2023-03

STABILIZATION OF BLACK COTTON SOILWITH SYNTHETIC CLOTH WASTE ASH AND LIME IN NEFAS MEWUCHA TOWN

ETSEGENET, TAMIRAT DESALEGN

BAHIR DAR UNIVERSITY

BAHIR DAR INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND WATER RESOURCE ENGINEERING

MASTERS OF SCIENCE THESIS ON

STABILIZATION OF BLACK COTTON SOILWITH SYNTHETIC CLOTH WASTE ASH AND LIME IN NEFAS MEWUCHA TOWN

BY

ETSEGENET TAMIRAT DESALEGN

March, 2023

Bahir Dar, Ethiopia

BAHIR DAR UNIVERSITY

BAHIR DAR INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND WATER RESOURCE ENGINEERING

STABILIZATION OF BLACK COTTON SOILWITH SYNTHETIC CLOTH WASTE ASH AND LIME IN NEFAS MEWUCHATOWN

BY

ETSEGENET TAMIRAT DESALEGN

A thesis submitted to the school of Research and Graduate studies of Bahir Dar Institute of Technology, BDU in partial fulfillment of the requirements for the degree of masters of science in Geotechnical Engineering

Advisor: Yebeltal Zerie (PhD)

March, 2023

Bahir Dar, Ethiopia

DECLARATION

This is to certify that the thesis entitled "Stabilization of black cotton soil with synthetic cloth waste ash and lime in Nefas Mewucha town", submitted in partial fulfillment of the requirements for the degree of Master of Science in Geotechnical Engineering under faculty of Civil and water resource Engineering, Bahir Dar Institute of Technology, is a record of original work carried out by me and has never been submitted to this or any other institution to get any other degree or certificates. The assistance and help I received during the course of this investigation have been duly acknowledged.

EtsegenetTamirat ________________	________	03/06/2023
Name of the candidate	signature	Date

BAHIR DAR UNIVERSITY

BAHIR DAR INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND WATER RESOURCE ENGINEERING

Approval of thesis for defense result

I hereby confirm that the changes required by the examiners have been carried out and incorporated in the final thesis.

Name of student: Etsegenet Tamirat Desalegn Signature_____ Date 03/06/2023

As a member of the board of examiners, we examined this entitled "Stabilization of black cotton soil with synthetic cloth waste ash and lime in Nefas Mewucha town" by Etsegenet Tamirat Desalegn. We hereby certify that the thesis is accepting for fulfilling the requirements for the award of the degree of Masters of science in "Geotechnical Engineering".

Board of Examiners

Name of Advisor: Dr. Yebeltal Zerie Beyene	Signature	Date 26/5/2023
Name of External Examiner: Dr. Tezera Firew Azmatch	Signature	Date March 28, 2023
Name of Internal Examiner: Dr. Elyas Assefa	Signature	Date March 30, 2023
Name of Chair Person: Gizachew Ayehualem	Signature	Date 02/06/2023
Name of Chair Holder: Fekadu Mulugeta	Signature	Date 02/06/2023
Name of Faculty Dean: Mitiku Damtie Yehualaw (PhD) Faculty Dean	Signature	Date 03/06/2023

Bahir Dar Institute of Technology Faculty of Civil and Water Resources Engineering

ACKNOWLEDGEMENTS

First of all, I would like to thank the omniscient and almighty God for everything that he did for me.

My deepest gratitude goes to my advisor Dr. Yebeltal Zerie for his professional and genuine guidance and valuable advice for my accomplishment. I am also grateful to thank all laboratory staffs of Bahir Dar institute of Technology for their genuine co-operation; without their collaboration, it would have been difficult to conduct the entire laboratory program. In the meantime, I would like to express my heartfelt thanks to all the people who helped me throughout my study.

ABSTRACT

Any civil engineering construction needs soil as its fundamental basis and the stability of these structures depends greatly on the quality of the underlying soils. Poorly engineered soils are difficult to build on and can cause serious issues for the structure that has been constructed on them. Treating the sub-grade soil is a very helpful procedure to prevent these issues. The process of stabilizing soil involves changing the characteristics of problematic soil using chemical or mechanical means in order to enhance its engineering property. Nowadays, using alternative materials with superior engineering quality has received a lot of attention. On the other hand, our nation has been producing more waste on a yearly basis. Reusing these wastes is one of the most alluring choices for managing them, and stabilizing soil with waste materials is particularly cost-effective because it supplies inexpensive and reusable materials. Effective use can be made of local resources. The purpose of this thesis is to examine the impact of adding various amounts of synthetic cloth waste ash and lime to black cotton soil found in Nefas Mewucha town. This study is done on the basis of experimental findings. The black cotton soil is treated by mixing lime and synthetic cloth waste ash by dry weight of soil with 2% and 4% of lime and 5%, 10% and 15% of synthetic cloth waste ash. Specific gravity, the Atterberg limit, free swell, compaction, the California bearing ratio (CBR), and unconfined compressive strength tests are performed in the laboratory and interpretations of test results are given based on different references. Finally, conclusions are made and recommendations for further studies are stated in detail. In addition, optimum amount of stabilizers is determined. Based on the laboratory tests, this study investigates that the combination of SCWA and Lime shows significant effect on engineering properties of the black cotton soil

Key words: *Black Cotton soil, Soil Stabilization, Synthetic cloth waste ash, Lime.*

TABLE OF CONTENTS

LIST OF ABBREVIATIONS

CBR	California Bearing Ratio
SCWA	Synthetic Cloth Waste Ash
L	Lime
BC	Black Cotton
FSI	Free Swell Index
UCS	Unconfined Compressive Strength
ASTM	American Society for Testing and Materials
USCS	Unified Soil Classification System
AASHTO	American Association of State Highway and Transportation Officials
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
CH	High plastic inorganic clay
PL	Plastic Limit
LL	Liquid Limit
PI	Plasticity Index
ERA	Ethiopian Roads Authority

LIST OF SYMBOLS

e	Axial Strain
Gs	Specific Gravity
L	Specimen deformation
L_o	Length of sample
q_u	Unconfined compressive strength
ρ_d	Dry density
Cu	Undrained shear strength
ρ	Wet Density

LIST OF FIGURES

LIST OF TABLES

CHAPTER ONE: INTRODUCTION

1.1. Background of the study

Expansive soils are those that can shrink and swell in response to changes in their moisture content, changing their volume. They typically contain an expansive clay mineral, such as smectite or vermiculite, which can absorb water, expands and increasing in volume when wet and contract when dry. Their volume grows as they take in more water(Chen, 1975).

Expansive soils are special types of clay soil that typically contains a preponderant amount of montmorillonite minerals. The qualities of these minerals largely determine how these soils expand and contract.

Volumetric variations of expansive soils make them unsuitable for construction activity since changes in water content make them abnormally swell and contract. This property of the soil is brought on by the existence of tiny clay particles, which, when in contact with water, expand and contract alternately, causing differential settlement of the structure.

(Chen, 1975)claimed that expansive soils evoked "black cotton soil" in arid regions of the world. Black cotton soils (BC soils) are mostly clayey soils with a grey to blackish appearance and a high amount of the expanding clay mineral montmorillonite. BC soils have high optimum moisture content and a low shrinkage limit. It is quite susceptible to changes in moisture and collapsible low-quality material. In Amhara Region, Nefas Mewucha town, there are certain locations covered with black cotton soils. These soils pose concern since they experience significant volume variations when wet and dry. Unless addressed and treated, this volume change causes extensive damage to structures and roads. It has been the geotechnical engineers' great responsibility to comprehend the properties of expansive soils and select the best method to enhance their engineering properties.

By stabilizing the soil through controlled compaction, proportioning and adding appropriate admixtures, the stability and bearing capacity of the soil are significantly increased. For the stabilization of black cotton soils, several studies used various kinds of organic, non-organic, expensive, and low-cost materials that were readily available.

A technique called soil stabilization aims to improve the engineering qualities of soil by chemically altering it through increasing or maintaining the stability of the soil mass. It is accomplished by using a variety of techniques, including the addition of fly ash, rice husk ash, chemicals, fibers, and other geo-materials like geo-synthetic, geo-grid, and geo-foam. Engineers can distribute a heavier load with less material over a longer life cycle by stabilizing the soil. This thesis utilizes SCWA and lime as stabilizers in order to study their impacts on the characteristics of black cotton soil.

1.2. Statement of the problem

Construction is essential to a nation's ability to develop economically, given its population and economic activity patterns. Buildings, roads, pipelines, airports, and other structures are seriously at risk from expansive soils both before and after construction. In rare instances, the extent of the harm could surpass those caused by hurricanes, tornadoes, and floods. Engineering infrastructures built on black cotton soils may experience damages ranging from small upkeep issues to significant aesthetic and structural problems. Additionally, rather than being concentrated in a small region, they produce property damages that are dispersed over larger areas. However, the economic damage brought on by expansive soils is substantial, and most of it might be avoided by properly identifying the issue and treating with the necessary preventive measures into the design, building, and upkeep of new structures.

In order to adequately assess the viability of such soils for hosting engineering projects, identification and quantification of the soil parameters that indicate shrink-swell potentials are crucial when development moves into expansive soil areas.

Stabilizing soil with synthetic cloth waste ash (SCWA) with lime is not a common practice in Ethiopia. However, both conventional and unconventional stabilizers have been developed and used in buildings. Numerous experts have conducted studies on a global scale to assess the suitability of various stabilizers, particularly those which expand soil stabilization.

The effectiveness of locally manufactured SCWA with lime stabilizer for soil stabilization, however, has not been studied in Ethiopia. Therefore, the purpose of this study is to examine the characteristics of these materials as soil stabilizers and assess the impact of their use.

1.3. Objectives of the study

1.3.1. General objective

The major goal of this research is to use locally accessible synthetic cloth waste ash and lime to reduce the expansiveness, swelling, and shrinkage issues of black cotton soil.

1.3.2. Specific objectives

- To study the engineering properties of black cotton soils.
- Analyzing the reactions of SCWA and lime with black cotton soil in various laboratory experiments.
- Determining whether mixing synthetic textile waste ash with lime improves the engineering behaviors of black cotton soils.
- Determining the optimum dosage of stabilizers required to enhance the characteristics of black cotton soil.

1.4. Scope of the study

This study basically focuses on the stabilization of black cotton soil which is problematic in nature using Synthetic cloth waste ash with lime in South Gondar zone, Nefas Mewucha district. Thus, the study is supported by series of laboratory experiments.

1.5. Significance of the study

It is recognized that several procedures, such as chemical and mechanical stabilization, can be used to enhance the engineering features of expansive soils, but these techniques are not cost-effective for our nation. Therefore, in order to solve this issue, readily available and inexpensive materials should be utilized for stabilization.

Additionally, this study will be used as an input for additional geotechnical researchers to fill the gap in the study and investigation of soil in the construction industry since the primary goal of soil stabilization in many engineering applications is to maintain the safety and economic viability of the structure.

1.6. Limitations of the study

While doing this study I have faced different challenges. These challenges were both theoretical and practical. Theoretically, it was difficult to get locally written literatures regarding the utilization of synthetic cloth waste as stabilizing material of black cotton soil and practically as some wastes of fiber were burned without blast furnace which made manipulation of the impact of intervening environmental variables a little bit difficult.

CHAPTER TWO: REVIEW OF LITERATURE

2.1. Review on expansive soils

Globally, expansive soils are an issue. Such soils are regarded as natural dangers that present difficulties for civil engineers, building companies, and owners. Buildings have occasionally been erected in less developed nations despite the presence of expansive soils. This was partially brought on by a dearth of historical evidence. Rapid urban infrastructure growth has made widespread soil issues increasingly noticeable. Therefore, it is necessary to address the issues related to these soils. Although expansive soils can be found all over the world, they are more common in arid and semi-arid areas. Since evaporation rates in these areas are higher than yearly rainfall, the soil is virtually never sufficiently moist. In soils with the ability to swell, adding water will result in ground heaving. Short bursts of rain are followed by protracted draughts, which cause cyclic swelling and shrinking phenomena in semi-arid environments. The type of material, type and quantity of clay minerals, micro-fabric, initial moisture content, and initial dry density are all factors in the ground heave that happens as a result of soil swelling potential (Matthews, 2016).

2.1.1. Characteristics of expansive soils

The expanding lattice type of clay minerals from the smectite group, of which montmorillonite is a significant part, are present in expansive soils and are what give them their name. Very weak van der-Waals interactions between the mineral's adjacent unit cells, significant isomorphous substitution during clay mineral formation, and extremely large negative surface charges are all characteristics of these clay minerals including extremely high specific surface area (400-900 m^2/g) and extremely high cation exchange capacity. It has long been known that these minerals behave very differently from non-expanding lattice type clay minerals like kaolinite, which can also be found in natural soil, in reaction to any external physico-chemical environment (Asuri and Keshavamurthy, 2016).

2.1.2. Origin of expansive soils

Like Ian Jefferson, in many parts of the world, expansive soils are common especially in dry and semi-arid areas together with areas where rainy weather returns after extended droughts. Geology (parent material), climate, hydrology, geomorphology and vegetation all affect where they are distributed (Jones et al., 2012). Basic igneous rocks or sedimentary rocks are the parent materials linked to expansive soils. In sedimentary rocks, they are part of the rock itself, but in basic igneous rocks, they are created by the breakdown of feldspar and pyroxene (Alemayehu Tefra and Mesfin, 1999).

2.1.3. Classification and identification of expansive soils

A soil is typically referred to as expansive if its volume variations are moisture sensitive. Due to their negative effects on the stability of the buildings they support, expansive soils are viewed as problematic soils. Therefore, correctly identifying and characterizing such soils becomes extremely important for a practicing geotechnical engineer. In turn, this will support the management of materials, time, money and human resources which are crucial elements in any project management.

2.1.3.1. Methods for mineralogical identification

Those methods fall under this group of techniques are:

A) Analysis of X-ray diffraction

The ratios of the different minerals present in colloidal clay are calculated using this method. Comparisons between the intensity of diffraction lines from various minerals and those from standard substances are particularly important (Chen, 1975).

B) Diffuse/differential thermal analysis

This technique is well known for being effective for controlling materials whose properties are change when heated. The test is used in conjunction with x-ray diffraction because it is ineffective when used alone to identify expansive soils(Chen, 1975).

C) Dye absorption

Minerals in a soil sample can be recognized by their distinctive hues, which are created when dyes are absorbed by the minerals. The hue that the absorbed dye takes on after being processed with acid in a clay sample depends on the clay's distinct minerals' aptitude for base exchange. If the chosen sample contains montmorillonite in amounts greater than roughly 5–10%, the presence of the mineral can be determined(Chen, 1975).

D) Chemical analysis

This technique is a useful complement to other approaches like X-ray diffraction. Chemical analysis can be used to determine the type of isomorphism and to reveal the origin and placement of the charge on the lattice in the montmorillonite group of clay minerals(Chen, 1975).

E) Resolution of an electron microscope

Clay minerals are examined under a microscope to provide a direct view of the substance. Even while two clays may have the same differential thermal curve and x-ray pattern, they will exhibit different morphological traits when seen with an electron microscope. Combining these methods will produce more effective and dependable results. However, due to the need for expensive, highly specialized, and sophisticated gear as well as expert analysis of the resulting data, these approaches are only used in research labs and have a limited application(Chen, 1975).

2.1.3.2. Indirect techniques

In order to assess the swell potential of soils, these approaches use soil index properties like the liquid limit, shrinkage limit, % clay size composition of soils, as well as some indices like the plasticity index, shrinkage index, and the like (Asuri & Keshavamurthy, 2016).

A) Properties relating to liquid limits

Liquid limit: In the laboratory, this upper bound plasticity limit is established using either the traditional Casagrande method or the fall cone method. The liquid limit of a soil is thought to be its ability to store water, which has been used as a gauge of its tendency to swell. The geotechnical engineering literature provides a variety of categorization techniques to identify the degree of soil swell potential based on the liquid limit of fine-grained soils (Table 2.1).

Table 2.1. Expansive soil classification based on liquid limit(Chen, 1988).

Swell potential	**Liquid limit (%)**
Low	<30
Medium/marginal	30–40
High	40–60
Very high	>60

Plasticity index: The plasticity index measures how much a soil's liquid limit differs from its plastic limit. The potential for soil swelling will increase as the plasticity index rises, indicating how plastic the soil is. Table 2.2 lists various methods for determining the expansiveness of the soil based on its plasticity index.

Table 2.2. Relation between swelling potential of clays & plasticity index(Chen, 1988).

Swell potential	Plasticity index (%)
Low	0–15
Medium	10–35
High	20–55
Very high	>35

The following restrictions apply to indirect approaches of characterizing soil swell potential (Asuri & Keshavamurthy, 2016):

- The liquid limit tests are based on strength and empirical principles.
- There is no specific mechanism governing the liquid limits of soils containing expanding lattice type clay minerals and soils containing non-expanding lattice type clay minerals.

- The mechanisms governing the test methods (such as the Casagrande method and fall cone method) and the mechanisms governing the liquid limits of fine-grained soils may not be compatible.
- The plasticity index's dependence on the liquid limit.

B) Properties related to shrinkage limits

Shrinkage limit is a measure of the volume stability of the in-situ soil because it reflects the lower bound water content for any volume reduction of a soil mass. The schemes for identifying the swell potential of fine-grained soils based on shrinkage limit are presented in Tables 2.3 and 2.4 (Sridharan and Prakash, 2016).

Table 2.3. Expansive soil classification based on shrinkage limit (Asuri and Keshavamurthy, 2016).

Swell potential	Shrinkage limit (%)
Low	>15
Medium	10–16
High	7–12
Very high	<11

Table 2.4. Expansive soil classification based on shrinkage limit (Chen, 1975).

Volume change	Shrinkage limit (%)
Non-critical	>12
Marginal	10–12
Critical	<10

Shrinkage Index: This is the distinction between the shrinkage limit and the plastic limit. Table 2.5 provides a classification of the swell potential of fine-grained soils based on their shrinkage index.

Table 2.5. Expansive soil classification based on shrinkage index (Sridharan and Prakashetal, 2016).

Degree of expansion/swellpotential	Shrinkage index (%)
Low	<15
Medium	15–30
High	30–60
Very high	>60

Numerous experts have highlighted that the shrinkage limit and shrinkage index are poor indicators of the soil's ability to swell. This is due to the fact that the shrinkage limit is not a plasticity trait of fine-grained soils and that a very different mechanism regulates shrinkage from that which regulates soil swelling. While the existence of expanding lattice type clay minerals regulates soil swelling, packing phenomena results in a limit to shrinkage that is principally regulated by the relative particle size distribution of fine-grained soils (Asuri & Keshavamurthy, 2016).

C) Particle size composition related properties

Several researchers have proposed criteria based on percentage clay size fraction (i.e., 0.002 mm size) or colloid content (i.e., content of particles of size less than 0.001 mm) to forecast the swell potential of fine-grained soils, in accordance with the scheme shown in Table 2.6.

Table 2.6.Expansive soil classification based on particle size composition (Chen, 1975).

Degree of expansion/swell potential	Percent clay size fraction
Low	<30
Medium	30–60
High	60–95
Very high	>95

Activity: The ratio of a soil's plasticity index to its percentage clay size fraction is typically used to assess a soil’s activity. The criteria listed in table 2.7 can be used to classify the soils depending on their activity.

Table 2.7. Expansive soil classification based on the activity (Asuri and Keshavamurthy, 2016).

Activity (A_c)	**Nature of the soil**	**Probable degree of swell potential**
<0.75	Inactive	Low
0.75–1.25	Normal	Medium
>1.25	Active	High

Strong restrictions apply to the classification criteria based on particle size-related properties:

• Rather than the proportion of clay size fraction present in the soil, the amount of expanding lattice type clay minerals present is what causes the soil to swell.

• The sedimentation analysis (example: hydrometer analysis), which is based on numerous erroneous assumptions, determines the percent clay size fraction or colloid content of a soil (Asuri and Keshavamurthy, 2016).

2.1.3.3. Direct techniques

The techniques falling under this category gauge a soil's capacity to swell directly.

A) Oedometer tests:

First Winterkorn and Fang (1986) claimed that the traditional oedometer swell tests may provide the most accurate and relevant evaluation of a soil's swellpotent ial.

$$S=60K\,(PI)^{2.44}$$

Where,

S=Swell Potential

K=3.6*10$^{\wedge -5}$and is Constant (Chen, 1975).

This equation's restriction is that it is only usable for soils having a clay percentage of 8 to 65%. Additionally, the calculated value matches the laboratory determined swell potential to within 33% (Chen, 1988).The total volume change of a soil from an air dry condition to a saturated condition under a surcharge of 7 KPa in an oedometer is the criterion for the classification of the degree of soil expansiveness according to USBR (Table 2.8).

Table 2.8. Expansive soil classification based on oedometer swell tests(Chen, 1988).

Swell potential	**% Expansion in oedometer (Holtz and Gibbs 1956)**	**% Expansion in oedometer (Seed et al.1962)**
Low	<10	0–1.5
Medium	10–20	1.5–5
High	20–30	5–25
Very high	>30	>25

The main drawbacks of oedometer tests are that they take a lot of time and effort to complete.

B) Tests for free swell

Test for Free Swell Value (FSV): Holtz and Gibbs first suggested this test (1956). This test involves slowly pouring 10 cm^3 of oven-dried soil that has passed a 425-micron sieve into a measuring jar with a capacity of 100 cm^3 and adding distilled water to it. The equilibrium volume of the sediment that forms is then recorded. The increase in soil volume represented as a percentage of the initial volume is then used to determine the free swell value.

Holtz and Gibbs (1956) did not recommend any particular free swell classification values for the soils. Nevertheless, they proposed a visual correlation between FSV and the volume change of undisturbed soil samples placed in the laboratory consolidometer from an air dry to a saturated condition under a load of around 7 KPa (Fig. 2.1). The soil mass contained in a 10 cm^3 dry volume is a variable, which is another drawback of this approach. The 10 cm^3 dry soil volume measurement is only based on personal judgment and does not take into consideration the variance in soil density, which depends on how high the soil is poured into the jar.

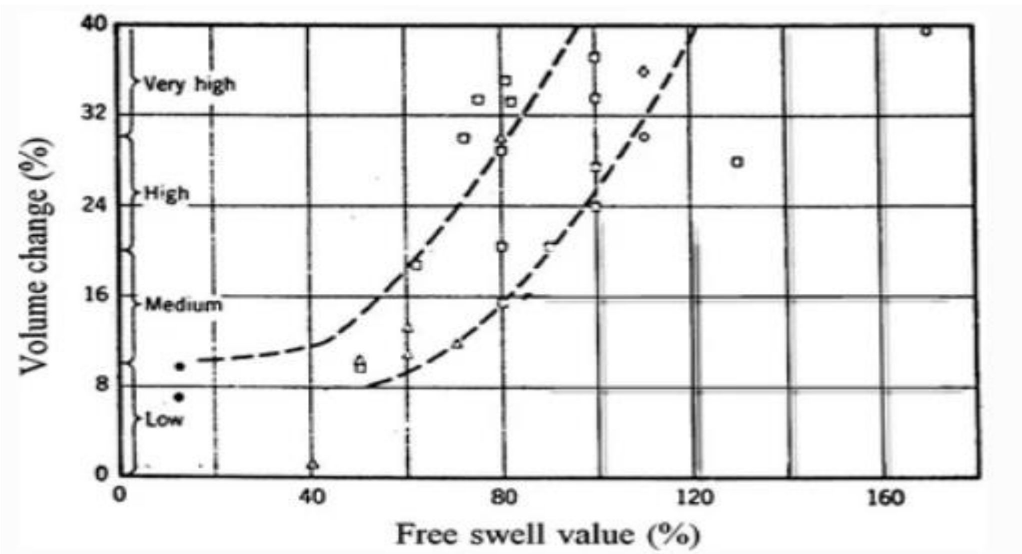


Figure 2.1. Relationship between free swell value and volume change (air dry to saturated condition under a load of 7 KPa) (Holtz and Gibbs 1956)

✓ **Differential Free Swell Test:** This approach gets over the drawback of the free swell volume test by requiring 10 g of initial dry soil mass for the test. In this method, the free swell index is defined by the following equation.

$$FSI(\%) = \frac{[Vd - Vk]}{Vk} 100$$

Where, Vd and Vk are the equilibrium sediment volumes of 10g oven-dried soil samples that pass through a 425-m sieve and are deposited, after at least a 24-hour equilibration time, in 100 ml graduated measuring jars containing distilled water and kerosene, respectively. Based on FSI, the soil's swelling potential is categorized in accordance with the rules in Table 2.9.

Table 2.9. Expansive soil classification based on FSI

Swell potential	**FSI (%)**
Low	<50
Medium	50–100
High	100–200
Very high	>200

The main drawback of the differential free swell method, according to Sridharan, is that it produces negative free swell indices for kaolinite-rich soils.

2.1.4. Expansive soil distribution in Ethiopia

Ethiopia is noted for having a lot of expansive soil. The central region of Ethiopia along the main trunk highways like Addis-Ambo, Addis-Woliso, Addis-Debre Birhan, Addis-Goha Tsion, and Addis-Modjo are some of the regions covered by expansive soils, although the breadth and range of distribution of this problematic soil have not been completely explored. Expansive soils are also known to cover a portion of Mekele, Gondar, Bahir Dar, Debre Birhan and Gambela (Uge, 2017).

2.1.5. Clay minerals

The term "clay minerals" refers to a class of hydrous aluminum silicates that dominate soils with clay particles smaller than 0.002 millimeters. These minerals are comparable to the primary minerals that come from the Earth's crust in terms of their chemical and structural makeup, but weathering causes changes to the geometric arrangement of atoms and ions inside their structures. Primary minerals often originate from igneous or metamorphic rocks and are formed at high temperatures and pressures. Although these minerals are relatively stable inside the soil, they may undergo changes if they are exposed to the surface's environmental factors. Quartz, micas, and certain feldspar are some of the principal minerals that are most resistant to disintegration, while other minerals, such as pyroxenes, amphiboles, and a variety of accessory minerals, are less resistant making secondary minerals possible. The end product of either altering the primary mineral structure (incongruent reaction) or neo-formation through precipitation or re-crystallization of dissolved elements into a more stable structure is the secondary minerals that result (congruent reaction). Because of their platy or flaky nature and the fact that one of its primary structural constituents is an extended sheet of SiO_4 tetrahedra, these secondary minerals are frequently referred to as phyllo- silicates (Phyllon, Greek for "leaf") (Barton & Karathanasis, 2002).

2.1.6. Structures of clay

The chemical underpinnings of a mineral, the geometric arrangement of its atoms and ions, and the electrical forces that hold it together all contribute to its composition. It makes sense that these eight elements are included in the elemental composition of soil minerals because they make up more than 99% of the Earth's crust. Despite this, it makes sense that silicon and oxygen predominate in the phyllo-silicate structure. All silicate structures are built on the silica tetrahedron. It is made up of one silicon ion at the center and four O_2 ions coordinated to the apexes of a regular tetrahedron (Fig.2.2). The tetrahedral sheet is a hexagonal network formed by an interlocking collection of these tetrahedral joined at three corners in the same plane by shared oxygen anions (Fig. 2.3). External ions are coordinated to one hydroxyl and two oxygen anion groups when they bind to the tetrahedral sheet. As the coordinating cation, an aluminum, magnesium, or iron ion is often surrounded by six oxygen atoms or hydroxyl groups, forming the eight-sided building block known as an octahedron (Fig. 2.2). The octahedral sheet is made up of the horizontal links connecting several octahedra. While many clay minerals' octahedral sheets resemble those of the minerals brucite ($Mg(OH)_2$) and gibbsite ($A1(OH)_3$), phyllo-silicates may also contain coordinating anions other than hydroxyls. The octahedral layer may contain trivalent or divalent cations.

When the cations are divalent (Mg, Fe^{2+}), the layer displays brucite-like shape, which preserves electrical neutrality. All three of the octahedron's potential cation sites are occupied, and the ratio of divalent cations to oxygen in this configuration is 1:2. Trioctahedral refers to both this configuration and the corresponding sheet created from an array of octahedral shapes. To preserve the charge balance when the cations are trivalent (Al, Fe^{3+}); one of every three octahedral cation sites is removed. In this configuration, the layer displays a di-octahedral structure akin to gibbsite and the ratio of trivalent cations to oxygen is 1:3. The fundamental building blocks of phyllo-silicates are layers of alumino-silicate, which are composed of tetrahedral, di, and tri-octahedral sheets connected by shared oxygen atoms. Varied clay mineral types have different sheet arrangements inside the alumino-silicate layers, which causes varying physical and chemical characteristics that distinguish the clay mineral classes (Barton and Karathanasis, 2002).

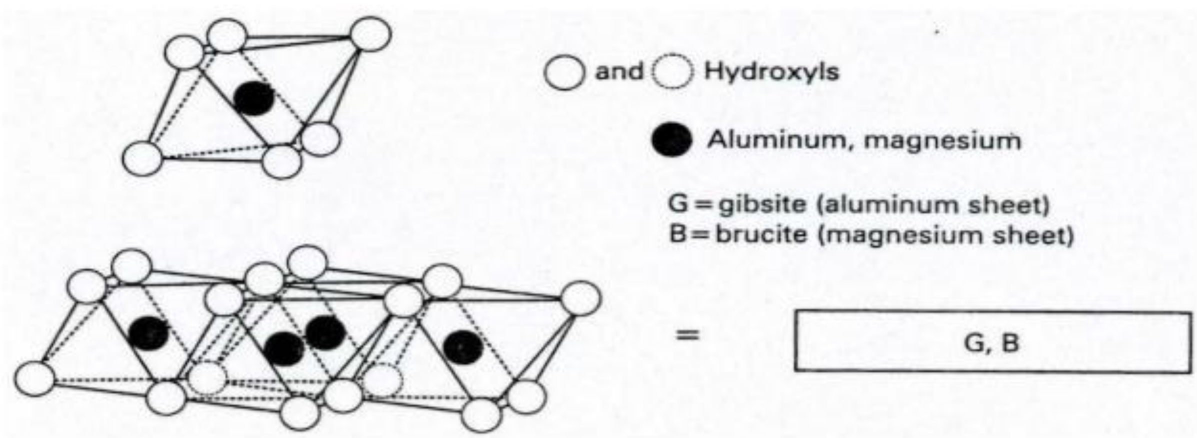


Figure 2.2. An octahedron and an octahedron Sheet

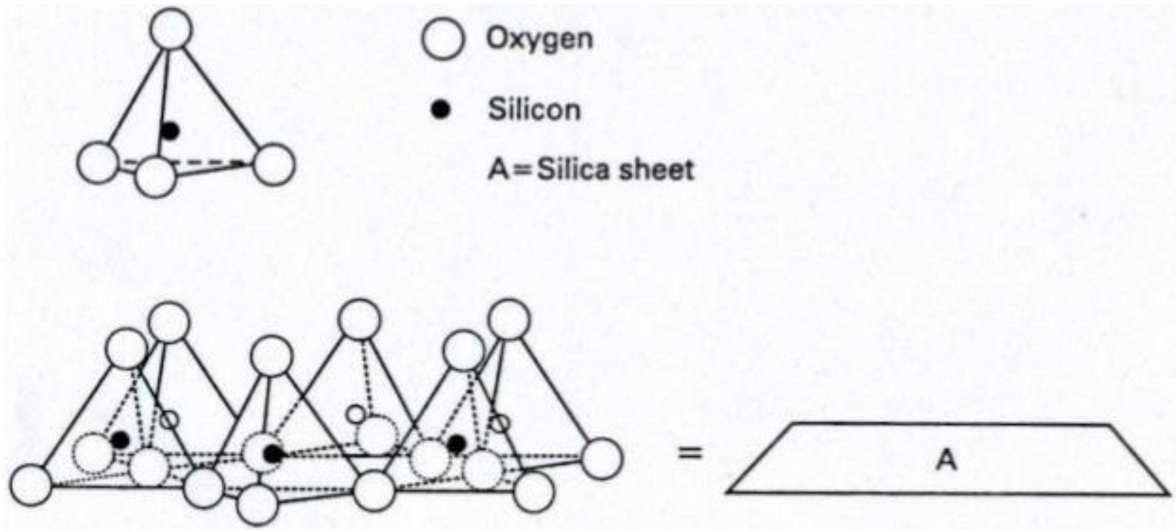


Figure 2.3. A Silica Tetrahedron and a Silica Sheet

2.2. Review of expansive soil stabilization

Improved shear strength characteristics increase the soil's bearing capacity, and this process is known as soil stabilization. It is necessary when the construction-ready soil is unfit to support structural loads. In general, soils have bad engineering qualities. The modification of soils to improve their physical characteristics is known as soil stabilization. Stabilization can boost a soil's shear strength and/or manage its shrink-swell characteristics, which increases a subgrade's ability to support foundations and pavements. In order to raise the shear strength of the soil mass in earth structures and decrease its permeability and compressibility, soil stabilization is used. Stabilization and its impact on soil show how additives react with it, how it affects strength, how it helps to increase and maintain soil moisture content, and how it can be used in construction methods (Afrin, 2017).

2.3. Techniques for soil stabilization

Several techniques can be used to stabilize soil. All of these techniques can be divided into two categories: mechanical stabilization and stabilization utilizing various admixtures.

2.3.1. Mechanical stabilization

By altering the gradation of the soil, mechanical stabilization improves the characteristics of the soil. This procedure involves applying mechanical energy to the soil in the form of rollers, rammers, vibrational techniques, and occasionally blasting in order to compact and densify the soil. The natural qualities of the soil material are what contribute to the stability of the soil in this manner. To create a composite substance that is superior to any of its constituent parts, two or more different kinds of natural soil are combined. In order to create a material that meets the necessary specifications, soils of two or more gradations are mixed or blended together (Afrin, 2017).

2.3.2. Stabilization by using different admixtures

A) Lime Stabilization
B) Cement Stabilization
C) Chemical Stabilization
D) Fly ash Stabilization
E) Rice Husk ash Stabilization
F) Bituminous Stabilization
G) Thermal Stabilization
H) Electrical Stabilization
I) Stabilization by Geo-textile and Fabrics
J) Recycled and Waste Products(Afrin, 2017).

2.4. Lime as a stabilizer

Lime provides an economical way of soil stabilization. The method of soil improvement in which lime is added to the soil to improve its properties is known as lime stabilization. The types of lime used to the soil are hydrated high calcium lime, monohydrated dolomite lime, calcite quick lime, dolomite lime.

Lime stabilization may refer to pozzolanic reaction in which pozzolana materials reacts with lime in presence of water to produce cementious compounds. The effect can be brought by either quicklime, CaO or hydrated lime, $Ca(OH)_2$. Slurry lime also can be used in dry soils conditions where water may be required to achieve effective compaction

(Rogers and Glendinning, 1993). Quicklime is the most commonly used lime; the followings are the advantages of quicklime over hydrated lime (White, 2005). higher available free lime content per unit mass - denser than hydrated lime (less storage space is required) and less dust - generates heat which accelerate strength gain and large reduction in moisture content according to the reaction equation, Cao + H $_2$O → Ca (OH)$_2$ + Heat.

Quicklime when mixed with wet soils, immediately takes up to 32% of its own weight of water from the surrounding soil to form hydrated lime; the generated heat accompanied by this reaction will further cause loss of water due to evaporation which in turn results into increased plastic limit of soil i.e., drying out and absorption. Addition of 2% lime will change the plastic limit to 40% so that the moisture content of the soil will be 5% below plastic limit instead of 10% above plastic limit(Afrin, 2017).

Sherwood looked into the reduction in plasticity that was first caused by cation exchange, in which sodium and hydrogen cations are exchanged for calcium ions because the clay mineral has a stronger affinity for those ions. Even in soils where the clay may already be saturated with calcium ions (such as calcareous soils), adding lime will raise the PH and hence enhance the exchange capacity. In a similar way to cement, when lime reacts with moist clay minerals, the PH rises, favoring the solubility of siliceous and aluminous chemicals. When these substances are combined with calcium, calcium silica and calcium alumina hydrates are produced, which are cementitious materials resembling cement paste. Clay minerals, powdered fly ash (PFA) and blast furnace slag are a few examples of naturally occurring pozzolana materials that contain silica and alumina. The method of lime stabilization is mostly utilized in geotechnical and environmental applications. Contaminant encapsulation, backfill rendering (using, for example, wet cohesive soil), highway capping, slope stabilization, and foundation enhancement using lime piles or lime-stabilized soil columns are a few applications. However, the process of stabilizing lime may be hampered by the presence of sulfur and organic compounds. Sulphate (for instance, gypsum) will react with lime and swell, which could affect the strength of the soil.

2.5. Fly ash stabilization

Fly ash stabilization has recently become increasingly significant due to its widespread availability. This approach is more expedient and less expensive than alternative approaches. It has been effectively used in geotechnical applications and has a long history of use as an engineering material.

In comparison to lime and cement, fly ash, a byproduct of coal-fired electric power plants, has less cementing qualities. The majority of fly ashes come from secondary binders, which by themselves are unable to provide the desired effect. However, it can chemically react with a small quantity of activator to create cementation compound, which helps strengthen the strength of soft soil (Ghais, 2015).

2.6. Summary of previous studies

Table 2.10. Summary of previous studies

No	Title	Author Name/ year	Conducted Tests	Major findings
1	Fly ash Utilization in Soil Stabilization	(Ghais, 2015)	Atterberg limits, soil compaction and California Bearing Ratio (CBR)	The author demonstrated that the effect of fly ash addition, on physical and mechanical properties improves the index properties of clay, reduces the liquid and plastic index, increases bearing capacity.
2	The Suitability and Lime Stabilization Requirement of Some Lateritic Soil Samples as Pavement	(Amu et al., 2011)	Natural moisture content, specific gravity, particle size analysis and Atterberg's limits, compaction, CBR, unconfined compression and undrained triaxial tests	Plasticity index is reduced as the lime increases. The compressive and shear strength were also improved.
3	Effect of lime and Fly Ash on Engineering Properties of black cotton Soil.	(Ramlakhan et al., 2013)	Sieve analysis, Liquid limits, Plastic limit, Compaction and CBR	The authors concluded that fly ash and lime can be used effectively in the stabilization of black cotton soil.

4	Investigation on the effects of combining lime and sodium silicate for expansive subgrade stabilization	(Dinku, 2014)	Atterberg limit test, Standard compaction test, California bearing ratio test	Sodium silicate decreases plasticity of expansive clay. It decreased shear strength and increased swelling properties of expansive clay, Compaction curves of expansive clay treated with sodium silicate or combination of lime and sodium silicate
5	Study of Engineering properties of black cotton soil with pond ash	(Sandyarani et al, 2018)	Atterberg's limits, standard proctor test, differential free swelling index, swelling pressure and wet sieve analysis	The authors concluded that the expansive soil cannot be successfully stabilized by pond ash.
6	Stabilization of expansive soil using bagasse ash &lime	(Meron Wubshet and Samuel Tadesse, 2014)	Atterberg limit, Free swell, Free swell index, Free swell ratio, Compaction & CBR	The plasticity index significantly decreased with addition of lime or bagasse ash combined with lime. The maximum dry density of the stabilized soil decreases with addition of lime, bagasse ash and bagasse ash combined with lime.
7	Stabilization of black cotton soil by using limestone	(Sandyarani et al, 2018)	Moisture content test, Specific gravity test, Particle size distribution and UCS	The addition of lime shows increase in the strength properties of the black cotton soil.

8	Effect of waste cloth on the properties of black cotton soil	(Sangmesh and Sharanakumar, 2020)	Atterberg limits, Standard proctor compaction test, Unconfined compression test and Direct shear test	The stabilizing materials shows increase in the strength properties of the black cotton soil.
9	Soil stabilization using waste clothes (cotton clothes and synthetic clothes)	(Bamrele et al., 2019)	Atterberg limits, Modified proctor test, California bearing ratio	From the experimental study, the authors concluded that, synthetic cloth waste stabilize the soil as compare to cotton cloth waste.
10	Effect of fly ash on geotechnical properties of soil	(Dixit et al., 2020)	Atterbergs limit, Grain size analysis, Specific gravity, Proctor compaction and California bearing	The authors investigated that all the investigated properties were decreased except CBR value and optimum moisture content.
11	Effects of snail shell ash on lime stabilized lateritic soil	(Nnochiri et al., 2018)	Compaction, California bearing ratio (CBR), Atterberg limits and UCS tests.	Results from these tests showed improvement in soil properties, also the values of the CBR and UCS increased considerably.

The researchers who studied expansive soil stabilization used a variety of materials, most of which were expensive and not readily available. This thesis filled a gap by using inexpensive and locally available materials.

CHAPTER THREE: MATERIALS AND METHODS

3.1. Introduction

This chapter describes the sample preparation, laboratory testing, and methods used in this study. The soil sample was taken from the sampling site and brought to Bahir Dar for laboratory analysis. It was at the soil laboratory of Bahir Dar University Institute of Technology that all laboratory tests were completed.

3.2. Overview of the research area

3.2.1. Location of study area

Nefas Mewucha town, which is 175 kilometers far from Bahir Dar and has a latitude and longitude of 11044'N 38028'E and an elevation of 3150 meters above sea level, is the study area’s location in Ethiopia's South Gondar zone of the Amhara regional state. At kebele 3, near Zenebu vicinity which has flat terrain and is heavily coated with black cotton soil, sampling was done.

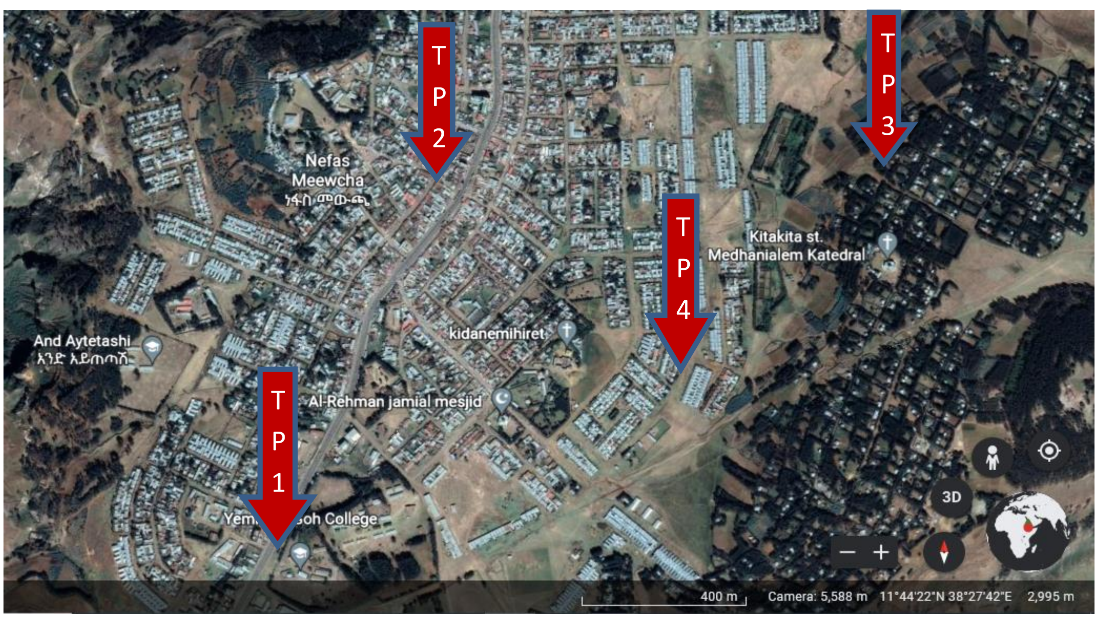


Figure 3.1. Locations of test pits

3.2.2. Climate

Nefas Mewucha town has a predominantly moderate climate. Its average annual temperature is 26^0C. The table below provides a summary of the typical climate of the town.

Table 3.1. Average climate of Nefas Mewcha town (Best travel months.com)

Months	Day	Night	Rain Days
January	27^0C	13^0C	0
February	29^0C	15^0C	1
March	30^0C	16^0C	1
April	30^0C	17^0C	1
May	28^0C	17^0C	5
June	26^0C	15^0C	11
July	23^0C	14^0C	21
August	23^0C	14^0C	25
September	24^0C	13^0C	15
October	25^0C	12^0C	4
November	26^0C	12^0C	1
December	26^0C	12^0C	0

3.2.3. Rainfall

According to the National Meteorological Agency (NMA,2022), the seasonal rainfall of Nefas Mewucha town ranges from 37.2mm to 461.1mm in the spring and from 450.6mm to 1004.6mm in the summer. While the standard deviation is 141 and the coefficient of variability 13%, the mean annual rainfall is 1052 mm.

3. 3. Materials utilized

3.3.1. Black cotton soil

Black cotton soils are extremely clayey soils whose color range from gray to black. They fall under the category of expansive soils, which grow while wet and contract when dried. They behave expansively and are quite disruptive. They are widespread in arid and semi-arid regions of the planet and are periodically shaken by changes in seasonal moisture content. During the rainy season, the ground will heave, and during the dry season, it will shrink. Structures built on the ground suffer significant harm from this cyclical swell-shrink movement. In this study, black cotton soil from all 4 kebeles of Nefas Mewucha town was gathered, and tests on the Atterberg limit and free swell were performed; consequently, to conduct further laboratory research, I have chosen one kebele (kebele 3, near Zenebu) that is covered by more expansive soil.

3.3.2. Lime

Quick lime and un-slaked lime are two common names for the inorganic chemical substance known as lime derived from limestone, a naturally occurring chemical. Quick lime, also known as calcium oxide, is a potent caustic component that is frequently used in the building industry to prepare plasters and mortar. Lime promotes soil strength while reducing soil density.

Medium, moderately fine, and fine-grained soils respond to lime by becoming less flexible, more workable, and stronger. Gain in strength is mostly caused by chemical interactions between lime and soil particles (Little, 1995).

3.3.2.1. Chemical and physical properties of quicklime

According to (Little, 1995), quicklime's reaction with water is crucial in practice because it forms the foundation for the creation of hydrated lime from quicklime. At a building site, quicklime is converted into hydrated lime by a slaking process, yielding a highly reactive substance for soil stability.

Table 3.2. Chemical & physical properties of Lime

Major & Minor oxides	Amount in percent (%)
Calcium Oxide (CaO)	83.3
Silicon Oxide (SiO_2)	2.5
Iron Oxide (Fe_2O_3)	2
Aluminum Oxide (Al_2O_3)	1.5
Magnesium Oxide (MgO)	0.5
Na_2O+K_2O	0.5
Co_2	5
Loss on Ignition (LOI)	1.25
Physical properties	
Specific Gravity (g/cm^3)	3.3
Color	White

3.3.3. Synthetic cloth waste ash (SCWA)

The design, manufacture, and distribution of garments, yarn, and other textile products are the main concerns of the textile industry. In this world, everyone needs clothing as well as other textile products like bags and shoes. However, one of the most polluting industries is the textile industry. Textile consumption and production both generate waste and the industrial process ends with wastes. In comparison to other stabilizer agents, the substance is thus readily available.

Synthetic cloth waste ash is one of the stabilizing materials used in this study. It was chosen because of its fibrous qualities, which contribute to the strength of the black cotton soil. The garbage from clothes and the textile factory was gathered. And the trash was burned, turning it into ash. Production will benefit from the gains in quality that come from managing both extracted garbage and waste that has been processed again. The waste synthetic cloth ash was gathered from the Bahir Dar textile factory, which burns waste synthetic fabric in a blast furnace at a temperature of 5500°C. The remaining waste cloth was gathered from various clothing shops in and around Nefas Mewucha town and burned.

3.3.3.1. Chemical and physical properties of synthetic cloth waste ash

Oxides of silicon, aluminum, iron, and calcium make up the majority of ash. Additionally, to a lesser extent, magnesium, potassium, sodium, titanium, and manganese are all present. I've sent the sample of SCWA to Institute of geological survey of Ethiopia for XRD test and the result of chemical and physical characteristics is listed in Table 3.3.

Table 3.3. Chemical and Physical Properties of SCWA (Geological survey of Ethiopia)

Major & Minor Oxides	Amount in Percent (%)
Silicon Oxide (SiO_2)	35.19
Aluminum Oxide (Al_2O_3)	19.96
Iron Oxide (Fe_2O_3)	9.04
Calcium Oxide (CaO)	3.44
Magnesium Oxide (MgO)	2.4
Sodium Oxide (Na_2O_3)	1.36
Potassium Oxide (K_2O)	0.4
Manganous Oxide (MnO)	0.16
Phosphorous Penta Oxide (P_2O_5)	1.98
Titanium di Oxide (TiO_2)	1.72
Water (H_2O)	0.89
Physical Properties	
Specific Gravity	2.08
Color	Very dark grey

3.3.4. Water

For some laboratory tests, such as Atterberg limits and specific gravity, it is advised to use pure water, per the standard test. However, tap water is used for all laboratory testing because there any distilled water is not available in the lab of Bahir Dar University Institute of Technology.

a) Synthetic cloth waste **b) Synthetic cloth waste ash** **c) Lime**

Figure 3.2. Materials used

3.4. Methodology

3.4.1. Sampling and preparation of material

The soil sample was first visually detected, and it was collected in a region of Nefas Mewucha town known as Zenebu. From four different test pits, a more expansive one was chosen, and a sample was taken at a depth of about 2.5 m to 3 m. This sample was then carefully packed in plastic bags, transported to the laboratory at Bahir Dar Institute of Technology, and dried in an oven there for 24 hours at a temperature of 110^{o}C. The sample was removed from the oven after drying for 24 hours, and various laboratory tests were run on it. Additionally, the soil sample was mixed with various ratios of waste synthetic cloth ash and lime separately, and related laboratory tests were also carried out. Figure 3.3 shows the general methodology applied to conduct this thesis grammatically.

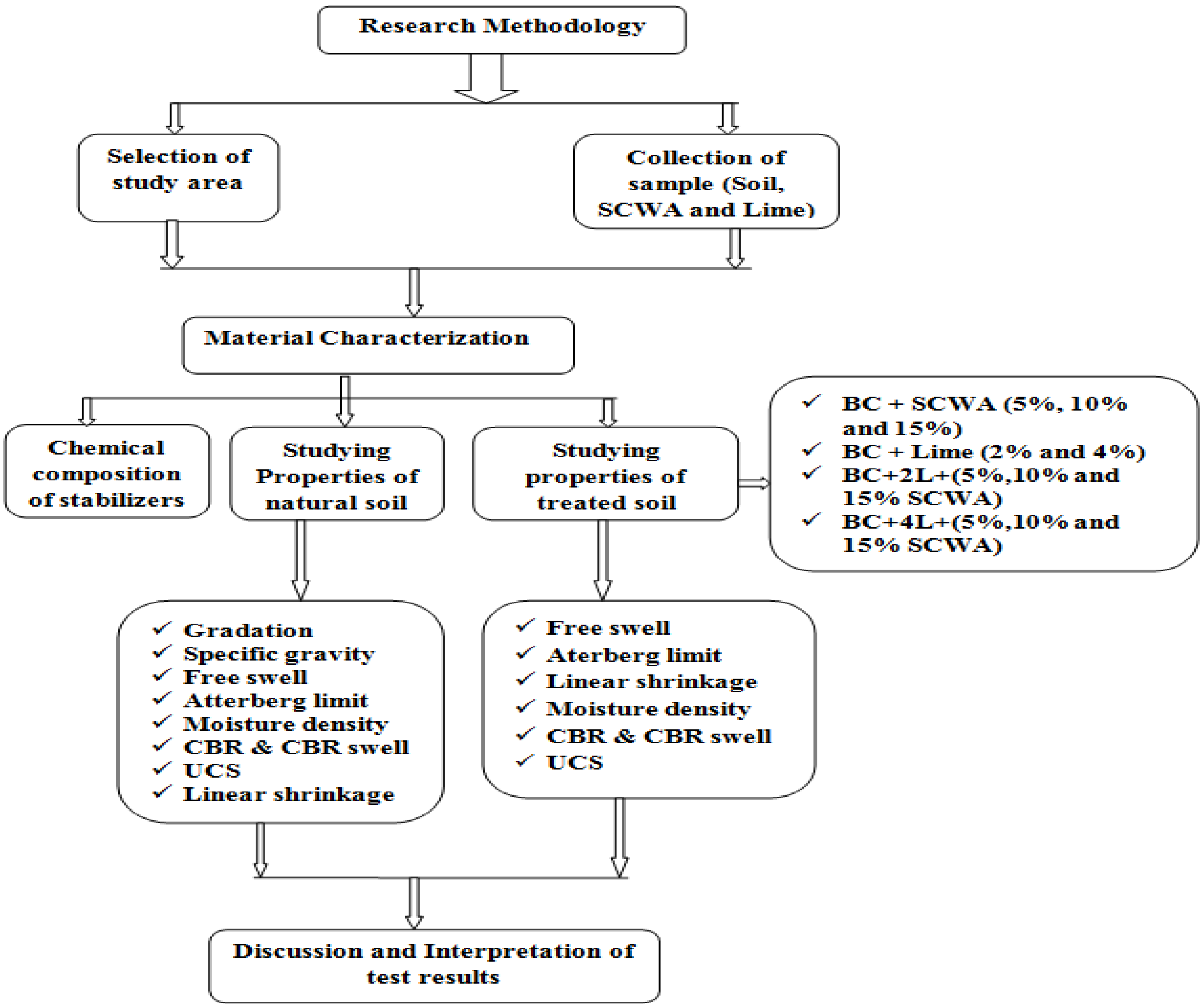


Figure 3.3. Research method adopted for this study

Please, note that BC= Black Cotton, L=lime and SCWA=Synthetic Cloth Waste Ash.

3.4.2. Mixing ratio

The addition of stabilizing additives was applied by combining natural soil with SCWA in 5% interval (5%, 10% and 15%) and lime in 2% interval (i.e., 2% and 4%).

Table 3.4. Summary of mixing proportion applied

<table>
<tr><th rowspan="4">BC</th><th>SCWA content (%)</th><th rowspan="4">BC</th><th>Lime content (%)</th><th rowspan="4">BC</th><th>Lime content (%)</th><th>SCWA content (%)</th><th rowspan="4">BC</th><th>Lime content (%)</th><th>SCWA content (%)</th></tr>
<tr><td>5</td><td>2</td><td rowspan="3">2</td><td>5</td><td rowspan="3">4</td><td>5</td></tr>
<tr><td>10</td><td rowspan="2">4</td><td>10</td><td>10</td></tr>
<tr><td>15</td><td>15</td><td>15</td></tr>
</table>

Table 3.4 shows that, the mixing process is done accordingly by;

- Adding lime on natural soil by 2% interval (i.e., 2% & 4%)
- Adding synthetic cloth waste ash on natural soil by 5% interval (i.e., 5%, 10% & 15%).
- Fixing lime at 2% and varying Synthetic cloth waste ash by 5% (i.e., 5%, 10% and 15%) by dry weight of the soil.
- Fixing lime at 4% and varying Synthetic cloth waste ash by 5% (i.e., 5%, 10% and 15%) by dry weight of the soil.

3.4.3. Frequency of conducted laboratory tests

The number of tests conducted for both the untreated & treated soil samples are tabulated in table 3.5.

Table 3.5. Number of tests conducted

No	Mixing Proportion	Laboratory tests conducted							
		Grain size analysis	Specific gravity	Free swell	Atterberg limit	Linear shrinkage	Modified proctor compaction	Three-point CBR	UCS
1	BC soil	1	1	1	4	2	1	1	4
2	BC+5SCWA			1	4	2	1	1	4
3	BC+10SCWA			1	4	2	1	1	4
4	BC+15SCWA			1	4	2	1	1	4
5	BC+2L			1	4	2	1	1	4
6	BC+4L			1	4	2	1	1	4
7	BC+2L+5SCWA			1	4	2	1	1	4
8	BC+2L+10SCWA			1	4	2	1	1	4
9	BC+2L+15SCWA			1	4	2	1	1	4
10	BC+4L+5SCWA			1	4	2	1	1	4
11	BC+4L+10SCWA			1	4	2	1	1	4
12	BC+4L+15SCWA			1	4	2	1	1	4

3.4.4. Curing period

The following table shows the applied curing time for untreated & treated soil.

Table 3.6. Curing Period

No	Mixing proportion	Laboratory Tests							
		Grain size analysis	Specific Gravity	Free swell	Atterberg limit	Linear shrinkage	Modified proctor Compaction	Three-point CBR test	UCS test
1	Untreated soil	No curing	No curing	No curing	No curing, 3,7 & 28 days of curing	No curing	No curing	Un soaked and 4 days of soaking	Uncured, 3,7 & 28 days of curing
2	SCWA treated soil	-	-	No curing	No curing, 3,7 & 28 days of curing	No curing	No curing	4 days of soaking	Uncured, 3,7 & 28 days of curing
3	Lime treated soil	-	-	No curing	No curing, 3,7 & 28 days of curing	No curing	No curing	4 days of soaking	Uncured, 3,7 & 28 days of curing
4	Treated soil by combined stabilizers	-	-	No curing	No curing, 3,7 & 28 days of curing	No curing	No curing	4 days of soaking	Uncured, 3,7 & 28 days of curing

3.4.5. Soil classification

The distribution of grain sizes in soils varies greatly. The plastic characteristics of soils can also vary greatly depending on the type and quantity of clay minerals present. Different engineering projects call for the field-based identification and classification of soil. Based on their grain-size distribution and plasticity, soils must be organized into distinct groups or subgroups for use in the design of foundations and earth-retaining structures, the construction of highways, and other related applications. There are two main systems that divide soils into different groups or subgroups. These are the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) Classification System. According to the AASHTO soil classification system, soils are typically divided into 12 sub-groups and seven major categories (A1 through A7). The soil types and index properties are represented by symbols in the Unified soil categorization system. According to their size, gradation, plasticity index, and liquid limit, soils are categorized. The USCS categorizes soils into three main groups: highly organic soils, fine-grained soils, and soils with a coarse grain. I used both classification schemes for this specific investigation (Das, 2008).

3.4.6. Laboratory evaluation

The following tests were carried out with the addition of waste cloth ash and lime on the obtained sample of soils in varied quantities to evaluate the effect of synthetic cloth waste ash and lime as a stabilizing agent of the black cotton soil. Table 3.7 lists laboratory tests conducted and their test methods.

Table 3.7. Laboratory tests conducted and test methods

Laboratory tests conducted	Test methods
Natural Moisture Content	ASTM D2216
Grain size Analysis	ASTM D422
Specific Gravity	ASTM D854
Atterberg Limit	ASTM D4318
Linear Shrinkage	ASTM D427
Modified Proctor Compaction Test	ASTM D1557
California Bearing Ratio (CBR)	ASTM D1883
Unconfined Compressive Strength (UCS)	ATM D2166

3.4.6.1. Natural moisture content

The purpose of this test was to quantify the moisture content of the soil mass. Understanding the water content of the soil is essential for controlling soil compaction, estimating the soil's maximum consistency and calculating the stability of all types of earthworks and foundations. It is the weight of water to solid ratio in a particular mass of soil, expressed as a percentage. Based on the ASTM D2216 standard reference, the test was carried out. A comparable amount of damp soil was obtained and placed in the clean moisture can. The moisture can with the moist soil was then weighted, recorded, and placed in the oven for 24 hours. Later, the moisture container containing the dry soil was taken out of the oven after 24 hours and let to cool. Finally, the dry soil's mass was computed, together with its natural moisture content (ASTM Standard D2216, 1998).

3.4.6.2. Grain size analysis

The percentage of various grain sizes present in a soil is calculated using both wet sieve analysis and hydrometer tests. A piece of soil that is retained on the No. 200 sieve underwent wet sieve analysis. Due to the black cotton soil's natural tendency to cling together when subjected to dry sieve analysis, wet sieve analysis is utilized to measure the quantity of silt present in the soil. However, in order to ascertain how the smaller particles were distributed, a hydrometer analysis was carried out. The tests were carried out using ASTM D422 as the standard reference.

3.4.6.3. Specific gravity test

A pycnometer is used to conduct a specific gravity test to ascertain the specific gravity of soil. According to its definition, it is the difference between the mass of a given volume of soil at a certain temperature and the mass of a given volume of distilled water at that same temperature. For this test, 25g of oven-dried soil that passed through a no. 10 sieve was employed. The ASTM D854 testing method was used to estimate the specific gravity of soils. Following is a table of specific gravity ranges for several types of soils;

Table 3.8. Ranges of Specific gravity (Chen, 1988)

Type of soil	Specific gravity
Sand	2.63 to 2.67
Silt	2.65 to 2.7
Clay and silt	2.67 to 2.9
Organic soil	1 to 2.6

3.4.6.4. Atterberg limits test

The Atterberg limits test involves figuring out the liquid limits and plastic limits of both treated and untreated soil samples, in accordance with ASTM D4318 testing methods. After being oven-dried, the soil sample utilized for this test was passed through a No. 40 sieve. The Casagrande cup containing the wet soil sample was grooved by making firm strokes with the grooving tool along the symmetrical axis of the cup through its centerline. The cup was then repeatedly lifted and dropped by turning the crank at a rate of two revolutions per second until the two halves of the soil pastes came into contact with each other for a length of about 13 mm. It was noted how many strokes were necessary to bring the two halves of the soil pat together at the groove's base.

To achieve this, four trials were conducted: the first between 15 and 20 drops, the second between 20 and 25 drops, the third between 25 and 30 drops, and the fourth between 30 and 35 drops. In each trial, a representative part of the soil sample was removed from the cup to determine its water content. In the end, the liquid limit for the water content corresponding to 25 strikes was determined from the graph of the water content vs the number of blows. The remaining soil sample from the liquid limit test was used for the plastic limit test. Distilled water was added, and the soil was thoroughly mixed to a consistency that allowed it to be rolled without sticking to the hands. A small amount of the soil mass was then taken and rolled between the palms. After four trials, the thread fragments were collected together and the soil's moisture content was determined by placing it into a moisture container. The plasticity index was then determined using the liquid and plastic limit that had been discovered. The liquid limit and plasticity index values were then used to classify the soil.

3.4.6.5. Free swell test

The expansiveness of a particular treated and untreated soil sample can be roughly estimated using this test. AASHTO and ASTM have not yet standardized this test. Holtz and Gibbs (1956) proposed the technique to assess the expansion potential of cohesive soils. 10 cubic centimeters of oven-dried soil sample that had been passed through a No. 40 sieve was slowly added to a 100 cubic centimeter graduated measuring cylinder that contained distilled water, and the mixture was left for 24 hours to settle entirely at the bottom of the cylinder. The final volume was then measured, and the following equation was used to obtain the free swell value in percent.

$$Free\ Swell(\%) = \frac{Final\ Volume - Initial\ Volume}{Initial\ Volume} \times 100$$

3.4.6.6. Linear shrinkage test

In this test, which is carried out in accordance with ASTM D427 testing method, the total linear shrinkage is determined from linear measurements taken along a standard bar with a length of 140 mm. Soil that previously had a moisture level within the liquid limit and that passes a No. 40 sieve is used to fill the semicircular section. The soil was blended after being added water. A 25 mm diameter semi-circular mold was made, and the top of the mold was completely filled with a soil sample. After that, the mold's initial length was measured, noted, and it was placed in the oven for 24 hours. It was cooled and the last length of the mold filled with dried soil after it had been dried for 24 hours was measured.

$$SL = \frac{Li - Lf}{Lf} \times 100$$

Where, SL=shrinkage limit, Lf= Final Length after oven dry (cm) Li= Initial Length before oven dry (cm).

3.4.6.7. Modified proctor compaction test

In order to evaluate the appropriate moisture content and the maximum dry density, modified proctor compaction test was performed on both untreated and treated black cotton soil. The test was conducted in compliance with ASTM D1557. Air-dried soil sample that had been passed through a sieve with a 4.75 mm opening was combined with some water and compressed in a mold using a modified proctor hammer in five equal levels with 25 blows for each layer. Through this process each trial's moisture content was determined, and From MDD Vs. OMC curvature, maximum dry density (MDD) and optimum moisture content (OMC) were calculated.

3.4.6.8. California bearing ratio (CBR) Test

It is a straight forward penetration test used to evaluate the durability of road sub-grades (soil beneath the pavement). To assess the strength of the untreated and treated black cotton soil, CBR and CBR swell tests were carried out in accordance with ASTM D1883 standard. At an OMC determined in modified compaction, samples are molded in three separate molds with a compaction intensity of 10, 30 and 65 blows. It was then submerged in water for four days. Before soaking, the moisture content was determined. CBR values were calculated at 2.5 mm and 5.0 mm of penetration. Then, 95% of the maximum dry density was used to calculate the CBR test results and swell percent was calculated using the swelling value.

3.4.6.9. Unconfined compressive strength (UCS) Test

According to ASTM D2166, the unconfined compressive strength (qu) is the compressive stress at which a cylindrical unconfined soil specimen fails a simple compression test. According to this test method, the unconfined compressive strength is determined by taking the greatest load per unit area that is achieved during a test, or the load per unit area at 15% axial strain, whichever occurs first. For unconfined compressive strength testing, only cohesive soils are permitted. It was applied to determine the clay's unconfined, undrained, unconsolidated shear strength. Only an axial force that causes the soil to fail was applied during the test, and there are no restricting stresses. The unconfined compressive strength tests were performed on both treated and untreated samples.

CHAPTER FOUR: RESULTS AND DISCUSSION

4.1. General

The results of laboratory experiments are briefly presented, and the impact of lime and the ash of waste synthetic cloth on the characteristics of untreated soil has been assessed. The laboratory tests performed on untreated soils as well as soils treated with synthetic cloth waste ash and lime including natural moisture content, free swell, specific gravity, grain size distribution analysis, Atterberg limit (LL and Pl), shrinkage limit, moisture density relationship (MDD & OMC), California bearing ratio (CBR) & CBR swell besides unconfined compressive strength tests.

4.2. Laboratory test results of untreated soil

On natural soil, many laboratory tests such as the Free swell index, Specific gravity, Atterberg limit (cured and uncured), Modified compaction, CBR (both soaked and un-soaked), and UCS(cured & uncured) were performed.

4.2.1. Natural moisture content

The natural moisture content was determined to be 38.56% based on laboratory testing done on natural soil.

4.2.2. Grain size distribution

To determine the distribution of various particle sizes, both sieve analysis and hydrometer analysis were performed. The natural soil has a composition of 0% gravel, 6.86% sand, 32.28% silt and 60.86% clay according to the findings of sieve and hydrometer testing. The graph below displays the distribution of particle sizes.

Sieve size (mm)	% Passing (%)
4.75	100.00
2.36	100.00
2	99.47
1.18	98.83
0.6	98.60
0.425	97.88
0.3	97.23
0.25	96.93
0.15	95.84
0.075	93.14
0.028	81.14
0.018	77.45
0.011	71.92
0.008	70.08
0.005	66.39
0.003	60.86
0.001	55.32

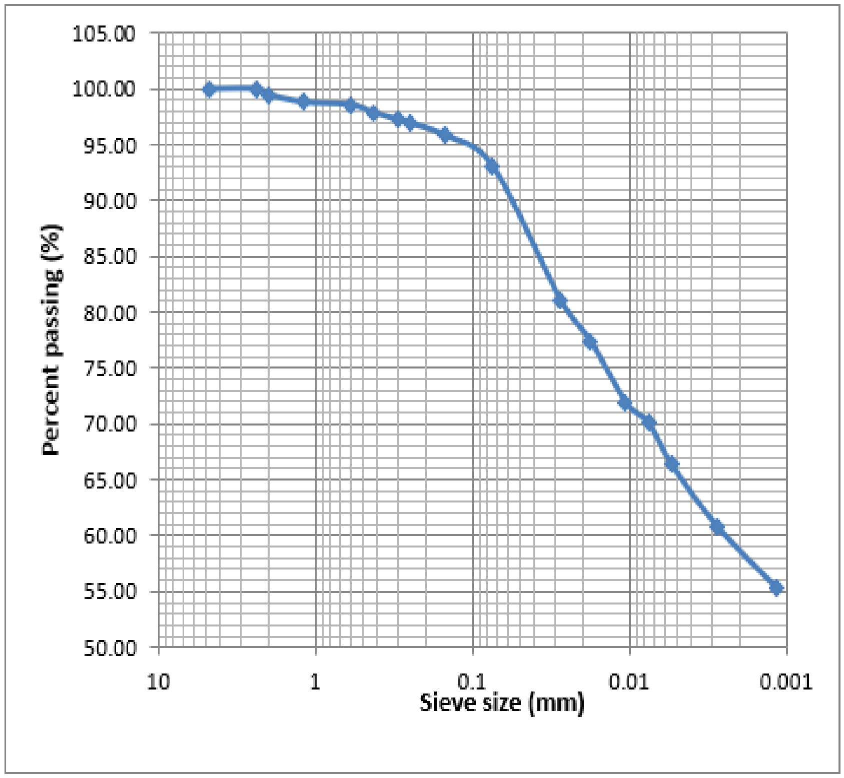


Figure 4.1. Grain size distribution curve of natural soil

Since more than 35% of the soil is passed through a 0.075mm sieve, the natural soil is categorized as A-7-5 (clayey soil) in accordance with the AASHTO classification system. According to the USCS classification system, this soil is classified as CH (CH - highly plastic inorganic clay).

4.2.3. Specific gravity

According to ASTM D854, the specific gravity of untreated (natural) soil is 2.69. And this group of soils includes those that exhibit clayey and silt features.

4.2.4. Atterberg limit

According to the findings of the laboratory tests, the limits for liquid and plastic are 87.43 and 35.94, respectively. However, the plasticity index (PI) is 51.49%. The sample's results for the liquid limit and plasticity index are much greater than the minimal values required by the ERA 2002 standard requirements, which call for a liquid limit of less than 60% and a plasticity index of less than 30%.

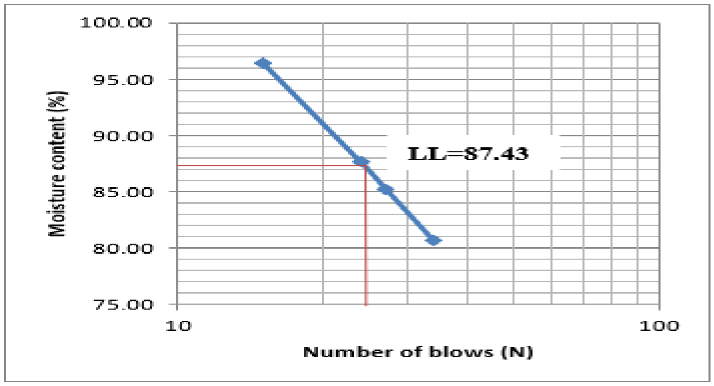


Figure 4.2. Flow curve of natural soil

4.2.5. Free swell

Natural soil has a free swell value of 110%, which is larger than 100% and indicates that the soil is highly expansive.

4.2.6. Linear shrinkage

According to ASTM D4943, two trials were used to calculate the native soil's linear shrinkage, with an average value of 17.66%. A soil with a high degree of expansion is deemed to have a linear shrinkage more than 8%.

4.2.7. Modified proctor compaction test

This experiment looked at the link between dry density and moisture content in natural soil. The outcome reveals that the maximum dry density (MDD) is 1.344 g/cm^3 and the optimum moisture content (OMC) is 40.2%.

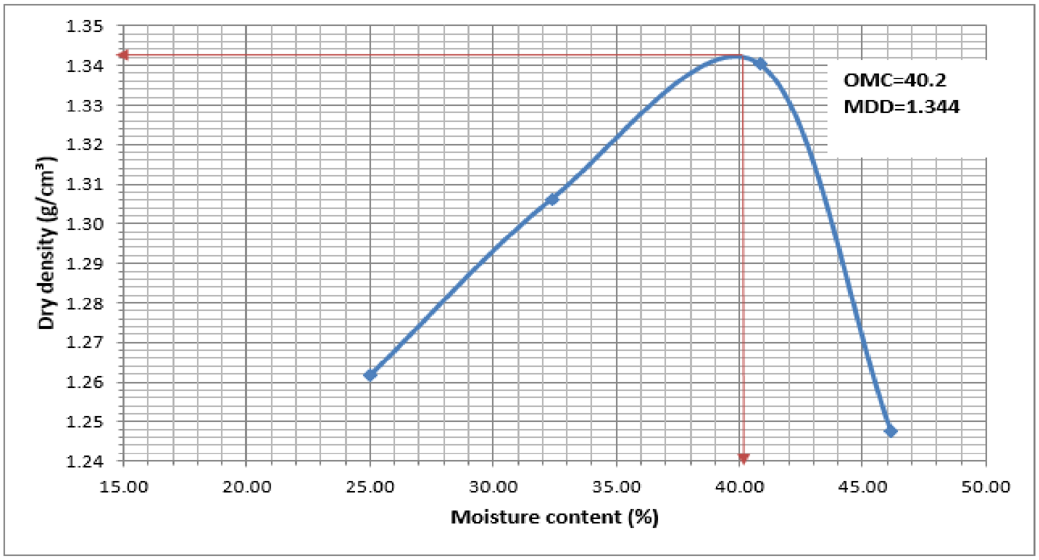
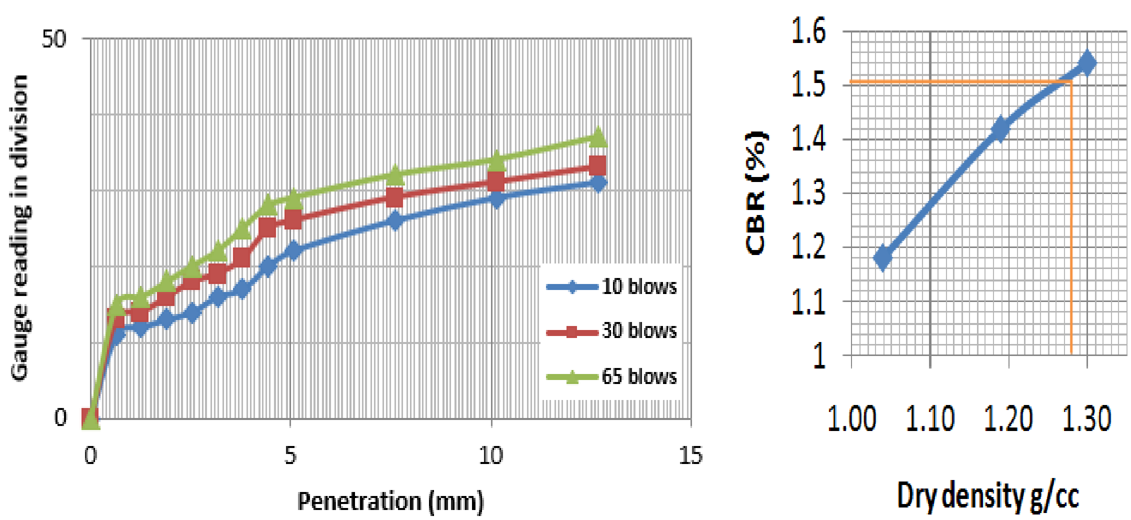


Figure 4.3. Dry density vs. Moisture content of natural soil

4.2.8. California bearing ratio (CBR) and CBR swell values

To determine the strength of natural soil, three-point CBR and CBR percent swell tests were carried out in accordance with ASTM D1883 standard specifications. The results were 1.51% and 14.9% respectively.



a) Penetration Vs Gauge reading curve b) Dry density Vs CBR curve

Figure 4.4. CBR value of natural soil

The minimum permissible CBR value for sub-grade material, according to ERA (2013), is 5%. Native soil cannot be utilized as sub-grade material without modification since a CBR value of 1.51% fails to meet the requirements of S1 sub-grade strength class, which has a very low load bearing capability. Based on their CBR ratings, Table 4.1 classifies sub-grades according to quality.

Table 4.1. Subgrade classification based on CBR

a) Quality of subgrade based on their CBR value

CBR Values (%)	Quality of sub grade
0-3	Very poor
3-7	Poor to fair
7-20	Fair
20-50	Good
>50	Excellent

b) Subgrade classes (ERA, 2013)

CBR Class	CBR Value (%)
S1	<3
S2	3,4
S3	5,6,7
S4	8-14
S5	15-30
S6	>30

4.2.9. Unconfined compressive strength

A UCS value of 156.01 Kpa has been determined for natural soil based on laboratory testing. This value is an indicator of stiff clay consistency of soil(Arora, 2004). Modification or stabilization should be done for the majority of construction activities to improve the workability and engineering qualities of such clay soils. The UCS values for soils of various consistency are displayed in Table 4.2.

Table 4.2. UCS of soils of different consistency (Arora, 2004)

UCS (Kpa)	Consistency of Soil
< 25	Very Soft
25-50	Soft
50-100	Medium
100-200	Stiff
200-400	Very Stiff
> 400	Hard

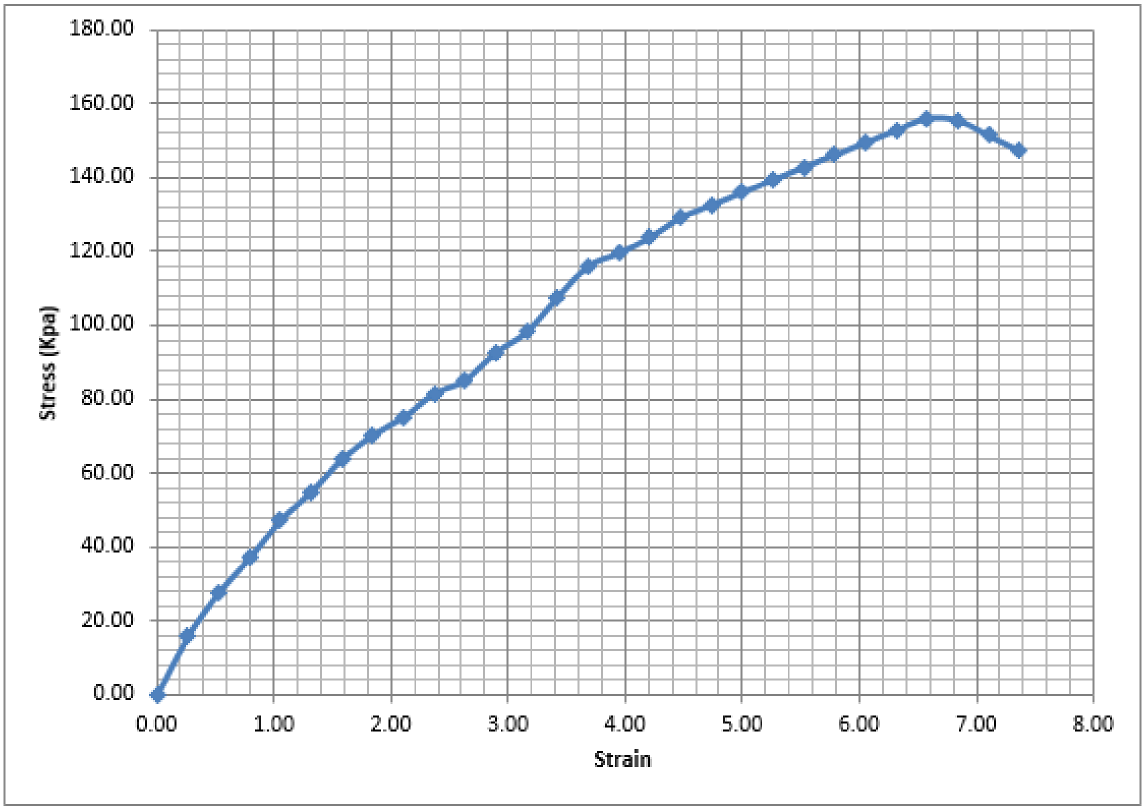


Figure 4.5. UCS of natural soil

4.2.10. Summary of natural soil laboratory test results

The natural soil cannot be used as a good building material without first being treated, according to all laboratory tests done on it. Thus, stabilization should be carried out using the proper stabilizing agents. The following is a summary of the findings of laboratory tests on untreated soil.

Table 4.3. Laboratory test results of natural soil

Laboratory tests conducted		Result
Natural moisture content (%)		38.56
Sieve & Hydrometer analysis	AASHTO	A-7-5 (Clayey soil)
	USCS	CH
Specific gravity		2.69
Atterberg limit (%)	Liquid limit (LL)	87.43
	Plastic limit (PL)	35.94
	Plasticity index (PI)	51.49
Free swell (%)		110
Linear shrinkage (%)		17.66
Modified proctor compaction	OMC (%)	40.20
	MDD (g/cm3)	1.344
Carifornia bearing ratio (CBR) (%)	Un-soaked	1.16
	Soaked	1.51
CBR percent swell (%)		14.9
Unconfined compression strength (UCS) (Kpa)		156.01
Color		Dark grey

4.3. Application of SCWA and lime on soil characteristics

For the purpose of examining their impact on the engineering qualities, different proportions of waste synthetic cloth ash and lime were combined with the natural soil. As previously mentioned, 2% and 4% of lime and 5%, 10% and 15% of synthetic cloth waste ash (SCWA) were separately mixed with black cotton soil.

4.3.1. Influences of SCWA and lime on Atterberg limit

4.3.1.1. Effect of synthetic cloth waste ash (SCWA) on Atterberg limit

For the soil mixed with synthetic cloth waste ash (SCWA) in proportions of 5%, 10%, and 15% by dry weight of soil, Atterberg limit tests were conducted. For the samples, it was done both with and without curing them for 3, 7 and 28 days. The table below provides a summary of the test findings.

Table 4.4. Effect of SCWA on Atterberg limits

Percentage of synthetic cloth waste ash (SCWA)		0%	5%	10%	15%
Laboratory tests conducted (uncured)	Liquid limit	87.43	82.65	80.3	78.73
	Plastic limit	35.94	34.55	35.67	37.42
	Plasticity Index	51.49	48.1	44.63	41.31
Laboratory tests conducted (3 days cured)	Liquid limit	87.15	80.5	77.14	75.68
	Plastic limit	35.78	34.52	35.68	37.4
	Plasticity Index	51.37	45.98	41.46	38.28
Laboratory tests conducted (7 days cured)	Liquid limit	86.97	77.8	76.15	74.16
	Plastic limit	35.72	34.32	35.6	36.92
	Plasticity Index	51.25	43.48	40.55	37.24
Laboratory tests conducted (28 days cured)	Liquid limit	86.25	77.45	73.82	70.17
	Plastic limit	35.12	34.49	35.65	38.4
	Plasticity Index	51.13	42.96	38.17	31.77

The liquid limit of black cotton soil for a sample that has been cured for 28 days is reduced as the amount of synthetic fabric waste ash is increases by 5%. This is due to cation exchange, which spreads out the soil's particles and significantly reduces the soil's cohesion. As SCWA is increased, the plastic limit (PL) of the soil-SCWA mixture fluctuates. The Plasticity Index (PI) however is reduced by 17.23%, 26.45% and 38.79%

while increasing the Ash Content of Synthetic Cloth Waste by 5%, 10% and 15%respectively.

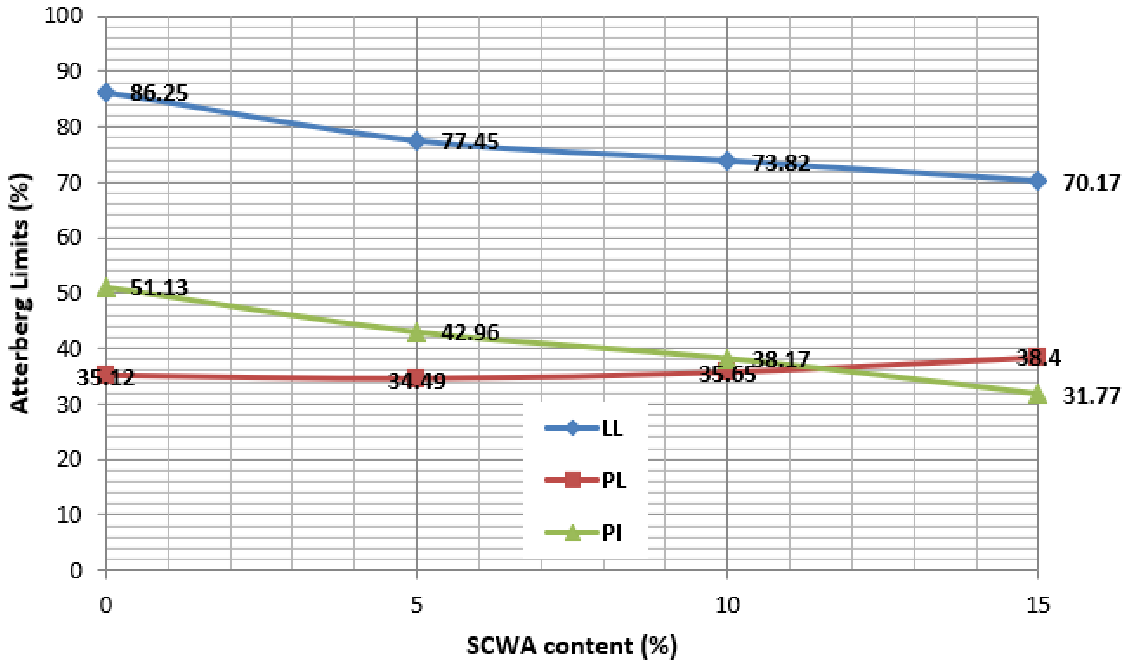


Figure 4.6. Varation of atterberg limits with increasing SCWA content at 28 days of curing

4.3.1.2. Lime's impact on Atterberg limit

When lime is added to the naturally occurring black cotton soil, the liquid limit (LL) becomes decreased as the lime content is increased by 2% and the curing process is prolonged. Additionally, the plastic limit (PL) exhibits a modest increase. However, there is a sharp decline in the plasticity index (PI), which indicates that the soil's capacity to swell has decreased. The cation exchange, which takes place when weaker soil cations are replaced by calcium ions from lime (Ca^{2+}), results in the filling of cavities by the agglomeration of particles process, which lowers the swell potential. The following table provides a summary of how adding lime to native black cotton soil is affected it.

Table 4.5. Effect of lime on Atterberg limits

Percentage of Lime		0%	2%	4%
Laboratory tests conducted (uncured)	Liquid limit	87.43	75.1	62.93
	Plastic limit	35.94	36.12	37.22
	Plasticity Index	51.49	38.98	25.71
Laboratory tests conducted (3 days cured)	Liquid limit	87.15	72.49	60.32
	Plastic limit	35.78	37.23	38.51
	Plasticity Index	51.37	35.26	21.81
Laboratory tests conducted (7 days cured)	Liquid limit	86.97	68.17	57.64
	Plastic limit	35.72	37.61	38.66
	Plasticity Index	51.25	30.56	18.98
Laboratory tests conducted (28 days cured)	Liquid limit	86.25	60.33	53.19
	Plastic limit	35.12	37.42	38.04
	Plasticity Index	51.13	22.91	15.15

Lime makes the diffuse double layer thinner, which raises the charge concentration and, in turn, the viscosity of the pore fluid. The inter-particle shear resistance rises as a result, sharply raising the plastic limit.

Additionally, the flocculation caused by the lime increases the inter-particle resistance to movement, increasing the plastic limit. Because lime needs more time to interact with soil, its effectiveness increases as the curing period lengthens, resulting in an instantaneous drop in the plasticity index (PI). Figure 4.7 displays the Atterberg limit values after 28 days of curing with increased lime content.

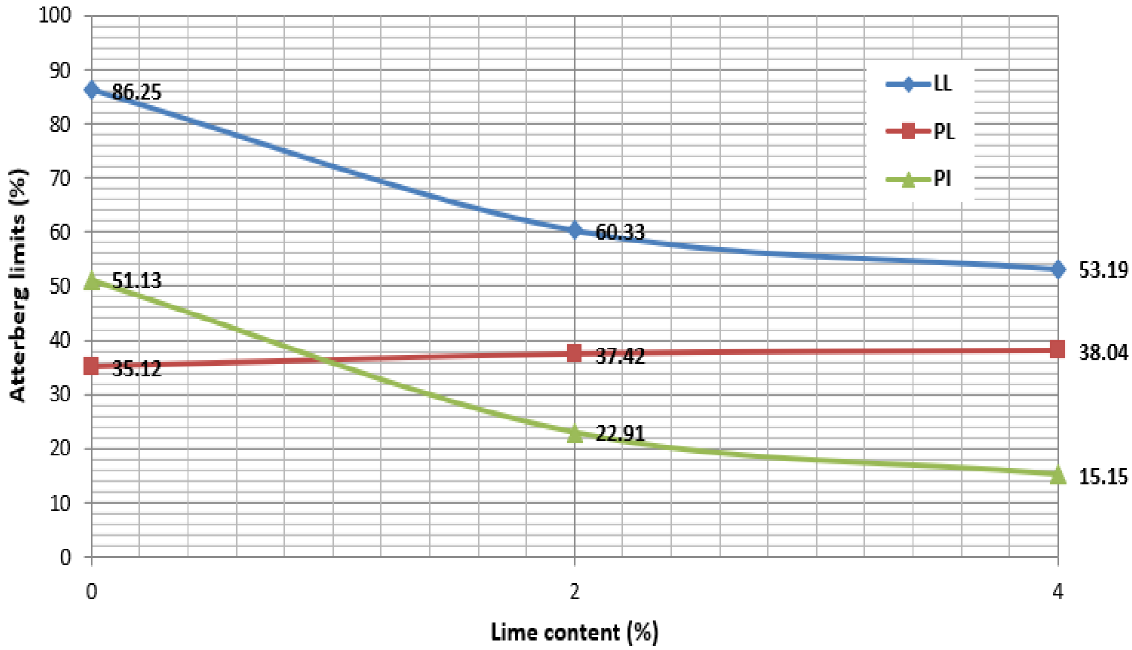


Figure 4.7. Varation of atterberg limits with increasing Lime content at 28 days of curing

4.3.1.3. Effect of combination of SCWA and lime on Atterberg limit

The mixture's liquid limit and plasticity index significantly decreased as the amount of stabilizers and curing time increased. The plastic limit (PL), however, rises. The test was conducted by combining 2% and 4% of lime with 5%, 10%, and 15% of synthetic cloth waste ash. The samples were cured for 3, 7 and 28 days for each group of mixes, and the combination of 4L15SCWA at the 28-day curing time resulted in the greatest reduction in plasticity index (75.63%), lowering the plasticity index of soil from 51.9% to 12.65%. This value of PI enables the soil to be categorized under low swelling potential according to (Chen, 1988). Figure 4.8 shows the impact of mixed stabilizers at various curing times.

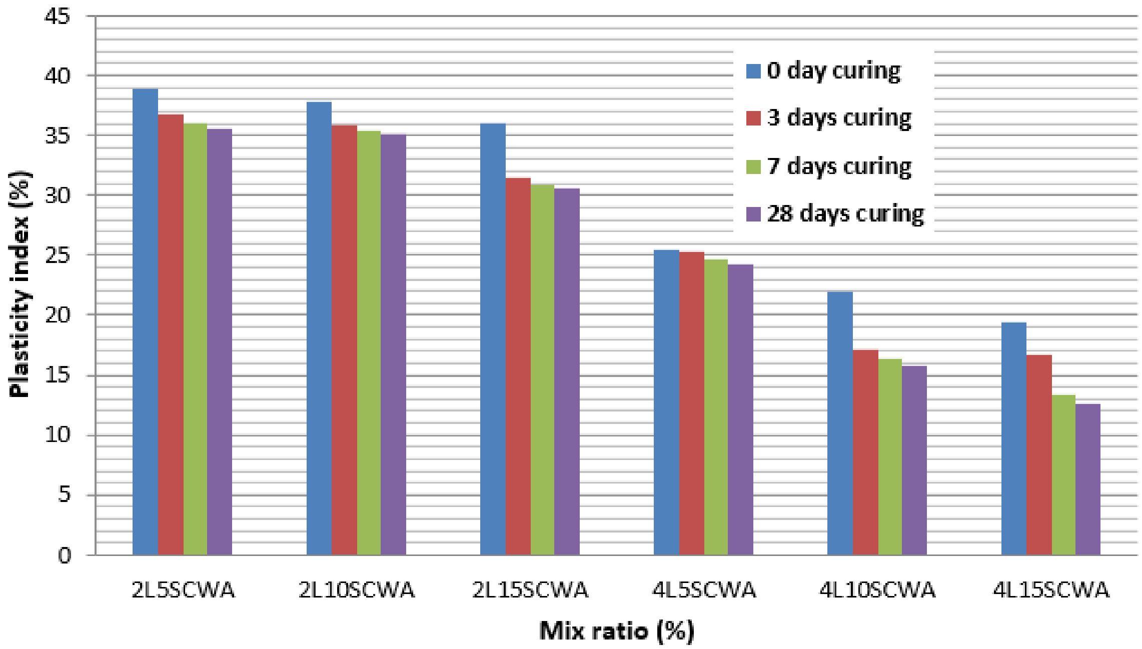


Figure 4.8. Effect of combined stabilizers on plasticity index at different curing periods

The increase in pore water concentration and decrease in the thickness of the diffuse double layer that hold on to the soil are both effects of cation exchange, which results in the release of stabilizer ions into the pore fluid. This study demonstrates that combining stabilizers yields greater results than doing so alone.

4.3.2. Influences of SCWA and lime on free swell

The outcome of the free swell experiment is significantly influenced by the mixture of waste ash from synthetic cloth and lime. The free swell index decreased from 110% to 55% when 4% of lime was added to the soil, and it decreased further to 70% when 15% of synthetic fabric waste ash was applied. The soil's free swell index value is reduced by 61.82% which decreases from 110% to 42% as a result of their combination (4L15SCWA), allowing the soil to be classified as having a lower degree of expansiveness (FSI<50). This outcome demonstrates that the stabilizers are more efficient when coupled as opposed to individually. Figure below illustrates how SCWA and lime together have an impact.

Mix ratio (%)	Free swell index
BC	110
5SCWA	90
10SCWA	90
15SCWA	70
2L	70
2L5SCWA	66
2L10SCWA	60
2L15SCWA	54
4L	55
4L5SCWA	51
4L10SCWA	46
4L15SCWA	42

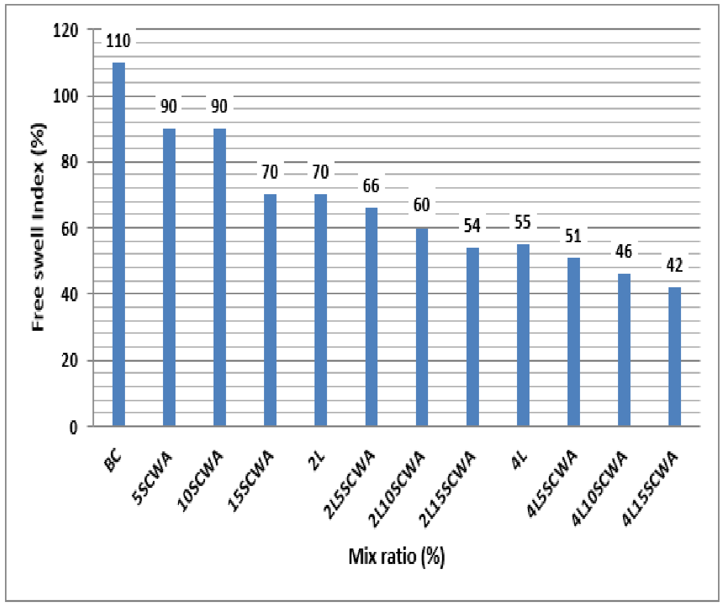


Figure 4.9. Influences of SCWA & lime on free swell

4.3.3. Effects of SCWA and lime on linear shrinkage

The linear shrinkage value of soil decreases as the amount of synthetic cloth waste ash and lime increase. The combination of 4L15SCWA has the greatest reduction in linear shrinkage, dropping it from 17.66% to 3.33% (a decrease of 81.14%). According to Murthy, the treated soil falls under the category of good quality of soil.

Table 4.6. Quality of soil based on linear shrinkage(Murthy, 2007)

Sr	Quality of soil
<5	Good
5 to 10	Medium good
10 to 15	Poor
>15	Very poor

4.3.4. Influences of SCWA and lime on moisture density (OMC & MDD) of soil

4.3.4.1. Effect of SCWA on moisture density relationship of soil

Figure 4.10 below shows a plot of the moisture density relation curve for natural soil mixed with 5%, 10% and 15% synthetic cloth waste ash. For comparison purposes, the curve of natural soil is also provided.

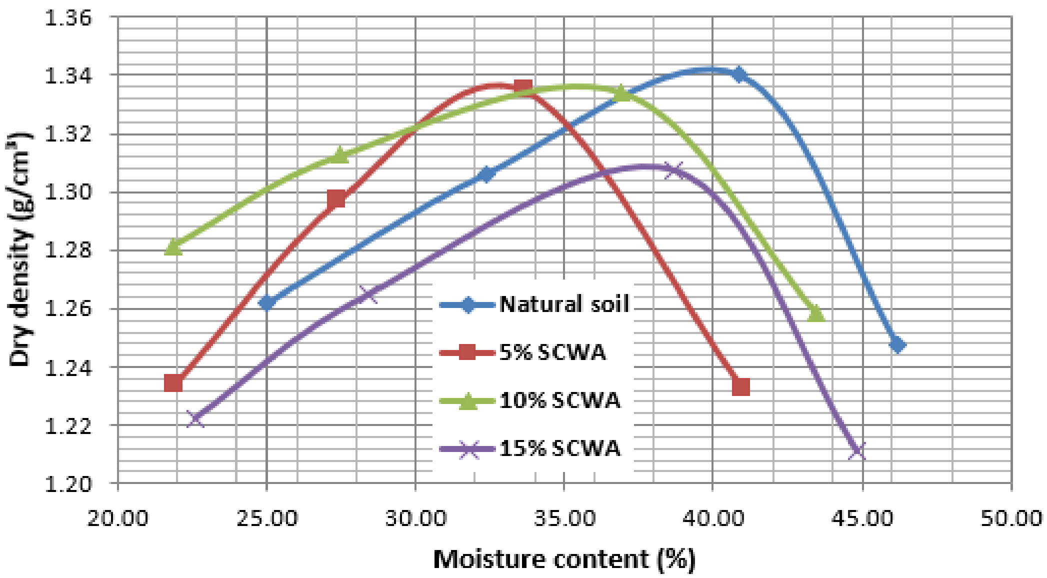


Figure 4.10. Effect of SCWA on moisture density relationship

As shown in the figure, the optimum soil moisture content initially decreases and then slightly increases as the amount of synthetic cloth waste ash rises by 5%. The optimum moisture content drops from 40.2% to 38%. However, at 15% of SCWA, the soil's maximum dry density decreases from1.34 g/cm^3 to 1.31 g/cm^3. The following table summarizes the impact of SCWA on the compaction behavior of natural soil.

Table 4.7. Influence of SCWA on moisture density relationship

Percentage	0%	5%	10%	15%
Moisture content (%)	40.2	33	36	38
Maximum dry density (g/cm^3)	1.344	1.338	1.336	1.31

4.3.4.2. Lime's effect on moisture density relationship of soil

Figure 4.11 shows the modified compaction curves of native soil mixed with 2% and 4% of lime by dry weight of soil.

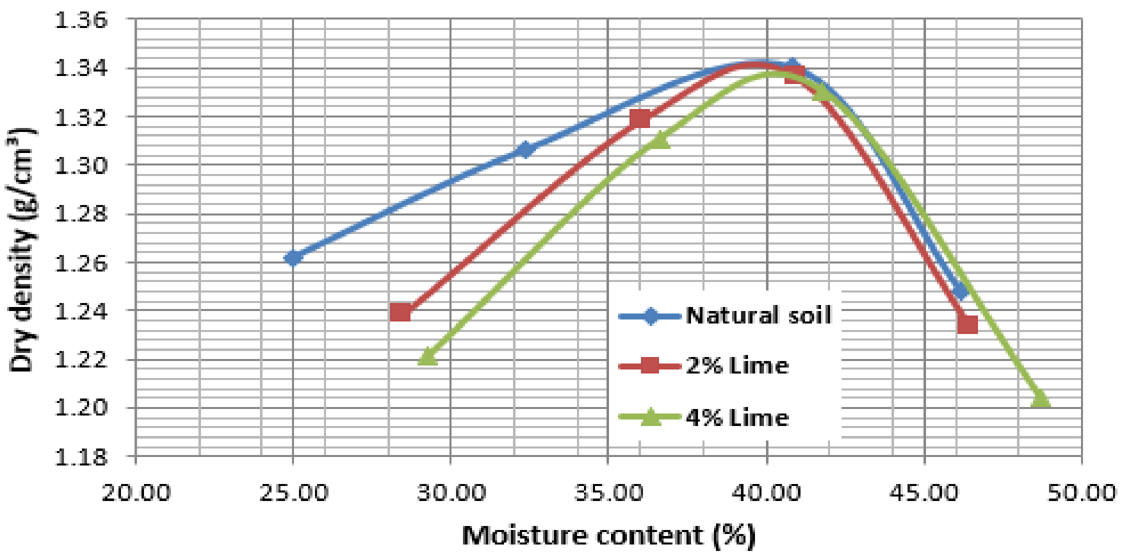


Figure 4.11. Effect of lime on moisture density relationship

The optimum moisture content of the soil initially decreases and then slightly rises at the highest utilization of lime (4% lime); it falls from 40.2% to 39.8% at 2% of lime and rises from 40.2% to 40.5% at 4% of lime. However, at 2% and 4% lime, respectively, the maximum dry density decreases somewhat, almost imperceptibly, from 1.344 g/cm^3 to 1.342 g/cm^3 and to 1.338 g/cm^3. The replacement of soil by lime in the mixture, results in a decrease in maximum dry density (MDD) with increasing lime contents. Conversely, an increase in optimum moisture content (OMC) results from the addition of more water by the additives to carry out chemical reactions. As the majority of studies demonstrate, this is accurate for lime contents exceeding 3% by dry soil weight. Table 4.8 summarizes how lime affects compaction behavior.

Table 4.8. Influence of lime on moisture density relationship

Percentage	**0%**	**2%**	**4%**
Moisture content (%)	40.2	39.8	40.5
Maximum dry density (g/cm^3)	1.344	1.342	1.338

4.3.4.3. Impact of SCWA and lime on moisture density relationship of soil

The combination of stabilizers often decreases the natural soil's maximum dry density (MDD) and slightly increases its optimum moisture content (OMC). The combination of 4% lime and 10% synthetic cloth waste ash results in the highest optimum moisture content, which is found to be 40.8%; the corresponding maximum dry density for this combination is found to be 1.32 g/cm^3. Figure 4.12 displays the compaction curves of combined stabilizers.

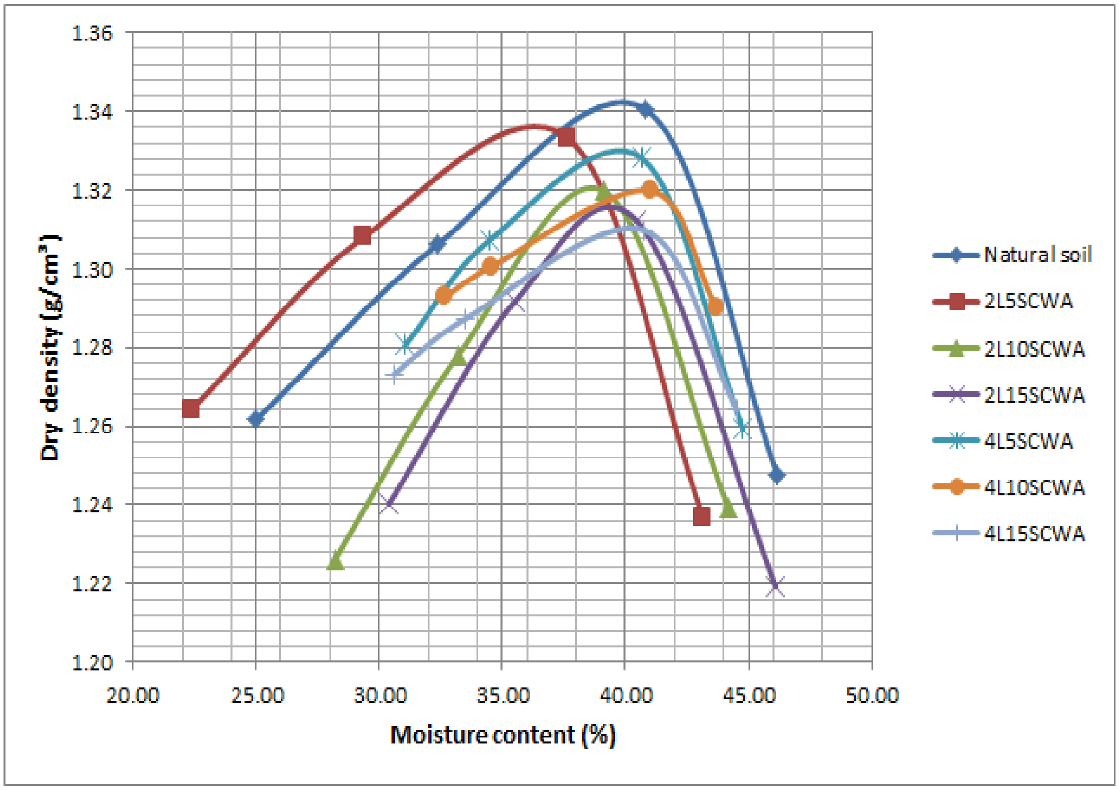


Figure 4.12. Effect of combined stabilizers on moisture density relationship

The mix requires minimum compaction effort to reach its optimum moisture content, as seen by the drop in maximum dry density. As was previously indicated, this is because stabilizer (SCWA) with lower specific gravity than natural soil has taken the place of clay particles.

4.3.5. Influences of SCWA and lime on California bearing ratio (CBR) values of soil

To investigate the effects of additives on the strength of native soil, soaked California bearing ratio tests were conducted for a period of 96 hours on both untreated soil and soil mixes with various percentages of synthetic cloth waste ash and lime.

4.3.5.1. Effect of SCWA on soil CBR values

The specimen was prepared by adding 5%, 10% and 15% of synthetic cloth waste ash to the native soil. The specimen was then subjected to a three-point CBR test. Figure 4.13 displays the final outcome.

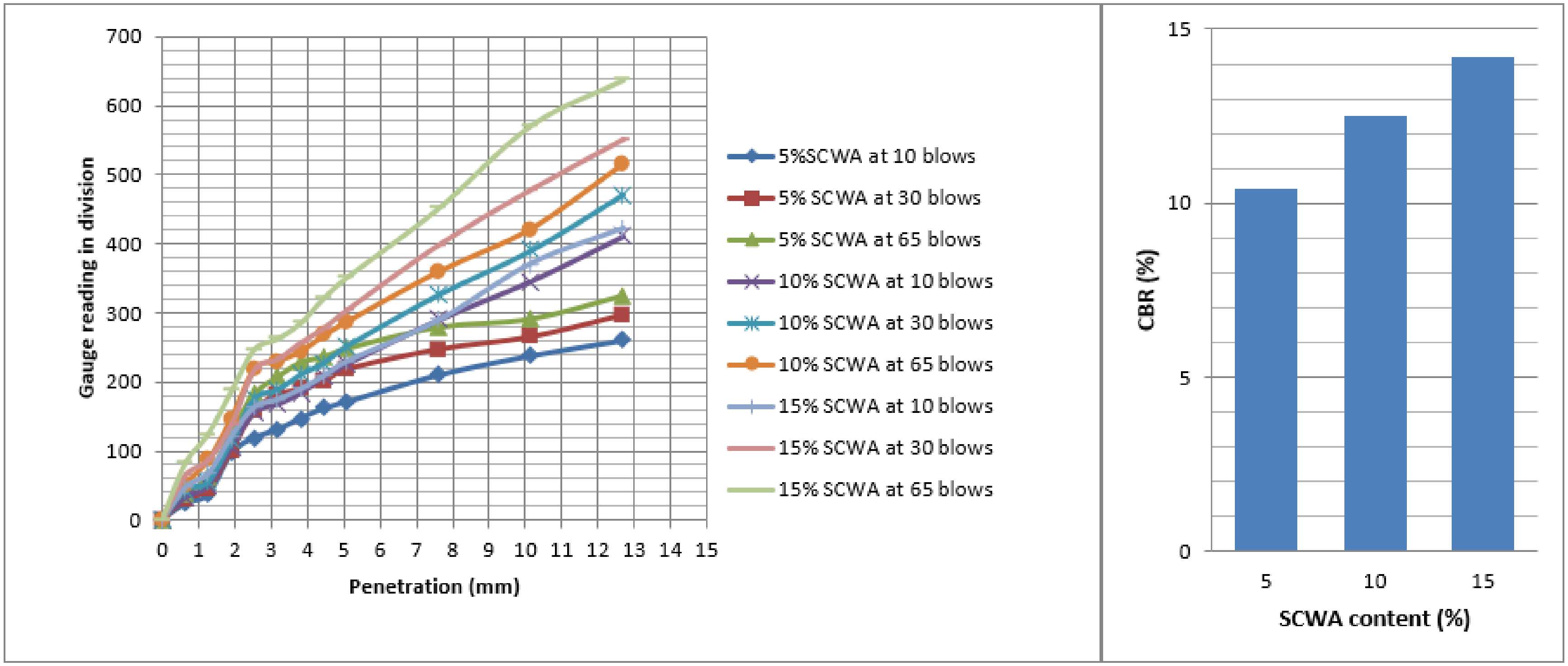


Figure 4.13. Effect of SCWA on CBR values

The CBR value increases to 10.4%, 12.5% and 14.2% with contents of 5%, 10% and 15% SCWA respectively, as seen in the above graph. According to the Ethiopian Roads Authority's (ERA, 2013)manual, the CBR value increased from 1.51% to 14.2% and happened at a concentration of 15% synthetic cloth waste ash. This value indicates that the specimen falls under the S4 sub-grade class. The reaction between clay particles and synthetic cloth waste ash produces cementious compound, which increases the CBR value.

4.3.5.2. Impact of lime on soil CBR values

Figure4.14 illustrates how adding 2% and 4% of lime to native soil affected the CBR values.

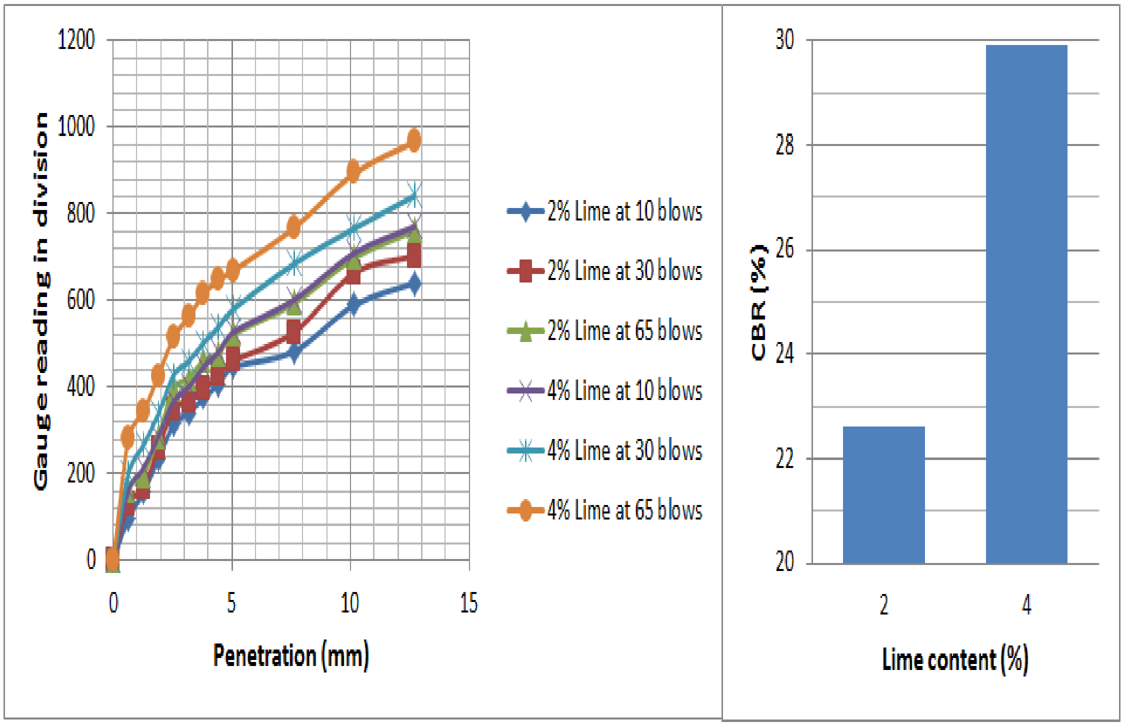


Figure 4.14. Effect of lime on CBR value

It has been noted that adding 2% lime increases the CBR value of natural soil from 1.51% to 22.6%, and adding 4% lime increases it to 27%. Based on ERA 2013 manual, this number enables the soil to be assigned to sub-grade class S5. The cementious compounds of calcium silicate hydrates and calcium aluminum hydrates that are formed by the chemical or pozzolanic reaction between lime and clay mineral elements interlock with the clay particles to produce a strong& long-lasting connection, increasing CBR value.

4.3.5.3. Effect of combination of SCWA and lime on CBR values of soil

This study has found that the combined impacts of stabilizers, as opposed to their solo effects, are more effective at increasing soil CBR values. The greatest CBR value is obtained when 4% lime and 10% SCWA are combined (i.e., when 4L10SCWA), and it is found to be 31%. The CBR value of 31%, as stated in the ERA manual 2013, indicates that the specimen is categorized under subgrade class of S5 and it can be utilized as good sub grade material. Table 4.9 provides a summary of the effects of individual and combined stabilizers on soil CBR values.

Table 4.9.Summary of effects of individual & combined stabilizers on CBR value

Contents of stabilizers (%)	CBR (%)
BC	1.51
BC+5A	10.4
BC+10A	12.5
BC+15A	14.2
BC+2L	22.6
BC+2L+5A	22.9
BC+2L+10A	24.9
BC+2L+15A	26
BC+4L	27
BC+4L+5A	27
BC+4L+10A	**31**
BC+4L+15A	25

The CBR swell of natural soil is decreased from 14.9% to 2.7% at the combination of 4L10SCWA. It is decreased by 81.88%. The test results of CBR swell are summarized in table 4.10.

Table 4.10. Summary of effects of individual & combined stabilizers on CBR swell value

Sample designation	10			30			65			Average CBR swell (%)
	Initial reading	final reading	CBR Swell (%)	Initial reading	final reading	CBR Swell (%)	Initial reading	final reading	CBR Swell (%)	
BC	108.7	124.4	14.44	108.5	126.4	16.50	111.3	126.7	13.84	14.9
5SCWA	110.2	125.1	13.52	107.3	120.1	11.93	114.5	127.6	11.44	12.3
10SCWA	109.6	123.5	12.68	111.7	124.2	11.19	107.1	118.2	10.36	11.4
15SCWA	109.8	123.4	12.39	111.5	126.4	13.36	117.9	129.5	9.84	11.9
2L	111.2	128	15.11	111.4	126.1	13.20	112.4	123.3	9.70	12.7
4L	107.6	122.5	13.85	109.2	122.2	11.90	112.3	121.2	7.93	11.2
2L5SCWA	111.5	128.3	15.07	113.2	127.7	12.81	117	127.5	8.97	12.3
2L10SCWA	108.7	124.9	14.90	112.5	126.1	12.09	114.1	123	7.80	11.6
2L15SCWA	114.3	131.1	14.70	115.3	129.2	12.06	118.3	127.3	7.61	11.5
4L5SCWA	110.5	124.6	12.76	118	129.5	9.75	119.2	124.4	4.36	9.0
4L10SCWA	114.7	119.1	3.84	117	121.1	3.50	119.4	120.2	0.67	**2.7**
4L15SCWA	110.4	117.1	6.07	115.7	121.9	5.36	111	113.5	2.25	4.6

4.3.6. Influences of SCWA and lime on Unconfined Compressive Strength of soil

The unconfined compressive strength test was performed on both treated and untreated naturally-occurring soil under both cured and uncured conditions. The cured conditions are at 3, 7 and 28 days of curing periods.

4.3.6.1. Effect of SCWA on UCS of soil

The UCS test was carried out for the remolded samples of soil mixed with 5%, 10% & 15% SCWA at curing times of 0, 3, 7 and 28 days. The samples were sealed by plastic for 3, 7 & 28 days then they were extruded & prepared for the test.

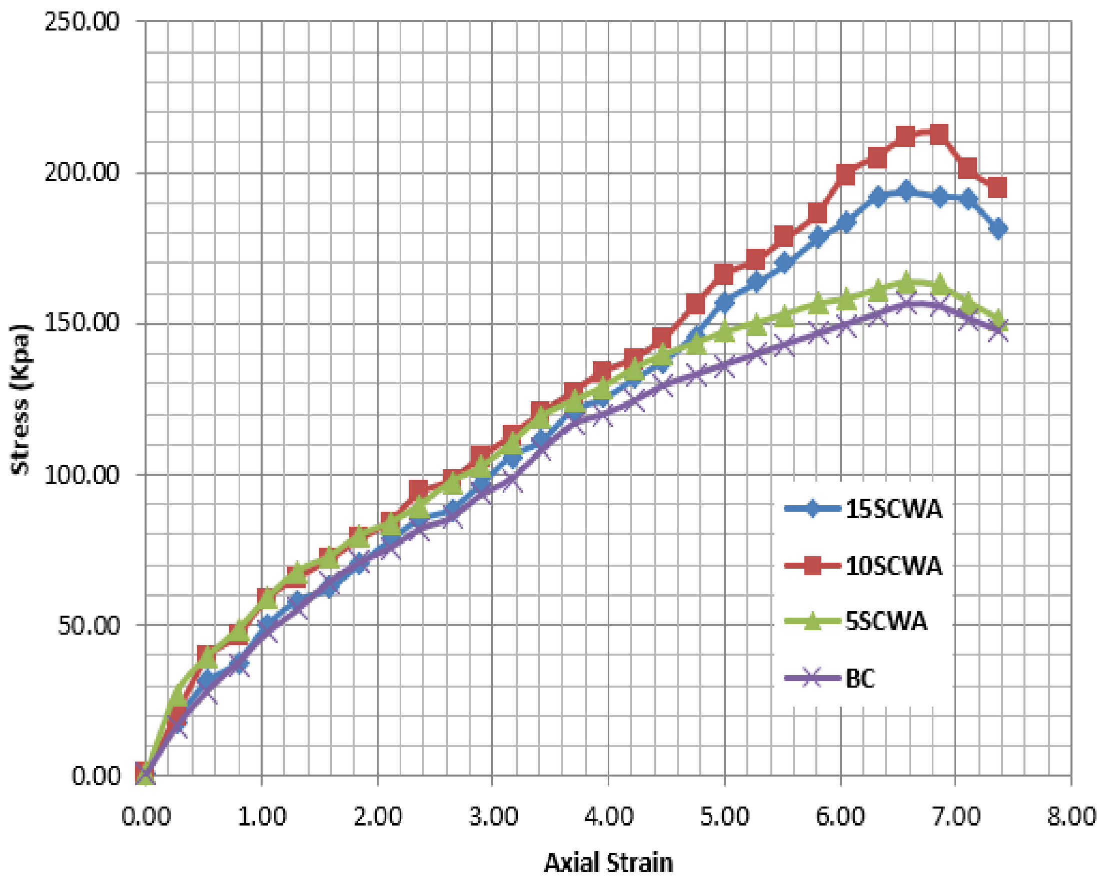


Figure 4.15. Effect of SCWA on UCS value

The stress-strain curve demonstrates that as the amount of synthetic fabric waste ash is increased by 5%, the unconfined compressive strength of soil somewhat improves. At 5% and 10% of SCWA, respectively, the UCS value of soil increased from 156.0Kpa to 163.44Kpa and 212.24Kpa, and it slightly decreases at 15% SCWA, which is 191.84Kpa. As a result of the weakening of the SCWA-soil link, the addition of SCWA had no effect on the UCS value.

4.3.6.2. Lime's impact on soil's unconfined compressive strength

Lime was added to native soil by 2% and 4% by dry weight of soil. The UCS value of soil was increased from 156.01Kpa to 289.95Kpa and 315.06Kpaat 2% and 4% of lime contents. The effects of lime on UCS values of soil are shown in figure4.16.

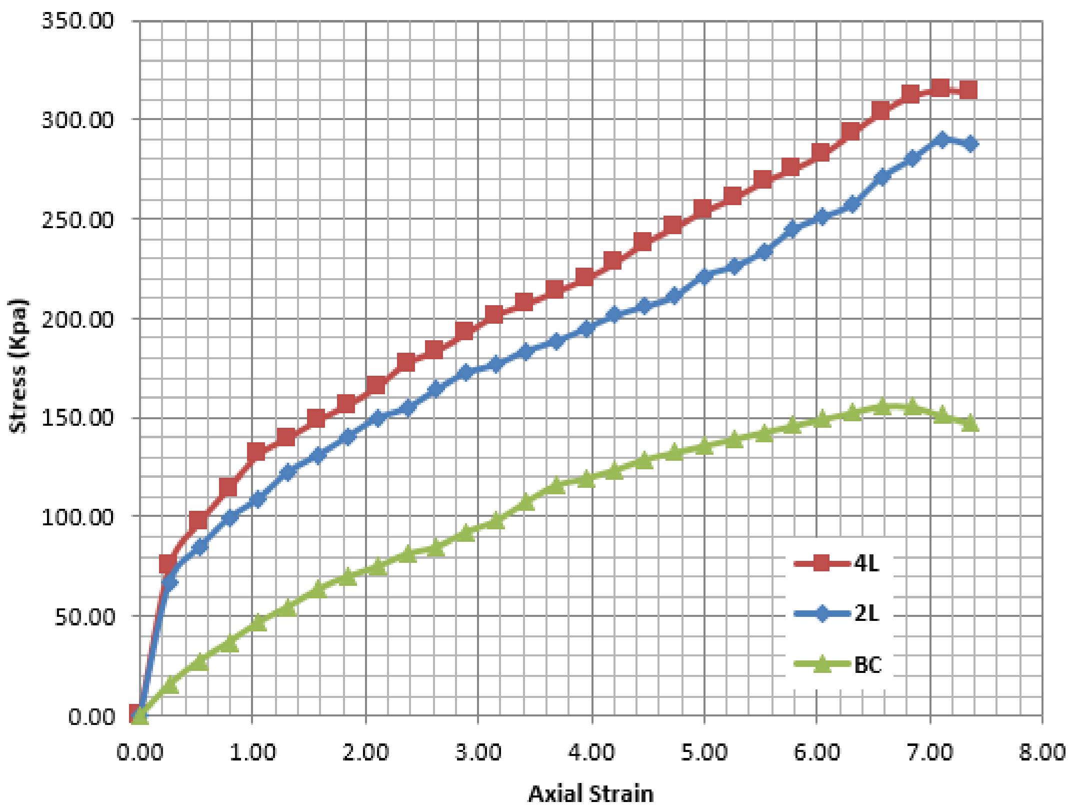


Figure 4.16. Effect of lime on UCS value

It has been found that lime addition significantly improves the UCS values of native soil. This is caused by the solid interlocking link between lime and soil, which raises the soil's capacity to withstand shear.

4.3.6.3. SCWA and lime's effects on soil's unconfined compressive strength

The SCWA content was varied by 5% (i.e., 5%, 10% and 15%) and the lime content by 2% (2% & 4%) for the UCS testing of native soil with SCWA and lime. Figure 4.17 displays the test findings. The blend of 4% lime and 10% synthetic cloth waste ash is found to give treated soil the highest value of unconfined compressive strength. The pozzolanic reaction of stabilizers with clay minerals causes them to interlock and increase the UCS value from 156.01Kpa to 424.93 Kpa at 28 days of curing period, which changes the consistency of soil to hard state.

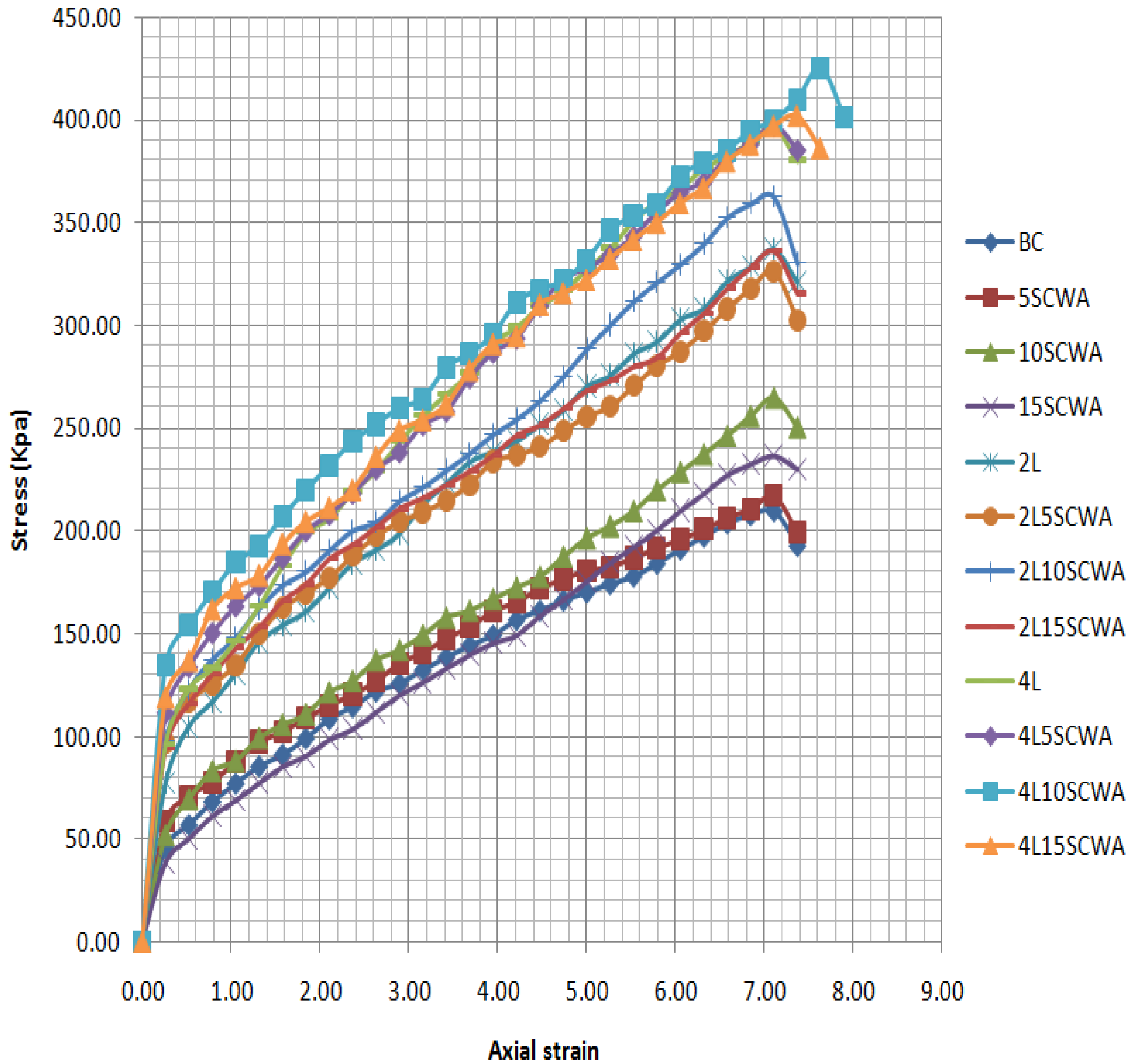


Figure 4.17. Effect of combined stabilizers on UCS value

4.3.7. Comparison with previous Findings

(Reshid, 2014) studied stabilization of black cotton soil with lime. He found that the addition of optimum lime content (i.e., 12%) resulted an improvement overall performance of the sub grade by increasing its strength and other workability criteria's. His result agrees with the findings of this study.

(Ramlakhan et al., 2013)investigated that effect of lime and fly ash on engineering properties of black cotton soil. They concluded that the maximum amount of 20% fly ash and 12% lime used effectively to improve all engineering properties of soil.

Table 4.11. Comparison of previous findings

Authors	Stabilizers	Optimum amount of stabilizers (%)	Laboratory test results											
			LL		PI		CBR		UCS		MDD		OMC	
			BC	Treated soil	BC	Treated soil	BC	Treated soil	BC	Treated soil	BC	Treated soil	BC	Treated soil
Reshid Musema	Lime	12%	64.14	45.66	33.11	15.1	2.04	8.92	276.21	791.73	1.51	1.46	25	29.3
Ramlakhan et al	Lime & fly ash	20FA12L	38.9	43.2	24.5	19.03	2.166	7.99	-	-	1.76	1.643	15.73	16.58
This study	SCWA & Lime	4L10SCWA	87.43	50.53	51.49	18.81	1.51	31	156.01	339.22	1.344	1.32	40.2	40.8

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1. Conclusions

The results of various laboratory tests lead to the following findings.

- ✓ It was found that the expansive soil exhibits no significant improvement in its geotechnical properties with increasing quantities of synthetic textile waste ash stabilizing agent alone is ineffective. But as it is mixed with various amounts of lime, it starts to work.
- ✓ When the SCWA and lime contents rise, the soil's free swell and linear shrinkage values decreased surprisingly.
- ✓ The liquid limit and plasticity index values at the combination of 4L15SCWA dropped by 46.85% & 75.26%respectively; at 28 days of curing, according to the findings of the Atterberg limit test.
- ✓ According to the compaction behaviors, the optimum moisture content increased while the maximum dry density decreased. The combination of 4L10SCWA yielded the optimum moisture content.
- ✓ The CBR value of black cotton soil significantly improved with the combination of 4L10SCWA, going from 1.51% to 31%. The CBR swell changed noticeably, falling from 14.9% to 2.7% for the same mix of stabilizers.
- ✓ The combination of 4L10SCWA, which alters the soil's consistency, increased the uncured unconfined compressive strength of black cotton soil from 156.1 Kpa to 424.93 Kpa.
- ✓ It is found that curing time has crucial effect on the strength of black cotton soil.
- ✓ Throughout this study, it is found that the combination of 4L10SCWA is the optimum content of stabilizers which improves all engineering properties of the black cotton soil.

In general, it was found that using various percentages of SCWA and lime significantly improved the engineering qualities of black cotton soil based on experimental research.

5.2. Recommendations

- ✓ Despite the fact that waste cloths are a readily available resource, there aren't enough studies that use them as a stabilizing factor, either as reinforcement or in the form of ash. Therefore, it is advised that additional research be done by combining such waste products with various additives.
- ✓ Due to the lengthy nature of the pozzolanic reaction of lime, the impact of prolonged curing should be evaluated.
- ✓ This study evaluates the engineering properties of black cotton soil that was only collected from four locations. But, since these soil deposits are dispersed throughout many areas, it is advised to evaluate soils collected from numerous locations in order to obtain better results.
- ✓ It is well established that using more stabilizers together results in better outcomes. In order to maximize SCWA's effectiveness, it is wise to combine it with more stabilizing agents than two or three.

REFERENCES

Afrin, H. (2017). *A review on different types soil stabilization techniques*.International journal of transportation engineering and technology, 3(2), 19. https://doi.org/10.11648/j.ijtet.20170302.12

AlemayehuT. and Mesfin, L. (1999). *Soil mechanics*.

Ali, A. A.-R. & M. F. A. G.*Expansive soils recent advances in characterization and treatment*.

Amu, O. O., Bamisaye, O. F., & Komolafe, I. A. (2011). *The suitability and lime stabilization requirement of some lateritic soil samples as pavement*. 2(1), 29–46.

Arora, D. K. R. (2004). *Soil mechanics and foundation engineering*. In standard publishers distributors (Vol. 7, Issue 2, p. 903). https://doi.org/10.1016/0013-7952(73)90044-6

ASTM Standard D2216. (1998). *Standard test method for laboratory determination of water (moisture) content of soil and rock by mass.*ASTM International, January, 1–5. www.astm.org

Asuri, S., & Keshavamurthy, P. (2016). *Expansive soil characterisation* : an appraisal. INAE Letters,*1*(1), 29–33. https://doi.org/10.1007/s41403-016-0001-9

Bamrele, S. K., Pro, A., Kumar, P., & Agarwal, S. (2019). *Soil stabilization using waste clothes (cotton clothes and synthetic clothes)*.June, 3655–3661.

Barton, C. D., & Karathanasis, A. D. *Clay minerals*.

Best travel months.com,2022

Chen, F. H. (1975). *Foundations on expansive soils*. Elsevier scientific publishing company. https://doi.org/10.1016/s0376-7361(08)70127-4

Chen, F. H. (1988). *Foundations on expansive soils*.

Das, B. M. (2008). *Advanced soil mechanics* (Third). Taylor & Francis.

Dinku A.and E. N. (2014). *Investigation on the effects of combining lime and sodium silicate for expansive subgrade stabilization.* Ehitabezahu Negussie and Abebe Dinku School of Civil and Environmental Engineering Addis Ababa Institute of

Technology , Addis Ababa University Ehitabe. 31, 33–44.

Dixit, A., Nigam, M., & Mishra, R. (2020). *Effect of fly ash on geotechnical properties of soil.*International Journal of Engineering Technologies and Management Research, 3(5), 7–14. https://doi.org/10.29121/ijetmr.v3.i5.2016.62

ERA. (2013). VOLUME II *: Rigid pavement design manual*. *II*.

Ghais, A. A. (2015). *Fly ash utilization in soil stabilization Abstract :*May 2014.

Jones, L. D., Survey, B. G., & Jefferson, I. F. (2012). *Expansive soils* (Issue July 2015). https://doi.org/10.1007/978-3-319-12127-7

Little, D. N. (1995). *Handbook for stabilization of pavement subgrades and base courses with lime*. Lime association of Texas.

Murthy, V. N. *Advanced foundation engineering*.

Nnochiri, E. S., Ogundipe, O. M., & Emeka, O. (2018). *E ffects o f s nail s hell a sh o n l ime s tabilized l ateritic*. 30(June), 239–253.

Ramlakhan, B., Kumar, S. A., & Arora, T. R. (2013). *Effect of lime and fly ash on engineering properties of black cotton soil.* 3(11), 535–541.

Reshid. (2014). *Stabilization of expansive soils with lime (A Case study on the Adura-Burbey DS6 road segment).*

Sandyarani et al. (2018). *Stabilization of black cotton soil by using lime stone.* International Research Journal of Engineering and Technology (IRJET), 05(08), 694–698.

Sangmesh, & Sharanakumar. (2020). *Effect of waste cloth on the properties of black cotton soil.*International Journal of Innovative Research in Engineering & Multidisciplinary Physical Sciences, 8(4), 11–14. https://doi.org/10.37082/ijirmps.2020.v08i04.003

Uge, B. U. (2017). *Performance, problems and remedial measures for roads constructed on expansive soil in Ethiopia.* A review. Civil and Environmental Research, *9*(5), 28–37.

World weather online.com,2022

Wubshet, M., & Tadesse, S. (2014). *Stabilization of expansive soil using bagasse ash & lime*. 32(December), 21–26.

APPENDICES

Appendix A: Natural moisture content test results

Natural Moisture Content

Can no	Mass of Can (g), M1	Mass of Can+wet soil (g), M2	Mass of Can+dry soil (g), M3	Mass of Moist soil (g), Mm	Mass of Dry soil (g), Ms	Mc(%)
L3	36.20	62.10	55.10	25.90	18.90	37.04
P101	34.60	66.40	57.30	31.80	22.70	40.09
						77.13
					Average Mc=	38.56

Mass of dry soil, M_s= M3-M1

Mass of moist soil, M_m= M2-M1

Mass of water, M_w= (M_w/M_s)*100

Appendix B: Specific gravity of natural soil

Pycnometer no	Mass of Empty pycnometer (g), M1	Mass of pycnometer+dry soil (g), M2	Mass of pycnometer+soil+water (g), M3	Mass of pycnometer+water (g), M4	Gs
9	51.60	76.80	185.00	169.00	2.74
10	56.60	81.60	187.80	172.40	2.60
9'	51.60	76.80	183.80	167.80	2.74
					8.08
				Average Gs=	2.69

$$Specific\ Gravity, Gs = \frac{(M2-M1)}{(M4-M1)-(M3-M2)}$$

Appendix C: Specific gravity of Synthetic cloth waste ash (SCWA)

Pycnometer no	Mass of Empty pycnometer (g), M1	Mass of pycnometer+dry soil (g), M2	Mass of pycnometer+soil+water (g), M3	Mass of pycnometer+water (g), M4	Gs
10'	55.23	80.24	185.37	172.18	2.12
11	51.16	76.65	180.24	167.36	2.02
10''	56.69	81.68	185.54	172.52	2.09
					6.23
				Average Gs =	2.08

$$Specific\ Gravity, Gs = \frac{(M2-M1)}{(M4-M1)-(M3-M2)}$$

Appendix D:Grain size analysis of natural soil

Wet Sieve Analysis

Sample taken= 1000gm

Weight of oven dried soil=72.79

Sieve size	Mass of Sieve	Mass of Sieve+soil	Mass of Retained Soil	Percent Retained	Cumm % retain	Percent Passing percent finer
4.75	721.09	721.09	0.00	0.00	0.00	100.00
2.36	334.78	334.78	0.00	0.00	0.00	100.00
2	320.44	325.75	5.31	0.53	0.53	99.47
1.18	353.37	359.81	6.44	0.64	1.18	98.83
0.6	315.72	317.93	2.21	0.22	1.40	98.60
0.425	378.04	385.31	7.27	0.73	2.12	97.88
0.3	262.41	268.87	6.46	0.65	2.77	97.23
0.25	264.73	267.75	3.02	0.30	3.07	96.93
0.15	265.54	276.45	10.91	1.09	4.16	95.84
0.075	372.95	399.95	27.00	2.70	6.86	93.14
Pan	605.77	609.94	4.17	0.42	7.28	92.72
			72.79	7.28		

Hydrometer analysis

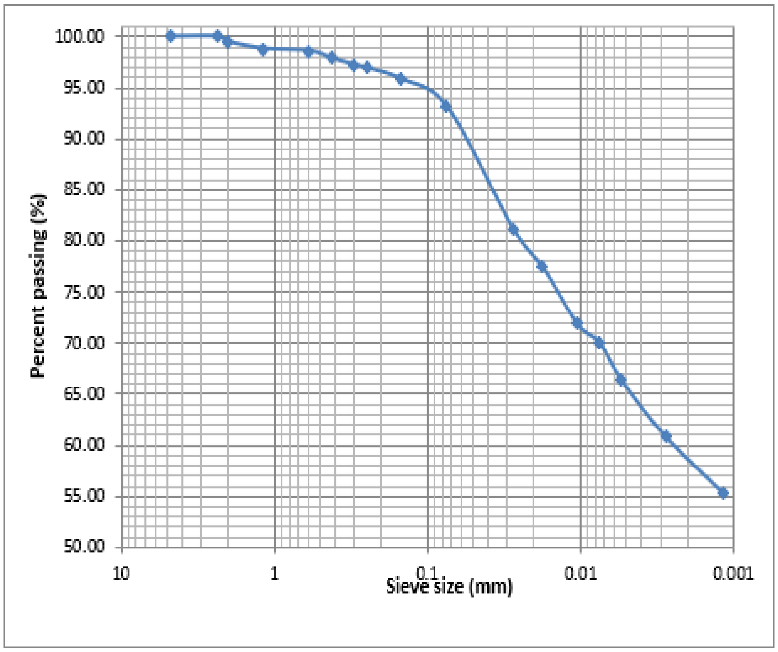
Sample taken= 50g

GS=2.69, Zero correction= 3, Meniscus correction=1

Elapsed Time (min)	Temprature (°C)	Actual Hydrometer reading, Ra	Correction for meniscus	L	K	D	Temprature correction factor	a	Corrected hydrometer reading Rc	% finer, P	% Adjusted finer, Pa
2	20	47	48	8.4	0.0135	0.027666767	0.00	0.99	44	87.12	81.14
5	20	45	46	8.7	0.0135	0.017807723	0.00	0.99	42	83.16	77.45
15	20	42	43	9.2	0.0135	0.010572606	0.00	0.99	39	77.22	71.92
30	20	41	42	9.4	0.0135	0.007556785	0.00	0.99	38	75.24	70.08
60	20	39	40	9.7	0.0135	0.005428052	0.00	0.99	36	71.28	66.39
240	20	36	37	10.2	0.0135	0.002783096	0.00	0.99	33	65.34	60.86
1440	20	33	34	10.7	0.0135	0.001163709	0.00	0.99	30	59.40	55.32

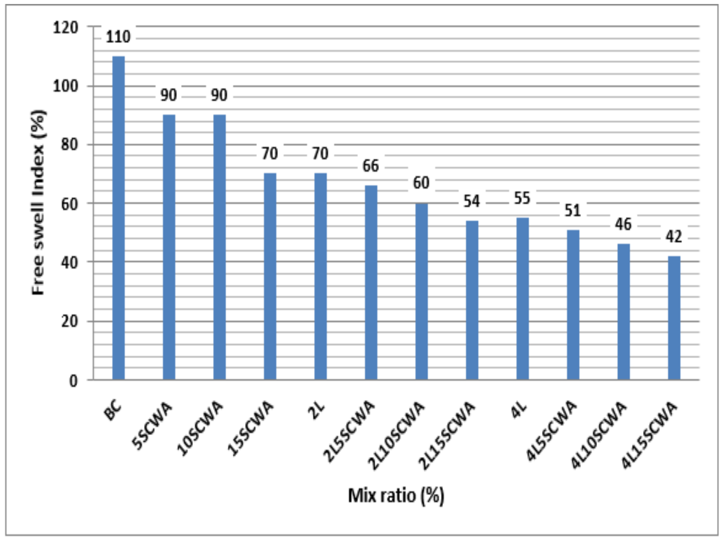
Sieve & Hydrometer Analysis combined curve

Sieve size (mm)	% Passing (%)
4.75	100.00
2.36	100.00
2	99.47
1.18	98.83
0.6	98.60
0.425	97.88
0.3	97.23
0.25	96.93
0.15	95.84
0.075	93.14
0.028	81.14
0.018	77.45
0.011	71.92
0.008	70.08
0.005	66.39
0.003	60.86
0.001	55.32



Appendix E: Free swell test results

Mix ratio (%)	Free swell index
BC	110
5SCWA	90
10SCWA	90
15SCWA	70
2L	70
2L5SCWA	66
2L10SCWA	60
2L15SCWA	54
4L	55
4L5SCWA	51
4L10SCWA	46
4L15SCWA	42

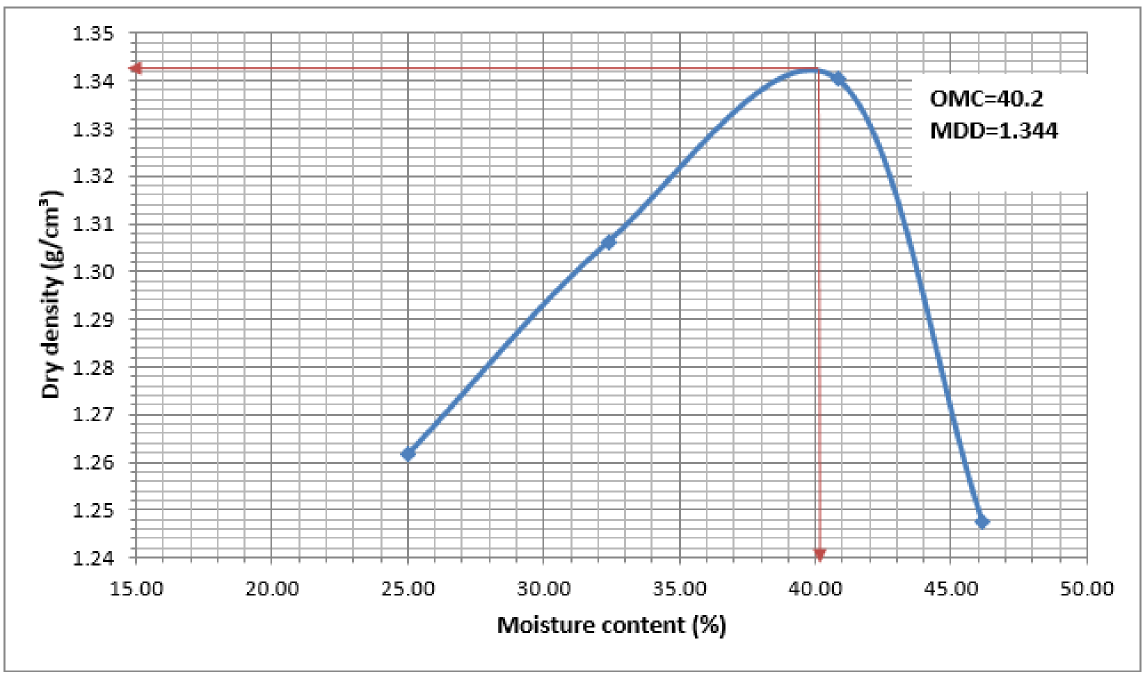


Appendix F: Shrinkage limit test results

Mix ratio	Trial 1		Trial 2		Average linear Shrinkage
	Initial length before oven dry (cm)	Final length after oven dry (cm)	Initial length before oven dry (cm)	Final length after oven dry (cm)	
BC	14	12	14	11.8	17.66
5SCWA	14	12	14	12.2	15.71
10SCWA	14	12.5	14	12.2	13.38
15SCWA	14	12.4	14	12.4	12.9
2L	14	12.6	14	12.7	10.67
2L5SCWA	14	12.8	14	12.9	8.95
2L10SCWA	14	13.1	14	12.7	8.55
2L15SCWA	14	12.9	14	12.9	8.53
4L	14	13	14	12.9	8.11
4L5SCWA	14	13.1	14	13.3	6.07
4L10SCWA	14	13.4	14	13.5	4.09
4L15SCWA	14	13.2	14	13.7	4.13

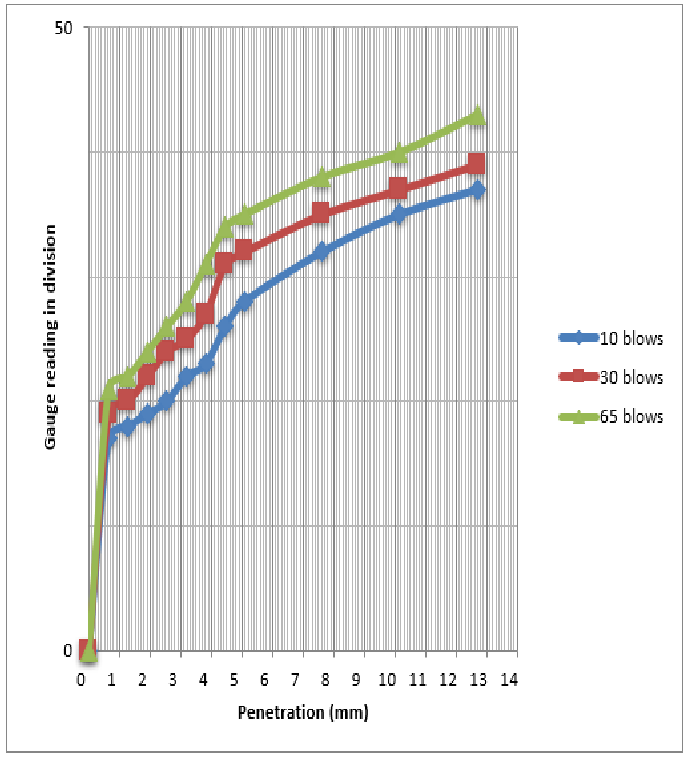
Appendix G: Modified compaction test result of natural soil

Can no	Mass of Empty mold (gm)	Mass of Mold+wet soil (gm)	Mass of Can (gm)	Mass of Can+wet soil (gm)	Mass of Can+dry soil (gm)	Mass of Wet soil (gm)	Mass of Dry soil (gm)	Mc (%)	Mass of wet soil (gm)	wet density	dry density
A	4548.50	6037.50	36.16	133.41	113.95	97.25	77.79	25.02	1489.00	1.58	1.26
B	4548.50	6180.80	35.74	125.98	103.91	90.24	68.17	32.37	1632.30	1.73	1.31
1	4548.50	6330.50	34.00	150.83	116.96	116.83	82.96	40.83	1782.00	1.89	1.34
L1	4548.50	6269.80	35.98	145.88	111.17	109.90	75.19	46.16	1721.30	1.82	1.25

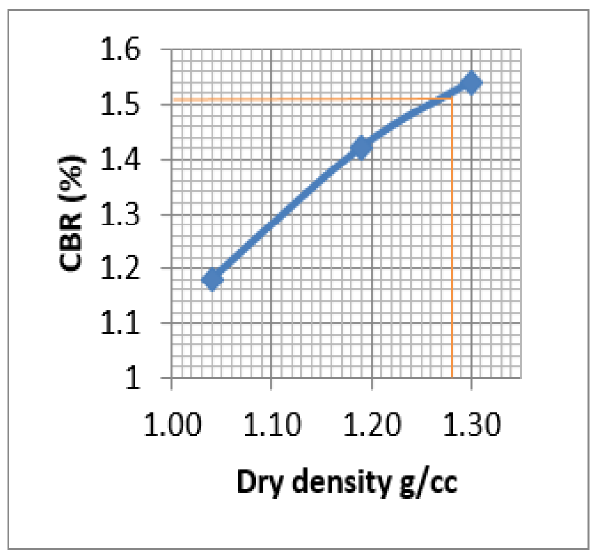


Appendix H: California bearing ratio (CBR) test result of natural soil

Penetration	Gauge reading @ 10	Gauge reading @ 30	Gauge reading @ 65
0	0	0	0
0.64	17	19	21
1.27	18	20	22
1.91	19	22	24
2.54	*20*	*24*	*26*
3.18	22	25	28
3.81	23	27	31
4.45	26	31	34
5.08	*28*	*32*	*35*
7.62	32	35	38
10.16	35	37	40
12.7	37	39	43

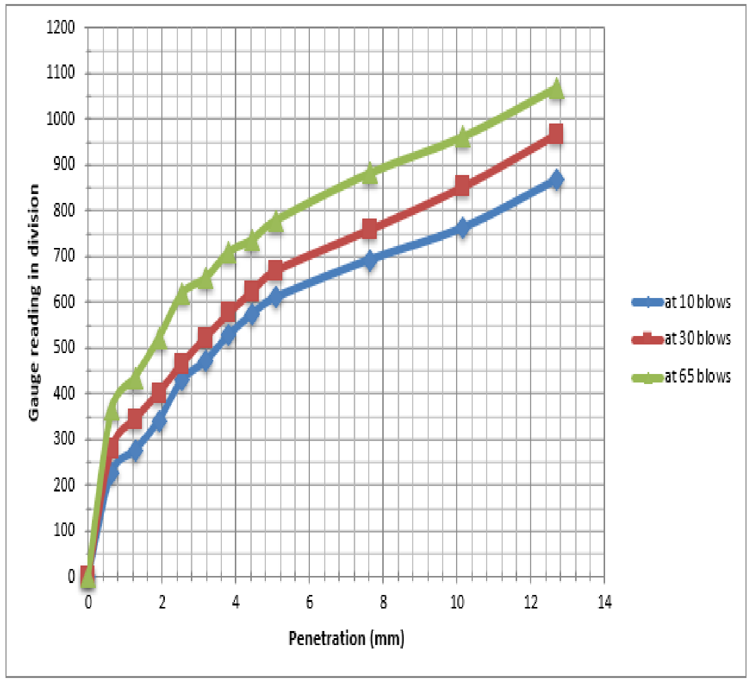


Dry Density (g/cc)	CBR (%)
1.04	1.18
1.19	1.42
1.3	1.54



Appendix I: California bearing ratio (CBR) test result of treated soil

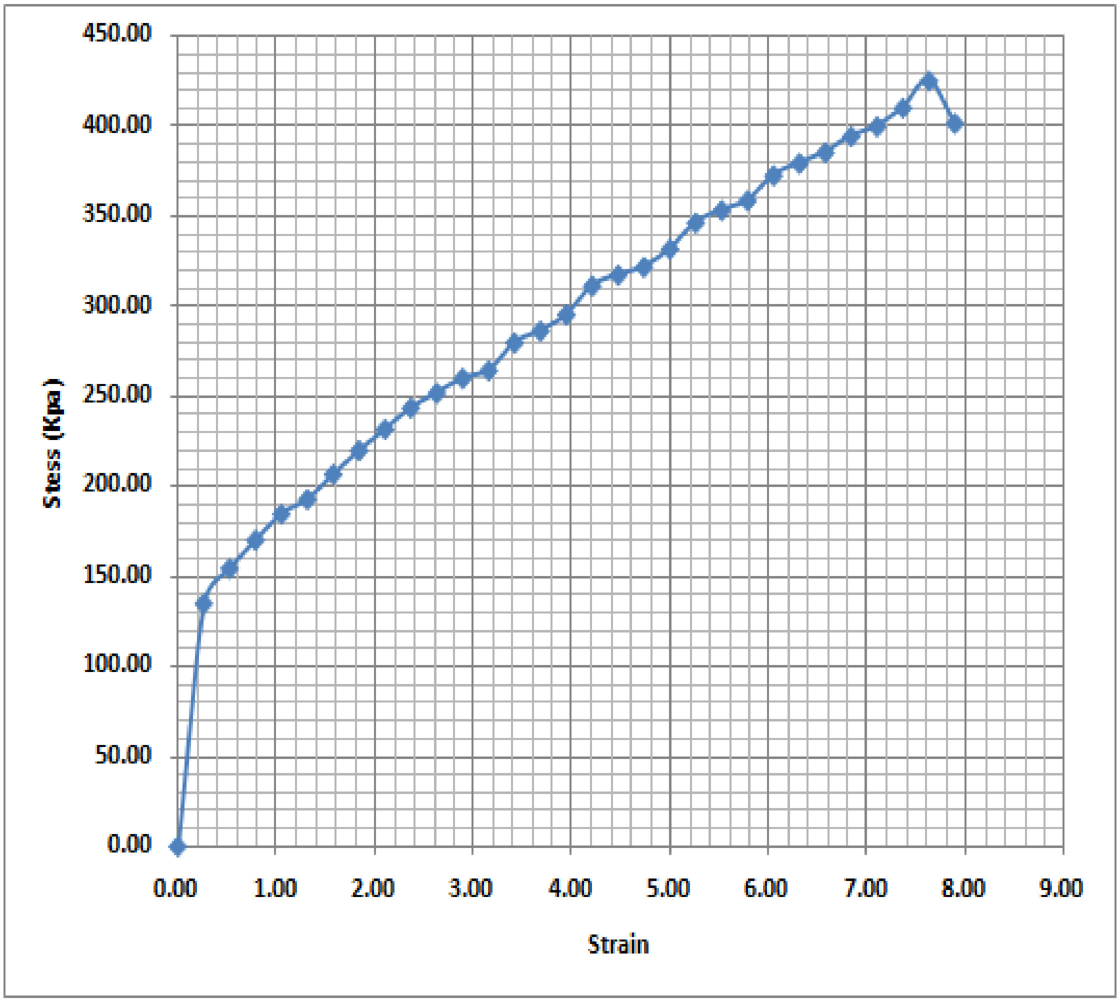
Penetration	Gauge reading @ 10	Gauge reading @ 30	Gauge reading @ 65
0	0	0	0
0.64	228	281	367
1.27	276	345	436
1.91	341	402	521
2.54	***429***	***466***	***619***
3.18	471	523	653
3.81	528	576	710
4.45	574	624	737
5.08	***610***	***669***	***779***
7.62	692	758	882
10.16	762	852	962
12.7	867	967	1069



Appendix J: UCS of treated soil by a combination of 4L10SCWAat 28 days of curing

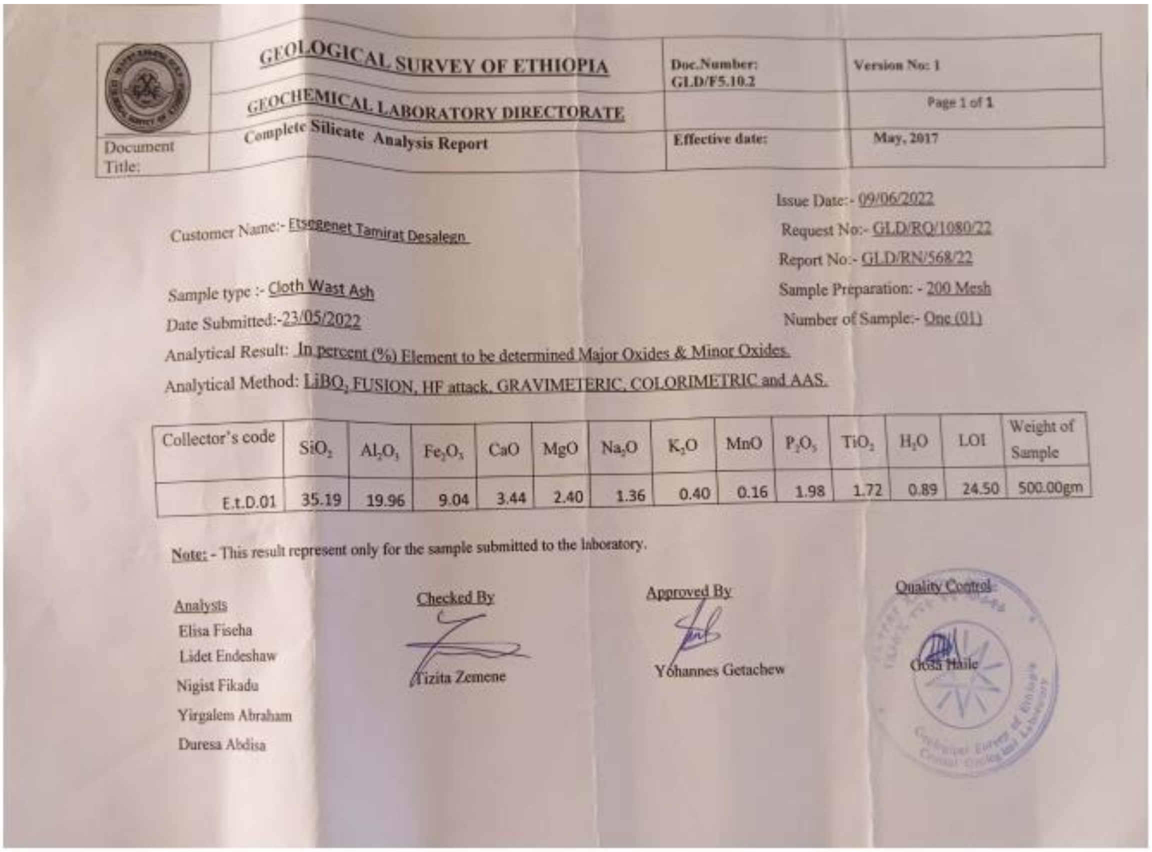
Diameter of Sample, mm	38	Ring Calibriation factor, N/di	44.93
Length of Sample, Lo (mm)	76	Wet unit weight, KN/m³	20.56
Water content (%)	36.98	Dry unit weight, KN/m³	15.01

Specimen Deformation, ΔL (mm)	Axial strain,e (%)=(ΔL/ Lo)*100	Proving Ring Reading,(div)	Applied load, P (KN)	corrected Area (m²) A'=Ao/1-e	Stress KN/m²,=P/A'
0	0.00	0	0.0000	0.0011	0.00
0.2	0.26	3.4	0.1528	0.0011	134.83
0.4	0.53	3.9	0.1752	0.0011	154.25
0.6	0.79	4.31	0.1936	0.0011	170.02
0.8	1.05	4.69	0.2107	0.0011	184.52
1	1.32	4.91	0.2206	0.0011	192.66
1.2	1.58	5.28	0.2372	0.0011	206.62
1.4	1.84	5.63	0.2530	0.0012	219.73
1.6	2.11	5.95	0.2673	0.0012	231.60
1.8	2.37	6.27	0.2817	0.0012	243.40
2	2.63	6.5	0.2920	0.0012	251.65
2.2	2.89	6.73	0.3024	0.0012	259.85
2.4	3.16	6.86	0.3082	0.0012	264.15
2.6	3.42	7.28	0.3271	0.0012	279.56
2.8	3.68	7.47	0.3356	0.0012	286.07
3	3.95	7.73	0.3473	0.0012	295.22
3.2	4.21	8.17	0.3671	0.0012	311.17
3.4	4.47	8.35	0.3752	0.0012	317.15
3.6	4.74	8.49	0.3815	0.0012	321.58
3.8	5.00	8.78	0.3945	0.0012	331.65
4	5.26	9.19	0.4129	0.0012	346.17
4.2	5.53	9.4	0.4223	0.0012	353.10
4.4	5.79	9.57	0.4300	0.0012	358.48
4.6	6.05	9.97	0.4480	0.0012	372.42
4.8	6.32	10.18	0.4574	0.0012	379.20
5	6.58	10.37	0.4659	0.0012	385.20
5.2	6.84	10.64	0.4781	0.0012	394.11
5.4	7.11	10.82	0.4861	0.0012	399.65
5.6	7.37	11.13	0.5001	0.0012	409.93
5.8	7.63	11.57	0.5198	0.0012	424.93
6	7.89	10.96	0.4924	0.0012	401.38



450.00
400.00
350.00
300.00
250.00
200.00
150.00
100.00
50.00
0.00
Stess (Kpa)
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
Strain

Appendix K: Chemical composition of SCWA (Geological survey of Ethiopia)



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

Issue Date:- 09/06/2022

Customer Name:- Etsegenet Tamirat Desalegn

Request No:- GLD/RQ/1080/22

Report No:- GLD/RN/568/22

Sample type :- Cloth Wast Ash

Sample Preparation: - 200 Mesh

Date Submitted:-23/05/2022

Number of Sample:- One (01)

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

Analytical Method: $LiBO_2$ FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS.

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI	Weight of Sample
E.t.D.01	35.19	19.96	9.04	3.44	2.40	1.36	0.40	0.16	1.98	1.72	0.89	24.50	500.00gm

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

Analysts
Elisa Fiseha
Lidet Endeshaw
Nigist Fikadu
Yirgalem Abraham
Duresa Abdisa

Checked By
Tizita Zemene

Approved By
Yohannes Getachew

Quality Control
Gosa Haile

Appendix L: Some Photos in laboratory


100 ml