STABILIZATION OF EXPANSIVE SOIL BY USING COFFEE HUSK ASH

MSc. THESIS

BY

REHMA MELKIE ALEMAYEHU

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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REHMA MELKIE ALEMAYEHU

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DECLARATION

I hereby declare that this thesis is my original work under the supervision of Dr.Ing. Samuel Tadesse, in school of Civil engineering, Hawassa University during the year 2017/18 as part of Master of Science program in Geotechnical engineering. I further declare that this work has not been submitted to any other University or institution for a degree in any other university, and all sources of materials used for this thesis have been duly acknowledged.

Name: Rehma Melkie Alemayehu

Signature: ___________________

Place and Date of Submission: Hawassa University, Hawassa, Ethiopia.

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SCHOOL OF GRADUATE STUDIES

EXAMINERS' APPROVAL SHEET-1

We, the undersigned members of the Board of Examiners of the final open defense by Rehma Melkie Alemayehu have read and evaluated her thesis entitled "**Stabilization of Expansive Soil by using Coffee Husk Ash (CHA)"**, and examined the candidate. This is therefore to certify that the thesis has been accepted in partial fulfillment of the requirement for the degree

Bereket Bezabeh (MSC)

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Name of the Chairperson	Signature	Date

Dr. Ing. Samuel Tadesse

--	--	--
Name of Major Advisor	Signature	Date

M. U. Jagadeesha

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Name of Internal Examiner	Signature	Date

Dr. Ing. Tensay G/medhin

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Name of External Examiner	Signiture	Date

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SGS Approval	Signature	Date

Stamp of SGS

Date:

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(Rehma Melkie)

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Official
ASTM	American Society for Testing and Materials
BS	British Standard
CAH	Calcium Aluminate Hydrate
CEC	Cation Exchange Capacity
CBR	California Bearing Ratio
CSH	Calcium Silicate Hydrate
ERA	Ethiopian Roads Authority
FSI	Free swell index
FSR	Free swell ratio
GSA	Groundnut shell ash
IS	Indian Standard
LL	Liquid Limit
PL	Plastic Limit
MDD	Maximum Dry Density
NRRDS	National Rice Research and Development Strategy
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
CHA	Coffee Husk Ash
RHA	Rise Husk Ash

SP	Swelling Pressure
UCS	Unconfined compressive strength
USA	United States of America
SNNPR	Southern Nations, Nationalities, and Peoples' Region

Units

gm	Gram
kg	Kilogram
km	Kilometer
kN	Kilo Newton
g/cm^3	Gram per centimeter cube
μm	Micrometer
kN/m^2	Kilo Newton per meter square
kPa	Kilo Pascal
mm	Millimeter
meq	Milli equivalent
^oC	Degree Centigrade
cc	Centimeter cube
cm	Centimeter cube
U.S.B.R	United States Bureau of Reclamation

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ABSTRACT

Expansive soils have high potential for swelling and shrinking as a result of change in moisture content and they are one of the major soil deposits in Ethiopia. Road constructed on an expansive soil needs special attention during the design and construction stages. This study investigates the feasibility of using coffee husk ash as an expansive soil stabilizer. Coffee husk is a byproduct of coffee beans: when coffee husk is burnt under controlled condition (muffle furnace) at 550 ^{o}C for 4 hour the resulting ash is coffee husk ash (CHA). To investigate the effect of adding CHA to an expansive soil, laboratory experiments such as atterberg limit tests, free swell index tests, Proctor density, CBR test and unconfined compressive strength (UCS) tests were performed for different percentages of coffee husk ash (CHA). Standard Procter testes were conduct to evaluate the compaction behavior and unconfined compressive tests were performed on samples of 3.6 cm diameter and 7.8 cm height for uncured, 7 and 14 days of curing.

The soil was stabilized with CHA in stepped concentration of 5%, 10%, 15% up to 30% by dry weight of the soil for different laboratory tests. Also the effect of curing period was evaluated over a period of 7 to 14 days for UCS and CBR tests. Analysis of the results shows improvement on the geotechnical properties of CHA stabilized soil. CHA reduces plasticity index by 68% up to 25% of CHA content, swelling ratio by 76% up to 20% CHA content and OMC decreased from 47% to 30% at 15% CHA content with an increase in MDD from 1.22 g/cm3 to 1.384 g/cm3. The UCS is also increased from 35.73kPa to314kPa and the un soaked and soaked CBR values increased from 16.2% to 21.5% and from 1.62 to 5.1% respectively after 14 day of curing and with the addition of 15% CHA content. After the addition of the optimum amount of the CHA the value of the laboratory tests are changed. From this investigation CHA is an alternative expansive soil stabilizing material sa it has changed and for further investigation use another stabilizing agent in addition to CHA to change the soil property from medium plastic to low plastic.

CHAPTER ONE

1 INTRODUCTION

1.1 General Back Ground of the Study

Soils that have characteristics of swelling during wet and shrinking during dry seasons are classified as an expansive soil. The shrink–swell properties of these soils detrimentally influence the construction project design, performance and lifetime, especially on lightweight civil engineering infrastructures [23]. Major engineering problems are volume changes due to cyclic swelling and shrinking behavior of the soils during the wetting and drying seasons respectively. The cyclic swelling and shrinking of the soils can lead to differential heave, settlement and creep, decrease in bearing capacity and shearing strength, and high erosion susceptibility and instability when exposed in natural slopes, road cuts, or open excavations [23]. Expansive soil (mainly black cotton soil) is one of the major soil deposits in Ethiopia, covering about 40% of the area [22]. Black cotton soil is a highly clayey soil. The black color in black cotton soil is due to the presence of titanium oxide in small concentrations and it has a high percentage of clay, which is predominantly montmorillonite in structure and black or blackish grey in color [29].

Most of the roads constructed in Ethiopia on the shrink swell soils are susceptible to damage before expected design lifetime. Early cracking of road surfaces constructed on expansive soils is a common failure of roads in the country. This happens a few years or sometimes even months after being opened for traffic. The high plasticity, compressibility, and swelling and shrinking nature of the soil are usually improved by removing the problematic soil and replacing it with selected material or stabilizing it with lime and cement.

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create a suitable soil material possessing the desired engineering properties. The process may include blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil ([24]; [18]).

Stabilization is used for a variety of engineering works; the most common application is in the construction of roads and air-field pavements [30]. Factors that should be considered in choosing stabilization materials are abundance, cost, and effectiveness. Over the times, cement and lime are the two main materials used for stabilizing soils. The over dependence on the utilization of industrially manufactured soil improving additives (cement, lime etc), have kept the cost of soil stabilization financially high. And also due to the massive growth of infrastructure projects, conventional construction materials are diminishing day by day or found short in supply at various locations in the country. On the other hand, large quantity of agricultural and domestic wastes produced from the different industries like fly ash, bagasse ash, rice husk ash, coffee husk ash etc. creates a potential negative impact on the environment causing air pollution, water pollution affecting the local ecosystem, and hence safe disposal of theses waste materials is required. Utilizing some of these materials as alternative materials for the construction is no doubt a best solution. Several methods of soil improvement using agricultural bi - products, usually coming from the combustion process, such as fly ash, bagasse ash and RHA have been developed and used successfully in practice and emphasis has to be given for maximum utilization of local material so that cost of construction may be minimized to a minimum extent. So the use of certain agricultural and domestic wastes as an expansive soil stabilizer is a best solution.

This paper will also try to assess the effectiveness of using Coffee husk ash (CHA) as an expansive soil stabilizer. Ethiopia produces a large volume of coffee beans every year, around 450,000 tons during 2013/2014alone [4]. Coffee husk is a bi product obtained from milling of coffee. The method of conversion of husk to ash is under controlled way by muffle furnace at 550 ^{o}C for 4 hours.

1.2 Statement of Problem

Expansive soils are one of the most abundant problematic soil in the world. Despite the fact that, billions of dollars in yearly damage losses have been attributed to these problematic soils, the state of practice in design and construction is severely limited by continued lack of understanding of expansive soil behavior and soil-structure interaction.

Over past decades, residential, commercial and infrastructural developments have been increased in most of our country. The construction of these projects have been undertaken on expansive soils due to un understanding of problematic nature of these soils and recently due to lack of suitable land for infrastructures and other developments. In addition to these the project consumes a lot of money as a result of removal and replacement.

This problem urges the need for wider application of cost effective and environmentally friendly technologies of improving soil properties, to be customized and adopted to the current road construction trend in the country.

Agricultural domestic waste of coffee husk ash is the alternative method of stabilizing agent and its locally available material. This stabilizing material were used to improve the compressive strength of locally available black cotton soil, to provide an economical solution and as stabilizing material to check the potential application.

1.3 Objectives of the Study

1.3.1 General Objective

The main objective of this study is to evaluate the technical feasibility of using CHA as expansive soil stabilizing material.

1.3.2 Specific Objectives

The specific objectives of this study are:

- To evaluate the effect of CHA on the properties of the expansive soil with respect to Atterberg limits, free swell, UCS, compaction and CBR.
- To evaluate the engineering properties of untreated and treated specimens.
- To determine optimum amount of stabilizing material (CHA) needed to attain the required Properties of black cotton soils that can be used as sub grade material.

1.4 Methodology of the Study

The research was conducted using literature review on expansive soils, field observations, collection of samples for laboratory testing and analysis of the laboratory test results. Certain laboratory tests were conducted on soil samples before mixing with the selected stabilizing material to check whether the collected sample was expansive or not. A construction site where soil samples for laboratory testing was selected at the Southern part of Addis Ababa around Akaki Kality sub city area. After the samples were confirmed to be expansive soil, full characterization and classification of the soil was made by conducting necessary laboratory tests.

The Coffee Husk used for this research work was obtained from Addis Ababa, Hana Mariam area and kept at a muffle furnace at a degree of 550 for 4 hours. And used to test the effect of Coffee husk ash on the engineering properties of expansive soil in the laboratory. Literature review was undertaken in order to provide a framework and understanding regarding expansive soils and problems associated with such soils. Emphasis was given to review previous works on treatment of expansive soils particularly to chemical stabilization techniques. Literature sources include maps, reports, books, journals and online materials available on internet.

Experiences of other countries with similar physical characteristics to Ethiopia have been referred to build up a picture on the procedures and methods adopted in dealing with expansive soils and agricultural by product stabilization.

The laboratory tests conducted before stabilization on natural soil samples include:

- Grain size analysis(sieve and hydrometer)
- Specific gravity
- Atterberg limits
- Free swell test
- California Bearing Ratio (CBR)
- Unconfined compressive strength (UCS)
- Proctor test
    - ✓ Optimum Moisture Content (OMC)
    - ✓ Maximum Dry Density (MDD)

The engineering properties of the soil samples after stabilization were tested at different percentages of coffee husk ash.

Laboratory tests that were conducted on treated Coffee husk ash-soil mixtures.

- Atterberg limits
- Free swell test
- California Bearing Ratio (CBR)
- Unconfined compressive strength (UCS)
- Proctor test
    - ✓ Optimum Moisture Content (OMC)
    - ✓ Maximum Dry Density (MDD)
- Percent swell of CBR

The test results are evaluated providing prime consideration to the effects of the CHA on engineering properties of the soil. The results of the tests from the natural untreated and treated specimens have been compared. Finally, improvements in the engineering properties of the sample have been analyzed and interpreted. This data together with data obtained from literature and other secondary sources have been compiled, and based on the results obtained, appropriate conclusions and recommendations have been made.

1.5 Significance of the Research

For many years, civil engineers have used additives such as lime, cement and bitumen to improve the qualities of readily available local soils. Laboratory and field strength tests have confirmed that the addition of such additives can increase the strength and stability of such soils. However, the cost of introducing these additives has also increased. Soil stabilization using coffee husk ash is an alternative method for the improvement of expansive soils.

Generally, this research were have the following importance:

- To improve the compressive strength of locally available black cotton soil.
- To provide an economical solution for soil stabilization using CHA.
- To check either CHA increase the physical properties of expansive soils or not.
- To determine the optimum amount of CHA used as stabilizing agent.

1.6 Scope of the Study

The findings of the research are limited to single soil type considered in this research which is expansive black clay located around Addis Ababa Akaki kality sub city area and stabilizing material collected from the stoked CH of Hana Mariam area. The results are also specific to the type of stabilizing material used and test procedures that have been adopted in the experimental work. Therefore, findings should be considered indicative rather than definitive for practical applications.

1.7 Organization of the Thesis

The present research is conducted using literature review on expansive soils and improvement methods, sampling of soil, laboratory testing and analysis on the soil from Addis Ababa around Akaki Kality sub city area. So as to present the results of the research in a more systematic manner, it is divided in to five chapters and the scheme of presentation is as follows.

Chapter one: the first chapter comprises the introduction part, which includes the general background of the study, statement of problem, objectives of the study, methodology, and significance of the research, scope of the study and organization of the thesis.

Chapter two: the second chapter tries to review different literatures that are related to this study, which mainly focus on expansive soils, soil stabilization & different stabilizing materials.

Chapter three: this chapter presents the description of materials and methods, including the description of the study area and sampling methods, materials and also the methods of laboratory analyses which all are used in order to meet the study objective.

Chapter four: this chapter comprises of the result and discussion of this research paper. It presents laboratory test results and discusses the core findings of the entire work by addressing the effects of coffee husk ash on the geotechnical properties of expansive soils one by one.

Chapter five: the final chapter presents conclusion and recommendations. This chapter summarizes the whole findings of research and finally it gives some recommendations based on the research findings and from the researcher point of view.

CHAPTER TWO

2 LITRITURE REVIEW

2.1 Expansive Soils

2.1.1 Genesis of Expansive Soils

Expansive soils are mainly formed from two parent materials. The first group comprises the basic igneous rocks, such as basalts of decant plateau in India, the dolerite sills and dykes in central region of south Africa and gabbros and norites west of pretoria North. In these soils the feldspar and pyroxene minerals of the parent rocks have decomposed to form montmorillonite and other secondary minerals. The second parent materials comprise the sedimentary rocks that contain montmorillonite as constituent which breaks down physically to form expansive soils. In North America, bed rock shale found in the Pierre formation and the more recent Laramie and Denver formation are examples of this type of rock. In Israel there are marls and lime stones. In South Africa the shale of Ecca series.

The montmorillonite was probably formed from two separate origins. The products of weathering and erosion of rocks in the highlands were carried by streams to the coastal plains. The fine grained soils eventually became shale accumulating in the ocean basin. Meanwhile volcanic eruptions, sending up clouds of ash, fell on the plains and seas. Finally, these ashes were altered to montmorillonite [17].

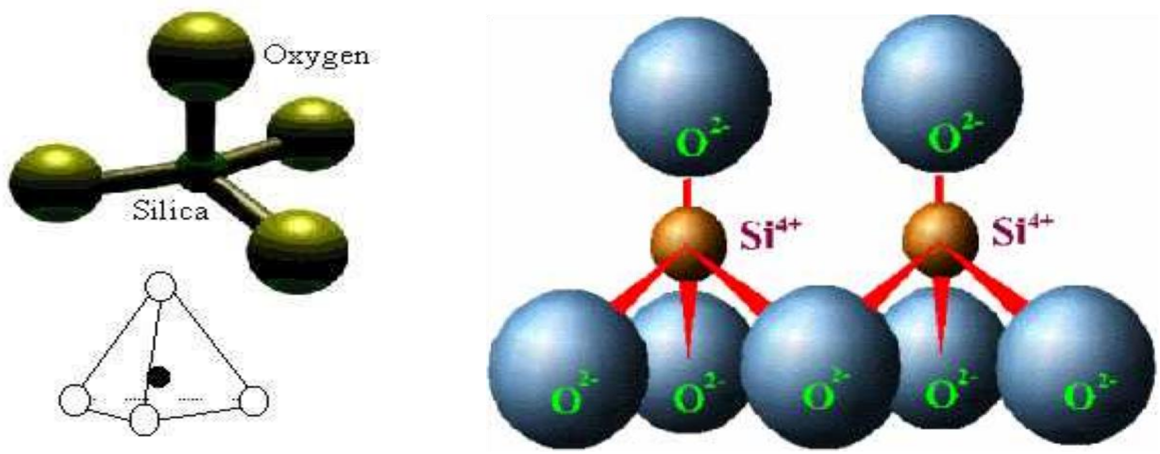
2.1.2 Mineralogy of Expansive Soils

The property of soils strongly depends on the grain sizes of soils. Broadly soils are classified as cohesive and non-cohesive soils (Granular). While the property of granular soils strongly depends on the gravity forces (the amount of mass contained in the soils) the behavior of clay soils strongly depends on surface tension forces, since clays have big specific surface as compared with their mass. The behavior of fine – grained soils depends to a large extent on the nature and characteristics of the minerals present.

Expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less. However, according to [17] the grain size alone does not determine clay minerals and he emphasized that the most important property of fine grained soils is their mineralogical composition.

Clay minerals are crystalline hydrous alumino-silicates derived from parent rock by weathering. The basic building blocks of clay minerals are the silica tetrahedron and the alumina octahedron and combined into tetrahedral and octahedral sheets to form the various types of clays [17]; [38]. Kaolinite, illite and montimorillonite (smectite) are the common groups of clay minerals most important in engineering studies [17].

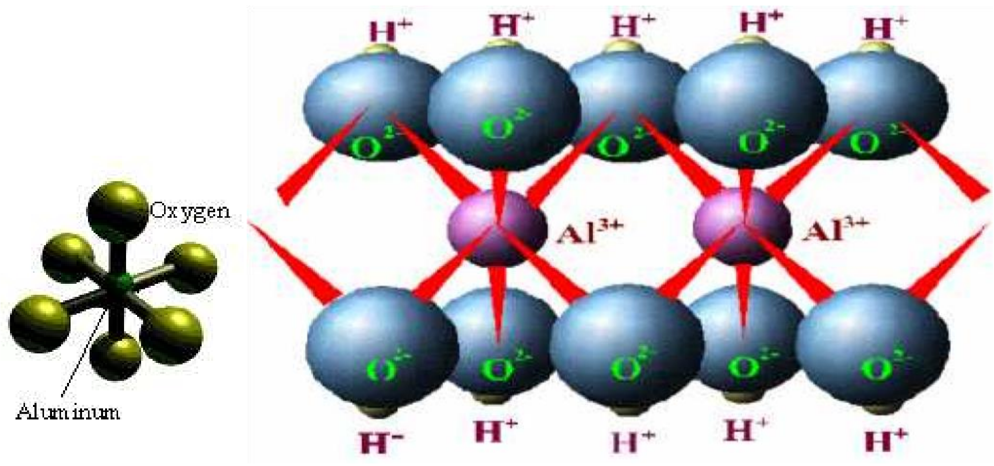
Clay soils consist of clay minerals. These minerals are crystalline material comprising of oxygen and silica. The structural unit of the silicates is a tetrahedron with a silica cation; positively charged, surrounded by four oxygen atoms, negatively charge (Fig 2.1). The silicates assemble themselves in to sheets.



a. Single Tetrahedron b. Silica Tetrahedral Sheet

Figure 2-1 The structural units of silicates

The silicate can contain other structural units like aluminum. By combining aluminum minerals, aluminum sheets can also be formed. They take the form of an aluminum ion surrounded by oxygen of hydroxyl in the formation of the octahedron: Figure 2.2



a. Single Octahedron b. Alumina Octahedral Sheet

Figure 2-2 Single Aluminum Octahedrons and Aluminum Sheets

The behavior of the soil mass depends on micro scale factors**.** These are:

- The amount and type of clay minerals in the soil
- The chemical structure
- The specific area of the clay particles
- The soil water chemistry contained within the voids.

The macro scale factors; plasticity and density are dictated by these micro scale factors.

The three important structural groups of clay minerals for engineering purposes are:

Kaolinite

Kaolinite is a typical two-layer mineral having a tetrahedral and an octahedral sheet joined to form a 1-1 layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water [38]. However, [23] in their study indicated the swelling property of kaolinite. Nelson (2010) described that kaolinite is formed by weathering or hydrothermal alteration of rocks rich in feldspar in low PH condition to favor the leaching out of ions like sodium, potassium, calcium, magnesium and iron. The bond that exists between layers in tight and hence it is difficult to separate the layers. As a result, kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently, kaolinite shows little swelling on wetting.

Illite

Illite has a basic structure similar to that of montmorillonite Fig. However, the basic illite units are bonded together by potassium ions which are non-exchangeable fig.2.3 because of this, the illite units are reasonably stable and so that mineral swells much less than montmorrillonite. From the above it follows that montmorillonite is the most active clay mineral that imparts swelling characteristics to clays.

Montomorillonite

Montomorillonite is the most common of all the clay minerals and is well known for its swelling properties. Its basic structure consists of an alumina sheet sandwiched between two silica sheets and symbolically represented as shown in fig 2.3 The basic montomorillonite units are stacked one on top of the other, but the bond between the individual units is relatively weak so that water is easily able to penetrate between the sheets and cause their separation and hence swelling.

Montmorillonite is extremely active and its activity decreases as the adsorbed cation exchanges in the following order:

Sodium (most active), lithium, potassium, calcium, magnesium and hydroxyl (most stable). As the result of exchange of adsorbed cation of montmorillonite by those listed above, its activity reduced. Making use of this fact, lime, cement and gypsum are used to stabilize montmorillonite soils (expansive soils) in the process of which the active sodium ions are replaced by less active calcium ions.

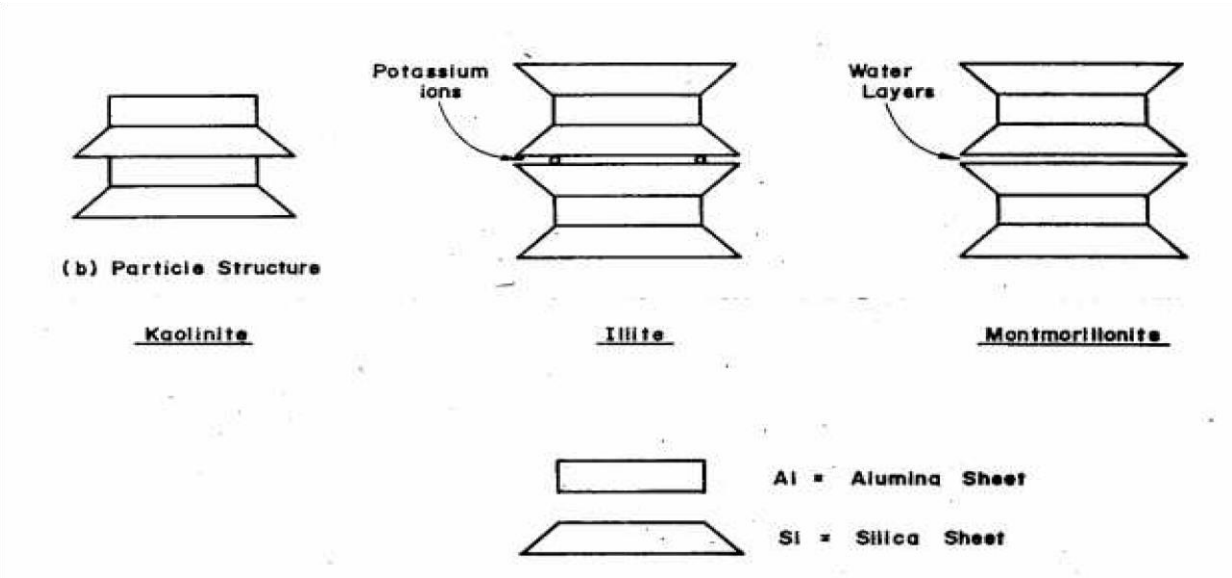


Figure 2-3 The Symbolic Structure of Clay Minerals (Teferra, and Leikun, 1999).

2.1.3 Distribution of Expansive Soil

Expansive soils are widespread in African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world case of expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc. [7].

The areal coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres [33]; as cited by [40]. They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis Gohatsion, and Addis-Modjo are covered by expansive soils. Also areas like Mekele and Gambella are covered by expansive soil.

2.1.4 Identification of expansive soils

The techniques of identification and classification of expansive soils generally consists of two important phases. The first is the visual identification and recognition of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss different ways that are commonly used to identify expansive soils.

2.1.4.1 Field Identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include [17]; [41]:

- Usually have a color of black or grey.
- Wide or deep shrinkage cracks.
- High dry strength and low wet strength.
- Stickiness and low traffic ability when wet.
- Cut surfaces have a shiny appearance.
- Appearance of cracks in nearby structures.

2.1.4.2 Laboratory Identification

Laboratory identification and classification of expansive soils can be categorized into mineralogical, indirect and direct methods.

2.1.4.2.1 Mineralogical Identification

Clay mineralogy is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques, the more common of which are five [17].

- X-ray diffraction
- Differential thermal analysis
- Dye adsorption
- Chemical analysis
- Electron microscope resolution

But these methods are not suitable for routine tests because of the following reason;

- They are time consuming;
- They require expensive test equipment; and
- The results can only have interpreted by specially trained technicians.

i. X – Ray diffraction

The most popular method, works on the principle that the beams of X- ray diffracted from crystals are similar to light reflections from the crystal larrice planes. X – Ray analysis is well known suited for identification of clay minerals because of wavelength of X – rays is of the same order of magnitude (about 1 A or 10-9 mm) as the atomic plane spacing of these minute crystals. The basal plane spacing is characteristic for each clay mineral group and gives the most intense reflection [41].

ii. Differential Thermal Analysis (DTA)

Differential thermal analysis is the other popular mineralogical method. DTA consists of simultaneously heating a sample of clay and an inert substance. The resulting thermographs, which are plots of the temperature difference (WT) verses applied heat, are compared to those for pure minerals. Each mineral shows characteristics endothermic and exothermic reactions on the thermo grams [41].

iii. Dye Absorption

Mineral can be identified by characteristic colors formed by dyes that are absorbed by the minerals of the soil sample. When a clay sample is pretreated with acid, the color assumed by the absorbed dye depends on the Base Exchange capacity of the various clay minerals present. The presence of the montmorillonite can be identified if the selected sample contains the mineral which is greater than about 5-10% [17].

iv. Chemical analysis

This method is a valuable addition to other methods such as X-ray Diffraction. In the montmorillonite group of clay minerals, the chemical analysis can be used to determine the nature of isomorphism and to show the origin and location of the charge on the lattice [17].

v. Electron microscope Revolution

Electron microscopes have provided a means of direct observing the clay particles. Only qualitative identification is possible based in size and shape of the particles using microscopy [41].

2.1.4.2.2 Indirect Methods

The second method is called indirect methods which comprises of the index properties, potential volume change method and activity methods. This method is a valuable tool in evaluating the swelling potential of the soils. It is advisable not to use the indirect tests directly, instead direct tests are also important to avoid an error in conclusions. These methods are related to laboratory soil identification and are vital for the intended purposes [17].

i. Atterberg's Limits

Atterberg's limits define the moisture content boundaries between states of consistency of fine grained soils. This shows a clay soil can exist in four distinct state of consistency depending on its water content. The water content at the boundaries between the different states are defined as the shrinkage, plastic and liquid limits.

Two useful indices may be computed from the atterberg's limit and the natural moisture content. These are the Plasticity Index (PI) and Liquidity Index (LI). The PI used extensively for classifying expansive soils and should always be determined during preliminary investigation [41].

Table 2-1 Relation between the Swelling Potential of Clays and the Plasticity Index (Chen, 1988)

Swelling potential	**Plasticity index**
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

While it may be true that high swelling soil will manifest high index property, the converse is not true [17].

The plasticity characteristics and volume change behavior of soils are directly related to the amount of colloidal sized particles in the soil. For engineering purposes, the term colloid is used to describe a particle whose particle is controlled by surface force (i.e. electrostatic and adsorptive forces) rather than by gravitational forces. Colloid size is generally defined as being smaller than 0.001mm [41].

Atterberg limits and clay content can be combined into a single parameter called Activity. This term was defined by Skempton (1953). The activity is defined as follows

$$\text{Activity } (A_c) = \frac{Plasticity\ index(PI)}{\%\ by\ weight\ finer\ than\ 2\ \mu m} \quad (2.1)$$

Skempton suggested three classes of clays according to activity as inactive, for activities less than 0.75; normal, for activities between 0.75 and 1.25 and active for activities greater than 1.25. Active clays provide the most potential for expansion [41].

Table 2-2 Typical values of activities for various clay minerals (Nelson and Miller, 1992)

Mineral	**Activity**
Kaolinite	0.33 to 0.46
Illite	0.9
Montmorillite (Ca)	1.5
Montmorillite (Na)	7.2

ii. Free Swell Test

Free Swell Index is the increase in volume of a soil, without any external constraints, on submergence in water. This test is carried out by taking two representative oven dried soil samples each of 10 grams passing through 425-micron sieve and Pour each soil sample in to each of the two glass graduated cylinders of 100ml capacity. Fill one cylinder with kerosene and the other with the distilled water up to the 100ml mark and remove the entrapped air in the cylinder by gentle shaking and stirring with a glass rod. Allow the samples to settle in both the cylinders. Sufficient time, not less than 24 hours shall be allowed for soil sample to attain equilibrium state of volume without any further change in the volume of the soils then record the final volume of the soils in each of the cylinders.

$$\text{Free swell } (\%) = \frac{vd-vk}{vk} * 100 \quad (2.2)$$

Where:

Vd = Volume of the soil specimen read from the graduated cylinder containing distilled water.

Vk = Volume of the soil specimen read from the graduated cylinder containing kerosene.

iii.Swell Potential

Swell potential is expressed as the percentage of swell of laterally confined sample which has been soaked under a surcharge of 1 psi. Seed, Woodward and Lundgreen, established the following relationship for sample compacted following AASHTO compaction test to maximum density and optimum moisture content and allowed to swell in consolidometer under a surcharge of 1 psi [7].

$$S=60\ K\ (Ip)^{2.44} \qquad (2.3)$$

Where;

S= Swell potential

K=3.6x10-5 (and is a constant)

Table 2-3 Relationship between Swelling Potential of Clays and Plasticity Index (Teferra, and Leikun, 1999)

Swelling Potential	Plasticity Index
Low	0-15
Medium	10-35
High	20-55
Very High	55 and above

iv. Cation Exchange Capacity (CEC)

The CEC is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milli-equivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.4.

Table 2-4 Typical CEC Values of Basic Clay Minerals after Mitchell, 1976(Nelson And Miller, 1992)

Clay Mineral	**CEC(meq/100gm)**
Kaolinite	3 – 15
Illite	10 – 40
Montmorillonite	80 – 150

2.1.4.2.3 Direct Methods

The third and the last methods are called the direct measurements. These methods are the most useful data for practicing Engineers. These methods offer the most useful data by direct measurement; and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions. The direct measurements are the most satisfactory and convenient methods to determine the swelling potential and swelling pressure of expansive clay [17].

Direct measurements of expansive soils can be achieved by the use of the conventional one dimensional consolidometer. The consolidometer can be platform type, Scale type or other arrangement. The soil sample is enclosed between two porous plates and confined in a metal lying. The soil sample can be flooded both from the bottom and from the top [17].

2.1.5 Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils are based on indirect and direct prediction of swell potential as well as combinations to arrive at a rating.

There are a number of classification systems. The following are some of the common methods.

2.1.5.1 Classification Using General Methods

Soils are classified in general schemes; Unified soil classification system (USCS) and the American Association of State Highway and Transportation Officials Method according to index properties.

i. Unified Soil Classification Systems

This classification is based on plasticity chart and a correlation is made between swell potential and unified soil classification as follows.

Category	**Soil classification in Unified system**
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderate expansion	GW, SC, ML, MH
High volume change	CL OL, CH, OH
Peat	PT

The above classification system can be summarized as follow:

a. All sands and gravels exhibit little or no expansion.
b. All clayey gravels and sands and all silts exhibit moderate volume changes.
c. All clay soil and organic soils exhibit high volume change.

In the above classification soils rated as CL or OH may be considered as potentially expansive [41].

i. AASHTO Classification

As shown on Table 2.5 soils rated A-6 or A-7 by AASHTO can be considered potentially expansive [41].

Table 2-5 AASHTO Soil Classification Chart

General classification	Granular Materials(35% or less of total sample passing No. 200)							Silt –clay materials (more than 35% of total sample passing No.200)			
Group classification	A1		A3	A2				A4	A5	A6	A7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No 10	50 max										
No 40	30 max	50 max	51 min								
No 200	15 max	25 max	10 max	35max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fracture passing No 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		NP.	10 max	10 max	11 min	11 max	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

2.1.5.2 Classification Specific to Expansive Soil

A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas [41]. Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information. An indirect prediction of swell potential includes correlations based on index properties, swell and a combination of them.

Some of such classification systems are:

i. U.S.B.R Method

This method was developed by Holtz and Gibbs; to establish degree of expansion based on simultaneous consideration of shrinkage limit, plastic index, percent smaller than 0.001mm (1μ) free swell and percent swell under a pressure of 1 psi. [7].

Table 2-6 Classification Based On Bureau of Reclamation Method [17]; [45].

Colloid content, (%)	**Plasticity index, (%)**	**Shrinkage limit, (%)**	**Probable expansion, (%)**	**Degree of expansion**
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

ii. Activity Method (Seed, Classification Method)

[7] Classify clayey soil according to its swelling potential defined as the percent vertical swell under a pressure of 1 Psi of a laterally confined sample compacted following AASHTO compaction method to maximum density and optimum moisture content. Seed et al established the following expression. [7].

$$S=KC^x \tag{2.4}$$

Where

S = Swelling potential in percent

C= Percentage of colloids smaller than 2μ

X= a constant depending on the clay type and equal to 3.44 for clays tested

K=A factor depending on the type of clay minerals, calculated from the intercept of straight line relating log S to log C.

iii. Skempton's Method

Skempton classified **c**lays according to activities. This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity. Activity is defined as:

$$\text{Activity (Ac)} = \frac{plasticity\ index(PI)}{\%by\ weight\ finner\ than\ 2\mu m} \qquad (2.5)$$

Where

Ac= Activity

PI= plasticity index

Skempton suggested three classes of clays according to their activity shown in Table 2.7.

Table 2-7 Degree of Colloidal Activity (Teferra And Leikun, 1999).

Degree of Activity	**Activity**
Inactive clay	Ac < 0.75
Normal clay	0.75 < Ac < 1.25
Active clay	Ac > 1.25

Following this classification, montmorillinitic clay (expansive clay) is defined as active, illitic clay as normal and kaolinitic clay as inactive.

2.2 SOIL STABILIZATION

2.2.1 General

Soil stabilization is the process of improving the engineering properties of the soil thus making it more suitable. It is required when the soil available for construction is not suitable for the intended purpose.

In general, soil stabilization is the process of creating or improving certain desired properties in a soil material so as to make it useful for a specific purpose. Soil stabilization may be broadly defined as the alteration or preservation of one or more soil properties to improve the engineering characteristics and performance of the soil. When the mechanical stability of a soil cannot be obtained by combining materials, it may be advisable to stabilize the soil by adding lime, cement, bituminous materials or special additives. The methods of stabilization include physical processes such as soil densification, blends with granular material, use of reinforcements (Geogrids), undercutting and replacement, and chemical processes such as mixing with cement, fly ash, lime, lime by-products, and blends of any one of these materials. Soil properties such as strength, workability, swelling potential, and volume change tendencies may be altered by various soil modification or stabilization methods. Different types of materials are used to stabilize the soils depending upon the cost, type of structure to be constructed and also the climatic conditions. Materials such as lime, cement, fly ash, copper slag, rice husk ash, disposal of solid wastes, geosynthetics etc., are used to stabilize the soil [9].

2.2.2 Types of Soil Stabilization

[7] Broadly classified soil stabilization into two: Mechanical stabilization and Chemical Stabilization.

2.2.2.1 Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation [24];[35].

2.2.2.2 Chemical Stabilization

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen.

Chemical stabilizers can be broadly divided in to three groups:

- **Traditional stabilizers** such as hydrated lime, Portland cement and Fly ash;

- **Non-traditional stabilizers** comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and
- **By-product stabilizers** which include cement kiln dust, lime kiln dust etc.

Among these, the most widely used chemical additives are lime, Portland cement and fly ash.

Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable. The mechanisms of stabilization of the traditional stabilizers are detailed below [9].

Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation-agglomeration and pozzolanic reactions.

a) Cation Exchange

Clay minerals have the property of absorbing certain anions and cations and retaining them in an exchangeable state. The exchangeable ions are held around the outside of the silica – alumina clay – mineral structural unit and the exchange reaction doesn't affect the structure of the silica – alumina pocket. In clay minerals, the most common exchangeable cations are Ca^{2+}, Mg^{2+} ,H^+, NH_4^+, Na^+, frequently in about that order of general relative abundance. [15].

The existence of such charges is indicated by the ability of clay to absorb ions from the solution. Cations (positive ions) are more readily absorbed than anions (negative ions); hence, negative charges must be predominant on the clay surface. A cation, such as Na^+, is readily attracted from a salt solution and attached to a clay surface. However, the absorbed Na^+ ion is not permanently attached; it can be replaced by K^+ ions if the clay is placed in a solution of potassium chloride KCl. The process of replacement by excess cation is called cation exchange. Some are more strongly attracted than others, and the cations can be arranged in a series in terms of their affinity for attraction as follows:

$Al^{3+} > Ca^{2+} > Mg^{2+} > NH_4^+ > K^+ > H^+ > Na^+ > Li^+$

This series indicates that, for example, Al^{3+} ions can replace Ca^{2+} ions, and Ca^{2+} ions can replace Na^{+} ions. The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. The process is called cation exchange. [15].

For example

Na clay +$CaCl_2$ → Ca Clay + NaCl

b) Flocculation and Agglomeration

Flocculated structure occurs in clays. The clay particles have large surface area and, therefore, the electrical forces are important in such soils. The clay particles have a negative charge on the surface and a positive charge on the edges. Inter particle contact develops between the positively charged edges and the negatively charged faces. This results in a flocculated structure.

Flocculent structure is formed when there is a net attractive force between particles. When clay particles settle in water, deposits formed have a flocculated structure. The degree of flocculation of a clay deposit depends upon the type and concentration of clay particles, and the presence of salts in water. Clays settling out in a salt water solution have a more flocculent structure than clays settling out in a fresh water solution. Salt water acts as an electrolyte and reduces the repulsive forces between the particles.

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Due to change in texture, a significant reduction in the swelling of the soil occurs.

In general, the soils in a flocculated structure have a low compressibility, a high permeability and high shear strength.

c) Pozzolanic Reactions

The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calcium aluminate hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects. The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix [32].

Pozzolanic constituents produces calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). Rate of the pozzolanic reactions depends on time and temperature.

$$Ca^{2+} + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} = CSH \tag{2.6}$$

$$Ca^{2+} + 2(OH)^{-} + Al_2O_3 \text{ (Clay Alumina)} = CAH \tag{2.7}$$

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength.

A. Traditional Stabilizers

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers, lime probably is the most routinely used.

Lime Stabilization

Lime is prepared by decomposing limestone at elevated temperatures. Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementitious compounds [50], [21]. The two primary types of lime used in construction today are quick lime (calcium oxide) and hydrated lime (calcium hydroxide) Slurry lime also can be used in dry soils conditions where water may be required to achieve effective compaction [26]. Heating lime stone at elevated temperatures produces quicklime and the addition of water to quicklime produces hydrated lime. Equation 2.6 and 2.7 show the reaction from which quick lime and hydrated lime are produced [28].

$$CaCO_3 + \text{Heat} \rightarrow CaO + CO_2 \text{ Equation} \tag{2.8}$$

$$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat Equation} \tag{2.9}$$

Quicklime is the most commonly used lime; the followings are the advantages of quicklime over hydrated lime [46].

- Higher available free lime content per unit mass
- Denser than hydrated lime (less storage space is required) and less dust
- Generates heat which accelerate strength gain and large reduction in moisture content according to the reaction equation below

$$CaO + H_2O \rightarrow Ca(OH)\ 2 + Heat\ (65\ KJ/mole)$$

Hydrated lime in the form of lime is used in the majority of lime stabilization work. Quick lime represents approximately 10 percent of the lime used in the lime stabilization process.

Lime-soil reactions are complex and primarily involve a two-step process. When lime is added to clayey soils, two pozzolanic chemical reactions occur, Cation exchange and flocculation – agglomeration. In the cation exchange and flocculation – agglomeration reactions, the monovalent cations generally associated with clays are replaced by the divalent calcium ions. The cations can be arranged in a series based on their affinity for exchange:

$$Al^{3+} > Ca^{2+} > Mg^{2+} > NH_4^+ > K^+ > H^+ > Na^+ > Li^+$$

Any cation can replace the ions to its right. For example, calcium ions can replace potassium and sodium ions from clay. Flocculation – agglomeration produces a change in the texture of clay soils. The altered clay structure, as a result of flocculation of clay particles due to cation exchange and short-term pozzolanic reactions, results in larger particle agglomerates and more friable and workable soils [32].

The clay particles tend to clump together to form large particles, thereby (a) decreasing the liquid limit, (b) increasing the plastic limit, (c) decreasing the plasticity index, (d) increasing the shrinkage limit, (e) increasing the workability, and (f) improving the strength and deformation properties of a soil [15].

Pozzolanic reaction between soil and lime involves a reaction between lime and the silica and alumina of the soil to form cementing material. One such reaction is

$$Ca\ (OH)_2 + \underset{\substack{\uparrow \\ \text{Clay silica}}}{SiO_2} \rightarrow CSH$$

Where

C= CaO

S= SiO_2

H=H_2O

Although pozzolanic reaction processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions, cation exchange and flocculation/agglomeration. Extent of this strength gain may vary with soils depending on differences in their mineralogical composition. Therefore, mellowing periods, normally about one-day in length but ranging up to about 4-days, can be prescribed to maximize the effect of short term reactions in reducing plasticity, increasing workability, and providing some initial strength improvement prior to compaction. The second step, a longer-term pozzolanic based cementing process among flocculates and agglomerates of particles, results in strength increase which can be considerable depending on the amount of pozzolanic product that develops, and this, in turn depends on the reactivity of the soil minerals with the lime or other additives used in stabilization.

Cement Stabilization

Cement is the oldest binding agent since the invention of soil stabilization technology in 1960's. It may be considered as primary stabilizing agent or hydraulic binder because it can be used alone to bring about the stabilizing action required [50]; [21]. Cement reaction is not dependent on soil minerals, and the key role is its reaction with water that may be available in any soil [21]. This can be the reason why cement is used to stabilize a wide range of soils. Numerous types of cement are available in the market; these are ordinary Portland cement, blast furnace cement, sulfate resistant cement and high alumina cement. Usually the choice of cement depends on type of soil to be treated and desired final strength.

Hydration reaction is the primary mode of strength gain in soil cement. Free lime, Ca(OH)2, produced during the hydration process can comprise up to about 25 percent of the cement and water mix on a weight basis. This free lime can produce pozzolanic reaction between the lime and soil, which can continue as long as the pH is high enough to solubilize the soil minerals [11]. The process starts when cement is mixed with water and other components for a desired application resulting into hardening phenomena. The hardening (setting) of cement will enclose soil as glue, but it will not change the structure of soil [21]. The hydration reaction is slow proceeding from the surface of the cement grains and the center of the grains may remain un hydrated [50]. Cement hydration is a complex process with a complex series of unknown chemical reactions [34]. However, this process can be affected by

- presence of foreign matters or impurities
- water-cement ratio
- curing temperature
- presence of additives
- Specific surface of the mixture.

Depending on factor(s) involved, the ultimate effect on setting and gain in strength of cement stabilized soil may vary. Therefore, this should be taken into account during mix design in order to achieve the desired strength. Calcium silicates, *C3S* and *C2S* are the two main cementations properties of ordinary Portland cement responsible for strength development [44]; [21]. Calcium hydroxide is another hydration product of Portland cement that further reacts with pozzolanic materials available in stabilized soil to produce further cementitious material [50]. Normally the amount of cement used is small but sufficient to improve the engineering properties of the soil and further improved cation exchange of clay. Cement stabilized soils have the following improved properties:

- Decreased cohesiveness (Plasticity)
- Decreased volume expansion or compressibility
- Increased strength (PCA-IS 411, 2003).

B. By-product Stabilizers

Like traditional stabilizers, pozzolanic reactions and cation exchange are the primary stabilization mechanisms for many of the by-product stabilizers. Lime kiln dust (LKD) and cement kiln dust (CKD) are by-products of the production of lime and Portland cement, respectively.

Lime kiln dust (LKD) normally contains between about 30 to 40 percent lime. The lime may be free lime or combined with pozzolans in the kiln. The source of these pozzolans is most likely the fuel used to provide the energy source. LKDs may be somewhat pozzolanically reactive because of the presence of pozzolans or they may be altogether non-reactive due to the absence of pozzolans or the low quality of the pozzolans contained in the LKD. Cement kiln dust (CKD) is the byproduct of the production of Portland cement. The fines captured in the exhaust gases of the production of Portland cement are more likely (than LKD) to contain reactive pozzolans and therefore, to support some level of pozzolanic reactivity. CKD generally contains between about 30 and 40 percent CaO and about 20 to 25 percent pozzolanic material. [32] The purpose of this document is not to establish specific guidelines regarding composition of byproduct LKD or by-product CKD as the oxide composition of each can vary widely depending on the composition of the feed stock, the nature of the fuel, the burning efficiency, and the mechanism and efficiency of flue dust capture. For example, if coal is used, then ash produced as a by-product of burning coal could be captured in the bag house or other mechanism used to capture exhaust fines with the by-product lime. If the source of the LKD is from the production of dolomitic lime, then magnesium oxide may form a significant part of the LKD. Magnesium oxide, MgO, takes longer and is more difficult to fully hydrate than CaO, and upon hydration it expands. If the LKD contains more than about 5 percent MgO then care should be taken to insure full hydration of the MgO if this LKD is used for modification or stabilization [32].

Again, it is incumbent upon the agency involved to determine acceptable levels of oxides and trace elements that comprise the by-product.

As a general guide on the level of risk associated with the presence of oxides and trace elements in these by product stabilizers, the development of expansive mineral products may become intolerable when the SO_3 content exceeds about 3 percent or when the MgO content exceeds about 3 to 5 percent. The impact of organics can also be a problem as their presence can interfere with the availability of calcium to the soil or aggregate being treated. Several tests can be used to screen for the presence of organics. One quick test if loss on ignition (LOI). Although it does not identify the type of organic, which is definitely important, an LOI of greater than about 8 to 10 percent flags a potentially problematic quantity of organics.

C. Non Traditional Stabilizers

This standard practice is limited to traditional, chemical stabilizers like: Portland cement, lime and fly ash. However, it is important when considering treatment with these traditional products to broach the subject of non-traditional or alternative stabilizers.

The mechanism of stabilization for non-traditional stabilizers varies greatly among the stabilizers.

Asphalt may or may not be grouped as a traditional stabilizer depending on perspective. Asphalt is not a “chemical” stabilizer in the sense that it does not react chemically with the soil to produce a product that alters surface chemistry of the soil particles or that binds particles together. Instead asphalt waterproofs aggregate and soil particles by coating them and developing an adhesive bond among the particles and the asphalt binder. The process is dependent on the surface energies of the aggregate or soil and the asphalt binder. Consequently, since this mechanism is more physical than chemical, soils with very high surface areas are not amenable to asphalt stabilization and such stabilization is normally limited to granular materials such as gravels or sands, and perhaps some silty sands. As a visco-elastic, visco-plastic material, temperature and/or dilution methods are required to make asphalt stabilization effective in soils. Either lower viscosity liquid asphalts (normally developed by mixing bitumen with diluents) or emulsified asphalts are used in soil stabilization. Because the nature of asphalt stabilization is so mechanistically different from chemical stabilization, asphalt stabilization is not considered as a candidate in this standard practice [32].

The mechanisms of stabilization of other non-traditional stabilizers including sulfonated oils, enzymes, ionic stabilizers, etc. are discussed in detail by Petry and Little. Such stabilizers may have a role in modification and/or stabilization, especially when high soluble sulfate contents in the soil limits the applicability of calcium-based, traditional stabilizers [32]

2.2.3 Stabilization of Expansive Soil Using Agricultural Byproduct

Agricultural by-products are the outputs generated from the processing of various agricultural products. An attempt has been made by researchers to use agricultural bi – products as soil stabilizers. Some research works conducted on stabilization of expansive soil using agricultural byproducts, approve agricultural byproducts usually coming from the combustion process, such as fly ash and RHA, have pozzolanic properties. Pozzolanas are siliceous and aluminous materials, which in itself possess little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (ASTM 595). The other agricultural byproduct is the coffee husk ash.

This paper will also try to assess the feasibility of using Coffee husk ash (CHA) as an expansive soil stabilizer.

2.2.3.1 Coffee Husk Ash

Coffee generates large amount of coffee by-products/residues during processing. Depending upon the method of coffee cherries processing, different residues are obtained. Coffee husks are the major solid residues from the handling and processing of coffee, since for every 2Kg of coffee beans produced, approximately 1kg of husks are generated [37].

In Ethiopia 192000metric tons of coffee is Husk cast adrift as byproduct per year. Coffee grounds are highly pollutant due to the presence of organic material that demands a great quantity of oxygen in order to degrade. Burning of coffee husk generates about 18-23% of its weight as ash. Muffle furnace is used to convert the husk to ash at a degree of 550 ^{o}C for 4 hours.

2.2.3.2 Coffee Husk Ash as soil stabilizer

On reviewing of past studies and researchers a considerable amount of research concerning Stabilization of soil with additives such as cement, lime, lime- fly ash, salt and rice husk ash are available in the literature. But soil stabilization with coffee husk ash (CHA) is new method. One research work which is conducted on expansive soil stabilization using coffee husk ash confirm the suitability of this material for soil stabilization.

Coffee husk was heated in a temperature controlled furnace (muffle furnace) known amount of husk was heated in a crucible for 4 hours at a temperature of 550^{o}C and the ash obtained. The amount of ash decreased as ashing become more and more complete. CHA depend upon, whether the husk have undergone complete destructive combustion or have been partially burnt. The ash obtained at lower temperatures was blackish in color, indicating incomplete combustion due to unburned carbon. At higher temperatures more and more of carbon is oxidized to carbon di oxide. As the temperature rises beyond 700°C reduction in mass is due to disintegration of solid compounds, alkali metal salts, into gaseous components [14]. Different factors influencing ash properties are incineration condition (temperature and duration), rate of heating, geographic location, fineness, color and crop variety and year of crop production.

From the chemical analysis test the chemical composition of the coffee husk ash used in this study was obtained as shown in table 2.8. (As analyzed by Geological Survey of Ethiopia, Central Laboratory)

Table 2-8 Chemical composition of CHA

Chemical properties	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H2O	LOI
Test value	3.14	<0.01	0.16	0.14	0.32	<0.01	0.96	<0.01	0.031	<0.01	8.0	88.73

Color changes are associated with the completeness of combustion process as well as structural transformations of silica in the ash. Ash of white color is an indication of complete oxidation of the carbon, which is also an indication of availability of large portion of amorphous silica in the ash [19].

2.2.3.3 Availability of Coffee Husk in Ethiopia

Coffee is one of the world's most popular beverages and important products. It has grown steadily in commercial importance over the last 150 years. Globally, 25 million small producers rely on coffee for their living. Brazil, Vietnam and Colombia account for more than half of the world's production. The global coffee production per year on average accounts to approximately 7 million metric tons [43]. According to the International Coffee Organization (ICO), output of coffee brew in the 2011-12 seasons is estimated at 130 million bags (ICO, output of coffee brew in the 2011-12, www.ico.org.). The top five countries that produces coffee in 2015/16: Brazil, Vietnam, Colombia, Indonesia and Ethiopia are produces; 2592000, 1650000, 810000, 660000, 384000metric tons respectively. Ethiopia is the fifth country of coffee producer in the world and is considered to be the original habitat of Arabica coffee and Central Africa is reckoned to be the native of robust a coffee. Today, an estimated 525,000 hectares (5,250 km2) of coffee are planted in Ethiopia [51], although the actual area is probably in excess of 20,000 km2. The main coffee growing areas are found within Oromia Region and Southern Nations, Nationalities, and Peoples' Region (SNNPR), with modest production in Amhara Region and minor output in Benishangul-Gumuz Region. Coffee provides Ethiopia with its most important agricultural commodity, contributing around one quarter of its total export earnings [36]. Coffee generates large amount of coffee by-products/residues during processing. Depending upon the method of coffee cherries processing, different residues are obtained. Coffee husks are the major solid residues from the handling and processing of coffee, since for every 2Kg of coffee beans produced, approximately 1kg of husks are generated [37] and [2]. In Ethiopia 192000metric tons of coffee is Husk cast adrift as byproduct per year. Coffee grounds are highly pollutant due to the presence of organic material that demands a great quantity of oxygen in order to degrade. Proposed alternative uses for coffee husks include employing this solid residue as a supplement for animal feed, direct use as fuel, and fermentation for the production of a diversity of products (enzymes, citric acid and flavoring substances), use as a substrate for growth of mushrooms and use as adsorbents [43].

However, considering the high amounts generated, there is still a need to find other alternative uses for this solid residue. By converting the coffee husk in to coffee husk ash for the purpose of soil stabilizing agent is the other alternative.

2.2.4 Previous Studies

2.2.4.1 Cementations Stabilization

Musema, R., (2014) studied stabilization of expansive soil using lime in a case of Adura Burbey DS6 Road Segment. The experimental study involved Atterberg limit, moisture density relation and CBR tests. The conclusions and findings drawn from the study are;

- The addition of lime to the tested samples led to the reduction in liquid limit, plasticity index and an increase in plastic limit of the soil sample at a specific location.

- The plastic limit of the blended samples has not shown a clear trend as it increases from 0 to 2% and starts decreasing from 2% to 12%.
- Both samples have shown a reduction in plasticity index with the addition of lime. And the plasticity index of both samples has improved from its high swelling potentials to low swelling potentials with the addition of 12% lime.
- The reduction in plasticity limit is an indication of a marked increase in workability which in turn expedites manipulation and placement of the treated soil. Workability is improved because flocculation makes the clay more friable; this assists combination for effective mixing and compaction.
- The addition of lime has resulted in an increase of the shrinkage limit of the soil.
- The addition of lime has resulted in a reduction of the free swell of the soil. As the percentage of stabilizer increased, free swell ratio decreased.
- The addition of lime has a positive effect in increasing the specific gravity of the soil samples. The addition of lime for the studied soil has resulted in an increase in optimum moisture content and reduction of maximum dry density for the same compaction effort.
- The optimum lime content in improving the CBR of the soil from its poor subgrade quality too poor to fair class is found out as 12 percent.
- The addition of lime has resulted in an increase of the unconfined compressive strength of the tested samples.
- The addition of optimum lime content has resulted in an improvement of the overall performance of the subgrade by increasing its strength and other workability criteria‘s for the project.
- By addition of lime, the swelling percentage decreased considerably. The reduction was higher for lime added samples having more lime content.
- The addition of lime up to 12% does not bring a significant improvement in California bearing ratio which falls in the range of 7-20 percent. But the achievement in Improving the subgrade quality is cost effective because this will reduce the use of borrow materials on the project.

Wubeshet, M., (2013) studied the stabilizing effect of Bagasse ash on the engineering Properties of potentially expansive subgrade soils on samples collected from Addis Ababa. The experimental study involved Atterberg limit, moisture-density relation and CBR tests. The conclusions and findings drawn from the study are;

- The plasticity index slightly reduced with increased in bagasse ash content and curing has also an insignificant effect on the plasticity of the expansive soil. The optimum moisture content increased while the maximum dry density values decreased with increment of bagasse ash content.
- Free swell, free swell index and free swell ratio of the stabilized samples decreased with increasing bagasse ash content.
- CBR values slightly increased with the addition of bagasse ash. The change in CBR value is not significant for both cured and uncured samples. Addition of bagasse ash alone does not improve the strength of soils due to presence of only reactive silica with low amount of calcium content in bagasse ash.
- The plasticity index significantly decreased with addition of bagasse ash combined with lime and increased curing period. However, the addition of bagasse ash alone has a minor effect on the plasticity index of expansive soil.
- The addition of lime and bagasse ash together led to a more decrease of the maximum dry density and increase in optimum moisture content compared to the addition of lime and bagasse ash separately.
- The addition of bagasse ash in combination with lime improved the CBR value. The improvement is more significant when the sample is cured. Hence, combination of bagasse ash and lime can strongly improve the strength of the expansive soil.
- Unlike lime in combination with bagasse ash the improvement achieved by bagasse ash alone on the poor geotechnical properties of expansive soil was limited because lower amount of calcium in the bagasse ash. Hence, improvements achieved with up to 70 to 30% bagasse ash content were not satisfactory. However, the rate of swelling and heave decreased with increasing bagasse ash content of stabilized expansive soil. In this investigation bagasse ash stabilized expansive soil does not bring significant change for use it as a sub-grade material. Therefore, bagasse ash is not an effective

standalone stabilizer for highly plastic expansive soils. However, bagasse ash plus/in combination with lime can effectively stabilize this soils. The expansive soil stabilized with bagasse ash plus/in combination with lime can be used as a good subgrade material. So, combining two local materials (bagasse ash and lime) can effectively improve the poor geotechnical properties of this soils and help in increasing land resources availability for construction projects and reduce the amount of lime needed for the stabilization purpose.

Nigussie, E., (2011) studied stabilization of highly plastic clay collected from Addis Ababa using sodium silicate and its combination with lime. The experimental study involved Atterberg limit, moisture-density relation and CBR tests. The conclusions and findings drawn from the study are;

- 6% lime yielded significant improvement on plasticity and strength properties of expansive soils.
- No significant improvement in the engineering properties of the soil was attained by addition of sodium silicate.
- The addition of sodium silicate changes clayey sand sample of PI 16 % in to non-plastic and resulted in a minimum of 11.96 % reduction in PI of expansive soil PI 92 % which led to the belief that sodium silicate decreases plasticity of soils.
- Carbonation has detrimental reversing effects on plasticity index of soils attained by stabilization.
- Sodium silicate is not a suitable additive for montmorillonitic clay (expansive soil) stabilization. Mixing sodium silicate with lime is not a viable option for montmorillonitic clay (expansive soil) stabilization.
- Applying 1% sodium silicate by dry weight of the soil in concert with ordinary Portland cement has positive implications on strength development of soil cement.
- Sodium silicate has inhibiting effect on strength development of soil-cement when its quantity is increased to and beyond 2.5% by dry weight of the soil. Proper curing is mandatory for strength development of cementitious stabilizers. Curing also enhances strength of soils treated with sodium silicate.

- Varying the mode of mixing does not bring a change in dry density of expansive clay treated with sodium silicate; however, sodium silicate should not be directly applied to cementitious stabilizers as it forms hydration products before the soil can be engaged in the reaction.

Argu, Y., (2008) studied stabilization of light grey and red clay subgrade soil collected from Addis Ababa using SA-44/LS-40 chemical and lime. The experimental study involved Atterberg limit, moisture-density relation, swelling pressure and CBR tests. The conclusions and findings drawn from the study are;

- 8% lime yielded significant improvement on plasticity, swelling and strength properties of expansive soils.
- The applications of SA-44/LS-40 chemical alone are ineffective in improving the soaked CBR value of the red clay and light grey soils.
- The application of 0.30lit/m3 of SA-44/LS-40 chemical and 2% lime is an optimum proportion in increasing the soaked CBR value and reducing the swelling pressure of the light grey clay soil. The application of 0.08lit/m3 of SA-44/LS-40 chemical and 4% lime is an optimum proportion in increasing the soaked CBR value of the red clay soil.

Nebro, D., (2002) evaluated lime and liquid stabilizer called Con-Aid for stabilization of potentially expansive subgrade soil on samples collected form Addis-Jimma road which had indicated different pavement damages exacerbated by the presence of expansive soils. The experimental study involved Atterberg limit test, moisture-density relation, UCS, CBR and CBR swell. The findings and conclusions of the study can be summarized as follows:

- Addition of lime reduced maximum dry density and increased the optimum moisture content.
- 4% of lime by dry weight of the soil was optimum lime content to stabilize the soil even though increased quantity of lime led to increased strength.
- Addition of lime reduces the swelling potential but no significant improvement in the engineering properties of the soil was attained by addition of Con-Aid.

Tesfaye, A., (2001) studied improvement of expansive soil by addition of lime and cement on black cotton soil from different parts of Addis Ababa. Index properties, compaction characteristics and swelling pressure of soil-cement and soil-lime were determined using Atterberg limit test, moisture-density relations, free swell and swelling pressure tests. The conclusions and findings drawn from the study are;

- Expansive soil becomes moderately active to inactive based on the amount of lime and cement added.
- Swelling pressure of expansive soil decreases with increasing lime, cement and molding water content. 4-6% of lime and 9-12% of cement yielded significant improvement on plasticity and swelling properties of expansive soils.

2.2.4.2 RHA Stabilization

Haji Ali et al. (1992) found that the addition of rice husk ash (RHA) enhances not only the strength development but also the durability of lime-stabilized residual soil. Stabilized soil with the optimum RHA content suffers the least detrimental effects of saturation. Therefore, it can be inferred that the use of RHA in the chemical treatment of residual soil for construction of roads, airfields, etc. would require reduced annual maintenance costs [Haji et al., 1992].

A.S. Muntohar(2009) made a laboratory study on stabilization of Three types of soils namely residual soil, kaolinite soil and bentonite using RHA and cement. Experimental studies include evaluation of index properties of soil and compaction, along with characterization of materials by X-ray diffraction. The test results showed that both cement and RHA reduced the plasticity of soils. A decrease in MDD and an increase in OMC was also observed.

Musa Alhassan (2008) reported the use of rice husk ash for stabilization of lateritic soil of Maikunkela area of Minna, Nigeria. Performance of soil-RHA was investigated w.r.t. compaction characteristics, CBR and UCS. Test result indicated a general decrease in MDD and increase in OMC with increase in RHA content. In addition to this, improvement in CBR and UCS were observed under the application of RHA & max. UCS recorded at 6-8% RHA.

Fidelis Okafor et al (2009) studied Effect of RHA on geotechnical properties of soil classified as A-2-6 or SW for sub grade purpose. The investigation included evaluation of properties such as compaction, consistency limits and strength of soil with RHA content of 5%, 7.5%, 10% and 12.5% by weight of dry soil. The results obtained showed that the increase in RHA increased the OMC but decreased the MDD. Increase in RHA content reduces plasticity and increased volume stability as well as strength of soil. 10% RHA content was observed to be the optimum content.

Hossain Anwar, K. (2011) evaluated the effectiveness of Cement kiln dust, rice husk ash, and their combinations in different percentages (maximum up to 20%) as expansive clay soil stabilizer. Correlations between strength, modulus of elasticity and CBR were established. It is reported that, developed stabilized soil mixtures have shown satisfactory strength and durability characteristics and can be used for low-cost construction to build houses and road infrastructures.

Edeh J. E. et al (2012) made Laboratory evaluation of the characteristics of rice hush ash (RHA) stabilized reclaimed asphalt pavements (RAP) subjected to British Standard light; BSL (standard Proctor) compactive effort to determine the compaction characteristics and CBR values was carried out. Test results showed that the properties of RAP improved when treated with RHA, using up to 2% cement additive. The particle size grading improved from 100% coarse aggregates for 100% RAP to 10 - 90% coarse aggregate with 10 - 90% fines for the various RAP + RHA mixtures containing up to 2% cement. The CBR values also increased from 8 and 14% for the un soaked and soaked conditions, respectively, for 100% RAP content to 73 and 79% (soaked condition) for 89.25% RAP in the RAP/RHA mix proportions with 1.5% cement/89% RAP content in the RAP/RHA mix proportions with 2% cement content, with corresponding un soaked CBR values of 28 and 26%, respectively. Generally, soaked samples recorded higher CBR values than un soaked samples. The RHA stabilized RAP mix proportions with 89.25% RAP/1.5% cement content, and 89% RAP/2% cement content with CBR values of 73 and 79% (soaked for 24 hours) can be used as sub base or subgrade materials in road construction.

Ramakrishna and Pradeep Kumar (2006) had studied combined effects of RHA and cement on engineering properties of black cotton soil. From strength characteristics point of view, they had recommended 8% cement and 10% RHA as optimum dose for stabilization.

Sharma et al. (2008) had investigated the behavior of expansive clay stabilized with lime, calcium chloride and RHA. The optimum percentage of lime and calcium chloride was found to be 4% and 1% respectively in stabilization of expansive soil without addition of RHA. From UCS and CBR point of view, when the soil was mixed with lime or calcium chloride, RHA content of 12% was found to be the optimum. In expansive soil – RHA mixes, 4% lime and 1% calcium chloride were also found to be optimum.

Rao et al. (2011) had studied the effects of RHA, lime and gypsum on engineering properties of expansive soil and found that UCS increased by 548% at 28 days of curing and CBR increased by 1350% at 14 days curing at RHA- 20%, lime -5% and gypsum 3%.

Sabat (2012) had studied the effects of polypropylene fiber on engineering properties of RHA-lime stabilized expansive soil. Polypropylene fiber added were 0.5 to 2% at an increment of 0.5%. The properties determined were compaction, UCS, soaked CBR, hydraulic conductivity and Ps. The effect of 0,7 and 28 days of curing ware also studied on UCS, soaked CBR, hydraulic conductivity and Ps. The optimum proportion of soil: RHA: lime: fiber was found to be 84.5:10:4:1.5.

Sabat (2013) had studied the effect of lime sludge (from paper manufacturing industry) on compaction, CBR, shear strength parameters, coefficient of compression, Ps and durability of an expansive soil stabilized with optimum percentage of RHA after 7days of curing. The optimum proportion soil: RHA: lime sludge was found to be 75:10:15.

Ashango and Patra (2014) had studied the static and cyclic properties of clay subgrade stabilized with RHA and Portland slag cement. The optimum percentage of RHA was found to be 10% and Portland slag cement as 7.5% for stabilization of expansive soil. They concluded that the stabilized expansive soil was found suitable for subgrade of flexible pavement as, there was significant increase in strength and the stabilized soil was durable.

Nitsihit T. (2015) had studied Stabilization of expansive soil using rice husk ash. She concluded that RHA stabilized expansive soil does not bring significant change for using it as a sub-grade material. Therefore, RHA is not an effective standalone stabilizer for highly plastic expansive soils because of lower calcium amount in the ash. However, the UCS and the un-soaked CBR values increased appreciably with increasing RHA (10%) content. RHA may be used in admixture stabilization.

2.2.5 CHA Stabilization

On reviewing past studies and researches stabilization of expansive soils with CHA is a new method and couldn't find literatures efficiently.

Atahu et al. (2017) had studied the effect of Coffee husk ash (CHA) on geotechnical properties of expansive soils. Commonly used laboratory tests such as Atterberg limits, free swell index, standard Proctor tests, and unconfined compressive strength tests were performed to evaluate the behavior of expansive soil treated with coffee husk ash. The tested soil is clays of high plasticity, with high swell–shrink capacity and low permeability. The obtained result reveals that the addition of CHA reduces the plasticity of the soil and increases both the maximum dry density and the unconfined compressive strength.

It is also found that the addition of higher percentages of CHA results in a decrease of both the unconfined compressive strength and maximum dry density of the soil. The swell and shrinkage tests indicated that the swelling and shrinking capacity of the soil stabilized with the addition of 25% CHA reduced by more than half compared to the untreated soil.

CHAPTER THREE

3 MATERIALS AND METHODS

3.1 Study Area and Sampling

3.1.1 Location

The study area is located in Akaki kality sub city of Addis Ababa, Ethiopia. The latitude of Akaki kality, Addis Ababa, Ethiopia is 8.895831, and the longitude is 38.789162.and it is located at Ethiopia country in the district place category with the GPS coordinates of 8°53'44.9916"N and 38°47'20.9832"E with the elevation of 2140 meters.

3.1.2 Climate

The climate of the study area is semi-arid. The highest rainfall is mostly in August. During March and April, the area receive rainfall from Gulf of Aden and Indian Ocean and from Atlantic Equatorial during the main rainy seasons.

3.1.3 Soil Property of the Study Area

Expansive soils are common in Akaki Kality sub city of Addis Ababa, and are frequently encountered during highway and building construction. To minimize the damage of roadways from swelling action, it is necessary to recognize these types of soils and when encountered in the field the boundaries of the soils along the project have to be determined during preliminary soil survey [3].

In most areas of Addis Ababa black cotton soils are usually found, either as thin layer, with the darker color and are oriented parallel to the bedding. The bands range in thickness from, few centimeters to as much as ten meters from the existing ground surface. Residual soils are commonly seen in most parts of the area with varying thickness (0.5 to 16m). On the other hand, due to intensive erosional activities, there is poor soil development on most parts of the slopes. Black cotton soil is the dominant type of soils in the area, where erosion past it by deposition. The variation in the characteristics of soils resulted in variations of infiltration and water holding capacity. Generally, the mature basic and acidic rocks are weathered to form thick soil profiles. In places where young basalts and welded tuffs occur, the thickness of the soil cover is reduced [6].

3.2 Materials

3.2.1 Expansive Soil

The soil sample used for this study is a black clay soil obtained from a single test pit in a sub city of Akaki kality area, Addis Ababa. A disturbed soil sample was collected from the single test pit at a depth below 1.0 m in order to avoid the inclusion of organic matter.

3.2.2 Coffee Husk Ash

Coffee husk is a by-product of coffee. Ethiopia produces coffee abundantly; its husk requires a large area for storage and it is environmentally unfriendly. The main focus of this study is the utilization of waste (coffee husk) for road construction to improve the properties of expansive soil and to increase economic and environmental benefits. The coffee husk (CH) used in this study was collected from Hana Mariam area on the way from Akaki to Haile garment Addis Ababa and kept in muffle furnace at 550 ^{0}C for 4 hours to convert into coffee husk ash.

Figure 3-1 A coffee husk

3.2.2.1 CHA Preparation

The collected Coffee husk was burnt under controlled condition or by muffle furnace kept at a degree of 550 ^{o}C for 4 hours to obtain the ash. After complete burning, the ash is allowed to cool for another 24 hours, the burnt ash was taken out for grinding then after the coffee husk ash was sieved through ASTM sieve size #40(0.425 mm) and the fractions passing through the sieve were used throughout the tests. The sieved ash was immediately stored in air tight containers to avoid pre-hydration during storage or when left in open air. Figure 3.2 shows the products obtained after the process, i.e. coffee husk ash (CHA).

Figure 3-2 CHA, after burning (a) and, after grinding and sieving (b)

3.2.3 Water

According to the standard test, it is recommended to use distilled water to conduct certain laboratory tests like hydrometer test, atterberg limits, free swell index and specific gravity tests.

3.3 Methods

3.3.1 Preparation of Testing Samples

The collected soil samples were prepared for test by drying, pulverizing (for expansive clay) and screened through the sieve of 4.75 mm aperture before preparing the specimens for testing. To get uniform mix ratio of the natural soil and CHA for each test the collected soils and ash contents were oven-dried at $105^{0}C$ overnight to remove moisture. Then the oven dried samples were mixed thoroughly by hand in a large tray in a dry state.

3.3.2 Laboratory Testing and Analysis of Samples

The testing program conducted on the clayey soil samples included determination of the workability, strength and compaction characteristics of soils at their natural state. Tests for soil classification which included grain size distribution, free swell, specific gravity, and Atterberg's limit. These are indicative tests that are usually used for identifying whether the soil is expansive or not.

The conducted tests however included hydrometer analysis, Atterberg limits, wet sieve analysis, specific gravity, moisture density relation (compaction), free swell, unconfined compressive strength, CBR and percent swell of CBR to fully characterize and attain the objective of the research. On the other hand, the testing program conducted on the clayey soil samples mixed with different percentages of coffee husk ash, included Atterberg's Limit, free swell, compaction, unconfined compressive strength test, CBR and percent swell of CBR tests.

3.3.2.1 Grain Size Analysis test

Sieve analysis was carried out to determine the grain size distribution of soil and used in the classification. Wet sieve analysis was employed to determine the grain size distribution of the soil samples in accordance with AASHTO T-88 Test Method for particle size analysis of soils. In addition, hydrometer analysis was conducted to know the silt and clay fractions.

3.3.2.2 Atterberg's Limit

The nature and response of soil upon change to moisture content is determined by Atterberg's limit tests. This parameter is basic in the AASHTO and Unified soil classification systems. The Atterberg's limits depends on the type of predominant clay mineral available in the soil mass. If the predominant clay is montmorillonite the liquid limit can reach or even exceed 100%. It is also expected that the Atterberg's limit is less for illite dominated soil and even lesser for kaolinite dominated soils. In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. The test includes the determination of the liquid limits, plastic limits and the plasticity index for the natural soil and the soil-CHA mixtures in accordance with ASSHTO T 89-96 and T90-96 testing procedures.

3.3.2.3 Specific Gravity

Specific gravity measures of heaviness of the soil particles. The test includes the determination of the specific gravity for the natural soil. The specific gravity test was conducted on the soil in accordance with ASTM D 854- 02 testing procedure.

3.3.2.4 Free Swell Tests

Free swell test is used to study the swelling property of the soils. The test includes the determination of the free swell for the natural soil and soil-CHA mixture. Free swell index test is performed as per the procedure of IS: 2720 (Part 40) 1977 to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample.

3.3.2.5 Compaction

Soil compaction consists of closing, packing the soil particles together, so that increases the dry unit weight. Soil compaction only reduces the air void in the soil. These tests were performed in accordance with AASHTO-T 099-95 to determine the maximum dry density (MDD) and the optimum moisture content (OMC) of the soils. The degree of compaction of soil influences several of its engineering properties such as CBR value, compressibility, stiffness, compressive strength, permeability, shrink, and swell potential. It is, therefore, important to achieve the desired degree of relative compaction necessary to meet the required soil characteristics. The soil mixtures, with and without additives, were thoroughly mixed with various moisture contents. The first series of compaction tests were aimed at determining the compaction properties of the un stabilized soils. The second series tests were carried out to determine the proctor compaction properties of the clay upon stabilization with varying percentage of CHA.

3.3.2.6 Unconfined Compressive Strength (UCS)

The unconfined compressive strength test is applicable only in cohesive soils. The UCS values are determined in accordance with ASTM D 2166. The unconfined compressive strength tests were conducted for both natural and treated soil samples. The unconfined compressive strength tests (UCS) were performed on samples of 3.6 cm diameter and 7.6 cm height, after curing for 0, 7 and 14 days.

3.3.2.7 California bearing ratio (CBR) and CBR Swell

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. A soil mass is prepared in accordance with standards and tests were conducted in accordance with AASHTO T193-93 after the curing of 0, 7 &14 days curing.

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 General

In this chapter laboratory test results are presented and their analysis is briefly discussed. The relevant engineering property of the soil is evaluated both for natural and stabilized soil samples separately.

4.2 Properties of Materials Used in the Study

4.2.1 Natural Soil

A Black clay soil was used in this study. The following laboratory tests were conducted on the natural soil sample.

- Atterberg limits
- Grain size analysis
- Specific Gravity
- Natural moisture content
- Compaction
- Free swell
- UCS
- Soaked and Un-soaked CBR and CBR swell tests

From the summary of the test result shown in Table 4.1, the natural soil can be characterized as highly expansive and of high plasticity. On the particle size distribution curve almost 93.5% of the soil is passing through No. 200 sieve; it exhibits a liquid limit of 99%, a plastic limit of 44.65% and plasticity index of 54.35%. Hence, these values indicate that the soil is highly plastic clay. According to AASHTO soil classification system the soil falls under the A-7-5 and CH based on USCS soil classification system. The liquid limit and plasticity index values are very much greater than the Ethiopian Roads Authorities requirements, i.e., liquid limit less than 60% and plasticity index less than 30%. According to ERA's design manual of 2002, soils with swelling potential and clay content greater than 35% and 28% respectively have high degree of expansiveness.

The free swell result of the soil also indicates that the soil is highly expansive clay with a free swell of about 72.7%. The soil has a maximum dry density of 1.219g/cm^3, optimum moisture content of 47%, un soaked CBR value of 16.2% and the UCS value of 35.73kPa. Furthermore, the soaked CBR and percent swell of 1.62% and 8.36% respectively indicate that the soil has a very low load bearing capacity and high swelling potential when compared to ERA's specifications of CBR > 5% and percent swell of less than 2% which makes it unsuitable for construction without any suitable treatment measure.

However, the comparisons above between ERA design manual and laboratory results of the soil shows that, the soil sample do not full fill the requirements as a sub-grade and are determined to be unsuitable for sub-grade in road construction. Therefore, the sub-grade soil should be treated with appropriate improving methods before use as road sub grade. This can be done by different soil improvement techniques including removal of the soil and replacement with suitable material (capping layer), chemical stabilization, compaction, pre-wetting and making the appropriate pavement designs that considers the unsuitability of the soil. These results have been summarized in table 4.1.

Table 4-1 Test results of natural soil

Properties	Values
% of passing sieve # 200	93.5 %
Specific gravity	2.53
Liquid limit	99%
Plastic limit	44.65 %
Plasticity Index	54.35 %
Free swell	72.7 %
Natural moisture content	30.98 %
Compaction	MDD=1.22 g/cm^3, OMC=47 %
Soaked CBR (1 Pt)	1.62%
Un-Soaked CBR (1 Pt)	16.2 %
CBR Swell	8.36 %
UCS	35.73 kPa
USCS Classification	CH (Highly plastic clay)
AASHTO Classification	A-7-5
Color	Black

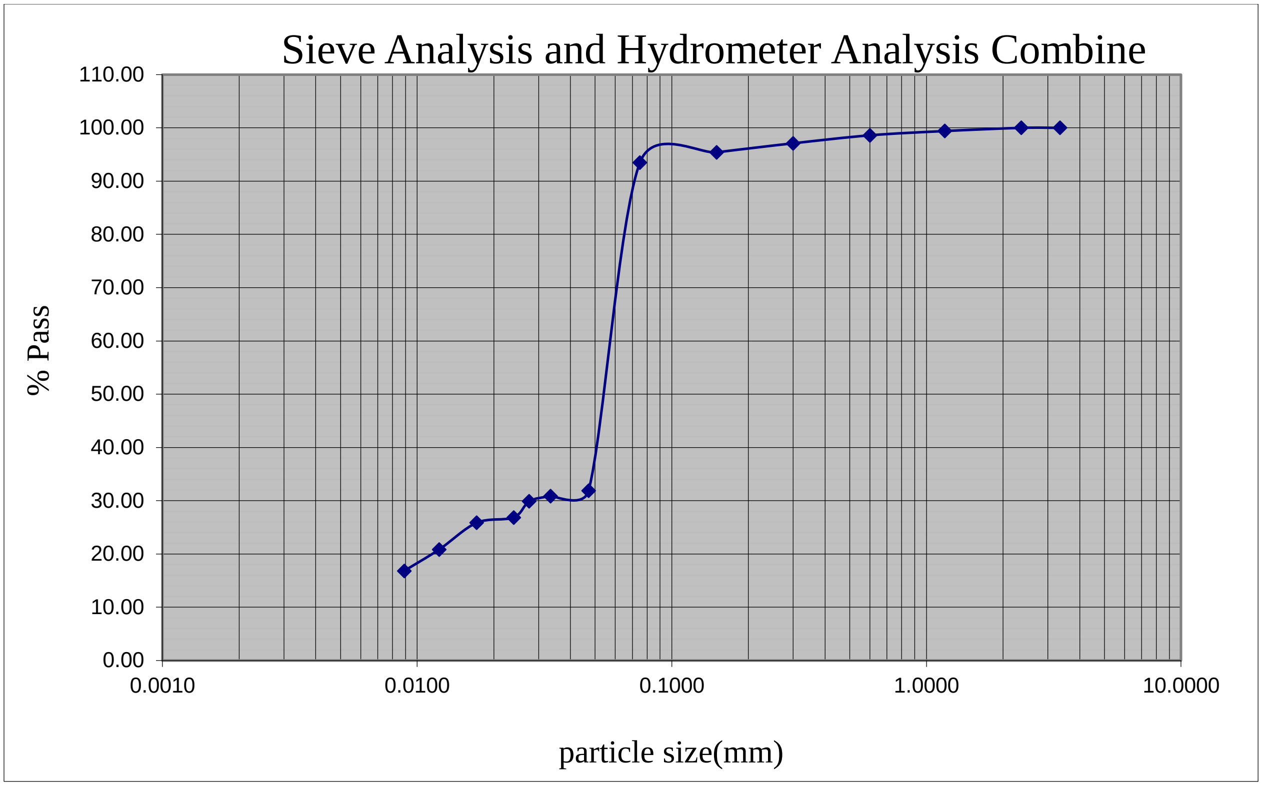


Figure 4-1 Particle size distribution of expansive soil sample

4.2.2 Coffee Husk Ash (CHA)

The properties of the CHA presented in table 2.8, while the laboratory test result is attached in Appendix 6.

4.3 Effects of Coffee husk ash (CHA) on engineering properties of soil

4.3.1 Effect of CHA on free swell

Soil swelling is an expansion in volume that causes significant problems leading to serious damage and economic consequences in construction sectors, mainly in road construction. The free swell index (FSI) test is performed on untreated and treated soil samples. Free swell tests were conducted by mixing CHA at different proportion of 0, 5, 10, 15, 20 and 25 percentages by dry weight of soil sample. The effect of CHA on the free swell index of the soil sample is graphically summarized in Table 4.2.

Table 4-2 Effect of Different percentage of CHA on Free swell index of natural soil

CHA%	0	5	10	15	20	25
FSI%	72.72	33.33	18.75	18.75	17.65	29.03
%Decrease of FSI	0	54	74	74	76	60

Percent decrease in the free swell index =$[\frac{FSIf - FSIi}{FSIi}]*100$ (4.1)

FSIf = value of FSI of the soil for respective percentage of CHA.

FSIi= value of FSI of untreated soil (zero percent CHA).

The results show that the addition of different proportion of CHA on soil samples shows significant change on FSI. The free swell index value decreased from 72.72% to 17.65%. This result was achieved at 20% CHA proportion to the soil sample, which shows about 76% decrease in the free swell index. This is the highest reduction in FSI test. After the addition of 20% more CHA content with the soil, it increased from 17.65% to 29.03%. Further increase in CHA increases the free swell as compared to 20%, indicating that 20% is the optimum value for CHA. The result obtained guaranteed, the soil stabilized using CHA show low degree of expansion as compared to untreated soil, as a result, the soil to have a free swell within the allowable requirements. Generally, increase in percentage of CHA increases potential swell of the soil.

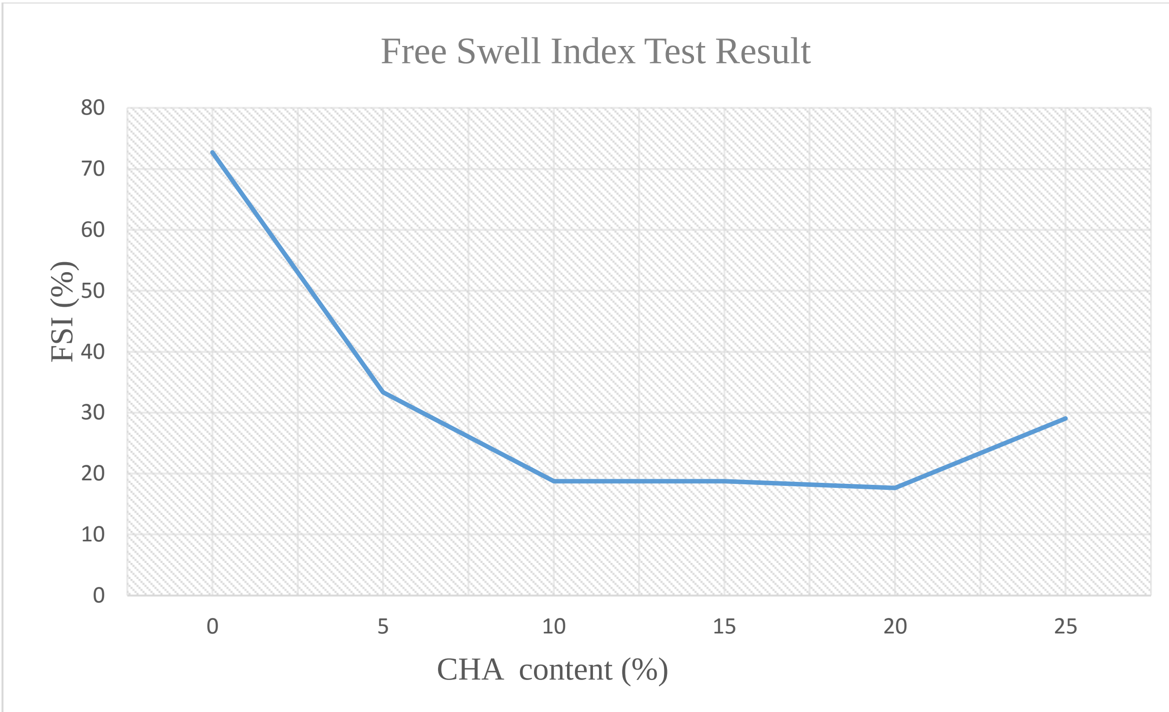


Figure 4-2 Effect of CHA on free swell index of the soil

4.3.2 Effect of CHA on Atterberg's limit of soil

One of the important and principal aims of the present study was to evaluate the changes of liquid limits, plastic limits and plasticity index with addition of CHA to the selected soil samples. To achieve this objective, liquid limit and plastic limit tests were conducted on CHA- soil mixtures according to consistency test of AASHTO T89 and T90, respectively.

Soil samples were first air dried and pulverized and then sieved with no 40 sieve. Soil passing no 40 sieve was mixed with 0%, 5%, 10%, 15%, 20%, 25% and 30% of CHA passing on no 40 sieve.

Samples of soil – CHA mixture showed the highest reduction in PI when they are treated with 25% CHA which is 79.58% reduction compared to untreated soil. It can be observed that CHA reduce the plasticity of soils significantly. After the addition of 25% more CHA content with the soil it increases the PI from 11.1% to 27.76%. Further increase in CHA increases the plasticity index as compared to 25%, indicating that 25% is the optimum value for CHA. The liquid limits significantly decrease with increasing CHA proportions. The values of plastic limits showed inconsistency, even to mention insignificant Changes observed.

The plasticity index also significantly decreases with increasing CHA proportion. This is because of the decrease in the liquid limit, as plasticity index is the difference between liquid limit and plastic limit.

According to the results observed from the laboratory test, one can judge that the behavior of soil sample was changed from very high plasticity soil to medium plasticity soil. Soils of plasticity index greater than 35% are classified as highly plastic and 10-35% medium plastic soil as shown in table 2.1.

Table 4-3 Effect of percentage of CHA on Atterberg's limit of natural soil.

CHA%	0	5	10	15	20	25	30
Liqid Limit%	99	90	91	81.3	72.7	63	77
Plastic Limit%	44.65	45.13	46.28	47.2	52.74	51.9	49.24
PI value%	54.35	44.87	44.72	34.1	19.96	11.1	27.76
%decrease of PI	0	17.44	17.72	37.26	63.28	79.58	48.92

$$\text{Percent decrease in plasticity index} = [(PI_f - PI_o)/ PI_o] *100 \tag{4.2}$$

PI_f = value of PI of the soil for respective percentage of CHA.

PI_o= value of PI of untreated soil (zero percent CHA).

The treatment of the soil with Coffee Husk Ash, have brought appreciable result in decreasing the plasticity of the soil. According to ERA 2002 specification, the maximum values of PI and LL were 30% and 60% respectively to use the soil as a sub grade material. The PI value is attained ERA specification requirement but liquid limit is not attained at 25% CHA. Hence the CHA is not successful in stabilizing expansive soils with regard to Atterberg's limit. Detailed test results are given in Appendix1.

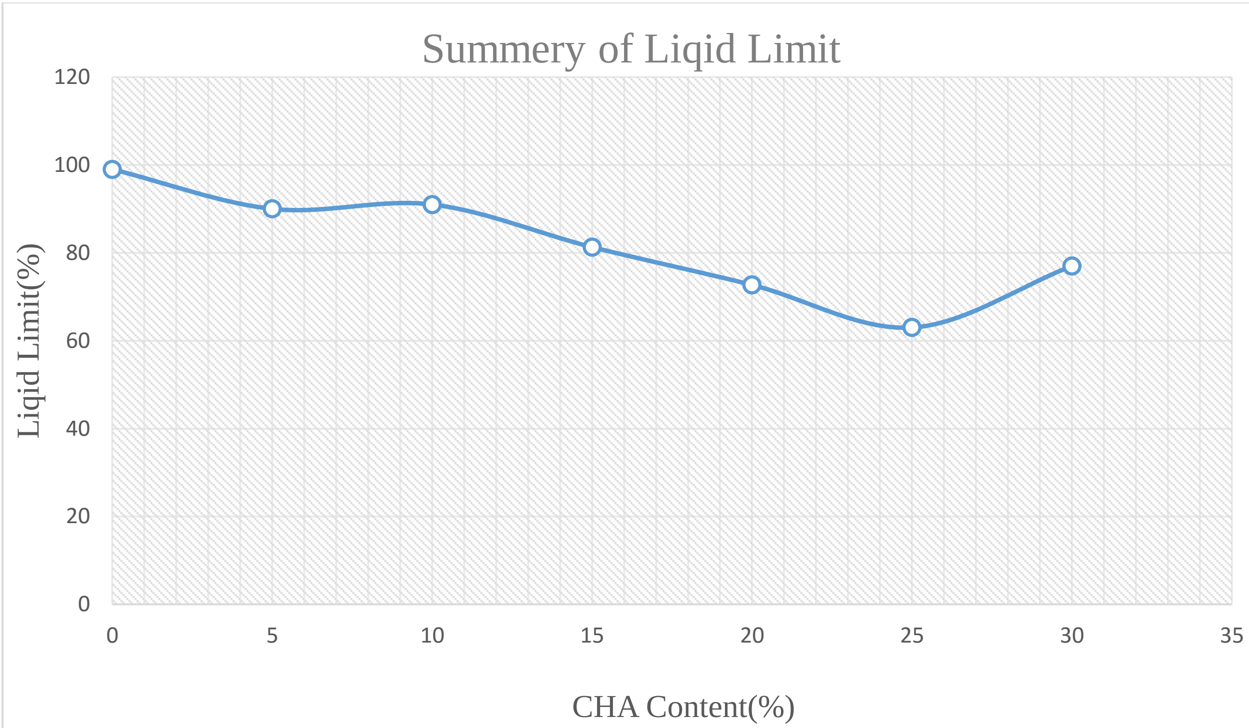


Figure 4-3 variation of liquid limit with different content of Coffee Husk Ash (CHA)

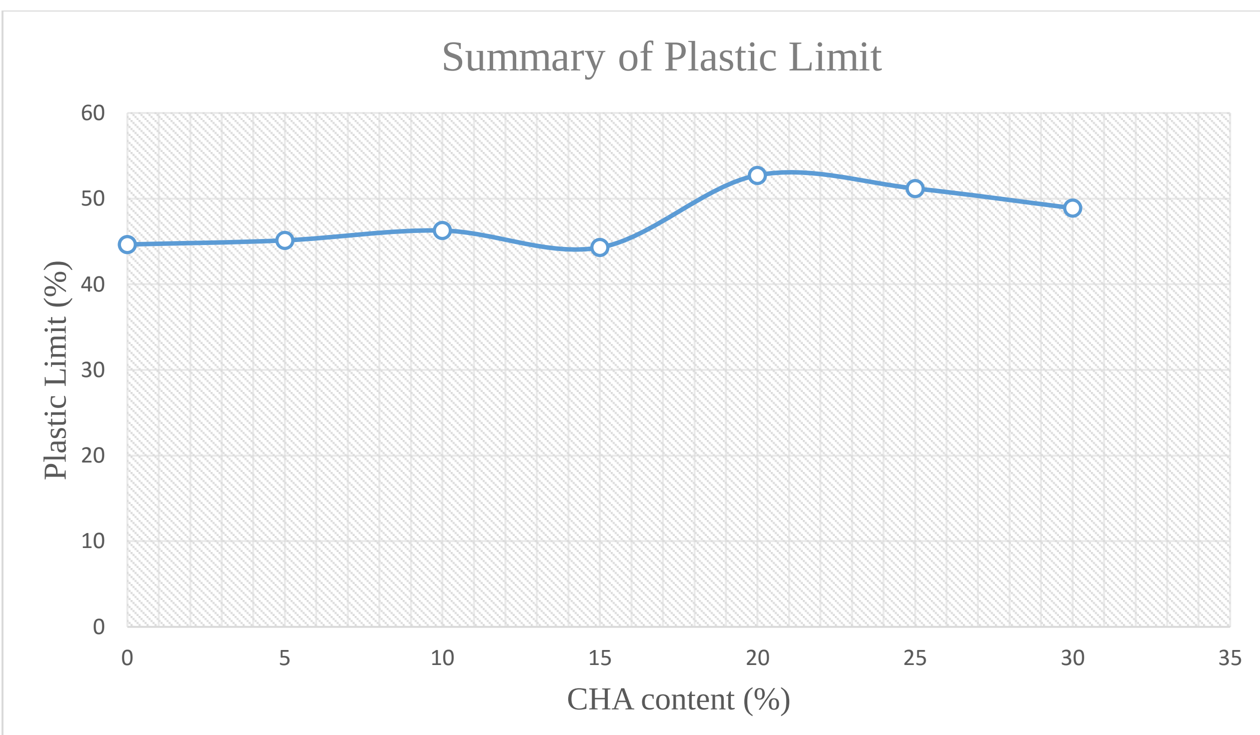


Figure 4-4 variation of plastic limit with different content of CHA.

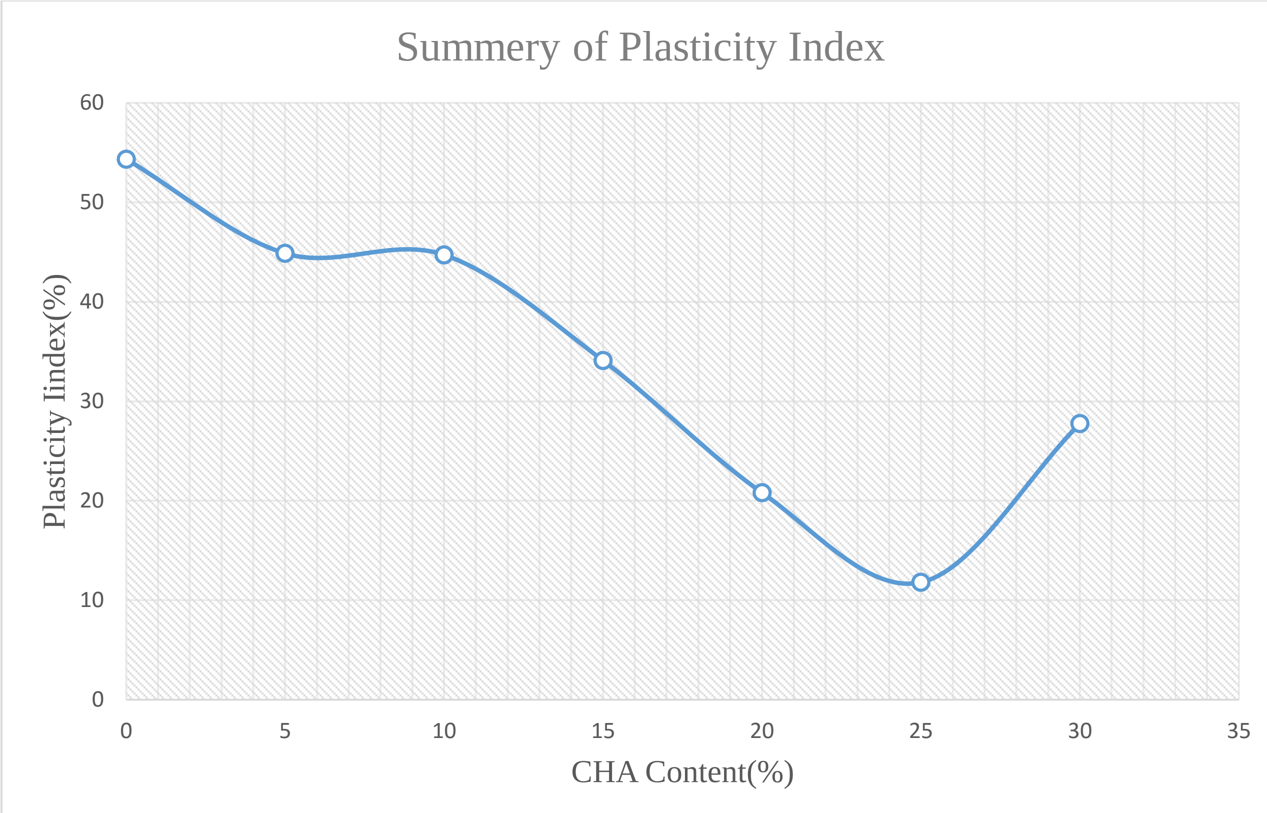


Figure 4-5 variation of plasticity index with different CHA

4.3.3 Effect of CHA on Moisture Density Relations of the Soil

Compaction is the artificial improvement of the mechanical properties of the soil, which increases the resistance and reduces the deformation capacity and void ratio. To determine optimum moisture content and maximum dry density of the expansive soil and treated soil, standard Proctor compaction tests were performed according to AASHTO T 099-95, at different percentages of CHA. Figure 4.6 and 4.7 shows the behavior of the soil due to the addition of different percentages of CHA, up to 25%. As the percentage of CHA increases there is an increase in dry density and a decrease in moisture content up to 15% CHA. The test results are shown below in table 4.4.

Table 4-4 Moisture density relation test results of coffee husk ash treated soil

	Moisture content and Dry Density							% CHA
MC (%)	31.39	33.33	42.51	48.15	57.84			0%
DD(gm./cc)	1.176	1.177	1.203	1.214	1.104			
MC (%)	30.89	34.95	37.58	38.77	40.36	41.50		5%
DD(gm./cc)	1.236	1.249	1.266	1.276	1.301	1.267		
MC (%)	18.48	19.62	24.66	25.81	32.50	39.29	40.42	10%
DD(gm./cc)	1.287	1.303	1.334	1.339	1.371	1.342	1.329	
MC (%)	23.62	26.47	27.80	29.08	30.53	35.05	38.82	15%
DD(gm./cc)	1.266	1.294	1.350	1.374	1.382	1.350	1.294	
MC (%)	26.81	30.56	37.50	38.97	43.67	48.28		20%
DD(gm./cc)	1.271	1.282	1.336	1.339	1.271	1.205		
MC (%)	30.69	35.38	38.22	40.53	47.47			25%
DD(gm./cc)	1.275	1.281	1.284	1.326	1.259			

The addition of CHA increase the maximum dry density of the natural soil from 1.219 g/cm3 to 1.384g/cm3 at 15% CHA. This result implied that there is a maximum increase in maximum dry density of a natural soil by13.5%.

However, increase in CHA percentage decrease optimum moisture content of the natural soil from 47% to 30% for the addition of 15% CHA. This result assures that there was 36% decrease in the OMC of the soil sample. After the addition of 15% more CHA content with the soil it increases the OMC and decreases the MDD from 30% to 40.5% and 1.384 to 1.328g/cm^3 respectively. Further increase in CHA increases the OMC and decreases the MDD as compared to 15%, indicating that 15% is the optimum value for CHA.

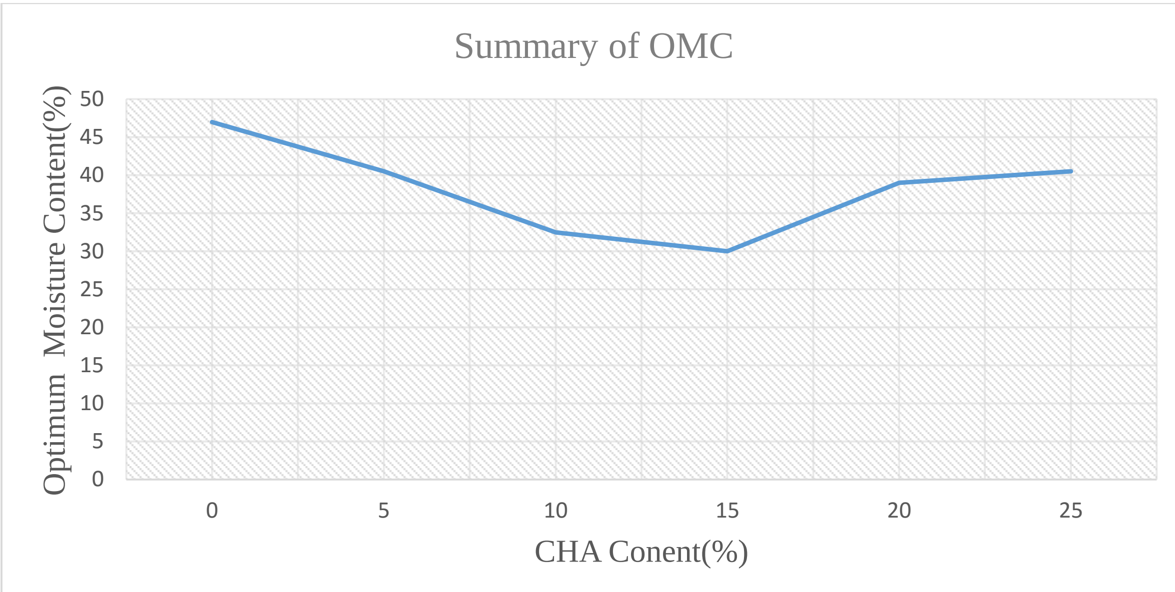


Figure 4-6 Variation of OMC with addition of different CHA content

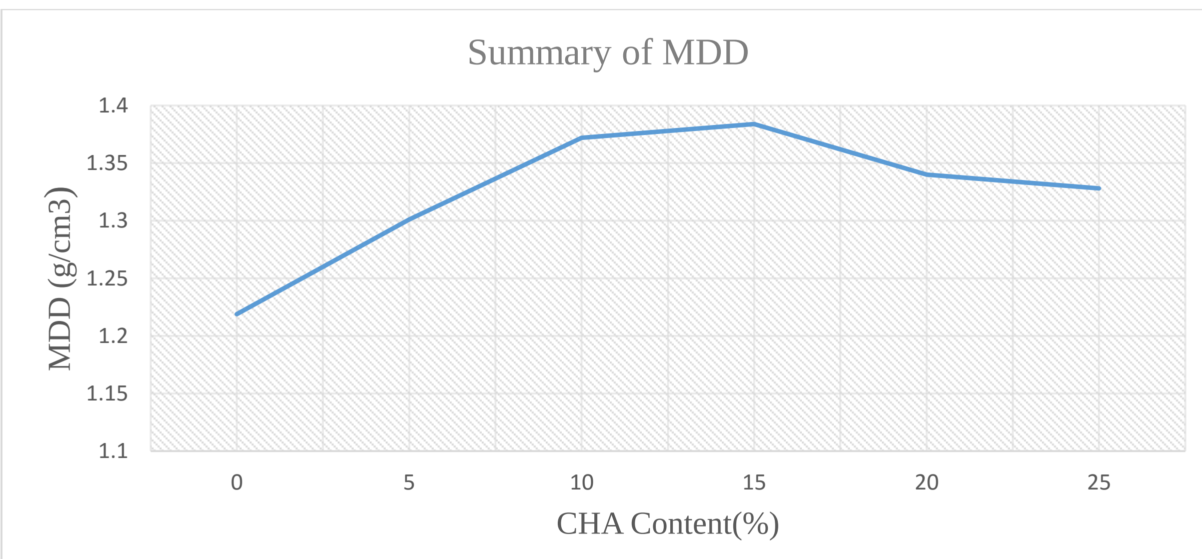


Figure 4-7 Variation of MDD with addition of different CHA content

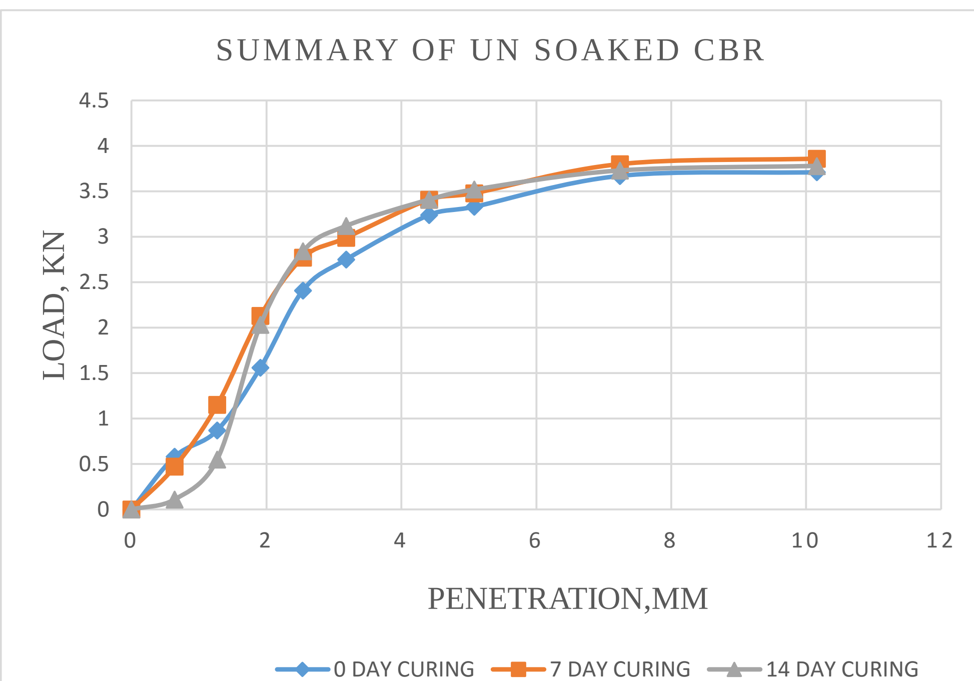
The details of compaction curves are shown in Appendix 2 for 0%, 5%, 10%, 15%, 20 and 25% of CHA contents.

4.3.4 Effect of CHA on CBR and CBR-Swell Values

4.3.4.1 Effect of CHA on Un soaked CBR of the soil

Un-soaked values of CBR showed an increasing trend with 15% of CHA content. There is an increase of un soaked CBR from 16.2 % to 18.27% with the addition of 15% CHA and 0 day of curing. Figure 4.9 shows the effect of curing day on un soaked CBR value of 15% CHA treated soil. The maximum un-soaked CBR value has reached 21.5% for 15% CHA content and 14 days of curing. The load bearing capacity of the sample shows significant increment with CHA treatment and also curing have an effect on the un soaked CBR values.

Hence, the CHA treated black cotton soil meet the minimum criteria as specified by Ethiopian Roads Authority Pavement design manual (2002) specification for materials suitable for use as sub grade material.



<u>0 Day curing</u>

CBR=18.27%

<u>7 Day curing</u>

CBR=21.02%

<u>14 Day curing</u>

CBR=21.5%

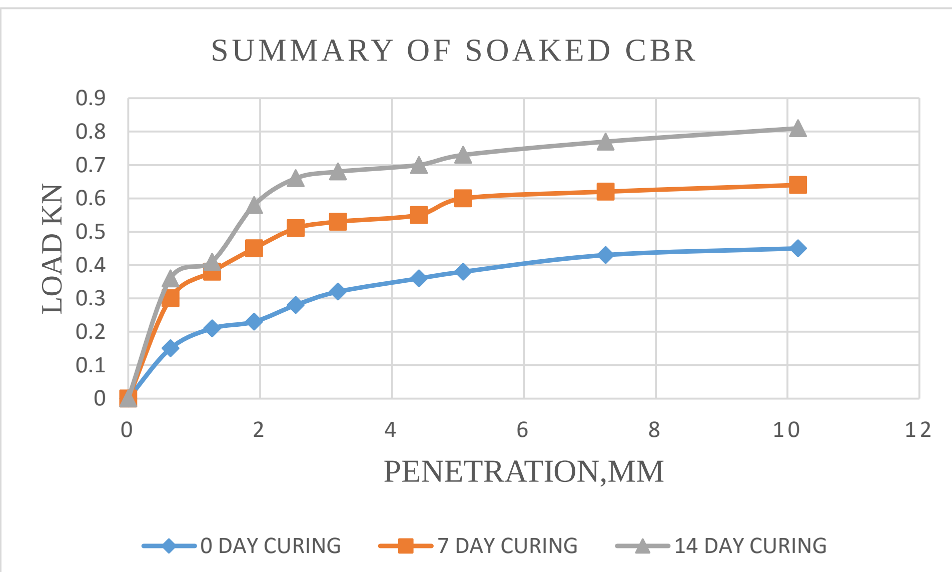
Figure 4-8 effect of curing day on un soaked CBR value 15% CHA treated soil.

4.3.4.2 Effect of CHA on soaked CBR of the soil

Air dried and pulverized soil passing no 4 sieve was mixed with CHA at optimum moisture content and compacted in a CBR mold at maximum dry density. Then CBR tests were conducted after the sample was soaked for 96 hrs for uncured condition. For the cured condition the sample was cured in the test tube after compaction for 0, 7 and 14 days by sealing the tube with plastic in order to avoid moisture content reduction.

There is an increase of soaked CBR from 1.62 % to 2.1% with the addition of 15% CHA and 0 day of curing. The maximum value of soaked CBR 5.1% was obtained for 15% of CHA content and 14 days of curing. So it can be accomplished that the maximum CBR were obtained at 15% of CHA and further addition of CHA, decreases the CBR of the expansive soil. It is observed that the CBR increase as the curing period increases.

However according to Ethiopian Roads Authority pavement design manual specification CHA treated soil satisfied a minimum CBR value not less than 5%. Hence one can conclude that an optimum increase of 15% CHA can peak out expansive soils from problematic sub grade material to fair sub grade material. The effect of curing day on 15% CHA is shown on figure 4.8.



<u>0 Day curing</u>

CBR=2.1%

<u>7 Day curing</u>

CBR=3.88%

<u>14 Day curing</u>

CBR=5.1%

Figure 4-9 effect of curing day on soaked CBR value 15% CHA treated soil

4.3.4.3 Effect of CHA on CBR- Swell of the soil

The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the CBR swell of the soil. The initial dial reading of the soil of the dial indicator on the soaked CBR of mold is taken just after soaking the sample. At the end of ninety-six hours the final dial reading of the dial indicator is taken.

The effect of 15% CHA on the CBR-Swell for different curing days are shown in Figure 4.8. When treating the expansive soil at 0% CHA content, sample gives CBR-Swell of 8.36%. According to ERA specification manual the value of CBR swell should be less than 2% to use as sub grade material. The result shown that the soil needs treatment. The CBR-Swell decreases from 8.36 % to 2.39 % with the addition of 15% CHA and 0 day of curing. The maximum CBR-Swell value has reached 1.84% for 15% CHA content and 14 days of curing. It is observed that the CBR-Swell decrease as the curing period increases. This shows that the swelling potential of the sample decreased with CHA stabilization and it satisfies the ERA specification manual for use as sub grade material.

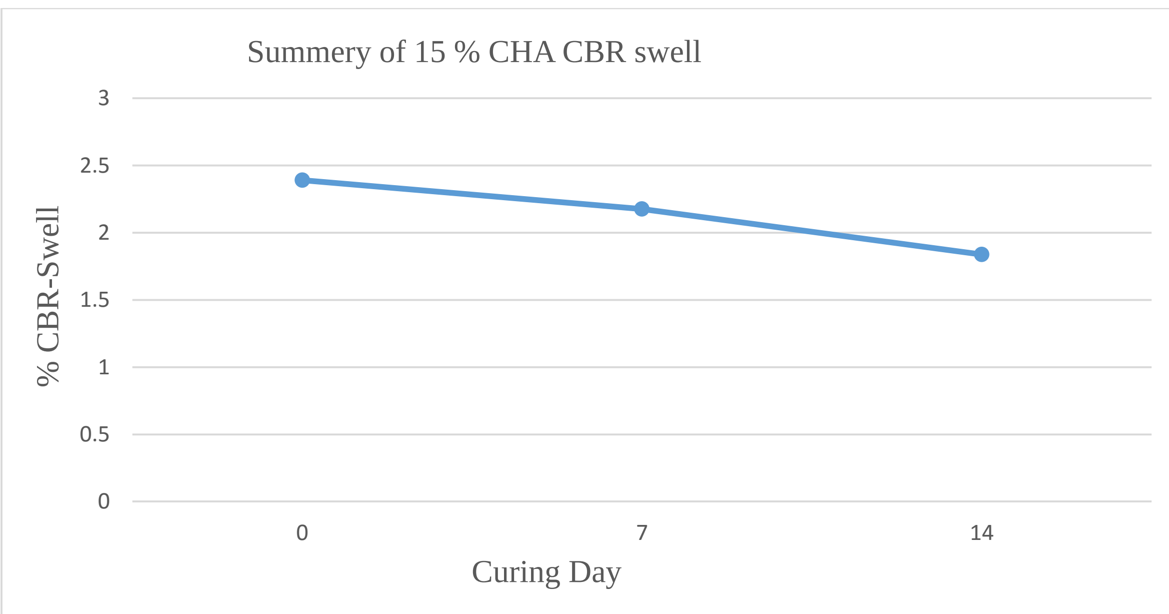


Figure 4-10 Effect of curing day on CBR-Swell of 15% CHA

4.3.5 Effect of CHA on UCS

The unconfined compressive strength of the remolded samples prepared at MDD and optimum moisture content with addition of 0% and 15% of CHA to the expansive soil. The prepared samples are tested after 0, 7 and 14 days. The sample was cured in the test tube after compaction for 0 (uncured), 7 and 14 days by sealing the tube with plastic in order to avoid moisture content reduction. Then the sample has extruded from the sampler and have been made for the test. The addition of CHA to soil samples shows considerable improvement. As the percentage of CHA increases from 0 to 15% the UCS indicates significant increment and consequently yields a higher load. The value of UCS for un treated soil and variation of unconfined compressive strength for soil at 15% of CHA is also shown in Figure 4.11. There is an increase of unconfined compressive strength from 35.73 kPa to 198.3 kPa with the addition of 15% CHA and 0 day of curing (uncured). The maximum value of unconfined compressive strength 314 kPa was obtained at 15% of CHA content and 14 days of curing. So it can be accomplished that the maximum unconfined compressive strength was obtained at 15% of CHA and further addition of CHA, decreases the UCS of the expansive soil. It is observed that the unconfined compressive strength increase as the curing period increases. Generally, CHA has positive impact on the UCS values for all 0, 7 and 14 days cured treated soil samples. Also the UCS of the treated soil increases with curing period increases.

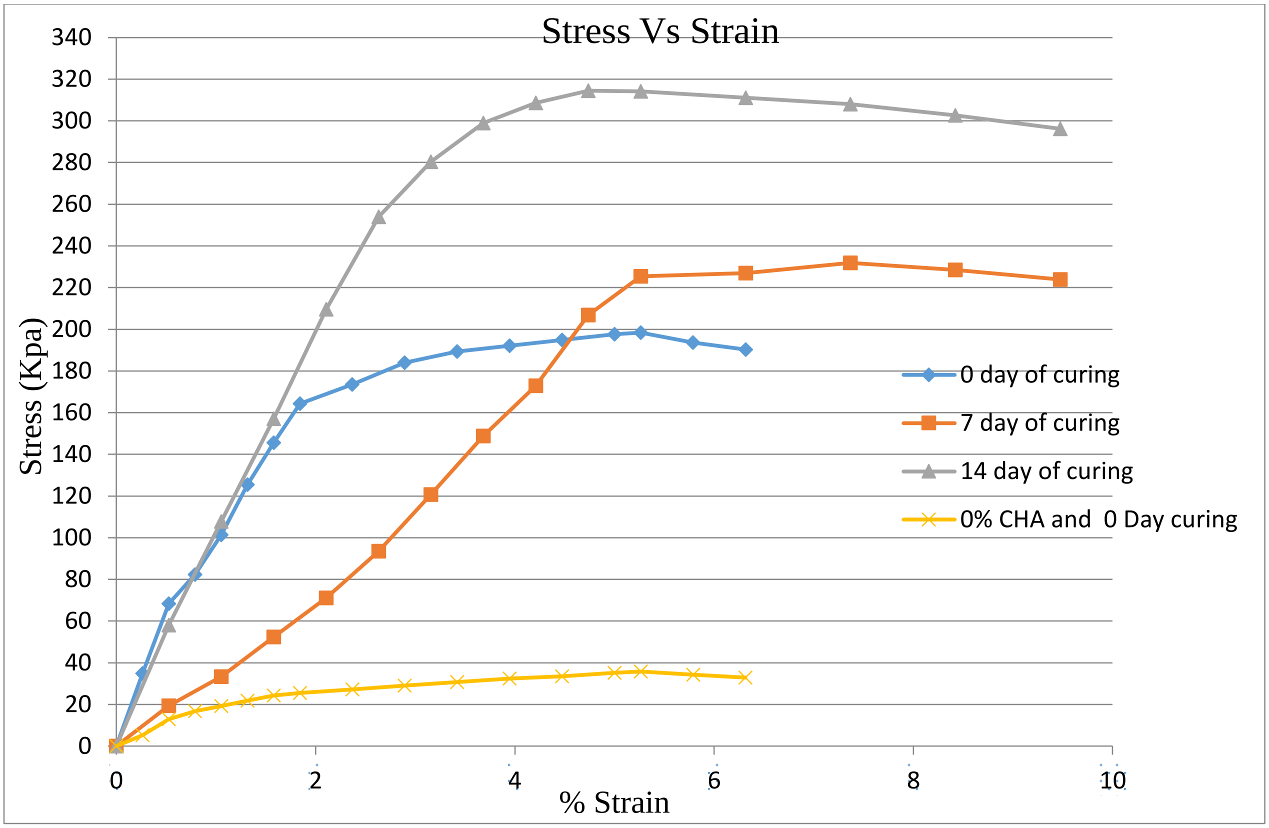


Figure 4-11 Summery of UCS curves for un treated and with the application of 15% CHA at different curing days

CHAPTER FIVE

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

An experiment was undertaken to investigate the effects of Coffee husk ash on some geotechnical characteristics of expansive soil. The following conclusions can be drawn from the results of the study;

The addition of different proportion of CHA on soil samples shows significant change on free swell index tests. But after 20% of CHA soil mixture the value of FSI is increased from 17.65% to 29.03%. Generally, increase in percentage of CHA increases potential swell of the soil.

The treatment of the soil with Coffee Husk Ash, have brought appreciable result in decreasing the plasticity of the soil from 54.35% to 11.1%. According to ERA 2002 specification, the maximum values of PI and LL were 30% and 60% respectively to use the soil as a sub grade material. The PI value attains ERA specification requirement but liquid limit is not attained at 25% CHA. Hence the CHA is not successful in stabilizing expansive soils with regard to Atterberg's limit.

The maximum dry density increased from 1.22 g/cm^3 to 1.384 g/cm^3 and optimum moisture content decreased from 47% to 30% up to 15% of CHA. After the addition of 15% more CHA content with the soil it increases the OMC to 40.5% and decreases the MDD to 1.32 g/cm^3.

The Un-soaked and Soaked values of CBR showed an increasing trend with 15%of CHA content; Results also show that curing enhances the un-soaked and soaked CBR value of expansive soil treated with CHA. And also the CBR-Swell shown an appreciable value at 15% CHA and 14 days of curing.

The UCS value found to increase appreciably at 15% of CHA content. The maximum increment of UCS for CHA treated soil is 314 kpa which was obtained for 15% CHA over 14 days of curing. It is observed that the unconfined compressive strength increase as the curing period increases.

Generally coffee husk ash can be an alternative expansive soil stabilizing material but it’s not effective.

5.2 Recommendations

This study tried to investigate the technical feasibility of CHA in stabilizing expansive soil, during the investigation CHA has improved the FSI, PI, compaction (MDD and OMC), UCS, CBR and percent CBR swell values except atterberg's limit of Liquid limit value. With regard to this result CHA can be used as alternative stabilizing material for highly plastic soil, as it has changed the property of expansive soil to medium plastic. But it cannot be concluded as effective.

Furthermore in order to investigate the ability of CHA in stabilizing expansive soil it is important to use appropriate muffle furnace which considers all corrected experimental value.

In addition to this the chemical and physical properties of CHA depends on incineration conditions (temperature and duration), geographic location, crop variety and year of crop production. Therefore, it's mandatory to assess the stabilizing potential of CHA from different sources and also using CHA with another stabilizing agent may alter (improve) the expansive soil property.

Finally the effectiveness of CHA in stabilizing expansive soils needs further study.

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APPENDICES

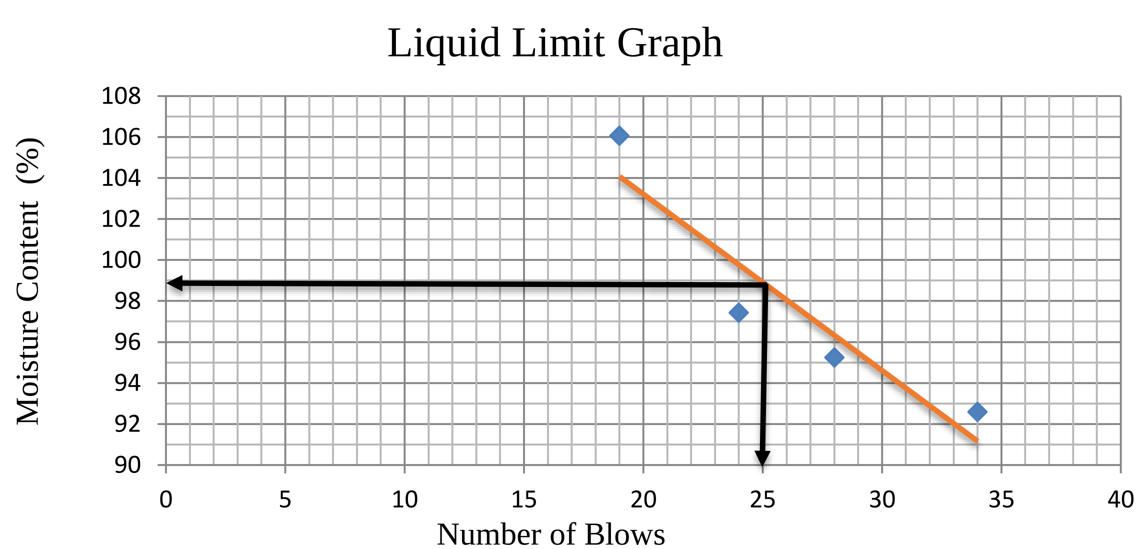
Appendix 1

Atterberg Limits with Varying Percentage of CHA

1. 0% CHA Content

LL=99%, PL=44.65% PI=54.35%

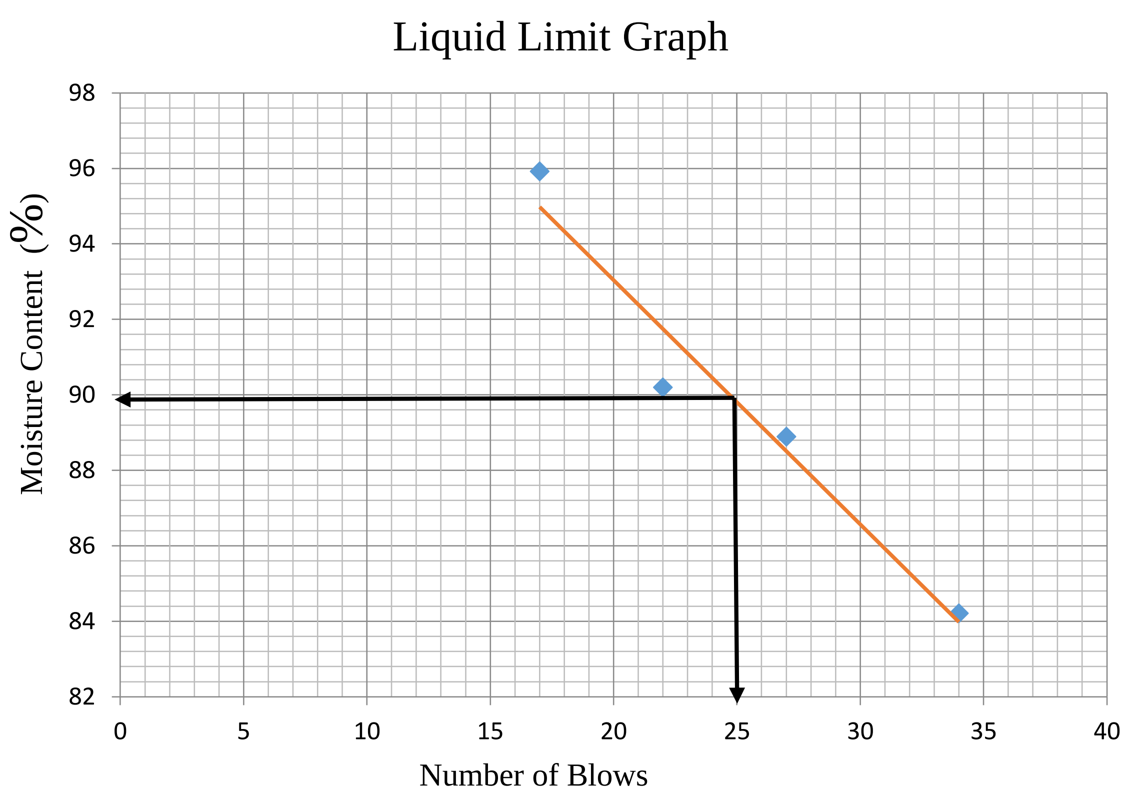
	[A] Liquid Limit					[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	19	24	28	34		
B	Can No.	A3	GTZ-22	A1	DV-5	G3	D5
C	Mass of can + Moist Soil, g	27.6	27.9	28.9	25.2	24.2	22.9
D	Mass of can + dry soil, g	24.1	24.1	24.9	22.7	23.1	21.9
E	Mass of can, g	20.8	20.2	20.7	20	20.7	19.6
F	Mass of water(C-D)	3.5	3.8	4	2.5	1.1	1
G	Mass of dry soil, g(D-E)	3.3	3.9	4.2	2.7	2.4	2.3
H	Moisture content, w(%)F/G	1.0606	0.9744	0.9524	0.926	0.458333	0.434783
	LL = 99%	106.06	97.436	95.238	92.59	45.83333	43.47826



2. 5% CHA content

LL=90%, PL=45.13%, PI=44.87%

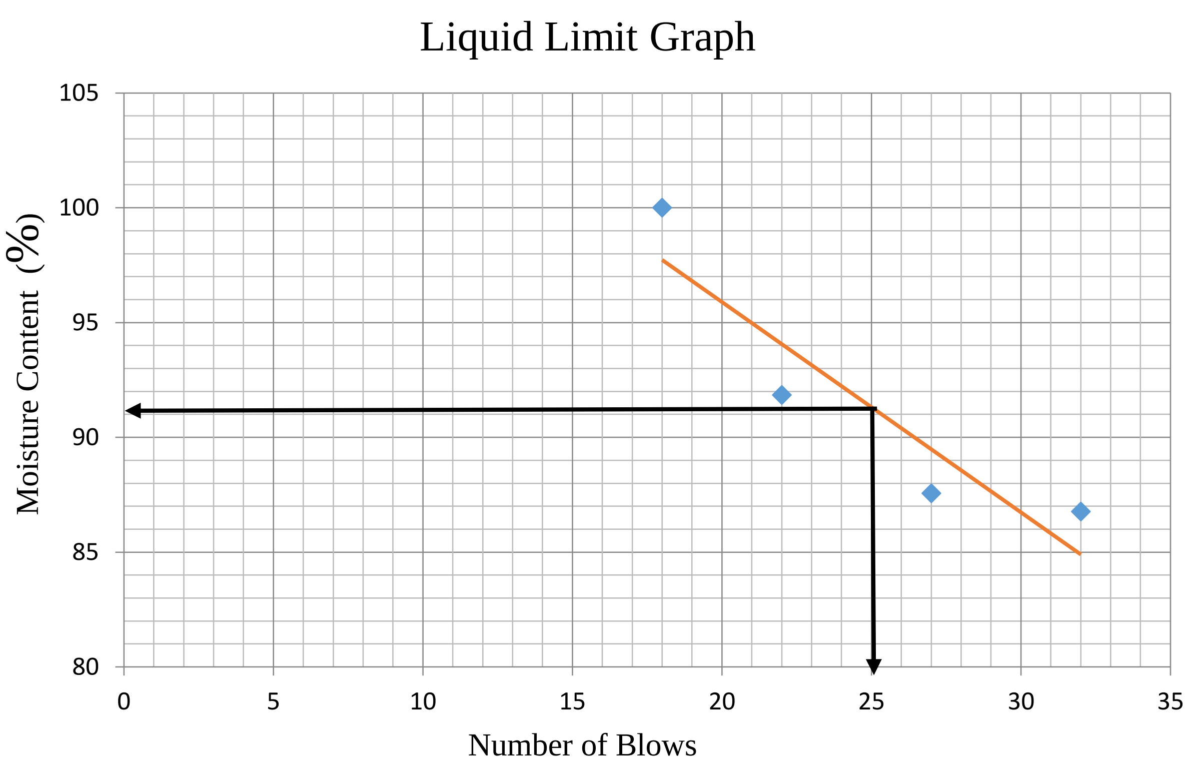
[A] Liquid Limit						[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	
A	Number of Blows	17	22	27	34		
B	Can No.	C3	FA3	G44	DV9	B4	B9
C	Mass of can + Moist Soil, g	34.6	30.9	31.3	26.6	23.8	23
D	Mass of can + dry soil, g	29.9	26.3	26.5	23.4	23	21.9
E	Mass of can, g	25	21.2	21.1	19.6	21.2	19.5
F	Mass of water(C-D)	4.7	4.6	4.8	3.2	0.8	1.1
G	Mass of dry soil, g(D-E)	4.9	5.1	5.4	3.8	1.8	2.4
H	Moisture content, w(%)F/G	0.9592	0.902	0.8889	0.842	0.444444	0.458333
	LL = 89%	95.918	90.196	88.889	84.21	44.44444	45.83333



3. 10% CHA Content

LL=91, PL=46.28, PI=44.72

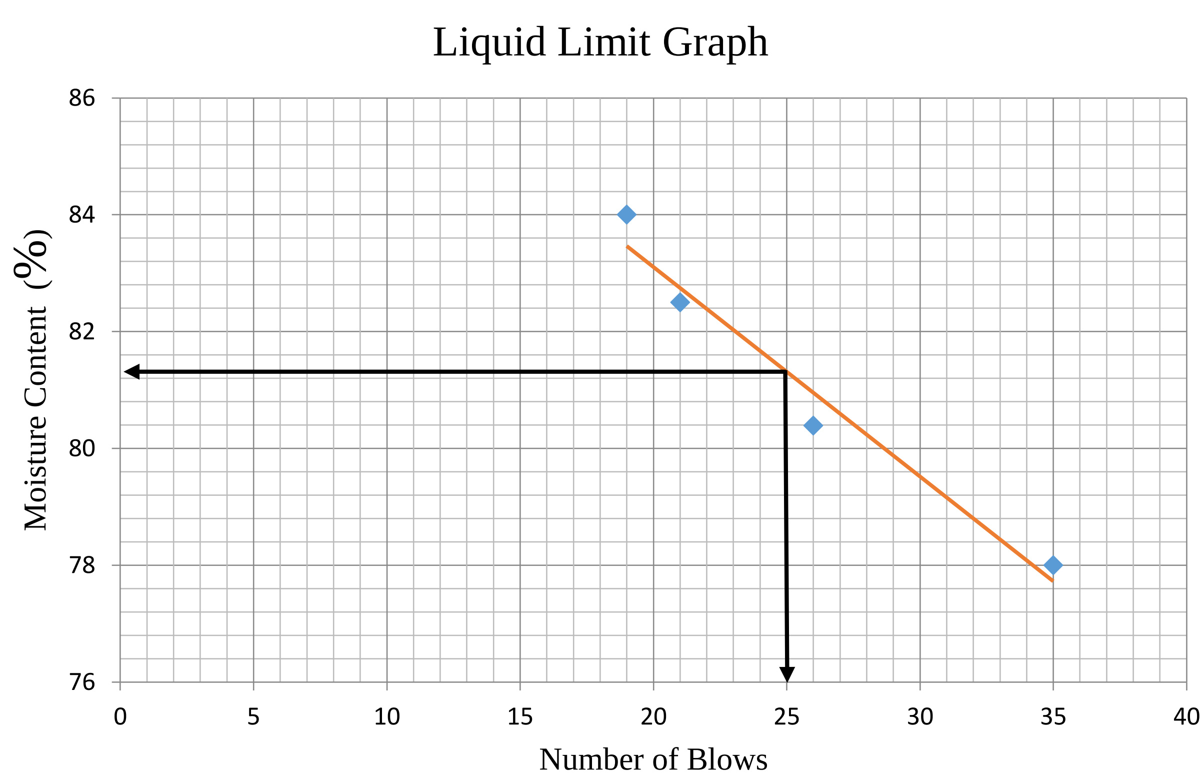
[A] Liquid Limit						[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	18	22	27	32		
B	Can No.	B4	DV2	B1	G2	G3	FA3
C	Mass of can + Moist Soil, g	31.5	33.6	28.74	33.4	25.2	25.4
D	Mass of can + dry soil, g	26.3	27.52	24.8	27.5	23.8	24.1
E	Mass of can, g	21.1	20.9	20.3	20.7	20.8	21.3
F	Mass of water(C-D)	5.2	6.08	3.94	5.9	1.4	1.3
G	Mass of dry soil, g(D-E)	5.2	6.62	4.5	6.8	3.2	2.9
H	Moisture content, w(%)F/G	1	0.9184	0.8756	0.86764706	0.46666	0.464285
	LL = 91%	100	91.843	87.556	86.7647059	46.666	46.4285



4. 15% CHA Content

LL=81.3, PL=47.2, PI=34.1

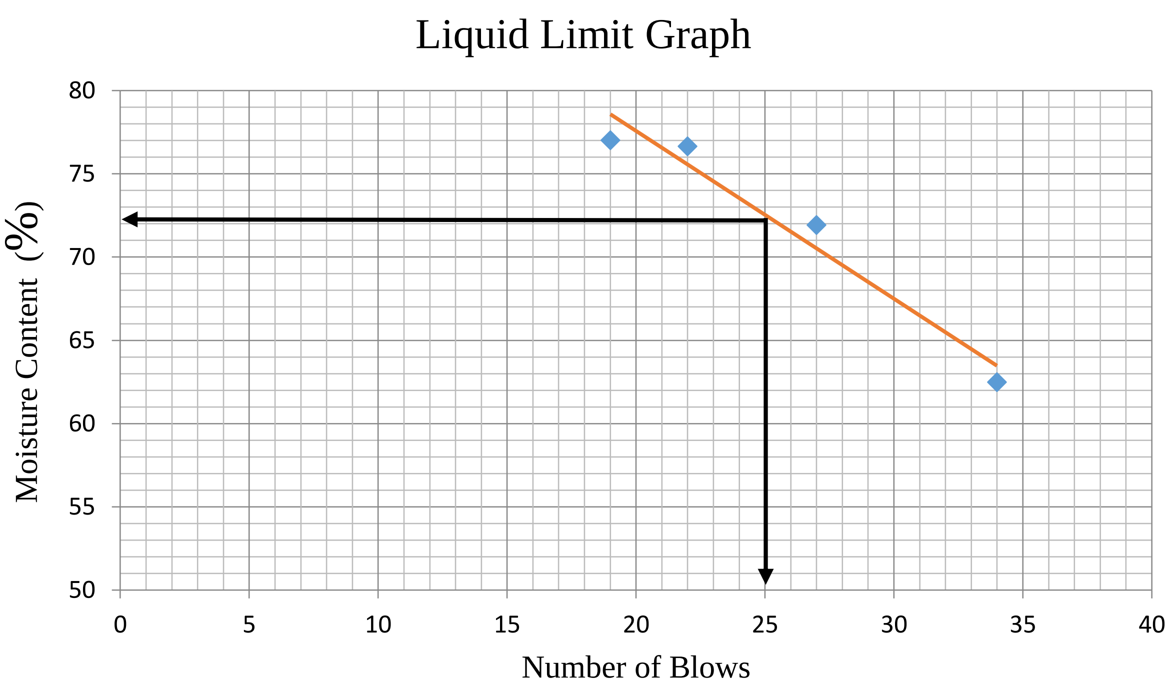
	[A] Liquid Limit					[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	19	21	26	35		
B	Can No.	B4	A3	DR	FA	G2	B9
C	Mass of can + Moist Soil, g	30.1	28	29.4	28.5	23	22.3
D	Mass of can + dry soil, g	25.9	24.7	25.3	24.6	22.2	21.4
E	Mass of can, g	20.9	20.7	20.2	19.6	20.5	19.5
F	Mass of water(C-D)	4.2	3.3	4.1	3.9	0.8	0.9
G	Mass of dry soil, g(D-E)	5	4	5.1	5	1.7	1.9
H	Moisture content, w(%)F/G	0.84	0.825	0.8039	0.78	0.470588	0.473684
	LL = 81.3%	84	82.5	80.392	78	47.05882	47.36842



5. 20% CHA Content

LL=72.2, PL=51.38, PI=20.82

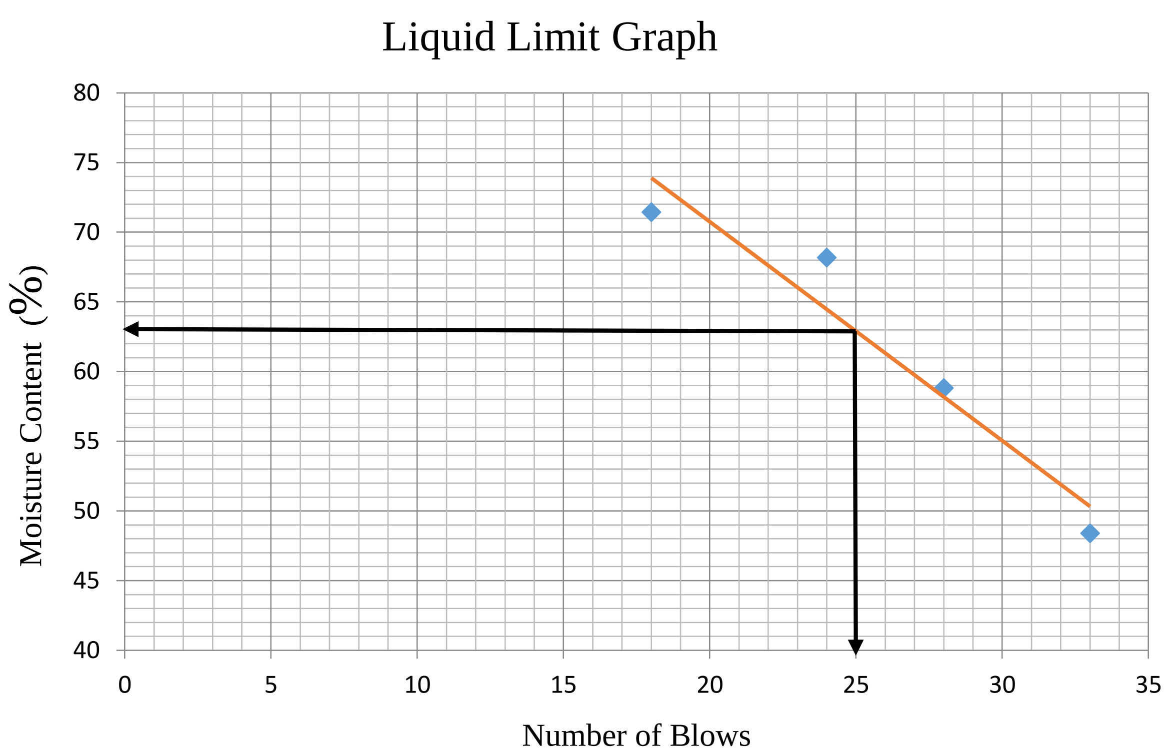
[A] Liquid Limit						[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	19	22	27	34		
B	Can No.	GTZ-2	B9	B4	Z11	DV2	DV5
C	Mass of can + Moist Soil, g	34	30.2	29.8	29.1	26.4	25.4
D	Mass of can + dry soil, g	28.3	25.6	25.7	25.6	24.5	23.6
E	Mass of can, g	20.9	19.6	20	20	20.9	20
F	Mass of water(C=D)	5.7	4.6	4.1	3.5	1.9	1.8
G	Mass of dry soil, g(D-E)	7.4	6	5.7	5.6	3.6	3.6
H	Moisture content, w(%)F/G	0.7703	0.7667	0.7193	0.625	0.527778	0.5
	LL = 72.2%	77.027	76.667	71.93	62.5	52.77778	50



6. 25% CHA Content

LL=63, PL=51.2, PI=11.8

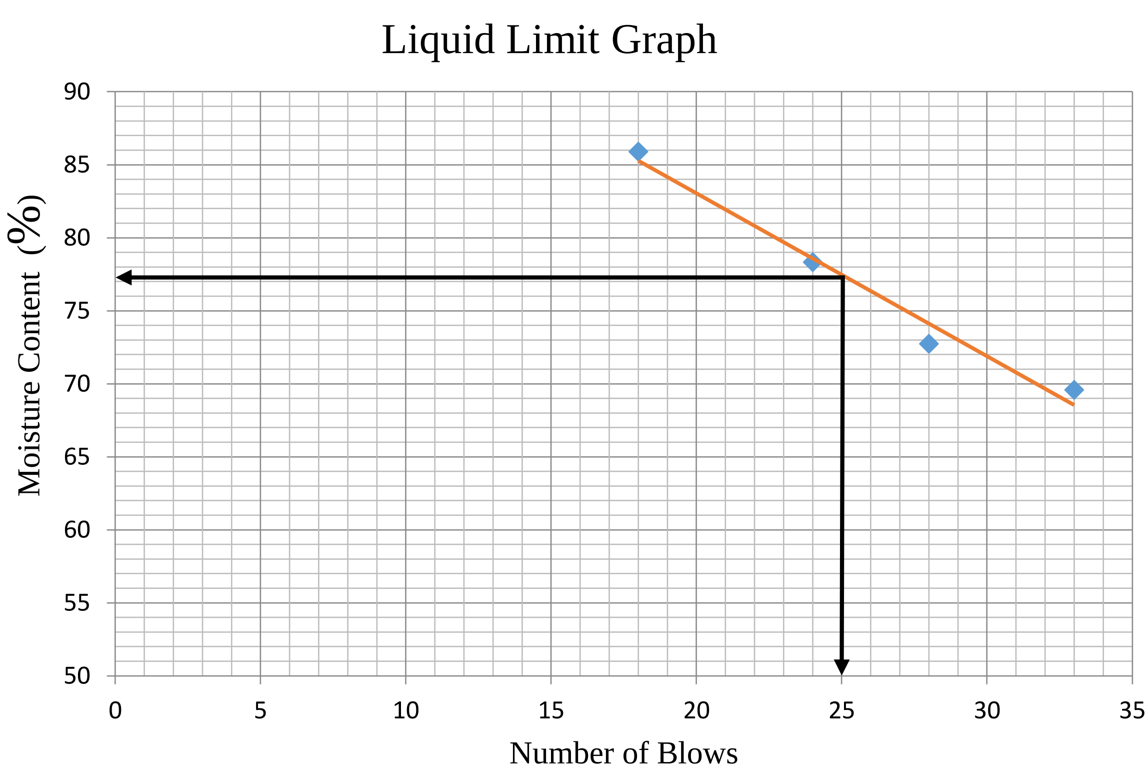
	[A] Liquid Limit					[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	18	24	28	33		
B	Can No.	G2	B4	DV2	G3	B9	DV5
C	Mass of can + Moist Soil, g	31.5	28.5	29	25.2	25	23.2
D	Mass of can + dry soil, g	27	25.5	26	23.7	23.2	22.1
E	Mass of can, g	20.7	21.1	20.9	20.6	19.6	20
F	Mass of water(C-D)	4.5	3	3	1.5	1.8	1.1
G	Mass of dry soil, g(D-E)	6.3	4.4	5.1	3.1	3.6	2.1
H	Moisture content, w(%)F/G	0.7143	0.6818	0.5882	0.484	0.5	0.52381
	LL = <u>63%</u>	71.429	68.182	58.824	48.39	50	52.38095



7. 30% CHA Content

LL=77, PL=49.24, PI=27.76

[A] Liquid Limit						[B] Plastic Limit	
	DETERMINATION No.	1	2	3	4	1	2
A	Number of Blows	18	24	28	33		
B	Can No.	A3	G3	B4	FA3	G44	B1
C	Mass of can + Moist Soil, g	35.2	31.3	28.5	27.4	26.1	25.1
D	Mass of can + dry soil, g	28.5	26.6	25.3	24.2	24.5	23.5
E	Mass of can, g	20.7	20.6	20.9	19.6	21.2	20.3
F	Mass of water(C-D)	6.7	4.7	3.2	3.2	1.6	1.6
G	Mass of dry soil, g(D-E)	7.8	6	4.4	4.6	3.3	3.2
H	Moisture content, w(%)(F/G)	0.859	0.7833	0.7273	0.696	0.484848	0.5
	LL = 77%	85.897	78.333	72.727	69.57	48.48485	50



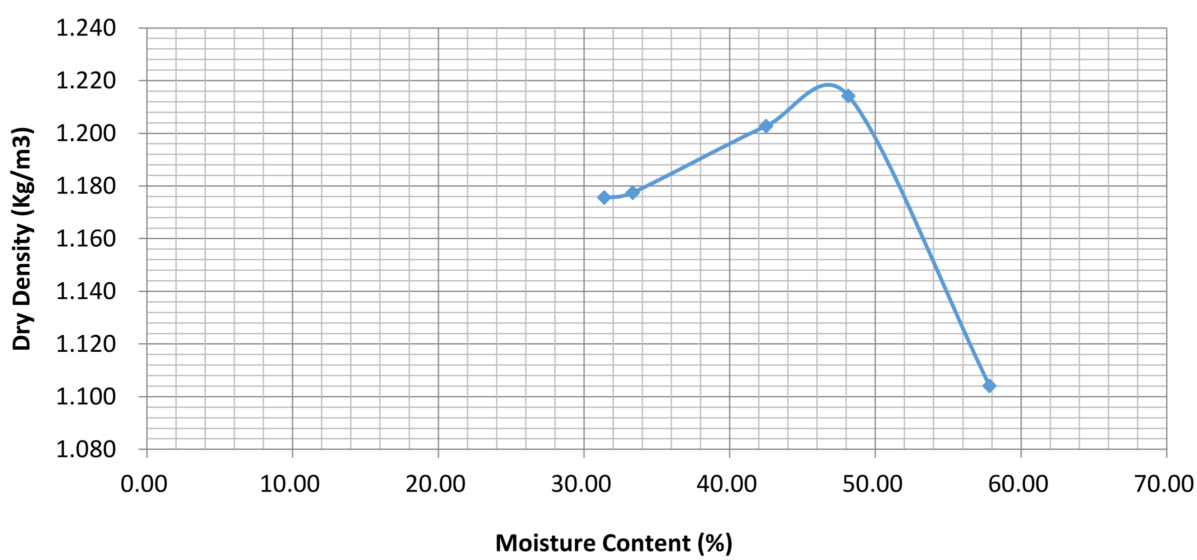
Appendix 2

Compaction curves with varying percentages of CHA

1. 0% CHA

BLOWS PER LAYER	25	No. OF LAYERS	3	RAMMER WEIGHT	2.5	
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)	944			
WEIGHT OF WET SOIL + MOLD (gm.)	4999.00	5022.90	5159.00	5239.00	5186.00	**NMC**
WEIGHT OF MOLD (gm.)	3540.90	3540.90	3540.90	3540.90	3540.90	
WEIGHT OF WET SOIL (gm.)	1458.10	1482.00	1618.10	1698.10	1645.10	
WET DENSITY (gm./cm3	1.54	1.57	1.71	1.80	1.74	
MOISTURE CONTENT : CONTAINER No.	D5	P23	G3	DV5	GTZ22	
WEIGHT OF WET SOIL + CONTAINER (gm.)	49.00	35.90	56.00	52.10	49.30	
WEIGHT OF DRY SOIL + CONTAINER	42.00	32.00	45.50	41.70	38.60	
WEIGHT OF CONTAINER (gm.)	19.70	20.30	20.80	20.10	20.10	
WEIGHT OF MOISTURE (gm.)	7.00	3.90	10.50	10.40	10.70	
WEIGHT OF DRY SOIL (gm.)	22.30	11.70	24.70	21.60	18.50	
MOISTURE CONTENT (%)	**31.39**	**33.33**	**42.51**	**48.15**	**57.84**	**20.9**
DRY DENSITY (gm./cm3)	**1.176**	**1.177**	**1.203**	**1.214**	**1.104**	

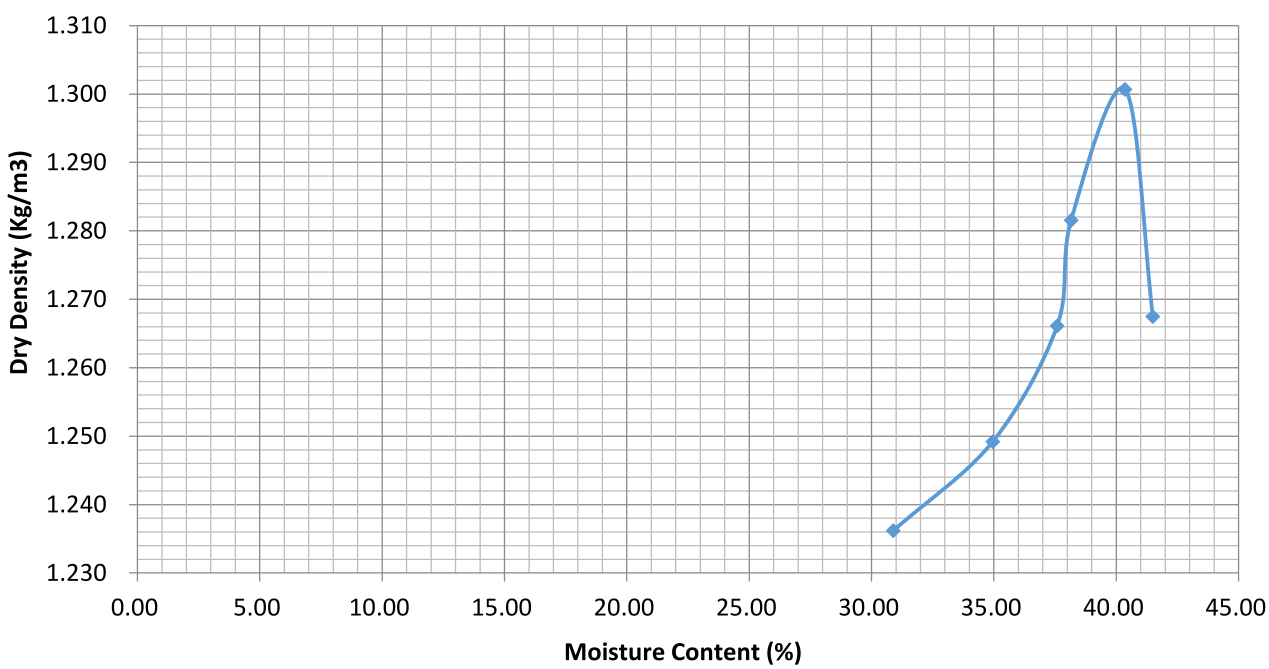
MDD=1.219 gm./cm^3 OMC=47%



2. 5% CHA

BLOWS PER LAYER	25	No. OF LAYERS		3	RAMMER WEIGHT			**2.5**
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)		944				
WEIGHT OF WET SOIL + MOLD (gm.)		5069.00	5133.00	5186.00	5213.00	5265.00	5234.60	**NMC**
WEIGHT OF MOLD (gm.)		3541.60	3541.60	3541.60	3541.60	3541.60	3541.60	
WEIGHT OF WET SOIL (gm.)		1527.40	1591.40	1644.40	1671.40	1723.40	1693.00	
WET DENSITY (gm./cm3)		1.62	1.69	1.74	1.77	1.83	1.79	
MOISTURE CONTENT : CONTAINER No.		B4	FA3	B1	D5	A1	GTZ22	
WEIGHT OF WET SOIL + CONTAINER (gm.)		55.00	49.00	61.30	51.20	44.00	41.00	
WEIGHT OF DRY SOIL + CONTAINER (gm.)		47.00	41.80	50.10	42.50	37.30	34.90	
WEIGHT OF CONTAINER (gm.)		21.10	21.20	20.30	19.70	20.70	20.20	
WEIGHT OF MOISTURE (gm.)		8.00	7.20	11.20	8.70	6.70	6.10	
WEIGHT OF DRY SOIL (gm.)		25.90	20.60	29.80	22.80	16.60	14.70	
MOISTURE CONTENT (%)		**30.89**	**34.95**	**37.58**	**38.16**	**40.36**	**41.50**	**20.9**
DRY DENSITY (gm./cm3)		**1.236**	**1.249**	**1.266**	**1.282**	**1.301**	**1.267**	

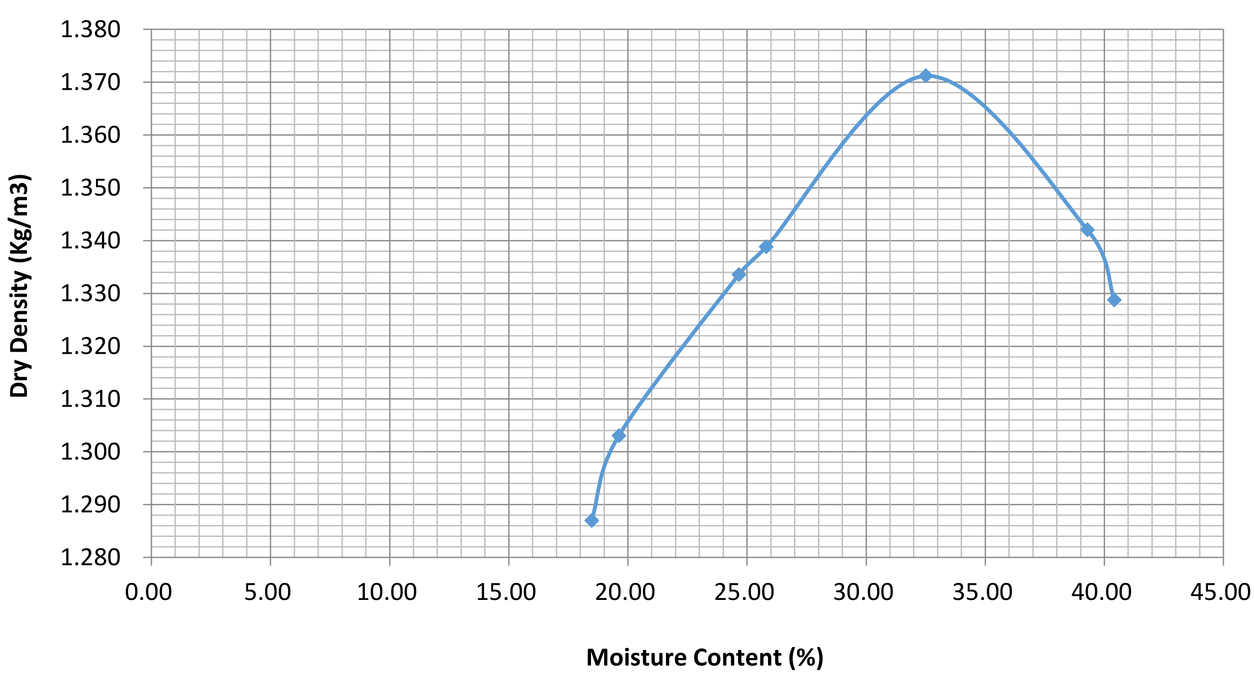
MDD= 1.301 gm./cm^3 OMC= 40.5%



3. 10% CHA

BLOWS PER LAYER	25	No. OF LAYERS		3	RAMMER WEIGHT			2.5	
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)		944					
WEIGHT OF WET SOIL + MOLD (gm.)		4981.00	5013.00	5111.00	5131.60	5256.70	5306.20	5303.00	**NMC**
WEIGHT OF MOLD (gm.)		3541.60	3541.60	3541.60	3541.60	3541.60	3541.60	3541.60	
WEIGHT OF WET SOIL (gm.)		1439.40	1471.40	1569.40	1590.00	1715.10	1764.60	1761.40	
WET DENSITY (gm./cm3)		1.52	1.56	1.66	1.68	1.82	1.87	1.87	
MOISTURE CONTENT : CONTAINER No.		DV-9	A3	DV-5	G2	G3	D5	DV-2	
WEIGHT OF WET SOIL + CONTAINER (gm.)		71.00	64.70	75.50	55.80	63.20	39.20	87.70	
WEIGHT OF DRY SOIL + CONTAINER (gm)		63.00	57.50	64.50	48.60	52.80	33.70	68.50	
WEIGHT OF CONTAINER (gm.)		19.70	20.80	19.90	20.70	20.80	19.70	21.00	
WEIGHT OF MOISTURE(gm.)		8.00	7.20	11.00	7.20	10.40	5.50	19.20	
WEIGHT OF DRY SOIL (gm.)		43.30	36.70	44.60	27.90	32.00	14.00	47.50	
MOISTURE CONTENT (%)		**18.48**	**19.62**	**24.66**	**25.81**	**32.50**	**39.29**	**40.42**	**14.20**
DRY DENSITY (gm./cm3)		**1.287**	**1.303**	**1.334**	**1.339**	**1.371**	**1.342**	**1.329**	

MDD=1.372 OMC=32.5

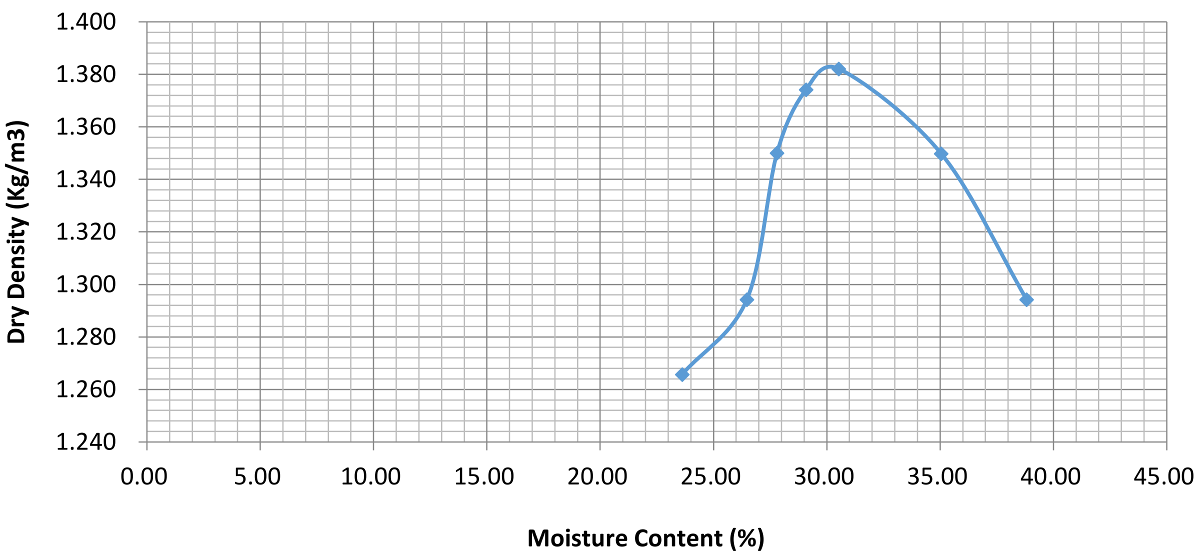


4. 15% CHA

BLOWS PER LAYER	25	No. OF LAYERS	3	RAMMER WEIGHT		2.5
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)	944			

WEIGHT OF WET SOIL + MOLD (gm.)	5018.00	5086.00	5169.70	5215.40	5244.00	5261.70	5236.90	**NMC**
WEIGHT OF MOLD (gm.)	3540.90	3540.90	3540.90	3540.90	3540.90	3540.90	3540.90	
WEIGHT OF WET SOIL (gm.)	1477.10	1545.10	1628.80	1674.50	1703.10	1720.80	1696.00	
WET DENSITY (gm./cm3)	1.56	1.64	1.73	1.77	1.80	1.82	1.80	
MOISTURE CONTENT : CONTAINER No.	A1	P23	A6	DV9	B1	G3	DV5	
WEIGHT OF WET SOIL + CONTAINER (gm.)	61.00	54.70	49.40	52.10	54.50	62.70	67.30	
WEIGHT OF DRY SOIL + CONTAINER (gm.)	53.30	47.50	43.20	44.80	46.50	51.80	54.10	
WEIGHT OF CONTAINER (gm.)	20.70	20.30	20.90	19.70	20.30	20.70	20.10	
WEIGHT OF MOISTURE (gm.)	7.70	7.20	6.20	7.30	8.00	10.90	13.20	
WEIGHT OF DRY SOIL (gm.)	32.60	27.20	22.30	25.10	26.20	31.10	34.00	
MOISTURE CONTENT (%)	**23.62**	**26.47**	**27.80**	**29.08**	**30.53**	**35.05**	**38.82**	**13.80**
DRY DENSITY (gm./cm3)	**1.266**	**1.294**	**1.350**	**1.374**	**1.382**	**1.350**	**1.294**	

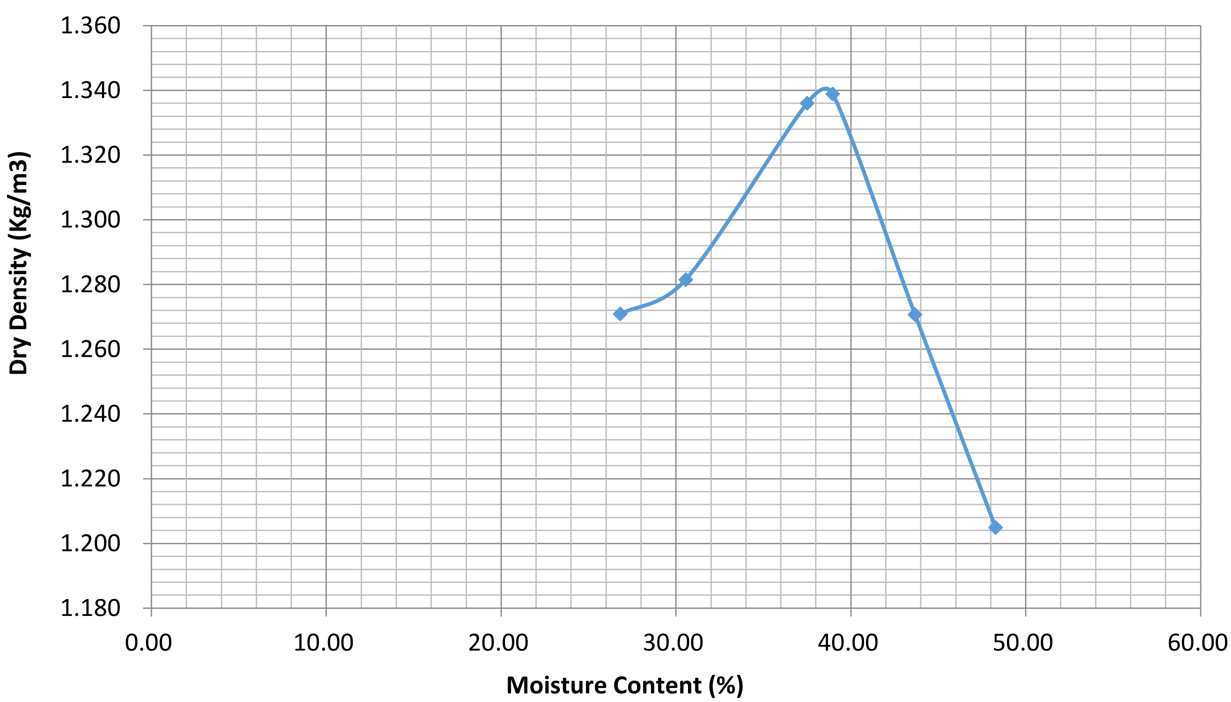
MDD= 1.384 gm/cm^3 OMC= 30%



5. 20% CHA

BLOWS PER LAYER	25	No. OF LAYERS	3	RAMMER WEIGHT			2.5
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)	944				
WEIGHT OF WET SOIL + MOLD (gm.)	5063.00	5121.00	5275.80	5298.00	5265.00	5228.20	**NMC**
WEIGHT OF MOLD (gm.)	3541.60	3541.60	3541.60	3541.60	3541.60	3541.60	
WEIGHT OF WET SOIL (gm.)	1521.40	1579.40	1734.20	1756.40	1723.40	1686.60	
WET DENSITY (gm./cm3)	1.61	1.67	1.84	1.86	1.83	1.79	
MOISTURE CONTENT : CONTAINER No.	G2	G8	Dr-A	D5	A3	D1	
WEIGHT OF WET SOIL + CONTAINER (gm.)	68.00	49.30	51.90	49.30	53.00	58.80	
WEIGHT OF DRY SOIL + CONTAINER (gm.)	58.00	42.70	43.20	41.00	43.00	46.20	
WEIGHT OF CONTAINER (gm.)	20.70	21.10	20.00	19.70	20.10	20.10	
WEIGHT OF MOISTURE (gm.)	10.00	6.60	8.70	8.30	10.00	12.60	
WEIGHT OF DRY SOIL (gm.)	37.30	21.60	23.20	21.30	22.90	26.10	
MOISTURE CONTENT (%)	**26.81**	**30.56**	**37.50**	**38.97**	**43.67**	**48.28**	**18.7**
DRY DENSITY (gm./cm3)	**1.271**	**1.282**	**1.336**	**1.339**	**1.271**	**1.205**	

MDD =1.34gm./cm3 OMC= 38.6%

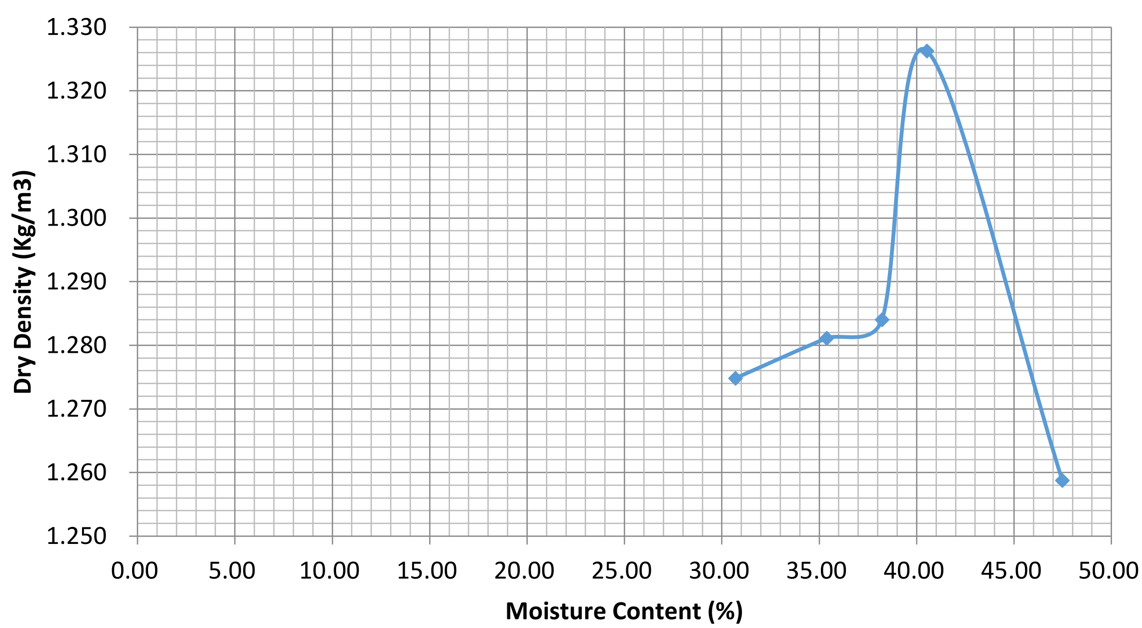


6. 25% CHA

BLOWS PER LAYER	25	No. OF LAYERS	3	RAMMER WEIGHT	2.5
MOLD DIAMETER	152mm	VOLUME OF MOLD (cm^3)	944		

WEIGHT OF WET SOIL + MOLD (gm)	5114.40	5178.90	5217.00	5301.00	5294.00	**NMC**
WEIGHT OF MOLD (gm)	3541.60	3541.60	3541.60	3541.60	3541.60	
WEIGHT OF WET SOIL (gm)	1572.80	1637.30	1675.40	1759.40	1752.40	
WET DENSITY (gm/cm3	1.67	1.73	1.77	1.86	1.86	
MOISTURE CONTENT : CONTAINER No.	G3	D5	A6	B1	GTZ22	
WEIGHT OF WET SOIL + CONTAINER (gm)	34.00	37.30	47.30	46.90	49.30	
WEIGHT OF DRY SOIL + CONTAINER	30.90	32.70	40.00	39.20	39.90	
WEIGHT OF CONTAINER (gm)	20.80	19.70	20.90	20.20	20.10	
WEIGHT OF MOISTURE (gm)	3.10	4.60	7.30	7.70	9.40	
WEIGHT OF DRY SOIL (gm)	10.10	13.00	19.10	19.00	19.80	
MOISTURE CONTENT (%)	**30.69**	**35.38**	**38.22**	**40.53**	**47.47**	**21.50**
DRY DENSITY (gm/cm3)	**1.275**	**1.281**	**1.284**	**1.326**	**1.259**	

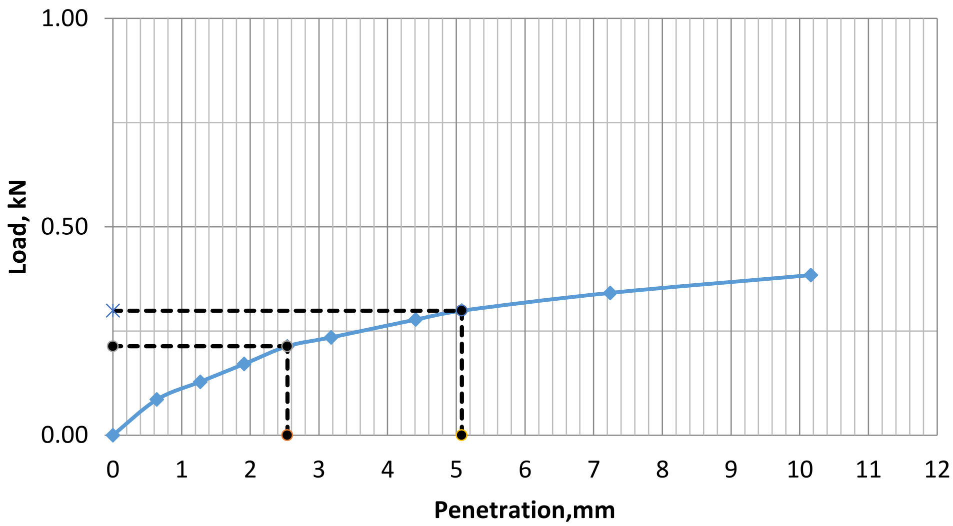
MDD=1.326 OMC= 40.5



Appendix 3

Soaked CBR Curves with Varying Percentages of CHA

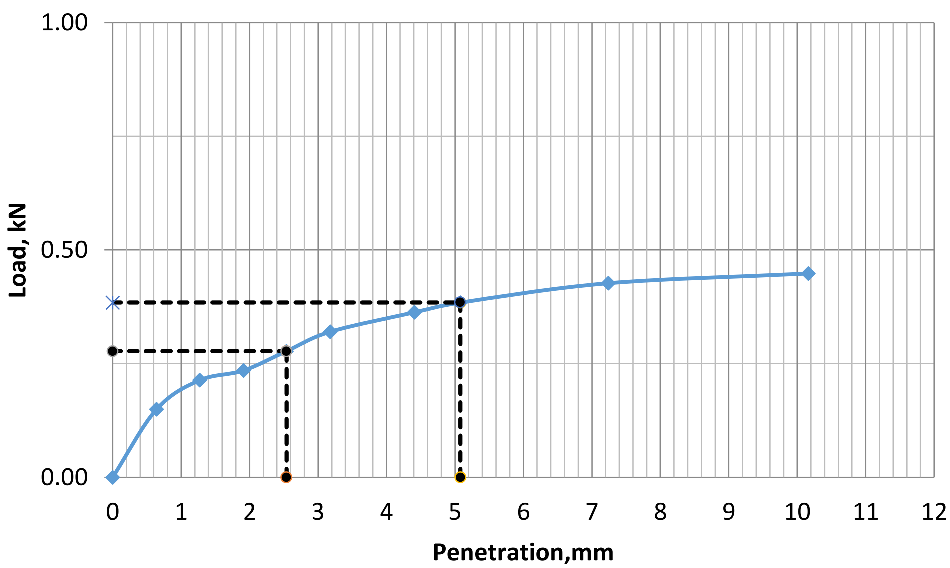
i. 0 %CHA (untreated soil)



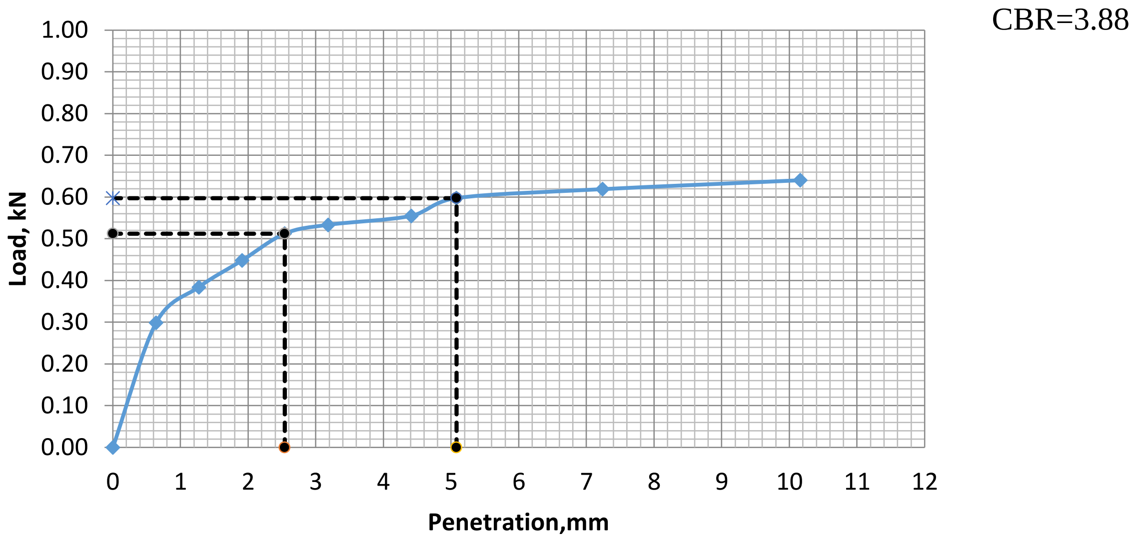
CBR=1.62

ii. 15 % CHA 0 day curing

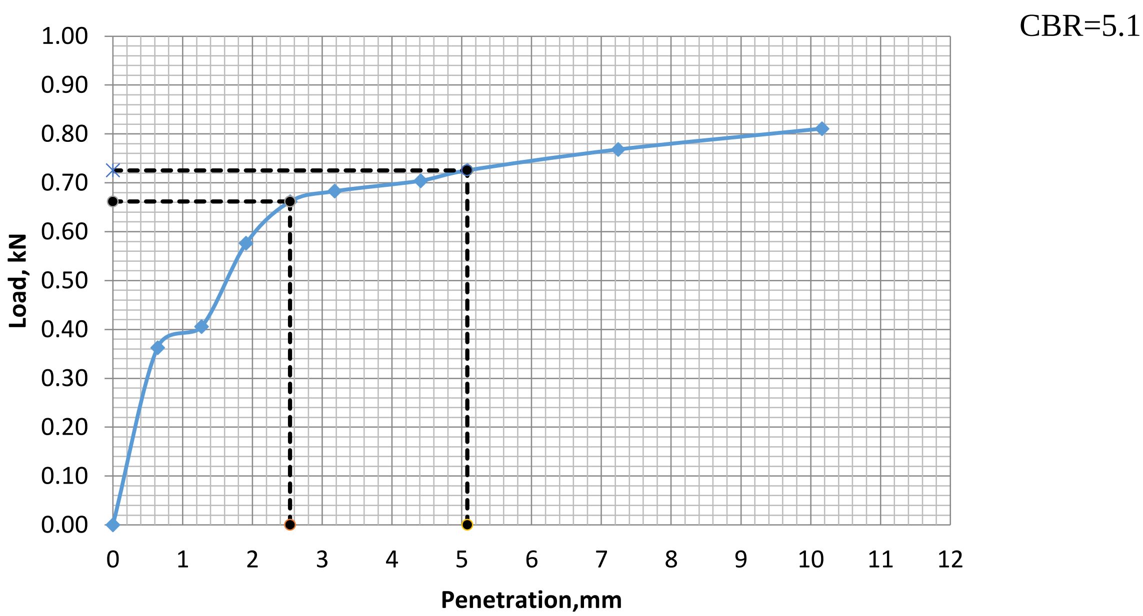
CBR=2.1



iii. 15% CHA 7 day curing



iv. 15% CHA 14 day curing

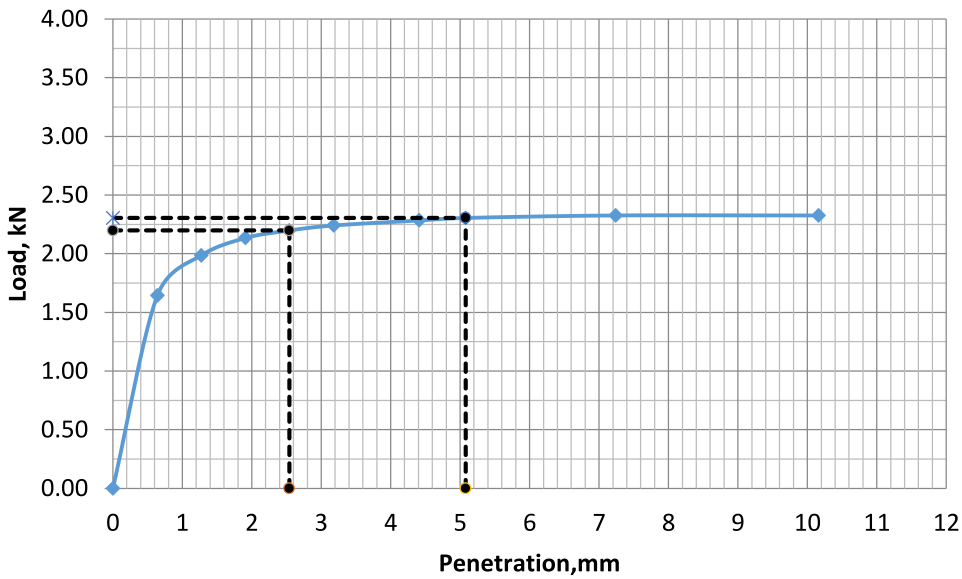


Appendix 4

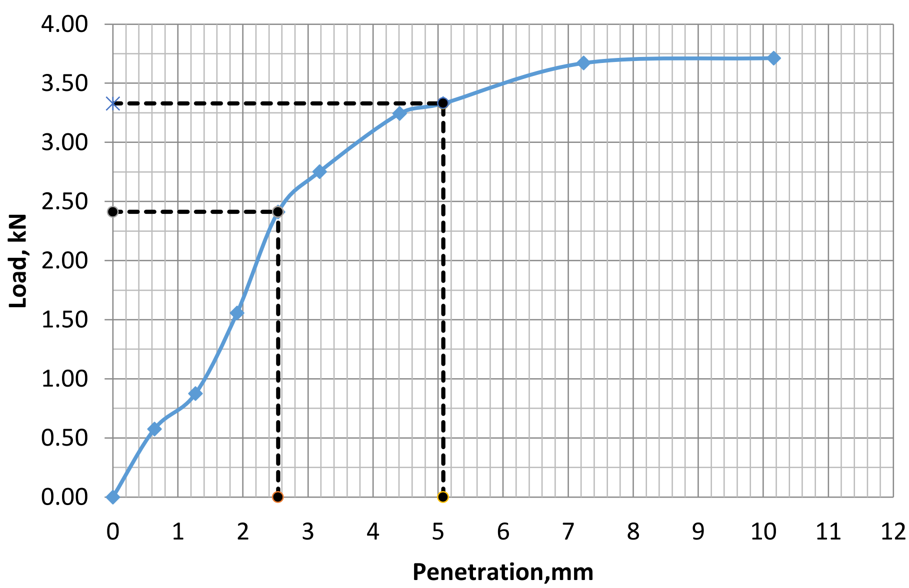
Un soaked CBR Curves with Varying Percentage of CHA

i. 0% CHA (untreated soil)

CBR=16.2

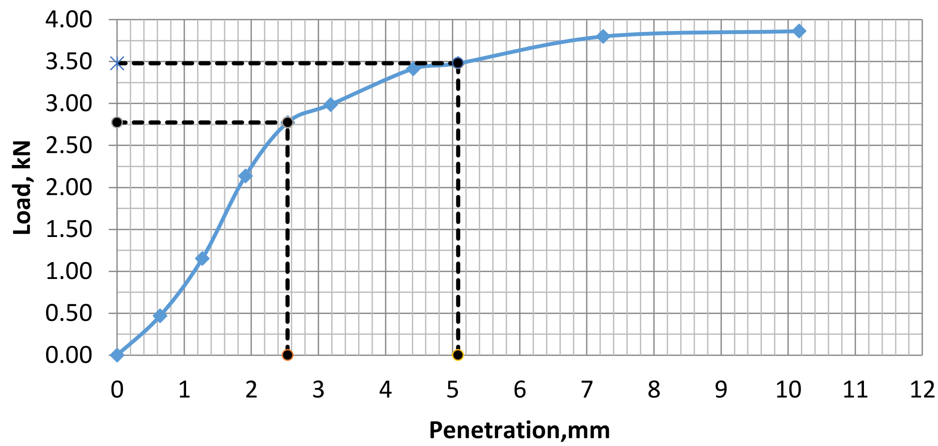


ii. 15% CHA 0 day curing



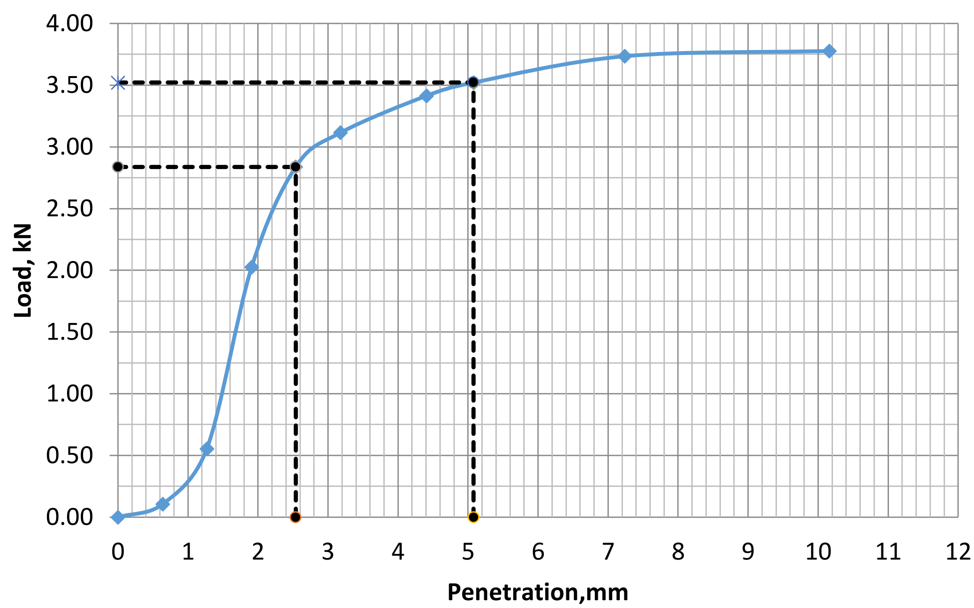
CBR=18.27

iii. 15% CHA 7 day curing



CBR=21

iv.15%CHA 14 day curing

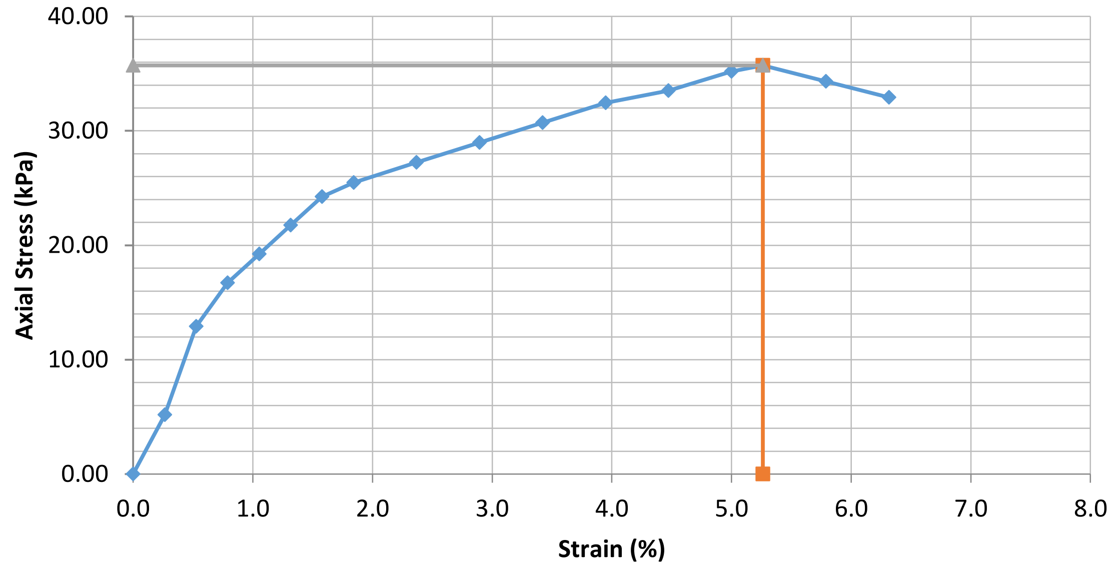


CBR=21.5

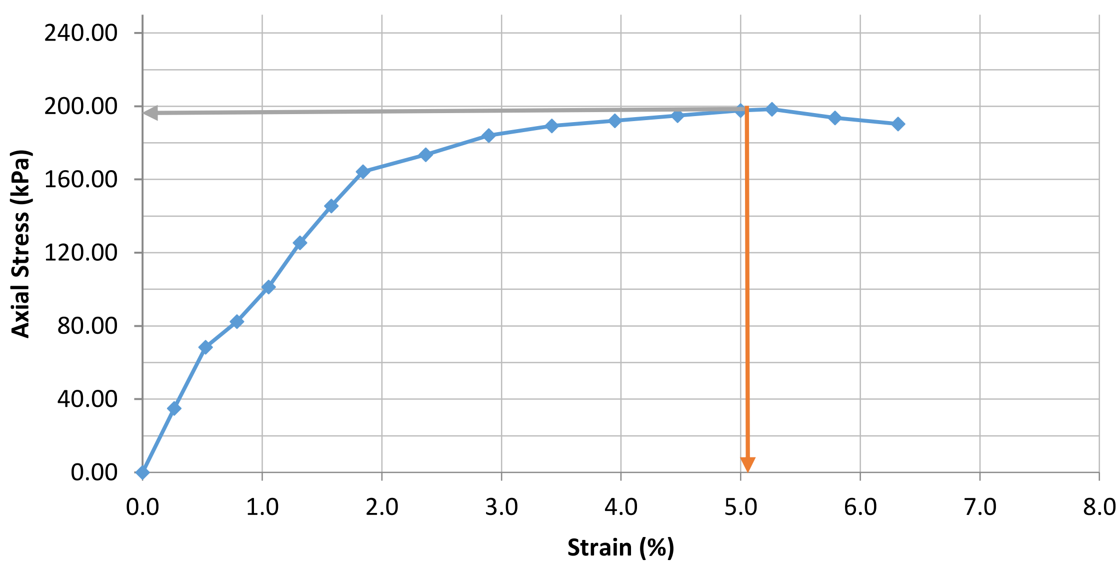
Appendix 5

UCS with Varying Percentages of CHA

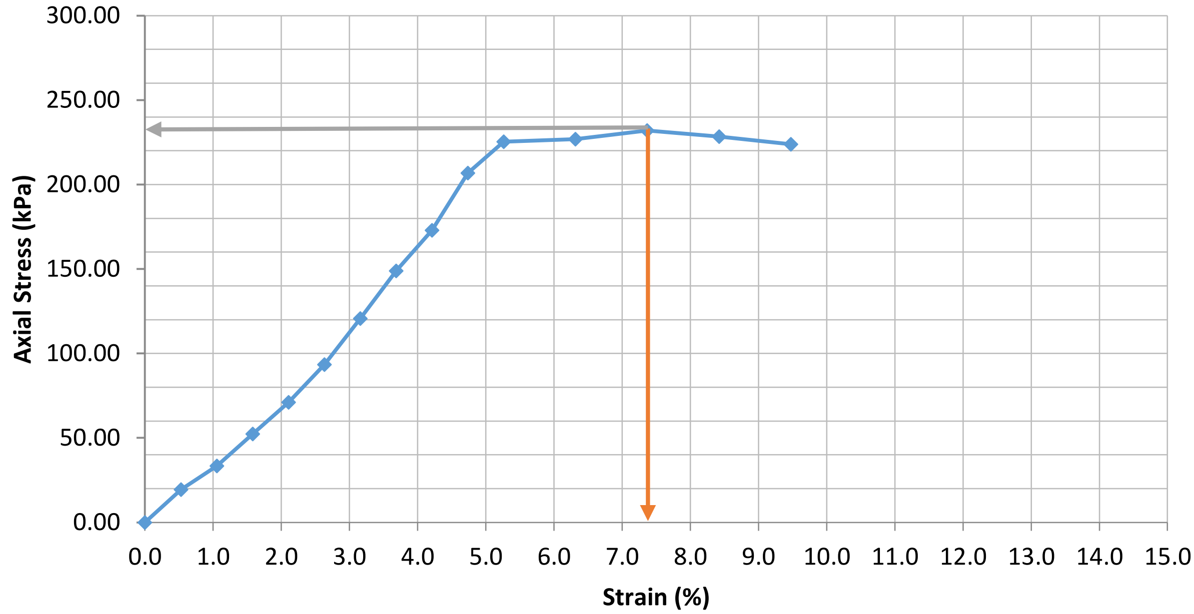
i. 0% CHA UCS=35.73 Kpa



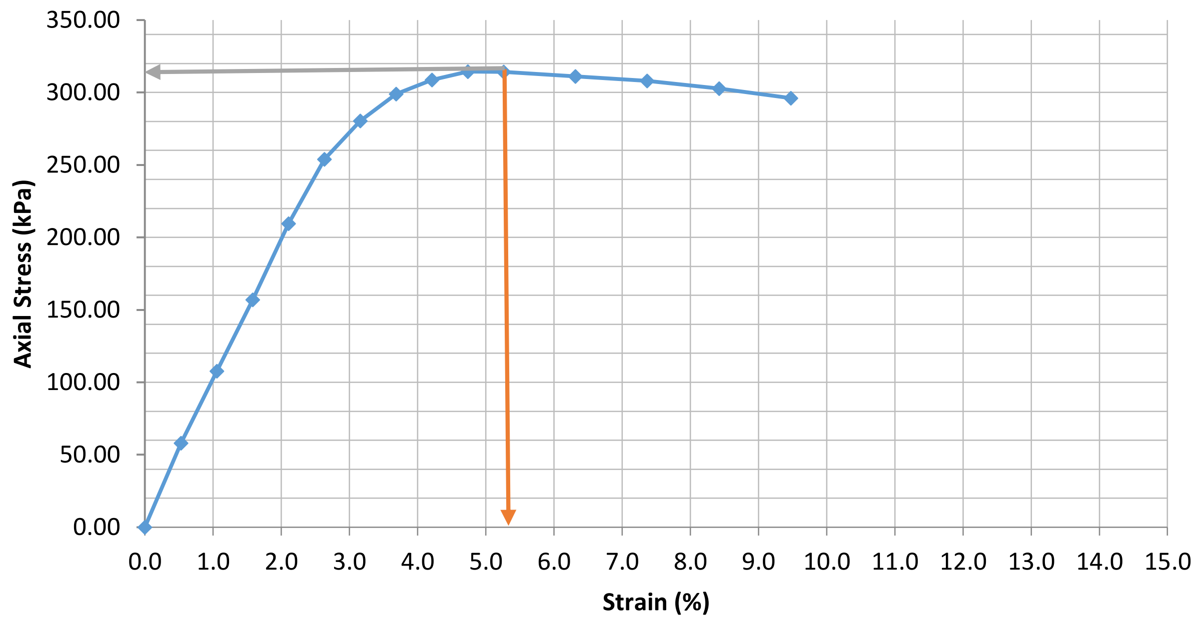
ii. 15% CHA and 0 day curing UCS=197.61Kpa



iii. 15% CHA and 7 day curing UCS=231.9Kpa



iv. 15% CHA and 14 day curing UCS=314Kpa



Appendix 6

Chemical Composition of CHA

	GEOLOGICAL SURVEY OF ETHIOPIA	**Doc.Number: GLD/F5.10.2**	**Version No: 1**
	GEOCHEMICAL LABORATORY DIRECTORATE		Page 1 of 1
Document Title:	**Complete Silicate Analysis Report**	**Effective date:**	**May, 2017**

Issue Date: - 09/05/2018

Customer Name:- Rehima Melke

Request No: GLD/TR/0271/18

Report No: GLD/TR/0290/18

Sample type: - coffe hask

Sample Preparation: - 200 Mesh

Date Submitted: - 24/04/2018

Number of Sample: One (1)

Analytical Result: In percent (%) Element to be determined Major Oxides & Minor Oxides

Analytical Method: LiBO2 FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS

Collector's code	SiO2	Al2O3	Fe2O3	CaO	MgO	Na2O	K2O	MnO	P2O5	TiO2	H2O	LOI
Good	3.14	<0.01	0.16	0.14	0.32	<0.01	0.96	<0.01	0.031	<0.01	8.00	88.73

Note: - This result represent only for the sample submitted to the laboratory.

Analysts	Checked By	Approved By	Quality Control
Tizita Zemene Dessie Abebe Tihitna Beletkachew Yohannes Getachew Nigist Fikadu	Tamiru Siraye	Gosa Haile	Awash Yirga

Geological Survey of Ethiopia Central Geological Laboratory