Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Chair

EFFECT OF MIXTURE OF CORNCOB ASH AND LIME ON THE ENGINEERING PROPERTIES OF BLACK COTTON SOIL FOUND IN JIMMA CITY.

A thesis submitted to school of graduate studies of Jimma University in partial fulfilment of the requirements for the Master's Degree of civil engineering in Geotechnical Engineering

By:

Sena Tesfaye

January, 2020

Jimma, Ethiopia

Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Chair

EFFECT OF MIXTURE OF CORNCOB ASH AND LIME ON THE ENGINEERING PROPERTIES OF BLACK COTTON SOIL FOUND IN JIMMA CITY.

A thesis submitted to school of graduate studies of Jimma University in partial fulfilment of the requirements for the Master's Degree of civil Engineering in Geotechnical Engineering

By:

Sena Tesfaye

Main Advisor: Prof. Emer T. Quezon, P.Eng (Ph.D.)

Co-Advisor: Mr. Habte T.(MSc)

January, 2020

Jimma, Ethiopia

Jimma University

Jimma Institute of Technology

School of Post Graduate Studies

Faculty of Civil and Environmental Engineering

Geotechnical Engineering Chair

EFFECT OF MIXTURE OF CORNCOB ASH AND LIME ON THE ENGINEERING PROPERTIES OF BLACK COTTON SOIL FOUND IN JIMMA CITY.

By:

Sena Tesfaye

APPROVED BY BOARD OF EXAMINERS

1. ______________________________ _____________ ____/_____/____
   External Examiner Signature Date
2. ______________________________ _____________ ____/_____/___
   Internal Examiner Signature Date
3. ______________________________ _____________ ____/_____/____
   Chairman of Examiner Signature Date
4. Prof. Emer T. Quezon, P.Eng _____________ 15/11/2019
   Main Advisor Signature Date
5. Mr. Habte T., (MSc.) _____________ ____/_____/____
   Co- Advisor Signature Date

DECLARATION

I, the undersigned, declare that this thesis entitled: "**Effect of Mixture of Corncob Ash And Lime on The Engineering Properties of Black Cotton Soil Found In Jimma City.**" is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for this thesis proposal have to be duly acknowledged.

Candidate:

Mr. Sena Tesfaye

____________________ ______________

Signature Date

As Master's Research Advisors, I hereby certify that I have read and evaluated this MSc Thesis prepared under my guidance by **Sena Tesfaye** entitled: "**Effect of Mixture of Corncob Ash And Lime on The Engineering Properties of Black Cotton Soil Found In Jimma City.**"

		15/11/2019
Prof. Emer T. Quezon, P.Eng	__________________	___________
Main Advisor	Signature	Date
Mr. Habte T., MSc.	__________________	___________
Co- Advisor	Signature	Date

ABSTRACT

Many of the soils of Ethiopia are covered by expansive soil, and are difficult to use in the construction of highways and airfields because of their high content of plastic clay and their expansive tendencies. The existence of expansive soil in Jimma area has caused damage to the light building, asphalt pavement, and buried utility lines. Several methods have been developed successfully to use pozzolanic materials such as Portland cement, lime, fly-ash, bitumen, and polymers for high strength concrete, soil improvement, and other civil engineering works. The over-dependence on the utilization of industrially manufactured soil-improving additives cement and lime has kept the cost of soil stabilization very high. These studies mainly focused on determining the effect of the mixture of corncob ash with lime on the Engineering properties of black cotton(BC) soil. Due to different soil problems, searching for the best soil stabilizer to overcome problems that occur by the BC soils is still being the main concern to achieve the required soil engineering properties. In order to address the aforementioned purposes, two subgrade soils were collected from Jimma City around Mendera Kochi(Green Sefer) and Merkato Sefer around the Police station. The soil samples collected from a local area within a depth of 1.5-2m. The laboratory tests which were carried out Moisture content, Specific Gravity, Grain size Analysis, Liquid limit test, plastic limit, plasticity index, permeability, UCS, CBR, swelling index, compaction, XRF and XRD. The samples tested on black cotton clay, black cotton clay+ % corncob ash, Black cotton clay + % corncob ash + % lime using different apparatus as per the procedures laid down in American standard of testing materials and Ethiopian road authority manuals. The data processing and analyzing techniques used were both descriptive and analytical methods. The test results indicated that the Natural sub-grade soils are A-7-5 as per the AASHTO soil classification system and CH as per USCS. As far as the engineering properties of natural subgrade soils were studied, the two soils were highly Swelling clay soils. The two soils have almost similar Engineering properties. Sample-1 has PI of 66.28 %, FSI of 100%, CBR of 1.24%, MDD of 1.54% and OMC of 24.5%. Likewise, Sample-2 also has PI of 66.58 %, FSI 87%, CBR of 1.54%, MDD of 1.57%, and OMC of 24%, and both samples Categorize as Illite, and smecitite minerollogically. As the amount of Lime ratio increases LL, PI, OMC, FSI, CBR swell decreased whereas PL, MDD, CBR are increased, rather than Corncob ash alone. But the amount of Lime decreasing with increasing CCA ash beyond 2% the results were decreased as the respective test. For this study, 2% CCA + 6% LIME was found out to be the optimum ratio which achieved by most of the geotechnical parameters in the study. The XRD results indicated a general reduction in peak intensities in all clay minerals that are present in the soil and, most significantly, in montmorillonite. Finally, Microscopic Analysis using SEM & EDX should be taken to show the presence of C-S-H & C-S-A-H in both CCA and Lime treated clay soils.
Keywords: *Black Cotton soil, CBR, Compaction, C-S-H, Engineering Properties, SEM*

ACKNOWLEDGMENT

I am deeply indebted to my thesis advisors, Prof. Emer T. Quezon and Mr. Habte T., who provided immense help and encouragement. Working with them, I enriched my scientific knowledge in general. I am also very grateful for their critical comments. I learned a lot from them when discussing the Expansive soil behavior.

I would like to thank my colleagues in the Group of Geotechnical Engineering Chair, to Mr. Haile and Ditta for their care and hospitality. Special thanks to my fellow Msc students, Temesgen Petros, Bruk Amde, Asmamawu Balcha, Ahmed Simeneh, Siflet Nigusse, Amente Assefa, Adugna lelisa, for all the great moments we had together.

I also give my gratitude to Wollega University who provided me with a full scholarship during two and a half years that guaranteed a smooth completeness of my thesis.

I would also like to thank my friends for being my reviewers, and the rest of my family for their continued support throughout all of my years of staying in the postgraduate class.

Last but not least, I am grateful to my parents, without their love, care and continuous support, this thesis would never be finished.

TABLE OF CONTENT

Contents Page No.

LIST OF TABLE

LIST OF FIGURES

LIST OF ACRONYM AND ABBREVIATIONS

AASHTO	American Association State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	Californian bearing Ratio
CCA	Corn Cob Ash
CSH	Calcium Silicate Hydrates
ERA	Ethiopian Roads Authority
FAO	Food and Agriculture Organization
FSI	Free Swell Index
IS	Indian Standard
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
SL	Shrinkage Limit
UCS	Unconfined compressive strength
USCS	Unified Soil Classification System
XRD	X-Ray Diffraction
XRF	X-Ray Flourecence

1. INTRODUCTION

1.1 Background of the study

Soil is a broad term utilized in engineering applications which includes all deposits of loose material on the earth's crust that is created by weathering and erosion of underlying rocks. Soil is one of the most encountered materials in Civil Engineering and is used in many Engineering Structures such as embankments, dams, and pavements. Thus, the behavior of the soil at the location of any project and the interactions of the earth materials during and after the construction of the facility has a major influence on the success, economy, and safety of the work [1]. However, not all naturally occurring materials are appropriate for construction. Problematic soils, such as expansive clay cause major problems in the design, construction, and maintenance of pavements.

Expansive soils are a worldwide problem posing many challenges to Civil Engineers, Construction Firms and owners. The damages from expansive soils are minor maintenance and aesthetic concerns, but often they are much worse, causing major structural distress. They are considered as a potential natural hazard that can cause extensive damage to engineering structures. Expansive soils exist in many parts of soils around the world, but particularly in arid and semiarid regions. Thus, an expansive soil exhibits high swelling, shrinkage and plasticity characteristics. Several methods have been developed successfully to use pozzolanic materials such as Portland cement, lime, fly-ash, bitumen and polymers for high strength concrete, soil improvement, and other civil engineering works. Over the times, these materials have rapidly increased in price due to the sharp increase in the cost of energy since the 1970s [2]. The over-dependence on the utilization of industrially manufactured soil improving additives cement and lime have kept the cost of soil stabilization very high. Cohesive clayey soils are generally poor materials for foundations and involve pretreatment prior to construction [2].

Considerable research has been done on stabilization of these soils using CCA as a viable alternative of conventional pozzolans. The application of agro-wastes as a soil stabilizing agent for cost-effective and environmentally safe construction practices is desperately needed in agriculture-based developing countries like Ethiopia. The benefit from applying

Corncob ash for soil stabilisations with lime is related to chemical reaction between calcium hydroxide produced by lime with pozzolan that is supplied from CCA.

Similar to cement reaction with soil this chemical reaction can be explained by two individual processes: (1) short term reaction, consisting of cation exchange and flocculation as a result of the reaction between clay, CCA and lime; and (2) long-term reaction, involving time and temperature dependent pozzolanic activity, in which new cementations compounds-calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH) responsible for long-term strength in soils are produced.

However, no research is available to date on the potential of locally produced CCA with Lime to improve problematic soils/clayey in Ethiopia and also the mineralogical characters of CCA stabilized BCS remain unknown up to now.

1.2 Statement of the problem

Expansive soil is widely spread in the world. Expansive soil is called the 'hidden hazard,' and it was reported that the economic losses caused by expansive soil amount to $2.3 billion per year in 1973, far more than that from the total losses caused by floods, earthquakes, and windstorms put together in the U.S.A. In Japan expansive soil is called 'problem soil,' for it often brings about foundation deformation and mud pumping of many roadbeds, the heaving of tunnel arches and landslides in embankments, etc. [3]. Especially in the African continent such as South Africa, Ethiopia, Nigeria, Kenya, Mozambique, Morocco, etc. Many of the soils of Ethiopia is covered by BC soil and are difficult to use in the construction of highways and airfields because of their high content of plastic clay and their expansive tendencies [4].

Due to economic growth in the country, rapid development is occurring in the field of transportation, especially the road sector in our country. Roads are vital to link our communities and sustain the economy and quality of life in society. Roads constructed over the expansive soil observed with high maintenance expenditure in spite of high capital cost [5]. Expansive soils occurring above water table undergo volumetric changes with changes in water content. Increase in moisture content causes the following effects:

- ✓ Swell - Shrink Characteristics
- ✓ Horizontal Thrust
- ✓ Creep and Landslide
- ✓ Typical Structural Distress Patterns
- ✓ Differential settelment

The existence of expansive soil in Jimma area has caused damage to the light building, asphalt pavement, and buried utility lines. For construction, maintenance and widening of roads, the huge quantity of construction materials are needed. However, due to some reasons such as depleting resources, environmental concerns, etc., there is a scarcity of conventional local materials such as coarse and fine aggregates for the construction of the subbase and base layers of Asphalt pavement projects. CCA is an abundant waste product obtained from maize, According to the Food and Agriculture Organization (FAO) data, 589 million tons of maize were produced worldwide in the year 2000 (FAO Records; 2002). Maize production of Ethiopia increased from 2.34 million tonnes in 1998 to 8.12 million tonnes in 2017 were growing at an average annual rate of 7.57 %, and Ethiopia is 15th in producing Corn from the world and third in Africa only preceded by South Africa and Nigeria. Since CCA are renewable, biodegradable, environmentally friendly, low density and low-cost materials that have similarly physical and mechanical properties of other natural pozzolanic materials has the highly potential application in the area of construction as building materials and for soil improvement in support of subgrade beneath pavements and roads. However, very few studies on CCA stabilizing problematic soil have been performed so far; therefore, more investigations are essential in order to comprehend the engineering properties of CCA improving BCS soil in means of the individual or combination of quick lime and CCA adopted in ground improvement method.

As expected, such expansive soil improvement method aims to achieve these two main objectives simultaneously consisting of curtailing industrial waste by-product of CCA in line with diminishing quick lime dosage if the application of CCA, quick lime- CCA combination treated expansive soils can result in the appreciable improvement of shrink-swell behaviour and mechanical properties.

Consequently, the cost of good quality natural materials is increasing and they need to be hauled from distant quarries to the project site [6]. Engineers and urban planners should have to consider different soil stabilization and used the most limited resources of the city with zero carbon industry or green technology.

1.3 Research Question

The following are the research question which the study are based on.

1. What are the pozzolanic properties of corncob ash?
2. What are the Engineering properties of black cotton soil?
3. Does corncob ash and lime affect the Engineering properties of BCS ?
4. What is the appropriate percentage of CCA or lime that should be added to improve the geotechnical properties of black cotton soil?

1.4 Objective of the Study

1.4.1 General objective

The general objective of the research study is to determine the effect of the mixture of corn cob ash and lime on the Engineering properties of black cotton soil.

1.4.2 Specific objective

- To determine the pozzolanic property of corncobash, according to the ASTM standards.
- To determine the Engineering properties of black cotton soil.
- To investigate the impact of the mixture on the Engineering properties of black cotton soil.
- To recommend the appropriate percentages of corn cob and lime on the improvement of black cotton soils.

1.5 Significance of the Study

Searching for the best soil stabilizer to overcome problems occur by the Expansive or BC soils are still being the main concern, not only to achieve the required soil engineering properties but also by considering the cost and the effect to the environment.

Lime has been used effectively to improve the engineering properties of some local soils materials for construction of stabilized pavement layers, stabilized earth and support layer for the foundation of buildings. However, Lime is expensive, and its use is unsustainable, requiring the search for alternative materials for its partial replacement if not totally replace. This study seeks to provide experimental insights on the engineering properties of BCS stabilized with mixtures of Lime and corncob ash (CCA) to find out its suitability for use as a foundation, embankement, subgrade and base material

1.6 Scope and Limitation of the Study

In order to address the aforementioned purposes,two sample were taken from the city at location where expansive soil where found following different literature footprint and simple test or field survey. The sites for the test pits are to be selected systematically and randomly. Undisturbed and disturbed samples would be extracted from each test pit for a laboratory test. the depth of the pits varied from 1.5m to 2m depend on the water table effect. The experiment was limited to the sample found in sub area in Jimma city, due to the cost and time already set for the paper. Limited number of tests are conducted only to achieve the objectives of the present study

2. REVIEW OF RELATED LITERATURE

2.1 Black Cotton Soil

Black cotton soil is a term usually applied to any soil that has a potential for shrinking or swelling due to changes in its moisture content. Expansive soils owe their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil [7]. Expansive soils are clayey soils, mudstones or shales that are characterized by their potential for volume change on drying and/or wetting. According to [8] the clay content is relatively high, and the clay mineral montmorillonite dominates. They are characterized by their high strength when dry; very low strength when wet; wide and deep shrinkage cracks in the dry season; high plasticity and very poor trafficability when welled. Whenever insufficient attention is given to the deleterious properties of expansive soils, the results will be premature pavement failure evidenced by undulations, cracks, potholes, and heave.

Methods were developed for the identification and classification of expansive soils both locally and worldwide. And also Potentially expansive soils can typically be recognized in the lab by their elastic properties[7]. Inorganic clays of high plasticity, generally those with liquid limits exceeding 50% and plasticity index over 30, usually have the high inherent swelling property. Expansion of soils can also be measured in the laboratory directly, by immersing a remolded soil sample and measuring its volume change.

Studies of the mechanical and engineering behaviors of expansive soil have been emphasized all over the world because of their wide distribution and serious harm. Expansive soil is called the ‘hidden hazard,' and it was reported that the economic losses caused by expansive soil amount to $2.3 Billion per year in 1973, far more than that from the total losses caused by floods, earthquakes, and windstorms put together in the U.S.A [9]. In Japan expansive soil is called ‘problem soil,' for it often brings about foundation deformation and mud pumping of many roadbeds, the heaving of tunnel arches and landslides in embankments, etc. [3].

The term bentonite is often used in reference to expansive soils. This term refers to clays that are rich in montmorillonite. Bentonite is a highly plastic, swelling clay material containing primarily montmorillonite [10].

2.2 Engineering Mineralogy of the Expansive clay

Expansive soils have very high clay content. There are three main types of clay minerals. The parent materials associated with expansive soils according to [11] are either basic igneous rock or sedimentary rock. These are basic igneous rocks are Basalts, Dolomites, Dykes, Gabbro and Norris, and the Sedimentary rocks are Marls, Limestone, and shales. The Mineralogy of these clay is three common types of clay minerals.

2.2.1 Kaolinite

Kaolinite has a structural unit made up of aluminum sheets joined to silica sheet and is symbolized as indicated in the fig 2.1 below. The bond that exists between layers is tight, and it is difficult to separate the layers. As a result, Kaolinite is relatively stable, and water is unable to enter into or between the layers. Consequently, Kaolinite shows a low degree of expansiveness.

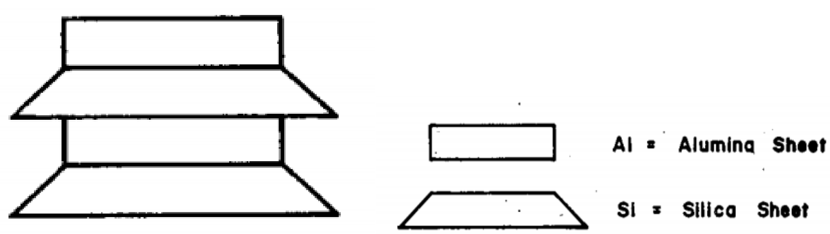


Fig 2.1 Structural unit of Kaolinite made of aluminum and silica sheet

2.2.2 Illite

It has a basic structure similar to that of montmorillonite fig 2.3. However, the basic illite units are bonded together by potassium ions which are non-exchangeable because of this,the illite units are reasonably stable and so that minerals swell much less than montmorillonite.

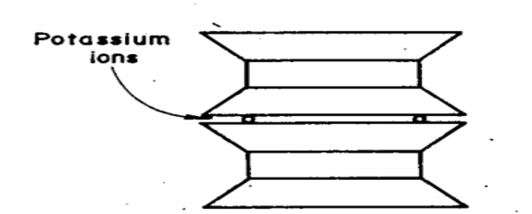


Fig 2.2 Structural unit of Illite made of aluminum and silica sheet bonded by potassium ion

2.2.3 Montmorillonite

It is the most common of all the clay mineral and is well known for its swelling properties. Its basic structure consists of an aluminum sheet sandwiched between two silica sheet and symbolically presents as fig 2.3. 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. It is extremely active, and its activity decreases as the absorbed cation exchanges in the following order.sodium (most active), lithium, potassium, calcium, magnesium, and hydrology(most stable). As a result of the exchange of absorbed cation of montmorillonite by these listed above, its activity is reduced.

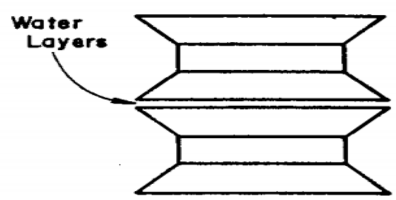


Fig 2.3 Structural unit of Montmorillonite made of aluminum and silica sheet bonded by weak bond

Montmorillonite is the most expansive type of clay mineral, and its structural formula is $Al_4Si_8O_{20}(OH)_4n(H_2O)$ [3]. Expansive soil is widely distributed in the globe and is found in more than 40 countries and regions [3] [12]. Ethiopia is one of the countries with a large distribution of expansive soil, which has been discovered in more than Half of its regions [4].

Making use of this fact, there are different additive materials used till now to stabilize the clay such as lime, cement and gypsum and in the process of which the active sodium ions are replaced by less active calcium ions.

2.3 Identification Of Expansive Soils

The identifcation methods used to identify the swell potential of expansive soils can generally be grouped into two categories. The first category mainly involves measurement of physical properties of soils, such as Atterberg limits, free swell, and potential volume change. The second category involves measurement of mineralogical and chemical properties of soils, such as clay content, cation exchange capacity, and specifc surface area [10].

Practicing geotechnical engineers typically use only the measurement of physical properties to identify expansive soils. However, the measurement of mineralogical and chemical properties is used routinely by agricultural and geological practitioners and should not be disregarded by the engineering community.

2.3.1 Methods Based on Physical Properties

2.3.1.1 Methods Based on Plasticity

Atterberg limits are commonly used to characterize soils and are used in some methods to identify expansive soils. Two indices defined on the basis of the Atterberg limits are the plasticity index, PI, and the liquidity index, LI. Many expansive soil identifcation methods make use of one or both of these indices. Peck, Hanson, and Thornburn (1974) suggested that there is a general relationship between the plasticity index of a soil and the potential for expansion, as shown in Table 2.1. However, Zapata et al

Table 2.1 Expansion Potential of Soils and Plasticity Index (Peck, Hanson, and Thornburn 1974)

Plasticity Index (%)	Expansion potential
0-15	Low
0-35	Medium
35-55	High
>55	Very High

showed that the expansion potential for remolded expansive soils correlates poorly to the plasticity index alone. They concluded that the correlation is significantly improved by correlating expansion potential with the product of plasticity index and percentage passing the No. 200 (75 μm) sieve. It is important to keep in mind that although the plasticity of a soil may be an indicator of expansive minerals, that in itself is not defnitive identification of an expansive soil. Atterberg limits and clay content can be combined into a parameter called activity, Ac.

This term was defned by Skempton (1953) as

$$\text{Activity}(Ac) = \frac{Plasticity\ Index}{\%\ \text{by weight fner than } 2\mu\text{m}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}1)$$

Skempton suggested three classes of clays according to activity. The suggested classes are "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. Typical values of activities for various clay minerals are shown in Table 2.2. Sodium montmorillonite has the most expansion potential, which is reflected by the extraordinarily high value of activity in Table 2.2.

Table 2.2 Typical Activity Values for Clay Minerals (Skempton 1953)

Mineral	Activity
Kalonite	0.33-0.46
Illite	0.9
Montmorillonite (Ca)	1.5
Montmorillonite (Na)	7.2

2.3.1.2 Free Swell Test

The free swell test consists of placing a known volume of dry soil passing the No. 40 (425 µm) sieve into a graduated cylinder flled with water and measuring the swelled volume after it has completely settled. The free swell of the soil is determined as the ratio of the change in volume from the dry state to the wet state over the initial volume, expressed as a percentage. A high-grade commercial bentonite (sodium montmorillonite) will have a free swell value from 1,200 to 2,000 percent.

Holtz and Gibbs (1956) stated that soils having free swell values as low as 100 percent may exhibit considerable expansion in the feld when wetted under light loading. Also, Dawson (1953) reported that several Texas clays with free swell values in the range of 50 percent have caused considerable damage due to expansion. This was due to extreme climatic conditions in combination with the expansion characteristics of the soil. Because of its simplicity and ease of operation, the free swell test is used as the sole swell potential index in the Chinese Technical Code for Building in Expansive Soil Areas (CMC 2003). However, even for the same type of soil, the results of the test can be influenced signifcantly by many factors, such as the amount of soil tested, degree of soil grinding, and drop height of the soil sample.

The Bureau of Indian Standards (1997) 2720 Part 40 uses the free swell index (FSI) method to indirectly estimate swell potential of expansive soils. In this test, two oven-dried soil specimens are each poured into a graduated cylinder. One cylinder is filled with kerosene oil and the other with distilled water. Both of the samples are stirred and left undisturbed for a minimum of 24 hours after which the final volumes of soils in the cylinders are noted.

The FSI is calculated as,

$$\text{FSI}= \frac{(\text{soil volume in water} - \text{soil volume in kerosene})}{\text{soil volume in kerosene}} \times 100\% \ldots\ldots\ldots\ldots\ldots \quad (2\text{-}2)$$

The expansion potential of the soil as classifed according to the FSI is shown in Table 2.3.

Table 2.3 Expansion Potential Based on Free Swell Index

Free Swell Index (FSI)	Expansion Potential
< 20	Low
20-35	Medium
35-50	High
> 50	Very High

2.3.1.3 Potential Volume Change (PVC)

The potential volume change (PVC) method was developed by T. W. Lambe (1960) for the Federal Housing Administration. It has been used by many State Highway Departments as well as some geotechnical engineers. The PVC test consists of placing a remolded soil sample into an oedometer ring. The sample is then wetted in the device and allowed to swell against a proving ring. The swell index is reported as the pressure on the ring and is correlated to qualitative ranges of potential volume change using the chart.

The advantage of the test is its simplicity. The disadvantage is that the stiffness of the proving ring is not standardized, there by allowing for different amounts of swelling to take place, depending on the stiffness of the proving ring. The swelling pressure that is developed will vary with the amount of swelling that the proving ring allows. Because the test uses remolded samples, the swell index and PVC values are more useful for identifcation of potential expansive behavior and should not be used as design parameters for undisturbed in situ soils

2.4 Geographic Distribution of Expansive Soils

Expansive soils deposits and problems associated with heaving soils have been reported on six continents and in more than 40 countries worldwide. Figure 2.1 shows the global distribution of reported expansive soil sites[13] [10].

In North America, mapping of expansive soils has been carried out by a number of investigators using geologic and agricultural soil maps and soil reports [10] . The map shows areas where expansive soils have been encountered, but it must be emphasized that expansive soils are not limited to those areas. Expansive soils are more prominent in the western and southern part of North America. The most severe problems occur in the western parts of the United States and Canada, where problems are primarily attributed to highly overconsolidated claystone and clayshale.

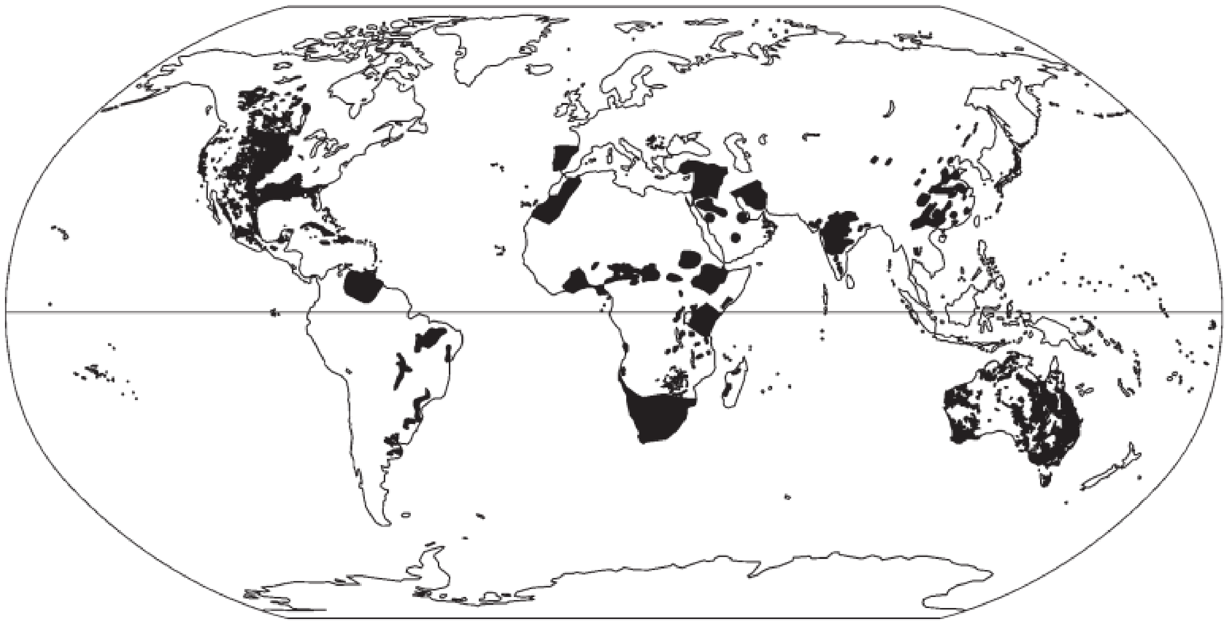


Fig 2.4. Global distribution of reported expansive soil sites [10]

2.4.1 Distribution of Expansive soil in Ethiopia

Expansive soil is widely spread in the world. Especially in the African continent such as South Africa, Ethiopia, Nigeria, Kenya, Mozambique, Morocco, etc. [11]. In Ethiopia, these are characteristics of the black clays (commonly known as "black cotton soils"), found in the central highlands and some other areas. However, red clays in the wetter regions of Ethiopia central and western highlands also show the nature of having high plasticity[11] [14]. The aerial coverage of expansive soils in Ethiopia is estimated to be 23.7 million acres as specified by Nebro. According to [15] Jimma expansive soils show similar properties with that of expansive soils found in Ethiopia.

Distribution of expansive soil is generally a result of geological history, sedimentation and local climatic conditions. Arid climatic conditions and severe weathering environment prevailing in north eastern part of Africa promote the widespread occurrence of expansive soils in this region. In Ethiopia, covering nearly 40% surface area of the country, expansive soils are observed in area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berehan, Addis Ababa - Gohatsion, Addis Ababa - Mojo. Also the cover the area like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction are being carried out [4], [15]. And Also on Table 2.2 Comparison of property ranges of Jimma expansive soils with other expansive soils found in Ethiopia [15].

Table 2.4 Comparison of property ranges of Jimma expansive soils with other expansive soils found in Ethiopia [15]

Item	**Location**			
	Jimma (current study, [2])	Addis Ababa* (Teklu et al.[23])	Bahr Dar [24]	Mekelle [23]
Sand, %	1-7	2 – 16	0.61- 17.1	3.8-19.2
Silt, %	42-51	9 - 46.40	10.23- 26.88	34.8-69.5
Clay, %	40-59	30 – 70	58.1-87	20.8-60.2
Liquid Limit, %	72-108	63 -108	78.5-112.05	48.6-89.7
Plasticity Index, %	36-68	45 - 76.4	46.46- 6.42	25.1-70.6
Specific Gravity, Gs	2.58-2.72	2.77-2.88	2.55-2.81	2.4-2.78
Free swell, %	80-160	51.8- 118	78-200	22-127
Swelling Pressure, kPa	135-210	36.9-420	80-520	50.2-262.9
UCS, kPa	85-285.6	96.7-267.2		352.4-565.9

2.4.2 Damages Due to Expansive Soil

Roads are vital to link our communities and sustain the economy and quality of life in society. Roads constructed over the expansive soil observed with high maintenance expenditure in spite of high capital cost [5]. These Problems Associated with Expansive Soils is very sensitive. Expansive soils occurring above water table undergo volumetric changes with changes in water content. Increase in moisture content causes the following effects:

1. Swell - Shrink Characteristics- This causes significant volume changes resulting severe damage to the foundations, buildings, roads retaining walls, canal linings, etc [16].

2. Horizontal Thrust- Increased water content in the soils adjacent to the foundation wall will cause the soils to expand and increase the lateral pressure applied to the foundation wall and it will cause minor cracking, bowing or movement of the wall and serious structural damage to or failure of the wall may occur [3].

3. Creep and Landslide - Expansive clay stone soils found as a layer under a more rigid top layer of soils, become unstable as the moisture content increases, allowing the claystone and top layers of the soil to move. If the soil is located on a slope, the top layer of soil can creep downhill or even cause a landslide. Consequently, a house with a weak foundation built on unstable slopes can be subjected to creeping of the structure downslope or to failure of the structures in a landslide [17].

4. Typical Structural Distress Patterns:-Buildings in arid areas tend to experience an edge lift, and conversely, in humid climates, the expansive soils may shrink when it dries, causing the edge to depress. The difference in water content between the interior and exterior of a building causes uplift force on the interior footings and walls, shrinkage settlement of the exterior walls and lateral thrust on the exterior walls [1].

5. Differential Settlement :- This can cause cracking, rutting and deformation in general distresses on road and runway pavements, failure of drainage structures (Bridges, Culverts) etc. the differential settlements creates series of bumps or corrugations, potholes on different road sections in various parts of the country which inturns reduces the riding quality of roads [17].

2.5 Techniques of soil stabilization

There are Various soil modification or stabilization which are necessary for designers/developers to take into consideration the local economic factors as well as environmental conditions and project locations for design. Each of them has there owned effect on the total cost of the project. Therefore, while selecting a stabilization technique for expansive soil, we consider a different mechanism.

2.5.1 Cement

Cement is generally the best type of admixture to use with soils. It is also commonly available but is often expensive [18]. The stabilization is due to cementitious links between the calcium silicate and aluminate hydration products and the soil particles. The degree of improvement depends upon the quantity of the cement used and the type of soil [19]. The amount of cement required to stabilize expansive soils ranges from 2% to 6% by weight [18].

However, its production consumes a huge amount of energy combined with emission of harmful gases which pollutes the atmosphere (Energy and Climate, 2012). Partial replacement of OPC with pozzolans has been discovered to be of very good solution to these problems and their usage is highly encouraged [20].

2.5.2 Lime

The use of lime as a stabilizing agent has been a popular method during the last few decades because it decreases the volume change of expansive soil [19]. Typically, lime addition to expansive soils initiates four types of reactions between lime and the silicate and aluminate constituents of the expansive clay. These are flocculation, cation exchange, carbonation, and pozzolanic reaction [21].

These reactions contribute to mineralogical and microstructural changes in the treated or stabilized soils. Due to this reason, lime-treated expansive soil behavior is significantly different from natural or untreated expansive soils. The pozzolanic reaction has significant influence, particularly on the expansive soil behavior. The amount of lime required to stabilize expansive soils ranges from 2% to 8% by weight [18]. Lime has been extensively used to improve the engineering properties of fine-grained soils [22] [23].

2.5.3 Fly ash

Fly ash is one of the additives that has been successfully used in stabilization of expansive soils. Fly ash is usually used as a pozzolanic material. It exhibits both pozzolanic and cementitious properties [19]. Fly ash is defined as the materials extracted from the flue gases of a furnace fired with coal. This admixture can promote flocculation of dispersed clay particles by means of a cation exchange process. The addition of fly ash to expansive soil reduces the swell potential and swelling pressure. Both the swell potential and swelling pressure were reduced by 50% at 20% fly ash. The swelling characteristics, namely, the free swell index, swell potential and swelling pressure, showed greater reductions at lower percentages of fly ash. At percentages of fly ash greater than 10%, the reduction gradually decreased [24].

According to [25], addition of lime significantly improved consistency, swelling and strength properties of the expansive soil. However, the presence of fly ash fundamental to further improve the soil behavior, due essentially to the occurrence of a larger amount of time-dependent pozzolanic reactions. Moreover it is always encouraged to use fly ash for stabilization where easily and economically available.

2.5.4 Waste Agricultural Materials

The application of agro-wastes as a soil stabilizing agent for cost-effective and environmentally safe construction practices is desperately needed in agriculture-based developing countries like Ethiopia.

2.5.4.1 Rice Husk

Rice husk (RH), a major agro-waste obtained from the food crop of paddy was generally considered a worthless waste of rice mills. It is now well established that rice husk ash (RHA) resulting from burnt rice husk has a great potential as a pozzolanic material. Chemically, RHA consists of 82-87% silica (SiO_2) which makes it an excellent substitute for conventional pozzolanic materials for soil stabilization [2].

2.5.4.2 Bagasse Ash

Bagasse ash has been reported to possess pozzolanic properties. It was reported that bagasse ash contains a large amount of silica and other relevant oxides which enhance good pozzolanic activity [26].

2.6 General Reviews About Corncob Ash

In our country, Ethiopia there are no much studies and significant experience on expansive soil improvement/stabilization. Particularly the performance of corncob ash to improve the engineering properties of black cotton soils is not much tried yet except it was used as charcoal in some area of the countries. However, the silica composition found in CCA is used for partial or total replacement of lime after some modification.

Ogunfolami reported that mixing of the CCA as a partial replacement with ordinary Portland cement can be carried out at the point of need i.e., on site, [27].

Adesanya and Raheem studied the use of CCA blended cement produced in the controlled circumstances. The studies revealed that the compressive strength properties of the CCA-blended cement concrete is less than that of sample made with plain concrete at early curing ages but significant improvement is noticed at later ages (after 90 days).Thus, there is necessary to look for ways to improve the strength characters during early ages.

Raheem et al. [27] Concluded that the addition of admixtures in corncob ash cement concrete increases compressive strength character at short term and long term curing periods irrespective of the type of binding materials used. There is a chances of increase in strength can be achieved at early ages by using accelerators. With plasticizer, high strength can be achieved at both short term and long term periods while with water reducing agents and retarder, greater strength can be achieved at long term only.

Olafusi and Olutoge reported [27], that although CCA is recommended to use as a partial replacement as a cementitious material in high strength concrete, but due to addition of CCA in production of high strength concrete would take more time to attain its designed strength and also the concrete made with CCA as admixture would require.

3. MATERIALS AND METHODS

3.1 Study Area Description

3.1.1 General

Jimma is the largest city in south-western Ethiopia. It is a special zone of the Oromia Region and is surrounded by Jimma Zone. It has a latitude and longitude of 7°40′N 36°50′E. Prior to the 2007 census, Jimma was reorganized administratively as a special Zone. The city is home to a museum, Jimma University, several markets, and an airport. Also of note is the Jimma Research Center, founded in 1968, which is run by the Ethiopian Institute of Agricultural Research. The Center specializes in agricultural research, including serving as the national center for research to improve the yield of coffee and spices. Jimma has a tropical rainforest climate. It features a long annual wet season from March to October [15].

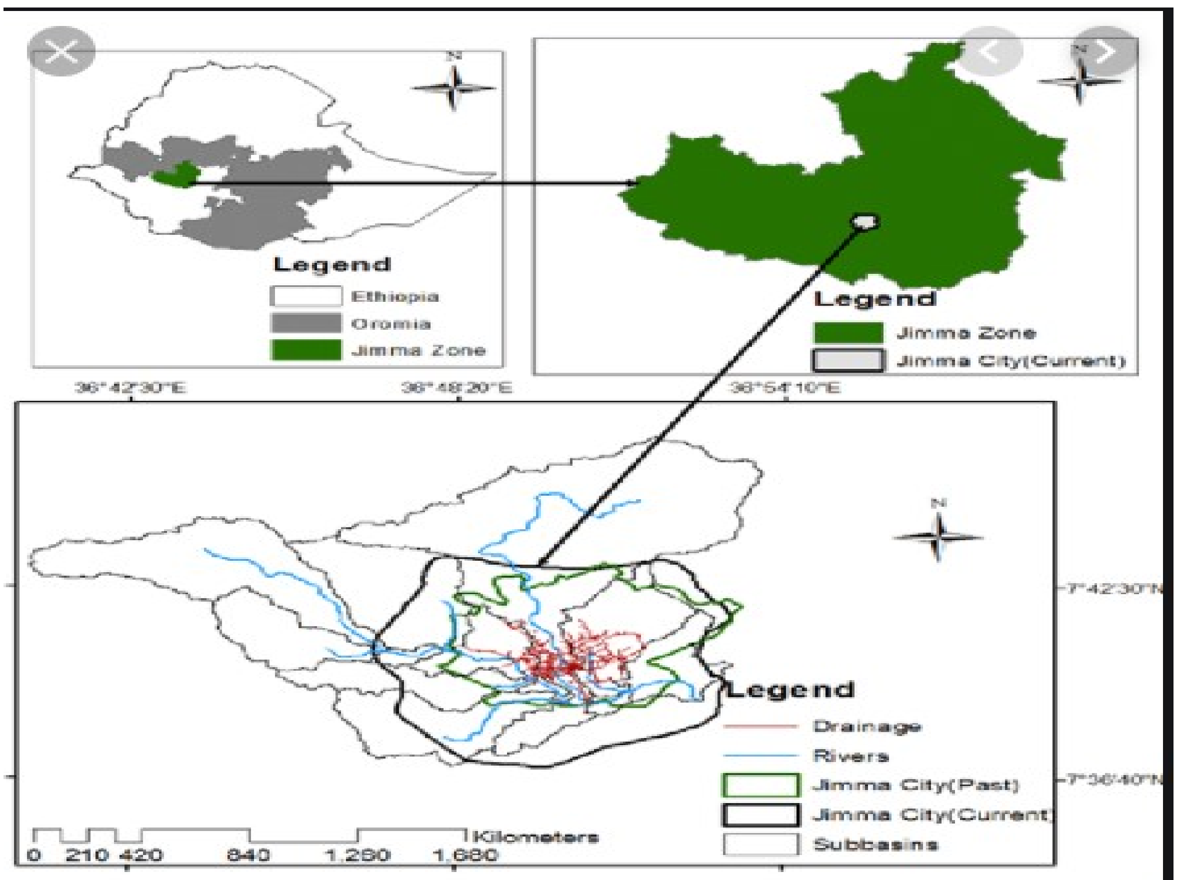


Fig 3.1. Map of Study Area in Jimma City (source: Google Map)

3.1.2 Climate

Jimma has a relatively cool tropical monsoon climate (Köppen climate classification: Am) under the Köppen climate classification. It features a long annual wet season from March to October. Temperatures at Jimma are in a comfortable range, with the daily mean staying between 20 °C and 25 °C year-round [15].

3.1.3 Geology

The main geologic formation of Jimma town is the Cenozoic tertiary volcanic rock of Nazareth Series and Jimma Volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the Trap series formation. Tuft and Alluvials are found in few amounts at different localities [15].

3.1.4 Demographies

Based on the 2007 Census conducted by the Central Statistical Agency of Ethiopia (CSA), this Zone has a total population of 120,960, of whom 60,824 are men and 60,136 women. With an area of 50.52 square kilometers, Jimma has a population density of 2,394.30 all are urban inhabitants. A total of 32,191 households were counted in this Zone, which results in an average of 3.76 persons to a household, and 30,016 housing units. The three largest ethnic groups reported in Jimma were the Oromo (46.71%), the Amhara (17.14%) and the Dawro (10.05%); all other ethnic groups made up 26.1% of the population. The majority of the inhabitants said they practiced Ethiopian Orthodox Christianity, with 46.84% of the population reporting they observed this belief, while 39.03% of the population were Muslim, and 13.06% were Protestant. The national 1994 census reported this town had a total population of 88,867, of whom 43,874 were men and 44,993 were women.

3.2 Sample Collection Methods

3.2.1 Sources of Data

Both Disturbed and Undisturbed soil samples will be collected from Mendera Kochi around kebele 5 and Merkato Sefer. Disturbed Samples**: -** are samples where the structure of the natural soil has been disturbed to a considerable degree by the action of the boring tolls or excavation equipment. Disturbed samples are satisfactory for performing classification tests such as, Specific gravity, Sieve analysis, Atterberg limits, Compaction test and CBR etc. Undisturbed Samples**: -** are samples, which represent as closely as is practicable, the true in-situ structure and water content of the soil. Undisturbed samples are required for determining reliable information on the shearing resistance and stress-deformation characteristics of a deposit.

3.2.2 Samping Techniques

The sampling technique used for this research were a purposive sampling which is nonprobability method, because the experimental investigation of the study was executed particularly on the weak subgrade soil samples, since this study pick out the samples in relation to some criterion, which are considered important for the particular study.

3.2.3 Research Design

The research methodologies that were used are quantitative and experimental approach so as to achieve the desired objective. The purpose of the research is to determine the effect of adding mixture CCA-Lime on the Engineering properties of black cotton soil of Jimma. To achieve the objective of the research the necessary step which took were mentioned as follows:

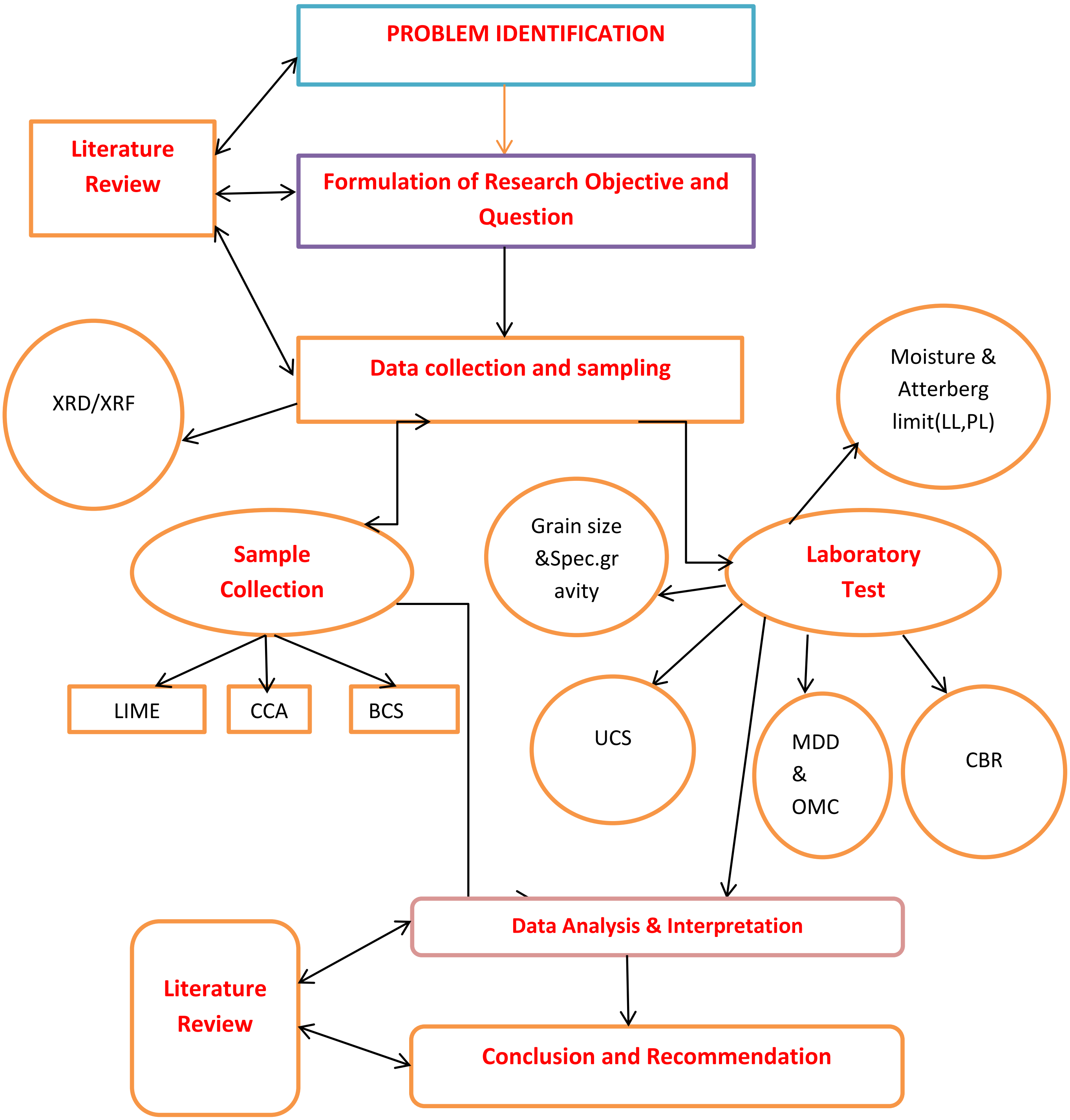


Figure 3.2: Flow Chart of Research Methodology

3.2.4 Study Variables

Independent variables

- ✓ Varying Lime content & Corncob ash in %
- ✓ Gradation, LL, PL, PI
- ✓ X-ray Diffraction
- ✓ X-ray Flouresence
- ✓ Compaction, Moisture Content
- ✓ Soaked, and CBR Swell,
- ✓ UCS

Dependent Variables

- ✓ Combined effects of CCA-LIME in Black cotton soil properties.

3.2.5 Sample Preparation

The BC soil was stabilized with CCA by a dry weight basis; the CCA added is defined by a percentage as the ratio of the weight of CCA to the dry weight of the soil. A series of experiments was performed on stabilized samples with different concentrations of CCA (0%, 2%, 4%, 6% and 8%). For CBR tests, BC soil-CCA mixtures were blended with an optimum amount of water and then compacted in the CBR mold.

Samples were cured at a room temperature (25 ^{o}C) for consecutive days for micro-structural and mineralogical analyses. At the end of the curing process, samples were oven-dried at 105 ^{o}C for 24 h, and then were powdered with a mortar and pestle until the material could pass through 63 mm sieve.

3.3 MATERIALS

3.3.1 Black Cotton Soil

The black cotton soil used in this investigation was obtained from mendera kochi and merkato sefer in Jimma City, Oromia (fig 3.1). The soil was collected by open excavation, from a depth of 2 m below ground level. The soil was dried and passed through Indian Standard sieve size of 4.25 mm.

3.3.2 Corncob Ash

A Corncob is the central core of an ear of maize. It is the part of the ear on which the kernels grow. The ear is also considered a "cob" or "pole," but it is not fully a "pole" until the ear is shucked, or removed from the plant material around the ear. Young ears, also called baby corn, can be consumed raw, but as the plant matures, the cob becomes tougher until only the kernels are edible. When harvesting corn, the corncob may be collected as part of the ear (necessary for corn on the cob), or instead may be left as part of the corn stover in the field. The innermost part of the cob is white and has a consistency similar to foam plastic. It is the agricultural waste product obtained from maize or corn.

3.3.3 Lime

The use of lime as a stabilizing agent has been a popular method during the last few decades because it decreases the volume change of expansive soil [19]. Typically, lime addition to expansive soils initiates four types of reactions between lime and the silicate and aluminate constituents of the expansive clay. These are flocculation, cation exchange, carbonation, and pozzolanic reaction [21]. For this study quick lime was used which is bought from local market.

The benefit from applying Corncob ash for soil stabilisations with quick lime is related to chemical reaction between calcium hydroxide produced by lime with pozzolan that is supplied from CCA.

3.4 Laboratory Testing and Analysis

Laboratory works conducted at Jimma Institute of Technology i.e, Civil Engineering laboratory, and Material Engineering Laboratory and Jimma College of Agriculture and veterinary Medicine. Quick lime collected from the market, while CCA Collected from the local farmer near to the city.

Tests for soil classification which included grain size distribution, free swell, specific gravity, and Atterberg limits. These are indicative tests that are usually used for identifying wether the soil is expansive or not. The conducted tests however included hydrometer analysis, Atterberg limits, specific gravity, moisture density relation, free swell, XRD, CBR and percent swell of CBR to fully characterize and attain the objective of the research.

3.5 Laboratory Testing Procedures

3.5.1 Specific Gravity Determination

This lab is performed to determine the specific gravity of soil by using a small pycnometer. Specific gravity is defined as the ratio of the mass of a given volume of a material to the mass of an equal volume of water. In effect, it tells us how much the material is heavier than (or lighter) than water. The particular specific gravity of a soil actually denotes the specific gravity of the solid matter of the soil and refers, therefore, to the ratio of the mass of solid matter of a given soil sample to the mass of an equal volume (i.e equal to the volume of the solid matter) of water. Alternatively, specific gravity of soil may be defined as the ratio of the unit mass of solids (mass of solids divided by volume of solids) in the soil to the unit mass of water.

In equation form, $G_s = \frac{M_s}{V_s \rho_w}$ *where*, G_s =specific gravity of soil M_S =mass of solid, g

V_s = volume of solid, cm^3, ρ_w =unit mass of water ($1 g/cm^3$)

The specific gravity of the samples will be determined on black cotton clay using ASTM D 854-83.

3.5.2 Grain-Size Analysis

For this study both wet sieve analysis and hydrometer analysis were done on black cotton clay ASTM D422-63 [28].

3.5.2.1 Sieve Analysis

The sieve analysis determine the grain size distribution curve of soil samples by passing them through a stack of sieves of decreasing mush opening sizes and by measuring the weight retained on each sieve. The sieve analysis is generally applied to the soil fraction larger than 75μm. The coarse grained soil (particles greater than 75μm) can be further sub divided in-to gravel fraction (size greater than 4.75mm) and sand fraction (75μm < size < 4.75mm).

3.5.2.2 Hydrometer Analysis

Basically the behaviors of fine-grained soils are influenced by the shape, arrangement of particles and geological history. However, there are some cases in which the grain size distribution of these soils is required, for example, in the design of filters for drainage system and rise of water in the capillary opening. The behavior of expansive soils is also a function of the proportion of clay fraction in the soil. Hydrometer analysis is a method used to determine the grain size distribution of fine grained soil having particles sizes smaller than 75μm using Hydrometer. It is based on the Stoke's law, which says that the larger the grain-size, the greater it setting velocity in a fluid.

3.5.3 Free Swell Index

This test would be performed by pouring slowly 10 gm of dry soil, 10gm of (soil+ corncob ash) passing through 425-micron sieve, in two different 100cc glass jar filled with distilled water. The swollen volume of (BCS), (BCS-CCA), (BCS, CCA, and lime) mixes are recorded as per IS 2720 part 40 (1985).

$$FSI = (V_w - V_k)/ V_k*100 \qquad \text{3.1}$$

Where FSI = Free Swell Index

V_w = Final volume in water

V_k= Final volume in kerosene

3.5.4 Atterberg Limit

The Atterberg limits are based on the moisture content of the soil. The liquid limit and plastic limit test conducted using Casagrande's liquid limit apparatus as per the procedures laid down in AASHTO T89-90. This lab is performed to determine the plastic and liquid limits of a fine grained soil. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a pat of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 in.) diameter threads without crumbling.

The methods described herein are performed only on that portion of a soil that passes the 425-µm (No. 40) sieve. Therefore, the relative contribution of this portion of the soil to the properties of the sample as a whole must be considered when using these tests to evaluate properties of a soil. The liquid and plastic limits of many soils that have been allowed to dry before testing may be considerably different from values obtained on non-dried samples. If the liquid and plastic limits of soils are used to correlate or estimate the engineering behavior of soils in their natural moist state, samples should not be permitted to dry before testing unless data on dried samples are specifically desired.

3.5.5 Modified Compaction

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compactive effort.

The compactive effort is the amount of mechanical energy that is applied to the soil mass. For construction of highways, airports, and other structures, it is often necessary to compact soil to improve its strength. Preparation of soil sample for proctor's Modified compaction test will be done as per AASHTO T180-95.

The optimum water content is the water content that results in the greatest density for a specified compactive effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density.

Soil placed as engineering fill (embankments, foundation pads, road bases) is compacted to a dense state to obtain satisfactory engineering properties such as, shear strength, compressibility, or permeability. Also, foundation soils are often compacted to improve their engineering properties.

Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved.

3.5.6 CBR of treated & Untreated Compacted Soils

The California bearing ratio tests was conducted on black cotton clay, black cotton clay+% CCA, black cotton clay + % CCA + % lime mixtures as per AASHTO T193-93 [29].

The CBR value is used as a criterion to evaluate the strength of the pavement sub-grade, sub-base and base course materials for road construction. In addition, it is used to estimate the potential use of additives for soil stabilization. The mold used for CBR tests has an internal volume of 2105 cm^3 and a height of 17.7cm, and the samples were compacted into this mold according to the standard compaction procedure. A surcharge mass of 4.5 kg was applied for soaked states and the samples were completely immersed into freshwater for 4 days.

The treated and untreated samples were prepared for tests, and then a penetration piston with a diameter of 4.9cm was pushed into the samples at a loading rate of 1.27mm/min. The CBR value was obtained by dividing the test load by a standard load at the same depth of penetration. Swelling data were also determined from the CBR tests. The results of CBR test computed for the collected samples were written in Table 4.10.

3.5.7 CBR swell

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 96 hours the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is 116.43mm, see Table 4.11.

Then CBR swell is given by:

CBR Swell = (Change in Length in mm during soaking /116.3) *100% ……………… (3.3).

3.5.8 X-ray Diffraction

X-ray diffraction is the most powerful technique used for analysis of minerals and offers mineral phase's identification and quantification. The standard operation describes the Whole rock approach where the sample is prepared into a random powder which helps in analysis of total amounts and identification of non-clay minerals present.

The analysis provides information, it is a high-tech, rapid, cheap and non-destructive technique for qualitative and quantitative analysis of crystalline compounds; about 95% of all solid materials in the soil are crystalline. When X-rays interact with a crystalline substance or powder, a diffraction pattern called a diffractogram is produced and can be quantified. Information obtained from this patterns include phase composition of a sample, types and nature of crystalline phases (minerals) present, crystal structure, amount of amorphous (OM) content, micro strain, size and orientation of crystallites. XRD has become an indispensable method for materials investigation, characterization and quality control. The angle and intensity of the diffracted beam recorded by a detector forms a diffraction pattern, which provides information about a sample [30].

PROCEDURE

Sample preparation

a. Milling - Samples are milled using a McCrone Grinding Mill with agate grinding elements in a jar. The unique grinding action of the mill rapidly reduces particles to sub micrometer sizes and mixes for homogenization required for quantitative and qualitative analytical methods (Approx. 10 µm). Make sure to retrieve as much sample as possible after milling. Centrifuge to remove ethanol, decant, dry and crush sample

Sample mounting (randomly oriented mounts)

a. Load the randomly prepared samples into the well of a low background sample holder and tap gently on the bench to help fill and pack to avoid sample displacement which causes peak shifts.
b. Using a sharp razor, tap the sample surface slowly in all directions to distort orientation a few times, and then continue to level while gently removing the loose excess sample powder by scrapping off from the edges of the well of the sample holder.
c. It is very important to have the correct sample level to the well surface since any error in the height of the sample will cause peak displacement.
d. After measurement the disc can be off loaded and washed with tap water and then re-used.

Sample measurement

a. Switch on the instrument and let it warm for thirty minutes.
b. Pull down the spherical handle of the stage and place onto sample holder into the sample position of the stage (Goniometer).
c. Lift the sample back into the sample measurement position by pulling up the spherical handle of the stage and slide down the instrument door. Press down door handle with force in order to close it correctly.
d. Activate the High Voltage
e. Set measurement parameters for a typical Lynx eye
f. Select start button to initiate acquisition.

Data acquisition

The intensity of diffracted X-rays is continuously recorded as the sample and detector rotate through their respective angles. A peak in intensity occurs when the mineral contains lattice planes with d-spacing appropriate to diffract X-rays at that value of θ. Results are presented as peak positions at 2θ and X-ray counts (intensity) in the form of a table or an x-y plot (shown above). Intensity (*I*) is either reported as peak height intensity, that intensity above background, or as integrated intensity, the area under the peak. When sample material is analyzed by XRD a range of diffracted peaks will occur and a diffractogram is obtained. The angle of each peak is used to identify the mineral phase while the intensity of the peak will indicate relative amount present.

The result of an XRD measurement is a Diffractogram showing:-

a. Phases present (Peak positions),

b. Phase concentrations (Peak heights),

c. Amorphous content (Background hump),

d. Crystallite size/strain (Peak widths).

3.6 Experimental Design Procedures and Analysis Techniques

The XRD was performed on untreated and CCA-Lime treated BC soils. The analysis was carried out to identify the changes in mineralogical phases presented in the samples using XRD 7000 diffractometer.

The XRD was carried out using CuKa radiation. The samples were scanned at an angle of 2q, and its ranges from 15 to 70 were chosen to provide enough X-ray diffraction peaks to identify the most common soil minerals. Match! (Crystal Impact) software was used to analyze the data obtained from diffractometer. The main use of powder diffraction is to identify components in a sample by a search/match procedure. It is commonly used to identify unknown substances, by comparing diffraction data against a database maintained by the International Centre for Diffraction Data, ICDD.

4. RESULTS AND DISCUSSION

4.1 Pozzolanic Properties of Corn Cob Ash

It is now well established that corncob ash (CCA) resulting from burnt corncob has great potential as a pozzolanic material. ASTM C-618 [36], describes pozzolana as a siliceous or silicious and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically reacts with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. The X-ray fluorescence result showed Pozzolanic property of CCA. It was observed that SiO_2 has the highest composition. Chemically, CCA consists of 63.8% silica (SiO_2) which makes it an excellent substitute for conventional pozzolanic materials for soil stabilization.

Table 4.1: Chemical Composition of Corncob ash

Chemical Constituents	% Composition	ASTM C-618 requirement in (%)	Remark
CaO	11.34	-	
SiO_2	63.8	35 min	✓
MgO	2.58	5 max	✓
Al_2O_3	10.19	-	
Fe_2O_3	4.85	-	
SO_3	2.53		
K_2O	4.71	-	

The chemical analysis of CCA resulted with the total compound content of Silicon Dioxide (SiO_2), Aluminum Oxide (Al_2O_3) and Iron Oxide (Fe_2O_3) was above the minimum of 70% as per ASTM C-618. It means the material was a good Pozzolan. The CCA sample had a cementitious compound like calcium oxide, alumina, and iron oxide with a total of about more than 20%. The rate of the pozzolanic reaction can also be controlled by external factors such as the mix proportions, the amount of water, temperature of reaction, curing condition and period.

4.2 Engineering Properties of the Natural Subgrade Soil

The Engineering properties of the Natural soil are given in Table 4.2. The soil has a specific gravity of 2.67 and 2.72 for both sample respectively and its bearing capacity or strength, as presented by its CBR and UCS values, is generally low. It is clayey (Fig. 4.1) and the clay minerals are suspected to be predominantly smecitite because of its expansive nature. Approximately 90% of the soil particles are finer than 75 µm and this fraction significantly influences the behaviour of the soil. The swelling and shrinkage characteristics of the soil are high.

Table 4.2: Geotechnical Properties of Black Cotton Soil

Properties	**Property**	**Values**	
		Mendera Kochi	**Merkato Sefer**
Moisture Content		43.54	40.16
Grain Size	Course (%)	0.42	2.35
	Sand (%)	9.14	10.49
	Silt &Clay (%)	90.86	89.51
Atterberg's Limit	Liquid Limit (%)	93	90
	Plasticity Index (%)	66.98	66.28
Compaction Test	MDD (kN/cu.m)	1.56	1.54
	OMC (%)	24.5	24
Swelling Test	Free Swell Index (%)	100	87
CBR (%)		1.24	1.54
UCS (kN/sq.m)		59	64
CBR Swell (%)		9.83	8.11

4.2.1 Chemical & Minerological Identification

The mineralogy of the natural soil samples were determined using indirect methods from plastic index and liquid limit chart as reviewed on literature review of this report.

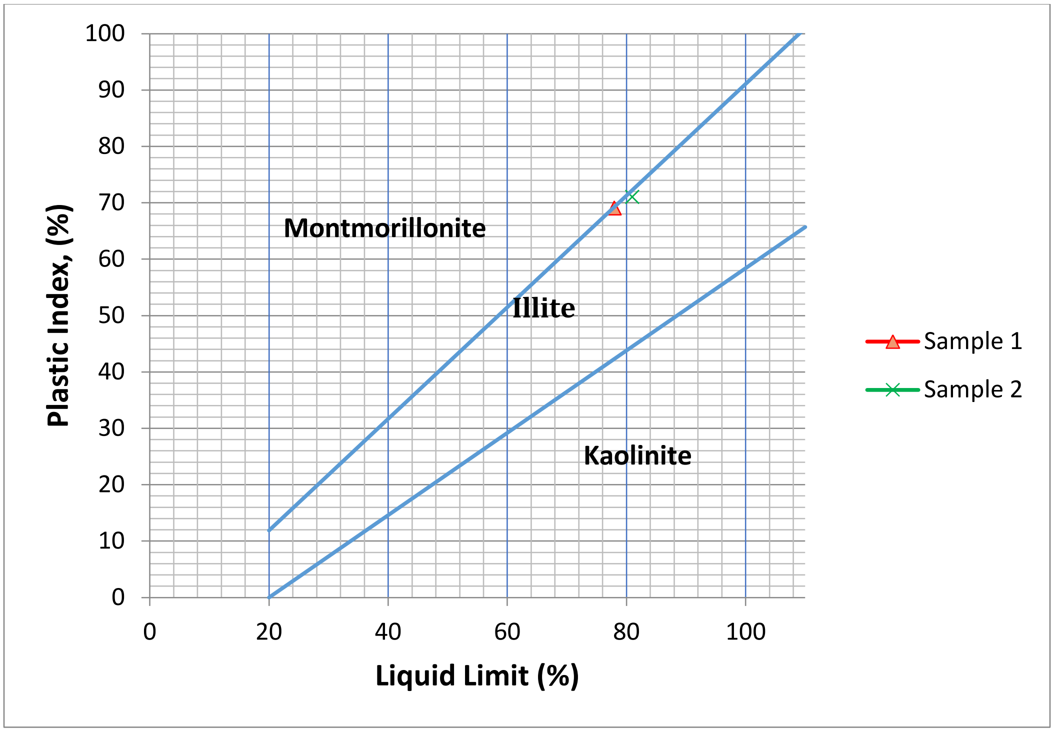


Fig 4.1: Mineralogical Identification of Expansive Soils using Indirect Method

As one would expect, the values tend to plot well above the A-line, and tend toward the U-line.

4.2.2 Particle size distribution

A basic element of a soil classification system is the determination of the amount and distribution of the particle sizes in the soil. To determine the distribution of coarser particles, 500g of the natural subgrade soil is taken and washed on sieve size of 75µm. A hydrometer test is conducted on 50gm of soil sample passing sieve No.200. The soil sample was soaked in chemical solution (Sodium hexa-meta phosphate) for 24 hours. The tabular experimental results are presented in appendix and the particle size distribution curves are shown in Fig below.

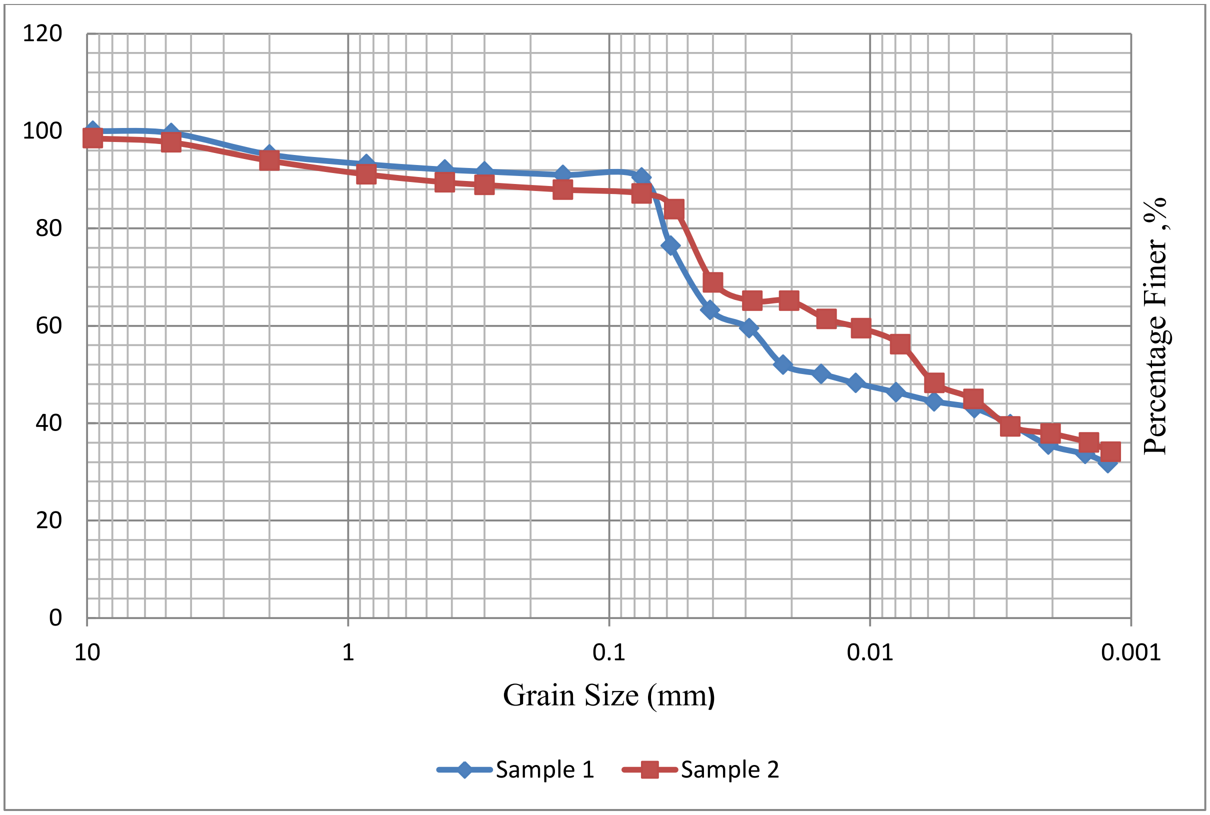


Fig 4.2: Particle size distribution curve

The soil for sample 1 is black clay, and almost 90.86% of the soil are passing through No.200 sieve and the soil for sample 2 is gray, and almost 89.51 % of the soil is passing through No.200 sieve as shown in figure 4.2. Almost the given soil sample were a fine silty clay soil.

4.2.3 Atterberg limit test

Atterberg limits (liquid limit, plastic limit) were determined according to AASHTO T 89 and 90 standard test method. The detailed tabular results of the Atterberg limits were shown in appendix. The results of Atterberg limit test computed for the collected samples were written in Table 4.6. According to the Ministry of Works and Urban Development of Ethiopia (MWUD, 2009), all greyish and/or brownish clays in Ethiopia with plasticity indices (PI) greater than 25% can be identified as expansive soils.

Table 4.3: Summary of Atterberg limits for the natural subgrade soil

Sample No.	100 Natural subgrade soil		
	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)
Sample 1	93	26.48	66.52
Sample 2	90	23.72	66.28

According to Atterberg limit test result as shown above soil sample 1 & 2 changed from liquid state to plastic state and got an average liquid limit of 93% and 90% respectively. The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 26.48% and 23.72% for sample 1 and 2 soil sample respectively. The soil sample also has Plastic Index of 66.52% and 66.28% for both soil sample respectively. As result of Plastic Index indicates both the native subgrade soil samples have poor for sub grade material unless it treated.

4.2.4 Soil Classification

The most widely used soil classification systems are AASHTO and USCS systems.

4.2.4.1 AASHTO Classification system

The AASHTO system uses similar techniques as that of USCS but the dividing line has an equation of the form PI= LL-30. It generally classifies a soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2 and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6 and A-7. As it can be observed from AASHTO Classification system plasticity chart is as Follows in Fig.4.3.

Table 4.4: Soil Classification

	Atterberg Limits			Soil Classification	
Sample No.	LL	PL	PI	USCS	AASHTO
1	93	26.48	66.52	CH/OH	A-7-5
2	90	23.72	66.28	CH/OH	A-7-5

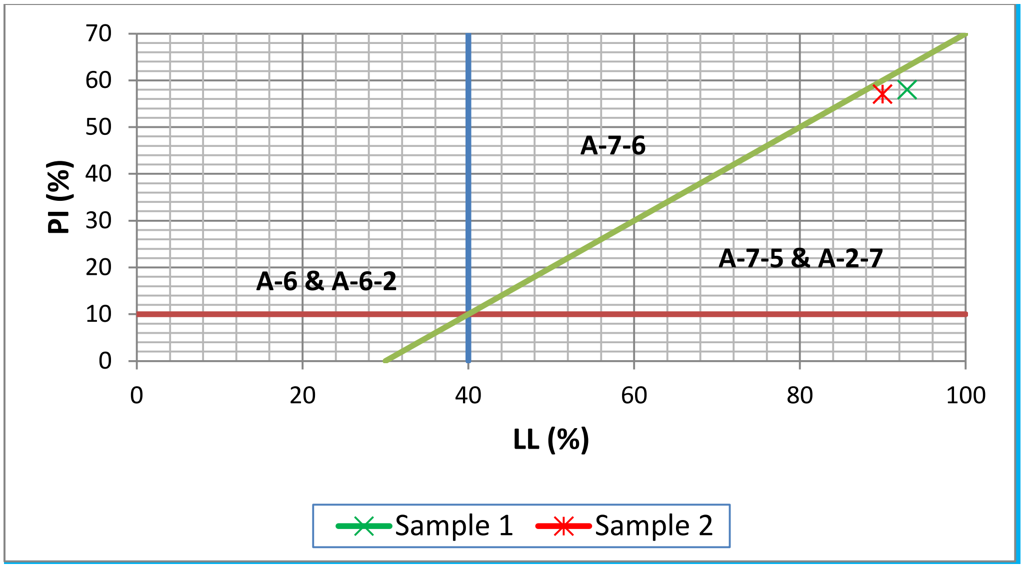


Fig 4.3: Soil to classification chart according AASHTO system

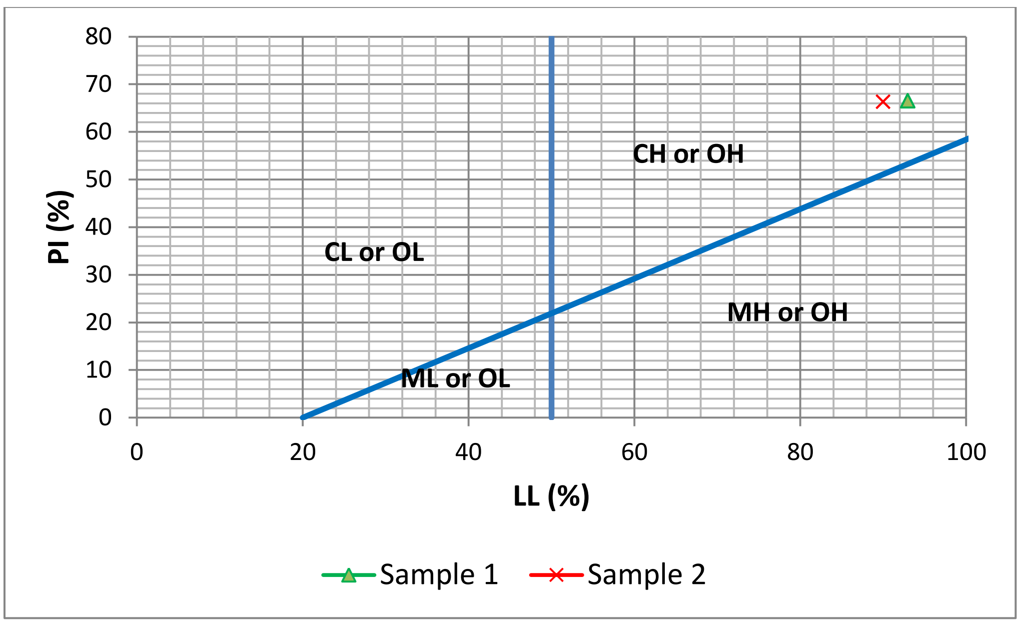


Fig 4.4: Plasticity chart of the studied soil using USCS

As results of atterberg limit test result Sample 1 (MK) and 2 (MS) subgrade soils has different Liquid limit and plastic Index, however according to AASHTO soil classification system both soil samples have classified under group A-7-5. with rating Fair-to- Poor to be used as subgrade material. Thus, the natural subgrade material is unsuitable to be used as subgrade material without employing some improvement methods.

4.2.4.2 Unified soil classification (USCS) system

This system describes a system for classifying minerals and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit and plasticity index and shall be used when precise classification is required (ASTM). The classification of the soils is presented in Fig 4.4.

According to USCS, if the Liquid limit are greater or equal to 50% the soil can be clay, silt, or organic depends on whether the soil coordinates plot above or below the A line. Since both soil sample has Liquid limit more than 50% and above A-Line, so they are classified under high to very high CH.

4.2.5 Specific Gravity of natural subgrade soil

This test was conducted on fined grained particles of materials used for the study and summary of the test results are tabulated as followed in Table 4.5.

Table 4.5: Specific gravity of the samples

Sample	**TP 1@2m Mendera Kochi**			**TP 2 @2m Merkato Sefer**		
Code	A	A1	A2	C	C1	C2
MP	31.71	29.07	27.81	32.13	30.07	32.81
MP + S	41.86	40.48	38.01	38.61	38.40	35.61
MP+S+W	134.77	132.049	129.86	134.23	133.009	128.01
MP+W	128.57	125.67	122.86	129.21	128.67	126.86
Ti	22	22	22	22	21	22
D TI	0.9978	0.9978	0.9978	0.9978	0.9978	0.9978
Tx	21	21	21	21	22	22
D TX	0.99802	0.9978	0.9978	0.99802	0.9978	0.9978
C MPS	128.59	125.67	122.86	129.23	128.67	126.86
K	0.9998	0.9998	0.9998	0.9998	0.9996	0.9996
GS	2.66	2.57	2.79	2.77	2.89	2.48
	2.67			**2.72**		

As Table 4.5 showed that sample 1 has an average specific gravity of 2.67 and 2.72 for sample 2. The specific gravity of solid particles of most soils varies from 2.5 to 2.9. For most of the calculations specific gravity (Gs) can be assumed as 2.65 for Cohesion less soils and 2.70 for clay soils. This result indicated both samples are dived under clay soil

4.2.6 Free swell index

The free swell Index of the study area soil was presented as,

Table 4.6: Free swell of the sample

Sample name	Free swell (%)
Mendera Kochi	100
Merkato Sefer	87

This result indicated that the two soils were Highly Expansive Soils. It was supported by Rao (2007) Soils are called highly expansive when the free swell index exceeds 50% and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying.

4.2.7 Compaction test results

Modified Proctor compaction tests were conducted on the soil to determine the relationship between the moisture content and dry density for specific compaction effort according to AASHTO T99-97. The soil sample 1 has optimum moisture content 24.5% and the maximum dry density is 1.57gm/cm^3. Also, the soil sample 2 has optimum moisture content 24% and the maximum dry density is 1.54gm/cm^3 as shown on figure 4.9 & 4.10. The results of MDD and OMC test computed for the collected samples were written in Table 4.9.

4.2.8 CBR test result

Strength of the soil has also been determined. A one point (56 blows) soaked CBR test was conducted according to AASHTO T193, summary of results as presented table 4.9 blow. According to laboratory result as presented in table 4.9, soil sample 1 had 1.24% soaked CBR value with 9.83% swell and soil sample 2 had 1.54% CBR value with 8.11% CBR swell. From the soaked CBR test, it was found that the natural subgrade soil has low CBR value, as compared ERA manual-2002 both subgrade soil samples does not satisfy the minimum requirements as sub-grade material. Also, CBR swell values are above the specified maximum value of 2%, hence this soil needs to be treated before use.

4.2.9 Summary of the natural subgrade test results

According to the laboratory test results of the natural subgrade soil sample obtained during the study, the proportion of fines passing no 200 sieve 98.48%, 97.65%, liquid limit 93%, 90%, and plasticity index 66.52%, 66.28%, for SAMPLE 1 and SAMPLE 2 soil sample respectively, both soils samples are classified in to A-7-5 as per the AASHTO and CH as per the USCS classification system.

The liquid limit and plasticity index values are very much greater than the ERA requirements, i.e., liquid limit less than 60% and plasticity index less than 30%. Accordingly, both samples show excess values in each parameter, and the soil in general thus had expansive property. The free swell index of 100% and 87% for SAMPLE 1 and SAMPLE 2 soil sample, respectively, also revealed that the soils are expansive soil since its free swell index is greater than 50%. Furthermore, the CBR and CBR percent swell of 1.24%, 1.54% and 9.83%, 8.11% for SAMPLE 1 and SAMPLE 2 soil samples respectively indicate that the soils have a low load-bearing capacity and high swelling potential when compared to ERA's specifications of CBR > 3% and percent swell of less than 2% which makes it unsuitable for construction without any suitable treatment measure.

However, the comparisons above between the ERA design manual and laboratory results of the soil show that the soil sample did not fulfill the requirements as a sub-grade and is determined to be unsuitable for sub-grade in road construction. Therefore, the subgrade soil should be treated with appropriate improving methods before use as a road subgrade.

4.3. Effect of the Mixture of CCA & Lime on the Engineering properties of the BC soil

4.3.1 Effect of the mixture on Atterberg Limits

Table 4.7 presents the results of Atterberg limits determined under different additive contents. The PI variations for both untreated and treated soils are shown in Fig. 4.5 & 4.6.

Table 4.7: Atterberg limit test result of treated & untreated expansive clay soils

Sample No.	Mixture of CCA & Lime		LL (%)	PL (%)	PI (%)	ERA(2000) Requirement for PI in (%)	Remark
	CCA (%)	Lime (%)					
1	0	0	93	26.48	66.52	< 30%	Poor
	8	0	83	49.85	33.15		Poor
	6	2	82	57.45	24.55		Satisfied
	4	4	81	57.56	23.44		Satisfied
	2	6	78	59.24	18.76		Satisfied
	0	8	76	65.58	10.42		Satisfied
2	0	0	90	23.72	66.28	< 30%	Poor
	8	0	86	53.54	32.46		Poor
	6	2	80	58.14	21.86		Satisfied
	4	4	79	61.85	17.15		Satisfied
	2	6	74	65.88	8.12		Satisfied
	0	8	72	69.97	2.03		Satisfied

The decrease in PI indicates an improvement in the workability of the soil. The higher the PI is, the greater the quantum of water that can be imbibed by the soil is, and hence the greater its swell potential would be. The soil showed an immediate decrease in PI upon the addition of lime. It is obvious that an addition of 4% of lime was sufficient to enhance the workability of the soil by reducing the PI from 66.52% to 23.44% and from 66.28% to 17.15% respectively for both samples. Increasing the lime content beyond 4% had a marginal effect on reducing the PI. This can be attributed to the chemical reactions between lime and soil including ion exchange and associated flocculation reactions [31].

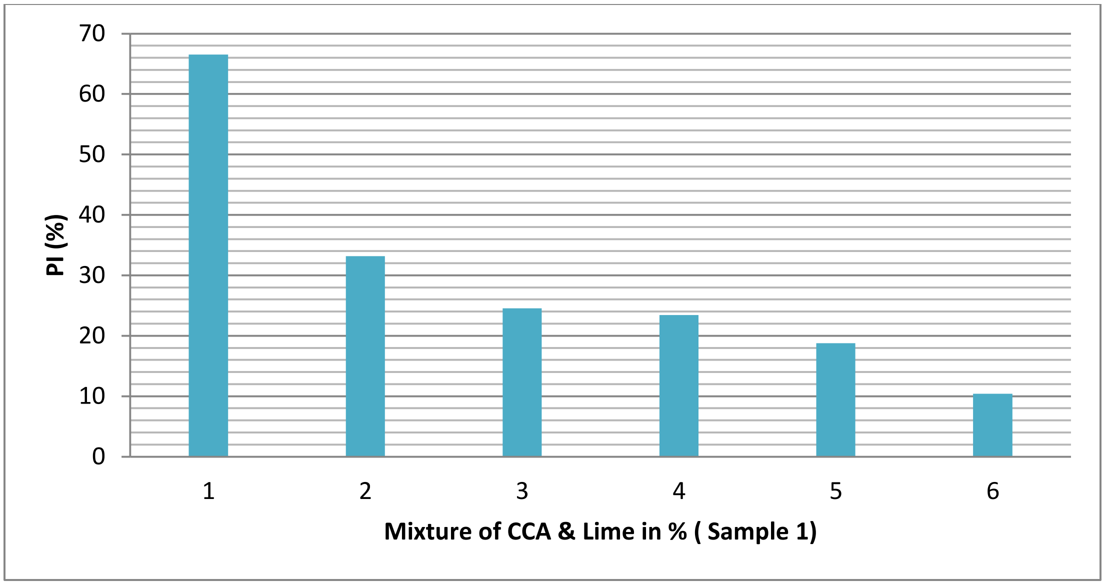


Fig 4.5: Plasticity index chart for treated & untreated soil Sample 1

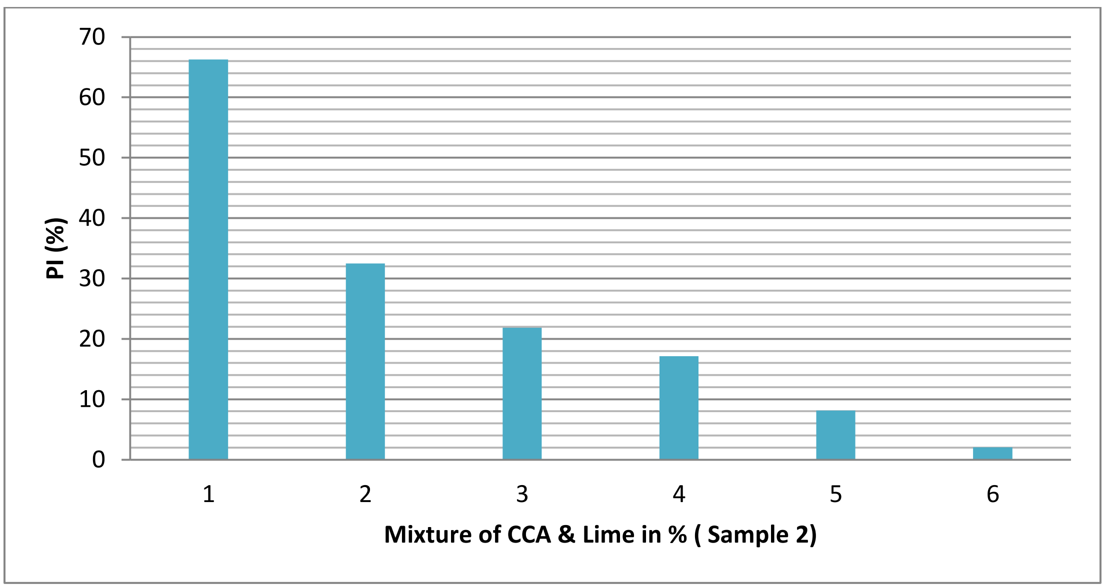


Fig 4.6: Plasticity index chart for treated & untreated soil Sample 2

The addition of lime to plastic soils causes a colloidal reaction which includes a replacement of naturally carried cations on clay surface by Ca 2+ cations, an increase in PH value and a reduction in double layer water [21] . This helps in flocculation and aggregation of colloidal clay particles, making them less plastic. The addition of CCA alone enhances the workability as a result of a reduction in the plasticity of the soil.

The addition of Corn Cob Ash alone had reduced the PI by about 20% and 21% when added at percentages of 8 % and 8 %, respectively. However, the combination of 2% CCA and 6% lime exhibited the highest effect on reducing the PI. The PI decreased from 66.52% to 10.42% from 66.28% to 2.03% respectively when 0% CCA and 8% lime mixtures were used, respectively.

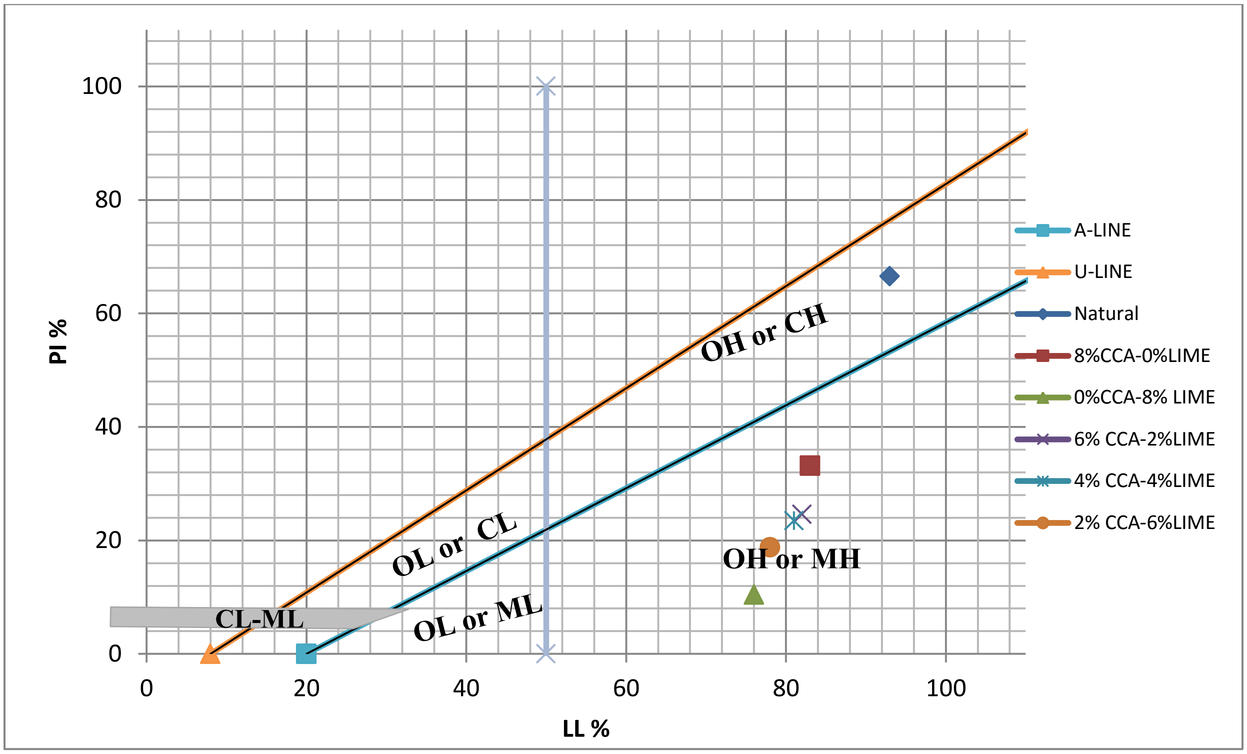


Fig 4.7: Casagrande plasticity chart for treated & untreated soil Sample 1

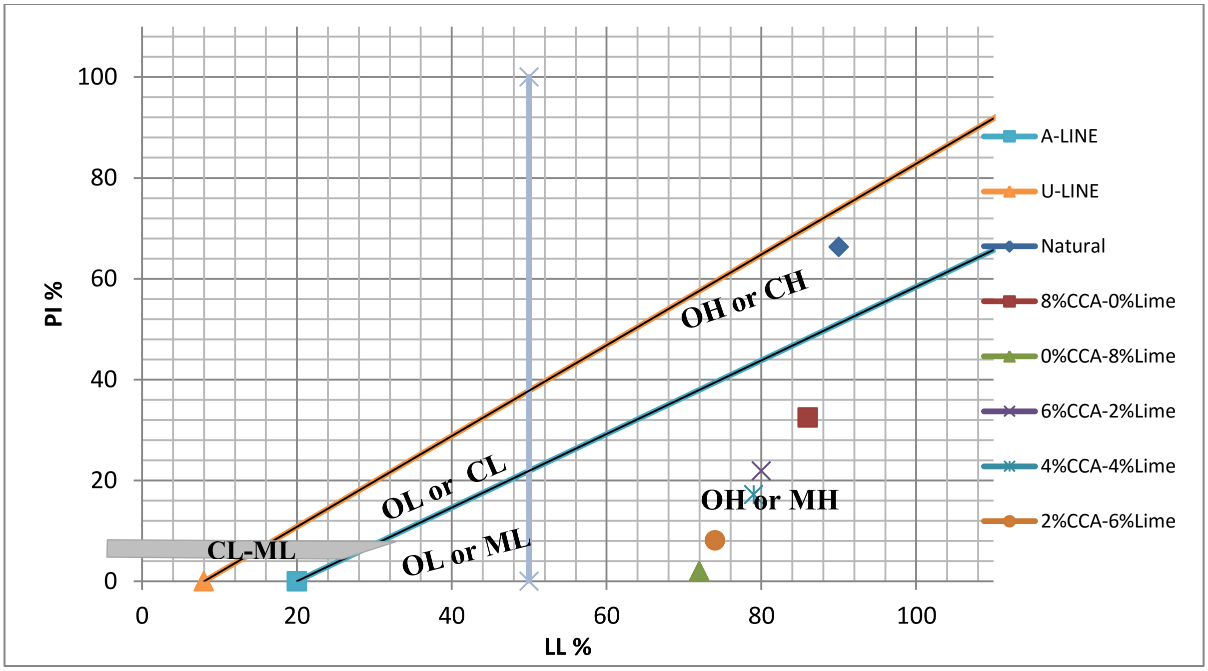


Fig 4.8: Casagrande plasticity chart for treated & untreated soil Sample 2

The Atterberg limits test results are plotted on the Casagrande plasticity chart in order to determine the soil classification in accordance with the USCS, as shown in Fig.4.7 & 4.8. It is clearly seen from the above figure that the soil is classified as CH class clay. After the addition of the mixtures, the soil falls in the class of MH soil.

All mixtures containing both CCA and lime moves the soil class from CH to MH. This renders the soil satisfactory for most construction operations even under severe environmental conditions.

4.3.2 Effects of the Mixture on Free Swell Index of soil

The free swell tests were conducted by mixing CCA with Lime at different proportion of percentages by dry weight of soil sample. The Effect of CCA and Lime on free swell index of the soil samples tabulated in Table 4.8 for both soil sample respectively.

Table 4.8: Effects of Corncob Ash mixed with Lime on Free Swell Index

Sample	**Mixture(%)**		**% FSI**	**IS 2720 Requirement**	**Remarks**
	CCA %	LIME %			
Sample 1	0	0	100	**FSI > 50 very high expansive**	Poor
	8	0	70		poor
	6	2	53		poor
	4	4	34		Satisfied
	2	6	24		Satisfied
	0	8	16		Satisfied
Sample 2	0	0	87	**FSI > 50 very high expansive**	Poor
	8	0	60		poor
	6	2	49		Satisfied
	4	4	44		Satisfied
	2	6	28		Satisfied
	0	8	19		Satisfied

The results showed that a significant reduction in the swell potential of the soil sample was observed by adding a different proportion of CCA and Lime. The free swell index value decreased from 100%, 87% to 16%, 19% for SAMPLE 1, and 2 soil samples, respectively, which was a significant change.

The result obtained guaranteed, the soil stabilized using CCA mixed with Lime show low degree of expansion as compared to untreated soil, as a result, the soil to have a free swell within the allowable requirements.

4.3.3. Effect of the Mixture of CCA & Lime on the Modified Compaction

The compaction tests were performed as per AASHTO T99-97 specifications outlined for modified Proctor compaction tests. Modified Proctor compaction tests were carried out on the soil-CCA-Lime mixture, Soil-CCA mixture and soil-Lime mixtures proportions. The compaction tests were performed for various combinations of soil-CCA-Lime mixtures as mentioned in Table 4.9. Fig 4.9 and 4.10 present the variation of MDD and OMC with different contents of CCA and Lime. The details of the test results are attached in Appendix.

The MDD and OMC have been found to increase with an increase in CCA dosage from 0% to 8%. Similar trends for lime, CCA and FA stabilized clay mixtures have been reported [24].

Table 4.9: Compaction test result of treated & untreated expansive clay soils

Sample No.	Mixture of CCA & Lime		MDD(g/cm^3)	OMC (%)
	CCA (%)	Lime (%)		
S #1	0	0	1.57	24.5
	8	0	1.49	25
	6	2	1.61	20
	4	4	1.62	18
	2	6	1.67	15
	0	8	1.76	13
S #2	0	0	1.54	24.0
	8	0	1.52	26.0
	6	2	1.56	23.5
	4	4	1.60	23.0
	2	6	1.65	19.0
	0	8	1.66	18.5

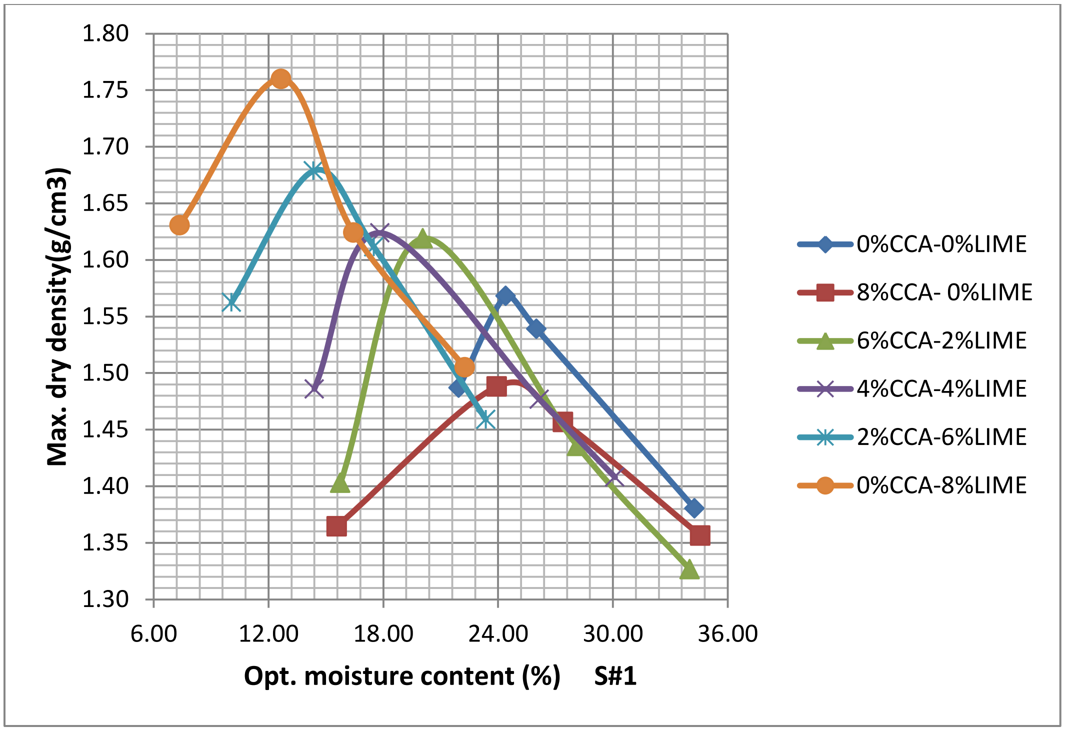


Fig 4.9: Summary of OMC and MDD of treated soil sample 1

It can be observed from Fig. 4.9 that OMC increases with the increase in CCA in mix mode stabilization. The increase in OMC has been attributed to the increased rate of pozzolanic reaction between CCA and soil components. A higher increase in OMC in the case of CCA stabilized soil specimens has been ascribed to the extra water needed for higher fineness and subsequent enhanced hydration.

Again from Fig.4.10, with CCA alone in mix mode stabilization, there is a further reduction in the MDD and enhancement in OMC. Relatively lower specific gravity values of CCA might be the causes for the reduction in MDD values and similar trend was observed by [32][13].

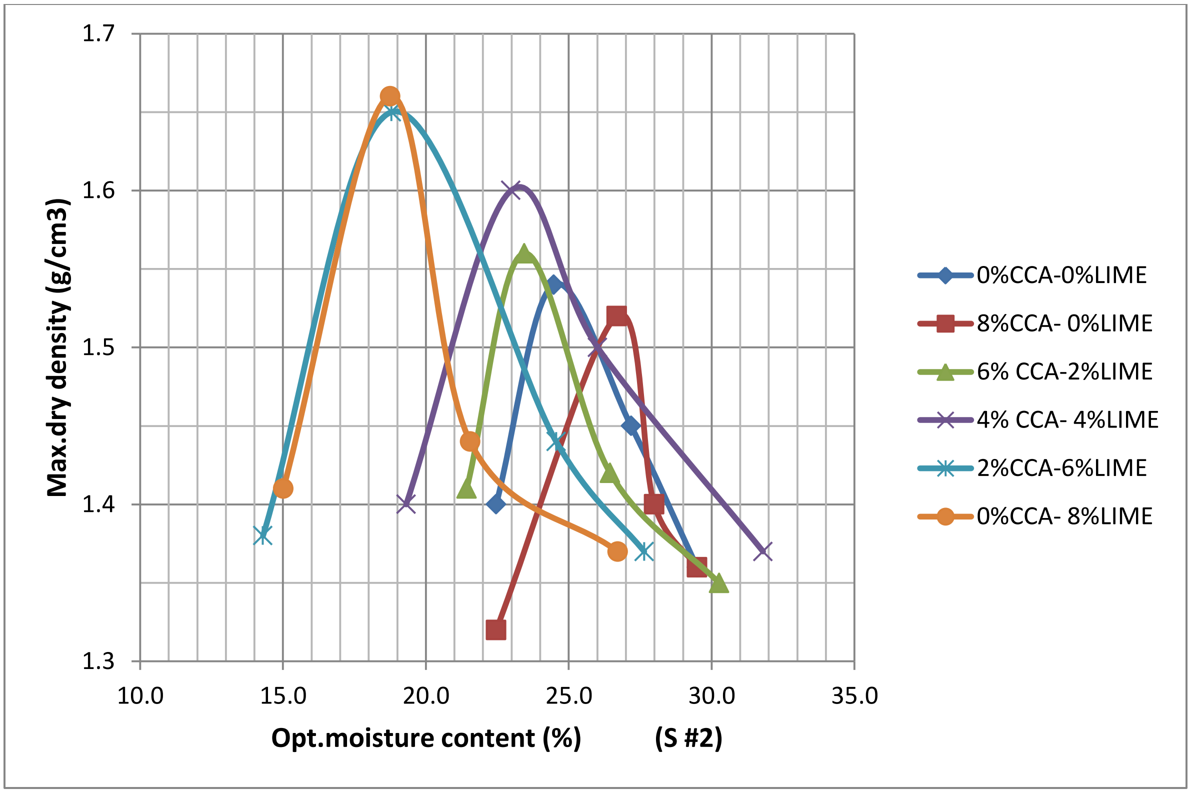


Fig 4.10 : Summary of OMC and MDD of treated soil sample 2

It is also observed that with increase in lime dosage in the soil samples, there is further decrease in OMC values. The fall in density is due to the quick reaction of lime with the soil which alters the base exchange aggregation and flocculation, thereby resulting in increased void ratio of the mixture. This subsequently leads to increases in the density of the overall mixture.

4.3.4 Effect of the Mixture On the California Bearing Ratio

The results of the CBR tests for varying percentage of CCA and Lime mixtures are represented by load versus penetration graphs and are plotted in Figure shown on Appendix for both conditions, respectively. The details of the laboratory results are attached in Appendix.

During the construction of roads, the strength of soils to be used is usually evaluated by their CBR values. The soaked CBR of the BC soil used in this study is 1.24 & 1.54%. Sub-grade materials having a CBR value of less than 3% needs a special treatment (ERA, 2013). According to AASHTO M 145 (1991), the BC soil used in this study is classified as A-7-5, in silt-clay groups. These groups range from fair to poor in quality to be used as a sub-grade material. After stabilization, it can be concluded that the CBR value increases as the percentage of CCA and Lime increases, as presented in Table 4.10.

Table 4.10: CBR values of treated and untreated soils

Sample	Mixture(%)		CBR value (%) 56 blows		Conditions	ERA Requirement	Remarks
	CCA %	LIME %	2.54	5.08			
S#1	0	0	1.24	1.15		>3%	Poor
	8	0	7.73	5.64			Satisfied
	6	2	8.11	7.94			Satisfied
	4	4	9.9	9.1			Satisfied
	2	6	11.63	9.44			Satisfied
	0	6	13.95	12.75			Satisfied
S#2	0	0	1.54	1.5		>3%	Poor
	8	0	6.22	5.64			Satisfied
	6	2	8.11	7.94			Satisfied
	4	4	11.39	9.6			Satisfied
	2	6	11.71	9.49			Satisfied
	0	8	14.7	14.28			Satisfied

The CBR values of soils treated with 8% CCA are found to be 5.64% and 5.64% in cases of both conditions, respectively. A similar trend was observed [33], it is observed that the stabilization of BC soil with CCA improves the CBR value, which is an indicator for the load carrying capacity improvement. The addition of CCA showed a significant improvement in both conditions. The improvement can be attributed to the reaction between the soil and CCA, Lime forming a cementitious material. The formation of this cementitious material bounds the particles together, by covering the soil grain and filling the inter-aggregate pores.

This increase was as a result of the formation of cementitious compounds, calcium-silicate hydrates (CSH) and calcium-aluminate-hydrates (CAH), by calcium from lime and the readily available silica and/or alumina from both the soil and Corncob ash. CSH and CAH are cementitious products similar to those formed in portland cement. They form the matrix that contributes to the strength of stabilized soil layers. Based on these results, it can be concluded that the BC soil treated with CCA-lime performs better as a sub-grade material, which requires lower thickness of the base course compared to the untreated BC soil. However, according to ERA pavement design manual specification, the CBR values of treated soil with 8% CCA alone is full fill the specification as a subgrade material in both soil sample.

4.3.5 CBR Swell of Treated And Untreated Stabilized Soils

The CBR swell test was also performed for varying combination of soil-lime-CCA mixtures. The CBR swell of untreated soil is found to be 9.83% and 8.11% for Sample 1 & 2 respectively, as mentioned in Table 4.11, which shows high swelling capacity. The amount of swelling and the magnitude of swelling pressure depend on the soil structure and fabric, the clay minerals presented in the soil, and the physio-chemical aspects of the soil such as cementation, cation valence, presence of organic matter and salt concentration.

Table 4.11: Swell from CBR test

% of Mixture		% CCA	0	8	6	4	2	0
		% Lime	0	0	2	4	6	8
Sample Name	Sample 1	% Swell	9.83	2.27	1.76	1.56	1.45	0.53
		Remarks	Poor	Poor	Satisfied			
	Sample 2	% Swell	8.11	2.33	1.58	0.9	1.22	0.74
		Remarks	Poor	Poor	Satisfied			

When the bottom layer of pavements is found to be a BC soil, it has a high risk of swelling and deformation, which will result in cracks on the surface of roads. Low-quality soils like BC soil with high swelling potential for subgrade construction can be improved through stabilization. The addition of CCA decreases the CBR swell with an increase in CCA dosage.

The usage of CCA as a stabilizing agent has also shown a significant change in CBR swell in addition to the CBR value. The decrease in CBR-swell of expansive soil is due cation exchange between the soil and the additives and flocculation and agglomeration of the soil particles. This is also due to replacement of some of the volume that is previously occupied by expansive clay minerals by Corncob ash. The formation of aggregations could account for the reduction of swell in expansive soils (Al-Rawas, 2002). Furthermore, the reduction in the swelling capacity of CCA-treated soil may also be related to the non-swelling property of CCA.

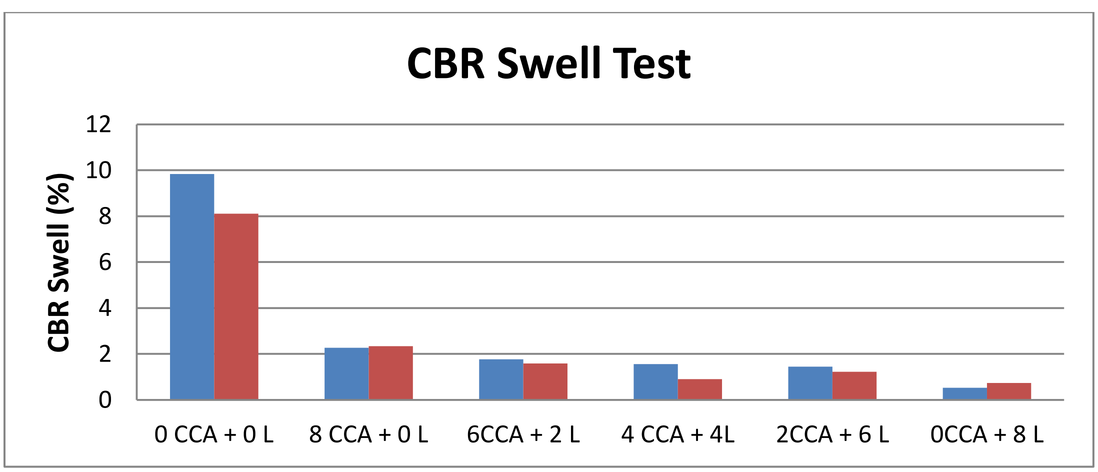


Fig 4.11: Summary of CBR Swell for Sample 1 and Sample 2 soil samples

4.3.6 Effect of the mixture on the mineralogy of the BC soil

XRD analysis has been carried out to identify the minerals present in BC soil, and investigate the effect of CCA and Lime on mineralogy of the soil. The XRD patterns of BC soil and soil treated with 0-8% CCA and 0-8% Lime are presented in Fig.12-16. The result indicates the presence of clay minerals, namely quartz and illite as similarly stated on different literatures [34]. The BC soil also contains some amount of montmorillonite mineral, which is responsible for the expansion character.

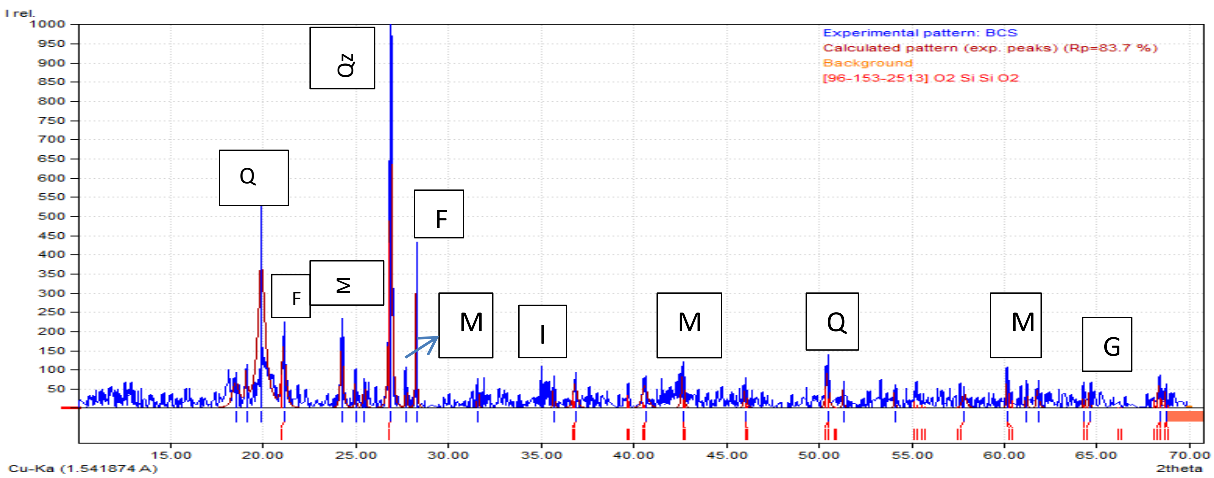


Fig 4.12: Mineralogical analysis by XRD of BC soil

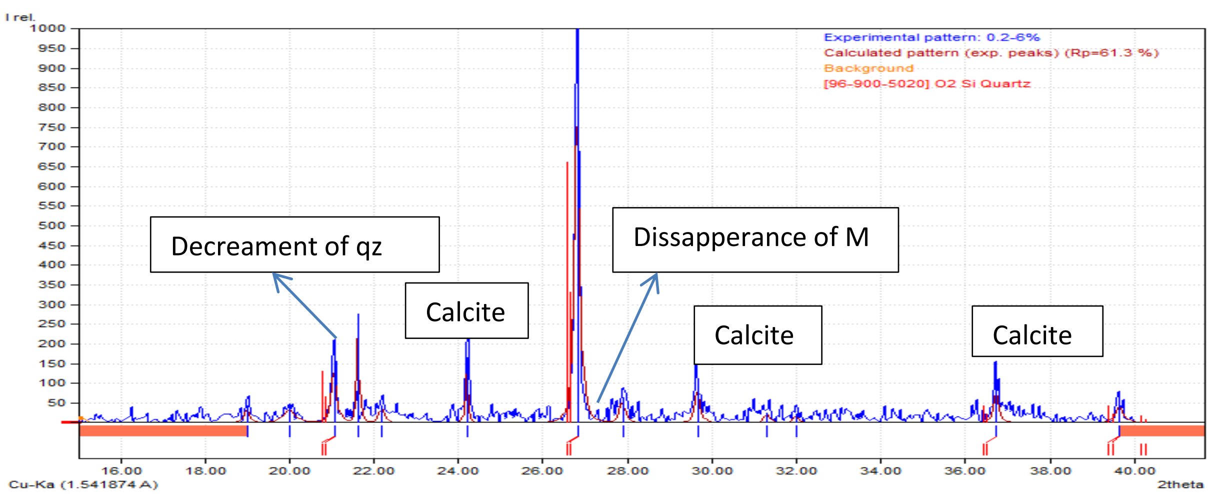


Fig 4.13: Mineralogical analysis by XRD of BC soil and samples treated with 2% CCA and 6% LIME.

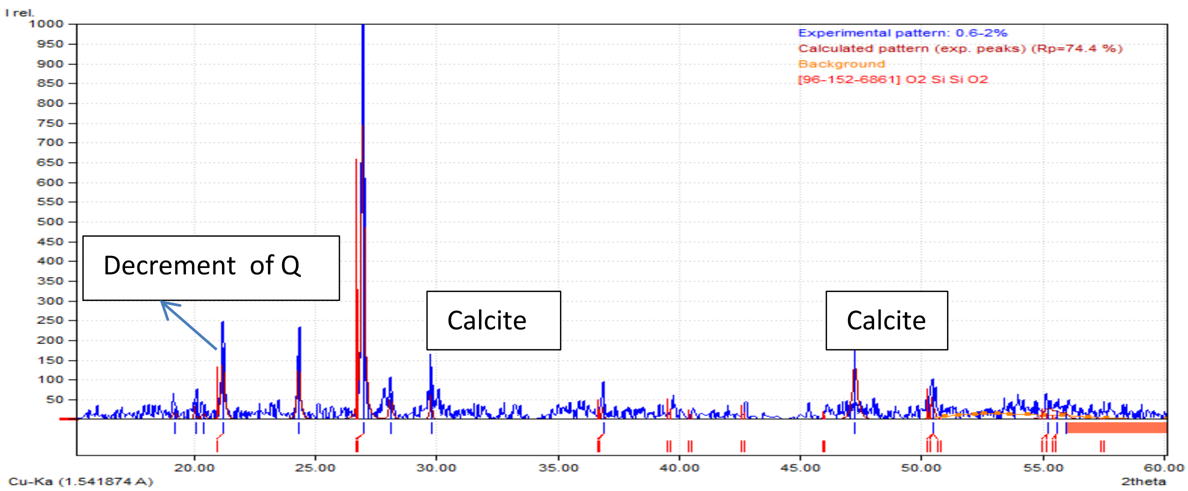


Fig 4.14: Mineralogical analysis by XRD of BC soil and samples treated with 6% CCA and 2% Lime.

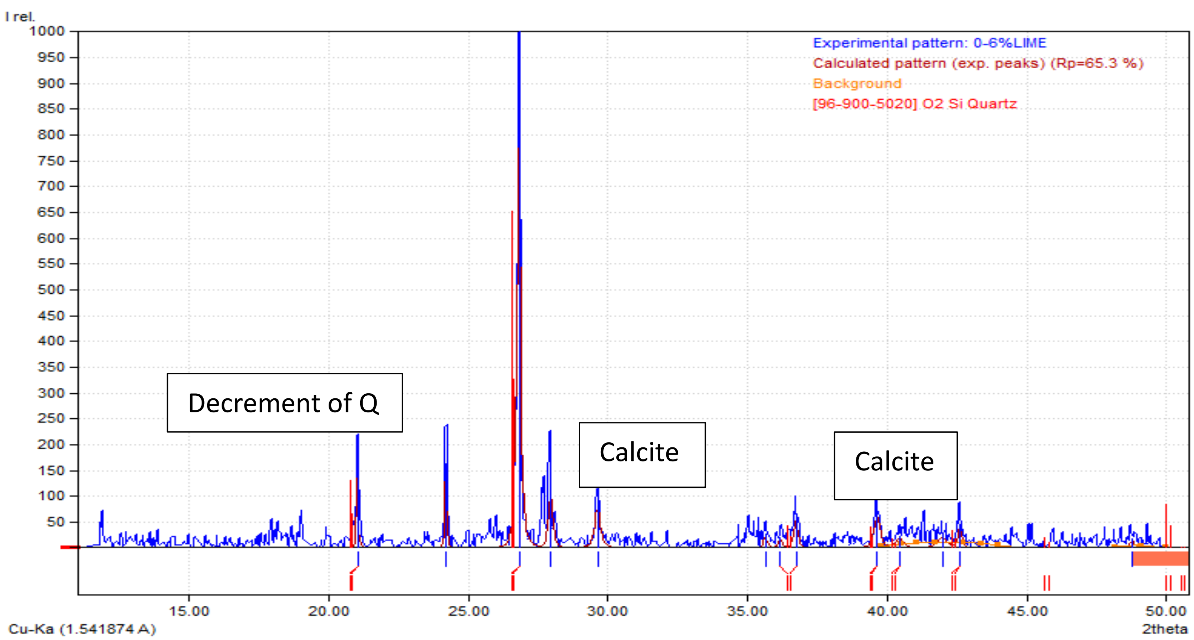


Fig 4.15: Mineralogical analysis by XRD of BC soil and samples treated with Lime Alone

The result of XRD shows the presence of elements such as Si, Al, Fe, Na and Mg in the BC soil. It indicates the presence of clay minerals such as montmorillonite (M), illite (I) and other non-clay minerals such as goethite (G), feldspar (F) and quartz (Q).

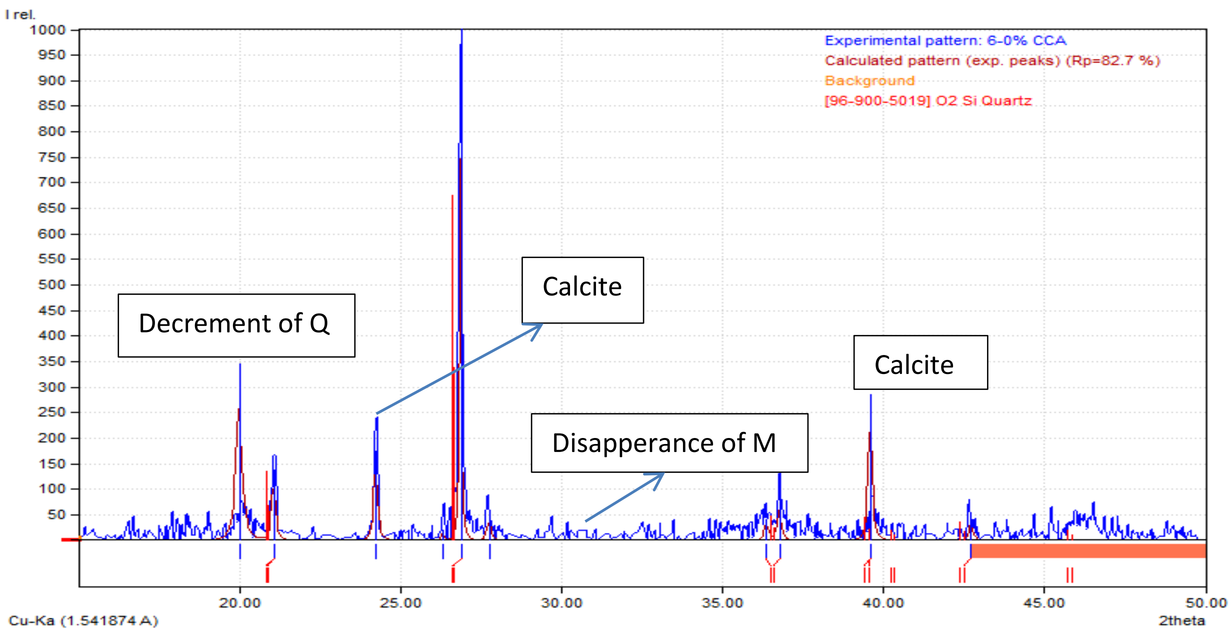


Fig 4.16: Mineralogical analysis by XRD of BC soil and samples treated with CCA Alone

XRD pattern shows the formation of stable compounds. The XRD pattern of CCA-treated soil shows the formation of calcite. In addition, reductions in the intensity of some minerals for CCA-LIME treated samples are also observed. For instance, a decrement in the intensity of quartz is observed for both samples treated with 8% Lime and 8% CCA [35]. Comparatively, XRD patterns indicated higher peaks of calcite for samples treated with 8% lime than that treated with 8% CCA. Apart from the reduction in peak intensity of quartz and the appearance of calcite, the disappearance of kaolinite is also observed in the samples treated with 6-2% CCA-LIME. These changes in XRD pattern and peak height could be attributed to the reactions between clay mineral and additive, these findings are consistent with the results that were presented by [34].

4.4 Analysis of the Appropriate Percentage of Corncob Ash And Lime, And Recommended Mixture

As observed from all testes was performed under this study, the maximum results were achieved at 2% CCA and 6% Lime by weight. Since most parameters achieve the ERA requirement and have got maximum strength or CBR value. The optimum percentage for the studied weak subgrade soils is at 2% CCA and 6% Lime. The change in the soil structure is a consequence of cation exchange caused by dissociated bivalent calcium ions in the pore water replacing univalent cations that are normally attached to the negatively charged individual expansive soil particles. The expansive clay undergoes major transformations in its structure when mixed with lime, these findings are consistent with the results that were presented by [21]. Flocculation and coagulation contribute to bring several expansive soil particles together to form larger sized aggregates.

5. Conculision & Recommendation

5.1 Conculision

This study aims to investigate the effect of the mixture of CCA-Lime on mineralogy, compressibility and strength characteristics of BC soil. To examine these properties, Atterberg Limits, Modified compaction, Free Swell index , CBR tests and finally XRD are conducted on untreated soil and treated soil with different percentages (0%, 2%, 4%, 6% and 8%) of CCA And Lime mixtures. The following conclusions are drawn:

- The General soil classification systems AASHTO and USCS shows for both samples are A-7-5 and CH respectively, this indicates that the soils are poor and unsuitable to use as a sub grade road construction material unless it improved.

- The chemical composition of Corncob ash test result indicates the combined percent composition of main oxides (SiO_2 + Al_2O_3 + Fe_2O_3) was 78.12% which is above the minimum of (70%) specified by ASTM C-618, which is acceptable as a good Pozzolana.

- The addition of Corncob ash reduces the plasticity characteristics of expansive soils. The Plasticity index (PI) decreased from 66.28% to 10% and from 66.58% to 2% Sample 1 and 2 soil sample respectively. The liquid limit decreases and the plastic limit increases with an increase in CCA content. The Liquid limit (LL) decreases with slight changes for both soil samples from control value 93%-76% and 90%-72% for Sample 1 and 2 soil sample respectively. However, the additives not shown significant change on liquid limit of the soil.

- The Plastic limits (PL) increases with stabilization of additives of both Mix-ratio from 26.48% to 65.58% and 23.72% to 69.97% for Sample 1 and 2 soil sample respectively, however Plastic limits of both soil samples radically increase when the mix-ratio of Lime increases rather than CCA.

- The free swell index of expansive soils can be effectively reduced by the addition of CCA. For the expansive soil used, the FSI was reduced by about 50% by adding 8% CCA.
- The MDD shows a slight increase and OMC shows a decrease in the treatment of weak subgrade soil with CCA-Lime additive agents. For Sample 1, MDD increases from 1.57g/cm^3 to 1.67 g/cm^3 and OMC decreases from 24.5% to 15.0%. For Sample 2, MDD increases from 1.54 g/cm^3 to 1.65 g/cm^3 and OMC decreases from 24% to 19% at optimum mix- ratio of at (2%CCA + 6%Lime). Generally, when increasing the percentage of Lime in CCA-Lime mix-ratio led increase in the maximum dry density and decrease optimum moisture content rather than CCA.

- For the CBR test, there was an initial increase from the control value of 1.24% to 11.63% for Sample 1 and 1.54% to 11.71% for Sample 2, at (2%CCA + 6%Lime). This was followed by a decline at increasing levels of percentage of CCA in CCA-Lime mix-ratio. However, all mix ratios proportions satisfied the minimum requirements as per ERA specification used as a road subgrade material.

- For CBR swell, all mix proportion percentages of Corncob ash-Lime for both soil samples are met the requirement specified by ERA pavement design manual as criterion for suitable material. However, CCA alone is improving plastic index, free swell index and CBR swell of highly plastic clay soils slightly. However, CCA mixed with Lime can effectively stabilize this Weak subgrade soil.

- As observed from all testes was performed under this study, the maximum results were achieved at 2% CCA and 6% Lime by weight. Since most parameters achieve the ERA requirement and have got maximum strength or CBR value.
- The XRD results show a general reduction in peak intenstites in all clay minerals that are present in the soil and most significantly in montmorillonite. But there is no correlation between the reduction in peak intensties and the swelling nature of the expansive soil.

From the result of stabilization analysis, for most cases Lime was better than CCA in improving sub grade soil properties although the stabilization of BC soil using CCA improves the bearing capacity, swelling properties and compressibility behavior of the BC soil. In addition, the potential use of this waste material as a stabilization agent has a positive effect on improving the Engineering properties of BC soil, and furthermore, it can address the associated disposal problems and environmental concerns.

5.2 Recommendation

- Microscopic Analysis using SEM & EDX should be taken to show the presence of C-S-H & C-S-A-H in both CCA and Lime treated clay soils. Since these cementitious phases induced significant improvement in the Engineering Properties of the investigated clayey soil, such as Workability, Compaction, Strength and Shrinkage.

- Further soil sample should be necessary to come up with good results. It is recommended to conduct stabilization by taking large number of samples characterizing the whole study area. Therefore, the findings should be considered as indicative.
- The impact of the stabilization on the construction cost comparing with other method of stabilization techniques should be studied further.
- It is recommended that additional parameters of Curing time effect, Evaluation of the effect of moisture and Temperature on optimum stabilizers ratio shall be considered should also be performed to have more realistic results.
- These study was conducted by mixing of CCA and lime at different percentages of mixing ratio and at lime and CCA alone at maximum ratio of (8%). It is recommended to the next investigator to conduct test at different percentages of CCA alone.

REFERENCES

[1] M. Dixit, "Damage mechanism in problematic soils," no. October, 2016.

[2] M. Aziz, M. Saleem, and M. Irfan, "Engineering behavior of expansive soils treated with rice husk ash," no. February, 2015.

[3] Y. Xu, "Bearing capacity of unsaturated expansive soils," pp. 611–625, 2004.

[4] B. U. Uge, "Expansive soils in Ethioipa : a Review," vol. 6, no. 5, 2017.

[5] R. M. Tailor and N. C. Shah, "Performance Improvement of Flexible Pavement on Swelling Subgrade Using Geotextile," *J. Traffic Logist. Eng.*, 2014.

[6] S. Patel and J. T. Shahu, "Engineering properties of black cotton soil-dolime mix for its use as subbase material in pavements," *Int. J. GEOMATE*, 2015.

[7] J. D. Rogers, R. Olshansky, and R. B. Rogers, "DAMAGE TO FOUNDATIONS FROM EXPANSIVE SOILS."

[8] R. M. Tailor and N. C. Shah, "APPLICATIONS OF INNOVATIVE MATERIALS FOR PERFORMANCE IMPROVEMENT OF FLEXIBLE PAVEMENT," vol. 8, no. 1, pp. 1197–1202, 2015.

[9] J. I. Adams and O. Hydro, "Foundations on expansive soils," pp. 135–136.

[10] J. D. Nelson, K. C. Chao, D. D. Overton, and E. J. Nelson, *Foundation Engineering for Expansive Soils*. 2015.

[11] "ALEMAYU TEFERA .Soil Mechanics.1999."

[12] B. Shi, H. Jiang, Z. Liu, and H. Y. Fang, "Engineering geological characteristics of expansive soils in China," vol. 67, pp. 63–71, 2002.

[13] H. Runqiu and W. U. Lizhou, "Stability Analysis of Unsaturated Expansive Soil Slope," vol. 14, no. 6, pp. 129–133, 2007.

[14] E. R. Authority, "Site Investigation Manual",2002.

[15] J. Jibril, "In-depth Investigation into Engineering Characteristics of Jimma Soils In-depth Investigation into Engineering Characteristics of Jimma Soils By," no. October 2014, 2017.

[16] B. U. Uge, "Investigation on the Swelling characteristics and unsaturated shear strength of expansive soils from Arba Minch in Ethiopia," vol. 2, no. 2, pp. 51–62, 2016.

[17] F. A. Yitagesu, "Spectroscopy to derive Engineering Parameters of Expansive Soils Expansive Soils," 2006.

[18] M. Mirzababaei and S. Yasrobi, "Effect of polymers on swelling potential of expansive soils," no. August, pp. 111–119, 2009.

[19] P. Taylor, A. R. Estabragh, M. R. S. Pereshkafti, B. Parsaei, and A. A. Javadi, "International Journal of Pavement Engineering Stabilised expansive soil behaviour during wetting and drying," no. December 2012, pp. 37–41.

[20] C. Mayowa, "Crystal and Microstructure Analysis of Pozzolanic Properties of Bamboo Leaf Ash and Locust Beans Pod Ash Blended Cement Concrete," 2016.

[21] S. A. A. Khattab and J. Fleureau, "Long-Term Stability Characteristics of a Lime-Treated," vol. 19, no. 4, pp. 358–366, 2007.

[22] C. R. Devi, S. Surendhar, P. Vijaya Kumar, and M. Sivaraja, "Bottom ASH as an Additive Material for Stabilization of Expansive Soil," *Int. J. Eng. Tech.*, 2018.

[23] B. Raju, "Role of amount and type of clay in the lime stabilization of soils," pp. 37–45, 2000.

[24] B. R. P. Kumar, R. S. Sharma, and M. Asce, "Effect of Fly Ash on Engineering Properties of Expansive Soils," no. July, pp. 764–767, 2004.

[25] M. M. E. Zumrawi and O. S. M. Hamza, "Improving the Characteristics of Expansive

Subgrade Soils Using Lime and Fly Ash," vol. 3, no. 12, pp. 1124–1129, 2014.

[26] M. Wubshet and S. Tadesse, "STABILIZATION OF EXPANSIVE SOIL USING BAGASSE ASH & LIME," vol. 32, no. December, pp. 21–26, 2014.

[27] P. M. M. Reddy, P. Thanmai, N. T. Kumar, C. Engineering, K. O. R. M. C. Engineering, and A. Pradesh, "Stabilization of Expansive Soils by Using Corncob Ash," no. June, pp. 5–8, 2018.

[28] S. T. Method, "Standard Test Method for Particle-Size Analysis of Soils 1," vol. 63, no. Reapproved, 1998.

[29] "AASHTO T (193-93) California Bearing Ratio.pdf." 2000.

[30] K. Flogeac *et al.*, "Characterization of soil particles by X-ray diffraction (XRD), X-ray photoelectron spectroscopy (XPS), electron paramagnetic resonance (EPR) and transmission electron microscopy (TEM) To cite this version : HAL Id : hal-00886300 Characterization of soil particles by X-ray diffraction (XRD), X-ray photoelectron spectroscopy (XPS), electron paramagnetic resonance (EPR) and transmission electron microscopy (TEM)," 2005.

[31] A. Al-Taie, M. M. Disfani, R. Evans, A. Arulrajah, and S. Horpibulsuk, "Swell-shrink Cycles of Lime Stabilized Expansive Subgrade," in *Procedia Engineering*, 2016, vol. 143.

[32] J. P. Sahoo, "Effect of Lime Stabilized Soil Cushion on Strength Behaviour of Expansive Soil," pp. 889–897, 2010.

[33] A. Roy, "Soil Stabilization using Rice Husk Ash and Cement," vol. 5, no. 1, pp. 49–54, 2014.

[34] M. K. Atahu, F. Saathoff, and A. Gebissa, "Strength and compressibility behaviors of expansive soil treated with coffee husk ash," *J. Rock Mech. Geotech. Eng.*, vol. 11, no. 2, 2019.

[35] S. Sathyapriya and P. D. Arumairaj, "Micro fabric and mineralogical studies on the stabilization of expansive soil using cement industry wastes," vol. 45, no. June, pp. 807–815, 2016.

[36] ASTM C618 (2003). Standard specification for coal fly ash and raw or calcined naturalpozzolan for use in concrete. American society for testing and materials, ASTM International: West Conshohocken, PA.

APPENDIX

Appendix A: Laboratory Test Result of Soil Sample 1

1. Moisture Content

Sample Name	Mendera Kochi		
Container No	T4C2	T2	DH
Mass of container, g	18.14	17.56	16.97
Mass of container + wet soil, g	65.66	52.39	34.45
Mass of ontainer + Dry soil, g	51.11	41.78	29.22
Mass of Water, g	14.55	10.61	5.23
Mass of Dry soil, g	32.97	24.22	12.25
Water content, %	44.13	43.81	42.69
Average Water Content	43.54		

2. Wet Sieve Analysis

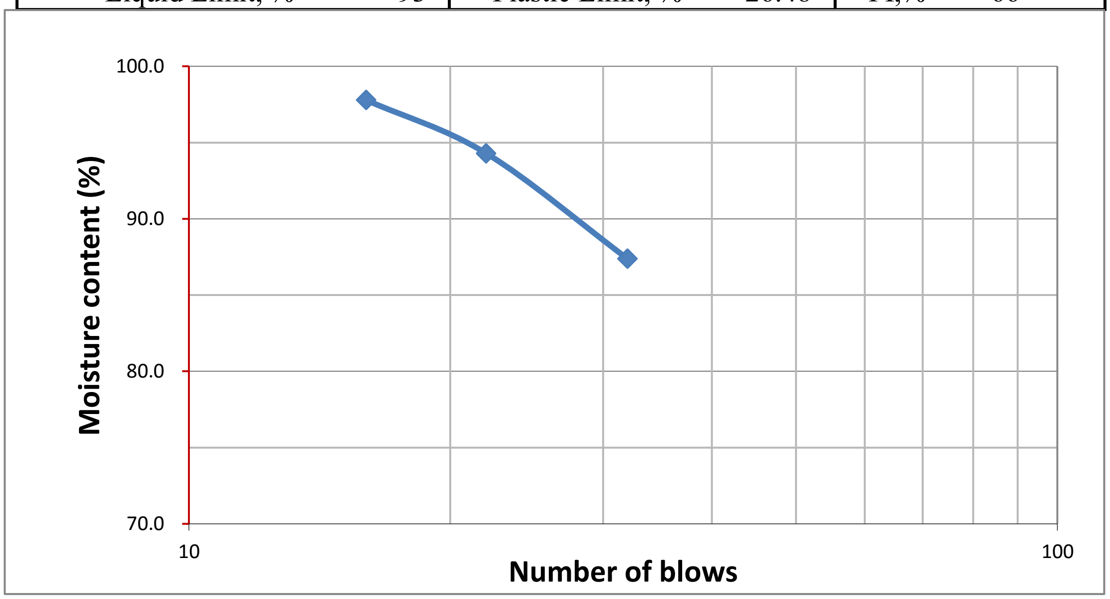
Test Pit	Sieve Size (opening)	Mass retained in g	Percentageretained	Cumulative percentage retained	Percentage finer particle
1 @1.5m	9.5	0	0	0	100
	4.75	4.2	0.42	0.42	99.58
	2	44.2	4.42	4.84	95.16
	0.85	19.8	1.98	6.82	93.18
	0.425	11.2	1.12	7.94	92.06
	0.3	3.9	0.39	8.33	91.67
	0.15	6.9	0.69	9.02	90.98
	0.075	5.4	0.54	9.56	90.44
	pan				
		Course	0.42		
		Sand	9.14		
		Silt & Clay	90.86		

3. Hydrometer Analysis

Elapsed time (min)	temprature	actual hyd. Reading	L	K	D	Ct	a	Corrected hyd. Reading (Rc)	persentage of finer %	corrected (Pa)
0.5	20	40	9.7	0.01317	0.0580	0.15	0.97435	40.65	79.21	76.41
1	20	40	9.7	0.01317	0.0410	0.15	0.97435	33.65	65.57	63.25
2	20	40	9.7	0.01317	0.0290	0.15	0.97435	31.65	61.68	59.49
4	20	34	10.7	0.01317	0.0215	0.15	0.97435	27.65	53.88	51.97
8	20	33	10.9	0.01317	0.0154	0.15	0.97435	26.65	51.93	50.09
15	20	32	11.1	0.01317	0.0113	0.15	0.97435	25.65	49.98	48.21
30	20	31	11.2	0.013018	0.0080	0.15	0.97435	24.65	48.04	46.34
60	20	30	11.4	0.013018	0.0057	0.15	0.97435	23.65	46.09	44.46
120	21	29	11.5	0.012868	0.0040	0.4	0.97435	22.9	44.63	43.05
240	22	27	11.9	0.013018	0.0029	0.65	0.97435	21.15	41.22	39.76
480	21	25	12.2	0.013018	0.0021	0.4	0.97435	18.9	36.83	35.53
960	21	24	12.4	0.01317	0.0015	0.4	0.97435	17.9	34.88	33.65
1440	21	23	12.5	0.01317	0.0012	0.4	0.97435	16.9	32.93	31.77

4. Atterberg limit

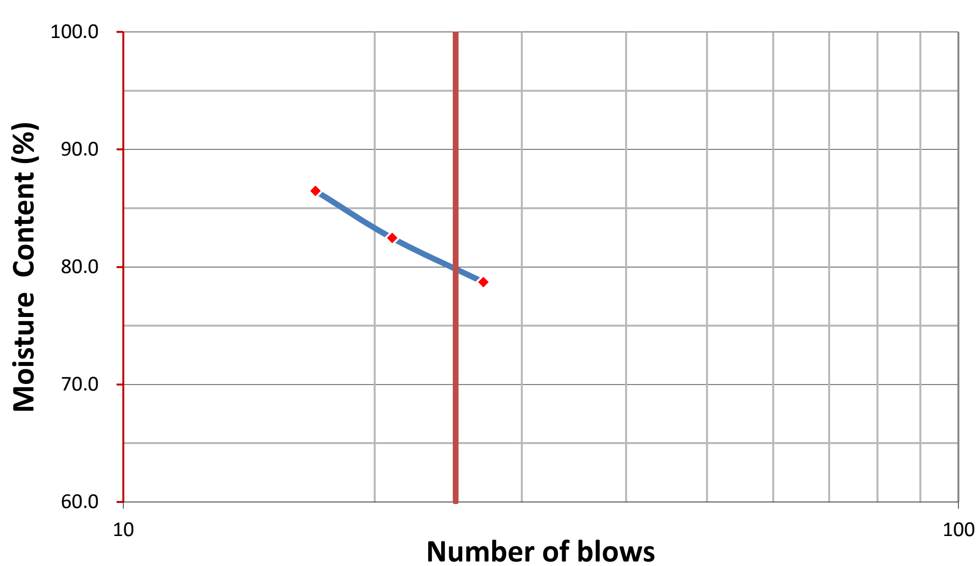
	0 - 0 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	G-2	G-3	I2	I1	I2
Mass of container, g	14.94	19.64	8.24	6.42	6.49
Mass of container + Wet soil, g	29.56	35.397	23.43	13.18	14.735
Mass of container + Dry soil, g	22.74	27.75	15.92	11.78	12.99
Mass of water, g	6.82	7.65	7.51	1.40	1.75
Mass of dry soil, g	7.81	8.11	7.68	5.36	6.50
Water content, %	87.38	94.29	97.79	26.12	26.85
No of blows	32	22	16	------	------
Liquid Limit, % = 93	Plastic Limit, % = 26.48			PI,% = 66	



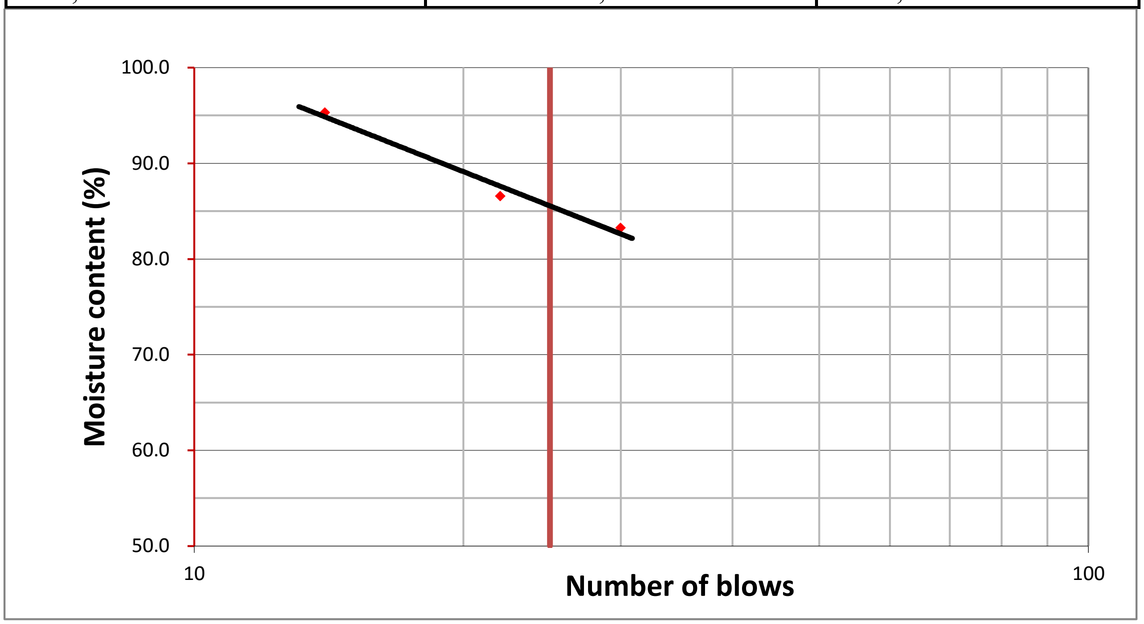
	8 - 0 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	I-I	P5	3	A1	A2
Mass of container, g	17.48	17.15	17.17	17.94	17.38
Mass of container + Wet soil, g	34.66	33.71	32.64	32.96	30.82
Mass of container + Dry soil, g	27.08	26.26	25.29	27.8	26.5
Mass of water, g	7.58	7.45	7.35	5.16	4.32
Mass of dry soil, g	9.60	9.11	8.12	9.86	9.12
Water content, %	78.96	81.78	90.52	52.33	47.37
No of blows	34	24	14	------	------
Liquid Limit, % = 83	Plastic Limit, % = 49.85			PI % = 33	

	2 - 6 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	Y1	Y2	Y3	21-I	22-I
Mass of container, g	6.34	6.21	5.54	6.34	6.45
Mass of container + Wet soil, g	21.44	23.61	20.42	17.93	17.69
Mass of container + Dry soil, g	15.00	15.92	13.66	13.28	13.87
Mass of water, g	6.44	7.69	6.76	4.65	3.82
Mass of dry soil, g	8.66	9.72	8.12	6.94	7.42
Water content, %	74.36	79.16	83.24	67.00	51.48
No of blows	36	24	15	------	------
Liquid Limit, % = 78	Plastic Limit, % =		59.24	PI, %=	19

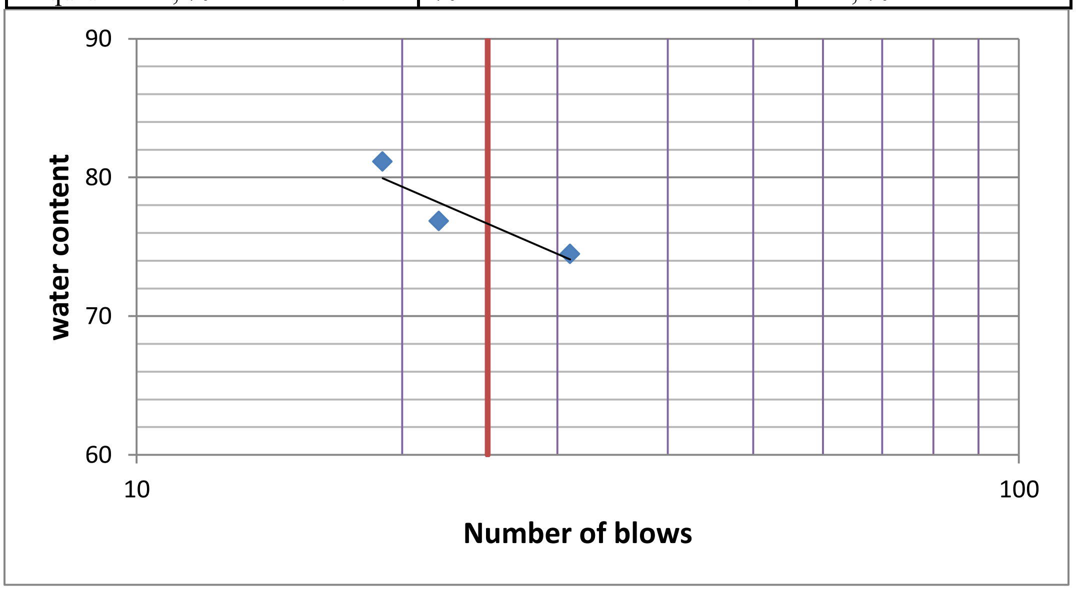
	4 - 4 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	3-1	3-2	3-3	31-1	32-2
Mass of container, g	5.45	5.97	24.39	5.93	16.27
Mass of container + Wet soil, g	19.80	22.5	39.4	15.64	24.87
Mass of container + Dry soil, g	13.48	15.03	32.44	11.36	22.58
Mass of water, g	6.32	7.47	6.96	4.28	2.29
Mass of dry soil, g	8.03	9.06	8.05	5.43	6.31
Water content, %	78.70	82.45	86.46	78.82	36.29
No of blows	27	21	17	------	------
Liquid Limit, % = 81	Plastic Limit, %		57.56	PI, %= 23	



	6 - 2 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	4-1	4-2	4-3	4-11	4-22
Mass of container, g	17.61	17.66	17.7	6.18	23.33
Mass of container + Wet soil, g	33.46	32.1	33.03	21.3	34.4
Mass of container + Dry soil, g	26.26	25.4	25.55	15.6	30.5
Mass of water, g	7.20	6.70	7.48	5.70	3.90
Mass of dry soil, g	8.65	7.74	7.85	9.42	7.17
Water content, %	83.24	86.56	95.29	60.51	54.39
No of blows	30	22	14	------	------
LL, % = 82	Plastic Limit, % = 57.45			PI, % = 25	



	0 - 8 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	A1	A2	A3	B1	B2
Mass of container, g	17.48	17.91	17.59	5.81	5.86
Mass of container + Wet soil, g	34.79	**33.58**	**31.9**	**17.91**	**18.1**
Mass of container + Dry soil, g	27.40	**26.77**	**25.49**	**13.64**	**12.79**
Mass of water, g	7.39	6.81	6.41	4.27	5.31
Mass of dry soil, g	9.92	8.86	7.90	7.83	6.93
Water content, %	74.50	76.86	81.14	54.53	76.62
No of blows	31	22	19	------	------
Liquid Limit, % = 76	Plastic Limit, % = 65.58			PI, % = 10	



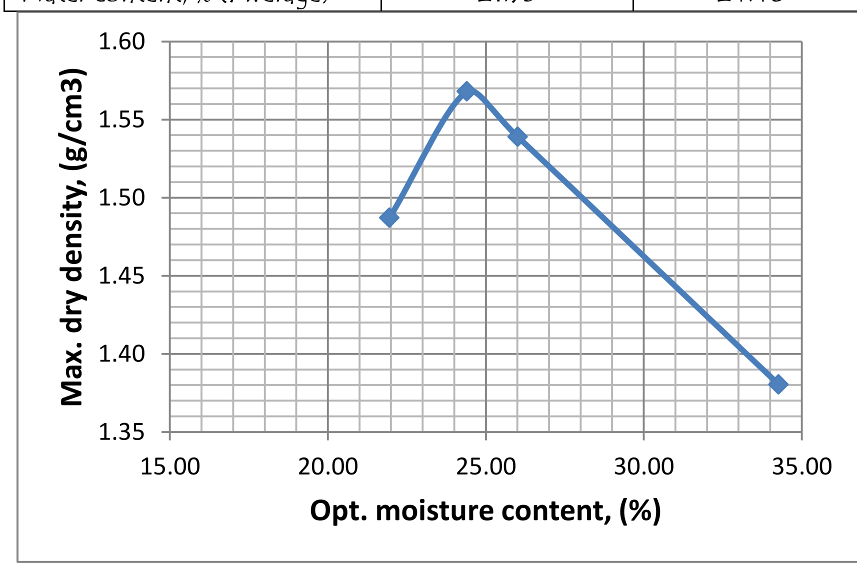
5. Compaction Test Result

5.1 0 - 0 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	10275.6	10558.5	10535	10358.2
Mass of Compacted soil, g	3740.2	4023.1	3999.6	3822.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.81	1.95	1.94	1.85
Water Contenet, %	21.95	24.40	26.01	34.27
Dry density, g/cm3	1.49	1.57	1.54	1.38

Water Content

Trial	1		2		3		4	
Conatainer Code	A1C	A18	A13	C19	M	M2	P15	J41
Mass of container, g	49.4	28.7	36.5	37.8	39.9	34.4	33.4	32.3
Mass of container + wet soil, g	213.7	193.6	173.7	206	159	246	169.5	187.7
Mass of ontainer + Dry soil, g	188.5	159.8	147.2	173	132	208	134.8	148
Mass of Water, g	25.2	33.8	26.5	33.5	27.6	38	34.7	39.7
Mass of Dry soil, g	139.1	131.1	110.7	135	91.6	174	101.4	115.7
Water content, %	18.12	25.78	23.94	24.87	30.13	21.89	34.22	34.31
Water content, % (Average)	21.95		24.40		26.01		34.27	

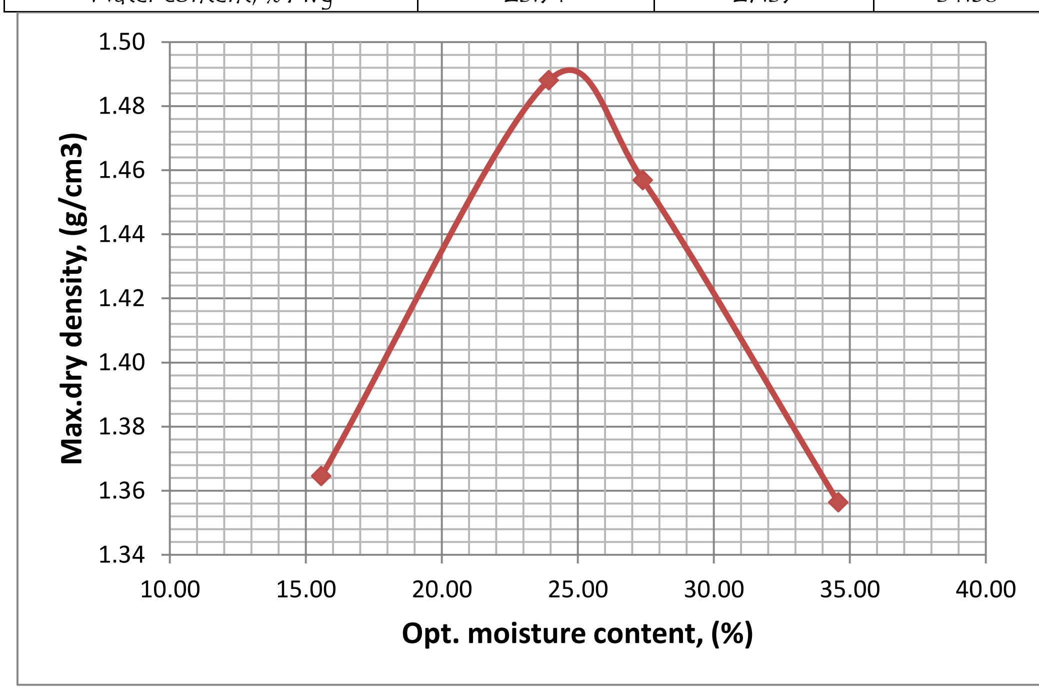


5.2 8 - 0 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6533.6	6533.6	6533.6	6533.6
Mass of mold + Compacted Soil, g	9785.4	10336.3	10360.5	10297.4
Mass of Compacted soil, g	3251.8	3802.7	3826.9	3763.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.58	1.84	1.86	1.83
Water Contenet, %	15.57	23.94	27.39	34.58
Dry density, g/cm3	1.36	1.49	1.46	1.36

Water Content

Trial	1		2		3		4	
Mass of container, g	17.4	17.1	17.2	17.6	17.1	17.7	35	35
Mass of container + wet soil, g	139.9	124.9	100.4	93	99.7	89.4	223	218
Mass of ontainer + Dry soil, g	115	105.2	83.4	76	78.8	70.7	196	195
Mass of Water, g	24.9	19.7	17	17	20.9	18.7	27	23
Mass of Dry soil, g	97.6	88.1	66.2	58.4	61.7	53	161	160
Water content, %	25.51	22.36	25.68	29.11	33.87	35.28	16.77	14.38
Water content, % Avg	23.94		27.39		34.58		15.57	

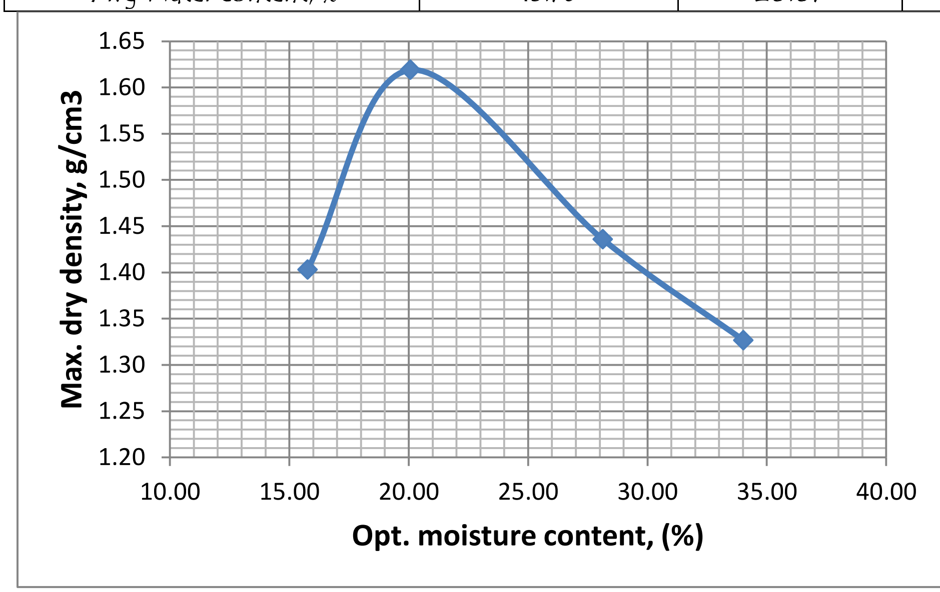


5.3 6 - 2 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	9885	10544.5	10329.3	10202.2
Mass of Compacted soil, g	3349.6	4009.1	3793.9	3666.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.62	1.94	1.84	1.78
Water Contenet, %	15.76	20.07	28.14	34.03
Dry density, g/cm3	1.40	1.62	1.44	1.33

Water Content

Trial	1		2		3		4	
Conatainer Code	GS3	T2C2	1A	T1C1	P1	G3T2	205	T2C1
Mass of container, g	17.5	17.6	17.8	17.5	17.8	17.7	17.7	17
Mass of container + wet soil, g	110	99.1	120.7	110.6	129.5	122.6	106.1	113
Mass of ontainer + Dry soil, g	106.8	81.3	107.4	91.8	112.7	93.4	83.5	88.5
Mass of Water, g	3.2	17.8	13.3	18.8	16.8	29.2	22.6	24.1
Mass of Dry soil, g	89.3	63.7	89.6	74.3	94.9	75.7	65.8	71.5
Water content, %	3.58	27.94	14.84	25.30	17.70	38.57	34.35	33.71
Avg Water content, %	15.76		20.07		28.14		34.03	

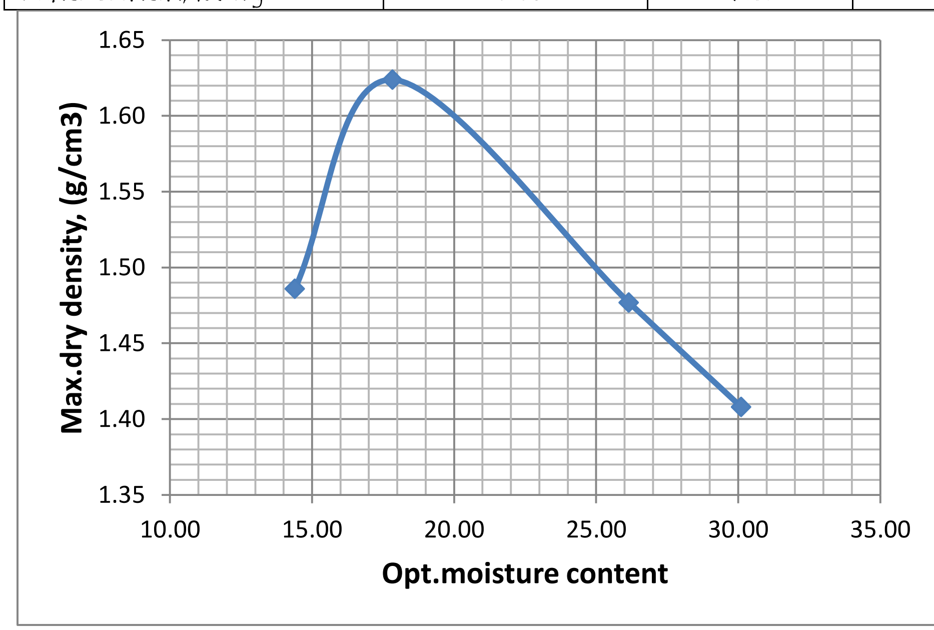


5.4 4 - 4 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	10040.5	10480.8	10377.3	10312.7
Mass of Compacted soil, g	3505.1	3945.4	3841.9	3777.3
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.70	1.91	1.86	1.83
Water Contenet, %	14.40	17.83	26.16	30.11
Dry density, g/cm3	1.49	1.62	1.48	1.41

Water Content

Trial	1		2		3		4	
Conatainer Code	T1	2P	P24	3	C2	HC51	10G	P5
Mass of container, g	18.1	17.3	17.5	17.1	17.5	17.7	17.6	17.1
Mass of container + wet soil, g	120.7	123.7	115	114.5	102.5	105.1	110.7	112.1
Mass of ontainer + Dry soil, g	103.6	115.1	99.3	100.9	87.7	84.3	88.4	90.9
Mass of Water, g	17.1	8.6	15.9	13.6	14.8	20.8	22.3	21.2
Mass of Dry soil, g	85.5	97.8	81.8	83.8	70.2	66.6	70.8	73.8
Water content, %	20.00	8.79	19.44	16.23	21.08	31.23	31.50	28.73
Water content, % Avg	14.40		17.83		26.16		30.11	

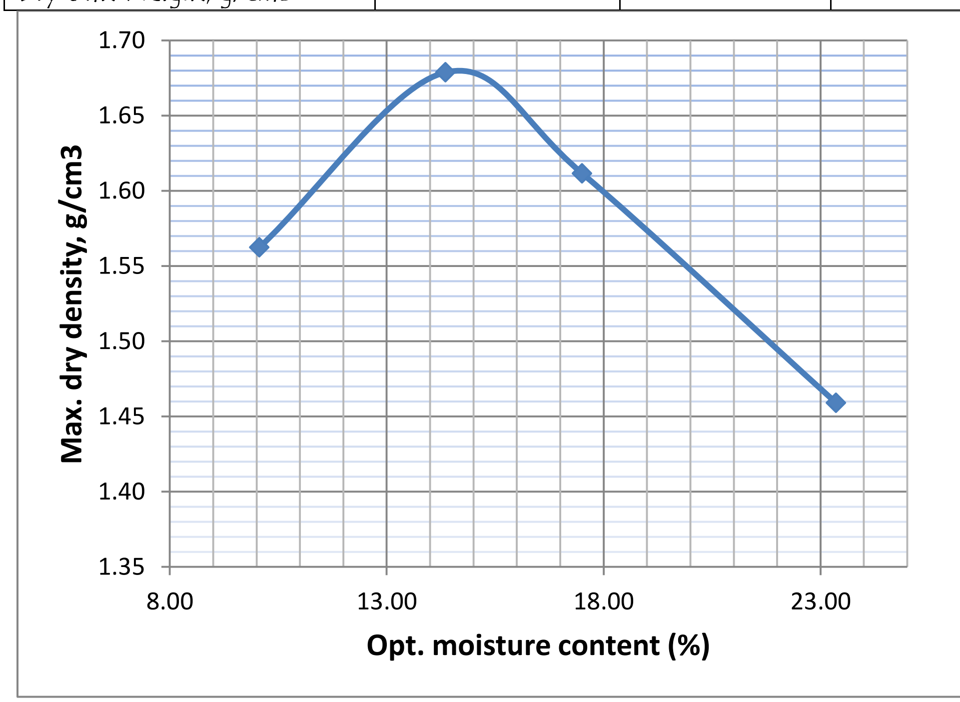


5.5 2 - 6 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	10081.5	10493.7	10440	10246.5
Mass of Compacted soil, g	3546.1	3958.3	3904.6	3711.1
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.72	1.92	1.89	1.80
Water Contenet, %	10.07	14.36	17.51	23.36
Dry density, g/cm3	1.56	1.68	1.61	1.46

Water Content

Trial	1		2		3		4	
Conatainer Code	LC51	G	F	S	C	A4	P65	ZE
Mass of container, g	25.4	32.3	36.3	25.8	32.7	38.3	37.8	33.1
Mass of container + wet soil, g	170.7	175.7	150	144.5	144	165	100.6	97.1
Mass of ontainer + Dry soil, g	152.6	167.7	135	130.6	127	147	91.4	82.5
Mass of Water, g	18.1	8	15.2	13.9	17.1	18.3	9.2	14.6
Mass of Dry soil, g	127.2	135.4	98.4	104.8	94	109	53.6	49.4
Water content, %	14.23	5.91	15.45	13.26	18.19	16.82	17.16	29.55
Water content, % (Average)	10.07		14.36		17.51		23.36	
Dry Unit Weight, g/cm3								

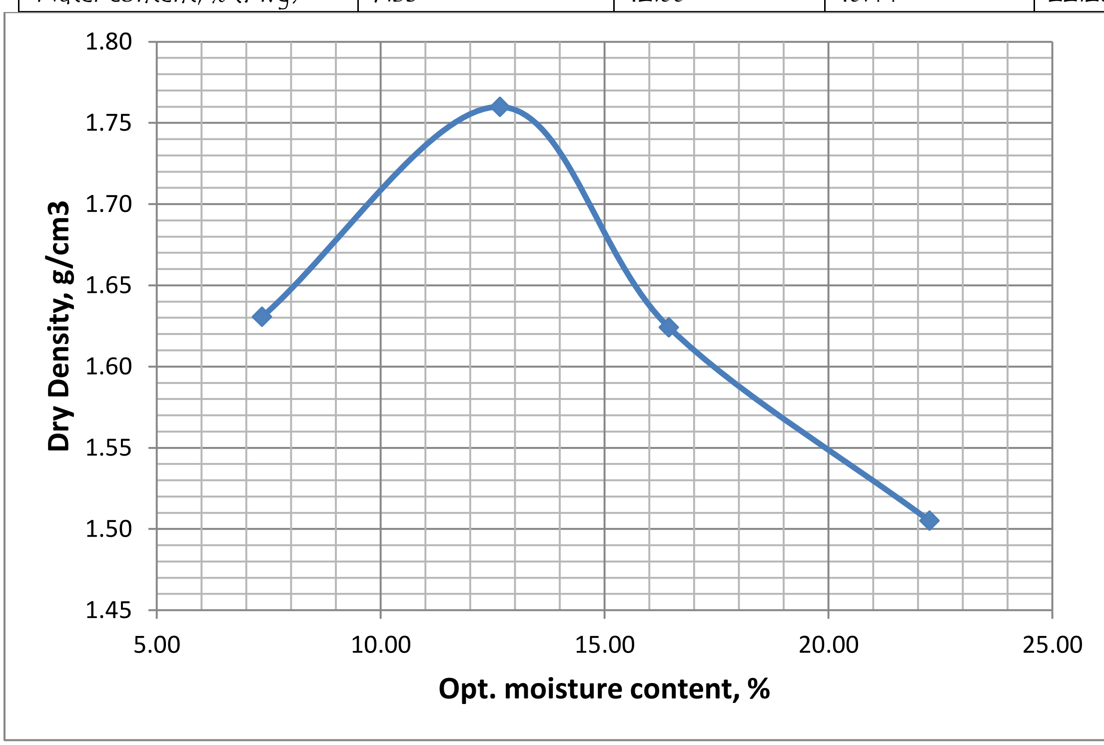


5.6 0- 8 % (CCA -LIME)

Determination No.	1	2	3	4
Mass of Mold, g	6608.6	6608.6	6608.6	6608.6
Mass of mold + Compacted Soil, g	10220	10699	10510	10404.5
Mass of Compacted soil, g	3611.4	4090.4	3901.4	3795.9
Volume of Mold, cm3	2063	2063	2063	2063
Bulk density, g/cm3	1.75	1.98	1.89	1.84
Water Contenet, %	7.35	12.66	16.44	22.26
Dry density, g/cm3	1.63	1.76	1.62	1.50

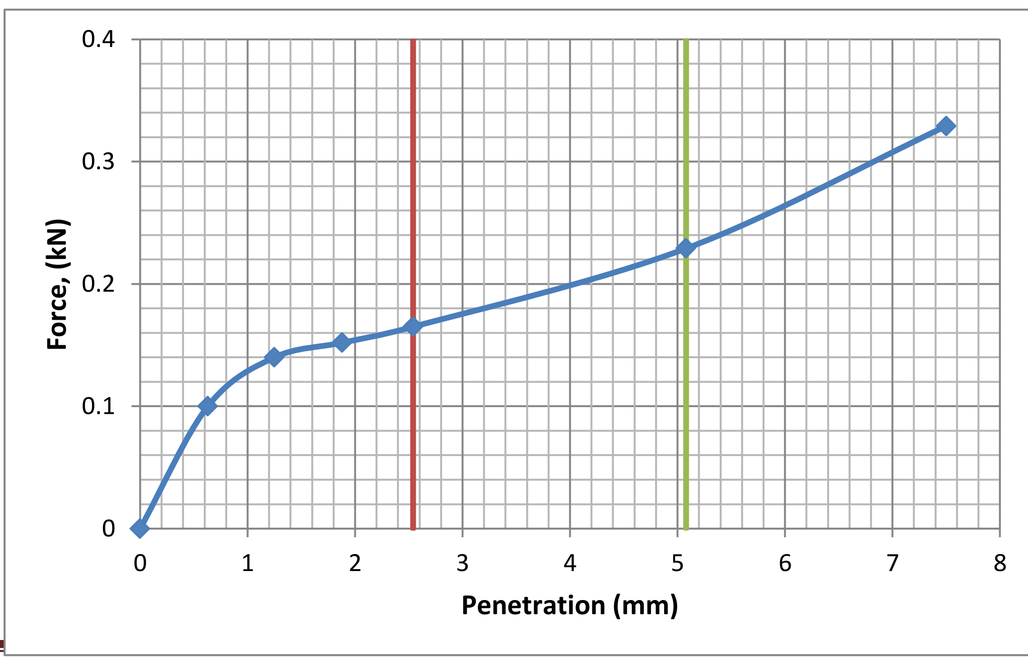
Water Content

Trial	1		2		3		5	
Conatainer Code	p2	HC51	G53	10G	UCN	P21	P2	HC51
Mass of container, g	17.5	17.4	17.4	17.4	17.5	17.7	16.7	17.7
Mass of container + wet soil, g	134.2	124.2	134.1	136.8	125.7	145.7	113.4	138
Mass of ontainer + Dry soil, g	124.7	118.3	125.3	119.3	114.6	123.1	94.1	118.3
Mass of Water, g	9.5	5.9	8.8	17.5	11.1	22.6	19.3	19.7
Mass of Dry soil, g	107.2	100.9	107.9	101.9	97.1	105.4	77.4	100.6
Water content,%	8.86	5.85	8.16	17.17	11.43	21.44	24.94	19.58
Water content, % (Avg)	7.35		12.66		16.44		22.26	

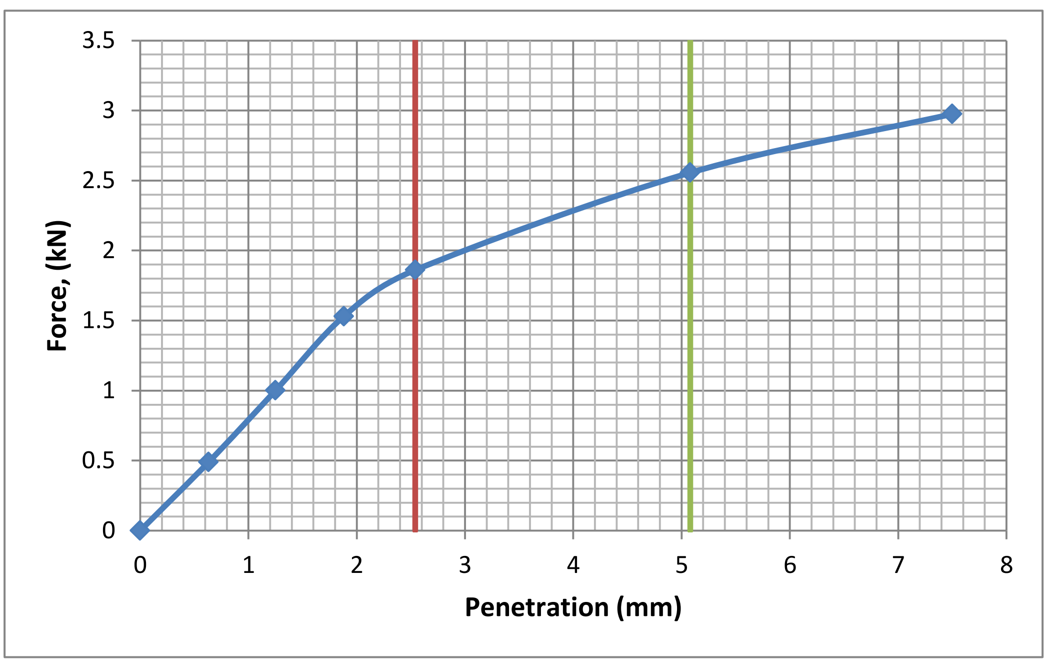


6. CBR test result

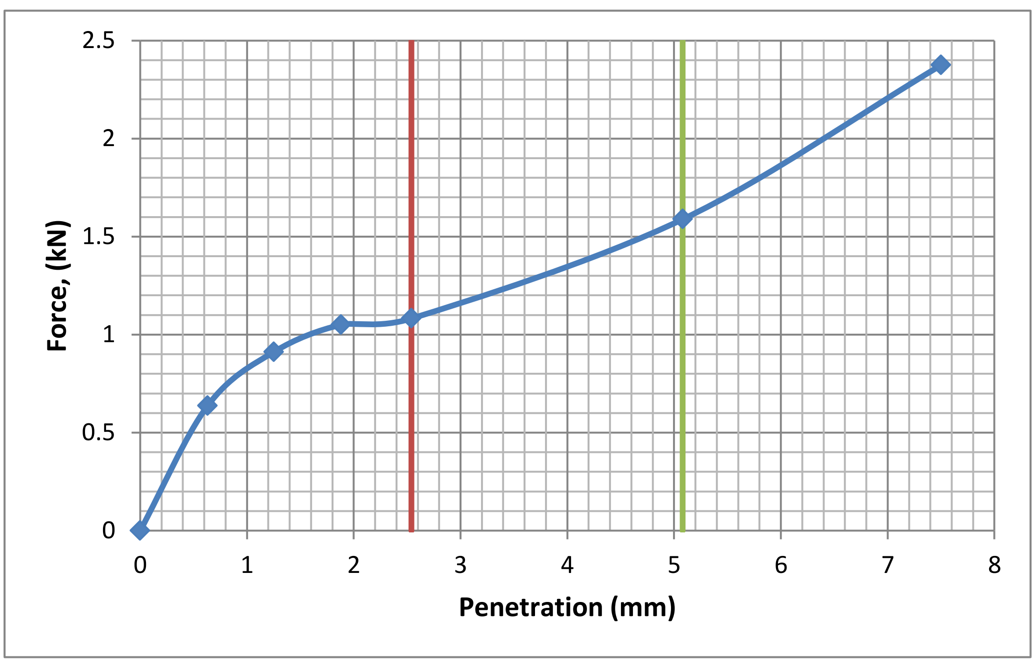
0 % CCA- 0% LIME					
Dry density of soil before and after soaking					
Dry density		Before Soaking	1.524		
		After Soaking	1.583		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.10				
1.25	0.14				
1.88	0.15				
2.54	0.17	1.24			
5.08	0.23	1.15			
7.50	0.33				
Proctor data		OMC (%)	24.5	MDD (g/cc)	1.57
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	17.94	9.83		
	Final	29.38			
Swell (%)	9.83				
MDD (g/cc)	1.57				
CBR	1.24				



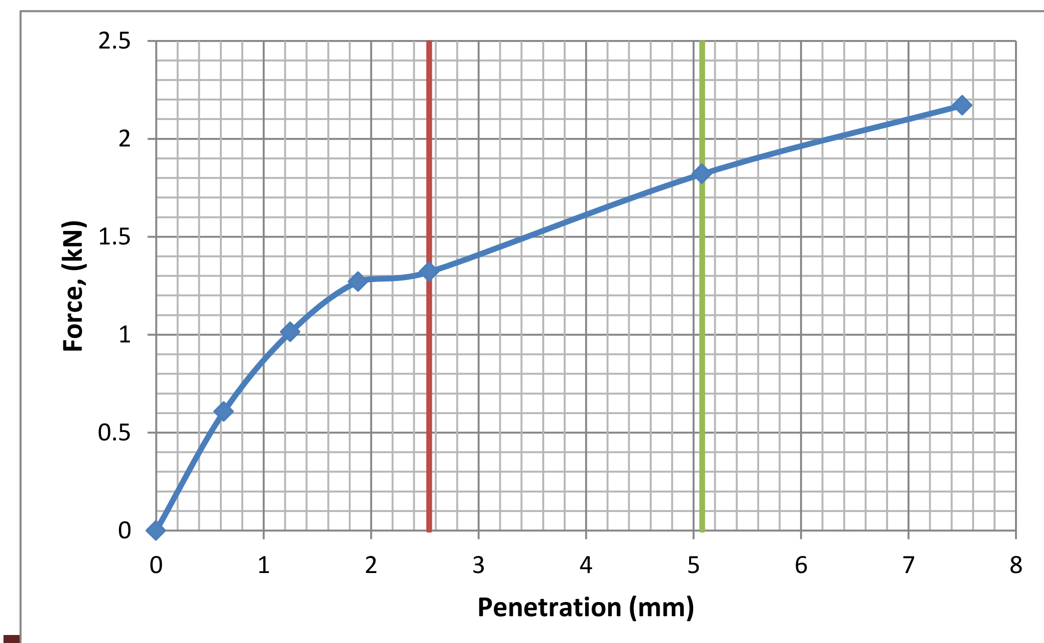
8% CCA- 0% LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.193		
	After Soaking		1.327		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	Load, KN	CBR %			
0.00	0.00				
0.63	0.14				
1.25	0.47				
1.88	0.90				
2.54	1.03	7.73			
5.08	1.13	5.64			
7.50	1.30				
Proctor data		OMC (%)	25	MDD (g/cc)	1.49
Swell Determination					
No. of blows	Gauge rdg(mm)		Swell in %		
56 blows	Initial	24.88	2.27		
	Final	27.52			
Swell (%)	2.27				
MDD (g/cc)	1.49				
CBR	7.73				



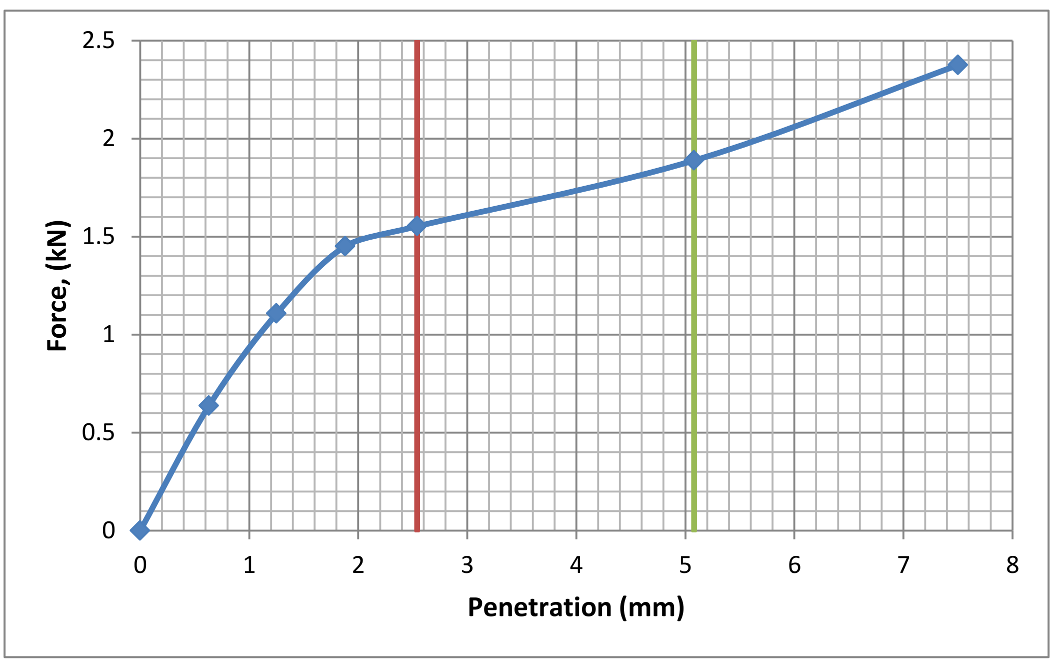
6% CCA- 2% LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.444		
	After Soaking		1.465		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.64				
1.25	0.91				
1.88	1.05				
2.54	1.08	8.11			
5.08	1.59	7.94			
7.50	2.37				
Proctor data		OMC (%)	20	MDD (g/cc)	1.61
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	23.41	1.76		
	Final	25.46			
Swell (%)	1.76				
MDD (g/cc)	1.61				
CBR	8.11				



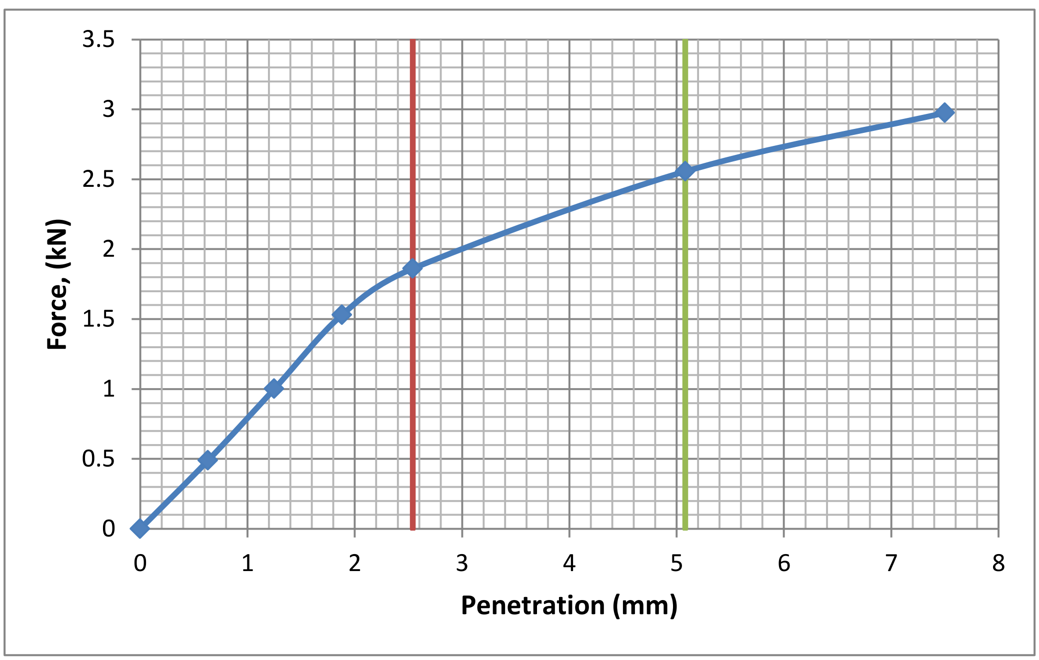
4% CCA- 4% LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.462		
	After Soaking		1.534		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.61				
1.25	1.01				
1.88	1.27				
2.54	1.32	9.90			
5.08	1.82	9.10			
7.50	2.17				
Proctor data		OMC (%)	18	MDD (g/cc)	1.62
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	27.28	1.56		
	Final	29.10			
Swell (%)	1.56				
MDD (g/cc)	1.62				
CBR	9.9				



2 % CCA- 6% LIME				
Dry density of soil before and after soaking				
Dry density	Before Soaking	1.494		
	After Soaking	1.542		
CBR penetration determination				
Penetration after 96 hrs Soaking Period				
Pen.mm	**Load, KN**	**CBR %**		
0.00	0.00			
0.63	0.64			
1.25	1.11			
1.88	1.45			
2.54	1.55	11.63		
5.08	1.89	9.44		
7.50	2.37			
Proctor data	OMC (%)	15	MDD (g/cc)	1.67
Swell Determination				
No. of blows	**Gauge rdg(mm)**		**Swell in %**	
56 blows	Initial	22.58	1.65	
	Final	24.50		
Swell (%)	1.65			
MDD (g/cc)	1.67			
CBR	11.63			



0% CCA- 8% LIME					
Dry density of soil before and after soaking					
Dry density		Before Soaking		1.541	
		After Soaking		1.548	
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.488				
1.25	1.00				
1.88	1.53				
2.54	1.86	13.95			
5.08	2.56	12.78			
7.50	2.98				
Proctor data		OMC (%)	13	MDD (g/cc)	1.76
Swell Determination					
No. of blows	**Gauge rdg(mm)**			**Swell in %**	
56 blows	Initial	25.88		0.53	
	Final	26.50			
Swell (%)	0.53				
MDD (g/cc)	1.76				
CBR	13.95				



Appendix B: Laboratory Test Result of Soil Sample 2

2.1. Moisture Content

Sample Name	Merkato Sefer		
Can Code	C1	C2	C3
Mass of container, g	*17.40*	*17.60*	*17.40*
Mass of container + wet soil, g	*54.66*	*53.39*	*54.65*
Mass of ontainer + Dry soil, g	*44.11*	*43.78*	*43.22*
Mass of Water, g	*10.55*	*9.61*	*11.43*
Mass of Dry soil, g	*26.71*	*26.18*	*25.82*
Water content, %	*39.50*	*36.71*	*44.27*
Average water Content	*40.16*		

2.2. Wet Sieve Analysis

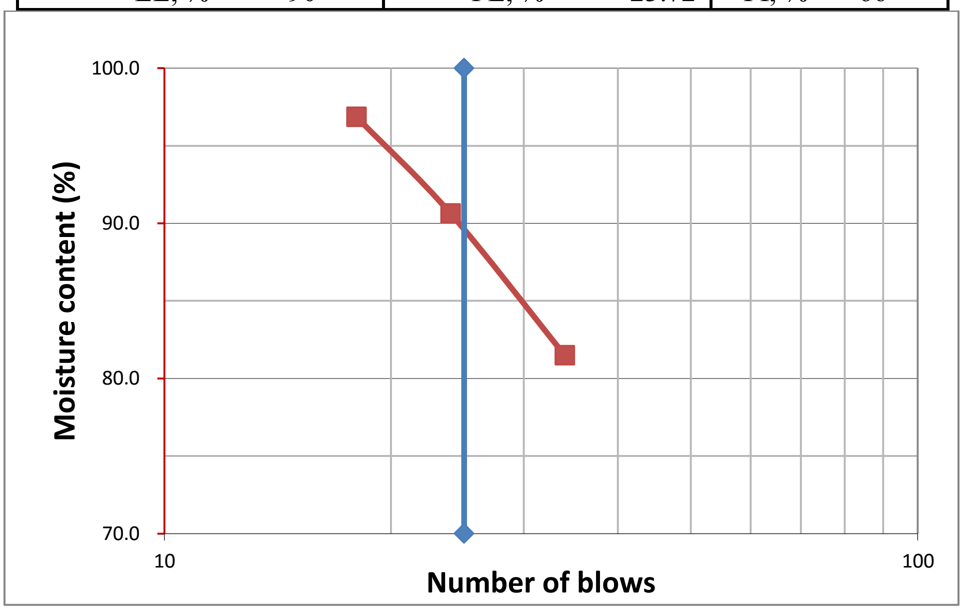
Pit No.	Sieve Size (opening)	Mass retained in g	Percentageretained	Cumulativepercentage retained	Percentage finer particle
2 @1.5m	9.5	15.2	1.52	1.52	98.48
	4.75	8.3	0.83	2.35	97.65
	2	37.6	3.76	6.11	93.89
	0.85	28.1	2.81	8.92	91.08
	0.425	16.2	1.62	10.54	89.46
	0.3	5.4	0.54	11.08	88.92
	0.15	9.8	0.98	12.06	87.94
	0.075	7.8	0.78	12.84	87.16
	pan				
		Course	2.35		
		Sand	10.49		
		Silt & Clay	89.51		

2.3. Hydrometer Analysis

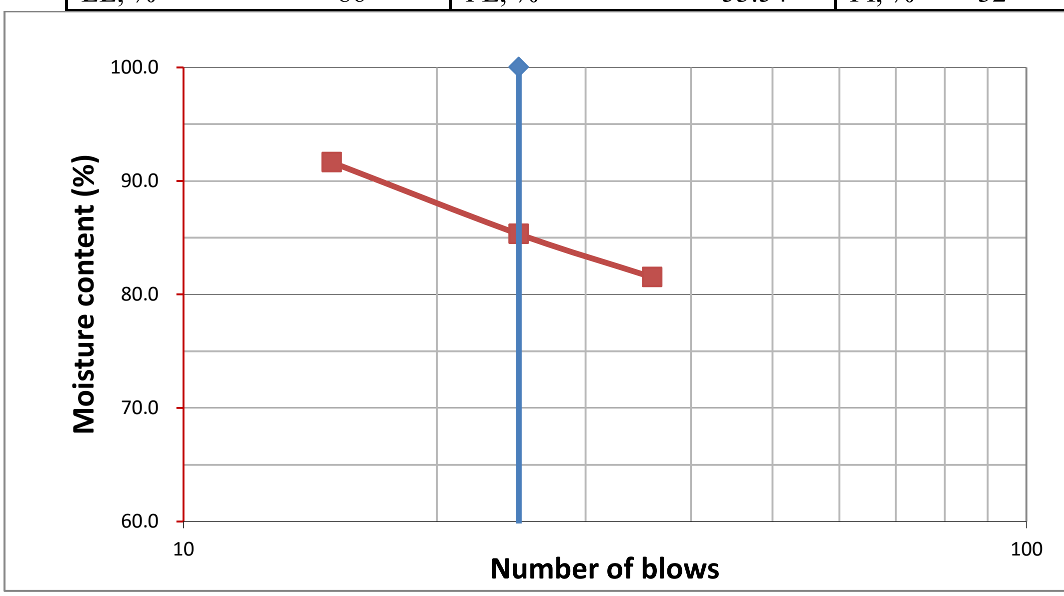
Elapsed time (min)	temprature	actual hyd. Reading	L	K	D	Ct	a	Corrected hyd. Reading (Rc)	persentage of finer %	corrected (Pa)
0.5	20	44	9.1	0.01317	0.0562	0.15	0.97435	44.65	87.01	83.93
1	20	43	9.2	0.01317	0.0399	0.15	0.97435	36.65	71.42	68.89
2	20	43	9.2	0.01317	0.0282	0.15	0.97435	34.65	67.52	65.13
4	20	41	9.6	0.01317	0.0204	0.15	0.97435	34.65	67.52	65.13
8	20	39	9.9	0.01317	0.0147	0.15	0.97435	32.65	63.63	61.37
15	20	38	10.1	0.01317	0.0108	0.15	0.97435	31.65	61.68	59.49
30	21	36	10.4	0.013018	0.0077	0.4	0.97435	29.9	58.27	56.20
60	20	32	11.1	0.01317	0.0057	0.15	0.97435	25.65	49.98	48.21
120	21	30	11.4	0.013018	0.0040	0.4	0.97435	23.9	46.57	44.93
240	21	27	11.9	0.013018	0.0029	0.4	0.97435	20.9	40.73	39.29
480	22	26	12	0.012868	0.0020	0.65	0.97435	20.15	39.27	37.88
960	22	25	12.2	0.012868	0.0015	0.65	0.97435	19.15	37.32	36.00
1440	22	24	12.4	0.012868	0.0012	0.65	0.97435	18.15	35.37	34.12

2.4 Atterberg limit test result

	0 - 0 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	Q1	Q2	Ws	ES12	ES13
Mass of container, g	13.23	10.56	12.30	5.32	5.09
Mass of container + Wet soil, g	30.20	25.6	27.3	11.18	10.35
Mass of container + Dry soil, g	22.58	18.45	19.92	9.8	9.6
Mass of water, g	7.62	7.15	7.38	1.38	0.75
Mass of dry soil, g	9.35	7.89	7.62	4.48	4.51
Water content, %	81.50	90.62	96.85	30.80	16.63
No of blows	34	24	18	------	------
LL, % = 90	PL, % = 23.72			PI, %= 66	

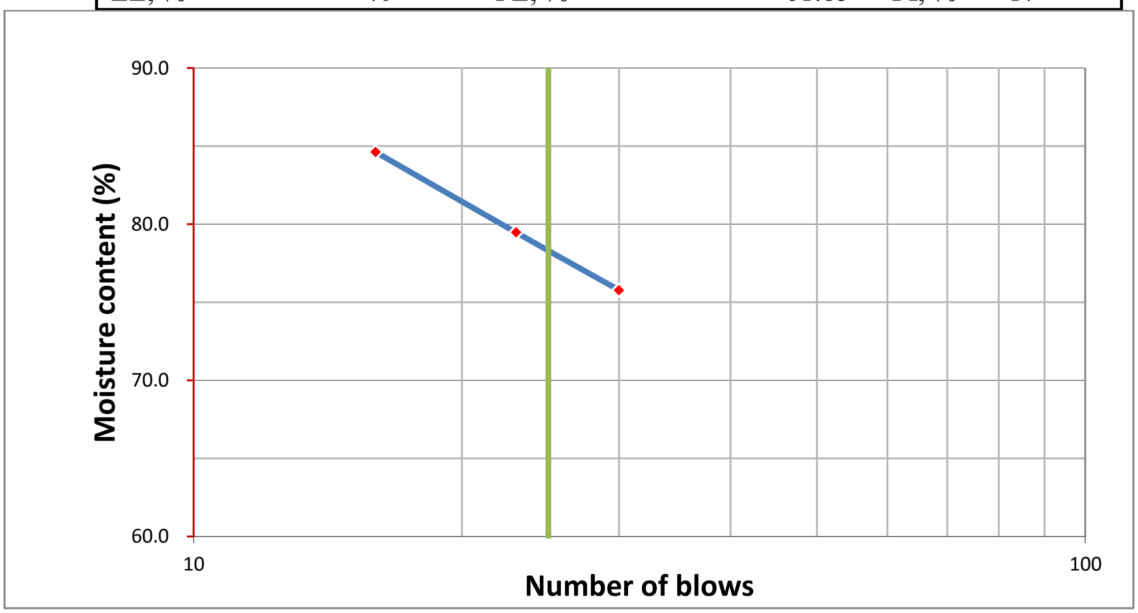


	8 - 0 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	A1	A2	A3	B1	B2
Mass of container, g	12.58	12.25	12.17	6.50	6.38
Mass of container + Wet soil, g	24.56	23.61	22.94	12.5	12.82
Mass of container + Dry soil, g	19.18	18.38	17.79	10.45	10.53
Mass of water, g	5.38	5.23	5.15	2.05	2.29
Mass of dry soil, g	6.60	6.13	5.62	3.95	4.15
Water content, %	81.52	85.32	91.64	51.90	55.18
No of blows	36	25	15	------	------
LL, % = 86	PL, % = 53.54			PI, % = 32	



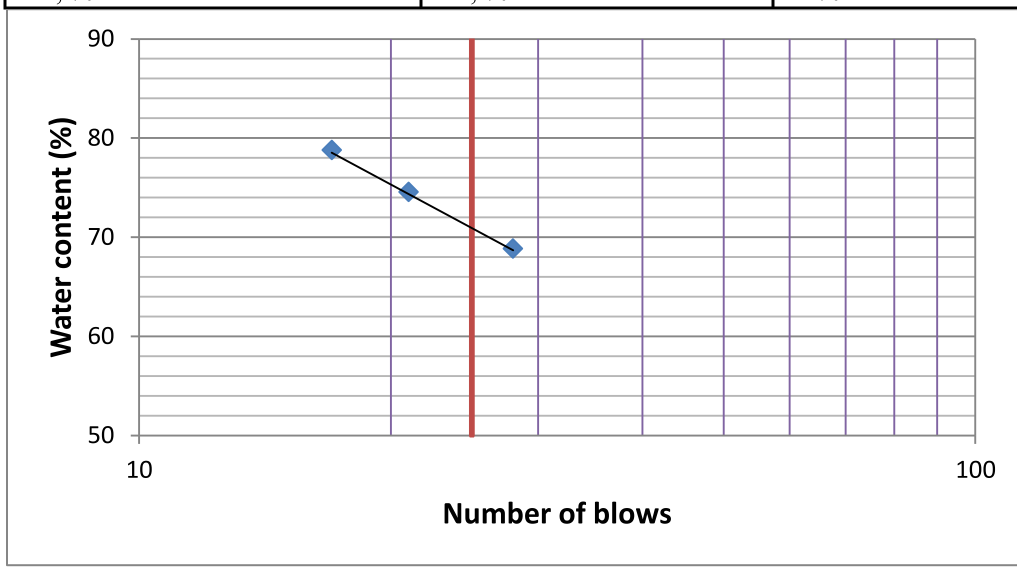
	6 - 2 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	WE12	WE13	WE14	E1	E2
Mass of container, g	17.01	17.3	17.4	7.21	7.29
Mass of container + Wet soil, g	30.26	31.1	33.36	14.12	14.12
Mass of container + Dry soil, g	24.6	24.9	25.85	11.11	12.2
Mass of water, g	5.66	6.20	7.51	3.01	1.92
Mass of dry soil, g	7.59	7.60	8.45	3.90	4.91
Water content, %	74.57	81.58	88.88	77.18	39.10
No of blows	33	22	16	------	------
LL, % = 80	PL, % = 58.14			PI, % = 22	

	4 - 4 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	Y23	W12	GF23	W1	W2
Mass of container, g	6.55	8.97	6.39	5.63	5.55
Mass of container + Wet soil, g	17.80	18.5	17.56	12.34	13.02
Mass of container + Dry soil, g	12.95	14.28	12.44	9.46	10.58
Mass of water, g	4.85	4.22	5.12	2.88	2.44
Mass of dry soil, g	6.40	5.31	6.05	3.83	5.03
Water content, %	75.78	79.47	84.63	75.20	48.51
No of blows	30	23	16	------	------
LL, % = 79	PL, % =		61.85	PI, %= 17	



Determination	2 - 6 % (CCA -LIME)				
	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	SC1	Y3	Tr4	W12	W13
Mass of container, g	17.40	17.50	17.40	4.24	4.25
Mass of container + Wet soil, g	29.46	30.61	28.42	10.23	10.29
Mass of container + Dry soil, g	24.56	24.95	23.381	7.98	7.77
Mass of water, g	4.90	5.66	5.04	2.25	2.52
Mass of dry soil, g	7.16	7.45	5.98	3.74	3.52
Water content, %	68.44	75.97	84.25	60.16	71.59
No of blows	32	23	15	------	------
LL, % = 74	PL, % =		65.88	PI, %= 8	

	0 - 8 % (CCA -LIME)				
Determination	Liquid Limit			Plastic Limit	
Trial No	1	2	3	1	2
Container No	A1	A2	A3	ZE1	Y
Mass of container, g	10.28	10.21	10.29	5.81	5.86
Mass of container + Wet soil, g	22.79	**23.58**	**21.91**	**11.91**	**12.1**
Mass of container + Dry soil, g	17.69	**17.87**	**16.79**	**9.44**	**9.49**
Mass of water, g	5.10	5.71	5.12	2.47	2.61
Mass of dry soil, g	7.41	7.66	6.50	3.63	3.63
Water content, %	68.83	74.54	78.77	68.04	71.90
No of blows	28	21	17	------	------
LL, % = 72	PL, % = 69.97			PI, %= 2	

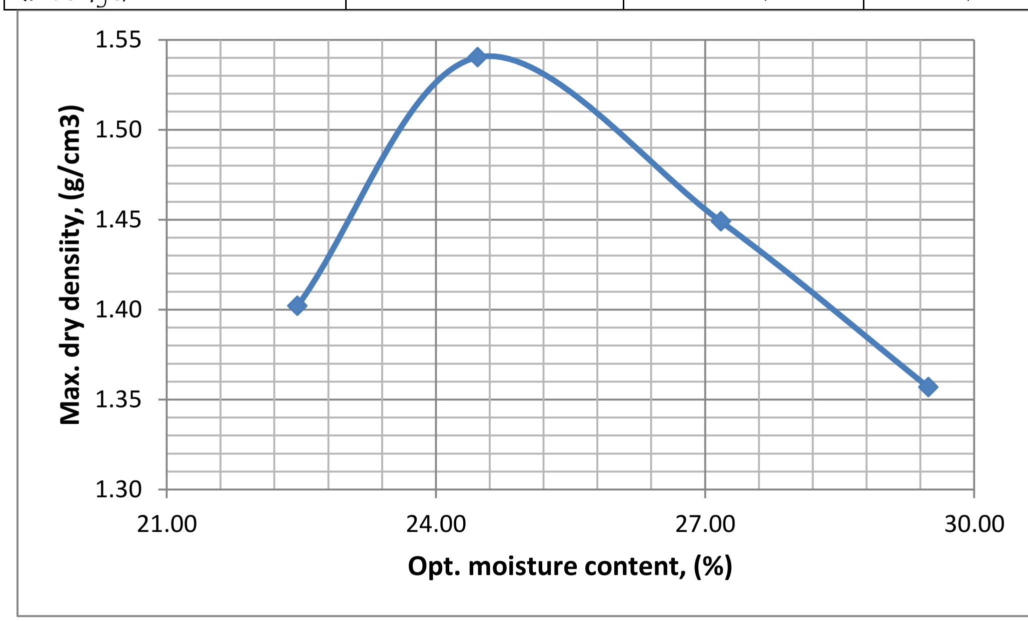


2.5 Compaction test result (0-0% CCA –Lime)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	10075.6	10488.5	10335	10158.2
Mass of Compacted soil, g	3540.2	3953.1	3799.6	3622.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.72	1.92	1.84	1.76
Water Contenet, %	22.46	24.47	27.18	29.49
Dry density, g/cm3	1.40	1.54	1.45	1.36

Water Content

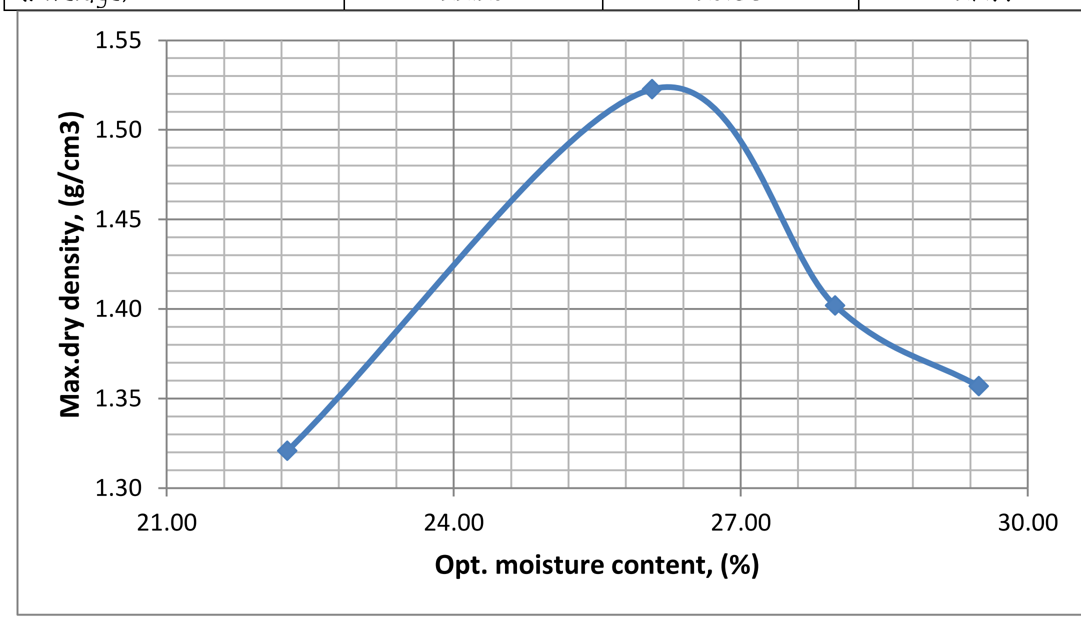
Trial	1		2		3		4	
Mass of container, g	28.4	26.7	25.5	27.8	24.9	25.4	34.4	32.3
Mass of container + wet soil, g	123.7	110.6	183.7	185	195.1	197	184.5	187.7
Mass of ontainer + Dry soil, g	104.5	96.8	152.2	154.5	151.5	168.5	154.8	148
Mass of Water, g	19.2	13.8	31.5	30.5	43.6	28.5	29.7	39.7
Mass of Dry soil, g	76.1	70.1	126.7	126.7	126.6	143.1	120.4	115.7
Water content, %	25.23	19.69	24.86	24.07	34.44	19.92	24.67	34.31
Water content, % (Average)	22.46		24.47		27.18		29.49	



2.5 Compaction test result (8-0% CCA –Lime)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	9865	10494	10235	10158.2
Mass of Compacted soil, g	3329.6	3958.6	3699.6	3622.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.61	1.92	1.79	1.76
Water Contenet, %	22.26	26.08	27.99	29.49
Dry density, g/cm3	1.32	1.52	1.40	1.36

Trial	1		2		3		4	
Mass of container, g	18.4	17.7	15.5	17.8	14.9	15.4	34.4	32.3
Mass of container + wet soil, g	99.7	95.6	108.7	118	99.1	99.7	184.5	187.7
Mass of ontainer + Dry soil, g	84.5	81.8	92.2	94.5	79.5	82.5	154.8	148
Mass of Water, g	15.2	13.8	16.5	23.5	19.6	17.2	29.7	39.7
Mass of Dry soil, g	66.1	64.1	76.7	76.7	64.6	67.1	120.4	115.7
Water content, %	23.00	21.53	21.51	30.64	30.34	25.63	24.67	34.31
Water content, % (Average)	22.26		26.08		27.99		29.49	

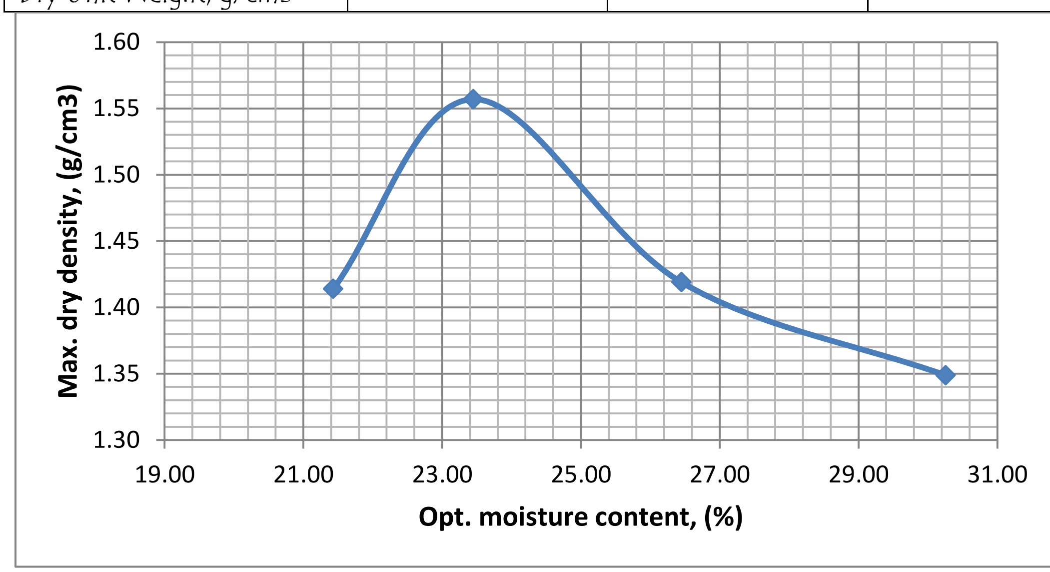


2.5 Compaction test result (6-2% CCA –Lime)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	10075.6	10498.5	10235	10158.2
Mass of Compacted soil, g	3540.2	3963.1	3699.6	3622.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.72	1.92	1.79	1.76
Water Contenet, %	21.43	23.45	26.45	30.26
Dry density, g/cm3	1.41	1.56	1.42	1.35

Water Content

Trial	1		2		3		4	
Mass of container, g	49.4	28.7	36.5	37.8	39.9	34.4	33.4	32.3
Mass of container + wet soil, g	94.7	93.6	112.7	109	149.1	142	134.5	127.7
Mass of ontainer + Dry soil, g	88.5	79.8	98.2	95.5	124.5	121.3	106.8	110
Mass of Water, g	6.2	13.8	14.5	13.5	24.6	20.7	27.7	17.7
Mass of Dry soil, g	39.1	51.1	61.7	57.7	84.6	86.9	73.4	77.7
Water content, %	15.86	27.01	23.50	23.40	29.08	23.82	37.74	22.78
Water content, % (Average)	21.43		23.45		26.45		30.26	
Dry Unit Weight, g/cm3								

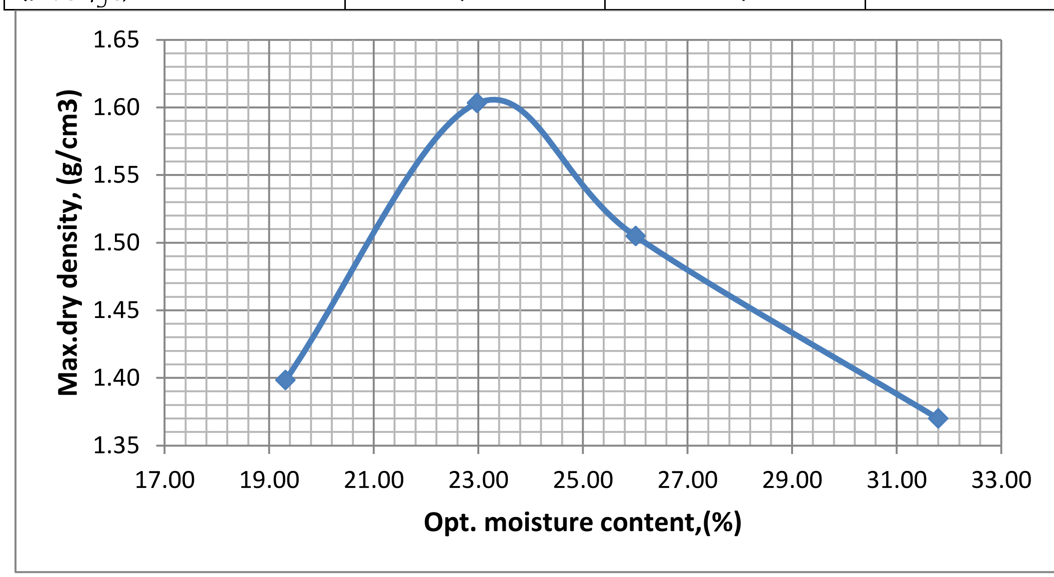


2.5 Compaction test result (4-4% CCA –Lime)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	9975.6	10600.5	10445	10258.2
Mass of Compacted soil, g	3440.2	4065.1	3909.6	3722.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.67	1.97	1.90	1.81
Water Contenet, %	19.31	22.98	26.01	31.80
Dry density, g/cm3	1.40	1.60	1.50	1.37

Water Content

Trial	1		2		3		4	
Mass of container, g	19.4	18.7	36.5	37.8	39.9	34.4	33.4	32.3
Mass of container + wet soil, g	113.7	103.6	154.7	166	159.1	246	159.5	177.7
Mass of ontainer + Dry soil, g	98.5	89.8	132.2	142.5	131.5	208	130.8	140.7
Mass of Water, g	15.2	13.8	22.5	23.5	27.6	38	28.7	37
Mass of Dry soil, g	79.1	71.1	95.7	104.7	91.6	173.6	97.4	108.4
Water content, %	19.22	19.41	23.51	22.45	30.13	21.89	29.47	34.13
Water content, % (Average)	19.31		22.98		26.01		31.80	

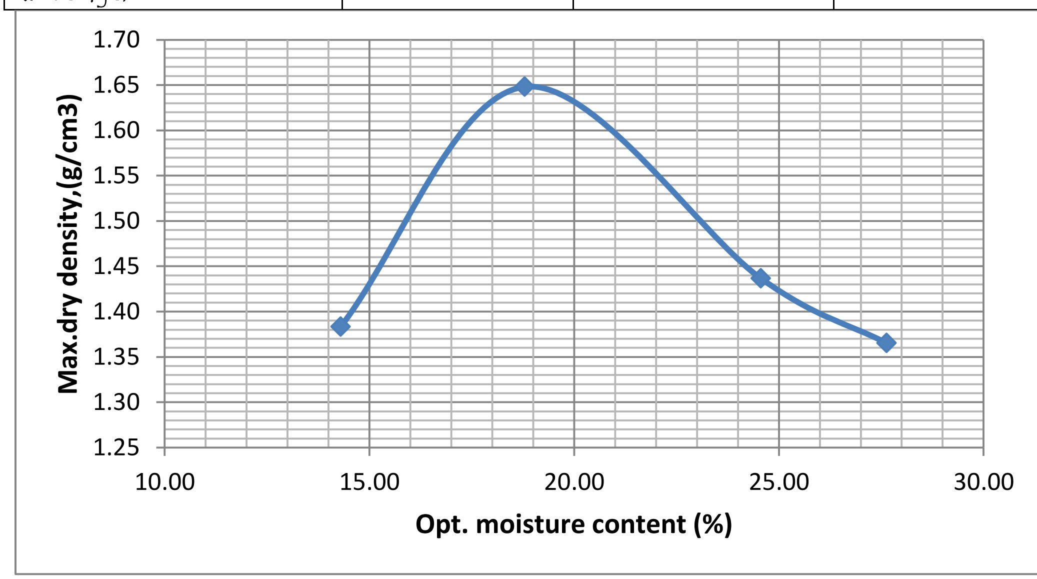


2.5 Compaction test result (2-6% CCA –Lime)

Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	9795.6	10572.5	10225	10128.2
Mass of Compacted soil, g	3260.2	4037.1	3689.6	3592.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.58	1.96	1.79	1.74
Water Contenet, %	14.30	18.80	24.56	27.64
Dry density, g/cm3	1.38	1.65	1.44	1.37

Water Content

Trial	1		2		3		4	
Mass of container, g	17.4	17.7	16.5	17.8	19.9	24.4	33.4	32.3
Mass of container + wet soil, g	223.7	203.6	124.7	118	129.1	146	142.5	134.7
Mass of ontainer + Dry soil, g	198.5	179.8	107.2	102.5	111.5	118	118.8	112.6
Mass of Water, g	25.2	23.8	17.5	15.5	17.6	28	23.7	22.1
Mass of Dry soil, g	181.1	162.1	90.7	84.7	91.6	93.6	85.4	80.3
Water content, %	13.91	14.68	19.29	18.30	19.21	29.91	27.75	27.52
Water content, % (Average)	14.30		18.80		24.56		27.64	

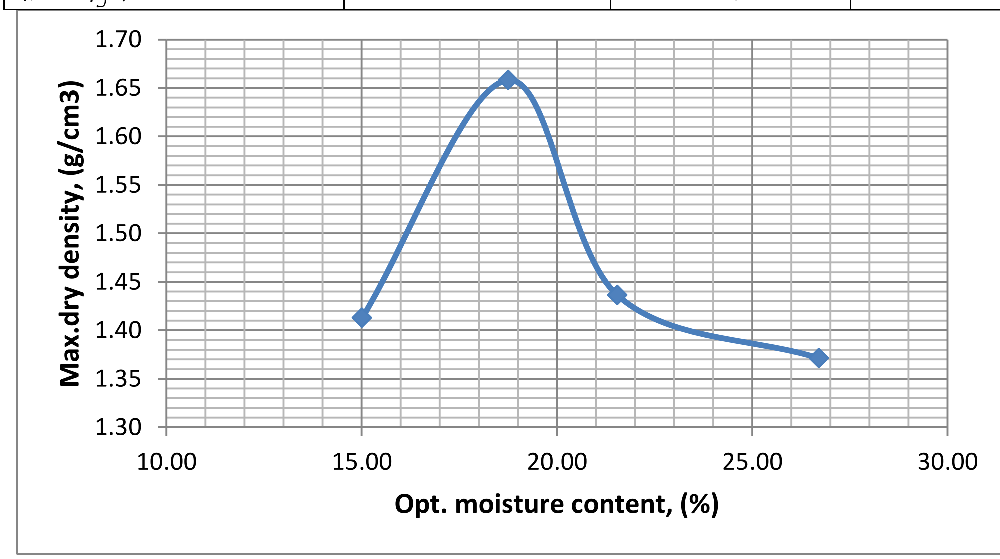


2.5 Compaction test result (0-8% CCA –Lime)

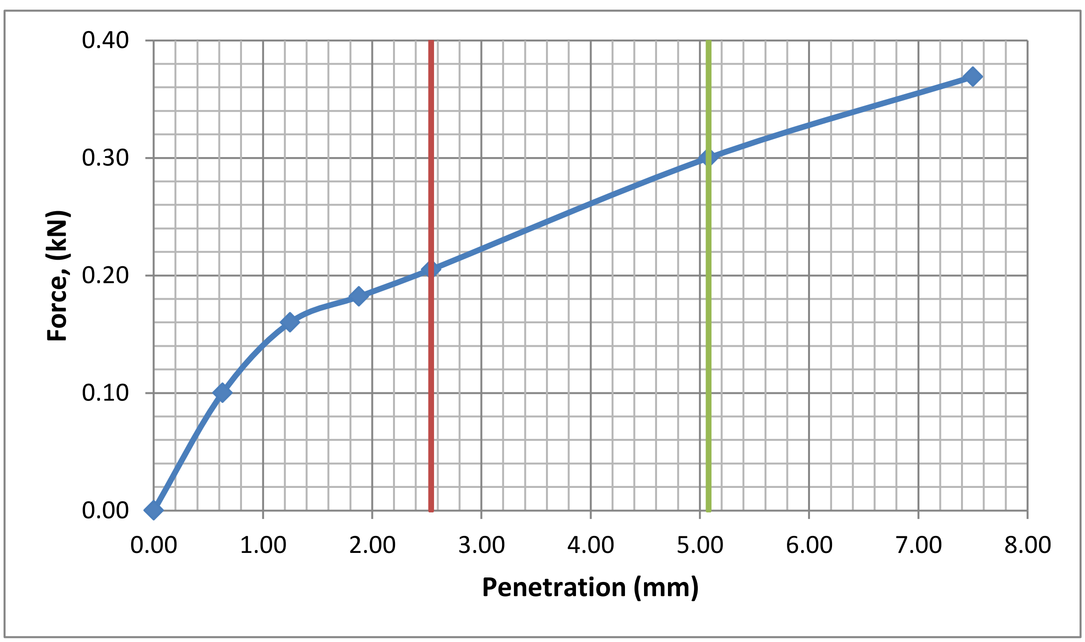
Determination No.	1	2	3	4
Mass of Mold, g	6535.4	6535.4	6535.4	6535.4
Mass of mold + Compacted Soil, g	9885.6	10595.5	10135	10118.2
Mass of Compacted soil, g	3350.2	4060.1	3599.6	3582.8
Volume of Mold, cm3	2062	2062	2062	2062
Bulk density, g/cm3	1.62	1.97	1.75	1.74
Water Contenet, %	15.01	18.75	21.55	26.71
Dry density, g/cm3	1.41	1.66	1.44	1.37

Water Content

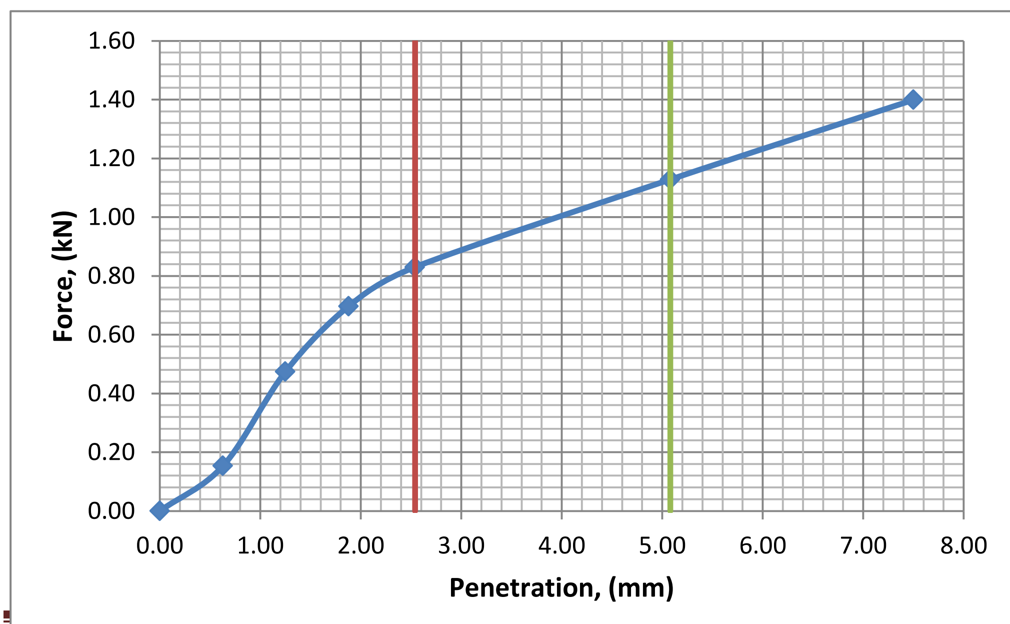
Trial	1		2		3		4	
Mass of container, g	19.4	18.7	16.5	17.8	29.9	24.4	33.4	22.3
Mass of container + wet soil, g	95.7	88.6	73.7	86	129.1	126	129.5	97.7
Mass of ontainer + Dry soil, g	86.5	78.8	67.2	72.5	111.5	108	114.8	78
Mass of Water, g	9.2	9.8	6.5	13.5	17.6	18	14.7	19.7
Mass of Dry soil, g	67.1	60.1	50.7	54.7	81.6	83.6	81.4	55.7
Water content, %	13.71	16.31	12.82	24.68	21.57	21.53	18.06	35.37
Water content, % (Average)	15.01		18.75		21.55		26.71	



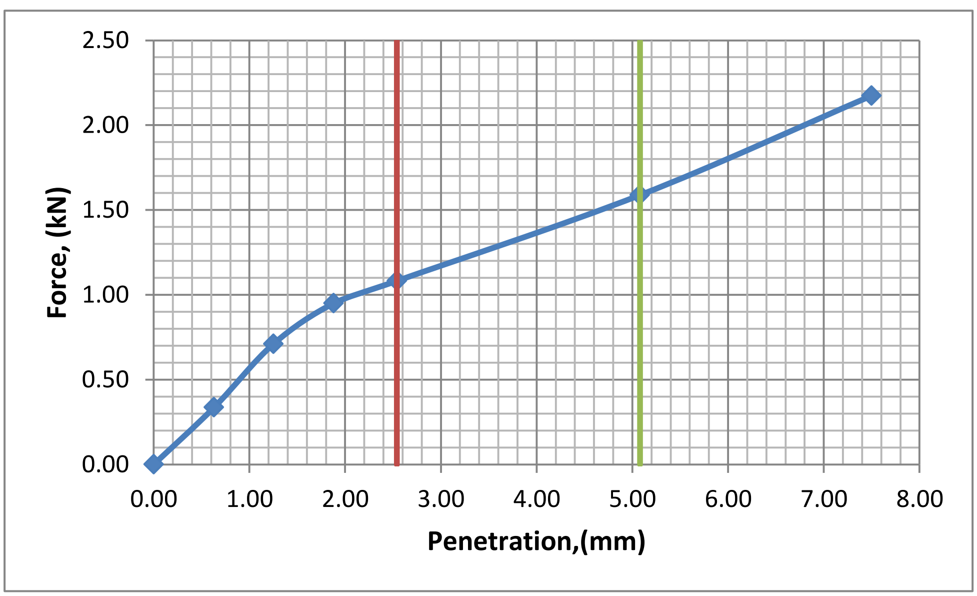
2.6 CBR test result		**0 % CCA- 0% LIME**			
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.53		
	After Soaking		1.51		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.10				
1.25	0.16				
1.88	0.18				
2.54	0.21	1.54			
5.08	0.30	1.50			
7.50	0.37				
Proctor data		OMC (%)	24	MDD	1.54
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	18.94	8.11		
	Final	28.38			
Swell (%)	8.11				
MDD (g/cc)	1.54				
CBR	1.54				



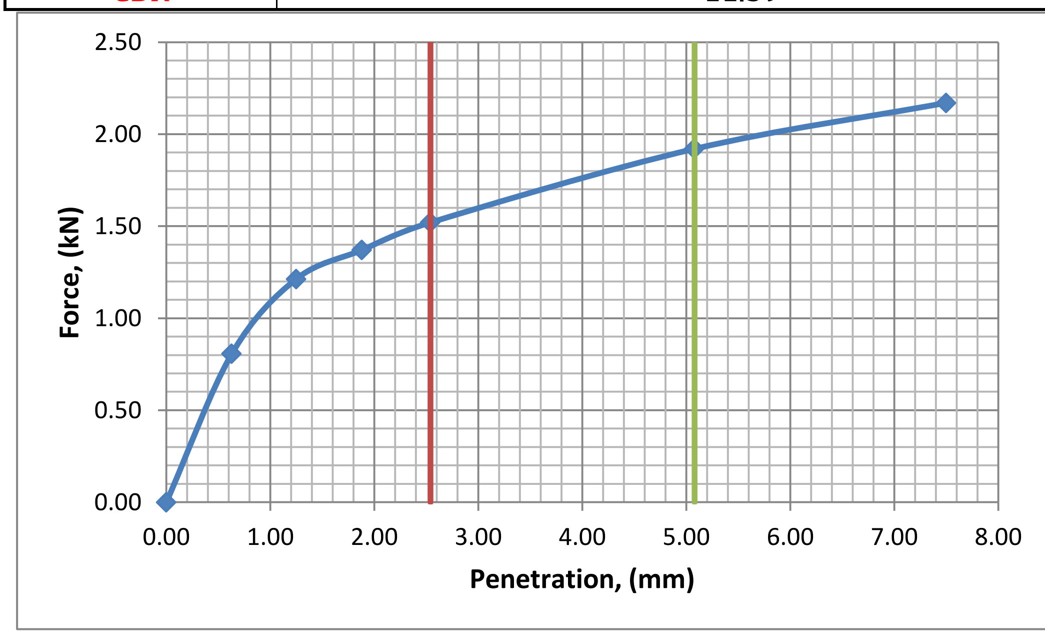
8 % CCA- 0% LIME				
Dry density of soil before and after soaking				
Dry density	Before Soaking		1.12	
	After Soaking		1.38	
CBR penetration determination				
Penetration after 96 hrs Soaking Period				
Pen.mm	**Load, KN**	**CBR %**		
0.00	0.00			
0.63	0.15			
1.25	0.47			
1.88	0.70			
2.54	0.83	6.22		
5.08	1.13	5.64		
7.50	1.40			
Proctor data	OMC (%)	26	MDD (g/cc)	1.52
Swell Determination				
No. of blows	**Gauge rdg(mm)**		**Swell in %**	
56 blows	Initial	20.67	2.33	
	Final	23.38		
Swell (%)	2.33			
MDD (g/cc)	1.52			
CBR	6.22			



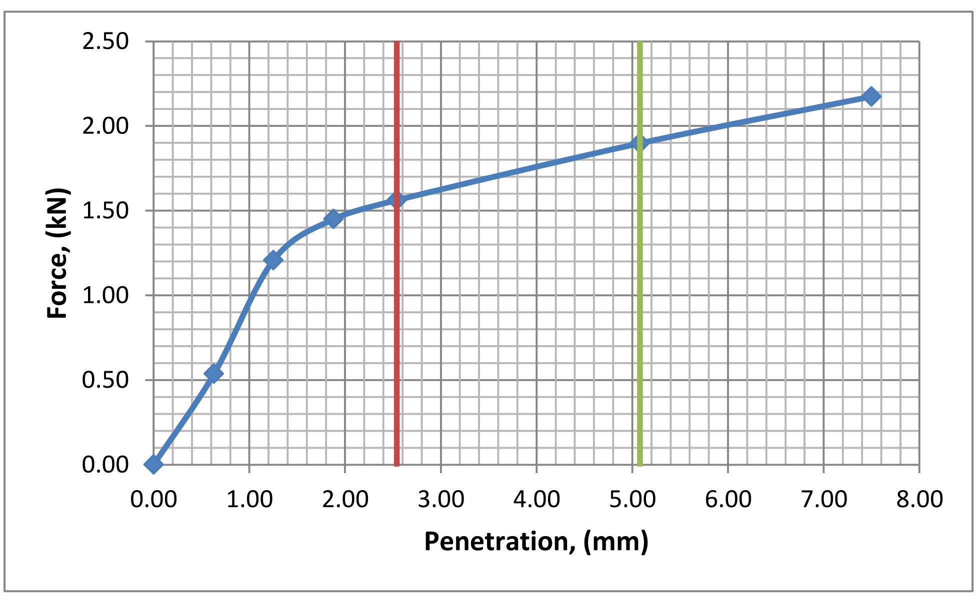
6 % CCA- 2% LIME				
Dry density of soil before and after soaking				
Dry density	Before Soaking		1.52	
	After Soaking		1.48	
CBR penetration determination				
Penetration after 96 hrs Soaking Period				
Pen.mm	**Load, KN**	**CBR %**		
0.00	0.00			
0.63	0.34			
1.25	0.71			
1.88	0.95			
2.54	1.08	8.11		
5.08	1.59	7.94		
7.50	2.17			
Proctor data	OMC (%)	23.5	MDD	1.56
Swell Determination				
No. of blows	**Gauge rdg(mm)**		**Swell in %**	
56 blows	Initial	21.54	1.58	
	Final	23.38		
Swell (%)	1.58			
MDD (g/cc)	1.56			
CBR	8.11			



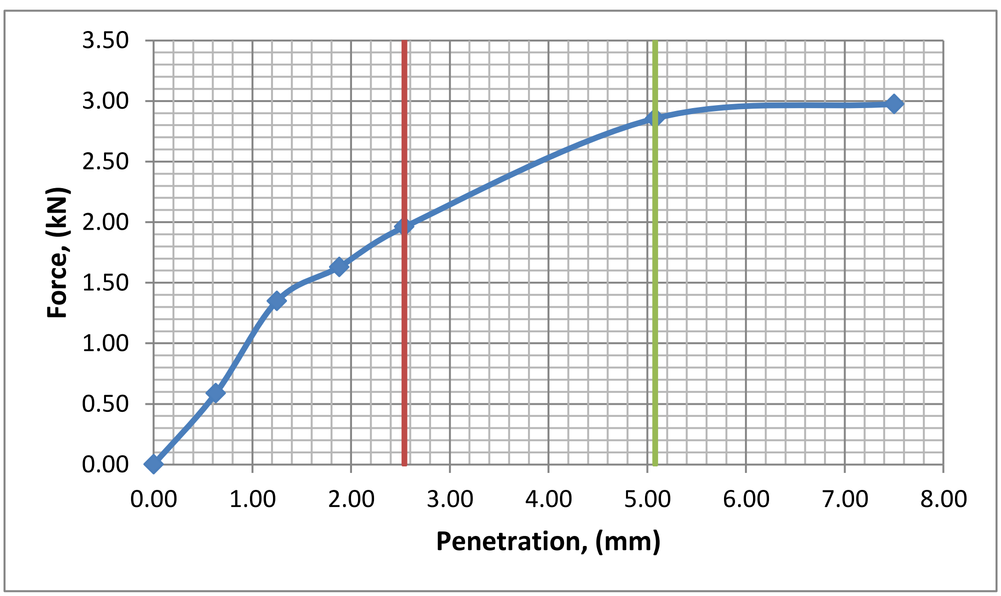
4 % CCA- 4 % LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.55		
	After Soaking		1.47		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.81				
1.25	1.21				
1.88	1.37				
2.54	1.52	11.39			
5.08	1.92	9.60			
7.50	2.17				
Proctor data		OMC (%)	23.0	MDD	1.6
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	20.33	0.90		
	Final	21.38			
Swell (%)	0.9				
MDD (g/cc)	1.6				
CBR	11.39				



2 % CCA- 6% LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.54		
	After Soaking		1.49		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.54				
1.25	1.21				
1.88	1.45				
2.54	1.56	11.71			
5.08	1.90	9.49			
7.50	2.17				
Proctor data	OMC (%)	19	MDD	1.65	
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	19.04	1.22		
	Final	20.46			
Swell (%)	1.22				
MDD (g/cc)	1.65				
CBR	11.71				



0 % CCA- 8 % LIME					
Dry density of soil before and after soaking					
Dry density	Before Soaking		1.57		
	After Soaking		1.53		
CBR penetration determination					
Penetration after 96 hrs Soaking Period					
Pen.mm	**Load, KN**	**CBR %**			
0.00	0.00				
0.63	0.588				
1.25	1.35				
1.88	1.63				
2.54	1.96	14.70			
5.08	2.86	14.28			
7.50	2.98				
Proctor data	OMC (%)	18.5	MDD (g/cc)	1.66	
Swell Determination					
No. of blows	**Gauge rdg(mm)**		**Swell in %**		
56 blows	Initial	20.82	0.74		
	Final	21.68			
Swell (%)	0.74				
MDD (g/cc)	1.66				
CBR	14.7				



Appendix C: Photo Taken During The Study

Figure A1: Photo taken during taking the sample

Figure A2: Photo taken during taking undisturbed BC soil sample

Figure A2: Photo taken while burning the corncob

Figure A3: Corn Cob Ash

Figure A4: Photo taken during testing the mineralogy of the mixture by XRD

Figure A5: Photo taken during setting a range for XRD result

Figure A5: Grinding the sample for The XRD

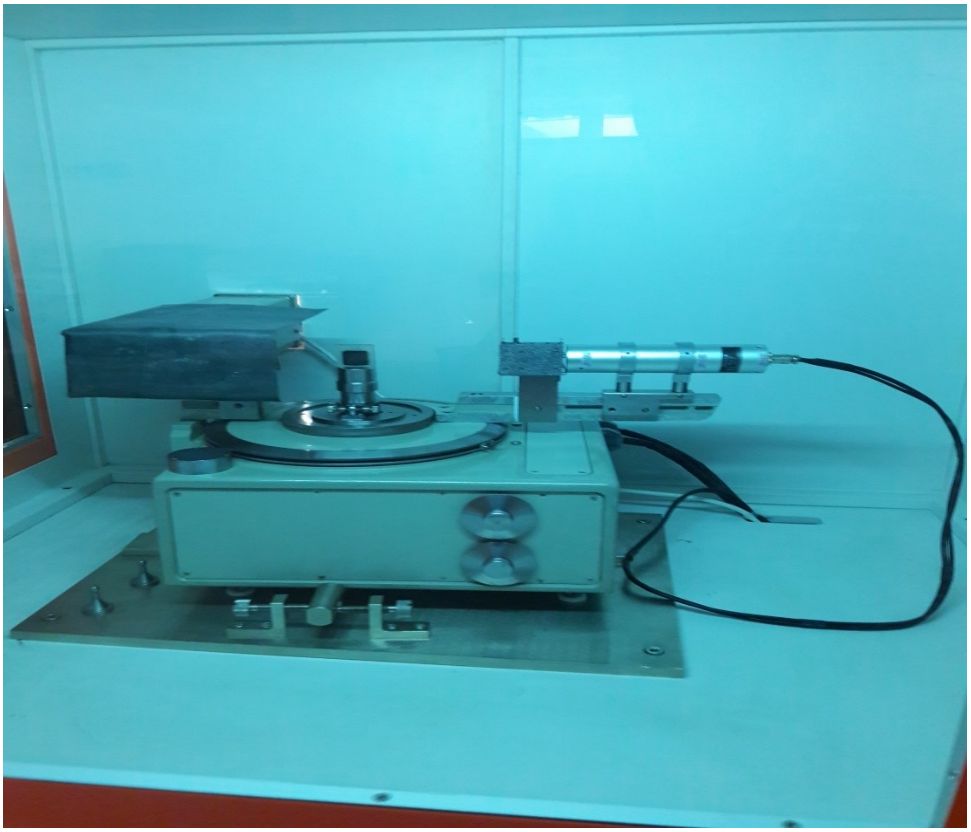


Figure A6: The inside part of the XRD machine

Figure A7: Mixing a BC soil with mixtures for the modified compaction

Figure A8: Soaking and mixing A BC soil with lime and corncob for the atterberg limit

Figure A9: Preparing a BC soil with mixtures for free swell index

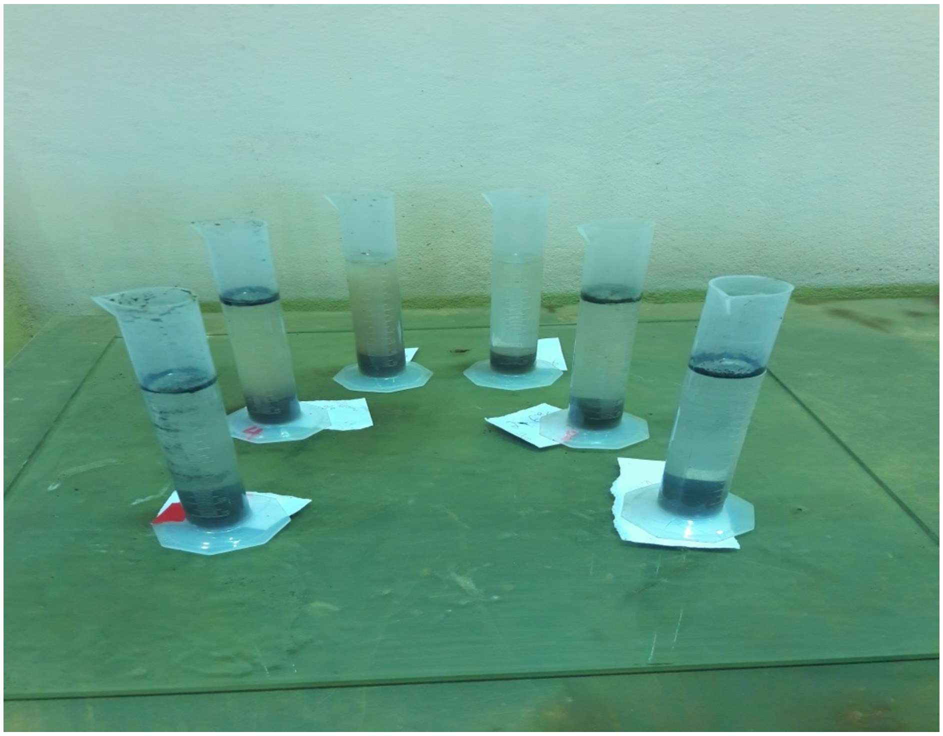


Figure A:10 a free swell Index reading After 24hrs

Figure A11:photo taken during reading hydrometer data

Figure A12: photo taken while mixing BC soil with sodium hexa methaphosphate

Figure A13: Photo taken while soaking BC soil for CBR test

Figure A14: Photo taken while reading CBR swelling

Figure A15: Photo taken while setting up a software for CBR penetration