JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Stabilization of Black Cotton Soil with Lime and Animal Bone Ash for the Improvement of Subgrade Soil

A final thesis submitted to the School of Graduate Studies of Jimma University in Partial Fulfilment of the Requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering)

By: Yan Asegid Chufa

May, 2021

Jimma, Ethiopia

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DECLARATION

I, the undersigned, declare that this thesis entitled: **"Stabilization of Black Cotton Soil with Lime and Animal Bone Ash for the Improvement of Subgrade Soil "** 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 have been duly acknowledged.

Candidate:

Mr. Yan Asegid Chufa ____________________ __________

Signature Date

As Master's Research Advisors, I hereby certify that we have read and evaluated this MSc research prepared under our guidance, by **Mr. Yan Asegid Chufa** entitled **"Stabilization of Black Cotton Soil with Lime and Animal Bone Ash for the Improvement of Subgrade Soil."**

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

Dr.Ing. Fekadu Fufa (PhD) __________________ _______________

Main Advisor Signature Date

Mr Markos tsegaye (PhD Cand.) _______________ ______________

Co-Advisor Signature Date

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ABSTRACT

Black cotton soils are characterized by large volume change with changes in moisture condition. These characteristics of these soils resulted in the failure of highway pavement and civil structure. These soils cover about 40% of the area in Ethiopia. The engineering properties of such soils can be improved through stabilization.

Therefore the aim of this study to investigate the stabilization of black cotton soil with lime, animal bone ash, and the mixture of both to improve the engineering properties of subgrade soil. Three black cotton soil samples were collected from different borrow pits in Konta-Ameya, the route called Chida, Ameya Cheka Bocha, and Ameya. Hydrated Lime was obtained from Sankale Lime Factory and bone samples were obtained in a fresh state from the abattoir (slaughterhouse) at Ameya town. The bone ash was prepared by calcination of the bone samples in a closed furnace at a temperature of 900°C for 8 hours.

The research was done with the addition of varying content of stabilizers an increasing content of 0, 2, 4, 6, and 8% lime and 0, 2, 4, 6, and 8% animal bone ash each by dry weight of the soil, was used to treat the soil. Furthermore, a mixture of both by keeping 6% animal bone ash constant and varying lime content to 2, 3, and 4% by dry weight of the soil.

For the analysis of the effect of the stabilizer on soil, a comparison was on the engineering properties of the native soil and stabilized soil. The comparison includes by carrying out: free swell, moisture content, grain size analysis, specific gravity, compaction test, Atterberg's limit test, linear shrinkage test, California Bearing Ration (CBR) test, and swelling test on both the native soil and stabilized soil according to AASHTO and ASTM.

The natural subgrade soils are A-7-5 according to the AASHTO soil classification system and CH as per USCS. Plasticity index of reduced from 59% to 13% at mix- the ratio of 6% BA + 4% lime. But adding BA in black cotton soil the plastic index decrease up to 6% BA then increases. The values for the MDD were noted to decrease with the addition of lime and mixture lime and bone ash content for reduced from 1.392 to 1.39 g/cm^3 at mix- the ratio of 6% BA + 4% lime. And the OMC was found to increase from 28.52% to 30% at mix- the ratio of 6% BA + 4% lime. However, for bone ash alone up to 6% BA maximum dry density increase and moisture content decrease. the maximum value of CBR was achieved at 6% BA + 4% L was 7.56% and 8% lime value of 10.72%, 4.99% on the addition of 6% of BA with black cotton soil. The value of CBR decreases further in addition of BA percentage.it shows 6% BA + 3% L *yields a similar result as the addition of* 6% lime *content.*

It is recommended that about 6% L alone and 6% BA + 3% L + 91 % S mixture could be used for the treatment of the soil to achieve a sub-grade material, thereby reducing the quantity (cost) of lime needed for stabilization and the environmental menace caused by the waste. It helps in reduction of the greenhouse of emission, as a result improved environment health.

Keywords: Animal bone ash, Black cotton soil, engineering property, lime, Soil stabilization.

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ACRONYMS

AASHTO	American Association Of Highway And Transportation Officials
AB	Animal Bone
ASTM	American Society For Testing And Materials
BCS	Black cotton soil
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CSA	Central Statics Agencies
ERA	Ethiopian Roads Authority
FS	Free Swell
LL	Liquid Limit
MC	Moisture Content
MDD	Maximum Dry Density
ND	Not Determined
OMC	Optimum Moisture Content
PDM	Pavement Design Manual
PI	Plastic Index
PL	Plastic Limit
SNNPRS	South Nation and Nationality people republic state
SP	Swelling Pressure
USCS	Unified Soil Classification System
USA	United States Of America

CHAPTER ONE

INTRODUCTION

1.1. Background

The unsuitability of untreated black cotton soil in roadwork is of great concern to the Civil Engineer. Black cotton soil is a typical example of expansive soils formed from the weathering of shaly and clayey sediments of basalts igneous rocks. It is predominantly composed of montmorillonite clay minerals which are characterized by large volume change with changes in moisture condition. This volumetric problem is reflected in the failure of roads and buildings constructed in areas where these soils are found [1], [2].

Found in the Northeastern part of Nigeria, Ethiopia, Cameroon, Lake Chad Basin, Sudan, Kenya, and South Zimbabwe. The soil is also found in India, Australia, and South-West of United States of America, South Africa and Israel [3],[4]. It is also found in the semi-arid regions of the tropics where the annual evaporation exceeds the precipitation [5].

It is approximated that about 40% of the country of Ethiopia is covered with expansive clay soil interrupting economic development and causing construction challenges [6]. This cost for the past 13 years, it is reported that 40% of the total road sector development expenditure in Ethiopia was allocated to rehabilitation and upgrading of trunk roads with an additional 11% utilized for maintenance works alone [7]. To solve this problem, stabilization should implement with different stabilizing additives to achieve the required specification of sub-soil materials.

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include the blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture, or plasticity, or act as a binder for cementation of the soil [8]. It also refers particularly to the mixing of the parent soil with other soil, cement, lime, bituminous products, silicates, and various other chemicals; natural or synthetic, organic, and inorganic materials as a stabilizer [9]. The cost of improving roads is expensive due to expansive soil that requires a replacement with imported materials or being stabilized. Moreover-dependence on industrially manufactured soil-improving additives (cement, lime, etc.) has kept the cost of construction of stabilized roads financially high.

However many modifications have been introduced to the concept with the use of conventional materials that help in reducing construction costs. The traditional stabilizers as lime, cement, and another are very expensive hence alternative means of using bone ash are will be sorted. Bones ash has potential availability considering the estimated 60 billion farm animals that are slaughtered every year worldwide. This number has been predicted at 120 billion by 2050 [10]. Ethiopia is home to 61.51 million cattle, 33.02 million sheep, and 38.96 goats and 1.93 million horses, 9.66 million donkeys, 0.37 million mule, and about 1.76 million camels in the rural areas of the country [11]. Meat production is the most important function of these animals in the country. This result bone is readily available and has a disposal problem an observed by [12] abattoir operations result in the generation of waste in various forms solid, liquid, semi-solid, semi-liquid that pollute the environment and pose serious threats to human health and quality of life. Also, Studies estimate that 20-50% of the weight of an animal carcass is not suitable for human consumption including blood, bone, urine, water, dissolved solids, intestinal content, and tissue waste [13], [14].

This study investigated the potential of calcination bone ash as an additive with lime stabilized black cotton soil for improvement of subgrade soil discovered a suitable complement for lime in soil stabilization. It proved viable and reduced highway construction costs. Besides, it ensures a reduction in environmental damage since using natural material revitalizes and conserves the environment, whereas chemical compounds such as cement have a detrimental impact on the environment.

1.2. Statement of the Problem

Black cotton soil is not suitable either for the building of houses or the construction of roads and causes major problems in the design, construction, and maintenance of pavements.

When unsuitable materials are encountered measures like avoiding the route, redesigning the pavement with thicker sections, or replacing the poor soil with good quality materials are practical but it is an uneconomical option and Particular problems associated with road construction over expansive soils are the seasonal volumetric change, low-bearing strength, shrinkage and crack and swell, and expansions [15].

Engineering problems related to expansive soils have been reported in many countries of the world as 3% of the world land area but are generally most series in arid and semi-arid regions. As a result, highly reactive soil undergoes substantial volume changes associated with the shrinkage and swelling process. Consequently, many engineering structures suffer severe distress and damage. Cracked foundations, pavement, floors and basement walls are topically types of damage done by swelling soils. Every year they cause billions of dollars in damage. Expansive soil is not as dramatic as a hurricane or wide areas rather than being constructed in a small locality [16].

Cities like Addis Ababa, Bahir-Dar, Mekelle, and Jimma as well as main trunk roads are situated on expansive soil. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres [17]. Currently, different construction activities are taking place in the road and building sector on expansive soil. Construction on expansive soils faces numerous problems and the causes of the problems need to be investigated in depth. Most of the roads constructed in Ethiopia on expansive soil fail before their expected design life, in some cases after a few months of completion [18].

Performance of Flexible Pavement depends on the functions of the component layers especially Subgrade. The subgrade is a compacted layer of soil that provides lateral support to the pavement. Frequently natural soils are composed of a high amount of fines which causes plasticity characteristics with adsorption of moisture under heavy loads and repeated traffic. Excess deformation leading several failures which requires a huge investment of money for their repairs. To reduce the excessive deformation of the soils and to increase the life period of the pavement there is a need to arrest their plastic characteristics and stabilization is one such technique to improve the natural soils by addition of industrial wastes. Accordingly, roads in the Konta zone experienced many types of failures such as cracks, sliding, large surface deformation and structural deformation of pavement layers and the subgrade. Therefore, to prevent the problems, engineers need to stabilize the existing weak soils before commencing the construction activities.

Expansive soil stabilization with lime and animal bone ash was Cost savings according to [12] because bone ash is typically by far cheaper than traditional stabilizers such as cement and lime and the production of traditional stabilizers, such as cement and lime, is environmental unfriendly processes which are manufactured by factory and use chemical. Also, the extraction of substantial amounts of non-renewable natural resources for road construction creates significant damaging impacts on the local environment and its inhabitants.

The discovery for the use of bone ash as an expansive soil stabilizer solved the disposal problem faced by abattoir agencies and also reduced the cost of improving infrastructure that is maintenance cost. Therefore, using a mixture of lime and bone ash economical solution for Ethiopia as it is available in large quantities According to [13], [14] between 2008 and 2011 has yielded approximately 192000 to 330000 tons of bone waste annually.

This study investigated the stabilization of black cotton soil with lime and animal bone ash for the improvement of subgrade soil.

1.3. Research questions

1. What are the engineering properties of existing black cotton soil in Konta-Ameya?
2. What are the engineering properties of soil stabilized under the addition of lime, AB ash and both mixture of stabilizers?
3. What optimum amount of stabilizing agent will be needed to attain the required properties of soils that can be suitable for subgrade?

1.4. Objective

1.4.1. General Objective

The main objective of this study is to investigate the stabilization of black cotton soil with lime and animal bone ash for the improvement of subgrade soil.

1.4.2. Specific Objectives

The specific objectives of the study are:

- To identify the engineering properties of existing black cotton soil.
- To evaluate the engineering properties of stabilized black cotton soils under the addition of lime, AB ash and both mixture of the stabilizer; and
- To determine the optimum content of stabilizer to improve the engineering property of black cotton soil.

1.5 Significance of the study

The production of industrially manufactured stabilizers, such as cement and lime released $C0_2$ into the atmosphere during the process of production, which is responsible for global warming. So, it is important to find another environmentally friendly option and cost advantage.

On the other hand, the extraction of a substantial amount of non-renewable natural resources for construction projects creates significant damaging impacts on the local environment and its inhabitants. Therefore, construction techniques implemented to solve the socio-economic problems need to found not only time and cost-effective but also environmentally friendly.

Other researchers will use the findings as a reference for further research and Provide information on the effective stabilization of black cotton soil with lime and animal bone ash for the improvement of subgrade soil.

1.6 Scope of the study

This study covered the stabilization of black cotton soil with lime and animal bone ash for the improvement of subgrade soil. Bone ash was collected from Konta-Ameya and the lime source was from Sankale Lime Factory. A representative soil from three different locations was collected and samples were disturbed and taken from 1.5 m depth weights 300 kilograms from Konta-Ameya. The soil samples were first air-dried and laboratory tests were conducted according to ASTM and AASHTO soil testing standard procedures.

The present study was conducted by taking limited parameters of Atterberg limits, free swell, moisture density relation, CBR, and CBR swell potential on stabilization by hydrated lime and animal bone ash. Also, the findings are considered indicative rather than definitive for the whole study area. Correspondingly, the study compared the results with standard specifications similarly a recommendation drawn and forwarded.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

Soil engineers did not recognize the problem of expansive soils until late 1930. Before 1920, infrastructures were believed to move depending on their respective self-weight. The light-weighted structures show relatively small cracks. Damages are noticed with different stages without the recognition of expansive soils. As a consequence of the structural damage, the potential problems associated with expansive soils have been recognized and preventive measures are being incorporated into new designs and Construction works [19]. Expansive soils Expand when they get wet and cracks when they dry. The swell and volumetric change of expansive soils have increased the interest of Engineers in the area. Expansive clays are different in that near to surface; clay often varies in density and moisture conditions from the wet season to the dry season. For example, near-or at-surface clays often dry out during periods of drought but then expand during the rainy season [20].

Those excessive volume changes cause serious distress and damage to engineering structures such as buildings and roads built on them. Pavements are particularly susceptible to damage made by expansive soils because they are lightweight and extended over large areas. The damage caused to the roads varied from the development of fine cracks on the road surface to premature pavement failures. As a result of these; vehicle operating cost increases, traffic accident increases, travel time increases and a lot of money is usually spent on rectifying the damages to pavements built on expansive soil.

Moreover, problems associated with construction over expansive soils are usually the seasonal moisture changes in sub-grade soils rather than the low bearing strength, as expansive soils are often relatively strong at equilibrium moisture content. Generally for road construction over expansive soils, it is essential to address the influence of the expansive soils both as naturally occurring undisturbed soils beneath the road and as compacted soil in the road formation. To solve this the practical treatment and design alternatives for highway construction on expansive clay soil separated into the following broad categories.

1. Re-route alignment or choose an alternative location to avoid the problem.
2. Pre-wet expansive soils to achieve post-construction equilibrium moisture contents

3. Remove or dig out expansive soil and replace it with non-expansive fill either entirely or a proportion (generally about 1 meter).
4. Prevent moisture changes in expansive soils utilizing barriers, both horizontal and vertical.
5. Improve the expansive soil by stabilization.

In general, Option 1 is not practical for highway use because the problem soils tend to occur over broad areas and option 2 tends to be impractical as well, while Option 3 and 4 have been the most commonly used methods. However, due to improvement in technology coupled with increased transportation costs, Option 5 is being used more often today and is expected to dramatically increase in the future.

2.2 Subgrade Soil

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is desirable to locate the pavement on the stronger soils if this does not conflict with other constraints. For this reason, the pavement engineer should be involved in the route selection process when choices made in this regard influence the pavement structure and the construction costs [21].

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density, and moisture content for the soil in question. This relationship must be determined in the laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction.

The moisture content (MC) of the subgrade soil is governed by the local climate and the depth of the water table on the road surface [21]. According to [21]volume 1 (Flexible pavements and gravel roads) chapter, three explains details concerning subgrade materials. According to the manual, the strength of the Subgrade soil is assessed by the type of soil, its density, and moisture content. According to [21]manual subgrades are classified from S1 to S6 based on the California bearing ratio (CBR) and are illustrated in the table below.

Table 2.1: CBR range subgrade class [21].

Serial No.	Class	CBR Range (%)
1	S1	< 3
2	S2	3,4
3	S3	5,6,7
4	S4	8-14
5	S5	15 - 30
6	S6	>30

According to the soil and materials investigation report, sections of the route with CBR greater than 5% and swell of about 2% can be used for Embankment construction that needs to be covered with blanketing material. From Bowls, 1992 CBR values and the quality of subgrades in pavement design are explained below.

Table 2.2: CBR range Subgrade quality [22]

Serial No.	CBR (%) Range	Subgrade Quality
1	0-3	Very poor subgrade
2	3-7	Poor to fair subgrade
3	7-20	Fair subgrade
4	20-50	Good subgrade
5	50+	Excellent subgrade

Unsuitable subgrade soil of Clay material having a Liquid Limit (LL) exceeding 60; or a Plasticity Index (PI) exceeding 30; or CBR value less than 3% at 95% of modified AASHTO compaction [23] method T-180 after four days soaking; or a swell value of more than 3% (with two surcharge rings) when determining per [23] T-193 at 95% of modified AASHTO compaction.

The AASHTO classification is given in [23] M145. It includes seven basic groups (A-1 to A-7) and twelve subgroups. Of particular interest is the Group Index, which is used as a general guide to the load-bearing ability of soil. The group index is a function of the liquid limit, the plasticity index, and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed an inverse ratio to its group index, i.e. a group index of 0 indicates a “good” subgrade material and a group index of 20 or more indicates a poor subgrade material.

Table 2.3: AASHTO soil classification system [23] M-145

General classification	Granular materials							Silt clay materials (more than 35% passing 75µm (No.200))			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis,% passing:											
2.00mm (No.10)	50 max.										
0.425mm (No.40)	30 max.	50 max.	51 min.								
75µm (No.200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	37 min.	38 min.	39 min.
Characteristics of fraction passing 0.425mm(No.40)											
Liquid Limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.
General type of significant constitute materials	Stone fragments gravel and sand		Fine sand	Silt or clayey gravel and sand				Silty soil		Clayey soil	
General rating as sub-grade	Excellent to good					Fair to poor					

2.3. Source of Expansive Soils /Source of Weak Subgrade

The parent materials for expansive soils can be classified into two groups. The first group comprises basic igneous rocks. This group comprises minerals named Feldspar and Pyroxene. The decomposition of these minerals forms an important mineral called smectite (montmorillonite) and other secondary minerals [5].

The formation of montmorillonite was probably the weathering and erosion in the highlands and carried by streams to the coastal plains. Besides, volcanic eruptions sending up clouds of ash felt on the plains and the seas with the ashes to be altered to montmorillonite [5].

The Presence of montmorillonite clay in these soils imparts them highs well–shrink potentials. Low rainfall has hindered the weathering of the active Montmorillonite mineral into low active clay types such as Illite and kaolinite. Further, the rainfall has not been sufficient to leach the clay particles far enough so that the overburden pressure can control the swell [19]. The second group comprises sedimentary rocks. The rock comprises smectite (montmorillonite) as a constituent and breaks down physically to form expansive soils. Smectite (montmorillonite) is one of the main sources of clay materials that form expansive soils [5].

2.4. Distribution of Expansive Soil

Expansive soils are widespread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria, etc. In other parts of the world case of expansive soils has been widely reported in countries like the USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela, etc. [24]. Besides, the aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres [17], [25]. They are widely spread in the central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. Also, areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure 2-1 [26], [27].

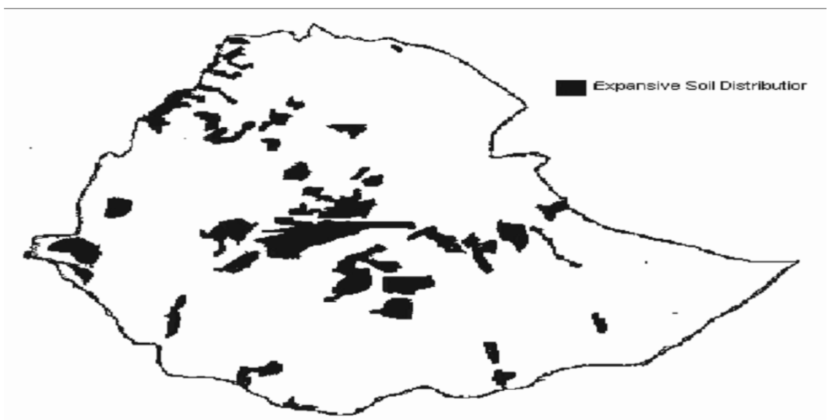


Figure 2.1: Distribution of expansive soil in Ethiopia [26], [27]

2.5. Mineralogy of Expansive Soils

The expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less. However, according to [5], the grain size alone does not determine clay minerals and he highlighted that the most important property of fine-grained soils is their mineralogical composition. The common groups of clay minerals are Kaolinite, illite, and montmorillonite (smectite) most important in engineering studies. Montmorillonite is the clay mineral that causes most of the expansive soil problems. The name Montmorillonite is used currently as a group name for all clay minerals with high expansiveness potential [5].

I. Kaolinite

Kaolinite is a typical two-layered mineral having a tetrahedral and an octahedral sheet joined to form a 1 to 1-layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water. The montmorillonite groups of clay minerals have a 2 to a 1-layer structure formed by an octahedron sandwich between two tetrahedrons [28]. These clay groups have a significant amount of magnesium and iron sandwiched into octahedral layers. The most important aspect of the montmorillonite clay mineralogy group is the ability for water molecules to be absorbed between the layers, causing the volume of the minerals to increase when they come in contact with water. The Illite clay minerals have a structure similar to that of kaolinite, but are typically deficient in alkalis, with less aluminum substitution for silicon, magnesium, and calcium can also sometimes substitute for potassium and illites are a non-expanding type of clay minerals [29].

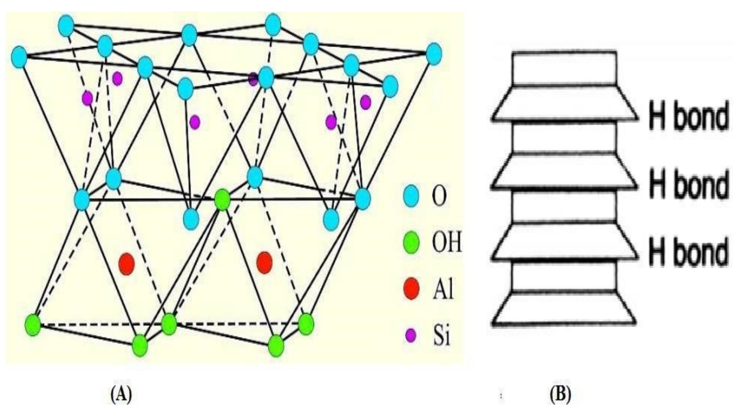


Figure 2.2: Diagrammatic and schematic representation of kaolinite [29]

II. Montmorillonite

Montmorillonite is a three-layer mineral having a single octahedral sheet sandwiched between two tetrahedral sheets [5]. The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the combined sheets due to these ions. Considerable swelling of Montmorillonite being can occur due to additional water observed between the combined sheets [30].

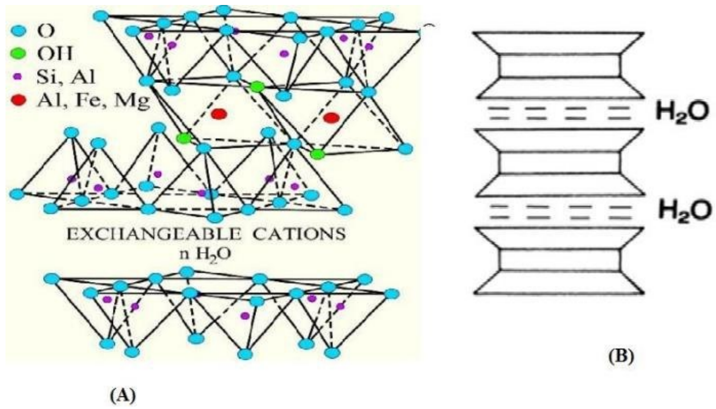


Figure 2.3: Diagrammatic and schematic representation of Montmorillonite [29]

III. Illite

Illite has a similar structure to that of Montmorillonite, but some of the silicon atoms are replaced by aluminum and also potassium ions are present between the tetrahedral sheet and adjacent crystals [5]. The layers of Illite clay minerals are more strongly bonded together than the Montmorillonites. In terms of Cation exchange capacity, inability to absorb and retain water, and physical characteristics Illite is intermediate in activity between clays of kaolinite and Montmorillonite. The layers of Illite clay minerals are linked together by fairly weak bonding to potassium ions held between them [30].

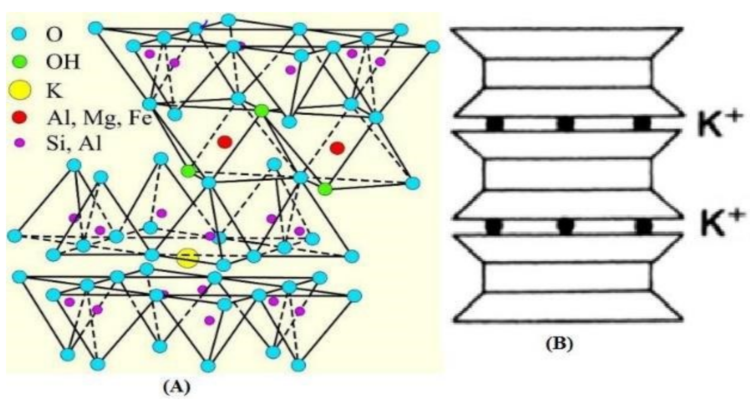


Figure 2.4: Diagrammatic and schematic representation of Illite [29]

2.6 Factors Affecting Expansive Soil Swelling and Shrinkage

According to [28], the expansive soil's swelling and shrinkage affecting factors are summarized in table 2.1.

Table 2.4: Factors affecting expansive soil swelling [28]

Factors	Description
Clay mineralogy	Clay soils that have clay minerals with higher swelling potential like Montmorillonite have hinge swelling potential. As the amount of clay mineral with a high swell, potential increases the swelling potential of the soil increases.
Initial water content	A small amount of initial water content, on the other hand, indicates a small degree of saturation. The tendency of soil to observe water will increase and this condition increases swelling potential.
Particle size	Fine particles in soil exist densely, and the finer the particle the higher will be its expansion potential.
Dry density	The higher the value of initial dry density implies, a closer particle spacing has a large swelling potential.
The concentration of pore fluid salts	The higher concentration of Cation in the pore field decreases expansion potential.
Pore field composition	Prevalence of monovalent Cation increases swelling potential while divalent inhabit shrinkage.
Climate	Arid climate courses desiccation of water content. This reduction of water content may lead to an increase in the swelling potential of clay.
Location of the water table	Fluctuating the location of the water table causes a variation of water content along with the depth of the clay stratum, and the water content variation affects the soil swell-shrinkage property.
The thickness of clay stratum and confining pressure	The high thickness of soil strata and large confining pressure reduces the soil's swelling potential.
Field permeability	Joints and fissures in the soil allow to pass water through and significantly affects the swelling capacity

2.7. Classifications and identifications of expansive soils

Most of the national codes of practice do not give characterization and classification of expansive soils. A simple user-friendly approach based on the free swell ratio, defined as the ratio of the sediment volume of soil in distilled water to that in carbon tetra-chloride or kerosene, is formulated considering the compatibility of the results with Odometer free swell tests and the soil clay mineralogy. Statistical illustrations are provided which indicate the assessment of soil expensiveness based on index properties is an overestimation. There is a consistency in the classifications based on odometer test results and the proposed approach [5].

Expansive soils can be recognized by using mineralogical identification, indirect index property tests, or direct expansion potential tests. The expansiveness of soil is governed by the type and proportion of clay minerals it contains. Knowing the type and proportion of the clay mineral in soil indicates the swelling potential [5].

2.7.1 Field Identification

It is easy to recognize expansive soils in the field during either dry or wet seasons. Their color varies from dark grey to black. During dry seasons, shrinkage cracks are visible on the ground surface with the maximum width of these cracks reaching up to 20 mm or more and they travel deep into the ground. A lump of dry black cotton soil requires a hammer to break. A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail. During rainy seasons, these soils become very sticky and very difficult to traverse. The appearance of cracking in the nearby structures is also indicative [2], [28].

2.7.2 Laboratory Identification

Several laboratory tests are useful in identifying expansive soils. Generally, these can be categorized as mineralogical identification, direct and indirect methods [5].

2.7.2.1. Mineralogical Methods

According to [28] type of clay, a mineral is a fundamental factor, which determines the expansive behavior of soil. A mineralogical test is used to identify this mineral. There are different types of techniques, which are used to identify the clay mineralogy. The common types of these techniques include X-ray diffraction, differential thermal analysis, and electron microscopy. Others also include infrared spectroscopy, dye adsorption, and radiofrequency electrical dispersion

2.7.2.2. Direct Method

The second method is called direct measurements. These methods are the most useful data for practicing Engineers. These methods offer the most useful data by direct measurement, and tests are simple to perform and do not require complicated equipment. Testing should be performed on many samples to avoid erroneous conclusions. direct measurements are the most satisfactory and convenient methods to determine the swelling potential and swelling pressure of expansive clay [5].

Direct measurements of expansive soils can be achieved by the use of the conventional one-dimensional consolidometer. The consolidometer can be platform type, Scale type, or other arrangements. The soil sample is enclosed between two porous plates and confined in a metal lying. The soil sample can be flooded both from the bottom and from the top [5].

2.7.2.3. Indirect Methods

This method has been used to investigate the swelling potential of soil by examining other parameters, which indirectly give information about the soil property. These include Index Property Tests, Cation Exchange Capacity, and Potential Volume Change tests [5]. The liquid limit and plasticity index are useful for determining the swelling characteristics of most of the clays and prepared a chart to support the identification [31].

Generally, the Ministry of Works and Urban Development (2009) described that in Ethiopia all grayish or brownish clays with a plasticity index greater than 25% can identify as expansive. The classification or rating from low potential to high heaving potential usually depends on the clay content and plasticity. These methods are related to laboratory soil identification and are vital for the intended purposes [28]

A. Atterberg Limits

Atterberg limits define the moisture content boundaries between states of consistency of fine-grained soils. Clay soil can exist in four distinct states of consistency depending on its water content. The water content at the boundaries between the different states is defined as the shrinkage, plastic, and liquid limits. Two useful indices may be computed from the atterberg limits and the natural moisture content. These are the Plasticity Index and Liquidity Index. The Plasticity Index is used extensively for classifying expansive soils and should always be determined during a preliminary investigation [28].

Table 2.5: Soil Expansivity Predictions by Liquid Limit [5], [33]

Degree of Expansion	Liquid Limit (%)	
	Chen	IS 1498
Low	<30	20-35
Medium	30-40	35-50
High	40-60	50-70
Very High	>60	70-90

Table 2.6: Soil Expansivity Predictions by Plastic Limit [5], [31], [33]

Degree of Expansion	Plastic Limit (%)		
	Holtz and Gibbs	Chen	IS 1498
Low	<20	0-15	<12
Medium	12-20	10-35	12-23
High	20-35	20-55	23-32
Very High	>35	>35	>32

Table 2.7: Relation between the swelling potential of clays and the plasticity index [5]

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

While it may be true that high swelling soil will manifest high index property, the converse is not true [5].

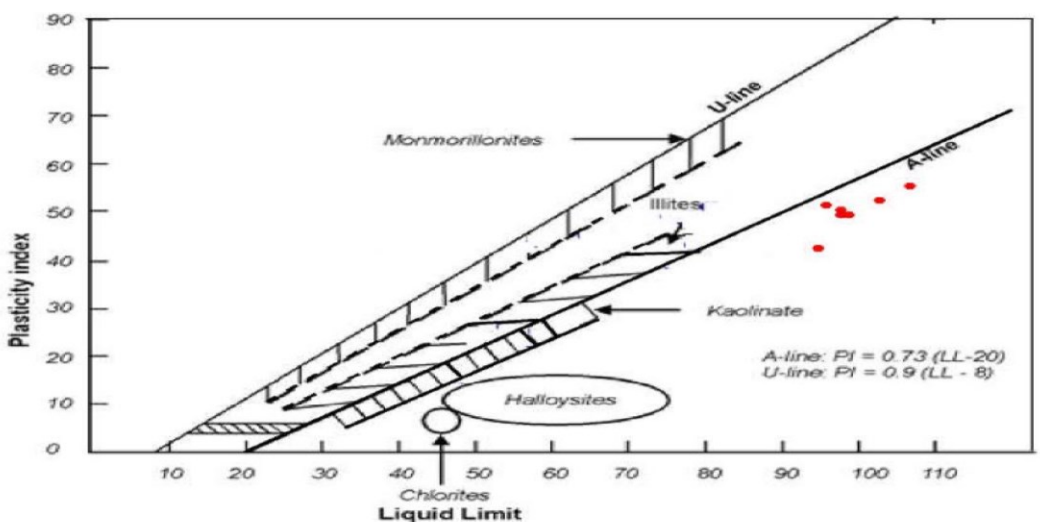


Figure 2.5: Plot of clay mineral on plastic index chart adapted from [16]

B. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling Potential of expansive clay. The procedure involves taking two oven-dried soil samples Passing through the 425µm sieve, 10cc each was placed separately in two 100ml graduated soil Samples. Distilled water was filled with one cylinder and kerosene in the other cylinder up to the 100ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index [19].

The free swell index is then calculated using Equation 2.1.

$$\text{Free swell index (\%)} = \frac{Vd - Vk}{Vk} * 100 \tag{2.1}$$

Where, V_d = the volume of soil specimen read from the graduated cylinder containing distilled water, and V_k = the volume of soil specimen read from the graduated cylinder containing kerosene.

The soils having a high free swell index value may show considerable volume Changes as compared to the soils having lower free swell index values. Mohan and Goel (1959) gave the following classification of degree of expansion based on the Free swell index values as given in Table 2.5, and the same has been suggested by Indian standard IS1498.

Table 2.8: Classification of degree of expansion based on the Free swell index [33]

Free swell index	Degree of expansion	Degree of severity
> 200	Very high	Severe
100 – 200	High	Critical
50 –100	Medium	Marginal
< 50	Low	Non-critical

C. Cation Exchange Capacity (CEC)

The CEC is the number of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milliequivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.6.

Table 2.9: Typical CEC values of basic clay minerals after Mitchell, 1976 [28]

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

D. Linear Shrinkage

The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple test. Theoretically, it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential [5]. [35] Suggests a relationship between linear shrinkage, shrinkage limit, and the potential of expansiveness as shown in Table 2.7.

Table 2.10: Shrinkage limit, linear shrinkage, and degree of expansion relationship [35]

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
< 10	> 8	Critical
10 –12	5 – 8	Marginal
> 12	0 – 5	Non-critical

2.8 Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in several different classification schemes. The classification system used for expansive soils is based on an indirect and direct prediction of swell potential as well as combinations to arrive at a rating. There are many classification systems. The following are some of the common methods.

2.8.1 General Classification

I. Unified Soil Classification Systems

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

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

The above classification system can be summarized as follow:

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

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

II. AASHTO Classification System

The AASHTO soil classification system is used to determine the suitability of soils for earthworks, embankments, and roadbed materials such as subgrade, sub-base, and base. According to this classification system, granular soils are soils in which 35% or less are finer than the No. 200 sieves (75 µm). Silt-clay soils are soils in which more than 35% are finer than the No. 200 sieves (75 µm). The system classifies soils into seven major groups, A-1 through

A-7. The first three groups, A-1 through A-3 are granular (coarse-grained) soils, while the last four groups, A-4 through A-7 are silt-clay (fine-grained) soils [28].

2.8.2 Classification Specific to Expansive Soil

The general classification systems are found to be suitable for identification of expansive soils or prediction of swelling characteristics or Expansion Potential, but it does not provide useful information. A parameter determined from the expansive soil identification tests has been combined in some different classification schemes to give the qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas [28]. Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information. An indirect prediction of swell potential includes correlations based on index properties, swell, and a combination of them. Some of such classification systems are enumerated below.

a) Method of Chen

As [5] Presented a single index method for identifying expansive soils using only plasticity index. Chen suggested four classes of clays according to their plasticity indices shown in table 2.7.

b) Method of Daksanamurthy and Raman (1973)

[36] Presented a single index method for identifying expansive soils using only liquid limits. They suggested four classes of clays according to their liquid limits as shown in Table 2.8.

Table 2.11: Relation between the swelling potential of clays and the liquid limit [19]

Swelling potential	Liquid limit
Low	$20 < LL \leq 35$
Medium	$35 < LL \leq 50$
High	$50 < LL \leq 70$
Very high	$LL > 70$

c) USBR Method

This method is developed by [31]; it is based on a direct correlation of observed volume change with colloid content, plastic index, and shrinkage limit. The classification is as given in Table 2.9.

Table 2.12: Classification based on the bureau of reclamation method [5], [37]

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

d) Method of Seed et al

After an extensive study on swelling characteristics of remolded, artificially prepared, and compacted clays, [38]. [5] has developed a chart based on activity and percent clay sizes as shown in Figure 2.4. The activity here is defined as;

$$AC = \frac{PI}{C - 10} * 100 \qquad 2.2$$

Where; A = activity, C= percentage of clay-sized finer than 0.002mm, PI= plasticity index

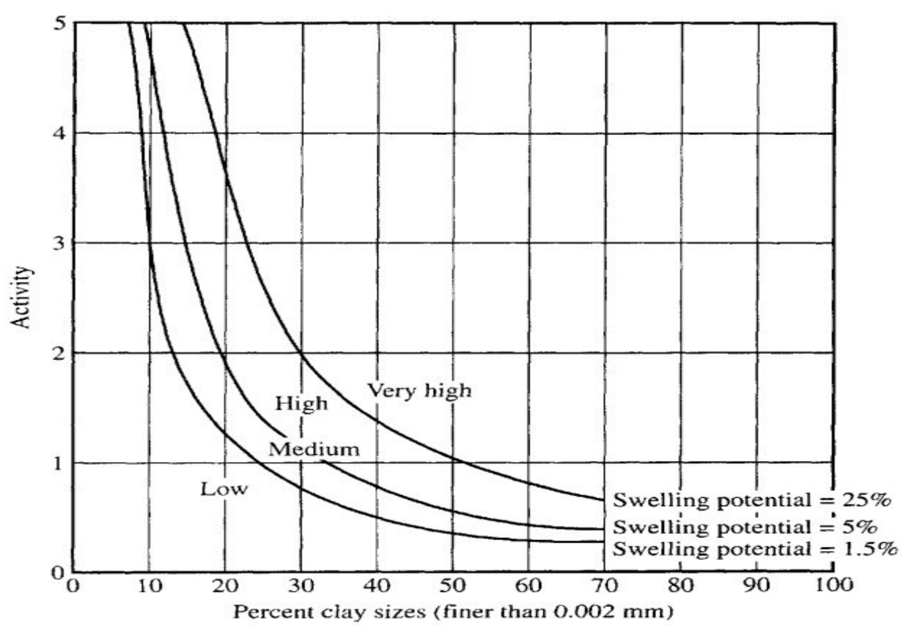


Figure 2.6: Classification chart for swelling potential according to [38].

e) Method of Skempton

This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity. Activity is defined as:

$$A = \frac{Ip}{C} * 100 \qquad 2.3$$

Where: Ip =plasticity Index, C=percentage of clay size finer than 0.002mm by weight Skempton suggested that three classes of clays according to their activity shown in Table 2.10.

Table 2.13: Relation between clay activity and potential of expansion [5]

Activity	Potential for expansion
Ac < 0.75	Low (inactive)
0.75 < Ac < 1.25	Medium (normal)
Ac > 1.25	High (active)

2.9. Black Cotton soils

Black cotton soils are inorganic clays of medium to high compressibility. They are characterized by high shrinkage and swelling properties. Because of their high swelling and shrinkage characteristics, black cotton soils have been a challenge to highway engineers. The Black cotton soil is very hard when dry, but loses its strength completely when in wet condition and owe their specific properties to the presence of swelling clay minerals, mainly montmorillonite. Because of the wetting and drying, massive expansion and contraction of the clay minerals take place. Contraction leads to the formation of wide and deep cracks. These cracks can be wide enough to make the terrain treacherous for animals [39].

[5] Observe that black cotton soils are imperfect to poorly drained, leaching of soluble weathering products is limited, the contents of available calcium and magnesium are high and the pH is usually above seven. Once they have reached their field capacity, practically no water movement occurs; this is due to the very low hydraulic conductivity of the soil. Flooding leading to road structure damage can be a major problem in areas with higher rainfall. Surface water may be drained by open drains, but mole' drainage is virtually impossible. Often, black cotton soils exhibit variations in properties, particularly strength and volume change properties following variation in their in-service moisture contents. These properties limit their performance as a support element for pavements. Typical problems include shrink-swell, settlement, collapse, erosion, or simply insufficient strength.

Particular problems associated with road construction over black cotton soils are commonly the seasonal volumetric changes in these soils. Typical distress is from seasonal wetting and drying whereby soils at the edge of the road wet up and dry out at a different rate than those under a bituminous surfacing. This mechanism causes permanent deformation (rutting) over the cross-section of the road and associated crack developments, first occurring in the shoulder area, subsequently developing in the carriageway [17].

2.10. Soil Stabilization

Soil stabilization is a process whereby increased strength and stability of the soil are attained mainly by mechanical or chemical means. The most common improvements attained through stabilization include better soil gradation, reduction of plasticity index or swelling potential, increase in durability and strength [8].

When unsuitable materials are encountered measures like avoiding the route, redesigning the pavement with thicker sections, or replacing the poor soil with good quality materials are practical but increasingly expensive options. With improved technological advances and concern for the reduction of non-renewable resources, improving the properties of soil using chemical additives is gaining increased popularity [15].

2.11. Methods of Soil Stabilization

2.11.1. Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil [6]. It is common to use both mechanical and chemical means to accomplish specified stabilization. The main methods of mechanical stabilization can be categorized into compaction, mixing or blending of two or more gradations, applying geo-reinforcement, and mechanical remediation [15].

[40]proves that mechanical stabilization is best suited for coarse-grained soils or aggregates at optimum or below optimum moisture contents. However, clayey soils are more effective under chemical stabilization. If the clayey soil is mixed with the specific stabilizer just enough to make it workable, better in texture and compactible regardless of the strength and durability, then it is referred to as modification; modification is restricted to the soil having AASTHO designation A-4, A-5, A-6, and A-7.

2.11.2. Chemical Stabilization

Chemical stabilization is a method of improving the engineering properties of a material by adding chemical substances. Chemical stabilization is used for a wide range of purposes including improving the bearing capacity and strength of pavement layers, dry temporary bypasses during rainy periods, delay certain chemical reactions that are harmful to road soils or aggregates, dry out the soil where the moisture content is too high for successful compaction, make the soil less permeable where necessary, reduce the plasticity of soils used in road construction and thereby reducing the effect of moisture variations, changing clay to a more granular and workable material and reducing swelling and shrinkage properties [41].

[42] classify chemical stabilizers into three groups:

- **Traditional stabilizers**: such as hydrated lime, Portland cement, and Fly ash;
- **Non-traditional stabilizers**: comprised of sulfonated oils, ammonium chloride, enzymes, polymers, potassium compounds and
- **By-product stabilizers**: which include cement kiln dust, lime kiln dust, etc.

Over and above these, there are inorganic and hydraulic products that are often clustered into non-traditional soil stabilizers. Perhaps a better term for them would be `hydraulic soil stabilizers' [43].

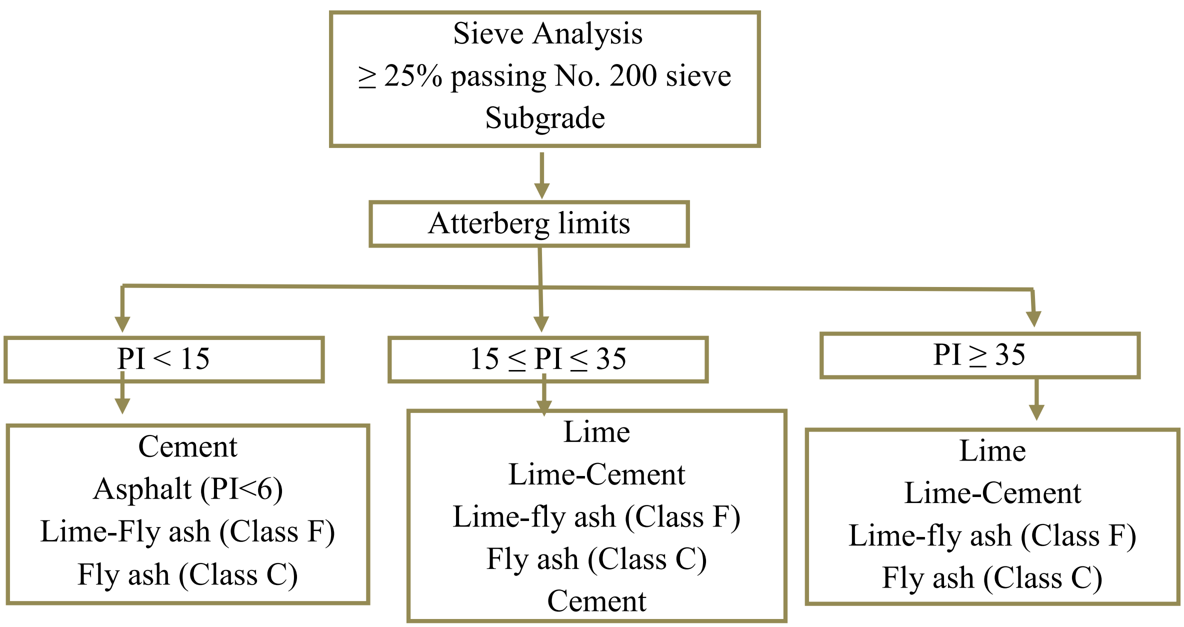


Figure 2.7: Decision tree for selecting stabilizers for sub-grade soils Adapted from [42]

This research evaluates the suitability of black cotton soil with a mixture of lime and animal bone ash for soil stabilization. Accordingly, the respective soil additives and their mechanisms of stabilization are briefly discussed in chapter three of this thesis.

2.12. Lime Stabilization

The use of lime to dry, modify, and stabilize soil is a well-established construction technique [44]. The treatment of pavement sub-grades with lime can considerably improve the engineering properties of a wide range of soils; typically medium, moderately fine, and fine-grained soils.

According to [23], lime has several effects summarized as follows;

- **Soil drying:** is a rapid decrease in soil moisture content due to the chemical reaction between water and quicklime and the addition of dry material into moist soil.
- **Modification:** is an improvement that occurs in the short term, during, or shortly after mixing (within hours). The modification reduces the plasticity of the soil and improves short-term strength to the desired level.
- **Stabilization:** is a longer-term reaction that is derived from the hydration of calcium-silicates and/or calcium aluminates in Portland cement or due to pozzolanic reactivity between free lime and soil or added pozzolans. A soil that is lime stabilized also experiences the effects of soil drying and modification [21].

2.12.1. Mechanisms of Lime Stabilization

Lime-soil reactions can broadly be grouped into initial and longer-term [23]. The initial reactions involve cation exchange by replacing the exchangeable Na^+ or K^+ ion in the clay with the Ca^{2+} ion of the lime. By this replacement of ions, the double water layer around the clay particle will decrease in thickness resulting in a significant change of the plasticity characteristics of the soil [6].

Longer-term reactions involve interactions between free lime $Ca(OH)_2$ and soil particles. These interactions are referred to as pozzolanic as they involve pozzolans, the alumina, and silica made available from the soil by the high pH limewater solution. When these pozzolans react with free lime and water, a cementing effect among particles as well as an alteration of surface mineralogy occurs. These pozzolanic reactions contribute to an increase in strength which can be considerable depending on the mineralogy of the soil [23].

The gain in strength associated with the formation of pozzolanic reactions is accelerated by heat, an advantage when using lime stabilization in hot climates. Strength also increases with time. During the first one or two days after construction, this increase is rapid. Thereafter, the rate slows down although strength gain continues provided the layer is well cured [21].

2.12.2. Mix Design and Strength Characteristics

When using lime as a stabilizer, the goal of the mixture design is to find the optimum lime content to adequately stabilize the soil to meet desired strength requirements [45]. Several procedures such as the Illinois procedure, Thompson Procedure, Eades and Grim Procedure, and the Texas Procedure, as summarized by [46] involve comparing results of strength testing using varying lime contents until a lime content that provides the maximum strength is encountered. For the Thompson Procedure, Eades, and Grim Procedure, the optimum lime content is first estimated by measuring the pH of several soil lime mixtures with varying lime contents. The lowest lime content that provides a pH of 12.4 is then used as the starting point for determining the optimum lime content. The Texas Procedure, as summarized by [46], first estimates the optimum lime content using the plasticity index of the soil and the percentage of soil passing the No. 40 sieves. After estimating the optimum lime content, strength testing is then used to verify the actual optimum lime content.

The optimum amount of lime added to expansive clay was estimated using the pH test method according to *ASTM D6276-99a*. This test method provides a means for estimating the soil-lime proportion requirement for the stabilization of soil. This test is performed on soil passing the 425µm sieve. The optimum proportion is determined at a pH of 12.4.

The Illinois procedure is based on the effect of lime on the plasticity index of the soil treated with various percentages of lime. The lime-soil-water mixture should be cured for one hour before testing. A plasticity index versus lime content is prepared. The design lime content may be designated as either that lime content above which no further appreciable reduction in *PI* occurs or a minimum lime content, which produces an acceptable *PI* reduction.

While the procedures outlined above help to identify the lime content that will provide the greatest strength, many factors influence the strength of soil-lime mixtures. The variability of these factors makes it practically impossible to pinpoint the strength that may be achieved for lime stabilization of a particular soil. Therefore, the strengths of soil-lime mixtures must be verified through strength tests such as CBR, unconfined compressive strength, or resilient modulus. Lime contents between 2 to 10 percent are typically capable of producing significant strength gains [47].

2.13. Animal Bone Ash Stabilization

One type of agricultural waste that is of great concern in both rural and urban areas of Ethiopia is abattoir or slaughterhouse waste. Abattoir wastes often contain horns, blood, bones, fat organic and inorganic solids, salts, and chemicals added during processing operation [48]. The bones partially trash are seen around most slaughters houses and market areas in major towns in Ethiopia that area called "Qera". These wastes are not decayed rather causing the environment polluted and unclean causing different environmental and health problems. Pressures [49]. Alternative means of disposing of this waste are sourced in this study that would be environmentally friendly as well as contributing to the income of people.

According to [50] to Bone is a dynamic tissue that performs mechanical, biological, and chemical functions. The main component of bone is hydroxyapatite, as well as amorphous forms of calcium phosphate possibly including carbonate. Bone chemical and physical properties are affected by age, nutrition, hormonal status, and disease. Borne Ash (BA) is a whitish powdery residue left from the burning (calcination) of bones. The ash is composed majorly of P_2O_3 and CaO_2 in the form of calcium phosphate Ca_3 $(PO_4)_2$ or modified hydroxyapatite (Ca5 $(PO_4)_3OH$).

The main chemical compositions of natural bone are expressed in terms of calcium Oxide and phosphorus pentoxide and at 32.1%, 28.8% respectively. Bone ash is obtained from the calcination of bone at approximately 1100^o C and then cooling, milling and sieving through the 0.425 mm mesh sieve. Bone Ash can also be made from bovine bones by burning the bones at 100^o C for about six to twelve hours and then placed in a muffle furnace heated up to 1000^o C. The leftover or residue is a mineral shell of hydroxyapatite, the calcium phosphate [51]. Elementary chemical composition of bone ash expressed in percent shown in table 2.4

Table 2.14: Selected chemical composition values of ABP using MP-AES [49]

Animal bone powder	Ca	Si	Al	Fe	Mg	Na	K	P
	58.64	1.8	0.21	0.66	0	0	0.63	29.3

Problem soil can also be improved with Cow Bone powder owing to its high content of CaO, an aspect that has not received attention before. Few studies record the use of BA in civil engineering. For example, Ayininuola and Denloye (2014) studied the effect of cattle bones on the strength properties of subgrade and sub-base soils. Cattle bones burn to ash and are added to soil at increasing proportions from 0 to 20%. The CBR of the soil increased with an increase in BA content until it reached 7% BA where after the CBR dropped steadily. Bone ash could thus be suitable for the stabilization of subgrade and sub-base soils up to 7%. This study shows the effectiveness of bone ash as soil stabilization.

This study has investigated the effect of animal bone ash addition to a mixture of black cotton soil and lime for the improvement of the soil's engineering properties. This was done through various laboratory tests by varying the respective proportions of the lime, bone ash, and also finding the optimum mixture for use of improving subgrade.

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Sampling site

Soil and bone samples were collected from Konta Special District along the route Cheka Bocha (7°06'09.99" N, 36°39'12.42" E and 2138 m), Ameya (7°06'21.23" N, 36°40'31.79" E and 2104 m) and Chida (7°10'03.64" N, 36°47'27.92" E and 1678 m), which is located in the Southern Nations, Nationalities, and Peoples Regional State (SNNPRS), Ethiopia. Also, hydrated lime was obtained from the market in Konta-Ameya which was produced by Sankale Lime Factory shown in Figure 3.1.

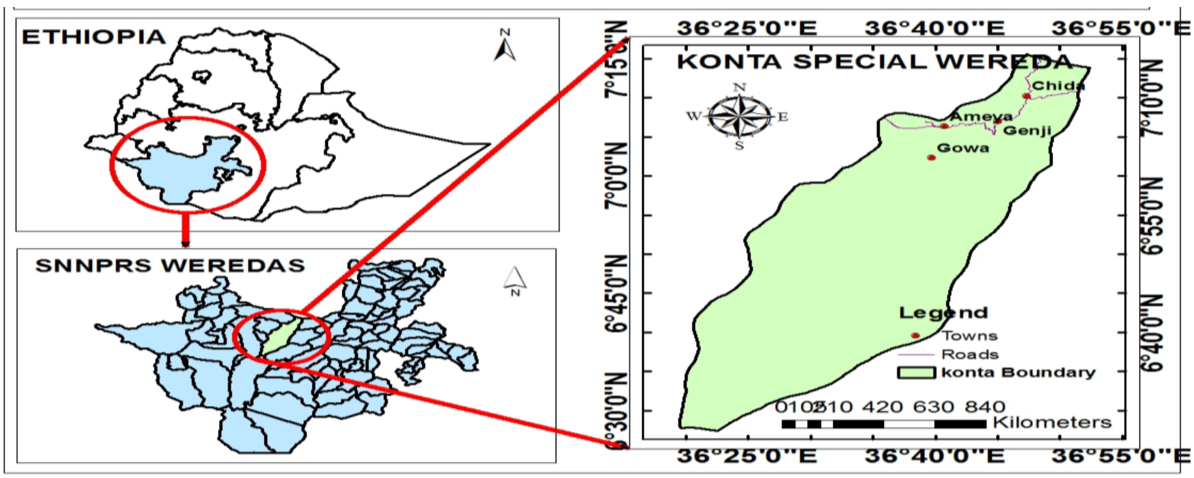


Figure 3.1: Location map of sampling sites (source http://www.earthexpoler.usgs.gov.com-using ArcGIS).

3.2 Study Design and Period

The study was conducted through a series of experimental investigation in the laboratory. The first step in the research work was identifying black cotton soil. The second step was sample collection. The third step was laboratory tests on natural and stabilized expansive soil. The fourth step was laboratory test data analyzed and interpreted then properties of expansive soil and its performances on requirement were addressed. Finally, the research findings and recommendations were expressed based on the laboratory test results.

The study period to conduct this thesis was completed within five months, starting from August 2020 up to the beginning of June 2021. The overall research design has shown in Figure 3.2.

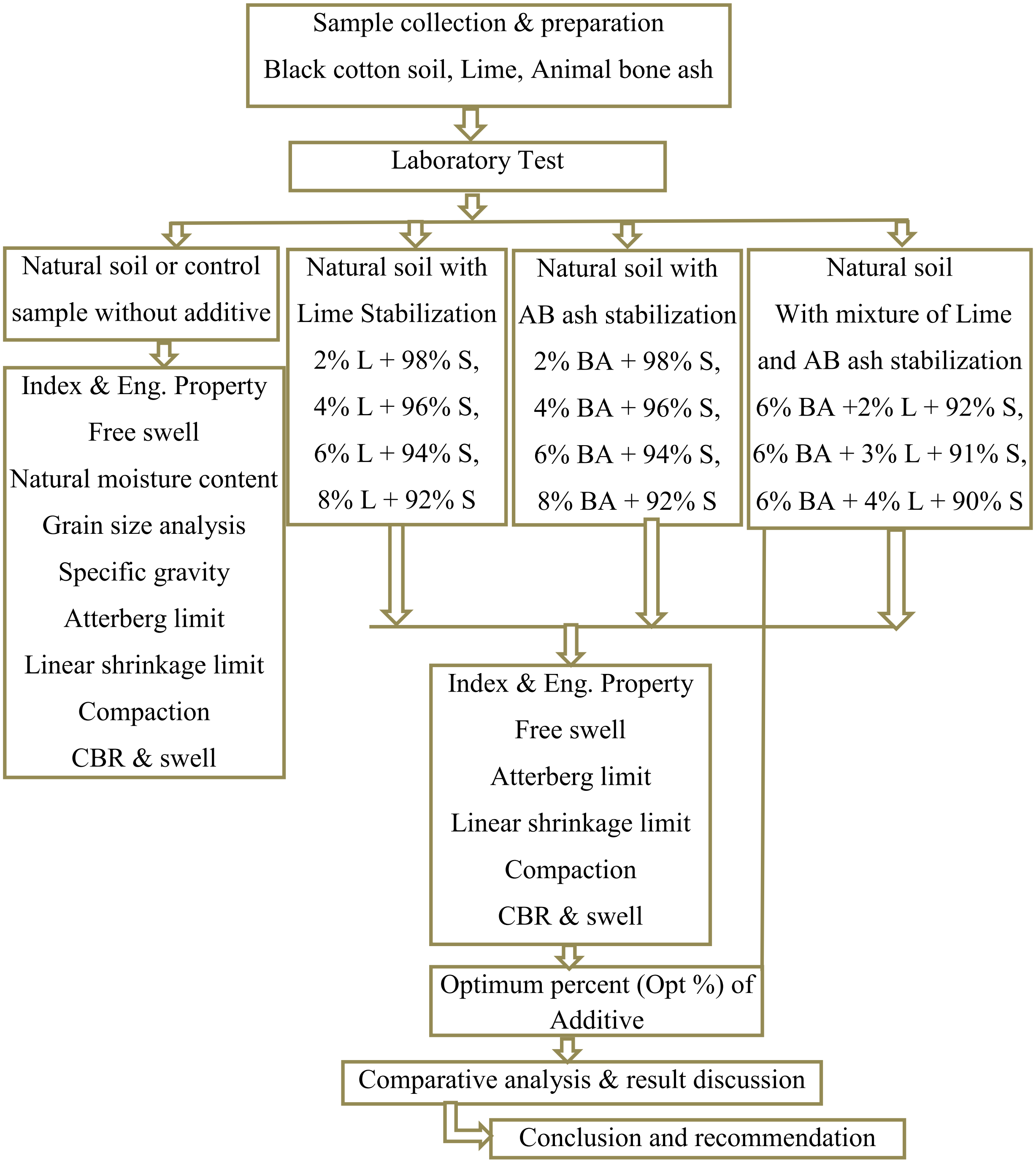


Figure 3.2: Flow chart of the study design

3.3 Study Procedure

The procedure utilized throughout the conduct of this research study is as follows: Review related literature on methods of stabilization, types of stabilizers, and properties of lime and bone ash includes articles, reference books, research papers, and standards specifications like ERA, AASHTO, and ASTM. Necessary data collection, laboratory tests, organization, comparison, and analysis were obtained, and then subsequently compared the results with preexisting literature and standard specifications. Conclusion and recommendation drew based on the results.

3.4 Study Variable

The study variables, which consist of the research, contained both independent and dependent variables.

3.4.1 Dependent Variables

The dependent variables are more related to the general objective of the study. Stabilization of black cotton soil with lime and animal bone ash for the improvement of subgrade soil was a dependent variable.

3.4.2 Independent Variables

The independent variables were compaction, gradation, atterberg's limit, moisture content, CBR. Specific gravity, linear shrinkage, and Contents of lime and bone ash.

3.5 Population

The population of this research was the black cotton soil, lime, and bone ash of the selected study area of Ameya town.

3.6 Sample collection techniques

The sampling technique used for this research was purposive sampling, which is a non-probability method. This sampling technique was proposed based on the information to determine the experimental investigation on the stabilization of the black cotton soil for the improvement of subgrade soil.

3.7 Materials collection

3.7.1 Black cotton soil

Moreover, based on observation and free swell test, expansive soil samples were selected around the Ameya town road segment which was under construction. Three boreholes were excavated using an excavator and shovel. According to [23] and [52], a 300 kilograms disturbed sample was collected at the depth of 1.5m to avoid the inclusion of organic matter shown in Figure 3-3.

Figure 3.3: Black cotton soil sample preparation (picture taken on 21/08/2020)

3.7.2 Lime

Fifty-kilogram Hydrated Lime was obtained from Sankale Lime Factory. The chemical composition of Sankale Hydrated Lime studied by [53] the composition result is presented in Table 3.1 and shown in Figure 3-4.

Table 3.1: Sankale Hydrated Lime chemical composition

Constituent	SiO_2	Al_2O_3	Fe_2O_3	Cao	MgO	Na_2O	K_2O	TiO_2	P_2O_5	MnO	SO_3
Percentage	6.21	2.18	3.57	59.47	3.91	0.61	0.79	0.3286	0.208	0.2785	0.58

Figure 3.4: Hydrated Lime (picture taken on 29/08/2020)

3.7.3 Animal Bone Ash

The animal Bone sample was obtained in the fresh state from the slaughterhouse (abattoirs) in at Konta-Ameya. The collected Animal Bone was dried and burnt under the controlled condition on the furnace of 900^0 C temperature to obtain ash form. After convert to ash then sieved through No.40 (0.425mm) to remove other unnecessary material. The fraction passing through the sieved was used during testing shown in Figure 3-5.

Figure 3.5: Preparation of bone ash (picture taken on 24/08/2020)

3.8 Sources of Data

Both primary and secondary data sources were used. The Primary sources of data for this study were experimental output and Secondary data was collected from different standards, journals, books, websites, and technical specifications.

3.9 Software and instruments

The following instruments and software were used for this study: meter tape, plastic bags, manual hand auger equipment, laboratory equipment, digital camera for documentation, MS Word, and Excel to analyze laboratory data were used in this study.

3.10 Data Collection Process

To attain the purpose of this research work, ethical considerations before, any data collection, a formal letter was obtained from Jimma institute of Technology.

Quantitative and qualitative data were utilized based on the necessary input parameters for the analysis by comparing them with AASHTO and ASTM manuals. The data collection process was included field visual inspection, field investigation, sampling, and laboratory tests and finally, the results from laboratory tests were compared with standard specifications.

3.11 Fieldwork,

After reviewing the literature and before conducting a laboratory test, the following activities were taken. Field observations were carried out and representative samples were taken to laboratory tests.

In the field, an observation was done by conducting a visual inspection and site inventory of the soil classification zones in the town. After finishing the initial visual inspection and categorizing the soil conditions of the road segments of the town and, the next step was to select the representative locations for sampling based on the availability of soil structures, which consists of different grain sizes that represent the types of soils with different grain sizes observed from the study area. From identified sites test pits were extracted for laboratory testing for further recommended road construction laboratory tests.

3.12 Sample preparation

Before treatment and testing, the sample was prepared by the method described in AASHTO T87-86. This process involves after the sample was collected from the site.

The natural soil sample was placed inside the thick-gauge plastic bags to prevent moisture and the natural moisture content of the sample is higher than desired for mixing therefore samples were air-dried to moisture content just below the target value. Then break up the soil aggregates by a rubber-covered mallet and adequately pulverized and then sieve analysis was conducted on properly pulverize natural soil. Sieving was conducted into three groups. The first team is soil samples passing #40 (0.425mm) sieve for Atterberg limits and free swell, 2.00mm sieve for specific gravity, and the third group is soil samples passing ¾ in (19 mm) for compaction and California bearing ratio.

On the other hand, the Animal Bone sample was obtained in a fresh state, crushed by a hammer to minimize size, and then was burnt under the controlled condition on a furnace for 8 hours at 900^0 C temperature to obtain the ash form. After conversion to ash then sieved through No.40 (0.425mm) to remove other unnecessary material and to ensure it is completely converted to ash. Finally, after preparation laboratory tests were done.

3.13 Experimental setup

The experimental setup was:

Dried soil sample, lime, and animal bone ash in percentages of the mixture of lime and animal bone ash (0%, 2%, 4%, 6%, and 8%) were mixed in proportion by weight to form various mixes. lime will first be added to the pulverized, sieved and air-dried soil sample and dry mixed thoroughly. AB ash was added after that and wet mixing was done by sprinkling the measured amount of water followed by a thorough mixing until a uniform soil-additive matrix was obtained.

Conduct laboratory tests with a mixture of lime and animal bone ash with different ratios to determine engineering properties to attain the set specific objectives. Also, each test was repeated three times to get a representative result.

The optimum stabilizing ratio for maximum improvement of the engineering properties of the soil was assessed and the result from the laboratory test was compared with the standard and specification of AASHTO and ERA. Finally, the research findings and recommendations were be forwarded based on the laboratory results.

3.14 Laboratory tests

The samples were collected from different sources subjected to various Geotechnical characterizations. The basic test such as sieve analysis, Atterberg limit, natural moisture content, compaction, Atterberg limit, and CBR of materials are investigated separately to know the natural properties of materials as per the relevant code of standard shown in Table 3.2.

Table 3.2: Laboratory test as per standard

Laboratory test	Standard
Free Swell Index test	IS 2720 part 40
Moisture Content	AASHTO T-256
Grain Size Analysis	AASHTO T-88
Specific Gravity	ASTM D 854-00
Atterberg Limits	AASHTO T90
Linear Shrinkage	(BS1377: Part 2:1990)
Soil Classification	AASHTO M-145
Proctor compaction test	AASHTO T-180
California Bearing Ratio and CBR Swell	AASHTO T-193 and AASHTO T-180

3.14.1 Free Swell Index Test

The test includes the determination of the free swell for the natural soil and soil-Lime Animal bone ash mixture. This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs (1956) and Indian standard IS 2720 (part 40) to measure the expansive potential of cohesive soils. But, in this research Indian standard IS 2720 (part 40) was used. The free swell test gives a fair approximation of the degree of the expansiveness of the soil sample. The procedure involves taking two oven-dried soil samples passing through the 425μm sieve, 10g each was placed separately in two 100ml graduated soil samples. Distilled water was filled with one cylinder and kerosene in the other cylinder up to the 100ml mark. The final volume of soil is computed after 24 hours to calculate the free swell index shown in Figure 3-6.

The level of the soil in the kerosene graduated cylinder shall be read as the original volume of the soil samples, kerosene being a non-polar liquid does not cause swelling of the soil. The level of the soil in the distilled water cylinder shall be read as the free swell level. The free swell index of the soil shall be calculated as follows:

$$\text{Free swell index } (\%) = \frac{Vd - Vk}{Vk} * 100 \qquad 3.1$$

Where, V_d = the volume of soil specimen read from the graduated cylinder containing distilled water, and V_k = the volume of soil specimen read from the graduated cylinder containing kerosene.

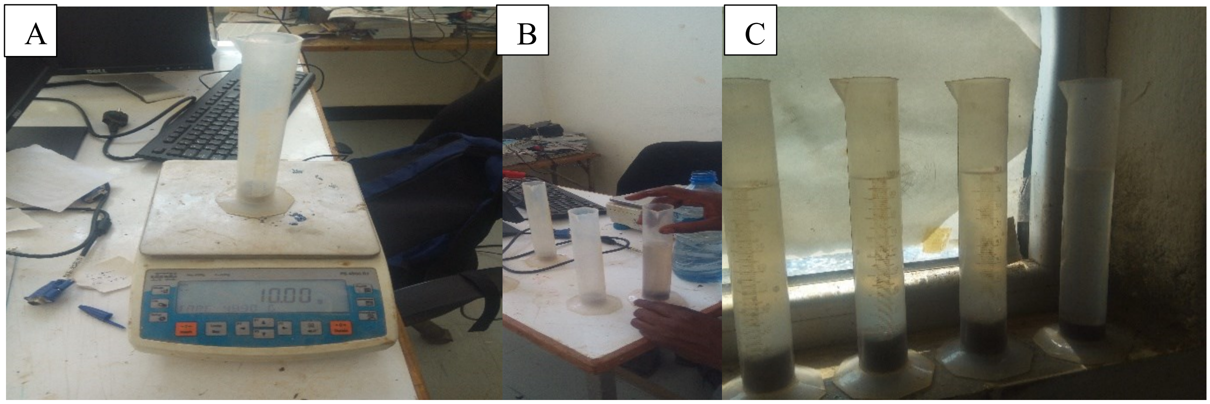


Figure 3.6: Free swell index test (taken on 27/08/2020)

3.14.2 Moisture Content (AASHTO T-256)

The oven-drying method was used to determine the moisture contents of the samples. In the oven-drying method, small, representative specimens obtained from large bulk samples were weighed as received, then oven-dried at 105°C for 24 hours. The sample is then reweighted, and the difference in weight is assumed to be the weight of the water driven off during drying. The difference in weight is dividing by the weight of the dry soil, giving the water content on a dry weight basis.

$$\text{Moisture content Mc } (\%) = \frac{\text{Wet weight} - \text{Dry weight}}{\text{Dry weight}} * 100 \qquad 3.2$$

3.14.3 Grain Size Analysis (AASHTO T-88)

This test was performed to determine the percentage of different grain sizes contained within the soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of finer particles. For this study, both wet sieve analysis and hydrometer analysis was done according to [52] D422-63. Lastly, the analysis will combined particle size distribution curve will be plotted as a figure.

3.14.4 Specific Gravity (ASTM D 854-00)

The weight of displaced water was calculated by comparing the weight of the soil and water in the flask with the weight of the flask containing only water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water shown in Figure 3-7.

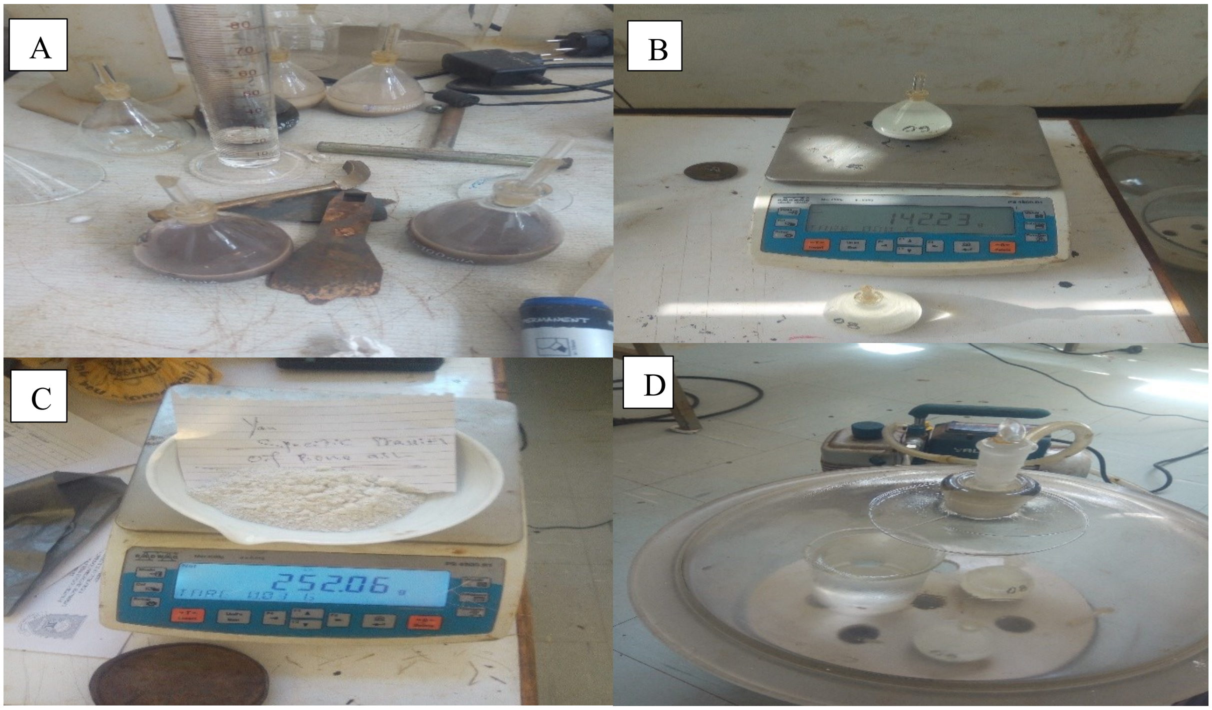


Figure 3.7: Specific gravity Test (picture taken on 15/09/2020)

3.14.5 Atterberg Limits (ASTM D424 or AASHTO T90)

Representative samples of each soil were subjected to Atterberg limits testing to determine the consistency of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 425 µm (No. 40) sieve. The liquid limit of each soil was determined by using the Casagrande apparatus. The plastic limit of each soil was determined by using soil passing through a 425-µm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index computed for each soil based on the liquid and plastic limit obtained is shown in Figure 3-8.

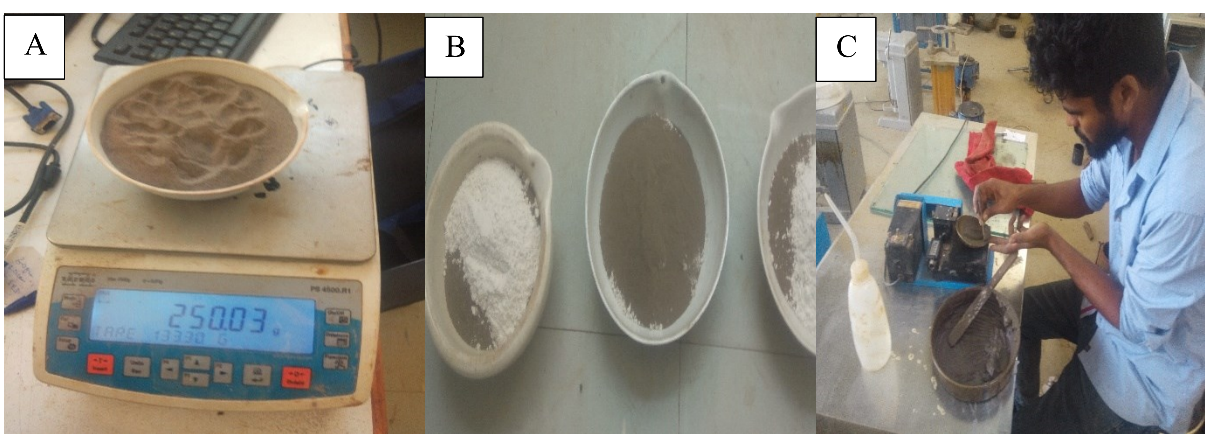


Figure 3.8: Atterberg limit determination (picture taken on 20/09/2020)

3.14.6 Linear Shrinkage

The linear shrinkage value is the way of quantifying the amount of shrinkage likely to be experienced by clayey material. Linear shrinkage test followed a British standard (BS1377: Part 2:1990) and covers the determination of total linear shrinkage from linear measurement on a standard bar of length 140 mm with a semicircular section of diameter 25 mm, the grove filled by a soil of the fraction passing 0.425 mm test sieve, originally having the moisture content of the liquid limit shown in Figure 3-9. The linear shrinkage is determined using Eq. (3.3).

$$LS = \frac{\text{Intial length} - \text{Oven dried length of a specimen}}{\text{Initial lenght}} * 100 \qquad 3.3$$

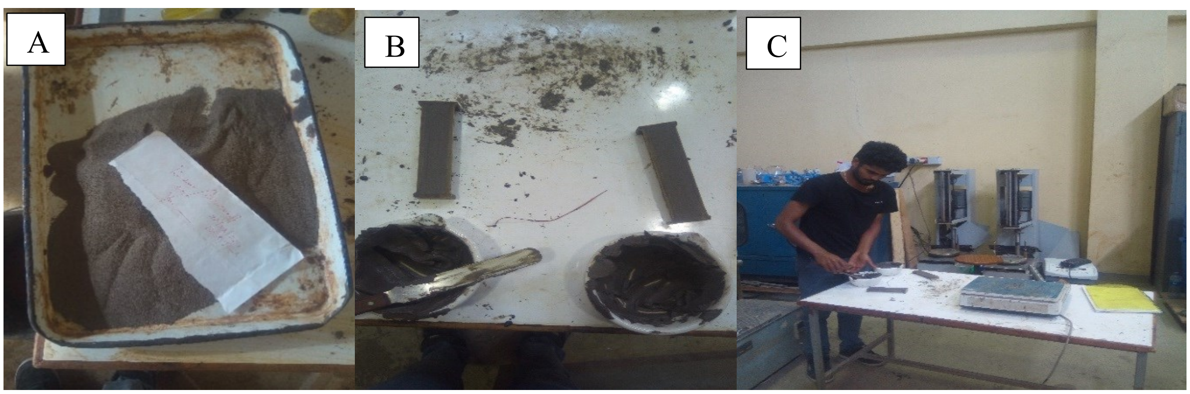


Figure 3.9: Linear Shrinkage Test (picture taken on 06/09/2018)

3.14.7 Soil Classification (AASHTO M-145)

The most widely used soil classification systems are AASHTO and USCS systems. Soil classified using the AASHTO Soil Classification System using particle size distribution and Atterberg limits. Soil classification is the arrangement of soils into different groups so that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO M-145 System. The AASHTO Classification system is useful for classifying soils for high way. According to the laboratory test, the result from the soil understudy will be classified as table 2.3 shown before.

3.14.8 Procter compaction test (AASHTO T-180)

This test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the material. It was done on the natural soil and various percentages of a mixture of lime and animal bone ash added to the black cotton soil and MDD and OMC were determined as shown in Figure 3-10.

Figure 3.10: Compaction test and procedures (picture taken on 13/09/2020)

3.14.9 California Bearing Ratio and CBR Swell (AASHTO T-193 and AASHTO T-180)

CBR test determined the strength of a given material and how it behaves under loading. This was determined by measuring the relationship between force and penetration when a cylindrical plunger of cross-sectional area 1935mm^2 is made to penetrate the soil at the given rate. At any penetration value, the ratio of the force to a standard force is defined as the California Bearing Ratio. The CBR test has consisted of the following procedures as key points to arrive at the result of the strength value deserved.

A. Compacting a sample at its optimum moisture content.
B. Applying a surcharge to the sample to represent the estimated thickness of pavement over the sub-base and subgrade materials.
C. Soaking the sample for four days.
D. Forcing 19.4 cm^2 plungers into the sample.

Road pavement structural design has usually based on 4-days soaked CBR values, to simulate the anticipated “worst-case” soil condition on the field. It was measured by placing the tripod with the dial indicator on the top of the 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 has taken just after soaking the sample. At the end of 96 hours, the final dial reading of the dial indicator was taken hence the swell percentage of the initial sample length shown in Figure 3-11.

$$\text{CBR } (\%) = \frac{\text{test load on the sample}}{\text{standard load on the crushed stone}} * 100 \tag{3.4}$$

$$\text{CBR swell} = \frac{\text{change in length in mm during soaking}}{116.30\text{mm}} * 100 \tag{3.5}$$

Figure 3.11: CBR test procedures (picture taken on 16/09/2020)

CHAPTER FOUR

RESULTS AND DISCUSSION

4.2 Properties of Materials

4.2.1 Physical properties of Animal Bone Ash

The major chemical compounds in the bone ash with their level of abundance rated in percentages shown in Table 4.1. The results show that the bone ash contained calcium oxide (CaO) and phosphate (P_2O_5) as the major oxides. The bone ash contains some of the elements (oxides) found in pozzolana. However, the total percentage of iron oxide, silicon oxide and aluminum oxide are less than the minimum of 70% specified by pozzolanas [52]. [52] defined pozzolana as siliceous or siliceous and aluminous materials which in themselves have little or no cementitious properties but in finely divided form and the presence of moisture, they react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. The CaO presents in the bone ash is capable of making ash a pozzolana in the presence of water. The P_2O_5 has the potential to act as a binding agent to cement particles of soil together and increase stability [54][55].

The physical properties of bone ash were summarized in Table 4.2 shows that plastic limit cannot be determined so the plasticity index is NP (non-plastic) as provided by [23].however should not exceed a PI of 6 to be used as subgrade material [21].

Table 4.1: Studies of the chemical composition of bone ash [54], [55]

Oxide Calcium	% Composition
Calcium Oxide (CaO)	48.21
Phosphate (P_2O_5)	37.77
Magnesium oxide (MgO)	1.29
Silicon Oxide (SiO_2)	0.12
Iron Oxide (Fe_2O_3)	0.09
Aluminum Oxide (Al_2O_3)	0.08
Moisture	0.15
Loss on ignition	0.21

Table 4.2: Physical properties test results of bone ash

Properties	**Symbol**	**Test result**
Liquid Limit, (%)	LL	ND*
Plastic Limit, (%)	PL	ND
Plasticity index, (%)	PI	NP
Specific Gravity	Gs	2.99

*ND = Not Determined

4.2.2 Properties of Natural Soils

The engineering properties of the natural soil before mixing with lime, animal bone ash, and mixture were presented in Table 4.3.

Table 4.3: Characteristics of soil samples

Parameters	Soil from Chida	Soil from ACB	Soil from Ameya
% passing No.200	97.03	92.07	89.42
PI (%)	59	55	53
LS (%)	22.87	20.85	18.84
AASHTO soil class	A-7-5(70)	A-7-5(61)	A-7-5(57)
USCS	CH	CH	CH
Gs	2.72	2.70	2.67
FSI (%)	104	99	95
MDD (g/cm^3)	1.3924	1.402	1.4121
OMC (%)	28.52	27.75	26.95
Soaked CBR (%)	1.27	1.90	2.53
CBR-swell (%)	4.67	3.73	3.08
Color	Black	Gray	Dark Gray

4.2.2.1 Grain Size Distribution

According to [5] the grain size alone does not determine clay minerals and the author emphasized that the most important property of fine-grained soils is their mineralogical composition.

The grain size distributions of the soil samples are given in Figures 4.1 and 4.2 and Figure 4.3 the detailed grain size analysis test results are attached in Appendix C.

The soil sample from Chida was black, and 97.03% of the soil was passing through the No.200 sieve (75 μm), this indicates that almost all the given soil sample was clay soil (Figure 4.1).

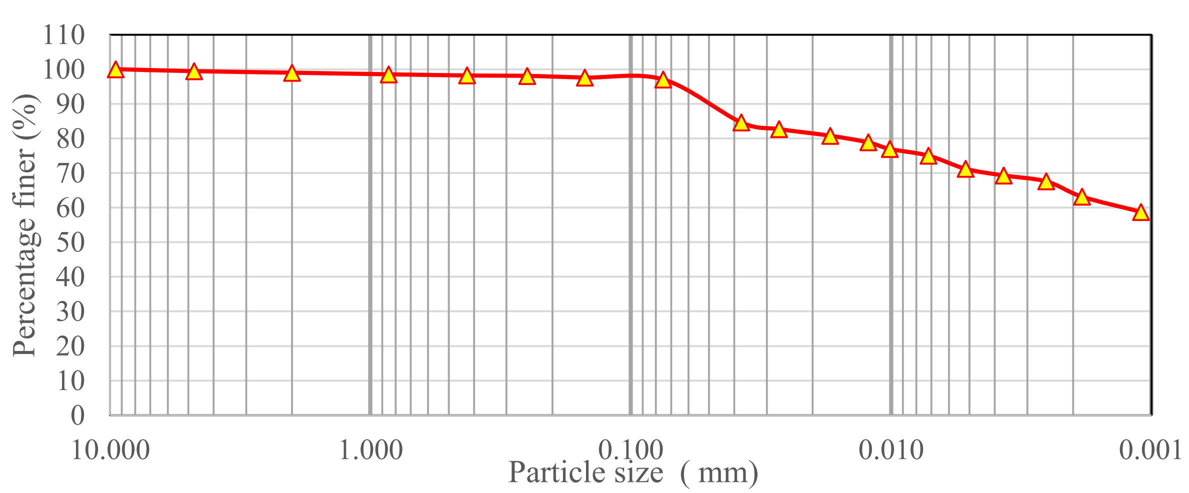


Figure 4.1: Grain size distribution curve of Chida natural soil samples

The soil sample from ACB was gray, and 92.07% of the soil is passing through No. 200 sieves (75 µm), this indicates that almost all the given soil sample was clay soil were presented in Figure 4.2.

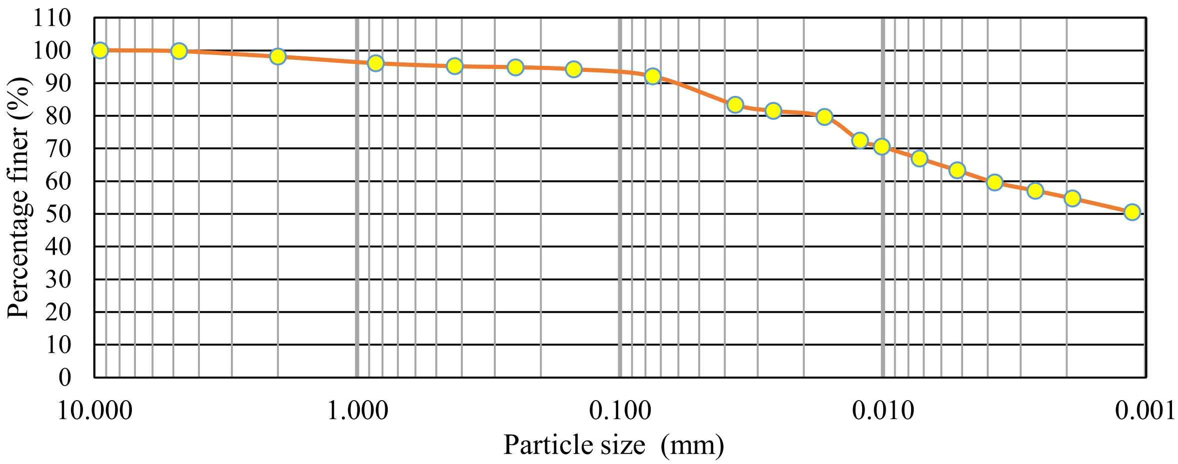


Figure 4.2: Grain size distribution curve of ACB natural soil samples

The soil sample from Ameya was dark gray, and 89.42% of the soil is passing through No.200 sieve (75 µm), this indicates that almost all the given soil sample was clay soil were presented in Figure 4.3.

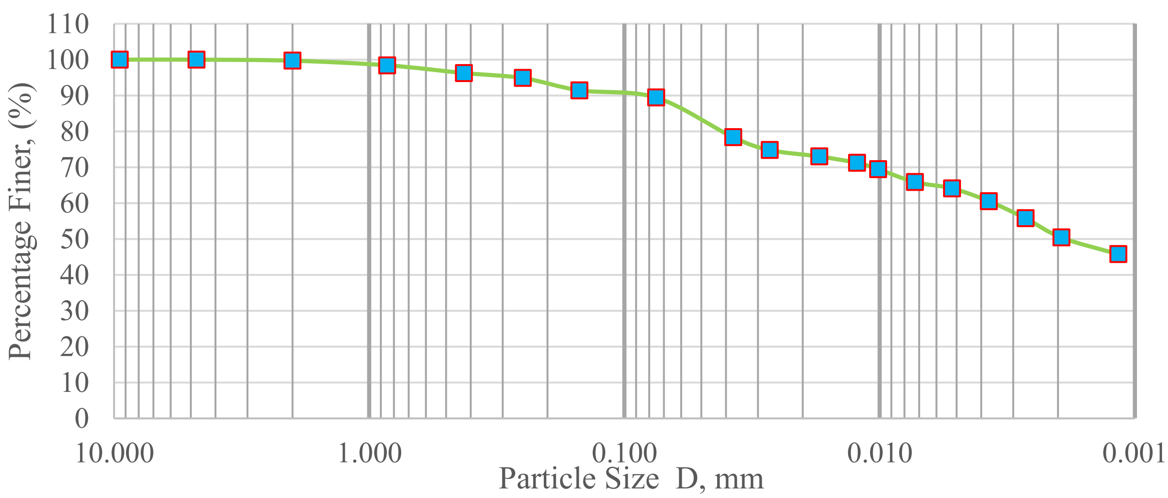


Figure 4.3: grain size distribution curve of Ameya natural soil samples

According to [56], about 40 to 60% of the black cotton soils have grain sizes less than 0.001 mm it is similar to the result of Chida, ACB and Ameya.

4.2.2.2 Atterberg's Limits

The liquid limit, plastic limit, and plastic index of the untreated sample as recorded in Table 4.4. The detailed laboratory analyses were given in Appendix D.

Table 4.4: Atterberg's Limits of natural soil samples

Sample location	Atterberg limits (%)		
	LL	PL	PI
Chida	96	37	59
ACB	93	38	55
Ameya	92	39	53

According to [57], a Liquid limit of less than 35% indicates low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity. Hence, these values indicate that soil samples are highly plastic clay. A high numerical value of the plasticity index is an indication of the presence of a high percentage of clay in the soil sample. The test results showed that the natural soil was not suitable for use as subgrade soil.

4.2.2.3 Soil Classification

According to the unified soil classification system, as shown in table 4.5, and figure 4.4, Ameya, ACB, and Chida soil samples lie above the A-line in the CH region, which means clayey soil with high plasticity.

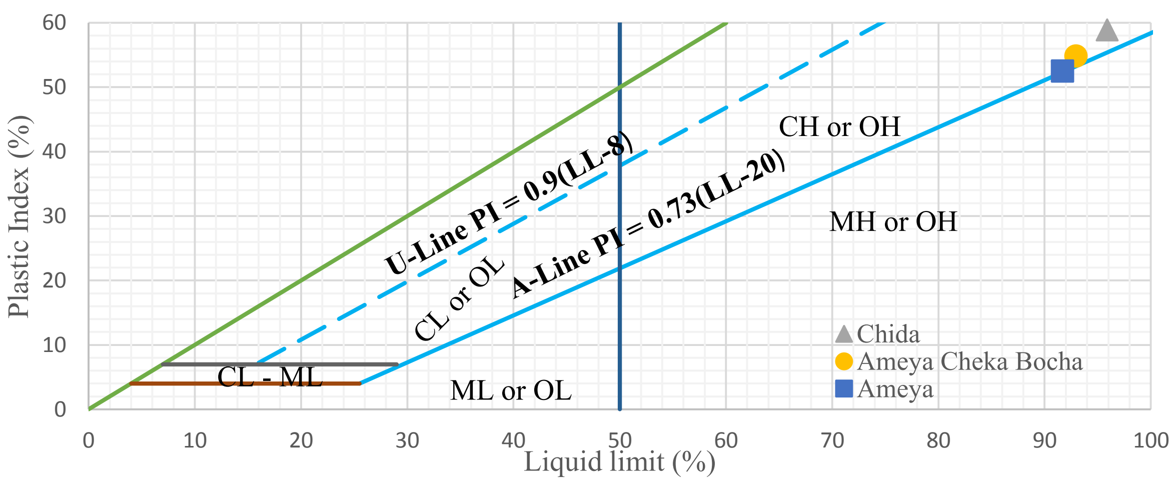


Figure 4.4: Plasticity chart of natural soil samples according to USCS

Table 4.5: Classification of soils based on USCS classification system

Sample location	Min. Sampling depth (m)	Quantity of grain size (%)				LL (%)	PI (%)	USCS Classification
		Gravel	Sand	Silt	Clay			
Chida	1.5	0.57	2.39	33.89	63.15	96	59	CH
ACB	1.5	0.26	7.67	37.38	54.69	93	55	CH
Ameya	1.5	0.00	10.58	39.01	50.41	92	53	CH

Table 4.6: Classification of soils based on AASHTO classification system

Sample location	Percentage of passing			LL (%)	PI (%)	LL-30	Group index	Soil group	Material type
	No.10	No.40	No.200						
Chida	98.98	98.18	97.03	96	59	66	70	A-7-5	Clay
ACB	98.06	95.14	92.07	93	55	63	61	A-7-5	Clay
Ameya	99.72	96.27	89.42	92	53	62	57	A-7-5	Clay

The group index is a function of the LL, the PI, and the amount of material passing the 0.075 mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed an inverse ratio to its group index, i.e. a group index of 0 indicates a "good" subgrade material and a group index of 20 or more indicates a poor subgrade material [21].

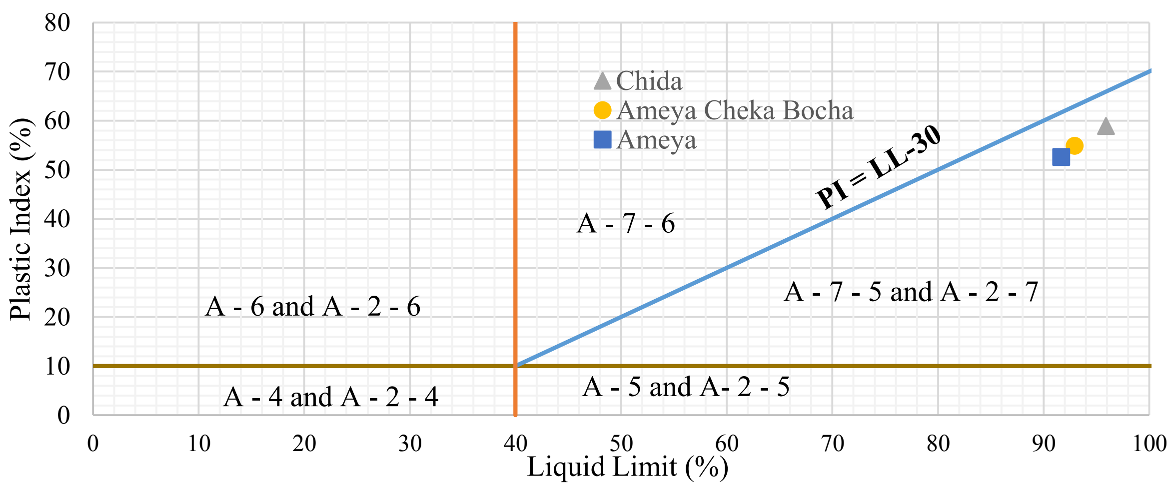


Figure 4.5: Plasticity chart of untreated soil samples according to AASHTO

According to the AASHTO soil classification system, the soil falls under the A-7-6 soil and CH (fat clay) according to ASTM soil class. Soils under this category are classified as a material of reduced engineering property to be used as a sub-grade material.

4.2.2.4 Specific Gravity

The Chida, ACB and Ameya soil sample has a specific gravity of 2.72, 2.70 and 2.67 respectively. The laboratory test results of both soil samples were summarized in Table 4.7 and the laboratory data analysis was attached in Appendix B.

Table 4.7: Gs of untreated soil samples

Sample Location	Chida			ACB			Ameya		
Gs at 20oc	2.73	2.71	2.72	2.71	2.69	2.69	2.67	2.68	2.66
Average Gs at 20oc,	2.72			2.70			2.67		

4.2.2.5 Linear Shrinkage

The Chida, ACB and Ameya soil sample has an LS of 22.87, 20.85 and 18.84% respectively. The linear shrinkage, LS, of the untreated soil samples were presented in Table 4.8 and Appendix E.

Table 4.8: Linear Shrinkage test results of the study area

Sample Location	Chida	ACB	Ameya
LS (%)	22.87	20.85	18.84

4.2.2.6 Free swell index

The free swell, FSI, values of the soils were given in Table 4.9. The free swell index value of both soil samples exceeds 50%, and such soils undergo volumetric changes leading to pavement distortion, cracking, and general unevenness due to seasonal wetting and drying.

Table 4.9: Free swell test results of the study area

Sample Location	Chida	ACB	Ameya
FSI (%)	103.50	99.00	95.00

4.2.2.7 Compaction Test

To determine the maximum dry density and optimum moisture content of the untreated soil samples, a modified Proctor compaction test has been conducted according to AASHTO T-180. The Chida soil sample has a maximum dry density and optimum moisture content of 1.39 g/cm^3 and 28.52% respectively. Similarly, The ACB soil sample has a maximum dry density and optimum moisture content of 1.40 g/cm^3 and 27.75%, and the Ameya soil sample has a maximum dry density and optimum moisture content of 1.41 g/cm^3 and 26.95 %. Detailed laboratory data was attached as Appendix F.

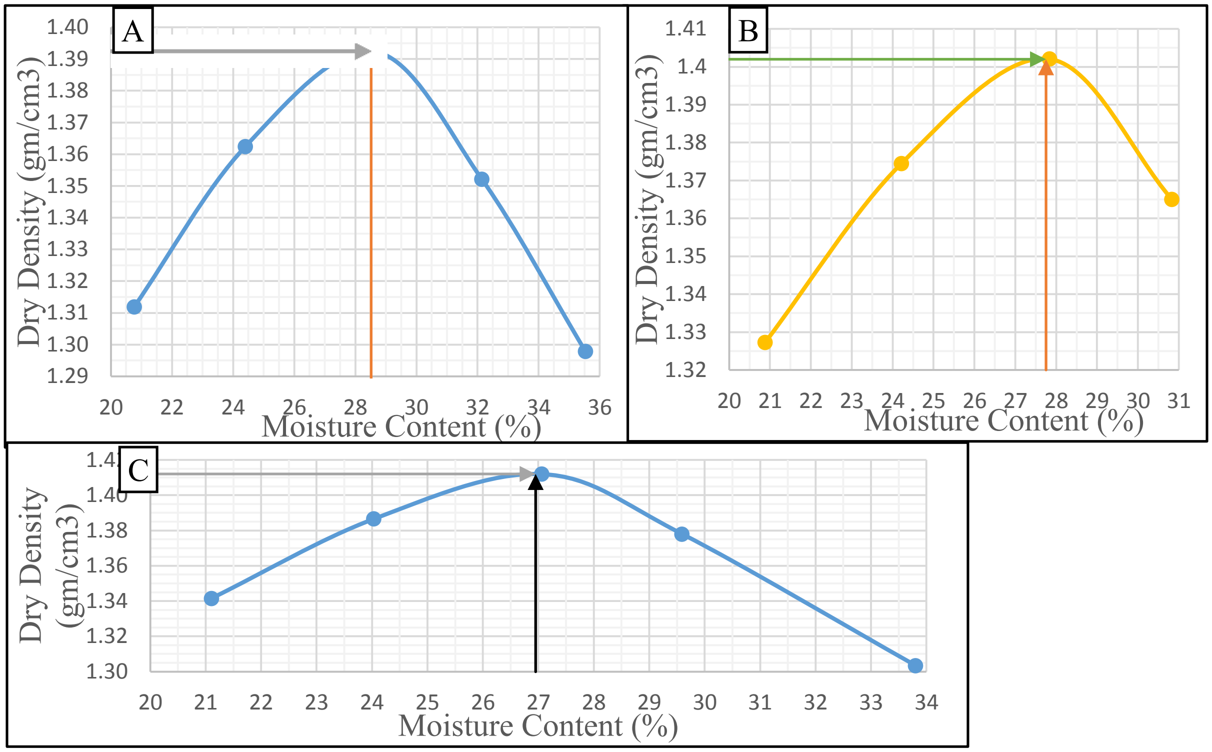


Figure 4.6: Dry density-moisture content relationship for untreated soil samples of Chida, ACB and Ameya respectively

4.2.2.8 California Bearing Ratio and CBR Swell

The CBR value of Chida, ACB and Ameya natural soil samples were shown in Figure 4.7. Chida soil sample had 1.27% CBR value at maximum dry density with 4.67% CBR swell, ACB soil sample had 1.90% CBR value with 3.73% CBR swells and Ameya soil sample had 2.53% CBR value with 3.08 CBR swell. The results indicate that soil samples have low CBR values that do not satisfy the minimum requirements as sub-grade material. According to [21] standard specification, subgrade soils with a CBR value of less than 5% require special treatment.

Hence, the soil was found to be highly plastic expansive clay with low bearing capacity and high swelling potential which fell below the standard recommendations for most highway Construction. According to [32], pavement design, a material with a CBR value of less than three is challenging to work and subgrade would lead to uneconomical pavement structures, it is recommended to cover with selected material or treating it with other stabilizing material. Therefore, the soil requires first modification and stabilization to improve its workability and engineering property.

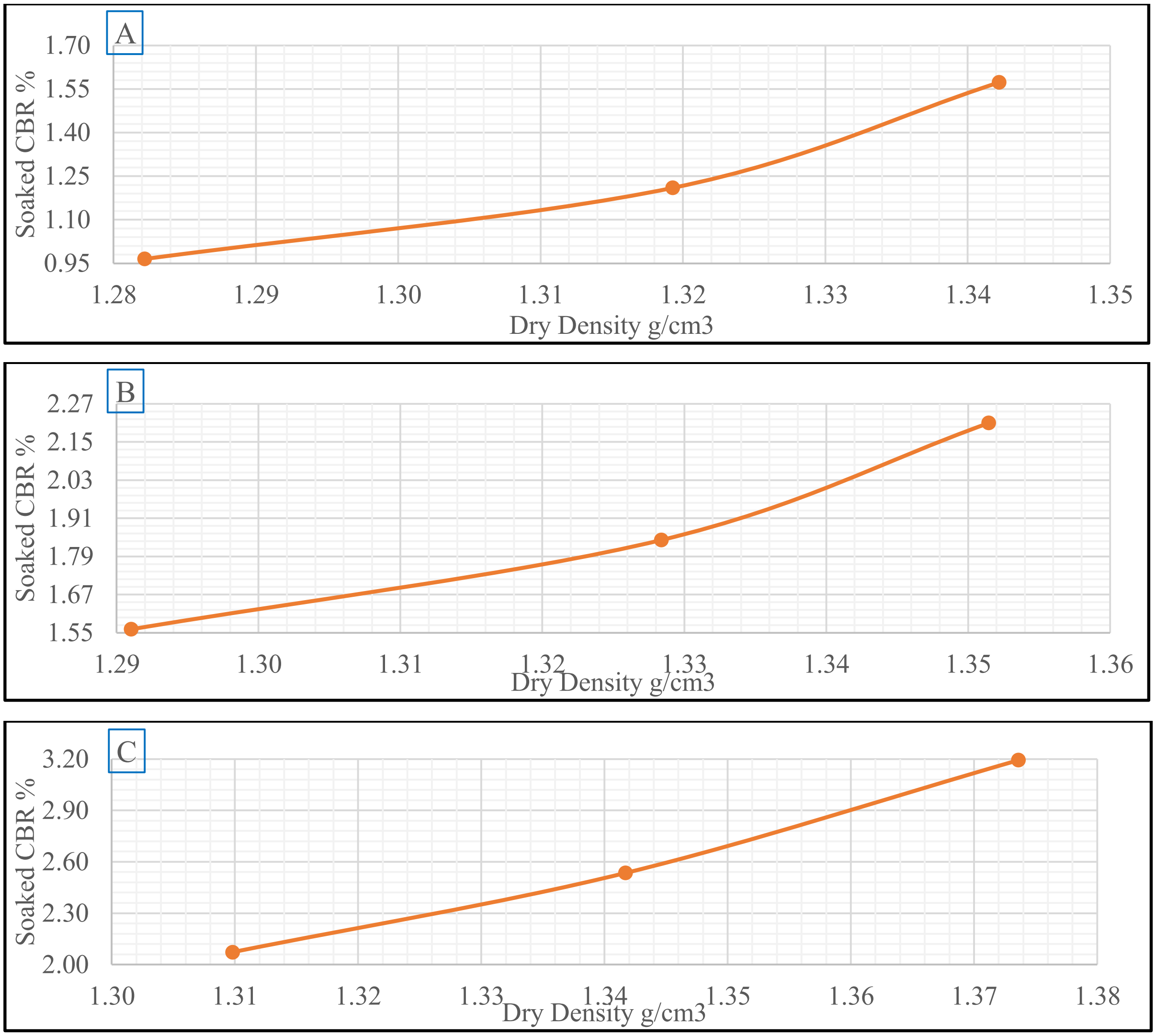


Figure 4.7: CBR test result of the (A) Chida, (B) ACB, and (C) Ameya soil samples

4.3 Stabilized Black Cotton Soil

4.3.1 The effect of Lime and Animal Bone Ash on Atterberg's Limits

The effect of lime and animal bone ash addition in varying proportions with natural black cotton soil had been studied and the variation in consistency limit for various additive mix-ratio was presented in Table 4.9. It was found that as the percentage of additive content lime increases the liquid limit decreases on the other hand the plastic limit increases then decreases. The initial increase in the plastic limit was due to the plastic nature of bone ash and the subsequent decrease beyond 6% BA content was due to cation exchange reaction. As a result, the PI also decreased followed by an increase in additives content. The summary of the laboratory test result was analyzed and given in Table 4.10.

Table 4.10: Effect of lime and animal bone ash content addition on Atterberg's limit

No	Sample type	Chida soil			ACB soil			Ameya soil		
	Atterberg's Limits (%)	LL	PL	PI	LL	PL	PI	LL	PL	PI
1	Natural soil	96	37	59	93	38	55	92	39	53
2	2% L + 98% S	93	52	41	91	55	36	88	62	26
3	4% L + 96% S	81	58	23	81	60	21	80	62	19
4	6% L + 94% S	73	61	12	73	62	11	72	62	10
5	8% L + 92% S	70	60	10	69	60	8	67	61	6
6	2% BA + 98% S	92	38	53	90	39	51	88	41	47
7	4% BA + 96% S	89	44	45	88	46	42	87	47	39
8	6% BA + 94% S	84	52	32	83	51	32	81	50	31
9	8% BA + 92% S	86	50	36	84	50	34	83	52	32
10	6% BA +2% L +92% S	79	59	20	78	60	18	77	61	16
11	6% BA +3% L+ 91% S	75	61	14	74	61	13	74	62	12
12	6% BA +4% L+ 90% S	72	59	13	70	60	10	69	61	8

As shown in Table 4.9, the LL decreases from control value 96 to 72%, 93 to 70%, and 92 to 69% for Chida, ACB, and Ameya soil samples, respectively. The Atterberg limit depends on the type of predominant clay mineral available in the soil mass. If the predominant clay is montmorillonite the LL can reach or even exceed 100%. It is also expected that the Atterberg limit is less for illite-dominated soil and even lesser for kaolinite-dominated soils. However, the additives have not shown significant change on the LL of the soil because the dispersing effect of the additive did not affect the liquidity natures of the soil but its PL only.

Figure 4.8, indicated that the highest reduction in plasticity index was observed when adding lime and animal bone ash at a 6% ratio but animal bone ash increase after that on the other hand mixture of both decreases continuously.

Generally, Addition lime, animal bone ash and mixture have shown an 83%, 45% and 78% reduction in plasticity index and modest change in liquid limit of both soil samples. The CaO present in the bone ash is capable of reacting with the fine particles of soils to aid stabilization. The P2O5has the potential to act as a binding agent to cement particles of soil together and increase its stability.

As shown in Table 4.9, the PI decreases from control value 59 to 13%, 55 to 10%, and 53 to 8% for Chida, ACB, and Ameya soil samples, respectively this result is similar to the result obtained by [16].

According to [5] degree of expansion is not critical when the value of PI and LL lower than 35% and 60% respectively and can be as subgrade material.

4.3.2 The effect of the addition of Lime and Animal bone Ash on Linear Shrinkage

The LS was presented in Table 4.10. Increment of additive content percentage, especially when the ratio of lime was higher than animal bone ash, the LS value was reduced. So the additive contents were effective to reduce the volume change when exposed to variable humidity and weather condition. According to [35] soils having LS values above 8%, between 5 and 8%, and less than 5% possess the critical, marginal, and non-critical degree of expansion, respectively.

The average LS for Chida, ACB and Ameya natural soils was under the critical degree of expansion with 22.87%, 20.85%, and 18.84% respectively. For Chida, ACB, and Ameya soil sample, 6% and 8% lime alone and 6% BA + 3% L and 6% BA + 4% L has significantly improved the natural soil sample into marginal and non-critical stage degree of expansion (Figure 4.9 and Table 4.11). The LS decreased with an increase in lime and animal bone ash. The cation exchange reaction results in flocculation of the clay particle thereby making the soil lose its plasticity and makes it behave more like silt. These reactions are responsible for the reduction in swelling and shrinkage characteristic of the soil as such improves its workability

Table 4.11: Effect of addition of lime and animal bone ash on LS

No	Sample type	Soil sample of Chida		Soil sample ACB		Soil sample Ameya	
	Mix- proportion (%)	LS (%)	Degree of Expansion	LS (%)	Degree of Expansion	LS (%)	Degree of Expansion
1	Natural soil	22.87	Critical	20.85	Critical	18.84	Critical
2	2% L + 98% S	16.48	Critical	15.80	Critical	15.12	Critical
3	4% L + 96% S	11.81	Critical	11.18	Critical	10.55	Critical
4	6% L + 94% S	7.00	Marginal	6.21	Marginal	5.42	Marginal
5	8% L + 92% S	4.34	Non-critical	3.94	Non-critical	3.54	Non-critical
6	2% BA + 98% S	22.66	Critical	20.59	Critical	18.51	Critical
7	4% BA+ 96% S	18.34	Critical	17.32	Critical	16.30	Critical
8	6% BA + 94% S	14.74	Critical	14.20	Critical	13.65	Critical
9	8% BA + 92% S	15.85	Critical	13.94	Critical	12.03	Critical
10	6% BA + 2% L + 92% S	11.28	Critical	10.65	Critical	10.02	Critical
11	6% BA + 3% L + 91% S	6.81	Marginal	6.02	Marginal	5.23	Marginal
12	6% BA + 4% L + 90% S	5.30	Marginal	4.90	Non-critical	4.51	Non-critical

4.3.3 The effect of Lime and Animal Bone Ash on Free Swell Index

The effect of lime and animal bone ash on the free swell index of the treated soil sample was tabulated in table 4.11 and plotted in figure 4.9. For the Chida soil sample, the highest to low reduction was attained when the sample was treated with 8% L, and 6% BA + 4% L that means 29% and 31.8 % respectively reduction was observed from its natural state which was 103.5%. For the ACB soil sample the maximum reduction of 26.2% was observed when 8% L and 29.4% reduction was observed after the addition of 6% BA + 4% L. but, Ameya soil sample the maximum reduction of 23.6% was observed when 8% L and 27.2% reduction was observed after addition of 6% BA + 4% L. This indicates that 8% L and 6% BA + 4% L was the optimum ratio of additive content to achieve a remarkable free swell index value. According to Is 1498:1970, (2016), soils having a free swell value above 100 can cause damage whereas free swell as low as 100% can cause considerable damage to lightly loaded structures, and soils having an FSI value below 50% seldom exhibits appreciable volume change even under light loads.

The behavior observed may be due to the partial replacement of the plastic soil particles with bone ash fines, an abrasive non-plastic material with a reduction in clay content.[54].According to the U.S.B.R classification Method [31] the soil sample which initially lies under, highly expansive was shifted to low expansive soil with a maximum percentage of stabilizer would be observable. Figure 4.10 and Table 4.12 shows that Chida, ACB, and Chida soil samples had reduced their swelling properties.

Table 4.12: Effect of addition of lime and animal bone ash on free swell index

No	Sample type	Chida		ACB		Ameya	
		FSI %)	IS 1498	FSI(%)	IS 1498	FSI %)	IS 1498
1	Natural soil	103.5	High	99.0	Medium	95.0	Medium
2	kerosene	0.0	Control	0.0	Control	0.0	Control
3	2% L + 98% S	73.6	Medium	69.7	Medium	66.3	Medium
4	4% L + 96% S	51.9	Medium	48.5	Low	45.5	Low
5	6% L + 94% S	40.8	Low	37.7	Low	35.0	Low
6	8% L + 92% S	29.0	Low	26.2	Low	23.6	Low
7	2% BA + 98% S	91.1	Medium	87.1	Medium	83.6	Medium
8	4% BA+ 96% S	72.1	Medium	69.0	Medium	66.2	Medium
9	6% BA + 94% S	58.6	Medium	56.1	Medium	53.8	Medium
10	8% BA + 92% S	44.2	Low	42.3	Low	40.6	Low
9	6% BA + 2% L + 92% S	57.7	Medium	54.7	Medium	52.0	Medium
10	6% BA + 3% L + 91% S	40.9	Low	38.3	Low	35.9	Low
11	6% BA + 4% L + 90% S	31.8	Low	29.4	Low	27.2	Low

4.3.4 The effect of Lime and Animal Bone Ash on Moisture density relationships

The values of the modified Procter compaction test for soil samples were presented and plotted in figure 4.11, figure 4.12, and figure 4.13 and a summary of the result presented in Table 4.13.

Table 4.13: Effect of lime and ABA content addition on Moisture Density Relation

No	Sample type	Soil of Chida		Soil of ACB		Soil of Ameya	
		OMC (%)	MDD (g/cm^{3})	OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)
1	Natural soil	28.52	1.392	27.75	1.402	26.95	1.412
2	2% L + 98% S	29.30	1.352	29.41	1.370	30.00	1.388
3	4% L + 96% S	32.40	1.329	32.00	1.342	31.80	1.357
4	6% L + 94% S	35.90	1.319	34.60	1.329	33.25	1.343
5	8% L + 92% S	37.80	1.310	37.00	1.315	36.25	1.321
6	2% BA + 98% S	27.00	1.443	26.30	1.452	25.60	1.462
7	4% BA+ 96% S	25.45	1.468	24.85	1.477	24.20	1.487
8	6% BA + 94% S	23.00	1.484	22.40	1.494	21.85	1.504
9	8% BA + 92% S	22.65	1.475	22.10	1.486	21.50	1.496
10	6% BA + 2% L + 92% S	27.67	1.406	27.10	1.418	26.50	1.430
11	6% BA + 3% L + 91% S	29.40	1.401	28.20	1.410	27.10	1.420
12	6% BA + 4% L + 90% S	30.00	1.390	29.00	1.395	28.40	1.402

The MDD shows a slight reduction and OMC shows an increment in the treatment of Chida soil sample with an increase of lime (Table 4.13) According to Didier (2000), Lime addition decreases the maximum dry density of expansive soils.

The MDD decreases from 1.39 to 1.31 g/cm^3 and OMC increases from 28.52 to 37.8%. Similarly, for lime with animal bone ash MDD decrease and OMC increase but for animal bone ash alone it is the reverse which is MDD increase to 1.48 g/cm^3 and OMC decrease to 22.65%.

For the ACB soil sample, as shown in Table 4.12, the MDD shows a slight reduction and OMC shows an increment in the treatment of the Chida soil sample with an increase of lime. The MDD decreases from 1.4 g/cm^3 to 1.32 g/cm^3 and OMC increases from 26.95 to 36.25%. Similarly, for lime with animal bone ash MDD decrease and OMC increase but for animal bone ash alone it is the reverse which is MDD increase to 1.5 g/cm^3 and OMC decrease to 22.5%.

For the Ameya soil sample, as shown in Table 4.12, the MDD shows a slight reduction and OMC shows an increment in the treatment of Chida soil sample with an increase of lime this result is in harmony with the findings of [17]. The MDD decreases from 1.41 g/cm^3 to 1.32 g/cm^3 and OMC increases from 27.75% to 37%. Similarly, for lime with animal bone ash MDD decrease and OMC increase but for animal bone ash alone it is the reverse which is MDD increase to 1.49 g/cm^3 and OMC decrease to 22.1%.

The reason for the increase in MDD could be due to lime and BA that occupied the soil voids and in addition, the flocculation and agglomeration of the clay particles as a result of an exchange of ions. The increase in MDD could also be attributed to a high specific gravity of the additives replacing the soil particles with lower specific gravity. The reduction may be attributable to the absorption capacity of the bone ash due to its porous properties.

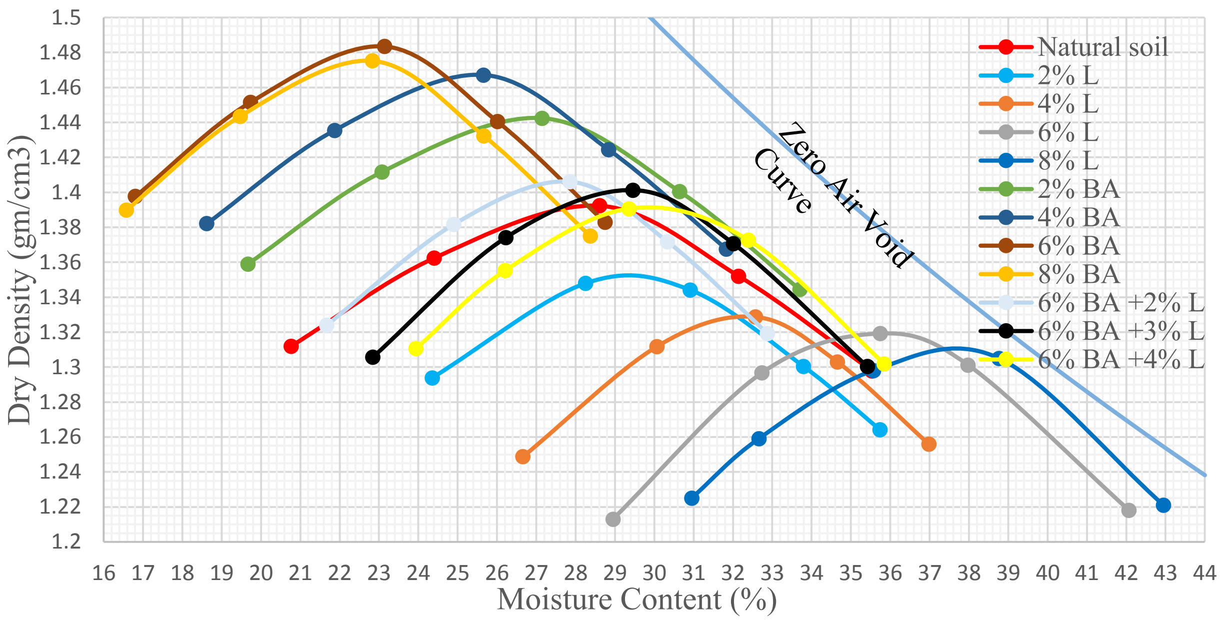


Figure 4.8: Summary of OMC and MDD of stabilized Chida soil sample

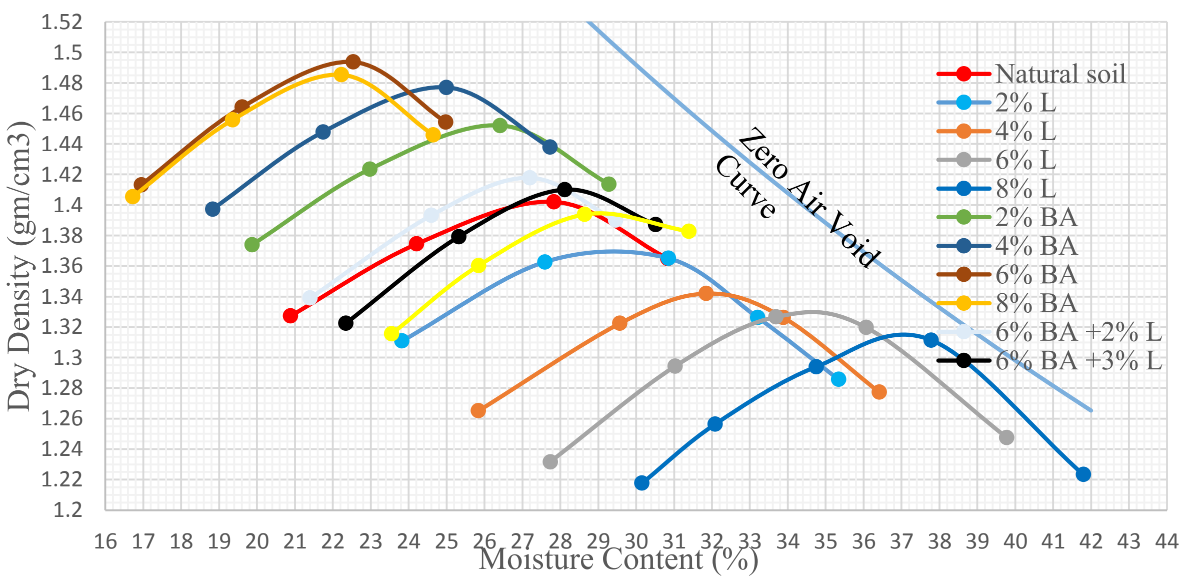


Figure 4.9: Summary of OMC and MDD of Stabilized ACB soil sample

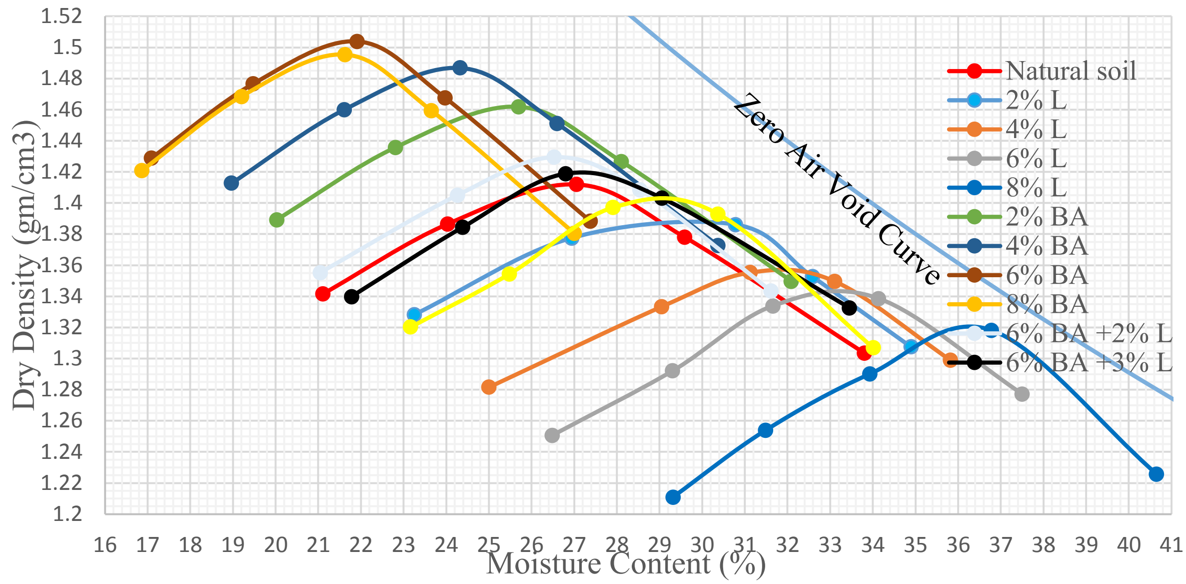


Figure 4.10: Summary of OMC and MDD of Stabilized Ameya soil sample

4.3.5 The effect of the addition of Lime and Animal Bone Ash on CBR value

The addition of bone ash increased the soaked CBR of all the samples. The CBR value of Chida soil increased from to maximum of 2.72% at 6% bone ash stabilization then decrease, while those of ACB and Ameya increase from 1.89 and 2.53% to 4.02 and 4.99% respectively at 6% bone ash stabilization. Therefore, since all the samples fall short of this standard, they are inadequate as sub-grade materials in highway construction.

CBR values of natural sub-grade soils of the three samples did not fulfill the requirement of sub-grade soils as per ERA standard (CBR> 5%).

The addition of lime increased the soaked CBR of all the samples. The CBR value of Chida soil increased from 1.27% to a maximum of 6.16% at 6% lime stabilization, while those of ACB and Ameya increase from 1.89 and 2.53% to 5.52 and 6.6% respectively at 4% lime stabilization. Therefore, all the samples fulfill the requirement of sub-grade soils as per ERA standard (CBR> 5%) adequate as sub-grade materials in highway construction.

The addition of lime and bone ash increased the soaked CBR of all the samples. The CBR value of Chida soil increased from 1.27% to a maximum of 6.31% at 6% BA + 3% L stabilization, while those of ACB and Ameya increase from 1.89 and 2.53% to 6.03 and 9.14% respectively at 6% BA + 2% L stabilization. Therefore, since all the samples fulfill the requirement of sub-grade soils as per ERA standard (CBR> 5%) adequate as subgrade materials in highway construction

The soaked CBR Test Result of both soil samples alongside ERA requirement was presented in figure 4.14, table 4.14, 4.15, and 4.16 detailed test results are given in Appendix-G.

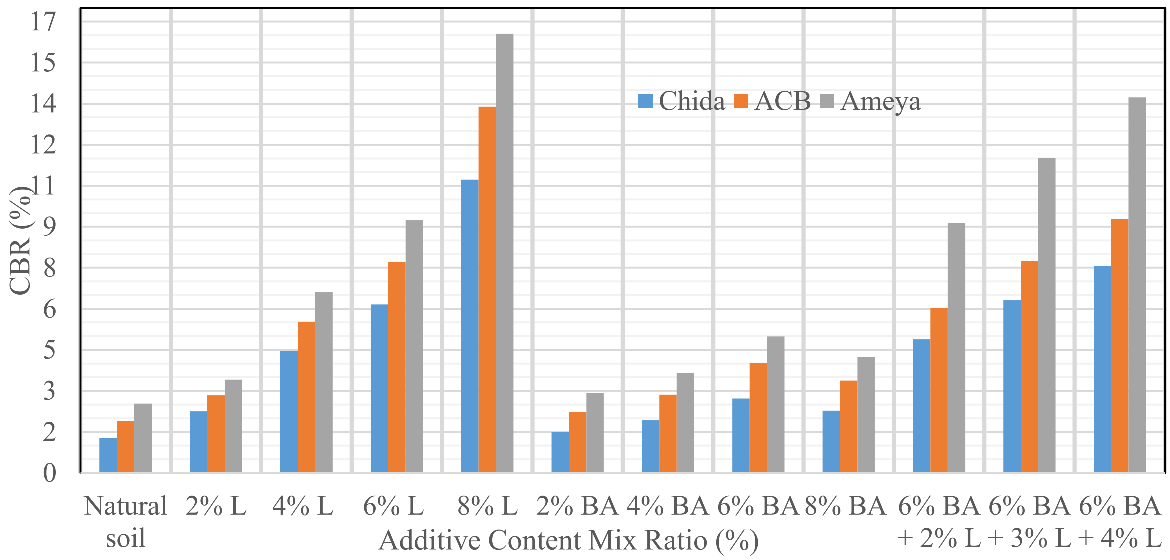


Figure 4.11: Summary of CBR test results of Chida, ACB, and Ameya soil Samples

As shown from figure 4.13, Chida, ACB and Ameya soil samples treated by 8% lime alone and 6% BA + 4% L showed more improvement than Animal bone ash alone. The combination of Lime and animal bone ash can strongly improve the strength of expansive soils.

Table 4.14: Summary of CBR Test results for Chida Stabilized soil sample

Soil Sample of Chida	CBR Value (%)						CBR @ 95% MDD	CBR Swell (%)	Subgrade Class
	65 blows		30 blows		10 blows				
	2.54	5.08	2.54	5.08	2.54	5.08			
Natural soil	1.57	1.48	1.21	1.14	0.97	0.91	1.27	4.67	S1
2% L + 98% S	3.45	3.25	2.65	2.50	2.12	2.00	2.26	2.75	S1
4% L + 96% S	6.71	6.33	5.16	4.87	4.13	3.90	4.45	2.24	S2
6% L + 94% S	8.16	7.70	6.28	5.93	5.33	5.04	6.16	2.07	S3
8% L + 92% S	12.49	11.79	10.41	9.83	8.22	7.76	10.72	1.82	S4
2% BA + 98% S	1.56	1.48	1.42	1.34	1.21	1.14	1.50	3.03	S1
4% BA+ 96% S	2.25	2.13	1.88	1.77	1.52	1.44	1.92	2.46	S1
6% BA + 94% S	3.07	2.90	2.56	2.42	2.30	2.18	2.72	2.28	S1
8% BA + 92% S	2.49	2.35	2.07	1.96	1.66	1.57	2.28	2.34	S1
6% BA + 2% L + 92% S	5.52	5.21	4.60	4.34	4.14	3.91	4.88	2.10	S2
6% BA + 3% L + 91% S	8.36	7.89	6.43	6.07	5.46	5.16	6.31	1.95	S3
6% BA + 4% L + 90% S	7.90	7.46	7.19	6.78	6.11	5.77	7.56	1.91	S3

Table 4.15: Summary of CBR Test results for ACB Stabilized soil sample

Soil Sample of ACB	CBR Value (%)						CBR @ 95% MDD	CBR Swell (%)	Subgrade Class
	65 blows		30 blows		10 blows				
	2.54	5.08	2.54	5.08	2.54	5.08			
Natural soil	2.20	2.08	1.84	1.73	1.56	1.47	1.898	3.73	S1
2% L + 98% S	3.23	3.05	2.76	2.60	2.35	2.21	2.832	2.20	S1
4% L + 96% S	6.88	6.49	5.29	5.00	4.50	4.25	5.528	1.79	S3
6% L + 94% S	9.19	8.67	7.07	6.67	5.65	5.34	7.700	1.65	S3
8% L + 92% S	16.14	15.24	12.42	11.72	9.93	9.38	13.393	1.45	S4
2% BA + 98% S	2.34	1.95	2.13	1.77	1.81	1.51	2.235	2.42	S1
4% BA+ 96% S	3.13	2.61	2.60	2.17	2.21	1.85	2.861	1.97	S1
6% BA + 94% S	4.43	3.69	3.69	3.08	3.32	2.77	4.018	1.82	S2
8% BA + 92% S	3.76	3.13	3.13	2.61	2.82	2.35	3.376	1.87	S2
6% BA + 2% L + 92% S	6.65	5.54	5.54	4.62	4.99	4.16	6.030	1.68	S3
6% BA + 3% L + 91% S	9.64	9.11	7.42	7.00	6.31	5.95	7.750	1.56	S3
6% BA + 4% L + 90% S	11.18	10.56	8.60	8.12	6.88	6.50	9.280	1.53	S4

Table 4.16: Summary of CBR Test results for Ameya Stabilized soil sample

Soil Sample of Ameya	CBR Value (%)						CBR @ 95% MDD	CBR Swell (%)	Subgrade Class
	65 blows		30 blows		10 blows				
	2.54	5.08	2.54	5.08	2.54	5.08			
Natural soil	3.18	3.01	2.53	2.39	2.07	1.96	2.53	3.08	S1
2% L + 98% S	3.73	3.52	3.39	3.20	2.78	2.62	3.41	1.82	S2
4% L + 96% S	7.40	6.99	6.73	6.36	5.52	5.21	6.60	1.48	S3
6% L + 94% S	10.82	10.2	9.01	8.51	7.66	7.23	9.24	1.37	S4
8% L + 92% S	17.49	16.5	14.6	13.77	12.4	11.7	16.06	1.20	S5
2% BA + 98% S	2.97	3.26	2.48	2.72	1.98	2.18	2.92	2.00	S1
4% BA+ 96% S	3.71	4.07	3.09	3.40	2.78	3.06	3.65	1.62	S2
6% BA + 94% S	5.08	5.57	4.23	4.64	3.81	4.18	4.99	1.50	S2
8% BA + 92% S	4.32	4.74	3.60	3.95	3.24	3.56	4.24	1.54	S2
6% BA + 2% L + 92% S	10.25	9.68	9.32	8.80	7.64	7.22	9.14	1.39	S4
6% BA + 3% L + 91% S	11.73	12.9	9.77	10.72	8.80	9.65	11.52	1.28	S4
6% BA + 4% L + 90% S	14.96	14.1	12.5	11.77	10.6	10.0	13.73	1.26	S4

4.3.6 The effect of the addition of Lime and Animal Bone Ash on CBR Swell Test

CBR swell showed significant reduction with the addition of lime. CBR swell of expansive Clay treated with 6 % lime yields 1.2 % and 6% BA + 3% L yields 1.28 % swelling. This finding is also in harmony with results obtained by [17].

The lime and Animal bone ash additive mixtures compacted in CBR molds at Optimum moisture content and maximum dry density gauged for swelling properties before and after soaking for four days to evaluate the percent of the swell. The test result at different mix-ratio for Chida, ACB and Ameya soil samples was plotted in table 4.13, table 4.14, table 4.15 and figure 4.15.

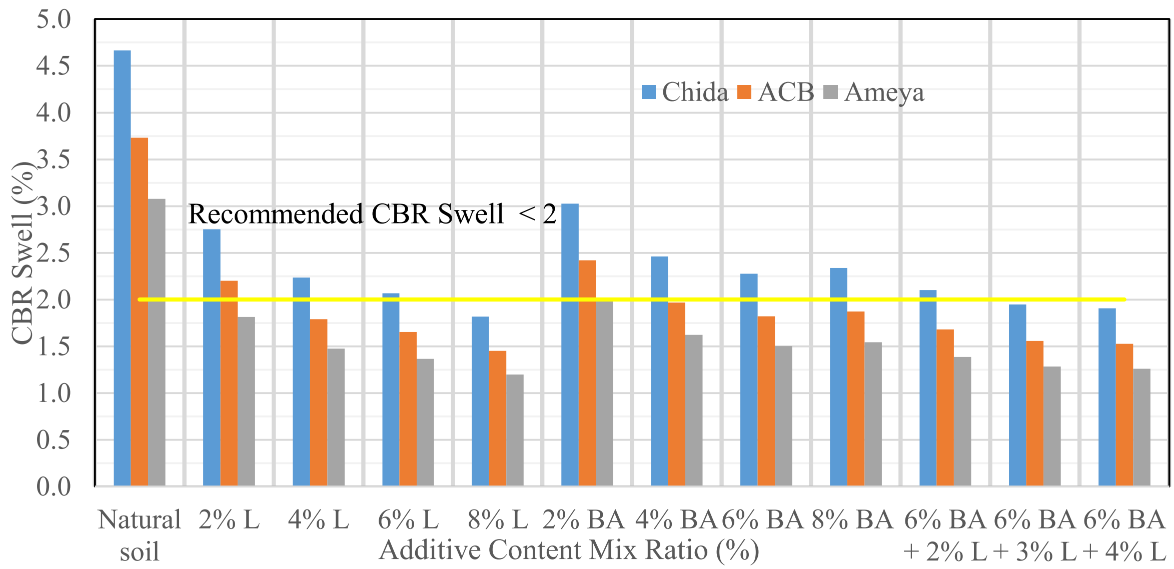


Figure 4.12: Summary of CBR Swell results of Chida, ACB, and Ameya soil Samples

Figure 4.15, shows Chida, ACB and Ameya natural soil samples have the properties of swelling and potentially expansive soil. Though, when lime and animal bone ash were added with different mix-ratio the CBR swell value reduces. The reduction in CBR Swell was due to cation exchange and flocculation and agglomeration of the soil particles and variation in clay mineralogy of the expansive soils. This was happened due to the replacement of some of the volume that was previously occupied by expansive clay minerals (montmorillonite and illite clay minerals) by Lime. Using both the stabilizers improves the stability and strength of the subgrade soils.

The strength of the subgrade is the principal factor in determining the thickness of the pavement, but deterioration due to frost action must also be taken into account.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the results obtained in the experimental investigation, the following conclusions have been drawn.

- The existing soil types were highly expansive and had a high degree of expansion, high plastic index, and poor strength. According to USCS and AASHTO classification system, the Chida soil sample was categorized as CH and A-7-5 (70), the ACB soil sample was categorized as CH and A-7-5 (61) and the Ameya soil sample was categorized as CH and A-7-5 (57) respectively. Thus, the natural soil was very poor in strength to be used as a subgrade material as per [21] specification. The engineering properties of the studied black cotton soil revealed that it was not suitable to use as a sub-grade material unless its undesirable properties are improved.
- The free swell index of black cotton soil used for Chida, ACB, and Ameya soil samples reduced from 103.5% to 31.8% for ACB from 99% to 29.4% and for Ameya 95% to 27.2% respectively at mix- the ratio of 6% BA + 4% lime + 90% soil.
- The liquid limit, Plastic limit, and plastic index were significantly improved to be in the range of subgrade material. Plastic limits of both soil samples significantly increased when the percentage of lime and mixture of lime and bone ash which plasticity index of Chida, ACB, and Ameya soil samples reduced from 59% to 13% for ACB from 55% to 10% and for Ameya 53% to 8% respectively at mix- the ratio of 6% BA + 4% lime + 90% soil. But adding BA in black cotton soil the plastic index decrease up to 6% BA then increases.
- The values for the maximum dry density were noted to decrease with the addition of lime and mixture lime and bone ash content for Chida, ACB, and Ameya soil samples reduced from 1.392 g/cm^3 to 1.39 g/cm^3 for ACB from 1.402 g/cm^3 to 1.395 g/cm^3 and for Ameya 1.412 g/cm^3 to 1.402 g/cm^3 respectively at mix- the ratio of 6% BA + 4% lime +90% soil. And the moisture content was found to increase for Chida, ACB and Ameya soil samples from 28.52 to 30% for ACB from 27.75 to 29% and Ameya 26.95 to 28.4% respectively at mix- the ratio of 6% BA + 4% lime + 90% soil. However, for bone ash alone up to 6% BA maximum dry density increase and moisture content decrease.

- The addition of lime and lime with bone ash additive content improved the CBR values of Chida, ACB, and Ameya soil samples. The improvement is more significant when the sample was cured because curing allows pozzolanic reactions. Hence, the combination of lime and bone ash can strongly improve the strength of the expansive soil. As observed from the test result performed under this study, the maximum value of CBR for Chida, ACB and Ameya soil samples was achieved at 6% BA + 4% L + 90% S (with CBR value of 7.56%, 9.28%, and 13.73%), and 8% lime (with CBR value of 10.72%, 13.39%, and 16.06%) respectively.
- On the other hand addition of BA in black cotton soil, the CBR values are not significantly affected highly plastic expansive soil. The CBR value increases for Chida (1.27 to 2.72%) ACB (1.9 to 4.02%), and Ameya soil samples (2.53 to 4.99%) on the addition of 6% of BA with black cotton soil. The value of CBR decreases further in addition to BA percentage.
- The results obtained during this investigation as discussed in the previous sections show that stabilized black cotton soil with bone ash content alone does not bring significant change for using it as a sub-grade material. Therefore, bone ash is not an effective standalone stabilizer for highly plastic expansive soils. However, bone ash in combination with lime can effectively improve the black cotton soil. Therefore, black cotton soil treated with bone ash combined with lime can be used as a good sub-grade material.
- Generally, the most parameters of [21] specification requirement were and achieved the physical and engineering properties of expansive soil were improved by bone ash combined with lime in different mix-proportion. The optimum amount for adequate stabilization was determined to be 6% BA + 3% L + 91% S. Furthermore, the Addition of 6% BA + 3% L + 91% S yields almost the same result as 6% lime addition. Therefore, it was deduced that lime was partially replaced with 6% bone ash. Input lime was saved due to partial replacement of lime with 6% bone ash.
- Partial replacement of lime by bone ash provides extra environmental as well as technical benefits. Based on the results, it is recommended that about 6% BA + 3% L + 91% S could be used for the treatment of the soil to achieve a sub-grade material, thereby reducing the quantity (cost) of lime needed for stabilization and the environmental menace caused by the waste. It helps in reduction of the greenhouse of emissions.

5.2 Recommendations

Based on the findings of this research work and general practices in the road sector, the following recommendations are forwarded;

This research shows that black cotton soil not suitable for subgrade soil and must be set out to lead on the selecting techniques of stabilizing agents concerning its optimal values.

In this research work, it has been found that the animal bone ash has not shown significant improvement of engineering properties of the expansive subgrade soil to meet specification requirements for road subgrade material. But Lime and animal bone ash mixture stabilization have effective results in improving the engineering properties of expansive soils. It is found to be effective for 6% of the lime and 6% BA + 3% L can be taken as a weak subgrade stabilizer. This research recommends the following areas for further research on Lime, animal bone ash and weak subgrade soil strength.

- As this study was done for a specific area and specific stabilizers, it is recommended as more investigation shall be performed on different parts of the country by mixing with other stabilizers.
- Effects of lime and animal bone ash for weak subgrade soil stabilization is also one perspective to study further for additional choice of stabilizers.
- Effects lime and animal bone ash stabilized subgrades in pavement thickness reduction in flexible and rigid pavement design shall be investigated.

Finally, the results and findings of this research work may be considered as indicative only for further studies as these findings are based on limited parameters and a small number of samples. More elaborate sampling and testing of expansive soils from different origins are recommended before concluding the performance of Lime and animal bone ash as a stabilizing agent for expansive soil.

REFERENCE

[1] A. E. Madedor, “Engineering Properties of Black Cotton Soils of Nigeria and Related Pavement Design,” *NIBRRI Res. Pap.*, no. N1, 1983.

[2] V. N. S. Murthy, *Soil Mechanics and Foundation Engineering*. CBS Publishers & Distributors,New Delhi, 2008.

[3] R. M. Plait, “Determination of swelling pressures of black cotton soil—a method,” in *Proceedings of the third international conference on soil mechanics and foundation engineering*, 1953, vol. 1, pp. 170–172.

[4] S. A. Ola, “The geology and geotechnical properties of the black cotton soils of northeastern nigeria,” *Eng. Geol.*, vol. 12, no. C, pp. 375–391, 1978, doi: 10.1016/0013-7952(78)90019-4.

[5] F. H. (Fu H. Chen, *Foundations on expansive soils / F.H. Chen*. Amsterdam ; New York : New York, NY, U.S.A: Elsevier ; Distributors for the U.S. and Canada, Elsevier Science Pub. Co, 1988.

[6] Molenaar, “Road Materials, Part I: Cohesive and Non-Cohesive Soils and Unbound Granular Materials for Bases and Sub bases in Roads,” 2005.

[7] ERA, “ERA Research and Development Directorate Report Manual.” Addis Ababa, Ethiopia, 2011.

[8] T. N. A. T. A. F. Department Of The Army, “Air Force Afjman 32-1019 Soil Stabilization for Pavements,” no. October. TM 5-822-14/AF Jman 32-1019, Washington DC, pp. 1–57, 1994.

[9] K. J. Osinubi, V. Bafyau, and A. O. Eberemu, “Bagasse ash stabilization of lateritic soil,” in *Appropriate Technologies for Environmental Protection in the Developing World - Selected Papers from ERTEP 2007*, 2009, pp. 271–280, doi: 10.1007/978-1-4020-9139-1_26.

[10] T. Weis, “Towards 120 billion: Dietary change and animal lives,” *Radical Philosophy*, vol. 199. pp. 8–13, 2016.

[11] CSA (Central Statistical Agency), “Report on Livestock and Livestock Characteristics (Private Peasant Holdings): Agricultural Sample Survey 2018/19 [2011 E.C.], Volume II,” 2019.

[12] J. A. Nwanta, J. I. Onunkwo, V. E. Ezenduka, P. O. Phil-Eze, and S. C. Egege, “Abattoir operations and waste management in Nigeria: A review of challenges and prospects,” *Sokoto J. Vet. Sci.*, vol. 7, no. 2, pp. 61–67, 2008.

[13] A. O. Aniebo, S. N. Wekhe, and I. C. Okoli, “Abattoir blood waste generation in rivers state and its environmental implications in the Niger Delta,” *Toxicol. Environ. Chem.*, vol. 91, no. 4, pp. 619–625, 2009, doi: 10.1080/02772240802343404.

[14] J. Fearon, S. B. Mensah, and B. Vivian, “Abattoir operations, waste generation and management in the Tamale metropolis: Case study of the Tamale slaughterhouse,” vol. 6, no. 1, 2014.

[15] Caterpillar, “Introduction to Soil Stabilization,” 2006. QEDQ 1229 %3Cwww.cat.com%3E.

[16] Solomon, “Chemical Stabilization of Expansive Sub-grade Soil Performance Evaluation Selected Road Section in Northeastern Addis Ababa.” MSc. Thesis, Addis Ababa University, Addis Ababa, 2011.

[17] D. Nebro, “Stabilization of Potentially Expansive Subgrade Soil Using Lime and Con-Aid.” MSc. Thesis, Addis Ababa University, Addis Ababa, 2002.

[18] S. Awoke, “Further Investigation of Road Failures Constructed on Expansive Soils of Central Ethiopia Addis Ababa-Jimma Road as Case Study.” MSc Thesis, Addis Ababa University, 2006.

[19] A. A. Al-Rawas and M. F. A. Goosen, “Expansive Soils: Recent advances in characterization and treatment,” *Taylor Fr. e-Library*, vol. 53, no. 9, pp. 1689–1699, 2006.

[20] R. W. Day, *Geotechnical and foundation engineering: design and construction*. McGraw-Hill Professional, 1999.

[21] ERA, “The Federal Democratic Republic Of Ethiopia Ethiopian Roads Authority Pavement Design Manual Vloume I: Flexible Pavements,” *Pavement Des.*, vol. I, no. March, p. 281, 2013.

[22] J. E. Bowles, *Engineering properties of soils and their measurement.*, 4th ed. Boston: McGraw-Hill, Inc, 1992.

[23] AASHTO, *Standard Specifications for Transportation Materials and Methods of Sampling and testing*, 26th ed. Washington, Dc: American Association of State Highway and Transportation Officials, 2006.

[24] A. Teffera and M. Leikun, “Soil Mechanics.” Addis Ababa University, Ethiopia, 1999.

[25] I. Lyon Associates, “Laterite and Lateritic soils and other problem soils of Africa.” Baltimore, 1971.

[26] D. Tilahun, “Influence of Drainage Condition on Shear Strength Parameters of Expansive Soils.” MSc. Thesis, Addis Ababa University, Addis Ababa, 2004.

[27] D. Teklu, “Examining the swelling pressure of Addis Ababa Expansive soil.” MSc. Thesis, Addis Ababa University, Addis Ababa, 2003.

[28] J. Nelson and D. Miller, “Treatment of Expansive Soils Problem and Practice in Foundation and Pavement.” John Willey & Sons, Inc., New York, 1992.

[29] R. . Craig, *Soil mechanics*, 6th ed. London: spon press, 1997.

[30] M. Budhu, *Soil Mechanics and Foundations*. John Wiley & Sons.Inc, 2007.

[31] W. G. Holtz and H. J. Gibbs, "Engineering Properties of expansive clays," *Trans. ASCE*, vol. 121, pp. 641–663, 1956.

[32] Ministry of Works and Urban Development, "Universal Rural Road Access Program-The WayForward to Practical Demonstration of Road Construction on Expansive Soil Sub-grade, Federal Democratic Republic Of Ethiopia." Addis Ababa, 2009.

[33] Is 1498:1970, "Classification and identification of soils for general engineering purposes." Bureau of Indian Standards, pp. 1–24, 2016.

[34] D. Mohan and R. K. Goel, "Swelling pressures and volume expansion in Indian black cotton soils," *J. Inst. Eng. Ser. A*, vol. 40, no. 2, pp. 58–62, 1959.

[35] W. T. Altmeyer, R. F. Dawson, E. S. Barber, and L. A. DuBose, "Discussion of 'Engineering Properties of Expansive Clays,'" *Trans. Am. Soc. Civ. Eng.*, vol. 121, no. 1, pp. 664–675, 1956, doi: 10.1061/taceat.0007284.

[36] V. Dakshanamurthy and V. Raman, "A simple method of identifying an expansive soil," *Soils Found.*, vol. 13, no. 1, pp. 97–104, 1973.

[37] G. R. and A. Rao, *Basic Applide Soil Mechanics*. New Delhi, 2002.

[38] H. B. Seed, R. J. Woodward, and R. Lundgren, "Prediction of swelling potential for compacted clays," *Trans. Am. Soc. Civ. Eng.*, vol. 128, no. 1, pp. 1443–1477, 1963.

[39] N. Pulling, "Determination of optimum concentration of lime solution for soil stabilization." Master s Thesis: University Technology: Malaysia, 2005.

[40] S. K. Dhakal, "Stabilization of very weak subgrade soil with cementitious stabilizers," 2012.

[41] Gautrans, "Manual L 2/04, Stabilization Manual." Department of Public Transport Gauteng Provincial Government, Pretoria, 2004.

[42] D. N. Little and S. Nair, "Recommended practice for stabilization of subgrade soils and base materials," 2009, doi: 10.17226/22999.

[43] A. E. Campbell and D. Jones, "Soil Stabilization in Low-Volume Roads: Obstacles to Product Implementation from Additive Supplier's Standpoint," *Transp. Res. Rec.*, vol. 2204, no. 1, pp. 172–178, 2011.

[44] National Lime Association, "Mixture design and testing procedures for lime stabilized soil," *Tech. Br.*, 2006.

[45] Geiman and M. Christopher, "Stabilization of soft clay subgrades in Virginia phase I laboratory study." Virginia Tech, 2005.

[46] L. Chou, "Lime stabilization: Reactions, properties, design, and construction," *State Art Rep.*, vol. 5, pp. 564–605, 1987.

[47] D. N. Little, *Stabilization of pavement subgrades and base courses with lime*. 1995.

[48] K. C. Onyelowe, “Kaolin Stabilization of Olokoro Lateritic Soil Using Bone Ash as Admixture,” *Int. J. Constr. Res. Civ. Eng.*, vol. 2, no. 1, pp. 1–9, 2016, doi: 10.20431/2454-8693.0201001.

[49] B. B. Teshome, T. A. Mohammed, and T. W. Aure, “Investigating Mechanical Properties of Animal Bone Powder Partially Replaced Cement in Concrete Production,” *Int. J. Sci. Res. Eng. Dev.*, vol. 2, no. 6, pp. 477–490, 2019.

[50] G. M. Ayininuola and A. O. Denloye, “Influence of bone ash on soil California bearing ratio (CBR),” *J. Emerg. Trends Eng. Appl. Sci.*, vol. 5, no. 8, pp. 235–237, 2014.

[51] M. Mucalo, “Processing cow bone for human use, proceeding of the International Conference on Deep Mixing-Best Practice and Recent Advances.” Stockholm, 2010.

[52] D. ASTM, “Annual Book of ASTM Standards,” *Am. Soc. Test. Mater. West Conshohocken, PA, USA*, vol. 6, p. 3, 1996.

[53] A. Simons, D. Solomon, W. Chibssa, G. Blalock, and J. Lehmann, “Filling the phosphorus fertilizer gap in developing countries,” *Nat. Geosci.*, vol. 7, no. 1, p. 3, 2014, doi: 10.1038/ngeo2049.

[54] M. Ayininiuola and D. Akinniyi, “Bone Ash Influence on Soil Consolidation,” *Malaysian J. Civ. Eng.*, vol. 28, no. 3, pp. 407–422, 2016, doi: 10.11113/mjce.v28n3.434.

[55] G. M. Ayininuola and A. O. Sogunro, “Bone Ash Impact on Soil Shear Strength,” *Int. J. Environ. Ecol. Eng.*, vol. 7, no. 11, pp. 772–776, 2013.

[56] S. . Seehra, “Practical Problems of highway Construction in Black Cotton Soil Areas and In-Place Remedial Measures: A Case Study, NBM Construction information,” 2008.

[57] R. Whitlow, *Basic Soil Mechanics*, 3rd ed. New York: Longman, 1995.

APPENDIX

APPENDIX A: NATURAL MOISTURE CONTENT

Sample Location	**Chida**		
Trial Number	1	2	3
Can number	A13	G-5	ZE
A. Mass of can(Mc), gm	36.52	17.38	33.04
B. Mass of can + moist soil (Mcms), gm	99.35	98.35	102.58
C. Mass of can + mass of oven dried soil(Mcds), gm	80.31	74.50	82.04
Mass of water (Mw), gm Mw = B-C	19.04	23.85	20.54
Mass of dry soil(Ms), gm Md = C-A	43.79	57.12	49.00
Water Content(w), % Wc = Mw/Md	43.49	41.76	41.91
Average water content(w), %	**42.39**		

Sample Location	**ACB**		
Trial Number	1	2	3
Can number	E	LHE	A
A. Mass of can(Mc), gm	37.91	25.38	37.01
B. Mass of can + moist soil (Mcms), gm	98.03	103.62	106.78
C. Mass of can + mass of oven dried soil(Mcds), gm	80.56	82.17	86.27
Mass of water (Mw), gm Mw = B-C	17.47	21.45	20.51
Mass of dry soil(Ms), gm Md = C-A	42.65	56.79	49.26
Water Content(w), % Wc = Mw/Md	40.96	37.76	41.64
Average water content(w), %	**40.12**		

Sample Location	**Ameya**		
Trial Number	1	2	3
Can number	P67	A2	2
A. Mass of can(Mc), gm	35.50	25.18	34.62
B. Mass of can + moist soil (Mcms), gm	95.70	103.52	104.48
C. Mass of can + mass of oven dried soil(Mcds), gm	78.93	82.79	84.72
Mass of water (Mw), gm Mw = B-C	16.77	20.73	19.76
Mass of dry soil(Ms), gm Md = C-A	43.43	57.61	50.10
Water Content(w), % Wc = Mw/Md	38.61	35.98	39.45
Average water content(w), %	**38.01**		

APPENDIX B: SAMPLE SPECIFIC GRAVITY TEST ANALYSIS DATA

Sample Location	Chida		
Determination Code	9	B	8
Mass of dry, clean Calibrated pycnometer, Mp, in gm	31.36	30.96	30.77
A. Mass of oven dry sample(gm)	25.00	25.00	25.00
B. Mass of Pycnometer + water(gm)	127.75	128.10	125.90
C. Mass of Pycnometer + water + sample(gm)	143.60	143.89	141.73
Observed temperature of water,Ti	23.00	24.00	24.00
Temperature of pycnometer when Mpsw was taken, Tx, in oc	26.00	26.00	26.00
Temperature Correction factor K for Tx	1.00	1.00	1.00
Specific gravity at 20oc, Gs=A*k/(A+B-C)	2.73	2.71	2.72
Average Specific gravity at 20oc, Gs	2.72		

Sample Location	Ameya		
Determination Code	8	B	-
Mass of dry, clean Calibrated pycnometer, Mp, in gm	30.77	30.96	31.49
A. Mass of oven dry sample(gm)	25.00	25.00	25.00
B. Mass of Pycnometer + water(gm)	125.81	127.50	130.00
C. Mass of Pycnometer + water + sample(gm)	141.46	143.20	145.64
Observed temperature of water,Ti	27.00	27.00	27.00
Temperature of pycnometer when Mpsw was taken, Tx, in oc	29.00	29.00	29.00
Temperature Correction factor K for Tx	1.00	1.00	1.00
Specific gravity at 20oc, Gs=A*k/(A+B-C)	2.67	2.68	2.66
Average Specific gravity at 20oc, Gs	2.67		

Bone Ash			
Determination Code	09-D	08-G	-
Mass of dry, clean Calibrated pycnometer, Mp, in gm	31.36	30.96	31.49
A. Mass of oven dry sample(gm)	25.00	25.00	25.00
B. Mass of Pycnometer + water(gm)	125.65	127.70	124.87
C. Mass of Pycnometer + water + sample(gm)	142.23	144.23	141.73
Observed temperature of water,Ti	25.00	25.00	25.00
Temperature of pycnometer when Mpsw was taken, Tx, in oc	27.00	27.00	27.00
Temperature Correction factor K for Tx	1.00	1.00	1.00
Specific gravity at 20oc, Gs=A*k/(A+B-C)	2.96	2.95	3.07
Average Specific gravity at 20oc, Gs	2.99		

APPENDIX C: GRAIN SIZE DISTRIBUTION TEST ANALYSIS DATA

I. Wet Sieve and Hydrometer analysis

Chida Soil Sample					
Sieve Number	**Sieve size (mm)**	**Mass of Retain on Each Sieve (g)**	**Percentage of Retained Soil (%)**	**Percentage of cumulative Retained Soil**	**Percentage of Passing Soil Particle (%)**
3/8 in	9.500	0.00	0.00	0.00	100.00
4	4.750	2.87	0.57	0.57	99.43
10	2.000	2.24	0.45	1.02	98.98
20	0.850	2.29	0.46	1.48	98.52
40	0.425	1.71	0.34	1.82	98.18
60	0.250	0.61	0.12	1.94	98.06
100	0.150	2.36	0.47	2.42	97.58
200	0.075	2.74	0.55	2.97	97.03
Pan	pan	485.17	97.03	100.00	0.00
	Sum	**500.00**			

Hydrometer number	**152H**	**Weight of sample**	**50**	**gm**
Specific gravity	**2.72**	**Zero correction**	**6**	
Dispersing agent	**sodium hexa metaphosphate**	**Meniscus of correction**	**1**	

Time	**T (min)**	**Temp (°C)**	**Actual Hydro. Rdg. R**	**Hyd. Corr. for Meniscus R**	**L** Table 2, D422	**K** Table 3, D422	**D (mm)**	**C_T** Table 4, Lab Manual	**a** Table 1, D422	**Corr. Hydro. Rdg. R_c**	**% Finer P**	**% Adjusted Finer PA**
							0.075					97.03
3:00	1	21	49	50	8.1	0.013204	0.0376	0.2	0.986	44.2	87.16	84.58
3:02	2	21	48	49	8.3	0.013204	0.0269	0.2	0.986	43.2	85.19	82.66
3:05	5	21	47	48	8.4	0.013204	0.0171	0.2	0.986	42.2	83.22	80.75
3:10	10	21	46	47	8.6	0.013204	0.0122	0.2	0.986	41.2	81.25	78.84
3:15	15	21	45	46	8.8	0.013204	0.0101	0.2	0.986	40.2	79.27	76.92
3:30	30	21	44	45	8.9	0.013204	0.0072	0.2	0.986	39.2	77.30	75.01
4:00	60	21	42	43	9.2	0.013204	0.0052	0.2	0.986	37.2	73.36	71.18
5:00	120	21	41	42	9.4	0.013204	0.0037	0.2	0.986	36.2	71.39	69.27
7:00	240	25	39	40	9.7	0.012598	0.0025	1.3	0.986	35.3	69.61	67.55
11:00	480	24	37	38	10.1	0.012748	0.0018	1	0.986	33	65.08	63.15
3:00	1440	23	35	36	10.4	0.012898	0.0011	0.7	0.986	30.7	60.54	58.75

ACB Soil Sample					
Sieve Number	**Sieve size (mm)**	**Mass of Retain on Each Sieve (g)**	**Percentage of Retained Soil (%)**	**Percentage of cumulative Retained Soil**	**Percentage of Passing Soil Particle (%)**
3/8 in	9.500	0.00	0.00	0.00	100.00
4	4.750	1.28	0.26	0.26	99.74
10	2.000	8.42	1.68	1.94	98.06
20	0.850	10.04	2.01	3.95	96.05
40	0.425	4.58	0.92	4.86	95.14
60	0.250	1.58	0.32	5.18	94.82
100	0.150	3.09	0.62	5.80	94.20
200	0.075	10.65	2.13	7.93	92.07
Pan	pan	460.373	92.07	100.00	0.00
	sum	**500.00**			

Hydrometer number	**152H**	**Weight of sample**	**50**	**gm**
Specific gravity	**2.7**	**Zero correction**	**6**	
Dispersing agent	**sodium hexa metaphosphate**	**Meniscus of correction**	**1**	

Time	**T (min)**	**Temp (°C)**	**Actual Hydro .Rdg. R_a**	**Hyd. Corr. for Meniscus R**	**L** Table 2, D422	**K** Table 3, D422	**D (mm)**	**C_T** Table 4, Lab Manual	**a** Table 1, D422	**Corr. Hydr. Rdg. R_c**	**% Finer P**	**% Adjusted Finer PA**
							0.075					92.07
3:00	1	23	50	51	7.9	0.01297	0.0365	0.7	0.99	45.7	90.49	83.31
3:02	2	23	49	50	8.1	0.01297	0.0261	0.7	0.99	44.7	88.51	81.49
3:05	5	23	48	49	8.3	0.01297	0.0167	0.7	0.99	43.7	86.53	79.67
3:10	10	23	44	45	8.9	0.01297	0.0122	0.7	0.99	39.7	78.61	72.38
3:15	15	23	43	44	9.1	0.01297	0.0101	0.7	0.99	38.7	76.63	70.55
3:30	30	23	41	42	9.4	0.01297	0.0073	0.7	0.99	36.7	72.67	66.91
4:00	60	23	39	40	9.7	0.01297	0.0052	0.7	0.99	34.7	68.71	63.26
5:00	120	23	37	38	10.1	0.01297	0.0038	0.7	0.99	32.7	64.75	59.61
7:00	240	25	35	36	10.4	0.01267	0.0026	1.3	0.99	31.3	61.97	57.06
11:00	480	24	34	35	10.6	0.01282	0.0019	1	0.99	30	59.40	54.69
3:00	1440	23	32	33	10.9	0.01297	0.0011	0.7	0.99	27.7	54.85	50.50

Ameya Soil Sample					
Sieve Number	**Sieve size (mm)**	**Mass of Retain on Each Sieve (g)**	**Percentage of Retained Soil (%)**	**Percentage of cumulative Retained Soil**	**Percentage of Passing Soil Particle (%)**
3/8 in	9.500	0.00	0.00	0.00	100.00
4	4.750	0.00	0.00	0.00	100.00
10	2.000	1.41	0.28	0.28	99.72
20	0.850	6.46	1.29	1.57	98.43
40	0.425	10.78	2.16	3.73	96.27
60	0.250	6.98	1.40	5.13	94.87
100	0.150	17.04	3.41	8.54	91.46
200	0.075	10.22	2.04	10.58	89.42
Pan	pan	447.099	89.42	100.00	0.00
	sum	**500.00**			

Hydrometer number	**152H**	**Weight of sample**	**50**	**gm**
Specific gravity	**2.67**	**Zero correction**	**6**	
Dispersing agent	**sodium hexa metaphosphate**	**Meniscus of correction**	**1**	

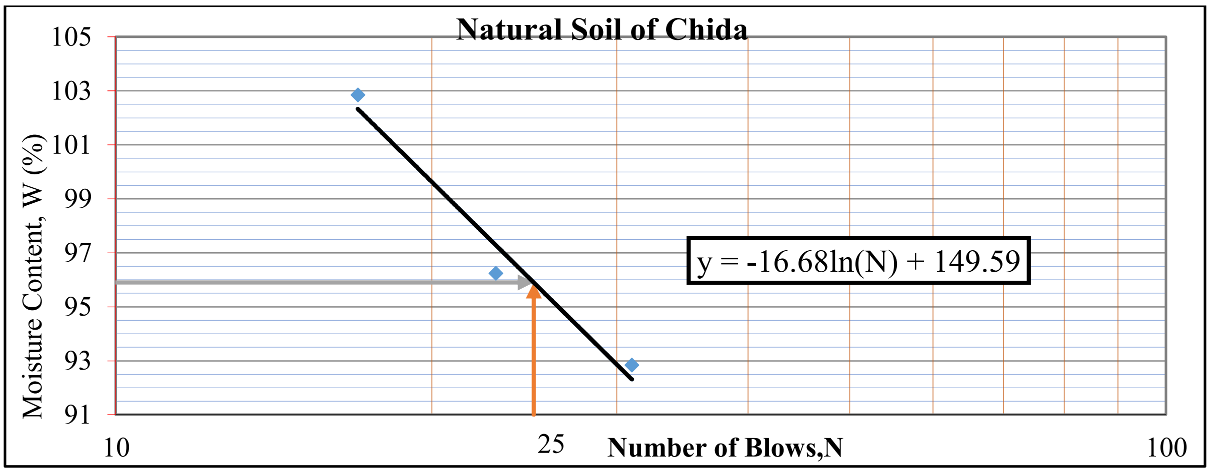
Time	**T (min)**	**Temp (°C)**	**Actual Hydro. Rdg**	**Hyd. Corr. for Meniscus**	**L Table 2, D422**	**K Table 3, D422**	**D (mm)**	**C_T Table 4, Lab**	**a Table 1, D422**	**Corr. Hydro. Rdg.**	**% Finer P**	**% Adjusted Finer PA**
							0.075					89.42
3:00	1	24	48	49	8.3	0.01301	0.0375	1	0.996	44	87.65	78.37
3:02	2	24	46	47	8.6	0.01301	0.0270	1	0.996	42	83.66	74.81
3:05	5	24	45	46	8.8	0.01301	0.0173	1	0.996	41	81.67	73.03
3:10	10	24	44	45	8.9	0.01301	0.0123	1	0.996	40	79.68	71.25
3:15	15	24	43	44	9.1	0.01301	0.0101	1	0.996	39	77.69	69.47
3:30	30	24	41	42	9.4	0.01301	0.0073	1	0.996	37	73.70	65.91
4:00	60	24	40	41	9.6	0.01301	0.0052	1	0.996	36	71.71	64.12
5:00	120	24	38	39	9.9	0.01301	0.0037	1	0.996	34	67.73	60.56
7:00	240	25	35	36	10.4	0.01286	0.0027	1.3	0.996	31.3	62.35	55.75
11:00	480	25	32	33	10.9	0.01286	0.0019	1.3	0.996	28.3	56.37	50.41
3:00	1440	23	30	31	11.2	0.01317	0.0012	0.7	0.996	25.7	51.19	45.78

II. Combined wet sieve analysis and hydrometer analysis

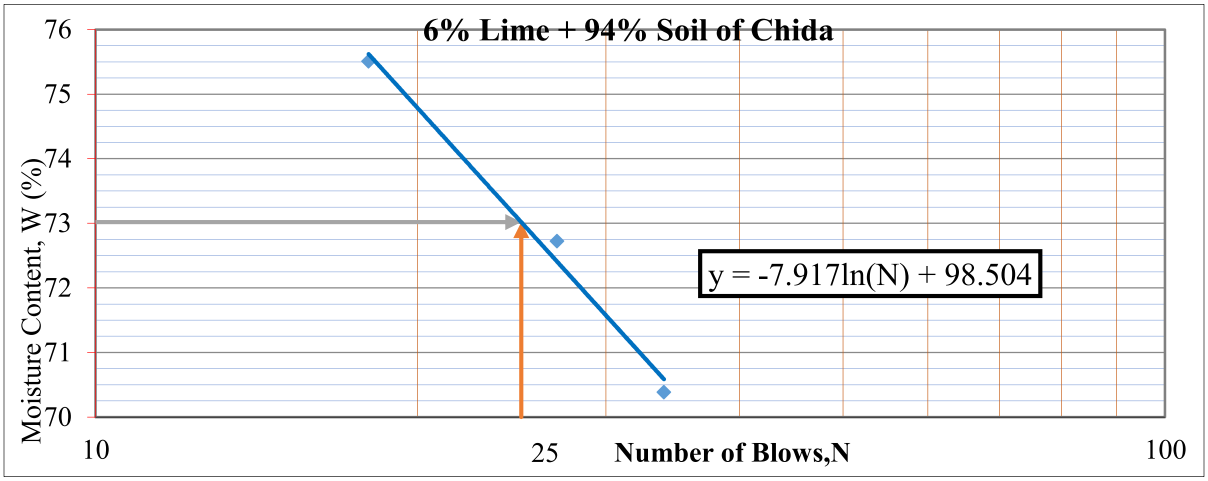
Diameter (mm)	Chida	ACB	Ameya
	Passing (%)	Passing (%)	Passing (%)
9.500	100.00	100.00	100.00
4.750	99.43	99.74	100.00
2.000	98.98	98.06	99.72
0.850	98.52	96.05	98.43
0.425	98.18	95.14	96.27
0.250	98.06	94.82	94.87
0.150	97.58	94.20	91.46
0.075	97.03	92.07	89.42
0.038	84.58	83.31	78.37
0.027	82.66	81.49	74.81
0.017	80.75	79.67	73.03
0.012	78.84	72.38	71.25
0.010	76.92	70.55	69.47
0.007	75.01	66.91	65.91
0.005	71.18	63.26	64.12
0.004	69.27	59.61	60.56
0.003	67.55	57.06	55.75
0.002	63.15	54.69	50.41
0.001	58.75	50.50	45.78

APPENDIX D: SAMPLE ATTERBERG'S LIMIT TEST ANALYSIS DATA

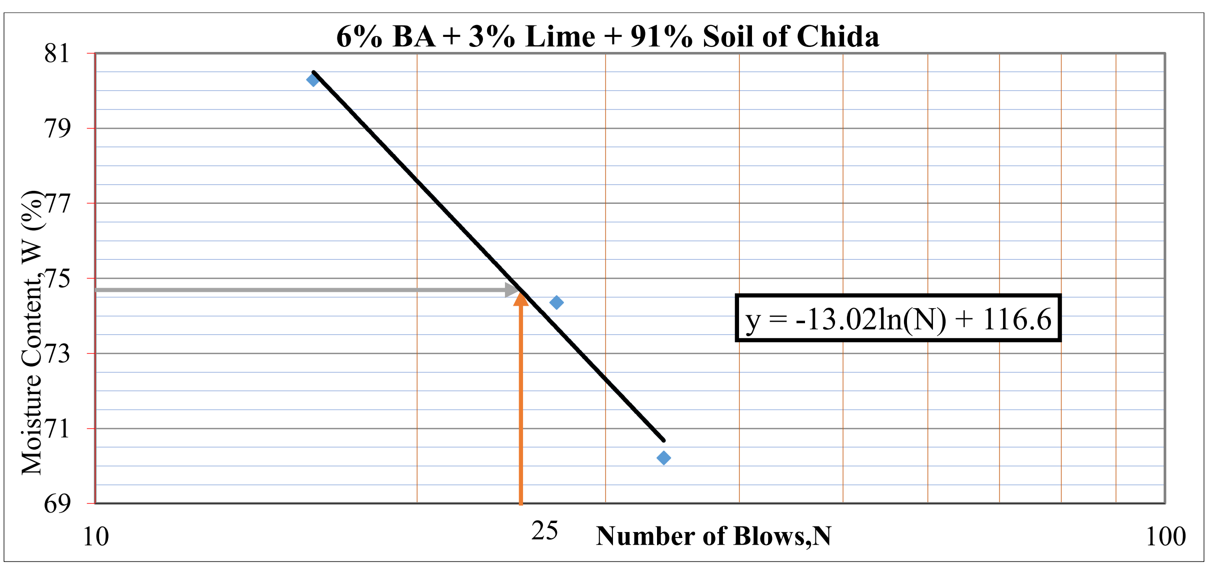
Sample Location: Chida	Natural Soil of Chida				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	31	23	17		
Sample Trial Number	1	2	3	1	2
Container Number	2*1	A36	16	G14	AA
Mass of Container + Wet Soil (g) = (W_1)	31.09	42.59	31.90	26.07	25.06
Mass of Container + Dry Soil (g) = (W_2)	19.17	30.30	19.63	24.44	23.51
Mass of Container (g) = (W_3)	6.33	17.53	7.70	20.11	19.25
Mass of Moisture (g) = (W_1 - W_2) = A	11.92	12.29	12.27	1.63	1.55
Mass of Dry Soil (g) = (W_2 - W_3) = B	12.84	12.77	11.93	4.33	4.26
Moisture Content (%) = (A / B) x 100	92.83	96.24	102.85	37.64	36.38
Liquid Limit (LL) (%):	95.90	**AV. Plas. Lim.**		**37.0**	
Plastic Limit (PL) (%):	37.0				
Plasticity Index (PI) (%): LL - PL	**58.9**				



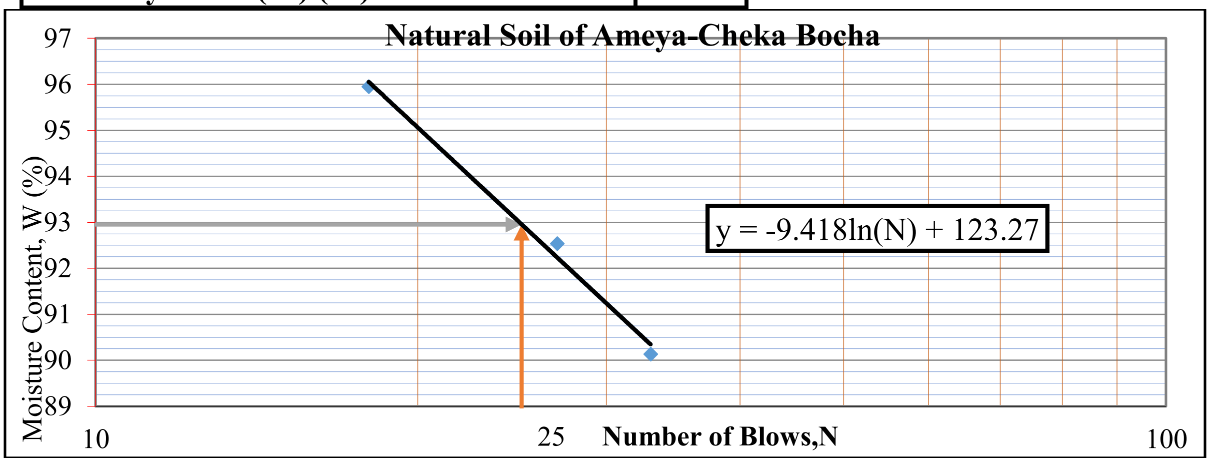
Sample Location: Chida	Additive Content: 6% Lime + 94% soil				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	34	27	18		
Sample Trial Number	1	2	3	1	2
Container Number	AA	G21	A5	B12	G3
Mass of Container + Wet Soil (g) = (W_1)	54.06	48.24	42.74	21.61	23.08
Mass of Container + Dry Soil (g) = (W_2)	39.68	36.27	31.21	19.89	20.51
Mass of Container (g) = (W_3)	19.25	19.81	15.94	16.97	16.43
Mass of Moisture (g) = (W_1 - W_2) = A	14.38	11.97	11.53	1.72	2.57
Mass of Dry Soil (g) = (W_2 - W_3) = B	20.43	16.46	15.27	2.92	4.08
Moisture Content (%) = (A / B)x 100	70.39	72.72	75.51	58.90	62.99
Liquid Limit (LL) (%):	73.02	**AV. Plas. Lim.**		**60.9**	
Plastic Limit (PL) (%):	60.95				
Plasticity Index (PI) (%): LL - PL	**12.07**				



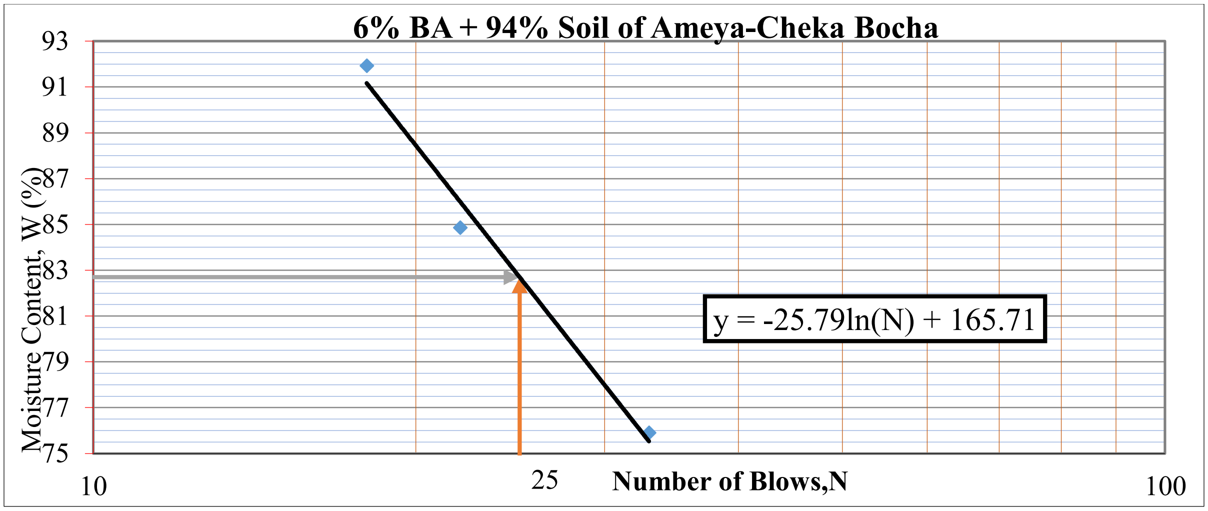
Sample Location: Chida	Additive Content: 6% BA + 3% L + 91% S				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	34	27	16		
Sample Trial Number	1	2	3	1	2
Container Number	A	2	G3	AA	B11
Mass of Container + Wet Soil (g) = (W_1)	25.07	42.49	37.65	31.94	29.01
Mass of Container + Dry Soil (g) = (W_2)	17.08	32.46	28.20	27.18	24.42
Mass of Container (g) = (W_3)	5.70	18.97	16.43	19.25	16.96
Mass of Moisture (g) = (W_1 - W_2) = A	7.99	10.03	9.45	4.76	4.59
Mass of Dry Soil (g) = (W_2 - W_3) = B	11.38	13.49	11.77	7.93	7.46
Moisture Content (%) = (A / B)x 100	70.21	74.35	80.29	60.03	61.53
Liquid Limit (LL) (%):	74.69	**AV. Plas. Lim.**		**60.78**	
Plastic Limit (PL) (%):	60.78				
Plasticity Index (PI) (%): LL - PL	**13.91**				



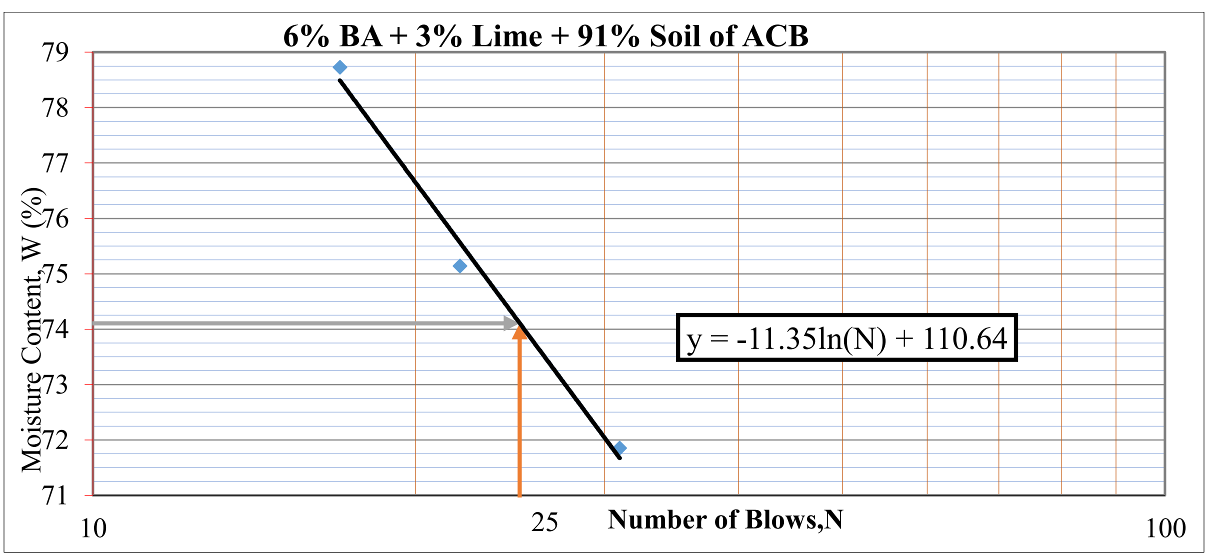
Sample Location: ACB	**Natural soil of ACB**				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	33	27	18		
Sample Trial Number	1	2	3	1	2
Container Number	L14	B10	42	B11	C14
Mass of Container + Wet Soil (g) = (W_1)	40.10	43.08	45.53	24.90	13.63
Mass of Container + Dry Soil (g) = (W_2)	30.32	31.79	34.63	22.66	11.56
Mass of Container (g) = (W_3)	19.47	19.59	23.27	16.96	5.94
Mass of Moisture (g) = (W_1 - W_2) = A	9.78	11.29	10.90	2.24	2.07
Mass of Dry Soil (g) = (W_2 - W_3) = B	10.85	12.20	11.36	5.70	5.62
Moisture Content (%) = (A / B)x 100	90.14	92.54	95.95	39.30	36.83
Liquid Limit (LL) (%):	92.95	**AV. Plas. Lim.**		**38.1**	
Plastic Limit (PL) (%):	38.1				
Plasticity Index (PI) (%): LL - PL	**54.9**				



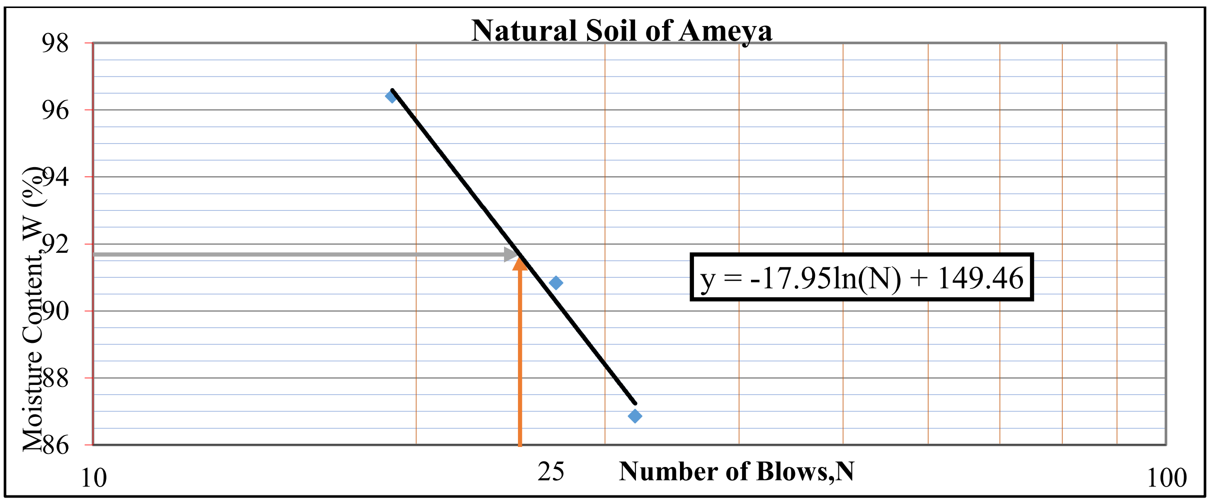
Sample Location: ACB	**Additive Content: 6% BA + 94% soil**				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	33	22	18		
Sample Trial Number	1	2	3	1	2
Container Number	B12	A5	G3	L14	2*1
Mass of Container + Wet Soil (g) = (W_1)	41.28	40.84	46.60	26.44	12.33
Mass of Container + Dry Soil (g) = (W_2)	30.79	29.41	32.15	24.08	10.31
Mass of Container (g) = (W_3)	16.97	15.94	16.43	19.47	6.30
Mass of Moisture (g) = (W_1 - W_2) = A	10.49	11.43	14.45	2.36	2.02
Mass of Dry Soil (g) = (W_2 - W_3) = B	13.82	13.47	15.72	4.61	4.01
Moisture Content (%) = (A / B)x 100	75.90	84.86	91.92	51.19	50.37
Liquid Limit (LL) (%):	82.70	**AV. Plas. Lim.**		**50.78**	
Plastic Limit (PL) (%):	50.78				
Plasticity Index (PI) (%): LL - PL	**31.91**				

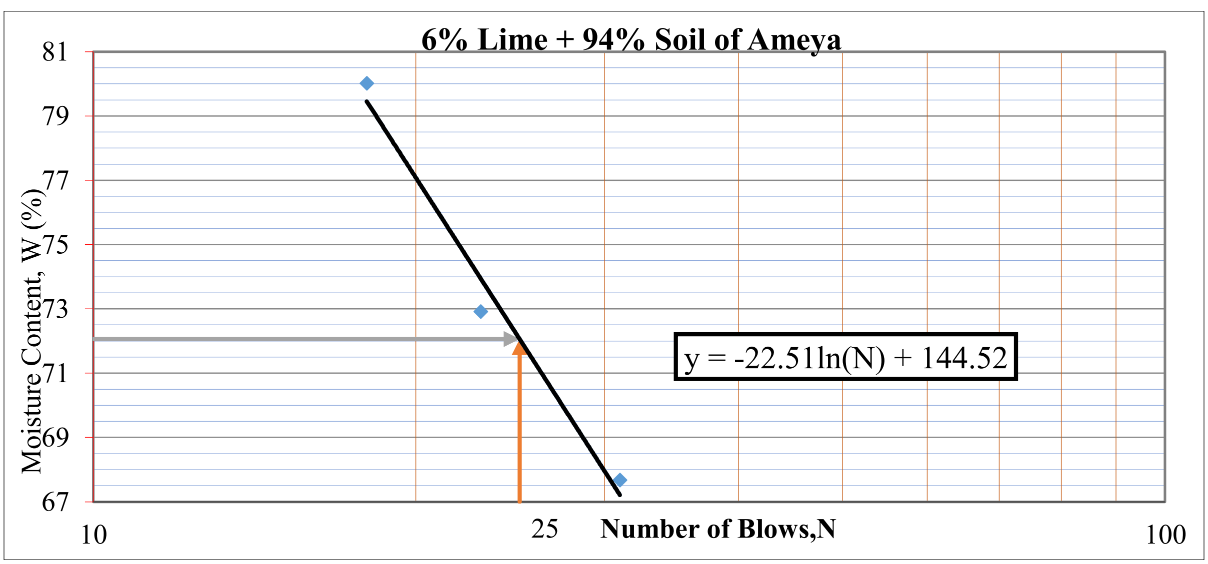


Sample Location: ACB	Additive Content: 6% BA + 3% L + 91% S				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	31	22	17		
Sample Trial Number	1	2	3	1	2
Container Number	G3	B10	AA	L14	G14
Mass of Container + Wet Soil (g) = (W_1)	35.54	41.50	40.09	29.01	28.82
Mass of Container + Dry Soil (g) = (W_2)	27.55	32.10	30.91	25.42	25.47
Mass of Container (g) = (W_3)	16.43	19.59	19.25	19.47	20.11
Mass of Moisture (g) = ($W_1 - W_2$) = A	7.99	9.40	9.18	3.59	3.35
Mass of Dry Soil (g) = ($W_2 - W_3$) = B	11.12	12.51	11.66	5.95	5.36
Moisture Content (%) = (A / B)x 100	71.85	75.14	78.73	60.34	62.50
Liquid Limit (LL) (%):	74.11	**AV. Plas. Lim.**		**61.42**	
Plastic Limit (PL) (%):	61.42				
Plasticity Index (PI) (%): LL - PL	**12.69**				

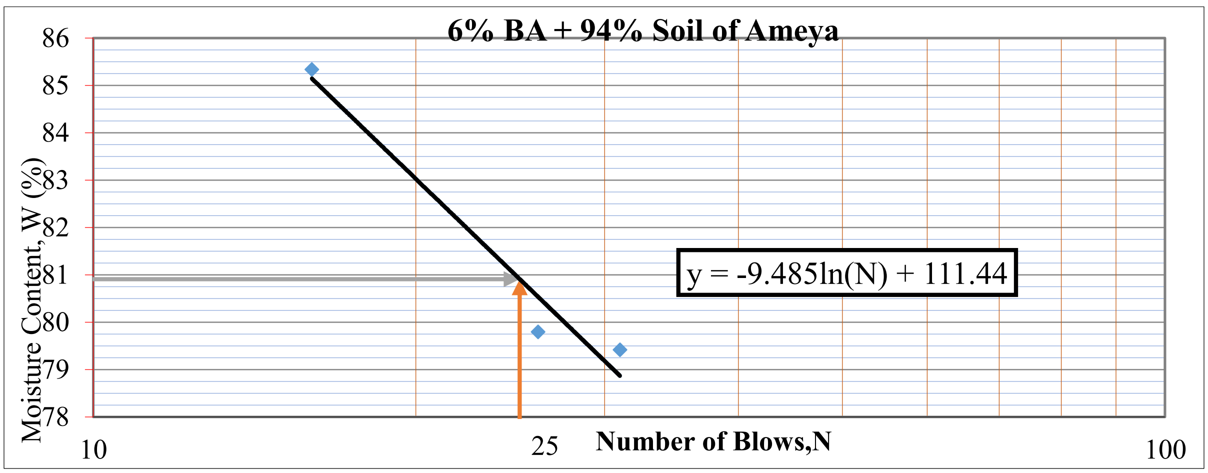


Sample Location: Ameya	**Natural soil of Ameya**				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	32	27	19		
Sample Trial Number	1	2	3	1	2
Container Number	42	L14	A7	2	A36
Mass of Container + Wet Soil (g) = (W_1)	49.58	44.26	44.33	25.73	24.20
Mass of Container + Dry Soil (g) = (W_2)	37.35	32.46	31.16	23.95	22.21
Mass of Container (g) = (W_3)	23.27	19.47	17.50	18.97	17.53
Mass of Moisture (g) = (W_1 - W_2) = A	12.23	11.80	13.17	1.78	1.99
Mass of Dry Soil (g) = (W_2 - W_3) = B	14.08	12.99	13.66	4.98	4.68
Moisture Content (%) = (A / B)x 100	86.86	90.84	96.41	35.74	42.52
Liquid Limit (LL) (%):	91.68	**AV. Plas. Lim.**		**39.1**	
Plastic Limit (PL) (%):	39.13				
Plasticity Index (PI) (%): LL - PL	**52.55**				





Sample Location: Ameya	**Additive Content: 6% BA + 94% soil**				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	31	26	16		
Sample Trial Number	1	2	3	1	2
Container Number	AA	A36	G21	A17	16
Mass of Container + Wet Soil (g) = (W_1)	39.29	41.83	38.22	28.62	16.30
Mass of Container + Dry Soil (g) = (W_2)	30.42	31.05	28.43	25.71	13.39
Mass of Container (g) = (W_3)	19.25	17.53	16.96	19.81	7.70
Mass of Moisture (g) = (W_1 - W_2) = A	8.87	10.79	9.79	2.90	2.90
Mass of Dry Soil (g) = (W_2 - W_3) = B	11.17	13.52	11.47	5.90	5.69
Moisture Content (%) = (A / B)x 100	79.42	79.79	85.34	49.19	51.00
Liquid Limit (LL) (%):	80.91	**AV. Plas. Lim.**		**50.1**	
Plastic Limit (PL) (%):	50.10				
Plasticity Index (PI) (%): LL - PL	**30.81**				



Sample Location: Ameya	**Additive Content: 6% BA + 3% L + 91% S**				
Determination	***Liquid Limit***			***Plastic Limit***	
Number of blows	32	23	18		
Sample Trial Number	1	2	3	1	2
Container Number	A36	2*1	AA	L14	G3
Mass of Container + Wet Soil (g) = (W_1)	37.03	30.32	40.14	28.98	25.70
Mass of Container + Dry Soil (g) = (W_2)	28.98	20.01	31.02	25.37	22.12
Mass of Container (g) = (W_3)	17.53	6.30	19.25	19.47	16.43
Mass of Moisture (g) = (W_1 - W_2) = A	8.05	10.31	9.12	3.61	3.58
Mass of Dry Soil (g) = (W_2 - W_3) = B	11.45	13.71	11.77	5.90	5.69
Moisture Content (%) = (A / B)x 100	70.31	75.20	77.49	61.19	62.92
Liquid Limit (LL) (%):	73.64	**AV. Plas. Lim.**		**62.05**	
Plastic Limit (PL) (%):	62.05				
Plasticity Index (PI) (%): LL - PL	**11.59**				

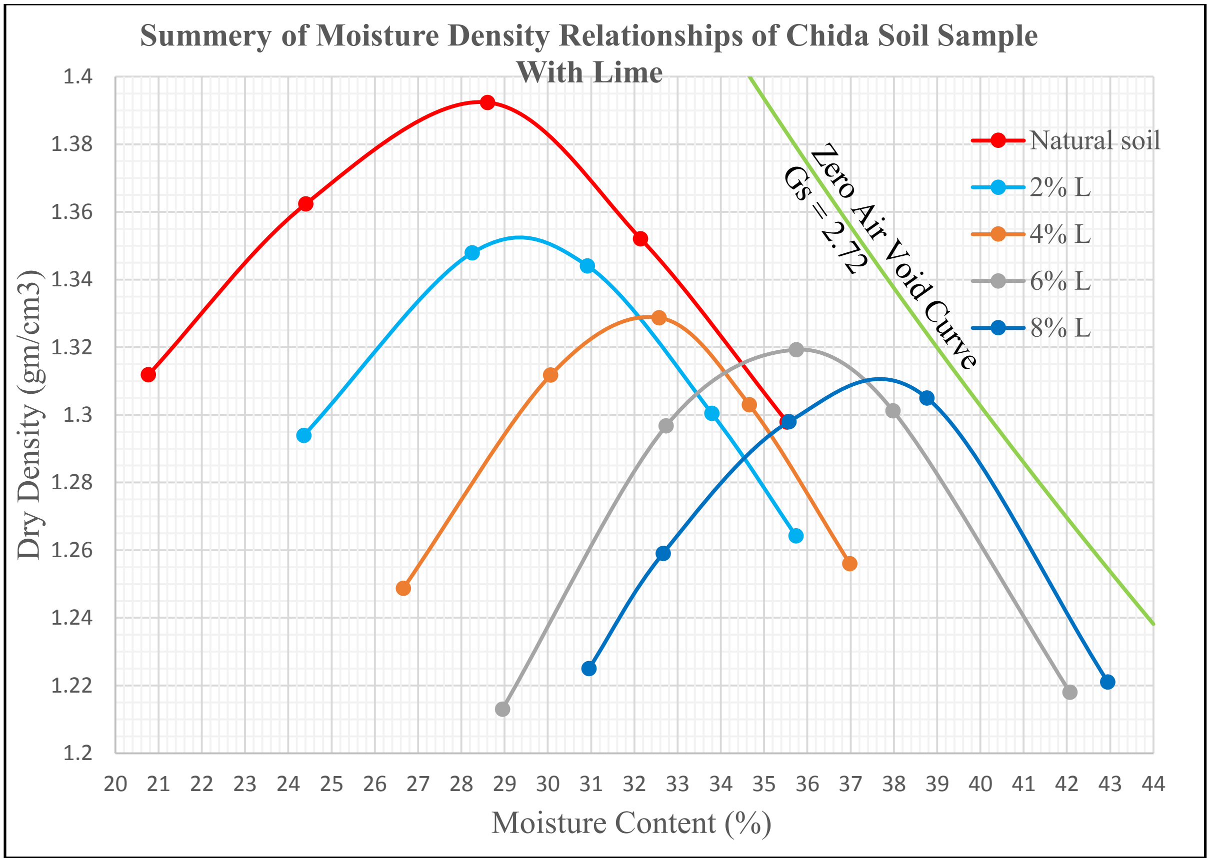
APPENDIX E: Linear Shrinkage Analysis Data

No	Sample type / Mix-Proportion of additives (%)	Length of Mold (cm)	Soil sample of Chida		Soil sample ACB		Soil sample Ameya	
			Oven dry length(cm)	LS (%)	oven dry length(cm)	LS (%)	oven dry length(cm)	LS (%)
1	Natural soil	14.00	10.80	22.87	11.08	20.85	11.36	18.84
2	2% Lime	14.00	11.69	16.48	11.79	15.80	11.88	15.12
3	4% Lime	14.00	12.35	11.81	12.43	11.18	12.52	10.55
4	6% Lime	14.00	13.02	7.00	13.13	6.21	13.24	5.42
5	8% Lime	14.00	13.39	4.34	13.45	3.94	13.50	3.54
6	2% BA	14.00	10.83	22.66	11.12	20.59	11.41	18.51
7	4% BA	14.00	11.43	18.34	11.58	17.32	11.72	16.30
8	6% BA	14.00	11.94	14.74	12.01	14.20	12.09	13.65
9	8% BA	14.00	11.78	15.85	12.05	13.94	12.32	12.03
10	6% BA +2% L	14.00	12.42	11.28	12.51	10.65	12.60	10.02
11	6% BA +3% L	14.00	13.05	6.81	13.16	6.02	13.27	5.23
12	6% BA +4% L	14.00	13.26	5.30	13.31	4.90	13.37	4.51

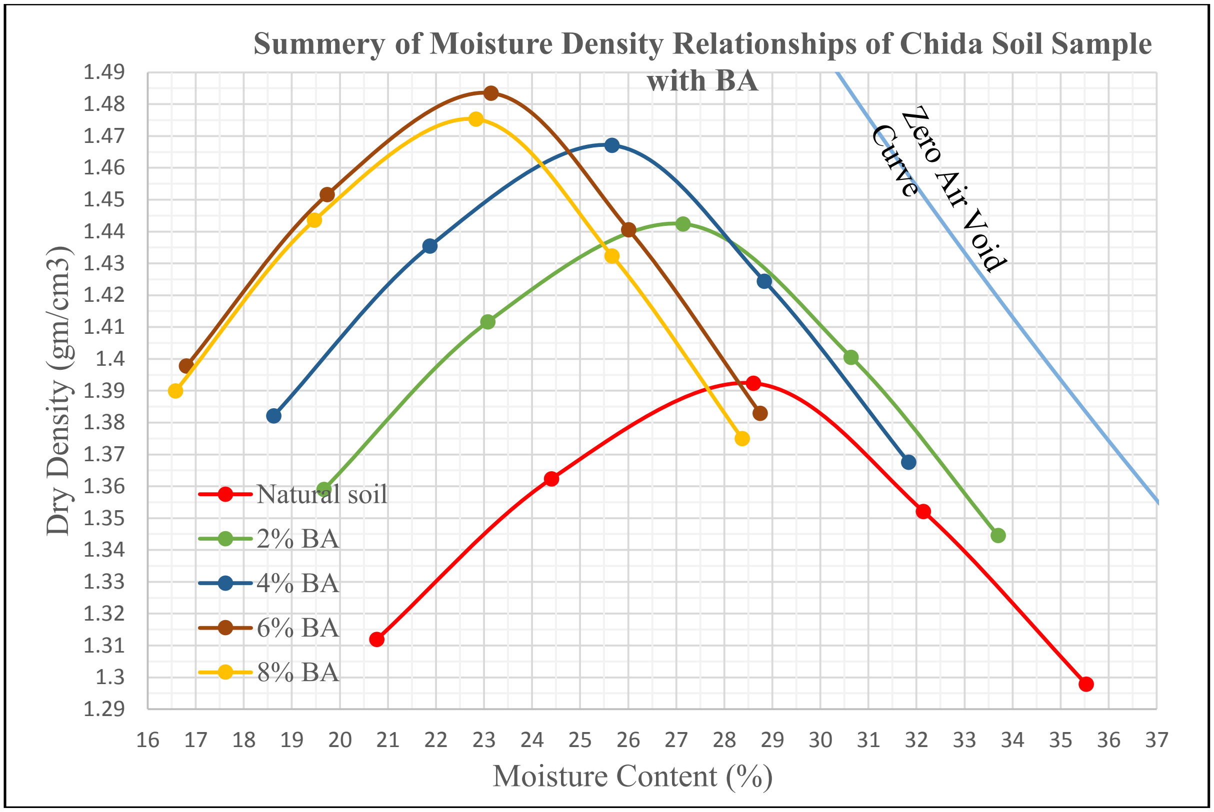
APPENDIX F: Compaction Test Analysis Data

Sample Location: Chida						Natural soil of Chida				
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6088.70		6323.40		6527.00		6518.60		6459.80	
Mold(gm)	2723.60		2723.60		2723.60		2723.60		2723.60	
Wet Soil(gm)	3365.10		3599.80		3803.40		3795.00		3736.20	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.58432		1.69482		1.79068		1.78672		1.75904	
Moisture Content Determination										
Container Code.	NC	G63	C3	190	F	D	T1	5	P6	P3
Wet soil + Container(gm)	75.6	90.1	90.1	113.5	98.1	87.6	100.5	94.3	95.9	101.6
dry soil + container(gm)	65.7	78.8	77.6	98.0	84.4	74.7	85.2	75.6	80.7	84.2
container(gm)	17.5	25.3	26.6	34.1	36.3	29.6	37.6	17.5	37.6	36.0
moisture(gm)	9.9	11.2	12.5	15.6	13.7	12.9	15.3	18.7	15.1	17.4
Dry soil(gm)	48.2	53.5	51.0	63.9	48.1	45.1	47.6	58.1	43.1	48.2
Moisture content %	20.6	21.0	24.5	24.4	28.6	28.6	32.1	32.2	35.0	36.0
Avg. Moisture content %	**20.76647**		**24.40296**		**28.60683**		**32.14542**		**35.53180**	
Dry Density gm/cm3	**1.31189**		**1.36236**		**1.39237**		**1.35209**		**1.29788**	
			OMC (%)			**28.52**	**MDD (gm/cm^3)**			**1.3924**

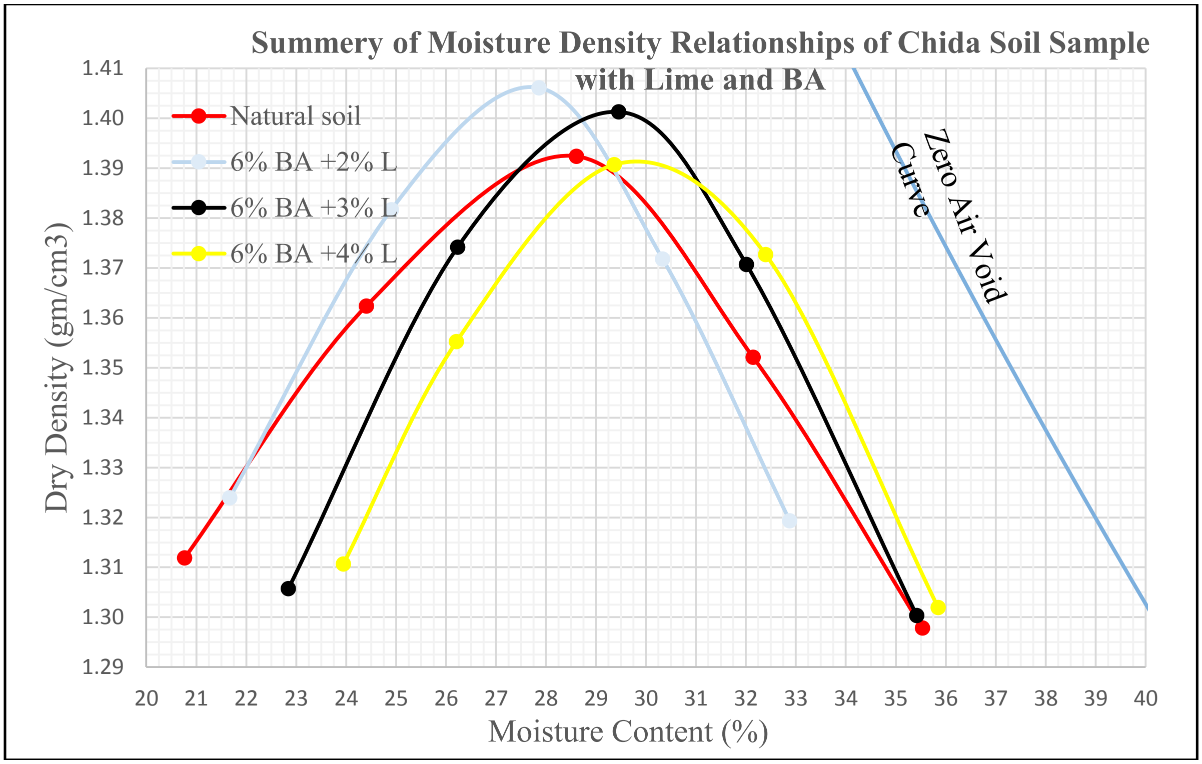
Sample Location: Chida						Additive Content: 6% Lime + 94% soil				
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6046.00		6379.50		6527.30		6537.10		6399.00	
Mold(gm)	2723.60		2723.60		2723.60		2723.60		2723.60	
Wet Soil(gm)	3322.4		3655.9		3803.7		3813.5		3675.4	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.56422		1.72123		1.79082		1.79543		1.73041	
Moisture Content Determination										
Container Code.	A13	F	G-5	P6	ZE	G63	E	LHE	190	D
Wet soil + Container(gm)	94.2	106.4	85.0	116.5	100.5	90.9	99.6	98.7	103.5	93.4
dry soil + container(gm)	81.3	90.6	68.2	97.2	82.8	73.5	82.6	78.5	82.9	74.5
container(gm)	36.5	36.3	17.4	37.6	33.0	25.3	37.9	25.4	34.1	29.6
moisture(gm)	12.9	15.8	16.8	19.3	17.6	17.4	16.9	20.3	20.6	18.8
Dry soil(gm)	44.8	54.2	50.8	59.6	49.8	48.2	44.7	53.1	48.8	44.9
Moisture content %	28.7	29.2	33.0	32.4	35.4	36.1	37.8	38.2	42.3	41.9
Avg. Moisture content %	**28.95038**		**32.73470**		**35.74344**		**37.97952**		**42.06882**	
Dry Density gm/cm3	**1.21304**		**1.29675**		**1.31927**		**1.30123**		**1.21801**	
			OMC (%)			**35.9**	**MDD (gm/cm^3)**			**1.319**



Sample Location: Chida				Additive Content: 6% BA + 94% soil					
Density Determination									
Test No.	1		2		3		4		5
Mold + Wet soil(gm)	6175.8		6399.5		6588.2		6563.6		6489.7
Mold(gm)	2708		2708		2708		2708		2708
Wet Soil(gm)	3467.8		3691.5		3880.2		3855.6		3781.7
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00
Bulk Density gm/cm^3	1.63267		1.73799		1.82684		1.81525		1.78046
Moisture Content Determination									
Container Code.	A13	G-5	ZE	E	LHE	A	P67	A2	2	12
Wet soil + Container(gm)	89.5	87.3	96.5	103.8	95.8	96.8	92.0	98.4	94.8	92.1
dry soil + container(gm)	82.6	76.3	86.1	92.9	82.3	85.9	80.8	82.6	81.3	80.8
container(gm)	36.5	17.4	33.0	37.9	25.4	37.0	35.5	25.2	34.6	41.2
moisture(gm)	6.9	11.0	10.4	10.9	13.6	11.0	11.1	15.8	13.5	11.4
Dry soil(gm)	46.0	58.9	53.1	55.0	56.9	48.9	45.3	57.4	46.7	39.6
Moisture content %	15.0	18.6	19.6	19.9	23.9	22.4	24.6	27.5	28.8	28.7
Avg. Moisture content %	**16.80672**		**19.73316**		**23.14500**		**26.01214**		**28.74827**
Dry Density gm/cm3	**1.39776**		**1.45156**		**1.48348**		**1.44054**		**1.38290**
			OMC (%)			**23**	**MDD gm/cm^3)**		**1.4838**

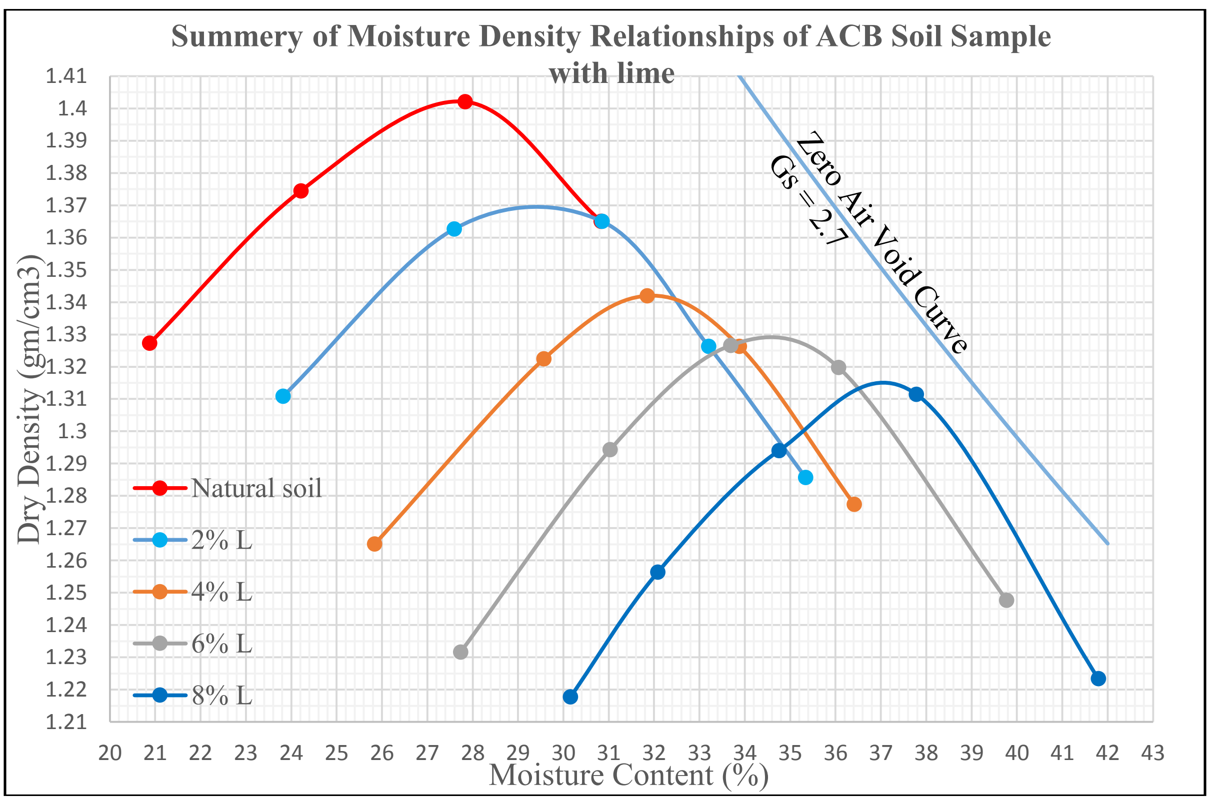


Sample Location: Chida					Additive Content: 6% BA + 3% Lime+ 91% soil					
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6114.9		6392.3		6560.9		6551.3		6448.2	
Mold(gm)	2708		2708		2708		2708		2708	
Wet Soil(gm)	3406.9		3684.3		3852.9		3843.3		3740.2	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.60400		1.73460		1.81398		1.80946		1.76092	
Moisture Content Determination										
Container Code.	J41	P2	A1	P65	A3	A4	A2	3	P2	G-5
Wet soil + Container(gm)	95.8	84.4	93.8	105.3	102.3	78.4	93.0	100.5	83.4	81.1
dry soil + container(gm)	83.7	72.4	80.2	91.4	86.6	64.4	76.6	82.5	66.3	64.2
container(gm)	32.7	17.6	28.7	37.8	32.8	17.3	25.2	26.6	17.6	17.4
moisture(gm)	12.1	12.0	13.6	14.0	15.7	14.0	16.4	18.0	17.0	16.8
Dry soil(gm)	51.0	54.8	51.5	53.6	53.8	47.1	51.4	55.9	48.7	46.9
Moisture content %	23.7	22.0	26.4	26.1	29.3	29.6	31.9	32.1	34.9	35.9
Avg. Moisture content %	**22.84335**		**26.23038**		**29.45365**		**32.01085**		**35.42091**	
Dry Density gm/cm3	**1.30573**		**1.37416**		**1.40126**		**1.37069**		**1.30033**	
			OMC (%)			**29.4**	**MDD (gm/cm^3)**			**1.401**

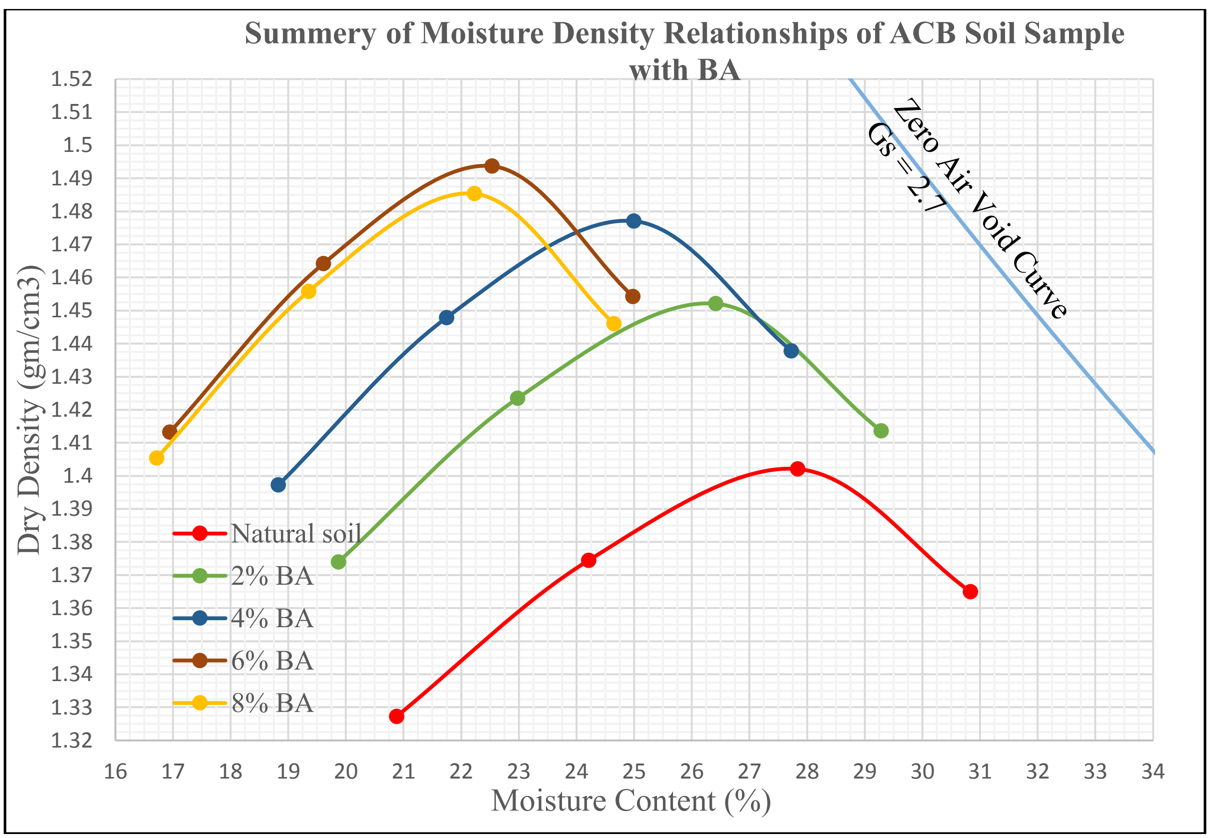


Sample Location: ACB					**Natural soil of ACB**			
Density Determination								
Test No.	1		2		3		4	
Mold + Wet soil(gm)	6115.80		6334.20		6514.90		6501.10	
Mold(gm)	2708.00		2708.00		2708.00		2708.00	
Wet Soil(gm)	3407.80		3626.20		3806.90		3793.10	
Mold cm^3	2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.60443		1.70725		1.79233		1.78583	
Moisture Content Determination								
Container Code.	G63	5	190	P3	C3	P6	D	NC
Wet soil + Container(gm)	64.30	76.47	98.74	91.96	96.62	99.54	92.07	90.39
dry soil + container(gm)	57.61	66.22	86.12	81.06	81.33	86.09	77.50	73.03
container(gm)	25.30	17.54	34.10	35.95	26.60	37.59	29.60	17.47
moisture(gm)	6.69	10.25	12.62	10.90	15.29	13.45	14.57	17.36
Dry soil(gm)	32.31	48.68	52.02	45.11	54.73	48.50	47.90	55.56
Moisture content %	20.71	21.06	24.26	24.16	27.94	27.73	30.42	31.25
Avg. Moisture content %	**20.88077**		**24.21153**		**27.83455**		**30.83152**	
Dry Density gm/cm3	**1.32728**		**1.37447**		**1.40207**		**1.36498**	
	OMC (%)			**27.75**	**MDD (gm/cm^3)**			**1.402**

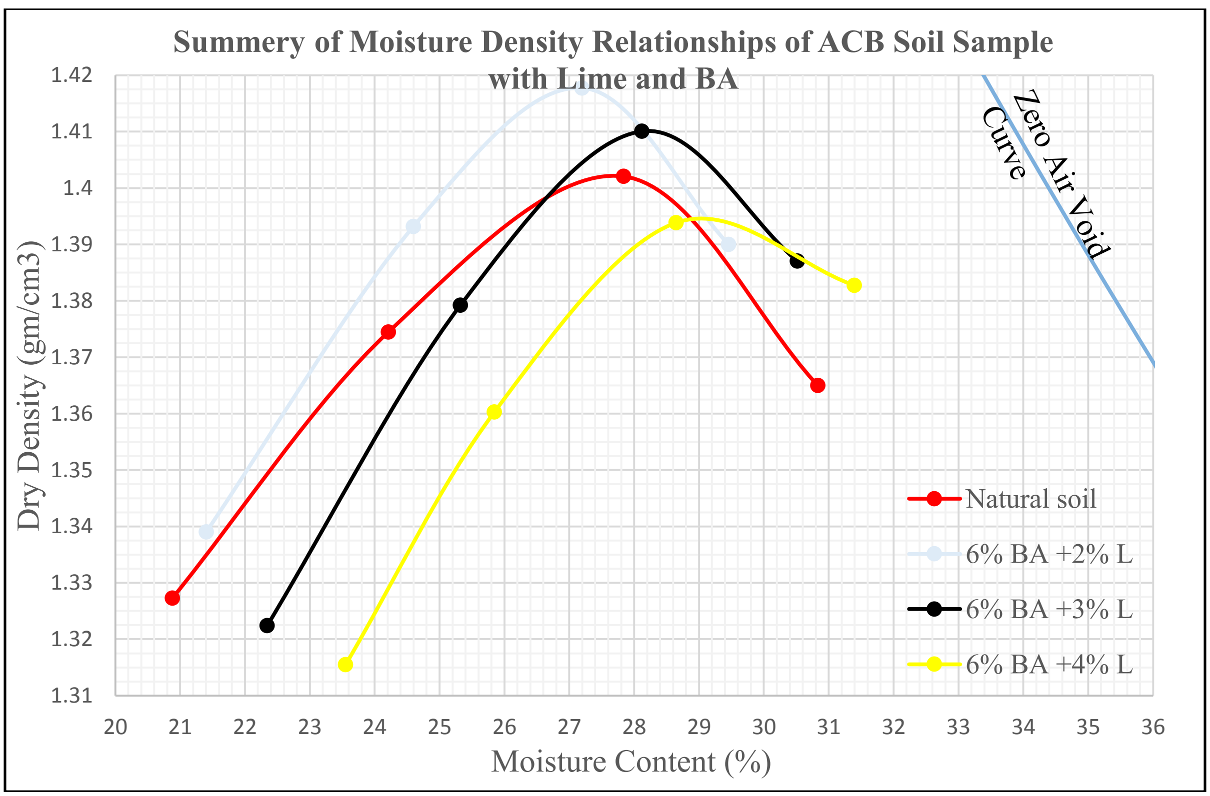
Sample Location: ACB					**Additive Content: 6% Lime + 94% soil**					
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6065.20		6325.80		6490.70		6537.80		6427.60	
Mold(gm)	2723.60		2723.60		2723.60		2723.60		2723.60	
Wet Soil(gm)	3341.6		3602.2		3767.1		3814.2		3704	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.57326		1.69595		1.77359		1.79576		1.74388	
Moisture Content Determination										
Container Code.	E	A	2	ZE	A2	12	G-5	LHE	P2	P67
Wet soil + Container(gm)	85.6	93.1	105.9	97.6	85.4	99.0	75.0	68.1	74.2	83.3
dry soil + container(gm)	75.3	80.9	89.1	82.3	70.2	84.4	59.8	56.8	58.2	69.7
container(gm)	37.9	37.0	34.6	33.0	25.2	41.2	17.4	25.4	17.6	35.5
moisture(gm)	10.4	12.2	16.8	15.4	15.2	14.5	15.3	11.3	16.1	13.6
Dry soil(gm)	37.4	43.9	54.5	49.2	45.0	43.2	42.4	31.4	40.6	34.2
Moisture content %	27.7	27.7	30.9	31.2	33.7	33.6	36.1	36.1	39.6	39.9
Avg. Moisture content %	**27.73617**		**31.03109**		**33.69133**		**36.06877**		**39.77272**	
Dry Density gm/cm3	**1.23165**		**1.29431**		**1.32663**		**1.31975**		**1.24765**	
			OMC (%)			**34.6**	**MDD (gm/cm^3)**			**1.329**



Sample Location: ACB					Additive Content: 6% BA + 94% soil			
Density Determination								
Test No.	1		2		3		4	
Mold + Wet soil(gm)	6218.4		6427.7		6595.5		6568.4	
Mold(gm)	2708		2708		2708		2708	
Wet Soil(gm)	3510.4		3719.7		3887.5		3860.4	
Mold cm^3	2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.65273		1.75127		1.83027		1.81751	
Moisture Content Determination								
Container Code.	T1	F	1	P2	P67	A13	5	D
Wet soil + Container(gm)	98.31	108.42	84.94	77.61	105.48	89.14	76.60	83.74
dry soil + container(gm)	90.31	97.06	74.09	67.72	92.25	79.74	65.41	72.37
container(gm)	37.64	36.33	18.40	17.61	35.52	36.52	17.54	29.60
moisture(gm)	8	11.36	10.85	9.89	13.23	9.4	11.19	11.37
Dry soil(gm)	52.67	60.73	55.69	50.11	56.73	43.22	47.87	42.77
Moisture content %	15.19	18.71	19.48	19.74	23.32	21.75	23.38	26.58
Avg. Moisture content %	**16.94733**		**19.60972**		**22.53509**		**24.97993**	
Dry Density gm/cm3	**1.41323**		**1.46415**		**1.49367**		**1.45424**	
	OMC (%)			**22.4**	**MDD (gm/cm^3)**			**1.494**

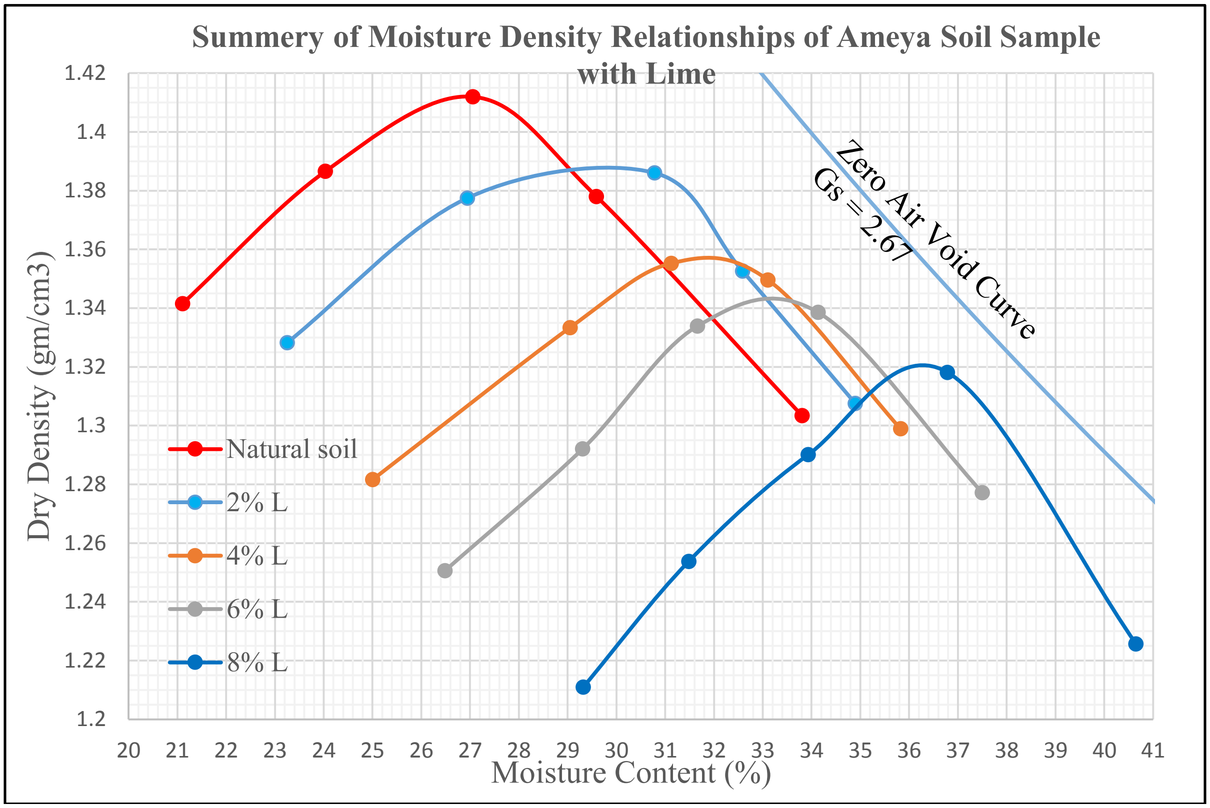


Sample Location: ACB				Additive Content: 6% BA + 3% Lime+ 91% soil				
Density Determination								
Test No.	1		2		3		4	
Mold + Wet soil(gm)	6144.3		6379.2		6545.1		6553.1	
Mold(gm)	2708		2708		2708		2708	
Wet Soil(gm)	3436.3		3671.2		3837.1		3845.1	
Mold cm^3	2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.61784		1.72844		1.80654		1.81031	
Moisture Content Determination								
Container Code.	3	T1	A2	J41	1	G-5	A1	P65
Wet soil + Container(gm)	89.22	99.26	95.87	98.13	86.26	85.80	93.61	106.73
dry soil + container(gm)	77.52	88.27	81.55	84.95	71.35	70.80	78.63	90.40
container(gm)	26.60	37.64	25.20	32.71	18.40	17.38	28.70	37.77
moisture(gm)	11.7	10.99	14.32	13.18	14.91	15	14.98	16.33
Dry soil(gm)	50.92	50.63	56.35	52.24	52.95	53.42	49.93	52.63
Moisture content %	22.98	21.71	25.41	25.23	28.16	28.08	30.00	31.03
Avg. Moisture content %	**22.34186**		**25.32115**		**28.11901**		**30.51497**	
Dry Density gm/cm3	**1.32240**		**1.37921**		**1.41005**		**1.38705**	
	OMC (%)			**28.2**	**MDD (gm/cm^3)**			**1.41**

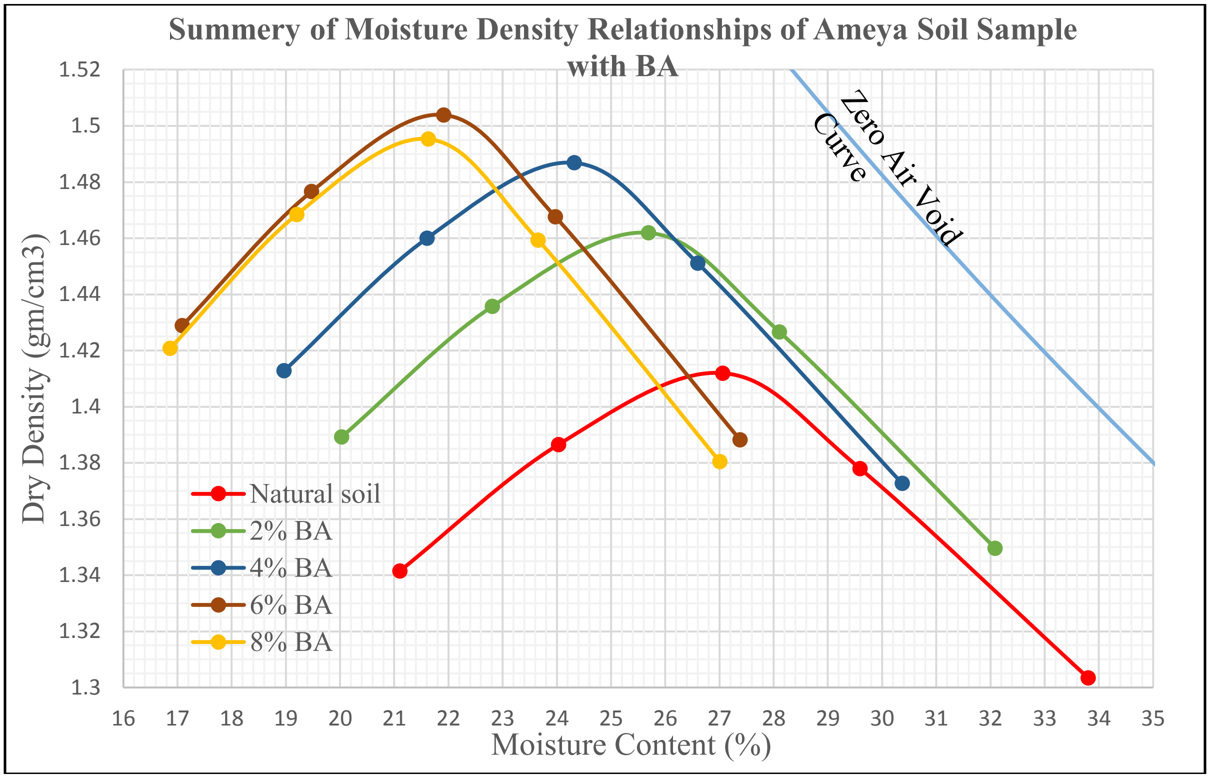


Sample Location: Ameya					**Natural soil of Ameya**					
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6174.40		6376.40		6534.10		6516.60		6427.90	
Mold(gm)	2723.60		2723.60		2723.60		2723.60		2723.60	
Wet Soil(gm)	3450.80		3652.80		3810.50		3793.00		3704.30	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.62467		1.71977		1.79402		1.78578		1.74402	
Moisture Content Determination										
Container Code.	P65	A2	A1	P2	A3	C2	A4	J41	G-5	3
Wet soil + Container(gm)	105.8	95.4	98.5	83.2	99.0	79.4	83.6	114.1	79.2	92.5
dry soil + container(gm)	93.9	83.2	84.9	70.5	85.0	66.2	68.6	95.4	63.6	75.9
container(gm)	37.8	25.2	28.7	17.6	32.8	17.6	17.3	32.7	17.4	26.6
moisture(gm)	11.9	12.2	13.6	12.7	14.1	13.2	15.0	18.7	15.7	16.6
Dry soil(gm)	56.1	58.0	56.2	52.9	52.2	48.6	51.3	62.7	46.2	49.3
Moisture content %	21.2	21.0	24.1	23.9	26.9	27.2	29.3	29.9	33.9	33.7
Avg. Moisture content %	**21.10539**		**24.03269**		**27.05850**		**29.59142**		**33.80409**	
Dry Density gm/cm3	**1.34153**		**1.38655**		**1.41196**		**1.37801**		**1.30341**	
			OMC (%)			**26.95**	**MDD (gm/cm^3)**			**1.4121**

Sample Location: Ameya					**Additive Content: 6% Lime + 94% soil**					
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6083.50		6272.30		6453.60		6537.10		6453.50	
Mold(gm)	2723.60		2723.60		2723.60		2723.60		2723.60	
Wet Soil(gm)	3359.9		3548.7		3730		3813.5		3729.9	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.58187		1.67076		1.75612		1.79543		1.75607	
Moisture Content Determination										
Container Code.	3	2	G-5	P2	ZE	C3	A4	LHE	190	A13
Wet soil + Container(gm)	92.5	99.0	87.2	88.1	103.3	97.8	81.5	93.4	101.8	94.0
dry soil + container(gm)	78.8	85.5	71.3	72.2	86.4	80.7	65.2	76.1	83.3	78.3
container(gm)	26.6	34.6	17.4	17.6	33.0	26.6	17.3	25.4	34.1	36.5
moisture(gm)	13.8	13.5	15.9	15.9	16.9	17.1	16.3	17.3	18.5	15.7
Dry soil(gm)	52.2	50.9	53.9	54.6	53.4	54.1	47.9	50.7	49.2	41.8
Moisture content %	26.4	26.5	29.4	29.2	31.6	31.7	34.1	34.2	37.6	37.4
Avg. Moisture content %	**26.48799**		**29.30815**		**31.65857**		**34.13411**		**37.50106**	
Dry Density gm/cm3	**1.25061**		**1.29208**		**1.33384**		**1.33854**		**1.27713**	
			OMC (%)			**33.25**	**MDD (gm/cm^3)**			**1.3433**



Sample Location:Ameya					Additive Content: 6% BA + 94% soil					
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6261.6		6455.2		6602.2		6572.5		6463.9	
Mold(gm)	2708		2708		2708		2708		2708	
Wet Soil(gm)	3553.6		3747.2		3894.2		3864.5		3755.9	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.67307		1.76422		1.83343		1.81944		1.76831	
Moisture Content Determination										
Container Code.	G-5	E	A	A2	12	2	P67	LHE	A13	ZE
Wet soil + Container(gm)	67.0	99.6	95.7	95.5	106.5	93.9	86.9	103.4	111.6	87.9
dry soil + container(gm)	59.9	90.4	86.0	84.1	95.0	83.1	77.3	87.9	95.4	76.1
container(gm)	17.4	37.9	37.0	25.2	41.2	34.6	35.5	25.4	36.5	33.0
moisture(gm)	7.1	9.2	9.6	11.4	11.6	10.8	9.7	15.5	16.2	11.7
Dry soil(gm)	42.6	52.5	49.0	58.9	53.7	48.5	41.8	62.5	58.9	43.1
Moisture content %	16.7	17.5	19.7	19.3	21.6	22.3	23.1	24.8	27.5	27.2
Avg. Moisture content %	**17.08799**		**19.47537**		**21.91242**		**23.97409**		**27.37862**	
Dry Density gm/cm3	**1.42890**		**1.47664**		**1.50389**		**1.46760**		**1.38823**	
			OMC (%)			**21.85**	**MDD (gm/cm^3)**			**1.504**

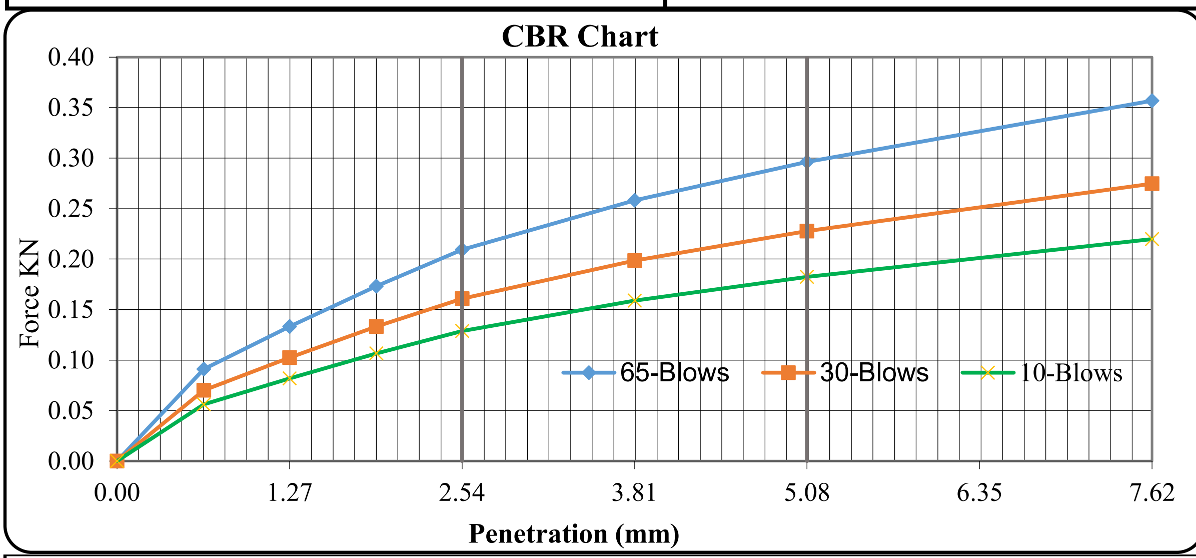


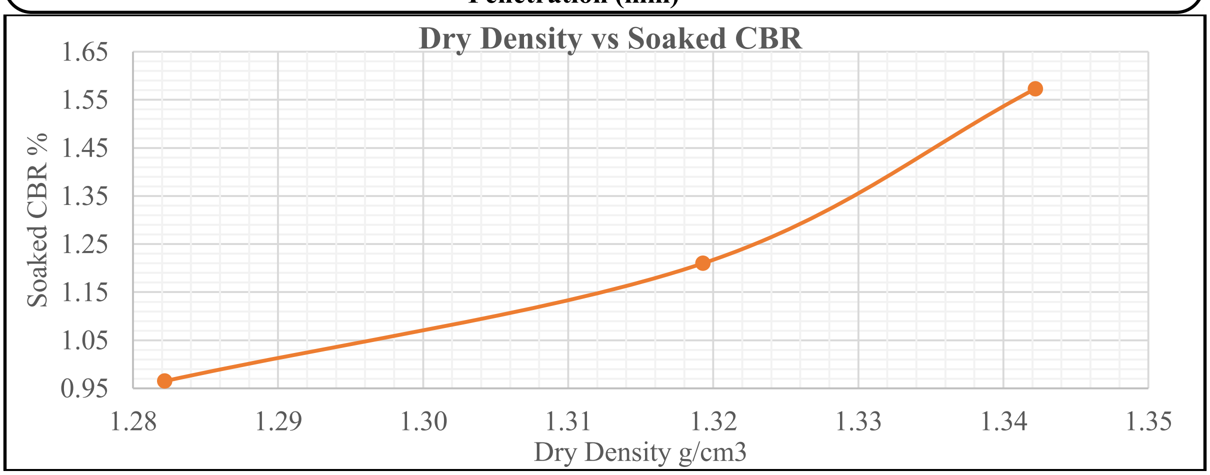
Sample Location: Ameya			Additive Content: 6% BA + 3% Lime+ 91% soil							
Density Determination										
Test No.	1		2		3		4		5	
Mold + Wet soil(gm)	6173.6		6365.5		6528.9		6553.9		6485.1	
Mold(gm)	2708		2708		2708		2708		2708	
Wet Soil(gm)	3465.6		3657.5		3820.9		3845.9		3777.1	
Mold cm^3	2124.00		2124.00		2124.00		2124.00		2124.00	
Bulk Density gm/cm^3	1.63164		1.72199		1.79892		1.81069		1.77830	
Moisture Content Determination										
Container Code.	A13	P65	J41	A1	3	A3	G-5	A2	1	A4
Wet soil + Container(gm)	101.3	103.7	97.3	93.3	90.4	96.0	84.5	91.1	86.2	81.0
dry soil + container(gm)	89.6	92.0	84.6	80.7	76.9	82.6	69.5	76.1	69.2	65.0
container(gm)	36.5	37.8	32.7	28.7	26.6	32.8	17.4	25.2	18.4	17.3
moisture(gm)	11.7	11.7	12.7	12.6	13.4	13.4	15.0	15.0	17.0	15.9
Dry soil(gm)	53.1	54.3	51.9	52.0	50.3	49.8	52.1	50.9	50.8	47.7
Moisture content %	22.0	21.6	24.5	24.3	26.7	26.9	28.7	29.4	33.5	33.4
Avg. Moisture content %	**21.78141**		**24.38889**		**26.79935**		**29.05511**		**33.45260**	
Dry Density gm/cm3	**1.33981**		**1.38436**		**1.41871**		**1.40303**		**1.33253**	
			OMC (%)			**27.1**	**MDD (gm/cm^3)**			**1.419**

APPENDIX G: SAMPLE CALIFORNIA BEARING RATIO (CBR) TEST ANALYSIS DATA

PENETRATION AND LOAD DETERMINATION									
NATURAL SOIL OF CHIDA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.091	0.133	0.173	0.209	0.258	0.296	0.357
	CBR (%)					**1.57**		**1.48**	
30-Blows	**Load (KN)**	0	0.070	0.102	0.133	0.161	0.199	0.228	0.275
	CBR (%)					**1.21**		**1.14**	
10-Blows	**Load (KN)**	0	0.056	0.082	0.107	0.129	0.159	0.182	0.220
	CBR (%)					**0.97**		**0.91**	

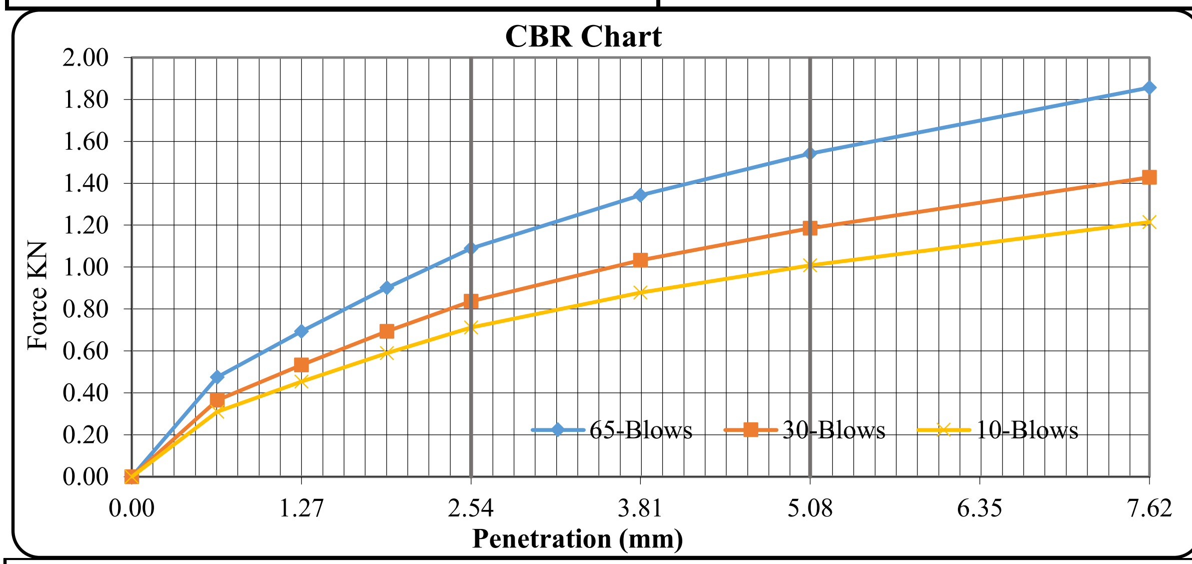
CBR RESULT SUMMARY			
OMC (%)	28.52		
MMDD	1.3924		
Dry Density at 95% of MDD	1.3228		
No of Blows	65	30	10
CBR Values (%)	1.57	1.21	0.97
DDBS g/cc	1.3422	1.3193	1.2822
CBR at 95% MDD	**1.265%**		

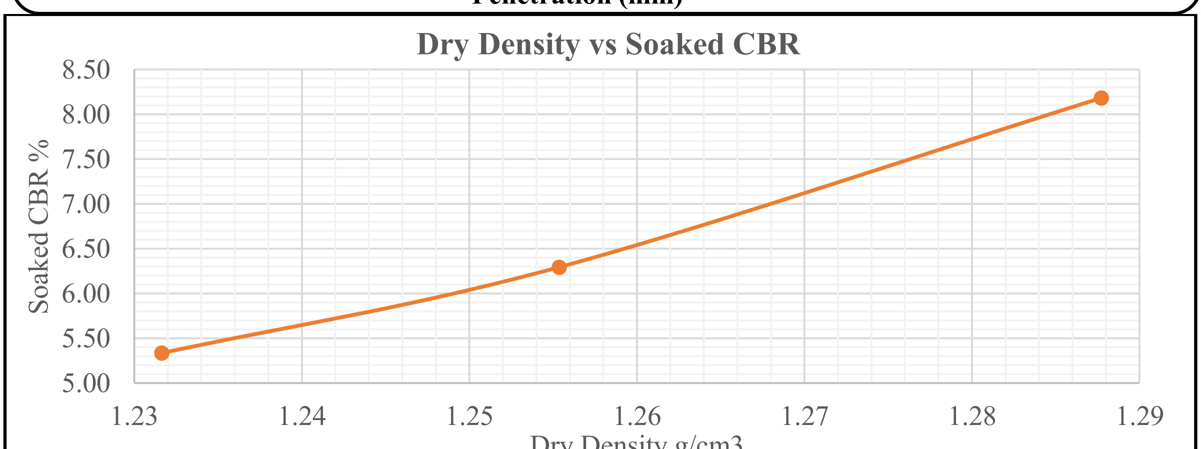




PENETRATION AND LOAD DETERMINATION									
6% LIME + 94% SOIL OF CHIDA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.474	0.693	0.901	1.088	1.343	1.541	1.857
	CBR (%)					**8.16**		**7.70**	
30-Blows	**Load (KN)**	0	0.365	0.533	0.693	0.837	1.033	1.185	1.428
	CBR (%)					**6.28**		**5.93**	
10-Blows	**Load (KN)**	0	0.310	0.453	0.589	0.712	0.878	1.007	1.214
	CBR (%)					**5.33**		**5.04**	

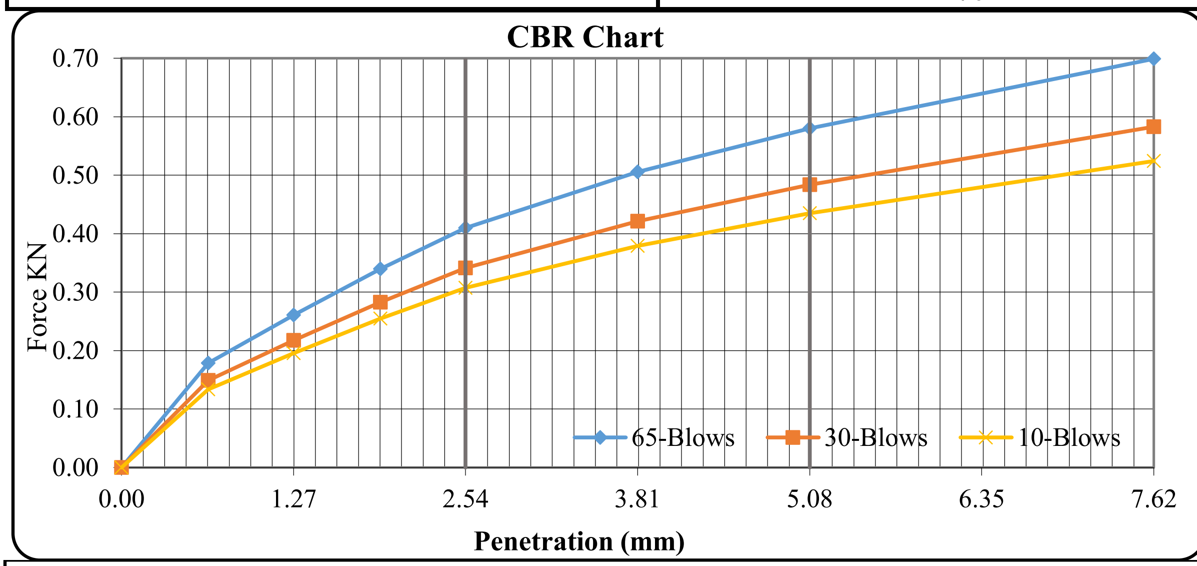
CBR RESULT SUMMARY			
OMC (%)	35.9		
MMDD	1.319		
Dry Density at 95% of MDD	1.2531		
No of Blows	65	30	10
CBR Values (%)	8.18	6.30	5.33
DDBS g/cc	1.2877	1.2554	1.2316
CBR at 95% MDD	**6.160%**		

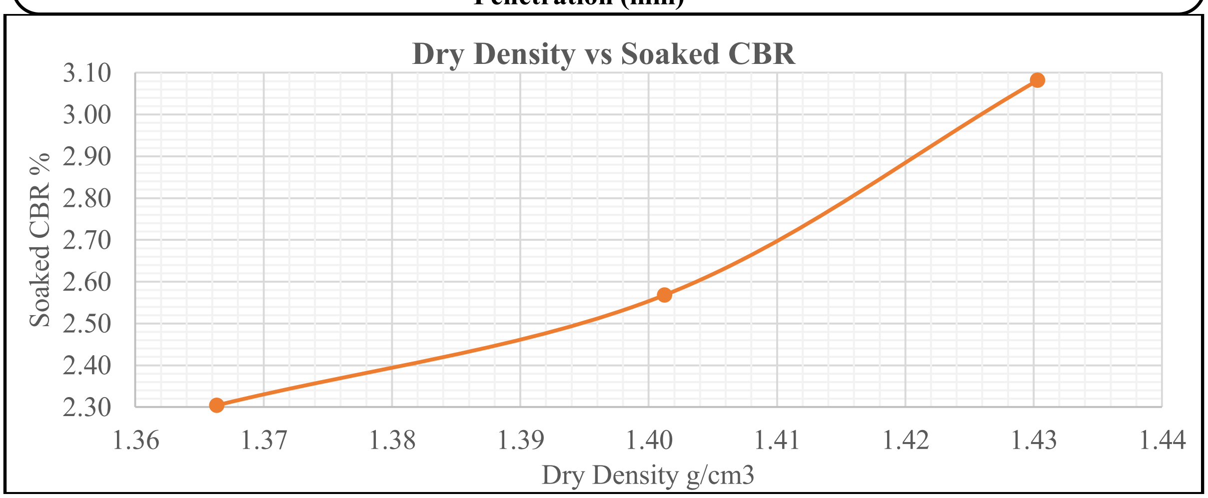




PENETRATION AND LOAD DETERMINATION									
6% BA + 94% SOIL OF CHIDA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.179	0.261	0.339	0.410	0.506	0.580	0.699
	CBR (%)					**3.07**		**2.90**	
30-Blows	**Load (KN)**	0	0.149	0.217	0.283	0.342	0.421	0.483	0.583
	CBR (%)					**2.56**		**2.42**	
10-Blows	**Load (KN)**	0	0.134	0.196	0.255	0.307	0.379	0.435	0.524
	CBR (%)					**2.30**		**2.18**	

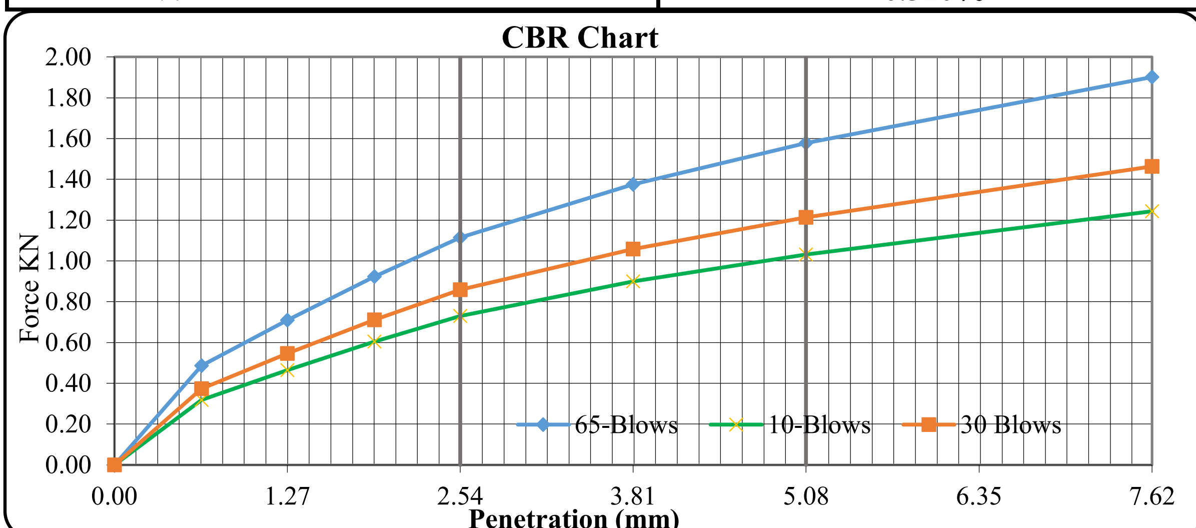
CBR RESULT SUMMARY			
OMC (%)	23		
MMDD	1.4838		
Dry Density at 95% of MDD	1.4096		
No of Blows	65	30	10
CBR Values (%)	3.08	2.57	2.30
DDBS g/cc	1.4303	1.4012	1.3664
CBR at 95% MDD	**2.716%**		

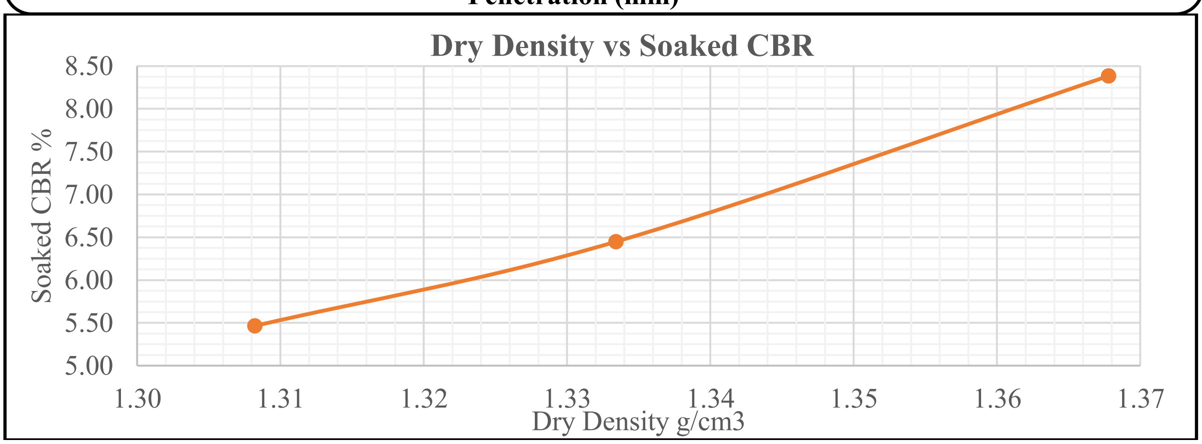




PENETRATION AND LOAD DETERMINATION									
6% BA +3% LIME+ 91% SOIL OF CHIDA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.486	0.709	0.923	1.115	1.376	1.578	1.902
	CBR (%)					**8.36**		**7.89**	
30-Blows	**Load (KN)**	0	0.374	0.546	0.710	0.858	1.058	1.214	1.463
	CBR (%)					**6.43**		**6.07**	
10-Blows	**Load (KN)**	0	0.318	0.464	0.604	0.729	0.900	1.032	1.244
	CBR (%)					**5.46**		**5.16**	

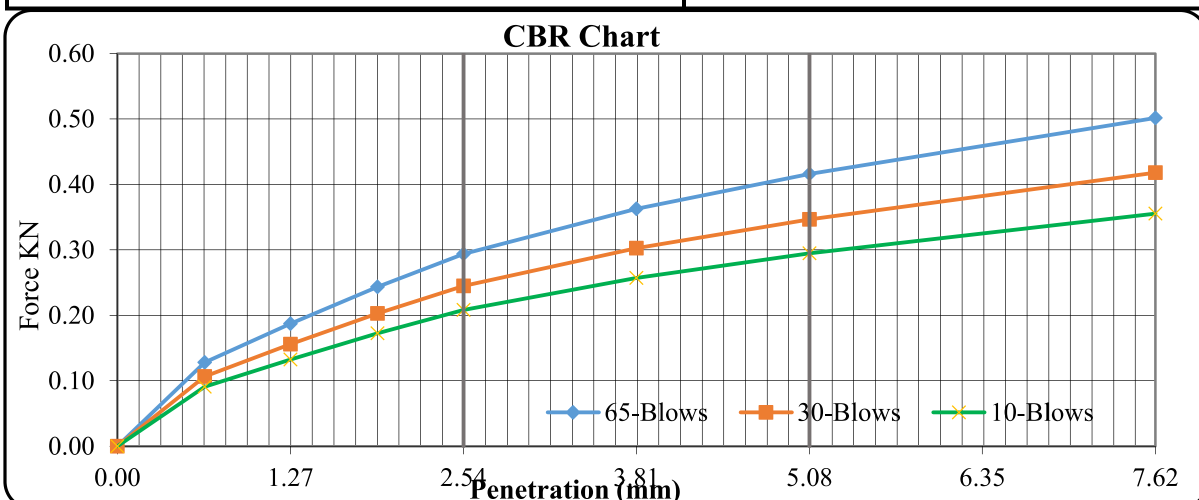
CBR RESULT SUMMARY			
OMC (%)	29.4		
MMDD	1.401		
Dry Density at 95% of MDD	1.3310		
No of Blows	65	30	10
CBR Values (%)	8.38	6.45	5.46
DDBS g/cc	1.3678	1.3334	1.3082
CBR at 95% MDD	**6.310%**		

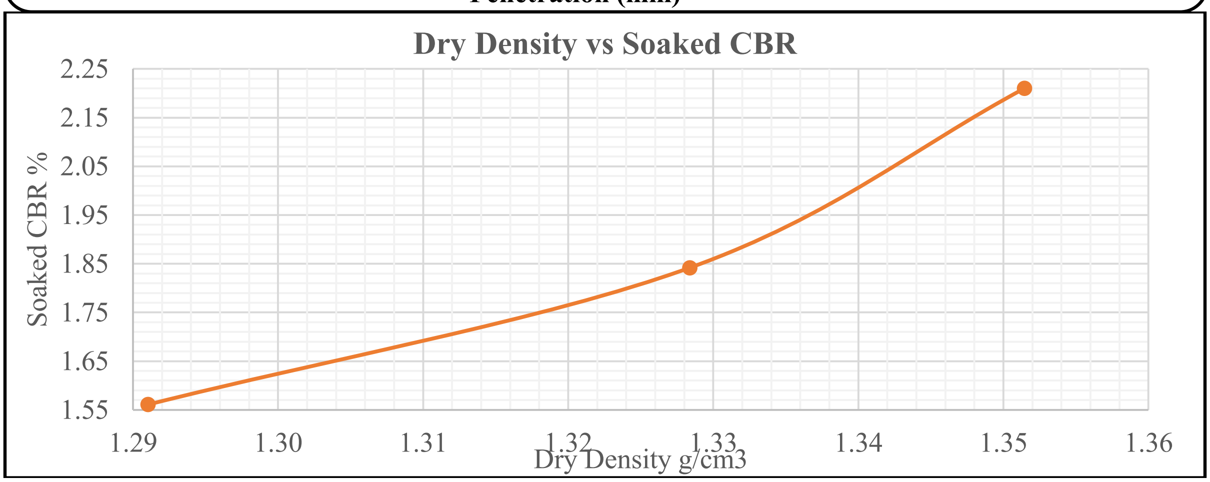




PENETRATION AND LOAD DETERMINATION									
NATURAL SOIL OF ACB									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0.000	0.128	0.187	0.243	0.294	0.363	0.416	0.501
	CBR (%)					**2.204**		**2.081**	
30-Blows	**Load (KN)**	0.000	0.107	0.156	0.203	0.245	0.302	0.347	0.418
	CBR (%)					**1.836**		**1.734**	
10-Blows	**Load (KN)**	0.000	0.091	0.132	0.172	0.208	0.257	0.295	0.355
	CBR (%)					**1.561**		**1.474**	

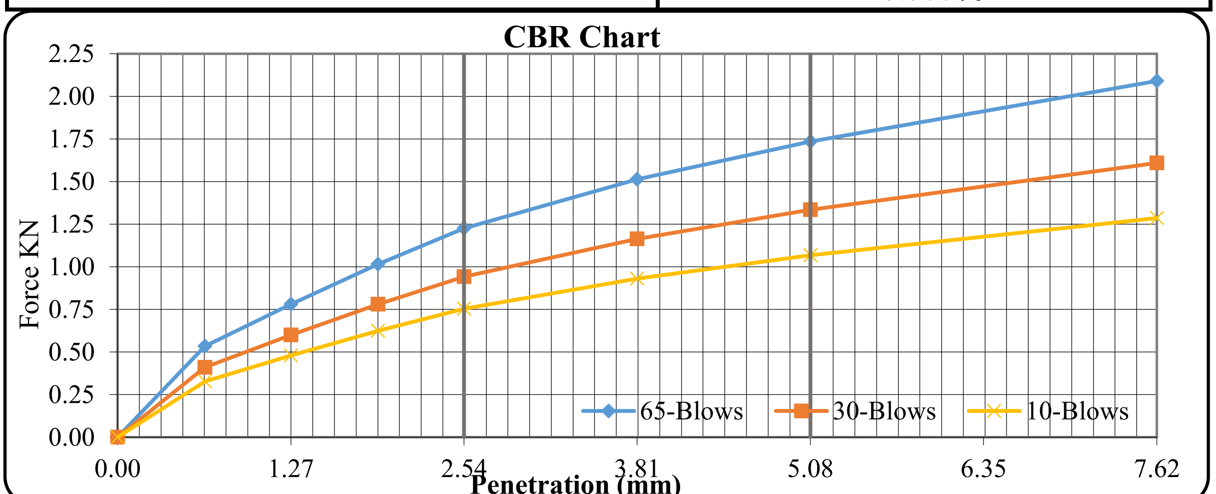
CBR RESULT SUMMARY			
OMC (%)	27.75		
MMDD	1.402		
Dry Density at 95% of MDD	1.3319		
No of Blows	65	30	10
CBR Values (%)	2.21	1.84	1.56
DDBS g/cc	1.3515	1.3284	1.2910
CBR at 95% MDD	**1.898%**		

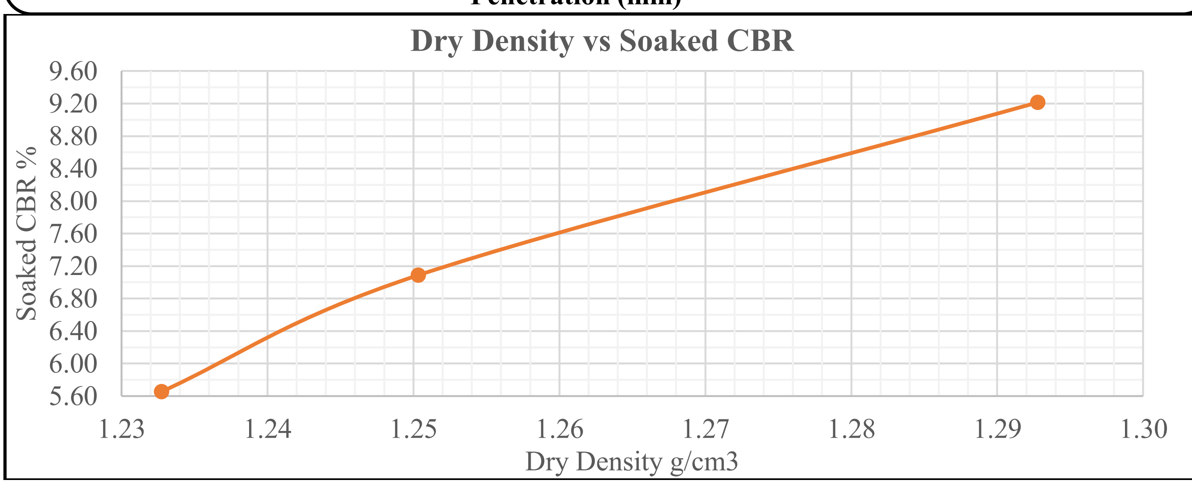




PENETRATION AND LOAD DETERMINATION									
6% LIME + 94% SOIL OF ACB									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.534	0.780	1.015	1.226	1.512	1.735	2.091
	CBR (%)					**9.19**		**8.67**	
30-Blows	**Load (KN)**	0	0.411	0.600	0.781	0.943	1.163	1.334	1.608
	CBR (%)					**7.07**		**6.67**	
10-Blows	**Load (KN)**	0	0.328	0.480	0.625	0.754	0.931	1.068	1.287
	CBR (%)					**5.65**		**5.34**	

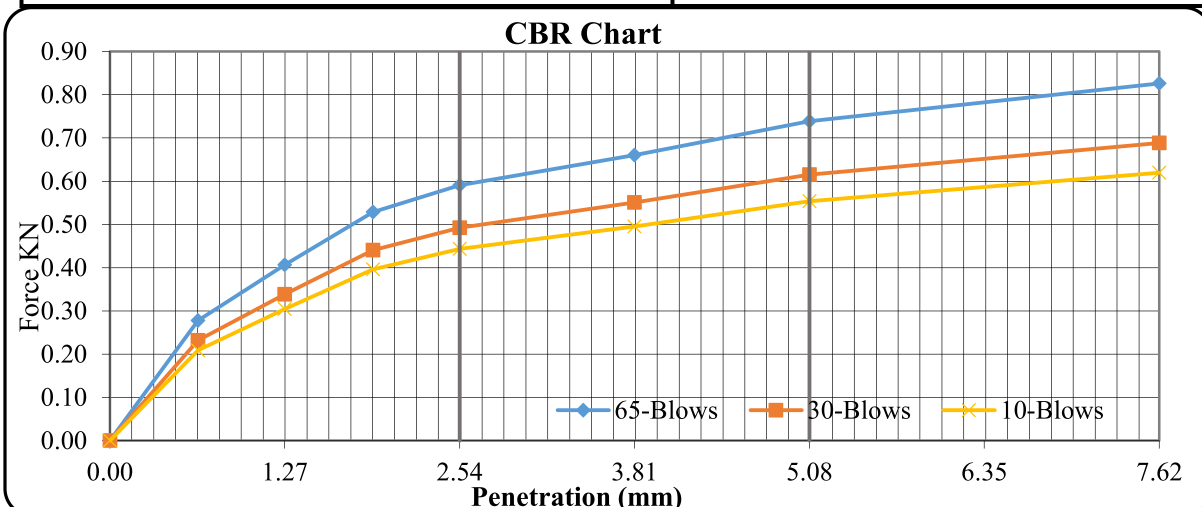
CBR RESULT SUMMARY			
OMC (%)	34.6		
MMDD	1.329		
Dry Density at 95% of MDD	1.2626		
No of Blows	65	30	10
CBR Values (%)	9.21	7.09	5.65
DDBS g/cc	1.2928	1.2503	1.2327
CBR at 95% MDD	**7.700%**		

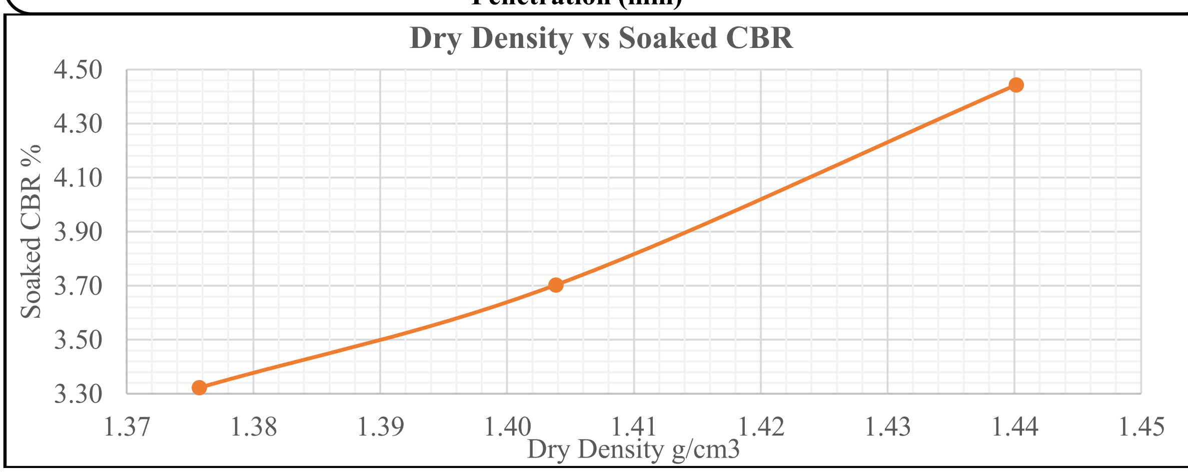




PENETRATION AND LOAD DETERMINATION										
6% BA + 94% SOIL OF ACB										
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.278	0.406	0.529	0.591	0.661	0.739	0.826
	CBR (%)					**4.43**		**3.69**	
30-Blows	**Load (KN)**	0	0.232	0.338	0.440	0.492	0.551	0.616	0.688
	CBR (%)					**3.69**		**3.08**	
10-Blows	**Load (KN)**	0	0.209	0.305	0.396	0.443	0.496	0.554	0.619
	CBR (%)					**3.32**		**2.77**	

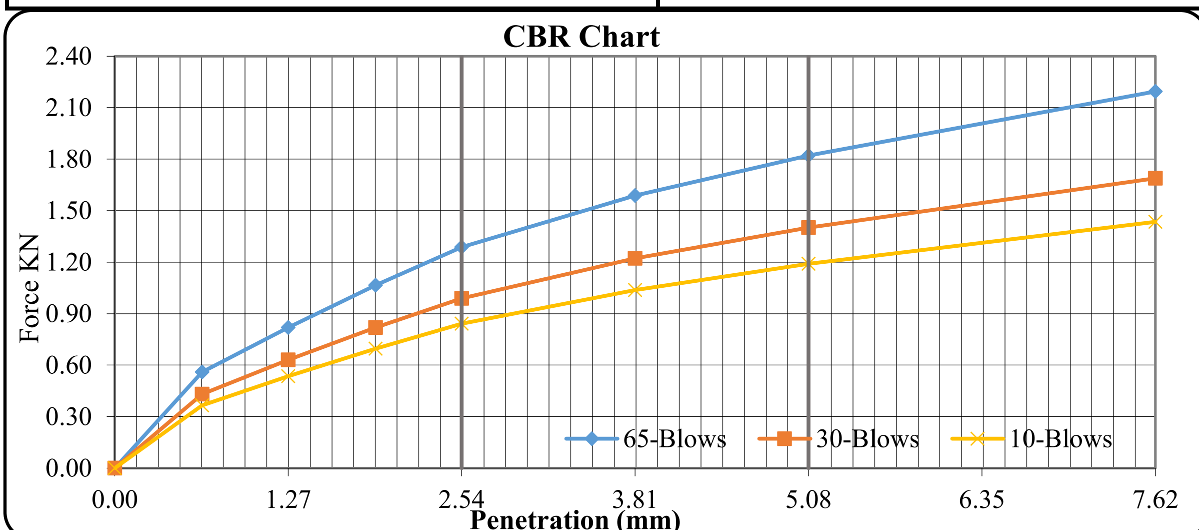
CBR RESULT SUMMARY			
OMC (%)	22.4		
MMDD	1.494		
Dry Density at 95% of MDD	1.4193		
No of Blows	65	30	10
CBR Values (%)	4.44	3.70	3.32
DDBS g/cc	1.4401	1.4038	1.3757
CBR at 95% MDD	**4.018%**		

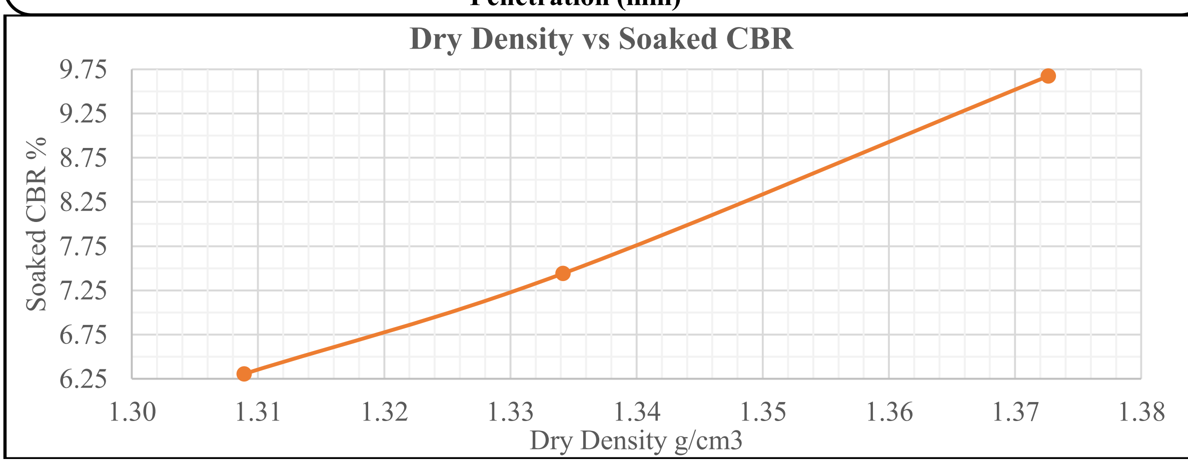




PENETRATION AND LOAD DETERMINATION									
6% BA + 3% LIME + 91% SOIL OF ACB									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.560	0.819	1.065	1.287	1.588	1.821	2.195
	CBR (%)					**9.64**		**9.11**	
30-Blows	**Load (KN)**	0	0.431	0.630	0.820	0.990	1.221	1.401	1.688
	CBR (%)					**7.42**		**7.00**	
10-Blows	**Load (KN)**	0	0.366	0.535	0.697	0.841	1.038	1.191	1.435
	CBR (%)					**6.31**		**5.95**	

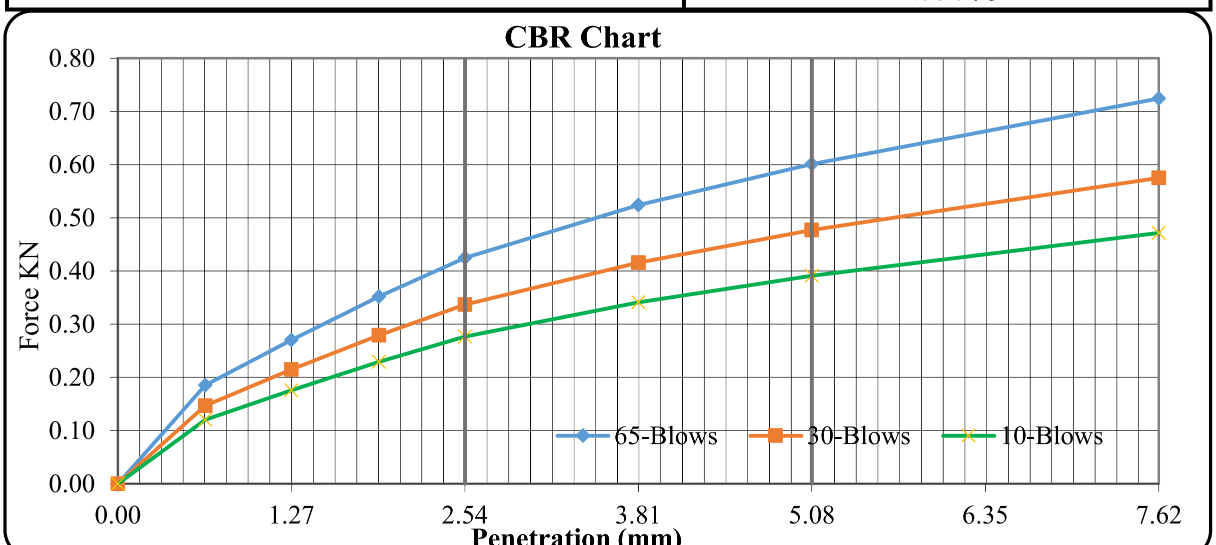
CBR RESULT SUMMARY			
OMC (%)	28.2		
MMDD	1.41		
Dry Density at 95% of MDD	1.3395		
No of Blows	65	30	10
CBR Values (%)	9.67	7.44	6.31
DDBS g/cc	1.3726	1.3342	1.3089
CBR at 95% MDD	**7.750%**		

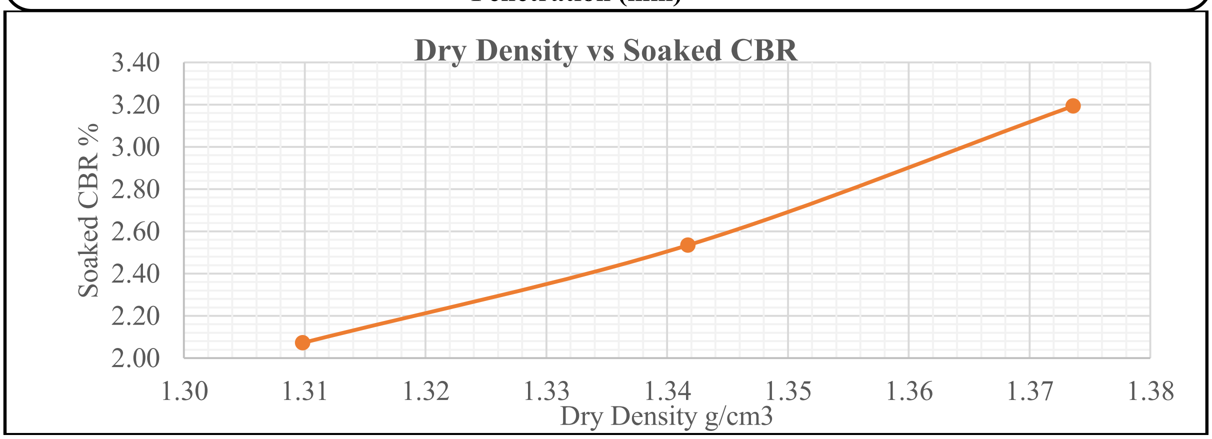




PENETRATION AND LOAD DETERMINATION									
NATURA SOIL OF AMEYA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0.000	0.185	0.270	0.352	0.425	0.524	0.601	0.725
	CBR (%)					**3.184**		**3.006**	
30-Blows	**Load (KN)**	0.000	0.147	0.214	0.279	0.337	0.416	0.477	0.575
	CBR (%)					**2.527**		**2.386**	
10-Blows	**Load (KN)**	0.000	0.120	0.176	0.229	0.276	0.341	0.391	0.472
	CBR (%)					**2.072**		**1.956**	

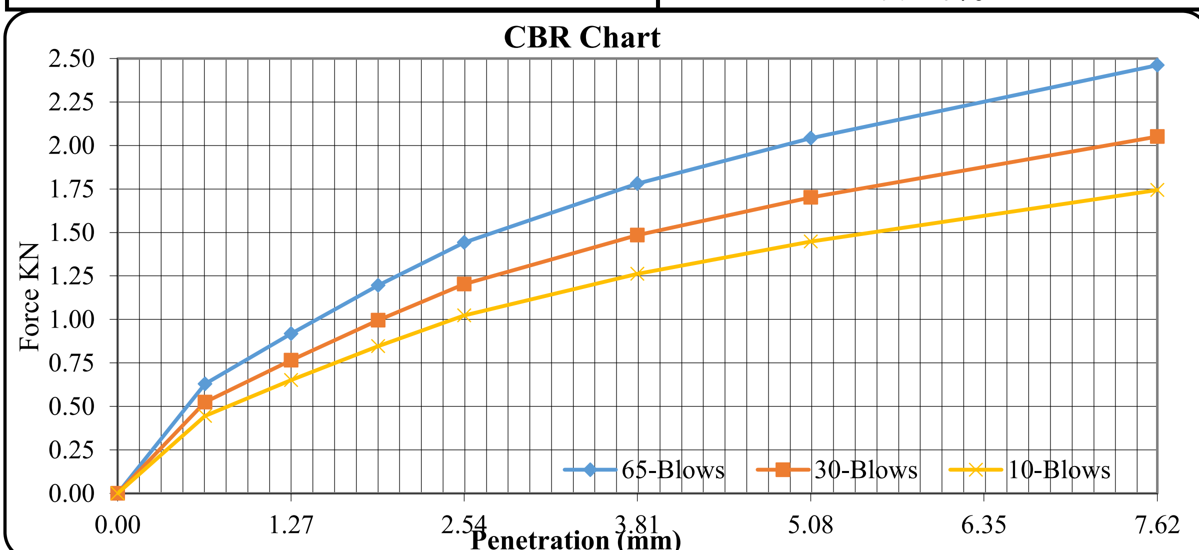
CBR RESULT SUMMARY			
OMC (%)	26.95		
MMDD	1.4121		
Dry Density at 95% of MDD	1.3415		
No of Blows	65	30	10
CBR Values (%)	3.19	2.53	2.07
DDBS g/cc	1.3736	1.3417	1.3098
CBR at 95% MDD	**2.530%**		

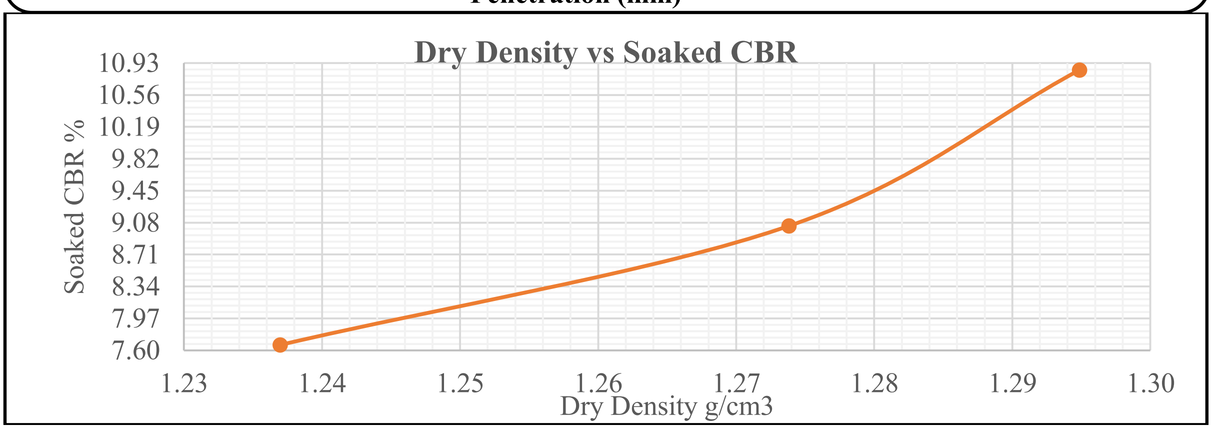




PENETRATION AND LOAD DETERMINATION										
6% LIME + 94% SOIL OF AMEYA										
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.628	0.918	1.195	1.443	1.781	2.043	2.462
	CBR (%)					**10.82**		**10.21**	
30-Blows	**Load (KN)**	0	0.524	0.765	0.996	1.202	1.484	1.702	2.051
	CBR (%)					**9.01**		**8.51**	
10-Blows	**Load (KN)**	0	0.445	0.650	0.846	1.022	1.261	1.447	1.744
	CBR (%)					**7.66**		**7.23**	

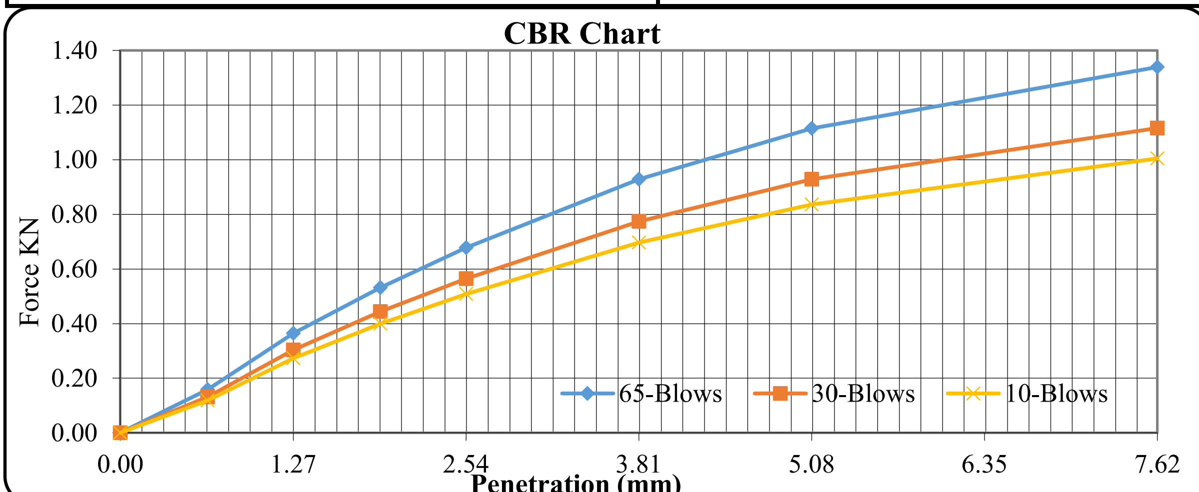
CBR RESULT SUMMARY			
OMC (%)	33.25		
MMDD	1.3433		
Dry Density at 95% of MDD	1.2761		
No of Blows	65	30	10
CBR Values (%)	10.85	9.04	7.66
DDBS g/cc	1.2949	1.2738	1.2370
CBR at 95% MDD	**9.240%**		

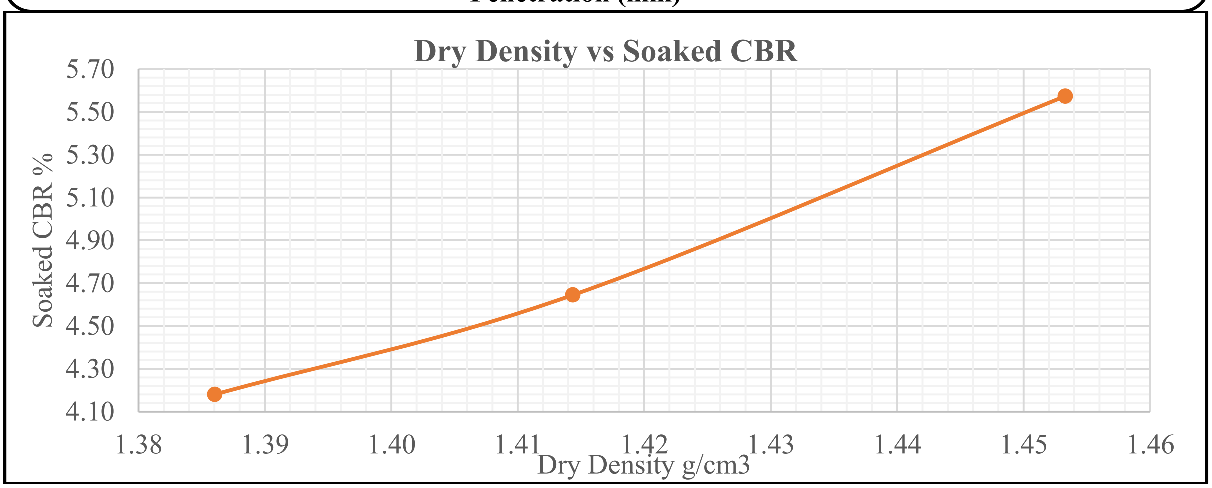




PENETRATION AND LOAD DETERMINATION									
6% BA + 94% SOIL OF AMEYA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.158	0.364	0.532	0.678	0.929	1.115	1.339
	CBR (%)					**5.08**		**5.57**	
30-Blows	**Load (KN)**	0	0.132	0.303	0.443	0.565	0.774	0.929	1.116
	CBR (%)					**4.23**		**4.64**	
10-Blows	**Load (KN)**	0	0.119	0.273	0.399	0.508	0.697	0.836	1.004
	CBR (%)					**3.81**		**4.18**	

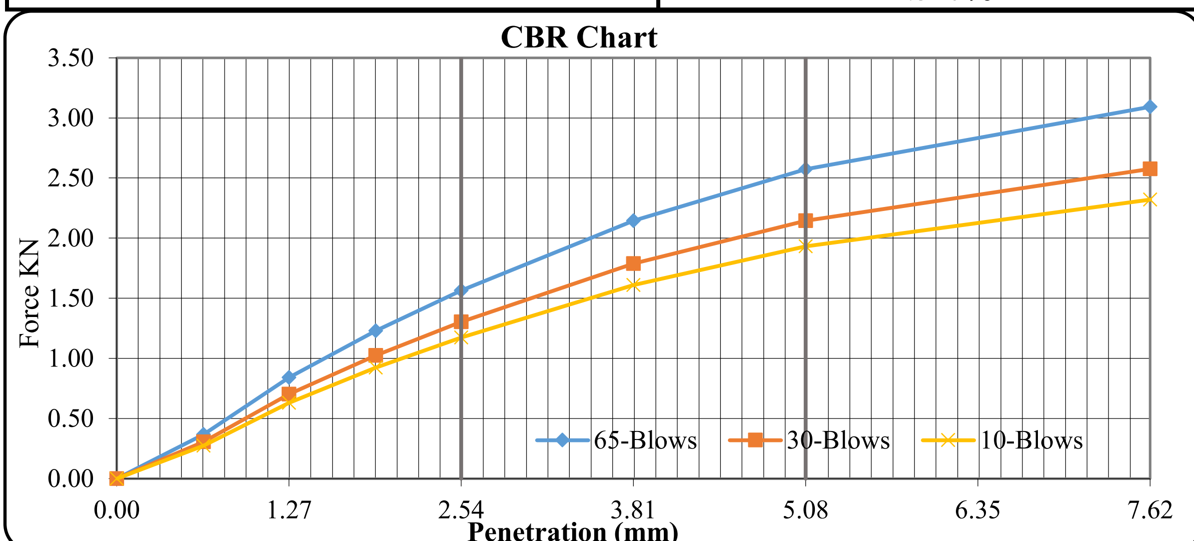
CBR RESULT SUMMARY			
OMC (%)	21.85		
MMDD	1.504		
Dry Density at 95% of MDD	1.4288		
No of Blows	65	30	10
CBR Values (%)	5.57	4.64	4.18
DDBS g/cc	1.4533	1.4143	1.3860
CBR at 95% MDD	**4.989%**		

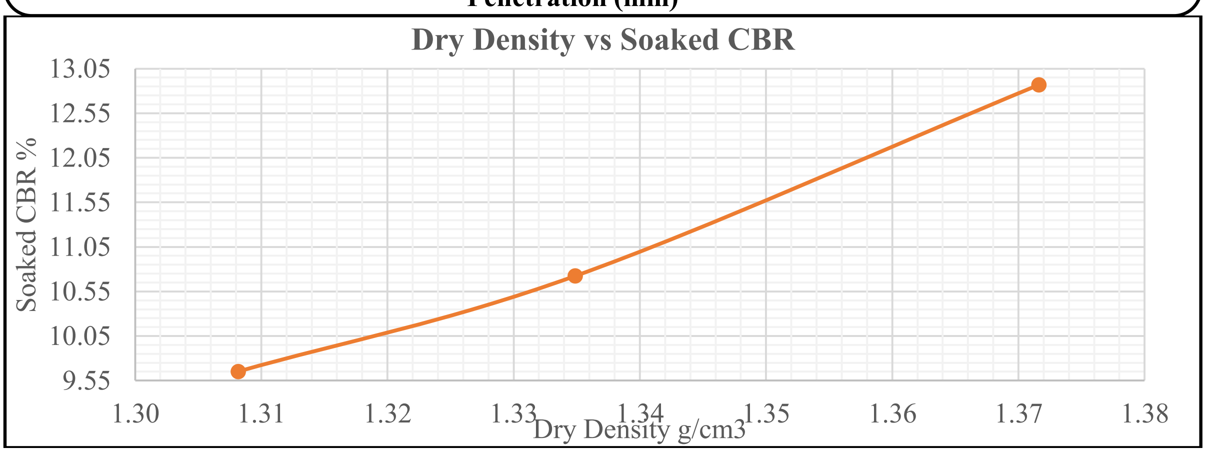




PENETRATION AND LOAD DETERMINATION									
6% BA +3% LIME+ 91% SOIL OF AMEYA									
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.365	0.840	1.229	1.564	2.145	2.574	3.092
	CBR (%)					**11.73**		**12.87**	
30-Blows	**Load (KN)**	0	0.304	0.700	1.024	1.304	1.787	2.145	2.577
	CBR (%)					**9.77**		**10.72**	
10-Blows	**Load (KN)**	0	0.274	0.630	0.921	1.173	1.609	1.930	2.319
	CBR (%)					**8.80**		**9.65**	

CBR RESULT SUMMARY			
OMC (%)	27.1		
MMDD	1.4195		
Dry Density at 95% of MDD	1.3485		
No of Blows	65	30	10
CBR Values (%)	12.87	10.72	9.65
DDBS g/cc	1.3716	1.3349	1.3082
CBR at 95% MDD	**11.520%**		





PENETRATION AND LOAD DETERMINATION										
6% BA +4% LIME + 90% SOIL OF AMEYA										
Penetration	**(mm)**	0	0.64	1.27	1.91	**2.54**	3.81	**5.08**	7.62
65-Blows	**Load (KN)**	0	0.869	1.270	1.652	1.995	2.462	2.824	3.404
	CBR (%)					**14.96**		**14.12**	
30-Blows	**Load (KN)**	0	0.724	1.058	1.377	1.663	2.052	2.354	2.836
	CBR (%)					**12.46**		**11.77**	
10-Blows	**Load (KN)**	0	0.616	0.899	1.170	1.413	1.744	2.001	2.411
	CBR (%)					**10.59**		**10.00**	

CBR RESULT SUMMARY			
OMC (%)	29		
MMDD	1.4015		
Dry Density at 95% of MDD	1.3314		
No of Blows	65	30	10
CBR Values (%)	15.00	12.50	10.59
DDBS g/cc	1.3510	1.3125	1.2906
CBR at 95% MDD	**13.730%**