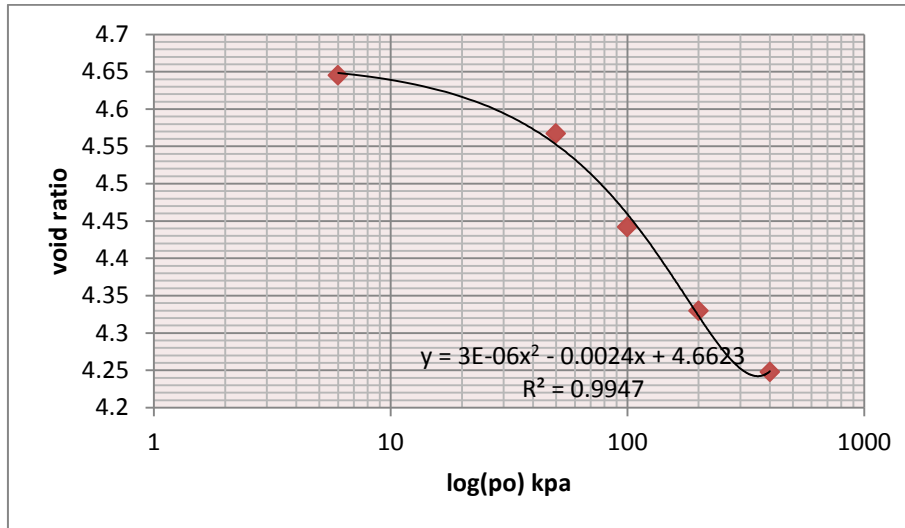


Figure .E.2consolidation dial reading for pt 4

Table E.3.consolidation dial reading for pt 5

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(m m)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.14	0.87	1.13	1.43
0.1	0.01	0.17	0.88	1.15	1.44
0.25	0.018	0.2	0.92	1.16	1.6
0.5	0.019	0.25	0.93	1.17	1.51
1	0.02	0.29	0.94	1.18	1.7
2	0.028	0.37	0.97	1.21	1.71
4	0.031	0.42	0.15	1.22	1.80
8	0.04	0.49	0.19	1.23	1.84
15	0.05	0.63	1.05	1.231	1.89
30	0.076	0.64	1.06	1.24	1.92
60	0.085	0.66	1.08	1.25	2.11
120	0.12	0.74	1.09	1.28	2.12
320	0.135	0.78	1.10	1.31	2.21
480	0.138	0.82	1.11	1.39	2.22
1440	0.14	0.87	1.13	1.43	2.41

	initial dial reading=0.01mm						
Pressure	ΔH	commulative ΔH	$H_f=H_i$ $-\sum \Delta H$	$H_v=H_f$ $-H_s$	$e_f=H_v/H_s$	$\Delta e=\Delta H/H_s$	$e=e_o-\Delta e$
Kpa							
6	0.01	-0.01	20.01	16.51	4.71714286	-0.002857143	4.717143
50	0.13	0.12	19.88	16.38	4.68	0.037142857	4.677143
100	0.73	0.85	19.15	15.65	4.47142857	0.208571429	4.505715
200	0.26	1.11	18.89	15.39	4.39714286	0.074285714	4.64
400	0.3	1.41	18.59	15.09	4.31142857	0.085714286	4.628572

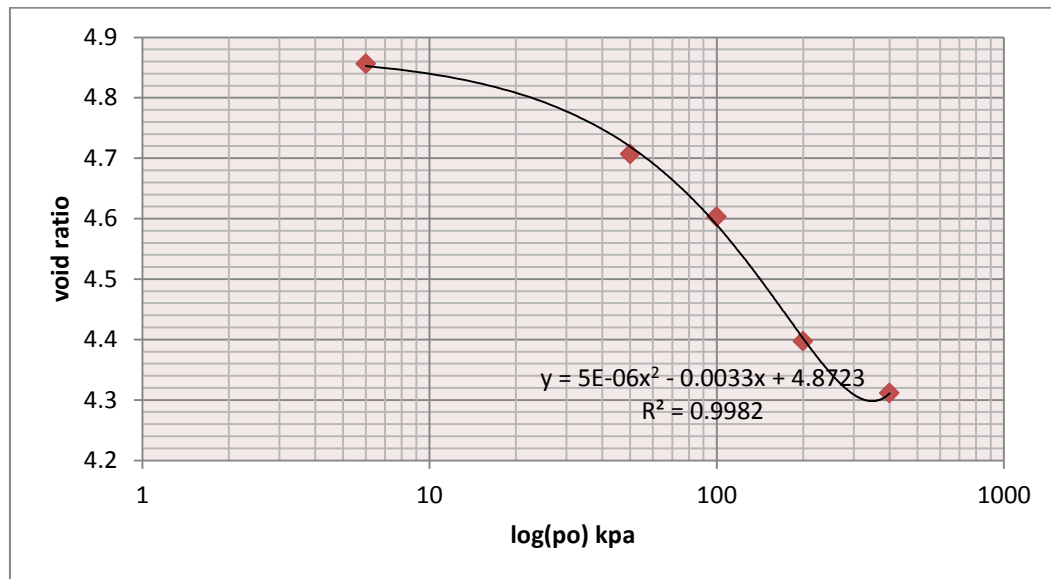


Figure.E.3. consolidation dial reading for pt 5

Table E.4. Consolidations dial reading for pt 8

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.15	0.82	1.13	1.3
0.1	0.01	0.17	0.87	1.14	1.45
0.25	0.02	0.2	0.9	1.15	1.5
0.5	0.02	0.25	0.92	1.16	1.55
1	0.03	0.3	0.95	1.17	1.6
2	0.03	0.36	0.98	1.19	1.73
4	0.04	0.41	0.99	1.2	1.84
8	0.05	0.49	1	1.22	1.89
15	0.06	0.53	1.03	1.23	1.93
30	0.07	0.59	1.04	1.24	1.99
60	0.085	0.66	1.06	1.26	2.1
120	0.1	0.75	1.08	1.29	2.13
320	0.11	0.78	1.09	1.33	2.16
480	0.135	0.79	1.1	1.4	2.2
1440	0.15	0.82	1.13	1.3	2.3

	initial dial reading =0.03 mm		0.03				
pressre kpa	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
6	-0.03	-0.03	20.03	16.49	4.65819	-0.008	4.722761
50	0.14	0.11	19.89	16.35	4.56001	0.0395	4.674738
100	0.67	0.78	19.22	15.68	4.4615	0.1893	4.52502
200	0.31	1.09	18.91	15.37	4.34181	0.0876	4.626715
400	0.17	1.26	18.74	15.2	4.29379	0.048	4.666263
800	1	2.26	17.74	14.2	4.0113	0.2825	4.4318

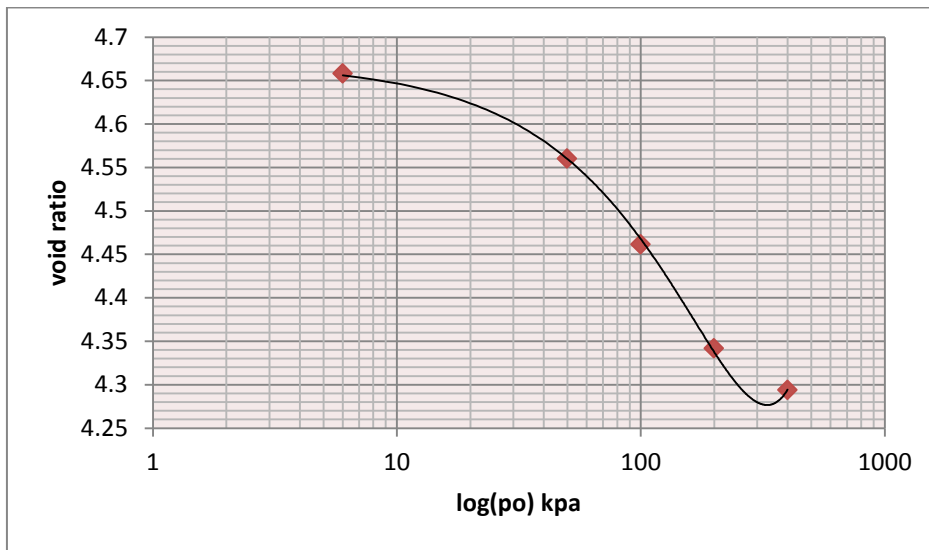


Figure E.4. consolidation dial reading for pt 8

Table E.5.consolidation dial reading for pt 9

time (min)	50kpa dial reading(mm)	100kpa dial reading(mm)	200kpa dial reading(mm)	400kpa dial reading(mm)
0	0.01	0.13	0.49	0.72
0.1	0.015	0.2	0.51	0.75
0.25	0.02	0.25	0.52	0.8
0.5	0.03	0.3	0.54	0.85
1	0.035	0.35	0.56	0.93
2	0.3	0.36	0.58	0.95
4	0.05	0.37	0.59	0.97
8	0.055	0.39	0.6	0.985
15	0.06	0.395	0.61	0.995
30	0.065	0.4	0.615	1.005
60	0.07	0.415	0.62	1.01
120	0.08	0.43	0.635	1.02
320	0.095	0.45	0.64	1.025
480	0.11	0.48	0.66	1.03
1440	0.13	0.49	0.72	1.1

	initial dial reading =0.04mm						
pressre	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
kpa							
6	-0.03	-0.03	20.03	16.63	4.891176471	-0.008823529	3.175491
50	0.115	0.085	19.915	16.515	4.84	0.033823529	3.132843
100	0.36	0.445	19.555	16.155	4.751470588	0.105882353	3.060785
200	0.23	0.675	19.325	15.925	4.683823529	0.067647059	3.09902
400	0.38	1.055	18.945	15.545	4.572058824	0.111764706	3.054902

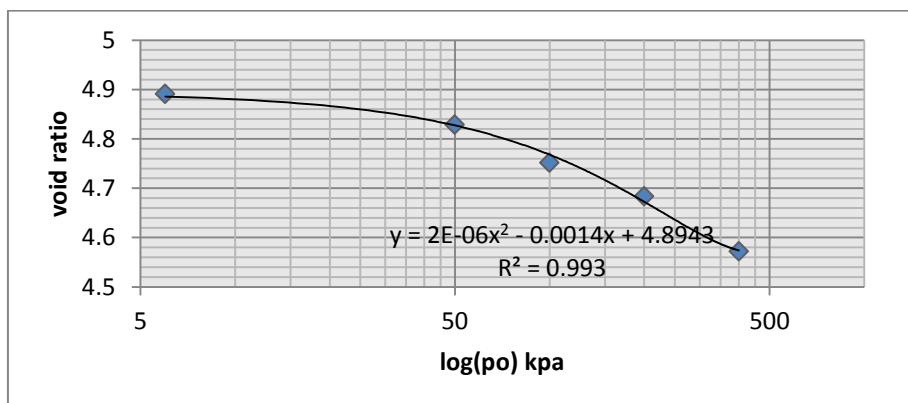


Figure.E.5. consolidations dial reading for pt 9

Table.E.6. consolidations dial reading for pt 12

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.15	0.82	1.13	1.42
0.1	0.01	0.18	0.85	1.14	1.48
0.25	0.021	0.21	0.91	1.15	1.51
0.5	0.022	0.26	0.93	1.16	1.57
1	0.028	0.29	0.94	1.17	1.59
2	0.035	0.31	0.96	1.18	1.72
4	0.038	0.44	1.01	1.11	1.85
8	0.045	0.45	1.04	1.15	1.88
15	0.058	0.51	1.05	1.21	1.10
30	0.064	0.58	1.06	1.26	1.11
60	0.074	0.67	1.07	1.28	2.13
120	0.1	0.71	1.08	1.29	2.15
320	0.11	0.76	1.1	1.31	2.18
480	0.125	0.78	1.2	1.35	2.19
1440	0.15	0.82	1.13	1.42	2.2

	initial dial reading =0.01 mm						
pressre kpa	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
6	-0.01	-0.01	20.01	16.51	4.7171429	-	4.717143
50	0.14	0.13	19.87	16.37	4.6771429	0.04	4.674286
100	0.67	0.8	19.2	15.7	4.4857143	0.191428571	4.522857
200	0.31	1.11	18.89	15.39	4.3971429	0.088571429	4.625715
400	0.29	1.4	18.6	15.1	4.3142857	0.082857143	4.631429

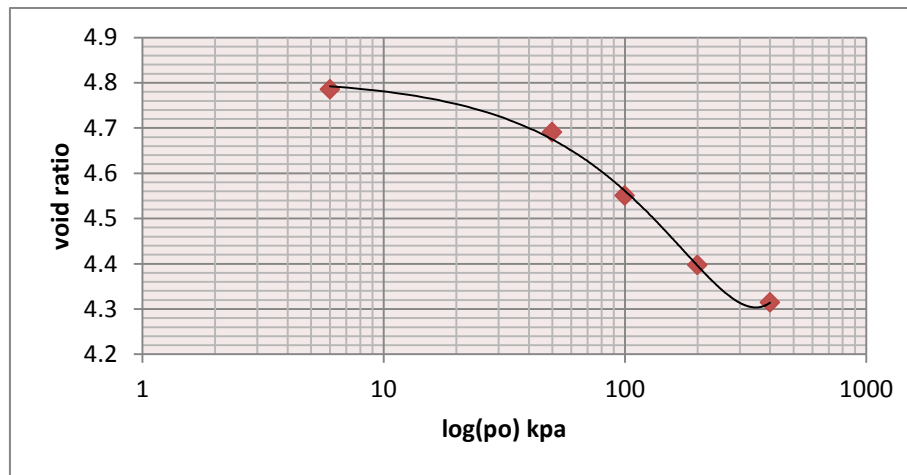


Figure .E.6.consolidation dial reading for pt 12

Table .E.7.consolidation dial reading for pt 13

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.15	0.82	1.13	1.42
0.1	0.01	0.18	0.88	1.13	1.45
0.25	0.025	0.21	0.89	1.14	1.46
0.5	0.028	0.24	0.91	1.15	1.56
1	0.029	0.32	0.95	1.16	1.7
2	0.031	0.29	0.97	1.18	1.72
4	0.036	0.38	0.99	1.21	1.78
8	0.041	0.44	1.01	1.22	1.81
15	0.058	0.48	1.02	1.23	1.91
30	0.064	0.52	1.04	1.25	1.95
60	0.076	0.59	1.05	1.27	2.2
120	0.088	0.63	1.07	1.28	2.14
320	0.099	0.76	1.08	1.31	2.15
480	0.135	0.8	1.1	1.38	2.28
1440	0.15	0.82	1.13	1.42	2.31

	initial dial reading =0.01 mm						
pressre	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
kpa							
6	-0.01	-0.01	20.01	16.57	4.816860465	-	4.717193
50	0.14	0.13	19.87	16.43	4.776162791	0.040697674	4.673588
100	0.67	0.8	19.2	15.76	4.581395349	0.194767442	4.519519
200	0.31	1.11	18.89	15.45	4.49127907	0.090116279	4.62417
400	0.29	1.4	18.6	15.16	4.406976744	0.084302326	4.629984
800	0.89	2.29	17.71	14.27	4.148255814	0.25872093	4.455565

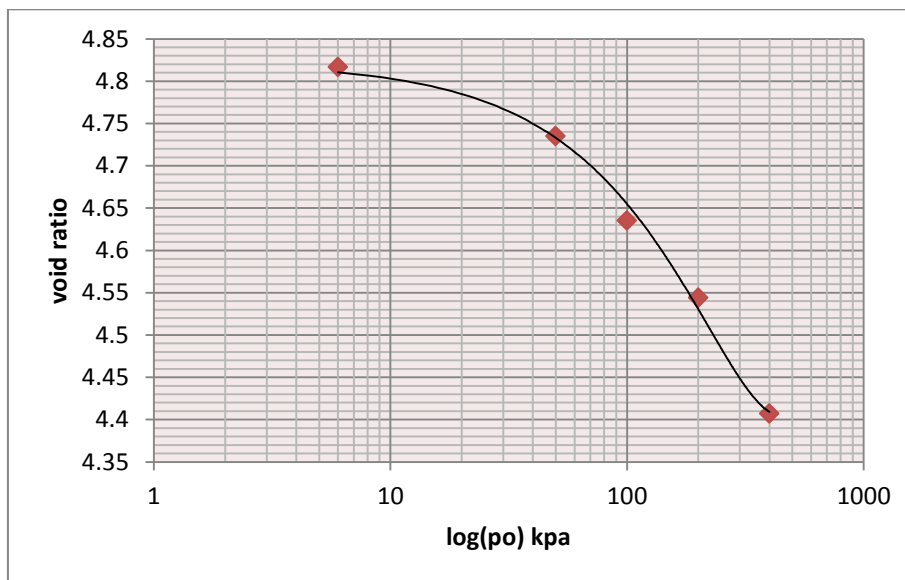


Figure E.7. consolidation dial reading for pt 13

Table .E.8.consolidation dial reading for pt 16

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.14	0.87	1.13	1.43
0.1	0.01	0.17	0.87	1.15	1.44
0.25	0.021	0.2	0.88	1.16	1.54
0.5	0.024	0.25	0.95	1.18	1.56
1	0.036	0.3	0.98	1.19	1.65
2	0.038	0.36	0.99	1.2	1.74
4	0.044	0.41	1.01	1.21	1.81
8	0.055	0.49	1.02	1.23	1.85
15	0.066	0.53	1.04	1.24	1.91
30	0.075	0.59	1.06	1.25	1.92
60	0.086	0.66	1.08	1.26	2.12
120	0.11	0.75	1.09	1.28	2.13
320	0.12	0.78	1.00	1.29	2.15
480	0.119	0.79	1.1	1.35	2.25
1440	0.14	0.87	1.13	1.43	2.31

	initial dial reading =0.02mm						
pressre	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
kpa							
6	0.01	-0.01	20.01	16.36	4.48219	-0.00274	4.717026
50	0.13	0.12	19.88	16.23	4.44658	0.0356164	4.67867
100	0.73	0.85	19.15	15.5	4.24658	0.2	4.514286
200	0.26	1.11	18.89	15.24	4.17534	0.0712329	4.643053
400	0.3	1.41	18.59	14.94	4.09315	0.0821918	4.632094
800	0.29	1.7	18.3	14.65	4.0137	0.0794521	4.634834

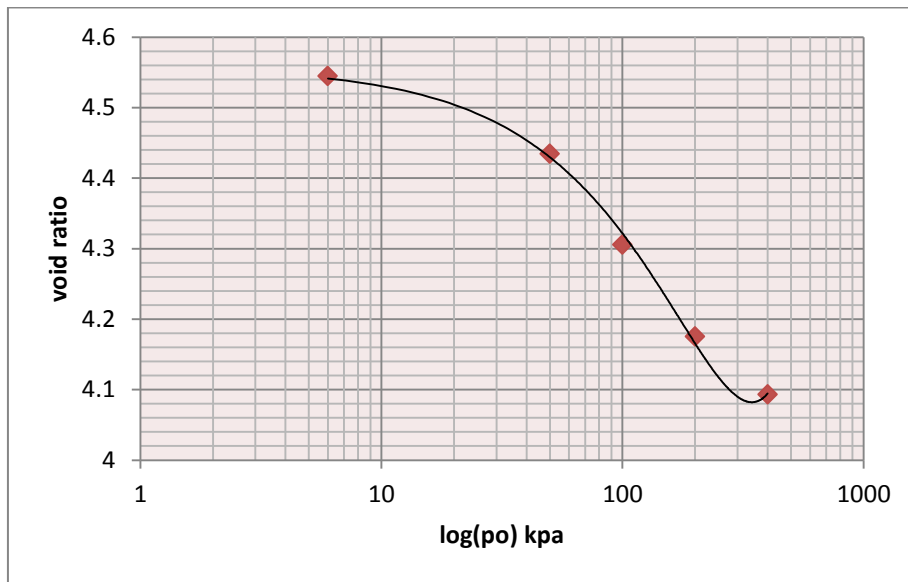


Figure .E.8. consolidation dial reading for pt 16

Table E.9.consolidation dial reading for pt 17

time	50kpa	100kpa	200kpa	400kpa	800kppa
(min)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)	dial reading(mm)
0	0	0.14	0.87	1.13	1.43
0.1	0.01	0.18	0.88	1.13	1.49
0.25	0.022	0.21	0.92	1.14	1.54
0.5	0.024	0.23	0.93	1.17	1.56
1	0.024	0.31	0.95	1.18	1.59
2	0.025	0.35	0.97	1.20	1.65
4	0.034	0.38	1.04	1.21	1.72
8	0.042	0.41	1.05	1.23	1.82
15	0.05	0.45	1.06	1.24	1.91
30	0.08	0.54	1.07	1.26	1.96
60	0.082	0.62	1.08	1.28	2.11
120	0.11	0.72	1.099	1.29	2.14
320	0.13	0.79	1.10	1.32	2.18
480	0.135	0.84	1.12	1.39	2.21
1440	0.14	0.87	1.13	1.43	2.31

	initial dial reading =0.01mm						
pressre kpa	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
6	-0.01	-0.01	20.01	16.35	4.467213115	-0.00273224	4.717018
50	0.13	0.12	19.88	16.22	4.431693989	0.035519126	4.678767
100	0.73	0.85	19.15	15.49	4.232240437	0.199453552	4.514832
200	0.26	1.11	18.89	15.23	4.161202186	0.071038251	4.643248
400	0.3	1.41	18.59	14.93	4.079234973	0.081967213	4.632319
800	0.29	1.7	18.3	14.64	4	0.079234973	4.635051

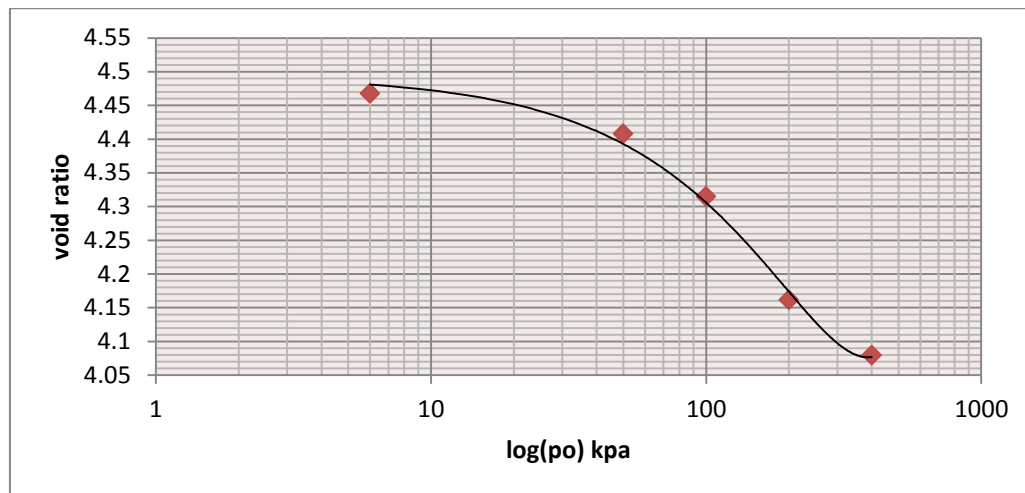


Figure .E.9. consolidation dial reading for pt 1

Table E.10..consolidation dial reading for pt 20

Time (min)	50kpa dial reading(mm)	100kpa dial reading(mm)	200kpa dial reading(mm)	400kpa dial reading(mm)
0	0.03	0.16	0.44	1.26
0.1	0.04	0.17	0.44	1.3
0.25	0.05	0.18	0.5	1.35
0.5	0.06	0.2	0.65	1.4
1	0.07	0.25	0.7	1.48
2	0.08	0.27	0.8	1.5
4	0.095	0.28	0.89	1.67
8	0.1	0.295	0.95	1.82
15	0.105	0.3	1.03	1.93
30	0.11	0.33	1.05	2.03
60	0.115	0.345	1.09	2.1
120	0.12	0.38	1.14	2.15
320	0.125	0.4	1.18	2.17
480	0.14	0.425	1.2	2.21
1440	0.15	0.44	1.26	2.3

	initial dial reading =0.03mm						
pressre	ΔH	commulative ΔH	$H_f = H_i - \sum \Delta H$	$H_v = H_f - H_s$	$e_f = H_v / H_s$	$\Delta e = \Delta H / H_s$	$e = e_o - \Delta e$
Kpa							
6	- 0.0 3	-0.03	20.03	14.21	2.4415807 6	- 0.0051546 4	3.00515 5
50	0.1 1	0.08	19.92	14.1	2.4226804 1	0.0189003 4	-0.0189
100	0.2 8	0.36	19.64	13.82	2.3745704 5	0.0481099 7	-0.04811
200	0.8 2	1.18	18.82	13	2.2336769 8	0.1408934 7	-0.14089
400	1.0 4	2.22	17.78	11.96	2.0549828 2	0.1786941 6	-0.17869

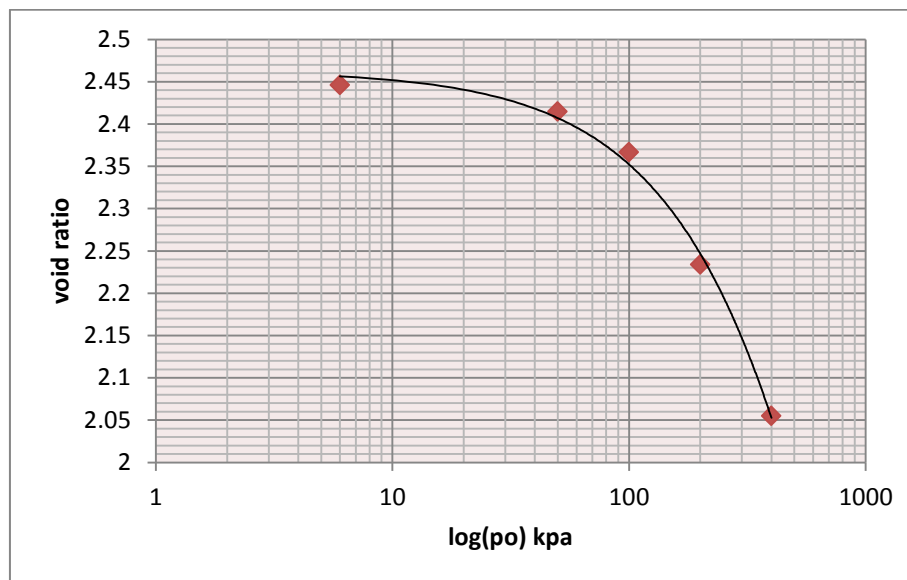


Figure .E.10. consolidation dial reading for pt 20