THE EFFECT OF TEFF STRAW FIBER AND LIME ON STRENGTH AND COMPRESSIBILITY OF SOIL: THE CASE OF BLACK COTTON SOIL OF SENDAFA TOWN, ETHIOPIA

SIFAN TESHOME GETACHO

A Thesis Submitted to

The Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirements for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

July, 2021

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DECLARATION

I hereby declare that this Master Thesis entitled "The Effect of Teff Straw Fiber and Lime on Strength and Compressibility of Soil: the Case of Black Cotton Soil of Sendafa Town, Ethiopia" is my original work. That is, it has not been submitted for the award of any academic degree, diploma, or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

Sifan Teshome	___________________	_________________
Name of the Student	Signature	Date

RECOMMENDATION

I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "The Effect of Teff Straw Fiber and Lime on Strength and Compressibility of Soil: the Case of Black Cotton Soil of Sendafa Town, Ethiopia" prepared under my guidance by Sifan Teshome submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Civil Engineering (Specialization in Geotechnical Engineering). Therefore, I recommend the submission of the revised version of the thesis to the department following the applicable procedures.

Yadeta Chemdesa (Ph.D.)	______________________	____________________
Advisor	Signature	Date

APPROVAL SHEET

I, the advisor of the thesis entitled "The Effect of Teff Straw Fiber and Lime on Strength and Compressibility of Soil: the case of Black Cotton Soil of Sendafa Town, Ethiopia" developed by Sifan Teshome, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Yadeta Chemdesa (Ph.D.)	______________	______________
Advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis by Sifan Teshome have read and evaluated the thesis entitled "The Effect of Teff Straw Fiber and Lime on Strength and Compressibility of Soil: the case of Black Cotton Soil of Sendafa Town, Ethiopia" and examined the candidate during the open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Civil Engineering (Specialization in Geotechnical Engineering).

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

a_v - Coefficeint of compressibility

C_c - Compression Index

C_r - Recompression index

Cu - Undrained cohesion

C_v - Coefficient of consolidation

e_o - Initial void ratio

e -Void ratio

H_s - Height of solid soil sample

m_v - Coefficient of volume change

p - Pressure

qu - Unconfined compressive stress

t_{90} - Time at 90% of primary consolidation

σ - Normal stress

LIST OF ABBREVIATIONS

AASHTO…………………..American Association of State Highway and Transportation Officials

ASTM……………………..American Standard of Testing Materials

CBR……………………….California Bearing Ratio

CH…………………………Inorganic Highly Plastic Clays

CL …………………………Inorganic Clays of Low Plasticity

FS…………………………. Free Swell

LL…………………………..Liquid Limit

MDD……………………......Maximum Dry Density

MH…………………………..Inorganic Silt of High Plasticity

ML…………………………..Inorganic Silt of Low Plasticity

NMC………………………...Natural Moisture Content

OMC………………………...Optimum Moisture Content

OH…………………………...Organic Silts and Organic Clays of High Plasticity

OL……………………………Organic Silts and Organic Clays of Low Plasticity

PI……………………………..Plasticity Index

PL…………………………….Plastic Limit

TP…………………………….Test Pit

UCS…………………………..Unconfined Compressive Strength

USCS…………………………Unified Soil Classification System

ABSTRACT

In developing countries like Ethiopia, there is high development of the construction industry. However, some soils at the site of construction might not be appropriate for supporting heavy structures. Many stabilization methods and materials exist for improving the strength and compressibility characteristics of the soil. Investigation of waste materials as geo material over the ordinary and market-oriented stabilizers has both economic and environmental significance. In this work, the effect of teff straw fiber, which is one of the waste products, on strength and compressibility characteristics of black cotton soil was investigated. The combined effect of fiber reinforcement and lime treatment was also investigated. To achieve the objective of this research, the sample of black cotton soil was collected from Sendafa town and different laboratory tests were conducted as per the ASTM manual to classify soil. Samples of soil reinforced with various contents (0%, 0.5%, 0.75%, and 1% by weight) and lengths (20mm, 40mm, and 60mm) of teff straw fibers and mixed with various percentages of lime (2%, 4%, and 6%) were prepared to conduct compaction test, UCS test, and consolidation test. The result showed that MDD decreased and OMC increased with increasing both fiber and lime percent, while unconfined compressive strength increased for both uncured and cured conditions until 0.75% for teff straw fiber with each length and 4% (optimum %) of lime. The strength increased by more than double when the soil is reinforced with optimum percentage (0.75%) of fiber content and lime (4%) treated as compared to when the soil is stabilized by lime (4%) only. Similarly, fiber reinforcement and lime stabilization significantly reduced the compressibility and swelling potential of soil. The finding of the work suggests that a combination of fiber reinforcement and lime stabilization is more effective for ground improvement than lime stabilization alone.

Key words: Black cotton soil, Teff straw fiber, strength, compressibility characteristics

CHAPTER ONE

INTRODUCTION

1.1. Background

All engineering structures are built on the ground. In the old-time engineers avoid construction on soil with poor bearing capacity or problematic soil like soft soil and swelling soil, but currently because of the limitation of land they tried to enhance its properties instead of avoiding it. Problematic soil can be the reason for serious damage to structures like settlement and deformation. To regulate this problem various soil improvement techniques such as mechanical method, soil reinforcement, or adding some admixtures within the soil are familiar to enhance the engineering properties of soil. The suitability of particular methods depends upon the site condition and economy. Among all these methods, the most popular technique for ground improvement is the soil reinforcement technique (Mehta, 2017). The first purpose of reinforcing soil mass is to enhance its stability, increase its bearing capacity, and reduce settlements and lateral deformation (Hejazi, 2011).

Using natural fibers to reinforce soil is an old idea. However, several researchers evaluated that natural fiber such as bamboo, oil palm fiber, palm kernel fiber, and coconut palm fiber that are locally available, relatively less expensive, low energy demand throughout production, environmentally friendly, and 100% bio-gradable as possible substitutes for steel reinforcement and steel mesh (Nwoke, 2011).

Modification of black cotton soil by chemical admixtures is also a common method for stabilizing the swell-shrink tendency of expansive soils. The benefits of chemical stabilization are that they reduce the swell-shrink tendency of the expansive soil and additionally render the soils less plastic (Eberemu, 2013). However, chemical stabilization improves the soil under compression and it's a considerable downside in rising the tensile behavior of soil. The tensile behavior of the soil is improved naturally by adding fiber reinforcement in clay soil (P. Murthi, 2020).

However, the combined effect of chemical admixture and natural fibers on soil properties is not well explored. Additionally, most of the conventional stabilizing agents are relatively expansive to be implemented, hence it becomes economic and environmental significance using waste materials as geo material over the ordinary and market-oriented stabilizers in the civil engineering construction works. The present work aimed to investigate the influence of teff straw fiber, which is one of the waste products, on the strength and consolidation characteristics of black cotton soil. The combined effect of teff straw fiber and lime was also investigated.

1.2. Statement of the Problem

Black cotton soil causes many problems to road and building constructed on it. In the rainy season, it absorbs water heavily which results in swelling and softening of soil. In the dry season reduction in water content, shrinks and produces cracks (Jadhav, 2014). The soil around Sendafa town is plastic clay which has a high volume change capacity between wet and dry states (Zemedkun, 2015). For those reasons, the improvement of engineering properties of soil is much more important for making a structural foundation. The methods commonly used to improve engineering properties of soil are reinforcing the soil by using natural fibers. The natural fiber reinforcement causes significant improvement in shear strength, and other engineering properties of the clay soil (Maja, 2018). It is also a common practice to alter undesirable properties of expansive soils by lime treatment (P. Murthi, 2020).

Now a day, the high cost and lack of reinforcing steel in several parts of the world have directed to increasing interest in the possible use of other locally accessible materials for construction. Natural fibers are cheap and locally available in several countries (Tadesse, 2019). Teff straw fiber is the byproduct of teff which is left over after harvesting as waste material. Therefore, it can be used as a potential cost-effective soil reinforcement material.

1.3. Significance of the Study

To the best knowledge of the researcher, there is no research report on improving compressibility characteristics of black cotton soil by using teff straw fiber and the combined effect of teff straw fiber and lime. This study has a valuable significance to understand the effect of teff straw fiber and lime on the strength and compressibility behavior of black cotton soil around Sendafa town. Since teff straw fiber is cheap, locally available and byproducts using it as soil reinforcement can minimize the effective cost of soil reinforcement material.

1.4. Objectives of Study

1.4.1. General Objective

The main objective of this study is to determine the effect of teff straw fiber and lime on the strength and compressibility characteristics of black cotton soil.

1.4.2. Specific Objectives

To meet the general objective of this research, the following specific objectives were designed:

- To determine the index properties of the Sendafa soil
- To determine the engineering properties of samples of the soil before and after reinforcing with teff straw fiber and mixing with lime
- To identify the optimum percentage of the fiber and lime in the soil reinforced with teff straw fiber and stabilized by lime

1.5. Scope of the Study and Limitation

1.5.1. Scope of the Study

In this study, samples of black cotton soil and teff straw fiber were collected from Sendafa town. To sample these soils, the first area where black cotton soils were found was identified by reviewing research done on this area and reviewed the literature on field identification of black cotton soils. Then, two pits were excavated for soil sampling and different laboratory tests were conducted as per the ASTM manual to classify soil. After that, the worst problematic soil was selected for sample preparation on untreated soil, fiber-reinforced soil, and lime stabilized/fiber-reinforced soil.

First, samples of soil reinforced with different contents of teff straw fibers (0.5%, 0.75%, and 1%) and different lengths (20mm, 40mm, and 60mm) were prepared. Then, a compaction test and unconfined compressive test were conducted on it to identify the optimum length and optimum percentage of fiber mentioned during the preparation of lime stabilized/fiber-reinforced samples and consolidation test.

Second, samples of lime stabilized and fiber-reinforced soil were prepared with different percentages of lime (2%, 4%, and 6%) and optimum length and percentage of fiber. Then, a compaction test and UCS test were conducted on it and optimum lime content was identified. Additionally, the effect of lime on compaction characteristics and strength of reinforced soil was described. Then, this optimum lime was added on the various percentage of fiber (0.5%, 0.75%, and 1%), and the effect of percentage of fiber on compressibility characteristics of soil stabilized by lime were discussed. Additionally, for consolidation test soil reinforced with different percentages of teff straw fiber and optimum length were prepared. Generally, the MDD and OMC, unconfined compressive strength, and consolidation characteristics that are used to calculate settlement were measured.

1.5.2. Limitation of the Study

The durability of teff straw fiber under soil reinforcement has not been addressed in this study. As a reason, that surface treatment of fiber with different chemicals used to increase the durability of fiber is need another extra investigation.

1.6. Organization of the Thesis

The thesis consists of four chapters. The first chapter is the introduction part and it discusses general overview of expansive soil, objective, statement of the problem, scope and limitation, and significance of the study. The second chapter is a literature review about expansive soil, method of reinforcement,consolidation, and previous works in detail. Chapter three deals with materials and methods used in the study in detail, and chapter four comprises laboratory data collection and analysis of the laboratory result of soil reinforced and stabilized with teff straw fiber and lime respectively by different ratios. Lastly, conclusion and recommendation of the thesis work are drawn.

CHAPTER TWO

LITERATURE REVIEW

2.1. General

The development of infrastructures like buildings, highways, railways, and other structures in recent past years has resulted in a shortage of good quality land for construction projects. For instance, expansive and collapsible soils are challenges to engineers because of their peculiar behavior of high swelling and shrinkage action. As a result, the engineers are certain to adopt construction on inferior and weak soil by applying ground improvement techniques. By ground improvement techniques the strength of the soil increases, its compressibility reduces and the performance underneath applied loading enhances (Ayele, 2017).

The reinforced soil technique has been gaining popularity in the field of civil engineering due to its highly versatile and flexible nature. It is used to build retaining walls, dams, earth dams, the foundation of heavy structures on soft soil foundations, viaducts, and other objects. Reinforcement of soils with natural and synthetic fibers is potentially an effective technique for increasing soil strength (Anusha R., 2011).

Soil stabilization with lime is a chemical technique that results in marked improvements in strength, compaction, and compressibility characteristics; a reduction in expansive soil's swell percentage, swelling pressure, and plasticity, which can cause unfavorable brittle failure. To reduce the plasticity of expensive soil, both synthetic and natural fibers have been added to lime stabilized soil (Yixian Wang, 2019).

2.2. Origin and Properties of Black Cotton Soil

Tropical expansive soils are often called 'Black Cotton' soils. The latter name is believed to have originated in India where locations of these soils are favorable for growing cotton (Kassahun, 2006). These soils are residual soil, which has been formed from basalt or trap and contains the clay mineral montmorillonite that causes excessive swelling and shrinkage characteristics of the soil. In the rainy season, it absorbs water heavily which results in swelling and softening of soil.

It also loses strength and becomes easy to compress. Black cotton soil tends to heave during wet conditions. In the dry season reduction in water content, shrinks and produces cracks. The swelling and shrinkage of the black cotton soil can lead to damage to the foundations of the buildings and road pavements (Jadhav, 2014).

Figure 1: (a) Typical cracks in BC soil during the dry season, (b) Typical crack scene on roads around Ambo, and (c) Serious damage on a house constructed on BC soil (M.K. Atahu, 2018)

2.3. Distribution of Expansive Soil

Potentially expansive soils can be found almost anywhere in the world; it is widely spread throughout the five continents. Expansive soils cover large parts of the United States, South America, Africa, Australia, and Asia. More than 30 countries have been reported for the occurrence of expansive soils. In Ethiopia, expansive soils are observed in areas such as central Ethiopia, following the major truck roads like Addis Ambo, Addis Welliso, Addis Debre Birhan, Addis Gohatsion, and Addis Modjo. Also, they cover areas like Mekele, Gambella, and the most southern, south-west, and south-east part of the capital Addis Ababa areas in which the most major recent constructions are being carried out (Kassahun, 2006).

2.4. Identification and Classification of Expansive Soil

2.4.1. Field Identification

During the dry and wet seasons, it is easy to recognize expansive soils in the field. Its color varies from dark gray to black. In the dry season, the widest shrinkage cracks can be seen on the ground surface. These cracks reach 20 mm or more and go deep into the ground. A piece of dry black cotton soil requires a hammer to break down. A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail. During rainy seasons, these soils become very sticky and very difficult to traverse (Uba, 2011).

2.4.2. Laboratory Identification

There are three methods to identify expansive soils in the laboratory.

i. Mineral Identification

The swelling potential of any clay can be evaluated by the identification of the constituent mineral of that clay. The techniques that can be used x-ray diffraction, differential thermal analysis, dye adsorption, chemical analysis, and electron microscope resolution (Gebrekrstos, 2005). However, Zewdie (2004) described that these methods are not suitable for routine tests because of the following reason; they are time-consuming, require expensive test equipment and, the results are interpreted by specially trained technicians.

ii. Indirect Methods

These methods include simple soil property tests that a practicing engineer resorts to using for identifying expansive soil. Such tests are easy and can be performed in an average soil mechanics laboratory, and yield excellent indices of expansive properties. The test used here is the Atterberg limit value. In this method, measurement of the plasticity and the shrinkage characteristics of the soil are conducted for identification of soils and provide a wide acceptable means of rating by using liquid limit, plastic limit, shrinkage limit, free swell tests (Zewdie, 2004).

iii. Direct Methods

Direct measurement of expansive soils can be achieved by the use of a conventional one-dimensional consolidation test (Gebrekrstos, 2005).

2.4.3. Classification of Expansive Soils

Soil classification is an important aspect of laboratory tests, which tells the characteristic of the soil under interest. There are several classification systems based on identification tests. Among these, the Unified Soil Classification System (USCS) and the American Association of State Highway Transport Officials (AASHTO) method make use of the liquid limit and plasticity index of the soil. Geotechnical engineers generally prefer the USCS while the AASHTO classification is common in state and country highway departments (Uba, 2011).

2.5. Soil Reinforcement Methods

Soil reinforcement is defined as a method to improve the engineering characteristics of soil such as shear strength, compressibility, density, and hydraulic conductivity (Hejazi, 2011).

There are several reinforcement methods for stabilizing problematic soils. 1 - Physical methods such as vibration, thermo-electrical, freeze and thaw. 2 - Mechanical method using fibrous materials from Geo-synthetic family (Geo-grid, Geo-textile, Geocomposite, Geo-net, and Geo-cell) and natural fiber groups. 3 - Chemical method using conventional materials, enzymes & polymeric resins (Maja, 2018). However, some of those methods may have the disadvantages of being ineffective and/or expensive. Therefore, new methods to increase strength characteristics and reduce the swell behaviors of problematic soils are still being researched. It is emphasized that randomly distributed fiber-reinforced soils have recently attracted increasing attention in many geotechnical engineering applications, not only in a scientific research environment but also in the executive real field (Hejazi, 2011).

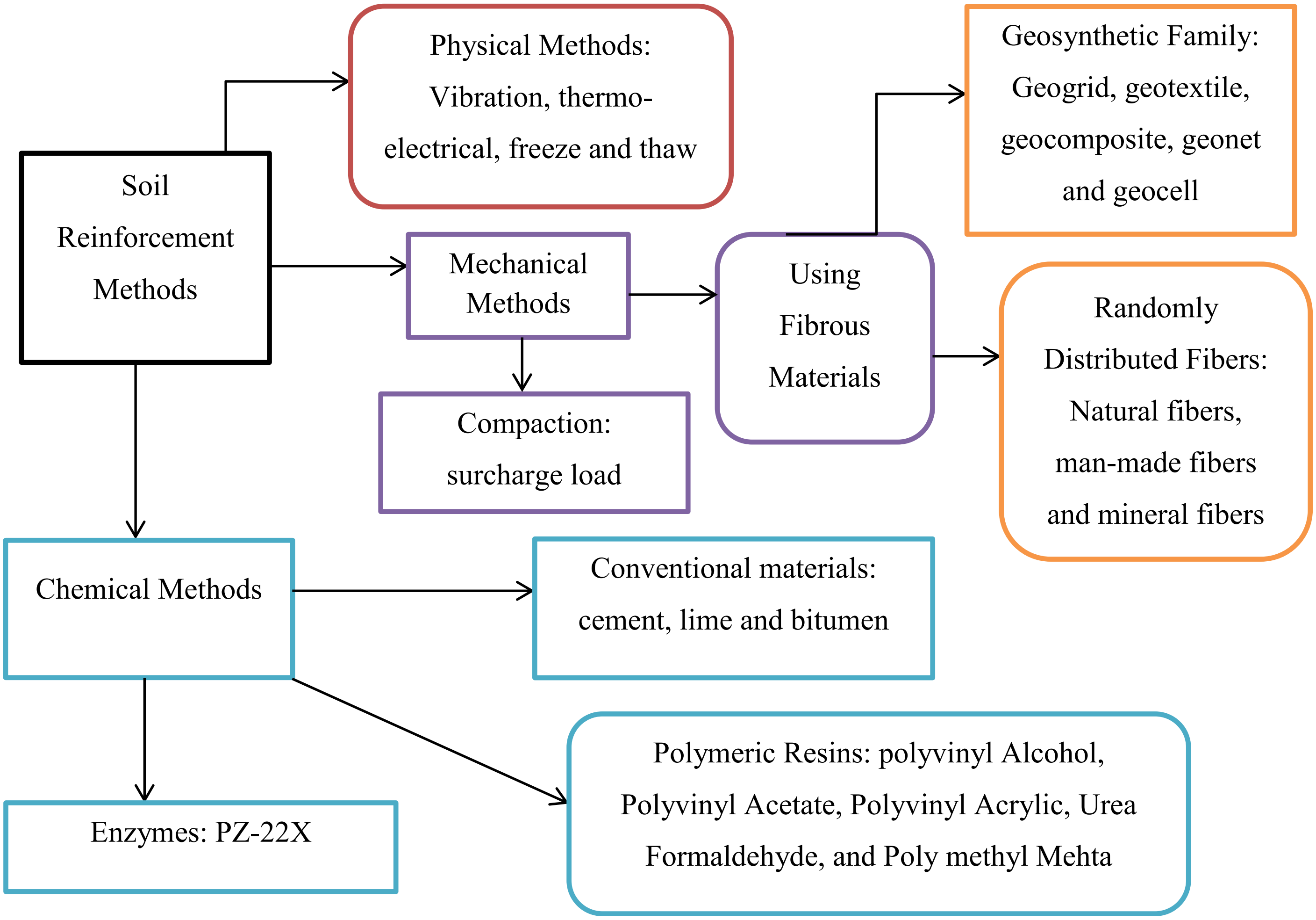


Figure 2: Different methods of soil reinforcement (Hejazi, 2011)

2.5.1. Natural Fiber as Soil Reinforcement

Natural fibers are byproducts and are found naturally. When these fibrous materials are randomly distributed in composite soil, they can reinforce problematic composite soils mechanically. The use of natural fiber in civil engineering for improving soil properties is advantageous because they are cheap, locally available, biodegradable, and eco-friendly. When fibers are randomly mixed with soil they increase the cohesion among the soil particles (Maja, 2018). Randomly distributed short fibers including natural, synthetic, and waste fibers have shown excellent potential for soil reinforcement applications, especially where the planar reinforcement using geosynthetics is not feasible due to space limitation, such as for the repair of a slope veneer or in partially failed slopes (Mirzababaei, 2018).

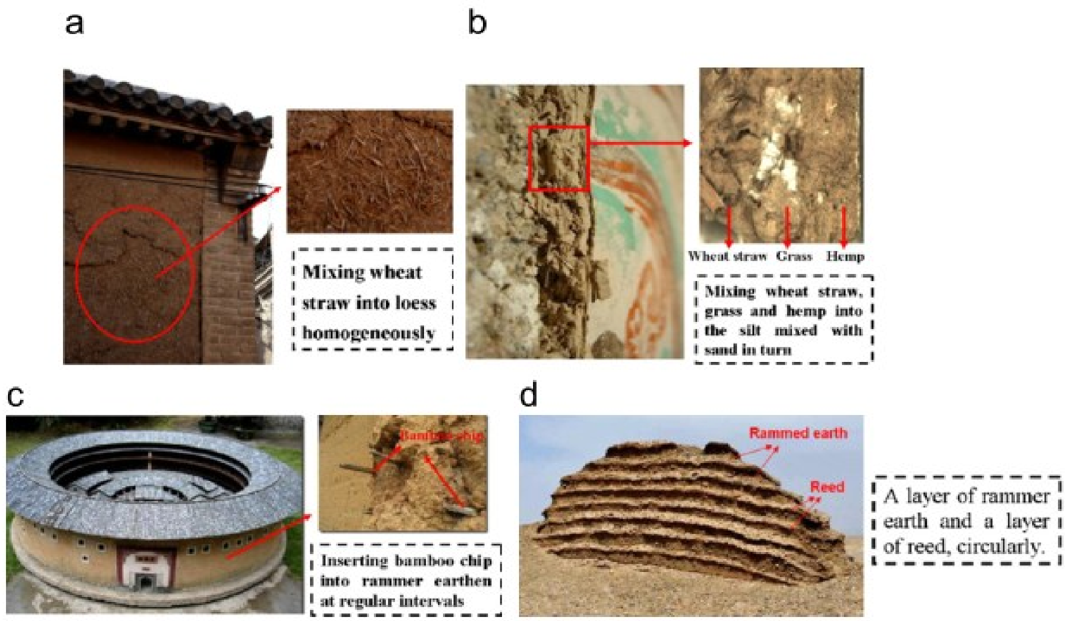


Figure 3: Traditional buildings constructed of reinforced soil (a) Rammed Residence, (b) Mogao Grottoes Mural, (c) Hakkas Earth Buildings, (d) Han Great Wall (Min Lia, 2012)

2.5.1.1. Properties of Teff Straw Fiber

Teff straw fiber is an organic material, which gets from the agricultural process. It is a durable, easily applicable, accessible, cheap, and not chemically hazardous material (Getnet Kassahun, 2019).

Teff is a small-grained cereal that has been grown as a food crop in East Africa for thousands of years. It is an essential food for the majority of the population in Ethiopia and Eritrea. It is cultivated in Ethiopia on about 2.59* 10^6 ha and occupies about 28% of the total crop area allocated to cereals delivering about 20% of the total cereal grain production and 17% of cereal crop residues production per year. The average teff grain yield of 1228 -3500 kg/ha is low compared to other cereals, but the by-product (teff stem/chid) is huge (Maja, 2018).

The teff straw fiber has the following properties (Maja, 2018):

- ✓ Its color is white and reddish-brown.
- ✓ It absorbs water from 40-50% (soaked for 24 hours) compared to the weight of its natural dry mass.
- ✓ It has an average length of 5cm and an average thickness of 1mm with an average hollow space (distorted or undistorted) of 0.3 mm.
- ✓ In dry condition, it breaks when bending about 270- 360 degree
- ✓ It attains high flexibility when wet.

2.6. Lime Stabilization

Lime stabilization is usually resorted to enhance the strength and reduce the compressibility of weak clay deposits. The safety of a structure is influenced by the soil compressibility as much as by the shear strength. One of the benefits of lime stabilization is that it imparts yield stress to the clay soils (Shivananda, 2005). When lime is used to treat highly plastic clay, the plasticity index decreases and the soil becomes friable and easy to be pulverized, having less affinity with water (Pankaj R. Modak, 2012). The clay soil stabilized by lime provides an abundance of calcium ions Ca^{2+} and magnesium ions Mg^{2+}. These ions tend to displace other common cations such as sodium Na^{+} and potassium K^{+}, in a process known as cation exchange.

Replacing sodium and potassium ions with calcium will significantly reduce the plasticity of the clay. The decrease in plasticity is usually associated with a decrease in the potential for swelling (Arvind Kumar, 2007).

2.6.1. Properties of Lime

It is a white, caustic, alkaline, crystalline solid at room temperature. It is relatively inexpensive and is manufactured by heating limestone, coral, seashells, or chalk, which are mainly $CaCO_3$, to drive off carbon dioxide. It has a melting point of $2600^{o}C$ (Ikeagwuani, 2016).

2.7. Shear Strength of Soil

The shear strength of soil is one of the most important geotechnical engineering properties. The bearing capacity of shallow or deep foundations, slope stability, retaining wall design, and pavement design are all influenced by the shear strength of the soil. The total stress on any plane can be determined by the normal stress, σ, which acts perpendicular to the surface, and the shearing stress which acts along the surface (Zewdie, 2004).

Zewdie (2004) has concluded moisture has a significant effect on the shear strength property of expansive soil. Thus, the shear strength of expansive soil reduces when the moisture content increased.

2.7.1. Unconfined Compression Test

The unconfined compression test is a special case of the triaxial compression test of soils where the compression and shear strengths of a soil prism, or cylinder, are measured under zero lateral stress ($\sigma_2 = \sigma_3 = 0$). The unconfined compression test is the simplest and quickest test for determining the unconfined compressive strength of the cohesive soils. The shear strength of cohesive soils (cohesion) is taken as equal to half the compressive strength. These values are used for checking the short-term stability of foundations and slopes, where the rate of loading is fast (Mir, 2015).

2.8. Consolidation

Consolidation is defined as the process of gradual transfer of an applied load from the pore water to the soil structure as pore water is squeezed out of the voids. The amount of water that escapes depends on the size of the load and the compressibility of the soil. The rate at which it escapes depends on the coefficient of permeability, thickness, and compressibility of the soil (Fikadu, 2015). Compressibility is an important parameter in soil investigation which indicates the rate of consolidation due to applied stress. This becomes more significant for geotechnical and geo-environmental engineering challenges such as the settlement calculation of foundation (Siddiqua, 2016).

2.8.1. Theories of Compression and Consolidation

Any structure constructs on the ground cause an increase of pressures on the underlying soil layers. The vertical compression of the soil under increased pressure is made up of the following components (Fikadu, 2015).

- A compression of solid mater which under usual loading is an account for very small compression.
- Compression of the pore fluid, which may be considered where the pores contain air, but negligible when the pore is filled with water.
- Reduction of the pore space by the expulsion of pore fluid, which forms the major component of the compression.

The settlement of the foundation in cohesionless soil and the elastic settlement of the foundation in clay can be assumed to occur as soon as the load is applied. The consolidation settlement of the foundation on clay will only take place as water seeps from the soil at the rate of depending upon the permeability of the clay. The settlement of saturated clay or organic soil can have three different components: immediate settlement, primary consolidation, and secondary compression (Fikadu, 2015).

1. Immediate settlement:

When a load is applied to partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to the compression of solid particles. The reduction in the volume of the soil just after the application of the load is known as initial consolidation or initial compression (Fikadu, 2015).

2. Primary consolidation:

The excess pore water pressure will decrease with time as water slowly flows out of the cohesive soil. This flow of water from cohesive soil (which has a low permeability) as the excess pore water pressures slowly dissipate is known as primary consolidation, or simply consolidation (Fikadu, 2015).

3. Secondary consolidation:

It occurs after essentially all of the excess pore water pressures have dissipated. The reduction in volume continues at a very slow rate even after the excess pore water pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in the volume is the called secondary consolidation (Fikadu, 2015).

2.8.2. Normally and Over Consolidated Soil

A normally consolidated soil is a soil that is subjected to an in situ effective overburden stress equal or greater than the pre-consolidated stress. Virgin consolidation settlement for applied stress exceeding pre-consolidated stress can be significant in soft and compressible soil such as plastic CH and CL clays, silts, and organic MH and ML soils. An over consolidated soil is a soil that is subjected to in situ effective overburden stress less than the pre-consolidated stress. Consolidation settlement will be limited to recompression from stress applied to the soil up to pre-consolidated stress (Nerea, 2004).

2.8.3. One-Dimensional Consolidation Test

The one-dimensional consolidation test, called the oedometer test, is used to find compressibility characteristics such as compression index (C_c), recompression index (C_r), coefficient of consolidation (Cv), coefficient of compressibility (av), and coefficient of volume change (mv). The compression index relates to how much consolidation or settlement will take place. The coefficient of consolidation relates to how long it will take for an amount of consolidation to take place (Fikadu, 2015).

The coefficient of compressibility (av) is represented by the slope of the void ratio versus applied pressure curves, an important indicator of the magnitude of soil compressibility. The coefficient of volume change (mv) is defined as the volume change per unit increase in effective stress for unit volume of soil (M.K. Atahu, 2018). The results of the typical oedometer test are usually presented in the form of an e-P, e-log P, and dial reading- time plots (Fikadu, 2015).

i. Compression index

Compression index Cc is numerically equal to the slope of the straight portion of the void ratio versus the log pressure plot from the e-log P curve. Its value is constant beyond the range of the recompression since beyond this point the plot of e against log P is a straight line.

Cc is an important index used to calculate the ultimate settlement of a foundation founded on a clay layer (Fikadu, 2015).

$$\mathrm{Cc} = \frac{\mathrm{e1} - \mathrm{e2}}{\mathrm{logP2} - \mathrm{logP1}}$$

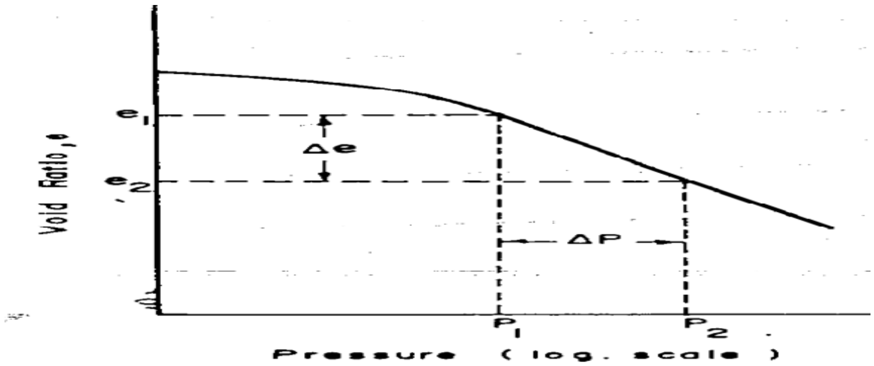


Figure 4: Determination of consolidation index (Fikadu, 2015)

ii. Coefficient of consolidation

The coefficient of consolidation, Cv is found experimentally from conventional laboratory 1-D consolidometer test results. Both Casagrande and Taylor's methods have been determined as follows (Fikadu, 2015).

1. The coefficient of consolidation Cv as determined by Casagrande's semi-logarithmic plot method is

$$C_v = \frac{(0.197)}{t50} H^2$$

Cv = coefficient of consolidation of stratum, in cm^2/sec

H = equivalent specimen thickness, in cm

t_{50} = time at 50 percent of primary consolidation, in sec

2. The Cv value as determined by Taylor's square root of the time fitting method is (0.848).

$$C_v = \frac{(0.848)}{t90} H^2$$

C_v = Coefficient of consolidation of stratum, in cm^2/sec

H = Equivalent specimen thickness, in cm

t_{90} = Time at 90 percent of primary consolidation, in sec

2.9. Related Previous Work

2.9.1. Teff Straw Fiber

(Maja, 2018) Have done an investigation on problematic sub-grade soil reinforcement using local natural Fibers such as Enset ventricosum and teff straw fibers. The researcher described that the effect of fibers percentage by weight in the soil on CBR, surcharged swell, dry density, moisture content, and their optimum improvements when reinforced lateritic soil. The percentage of individual fiber in the soil is taken as 0%, 0.25%, 0.5%, 0.75%, 1%, and 1.25% by air-dry weight. The result shows that MDD decreased and OMC increased with increasing fibers percent while CBR value increased until 1% for Enset fiber and 0.75% for Teff straw fiber with a fiber length of 6 cm. However, the degree of improvement of fiber mixed soil is 157 and 133 times greater than the soil without the fiber respectively. In general, MDD and CBR have a direct relation to each other. However, using fibers, MDD decreases with increasing CBR value. The effect of fiber is keeping CBR increasing even if the MDD is decreasing. This effect may be like the act of roots in the soil, binding soil particles by increasing shear strength among soil particles and fibers. Understanding the effect of fibers on shear strength needs further investigation.

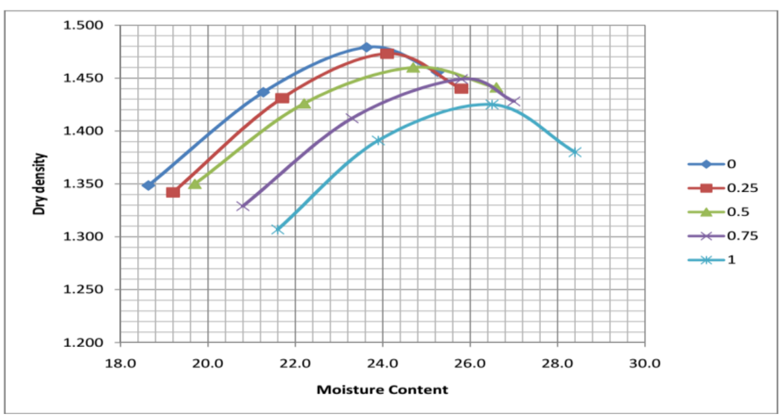
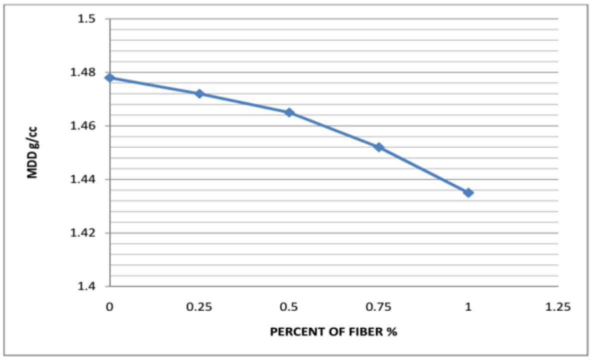
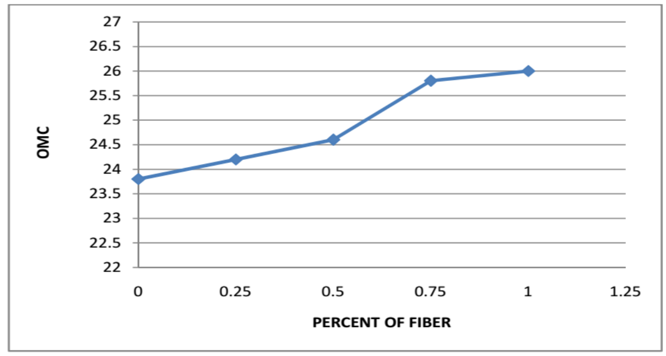


Figure 5: The relationship between dry density and moisture content at different teff straw fiber contents (Maja, 2018)



(a)



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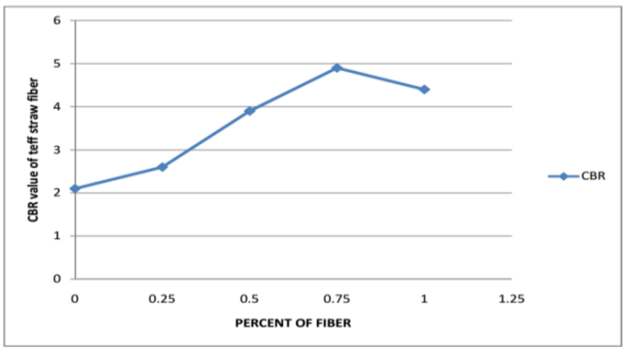

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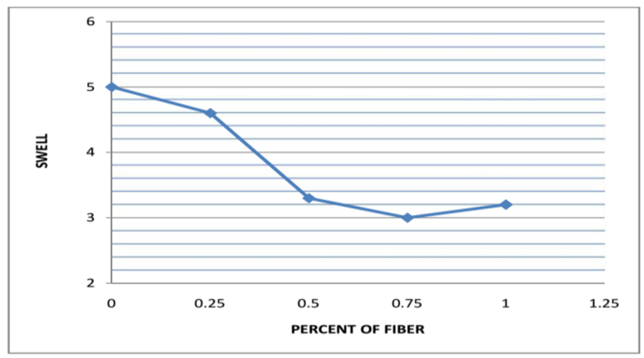


(d)

Figure 6: Relationship between percent of teff straw fiber and (a) MDD (b) OMC (c) CBR Value (d) Swell (Maja, 2018)

(Getnet Kassahun, 2019) Experiments have been carried out on the use of Eragrostis teff straw fiber reinforced and cement stabilized adobe unit specimens with a fiber proportion of 0.5%, 1%, 1.5%, 2% by weight, and 3.5% cement by weight. Compressive strength tests were performed after curing and drying for 14, 21, and 56 days. The test results show that compressive strength and water strength were improved when adobe units were reinforced with Eragrostis teff straw and stabilized with cement.

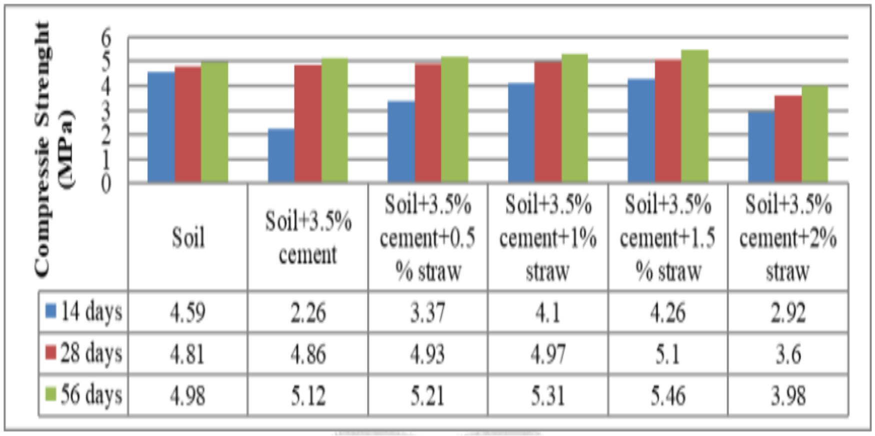


Figure 7: The compressive strength of adobe unit specimens for 14, 28, and 56 days of curing (Getnet Kassahun, 2019)

2.9.2. Lime

(R, 2017) Have done an investigation on stabilization of black cotton soil by using lime. The researcher conducted a compaction test, unconfined compressive strength test, and CBR test on soil treated by lime and untreated soil. UCS test was carried out by varying percentages of lime (3%, 6%, and 9%) and cured for a different period (3, 7, 14, and 28 days). The test was carried out in both soaked and unsoaked conditions. The CBR test was performed after 7 days of curing. From the test result, the investigator concludes that the maximum dry density decreased, and optimum moisture content remain almost the same with the addition of lime. The reduction of density is due to the resistance offered by the flocculated structure of the soil–lime mix against impact. Additionally, he concluded that the unconfined compressive strength of the black cotton soil treated with lime increased with an increase in the % of lime and curing period.

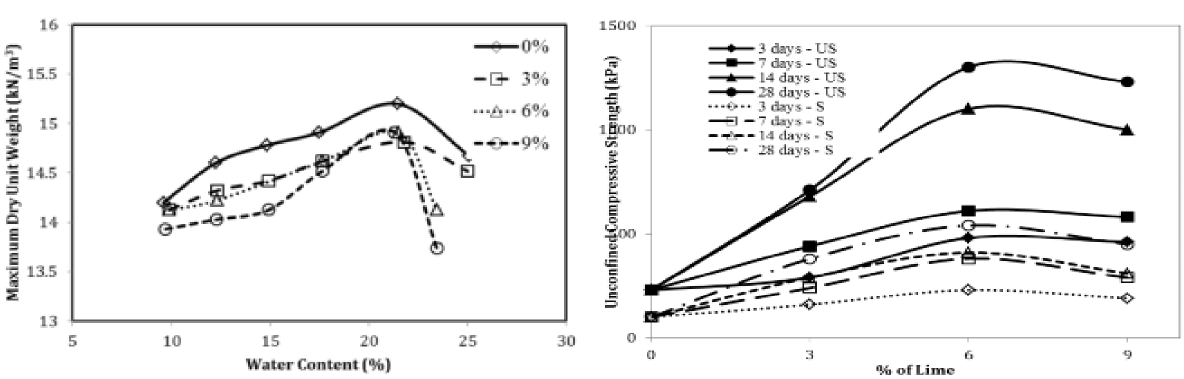


Figure 8: Variations in MDD and UCS with lime content and curing time (R, 2017)

(Nadgouda, 2010) have been investigated the effect of lime stabilization on the properties of black cotton soil by conducting Atterberg Limits, Standard Proctor Test, Differential Free Swell Test, Swelling Pressure Test, and California Bearing Ratio Test. The tests were performed on samples with lime content varying from 2.5% to 7.5% and then compared to the results obtained from the soil without lime. The Standard Proctor Test results show that when the lime content changes, the maximum dry density remains the same, and the optimum moisture content (OMC) decreased as lime content increased. The CBR value of the soil decreased with an increase in lime content up to 3.5%, then increase for further increased lime content. The optimum lime content observed was at about 3.5%. The author concludes that the addition of lime did not improve the compaction characteristics of the soil under investigation.

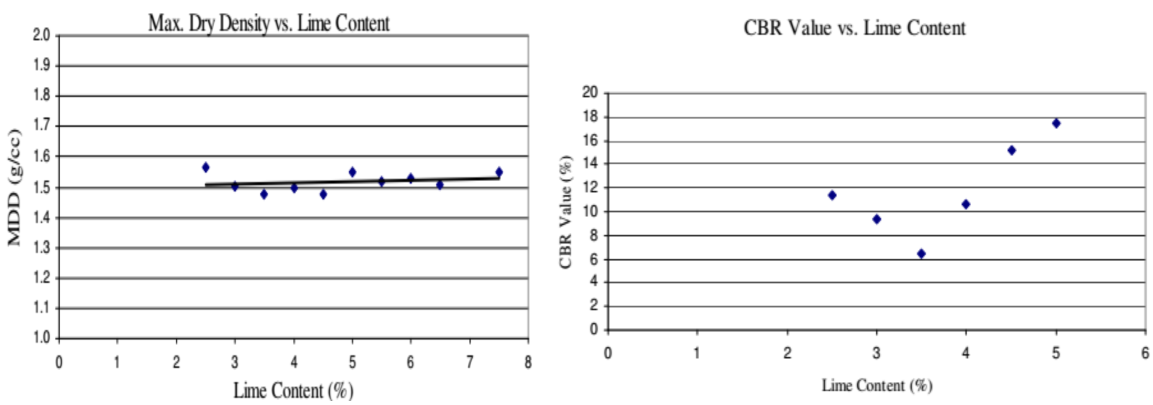


Figure 9: Variations in maximum dry density and CBR with lime content (Nadgouda, 2010)

(Yixian Wang, 2019) Have been studied the influence of wheat straw fiber reinforcement and lime stabilization on the mechanical behavior of clay. The samples were prepared on untreated soil, fiber-reinforced soil, lime-stabilized soil, and lime-stabilized/fiber-reinforced soil. Lime was added in three different amounts: 2%, 4%, and 6%, and 13 mm long wheat straw fiber slices with a cross-section of one-quarter that of the intact ones were mixed in at 0.2%, 0.4%, and 0.6% by dry weight of soil. Analysis of test results indicates that the combination of lime stabilized and soil fibers improved the strength and softening of the clay soil. At fixed fiber content, the cohesion decreased with a further increase in lime content above 4%. However, the angle of internal friction increased with increasing lime content from 4% to 6%. As a result, they got that the combination of 0.4% of fiber reinforcement and 4% of lime stabilization is the optimum combination to improve the shear strength of the soil.

Generally, the researchers conclude that a combination of fiber reinforcement and lime stabilization on shear strength parameters greater than those achieved when fiber and lime were used separately.

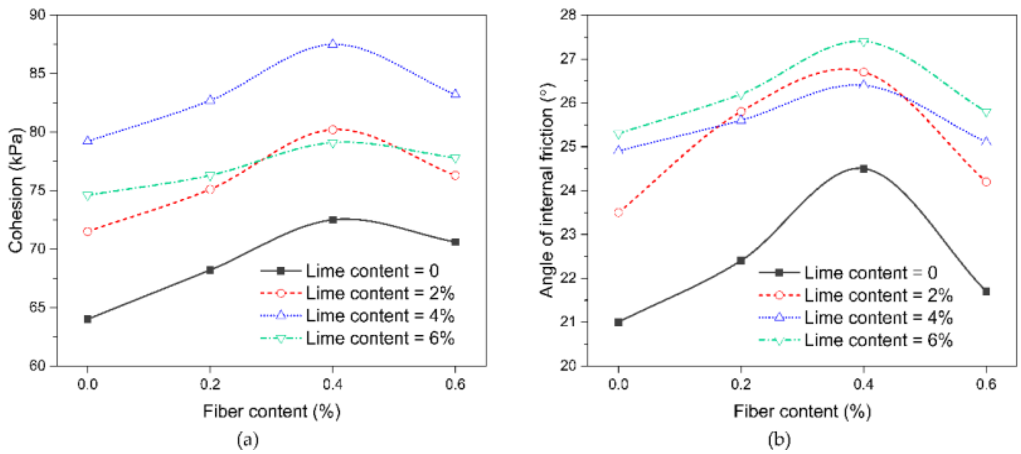


Figure 10: Shear strength parameters for soil samples with additives of different contents: (a) Cohesion and (b) Angle of internal friction (Yixian Wang, 2019)

(Ikeagwuani, 2016) Have done an investigation on the use of sawdust ash and lime on the compressibility characteristics of the black cotton soil. To achieve the object of this study one-dimensional consolidation test was conducted. Atterberg's limits, specific gravity, and particle size distribution were also performed on soil samples. The lime was added in various percentages of 0, 2, 4, 6, 8, and 10% by weight of black cotton soil to conduct Atterberg's limit test. The result shows that 4% lime addition to the black cotton soil produced the least plasticity index. During the consolidation stage, sawdust ash varied in 0, 4, 8, 12, 16, and 20% by weight of black cotton soil and lime of 4% by weight of the dry soil added to black cotton soil.

Analysis of the results shows that a combination of 16%sawdust ash (SDA) and 4%lime of the weight of black cotton were significant improvements in the compressibility characteristics of the black cotton soil.

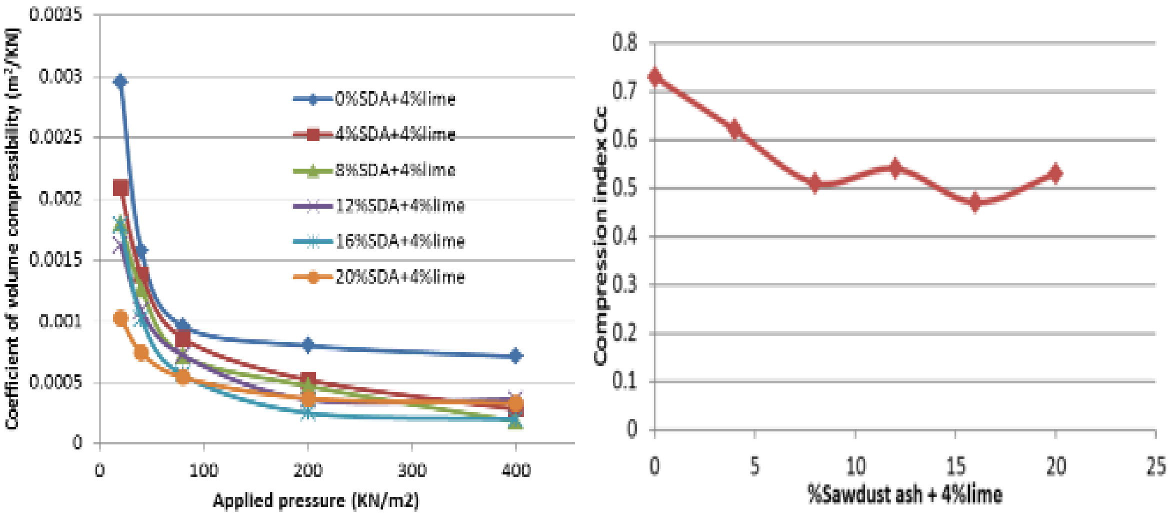


Figure 11: The effect of sawdust ash and lime on the coefficient of volume compressibility and compression index of black cotton soil (Ikeagwuani, 2016)

(Narasihma Rao, 2014) Have been studied the compressibility behavior of black cotton soil admixed with lime and rice husk ash. Standard Proctor tests and consolidation tests have been conducted on the soil with different percentages of lime and rice-husk ash separately and in combination in different proportions namely 1:1, 1:2, 1:3, and 1:4, ranging from 2 to 8% in increments of 2%. They found that the optimum moisture content increase and maximum dry density decrease with a percent increase of lime content. They also investigated that the compression index of the untreated soil was 0.266, which decreased on treatment with lime and RHA separately and in combination. Additionally, the authors conclude that the coefficient of consolidation increases with the increase of admixture content.

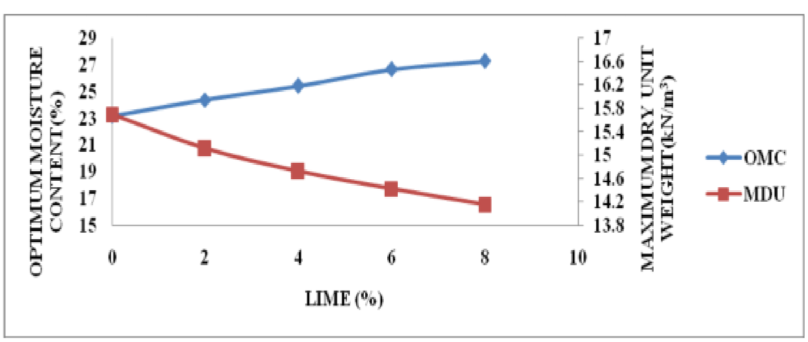
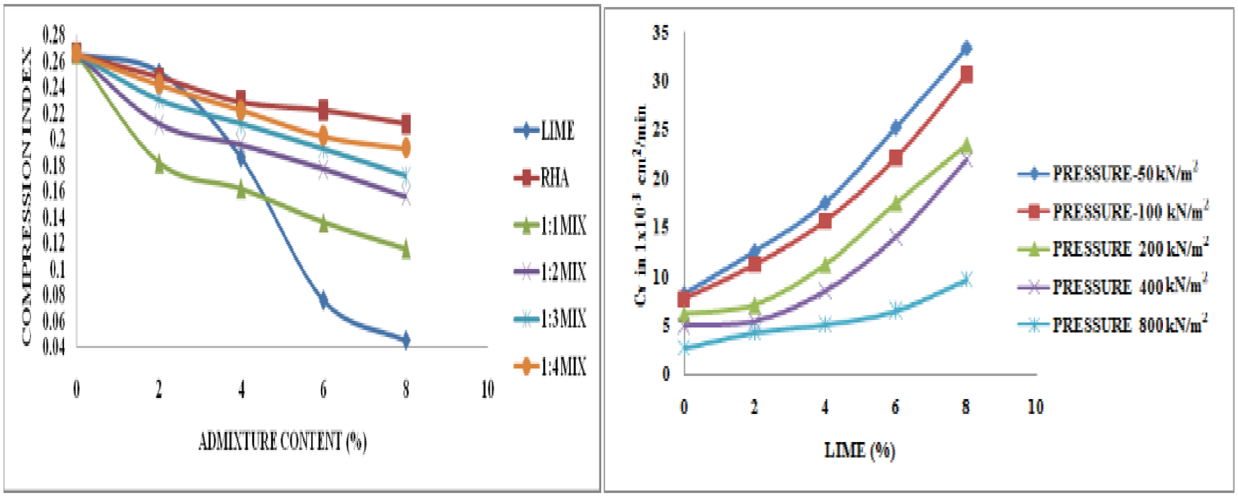


Figure 12: Effect of lime on maximum dry unit weight and optimum moisture content (Narasihma Rao, 2014)



a) b)

Figure 13: (a) Variation of compression index with percent admixture (b) Relationship between C_v and percent of lime (Narasihma Rao, 2014)

(Ankur Mudgal, 2014) Have been studied on the effect of lime and stone dust on some geotechnical properties of black cotton soil. First, an optimum value of lime was determined from standard Proctor tests performed on soil samples alone and their mixes with an increasing percentage of lime by weight (3, 6, 9, and 12%). After determining the optimum percentage of lime (=9%), Stone Dust was mixed by 5, 10, 15, 20, and 25% by weight in the lime-Black Cotton Soil and determined MDD and OMC, unconfined compression strength (UCS), and California bearing ratio (CBR) value of the new mix. The result shows that MDD of lime stabilized (=9%) black cotton soil increases up to 20% Stone Dust and further increase of the agent decreases the value. Additionally, for UCS and CBR, the strength increases up to 20% addition of Stone Dust in lime stabilized soil.

Table 1: Summary of pervious related works

No	Name of Researcher	Types of material	Material proportion (by weight)	Optimum percentage	Results
1	(Maja, 2018)	Enset straw fiber	0, 0.25, 0.5, 0.75, 1, and 1.25%	1%	MDD decreased and OMC and CBR increased as percentage of fibers increased.
		Teff straw fiber	0, 0.25, 0.5, 0.75, and 1%,	0.75%	
2	(Getnet Kassahun, 2019)	Teff straw fiber	0.5, 1, 1.5, and 2%	1.5% of fiber and 3.5% of cement	Compressive strength of adobe units increased with percentage of fiber and curing period.
		Cement	3.5%		
3	(R, 2017)	Lime	3, 6, and 9%	6%	MDD decreased, OMC the same and UCS increased with percentage of lime and curing time
4	(Yixian Wang, 2019)	lime	2, 4, and 6%	4%	a combination of fiber reinforcement and lime stabilization on shear strength parameters greater than those achieved when fiber and lime were used separately.
		Wheat straw fiber	0.2, 0.6, and 0.8%, length = 13mm	0.6%	
5	(Ikeagwuani, 2016)	Lime	2, 4, 6, 8, and 10%	4%	Coefficeint of volume compressibility and compression index decreased.
		Sawdust ash	4, 8, 12, 16, and 20%	16%	
6	(Narasihma Rao, 2014)	Lime and Rice husk ash	ranging from 2 to 8% in increments of 2%.	-	MDD decreased, OMC increased, compression index decreased and coefficient of consolidation increased with content of admixture
7	(Ankur Mudgal, 2014)	Lime	3, 6, 9, and 12%	9%	UCS and CBR, the strength increases up to 20% addition of Stone Dust in lime stabilized soil.
		Stone dust	5, 10, 15, 20, and 25%	20%	

CHAPTER THREE

RESEARCH METHODOLOGY

3.1. Description of Study Area

Sendafa is one of the fastest-growing towns and commercial centers in the Oromia Special Zone. It is 39km from Addis Ababa on Addis Ababa to Debreberhan road. Now a day, Sendafa is developing by constructing buildings, new roads, and industries. There is still a great potential for investment expansion in the zone mainly as a result of its location near to the capital. However, expansive soil covers different parts of the town (especially the industry zone and the newly to be establishing residential areas). From visual observation of the site soil, expansive soil-related cracks, and discussion with residents the soil in the town is to be expansive (Zemedkun, 2015).

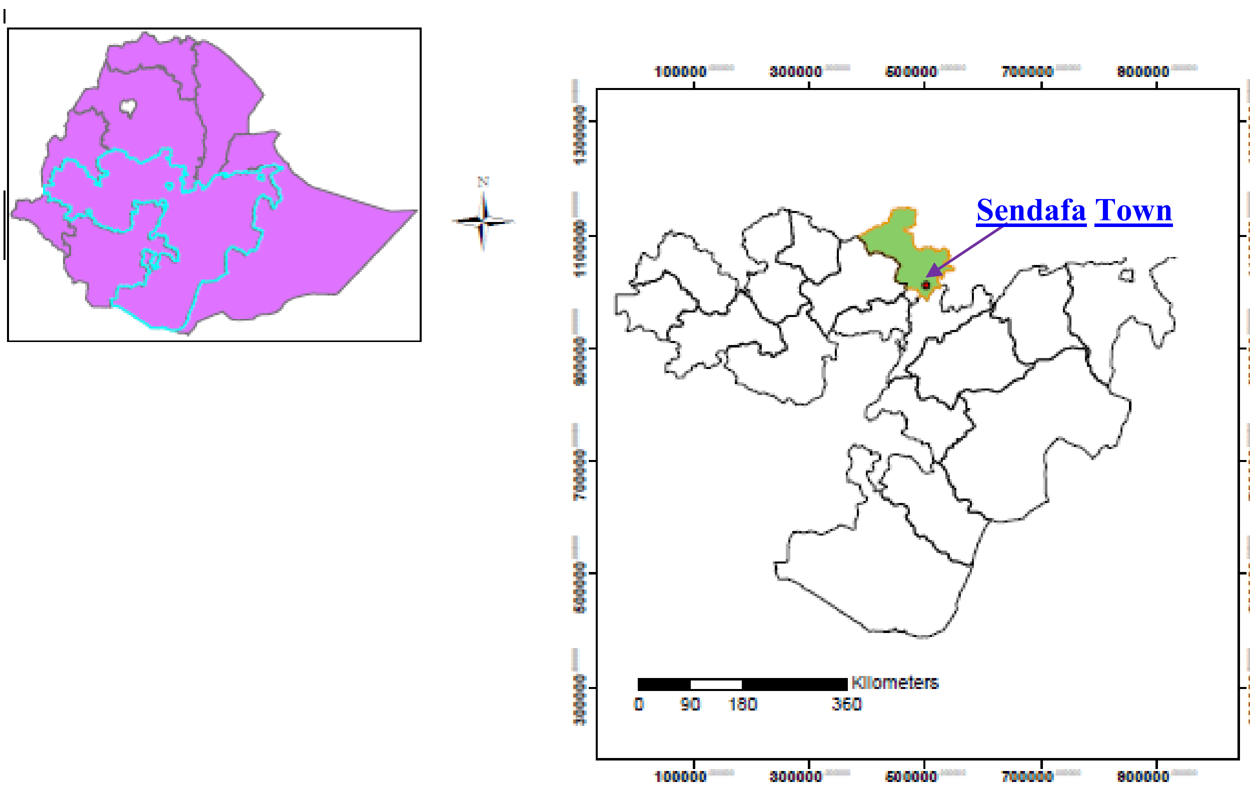


Figure 14: Location of the study area

3.2. Materials and Methods

The black cotton soil samples used in this investigation were collected from Sendafa town by excavating two pits. It was collected from 1.5m up to 1.75m depth. After 1.75m depth, the color of soil was changed from black to red color.

(a) (b)

Figure 15: (a) Typical crack of black cotton soil on the ground surface around study area (b) Test pit excavation-one

After pit excavations, to characterize the soils, laboratory tests such as natural moisture content, specific gravity, Atterberg limits, free swell test, particle grading, and hydrometer analysis of the soil were conducted on disturbed soil samples. In the end, depending on the result of the above tests the soil was classified by using the Unified Soil Classification System (USCS) and American Association of State High way and Transportation Officials (AASHTO) classification schemes. Teff straw fibers were also collected around Sendafa town from farming communities. Then physical characteristics of fiber such as water absorption, its average length, and angle of bending before the break were conducted in the laboratory. Lime was bought from a local market. Samples of soil reinforced with teff straw fiber, reinforced with fiber/stabilized by lime, and unreinforced soil were prepared to conduct laboratory tests such as compaction test, UCS test, and consolidation test. Finally, the effect of length and percentage of teff straw fiber on compaction characteristics and strength of black cotton soil, the effect of percentage of lime on MDD, OMC and strength of soil reinforced samples and the effect of lime and teff straw fiber on compressibility characteristics of soil were discussed.

3.3. Laboratory Test for Soil Classification

3.3.1. Natural Moisture Content

ASTM D2216: Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass was followed.

Undisturbed soil samples were collected from the study area and moisture content was determined in the laboratory. The moisture content of a soil mass is defined as the ratio of the weight of water to the weight of solids (dry weight) of the soil mass.

$$w = \frac{Mw}{Ms} * 100$$

3.3.2. Specific Gravity

The test was conducted as per ASTM D854: Standard Test Method for Specific Gravity of Soils. The specific gravity of soil solids, G_s, is the mass density of the mineral solids in soil normalized relative to the mass density of water. Additionally, a specific gravity of soil reinforced and lime stabilized samples was determined which used to calculate void ratio during consolidation test.

Figure 16: Specific gravity test for black cotton soil

3.3.3. Atterberg's Limit Test

The test was conducted as per ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

The liquid and plastic limit are water contents at which the mechanical properties of soil change. They apply to fine-grained soils and are performed on soil fractions that pass through 0.425μm sieve. The difference between liquid limit (LL) and plastic limit (PL) is defined as the plasticity index (PI).

$$PI = LL - PL$$

3.3.4. Free Swell Test

The procedure consists of pouring very slowly 10 cubic centimeters of that part of the dry soil passing No.40 sieve into a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately twenty-four hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted.

$$\text{Free swell} = \frac{\text{Final Volume} - \text{Initial Volume}}{\text{Initial Volume}} * 100\%$$

3.3.5. Wet Sieve Analysis

In wet sieve analysis, the oven-dried soil sample is soaked with water for 24hrs. Then, washed through 75μm sieve until the wash water becomes clear. The material retained on 75μm was dried in the oven and sieved through the set sieves. Then the material on each sieve was collected and weighed. The purpose of these tests is to determine the percentage of fines in soil. This information is used to classify the soil per the Unified Soil Classification System (USCS).

Figure 17: Soaked the soils in water for wet sieve analysis

3.3.5. Hydrometer Analysis

The test was conducted as per ASTM D422: To perform a hydrometer analysis, the soil is mixed with water and sodium hexametaphosphate (a dispersing agent) to create a slurry of dispersed soil particles. Information regarding how low the hydrometer floats in the slurry was recorded as a function of time, and this information is used to calculate points of grain size versus percent passing for the gradation curve.

Figure 18: Recorded hydrometer floats in a slurry with the function of time

As described in ASTM D422: Use Stoke's Law to calculate the particle diameter, D, in mm, corresponding to t in minutes:

$D = K * L/t$ Where, K = is a function of temperature and Gs (Appendix A, Table A-2)

L = Effective Depth Based on Hydrometer and Sedimentation of Cylinder of Specified Sizes (Appendix A, Table A-1)

Generally, from hydrometer analysis, the percentage of clay, silt, and sand found in the soil sample was identified.

3.3.6. Soil Classification System

ASTM D2487: Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) was followed.

USCS soil classification is a methodical procedure that follows these steps:

1) Decide if the soil is fine-grained or coarse-grained. If more than 50% of the soil passes the No. 200 sieves, it is fine-grained. Otherwise, it is coarse-grained.
2) For fine-grained soils, plot the LL and PI on the plasticity chart. The point will fall in the quadrant corresponding to the USCS group symbol, which will most likely be either silt (ML), lean clay (CL), or fat clay (CH).
3) For coarse-grained soils, determine if the soil is sand or gravel. The material retained by the No. 200 sieves is referred to as the coarse fraction. If more than 50% of the coarse fraction passes the No. 4 sieves, the soil is sand. Otherwise, it is gravel.

As described in ASTM D-3284 in the AASHTO soil classification system, soils are generally placed in seven major groups. Such as: A-1, A-2, A-3, A-4, A-5, A-6 and A-7. The required parameters for this classification system are grain size analysis, liquid limit, and plastic index.

3.4. Physical Characterization of Teff Straw Fiber

Water absorption of fiber was determined by soaking fiber in water for 24hrs and weighed dry fiber and saturated surface dry fiber. The length and diameter of fiber were measured by using the Varner caliper. The angle of bending before the break was measured by using a protractor.

3.5. Samples Preparation

The first sample of teff straw fiber was cut into different lengths (20mm, 40mm, and 60mm).

Figure 19: Prepared samples of teff straw fiber with different lengths

Then, a sample of soil was reinforced with different content (0.5%, 0.75%, and 1% by weight) and length of teff straw fiber and an unreinforced sample of soil compared to one another by conducting different laboratory tests. Additionally, the optimum length and percentage of teff straw fiber mentioned during the preparation of lime stabilized/fiber-reinforced samples were determined from the unconfined compressive strength test. Next, to determine the effect of teff straw fiber and lime on black cotton soil samples of lime stabilized/fiber-reinforced soil were prepared with different percentages of lime (2%, 4%, and 6% by weight) and optimum length and percentage of teff straw fiber. Then, a compaction test and UCS test were conducted on it and optimum lime content was identified. After that, this optimum lime was added to soil reinforced with various percentages of fiber for the consolidation test. Additionally, for consolidation test soil reinforced with different percentages of teff straw fiber and optimum length were prepared. In all these samples teff straw fiber and lime were distributed in the soil randomly.

Figure 20: Prepared soil reinforced and lime stabilized sample for a compaction test

Table 2: Mix proportions of black cotton soil, lime, and teff straw fiber

Material used and Mix Proportion (%)				Test Conducted		
Black Cotton Soil	Teff Straw Fiber Percentage	Teff Straw Fiber Length	Lime	Standard Compaction Test	UCS Test	One-Dimensional Consolidation Test
100	-	-	-	✓	✓	✓
99.5	0.5	20 and 40mm	-	✓	✓	-
99.25	0.75		-	✓	✓	-
99	1		-	✓	✓	-
99.5	0.5	60mm	-	✓	✓	✓
99.25	0.75		-	✓	✓	✓
99	1		-	✓	✓	✓
97.25	0.75	60mm	2	✓	✓	-
95.25	0.75		4	✓	✓	✓
93.25	0.75		6	✓	✓	-
96	0	60mm	4	✓	✓	✓
95.5	0.5		4	✓	-	✓
95	1		4	✓	-	✓

3.6. Laboratory Test for Samples

3.6.1. Compaction Test

The test was conducted as per ASTM D698, the standard proctor compaction test to determine the optimum moisture content and maximum dry density of each sample from the compaction curve. Every sample was compacted in 3 layers by a hammer that delivers 25 blows to each.

Figure 21: Standard compaction test

3.6.2. Unconfined Compressive Strength Test

ASTM D2166, the standard test method for unconfined compressive strength of cohesive soil was conducted on unreinforced, reinforced with fiber and fiber-reinforced/lime stabilized samples of soil. Unconfined compressive strength testing provides a quick and simple means to measure the unconfined compressive strength (qu) and undrained cohesion value (cu) of normally consolidated and lightly overconsolidated cylindrical specimens of cohesive soil. Unconfined compressive strength (UCS) samples were molded in a triaxial compressive strength mold of diameter 37mm and height of 76mm.

Samples were prepared in split mold at MDD and optimum moisture content (OMC). The prepared samples were tested without curing time and after cured for 7 days in a desiccator to maintain 100% relative humidity.

Mass of sample = Volume of mold*Maximum dry density of a sample

Water = Mass of sample*Optimum moisture content of the sample

Figure 22: Sample preparation for unconfined compressive strength test

As described in ASTM D2166: Axial deformation, ΔH is measured using a deformation indicator, and applied load, *P*, is measured using a load cell. Axial strain, ε, is expressed as:-

$\varepsilon = \Delta H / H_o$ Where, H_o is the initial height of the specimen

As the specimen deforms, its cross-sectional area increases, and the corrected area, A, is expressed as:

$A = A_o / (1 - \varepsilon)$ where, A_o is the initial cross-sectional area of the specimen

In this study, a proving ring was used instead of a load cell. Proving rings, *P* is calculated as:

$P = G_P * K_P$ where G_p is proving ring reading

K_p is proving ring factor

The major principal stress, σ_1, is expressed as- $\sigma_1 = P / A$

Unconfined compressive strength (qu) is equal to the maximum failure stress. Undrained cohesion (cu) is half of the unconfined compressive strength. Finally, from UCS test failure stresses, the stress-strain relationship and unconfined compressive strength of each sample were determined.

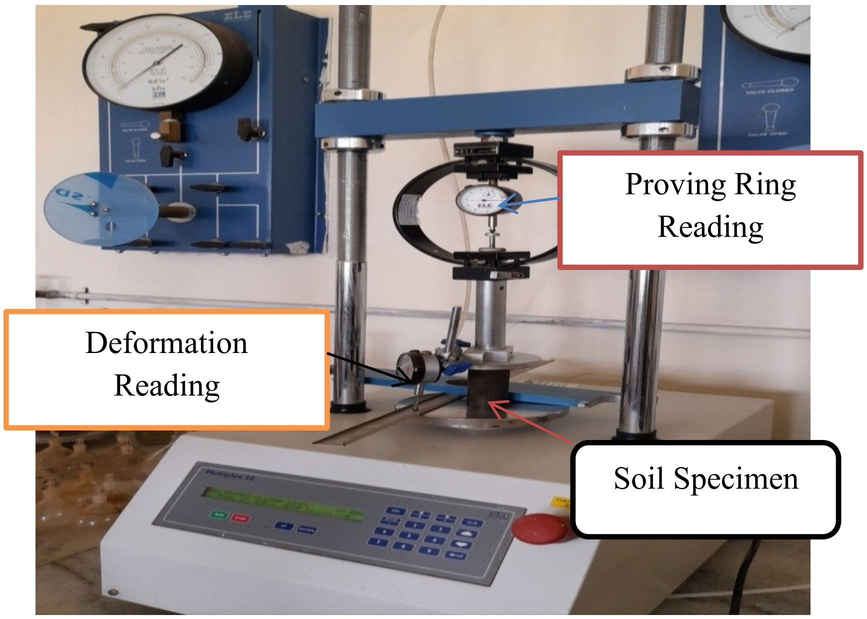


Figure 23: Unconfined compressive strength test machine

3.6.3. One-Dimensional Consolidation Test

The test was conducted as per ASTM D2435: The standard test method for one-dimensional consolidation properties of soil using incremental loading on the sample prepared for consolidation test. The load was maintained for 24 hours during which the soil consolidates with drainage from the porous stones. Afterward, the applied load was increased incrementally by doubling the applied stress at each stage. The numbers of the load stages were 8, 16, 32, 64, and 128 kg. During the loading process, water was provided into the cell so that the specimen remains fully saturated. At each loading stage, the deformation was measured at 6, 15, and 30 seconds, then at 1, 2, 4, 8, 16, 30 min, and 1, 2, 4, 8, and 24 hours, respectively. When the maximum load was reached, unloading stages were conducted by reducing the load. Typically, the load was reduced by a factor of 4 at each step.

After the test, the final height of the sample and its water content were measured. Finally, compressibility characteristics such as compression index, Cc, coefficient of volume compressibility, the coefficient of consolidation, Cv, and void ratio were determined from the test.

Figure 24: (a) Sample prepared for consolidation test (b) Consolidation test machine

As described in ASTM D2435: A curve of void ratio versus pressure was derived from the deformation at the end of primary consolidation for each load increment and coefficient of compressibility, a_v, and coefficient of volume compressibility, M_v, was determined. This curve is also plotted as a semi-log plot, which is used to calculate the compression index, C_c, and the recompression index, C_r. The compression and recompression indices are the slopes of the two portions of the curve. In this study coefficient of consolidation, Cv was calculated by using Taylor's square root of time fitting method. Height solid method used to calculate the void ratio.

$e_o = \frac{Gs}{\gamma d} - 1$ Where, e_o is initial void ratio

$H_s = \frac{Ho}{1+eo}$ H_s is height of solid sample

$\Delta e = {}^{Ho}/_{Hs}$ H_o is initial height of sample

$a_v = {}^{\Delta e}/_{\Delta p}$ ΔH is change in height of sample

$m_v = \frac{av}{1+eo}$ Δp is change in pressure

G_s is specific gravity

γ_d is dry density of sample

CHAPTER FOUR

RESULT AND DISCUSSION

4.1. Characterization of Soil

4.1.1. Natural Moisture Content

The laboratory results of NMC of test pit-one (TP-1) and test pit-two (TP-2) are 40.31% and 39.95% respectively. As described in ASTM D2216: In coarse-grained soils such as sands and gravels, the water content may range from a few percent in drier soils to over 20% in saturated soils. In fine-grained soils such as silts and clays, the possible range in moisture content is much higher due to the ability of clay minerals to absorb water molecules. Additionally, Zemedkun (2015) investigated that NMC of the Sendafa town falls in between 25.03% to 43.56 %. From the results, he concluded that the soils are highly plastic. The results of NMC of soil in this study also fall in similar ranges within the previous investigator. Therefore, the soils of the study area are characterized by high water holding capacity and plasticity.

4.1.2. Specific Gravity of Soil

The specific gravity of soil from TP-1 and TP-2 is 2.78 and 2.7 respectively. As described in ASTM D854, the specific gravity of coarse-grained soils such as sands and gravels, where the mineralogy is dominated by quartz and feldspar, is typically around 2.65. In fine-grained soils, specific gravity is more variable due to the presence of clay minerals and may range from 2.70-2.85. Therefore, both soils are fine-grained. But soil from test pit one has more clay mineralogy than that from test pit two.

4.1.3. The Free Swell Test

The free swell test result of soils from TP-1 and TP-2 are 70% and 40% respectively. Soils with free swell less than 50% are not likely to show expansive property, while soils with free swells above 50% could present swelling problems. The values indicate that soil from TP-1 has high swell potential than TP-2 and is expected to have a higher impact on the construction of structures.

4.1.4. The Atterberg Limit

The liquid limit and plasticity index of soils from TP-1 and TP-2 are 81.8% and 49.62%, and 78.7% and 38.22% respectively. According to ASTM D4318 liquid limit typically ranges anywhere from 20% for silts to over 100% for high-plasticity clays.

The plasticity index typically ranges anywhere from near 0% (a non-plastic soil) for silts to over 50% for high-plasticity clays. As a result soil from TP-1 is highly plastic than soil from TP-2.

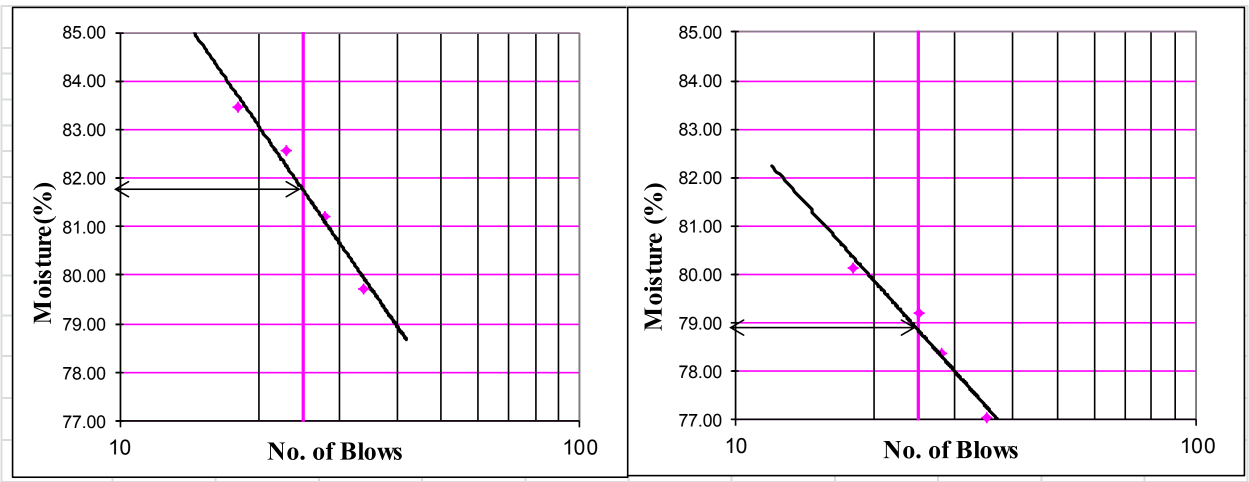


Figure 25: Liquid limit value of soil from TP-1 and TP-2

4.1.5. Grain Size Analysis

From the gradation curve (Figure 26), materials finer than 0.075 mm and finer than 0.002 mm are respectively found to be 92.03% and 45% for TP-1 and 90.55% and 33.6% for TP-2, confirming the fine-grained nature of the soils. However, soil from TP-1 has a high percentage of clay fraction than the soil of TP-2 which is in agreement with the relatively high plasticity and swelling potential of TP-1.

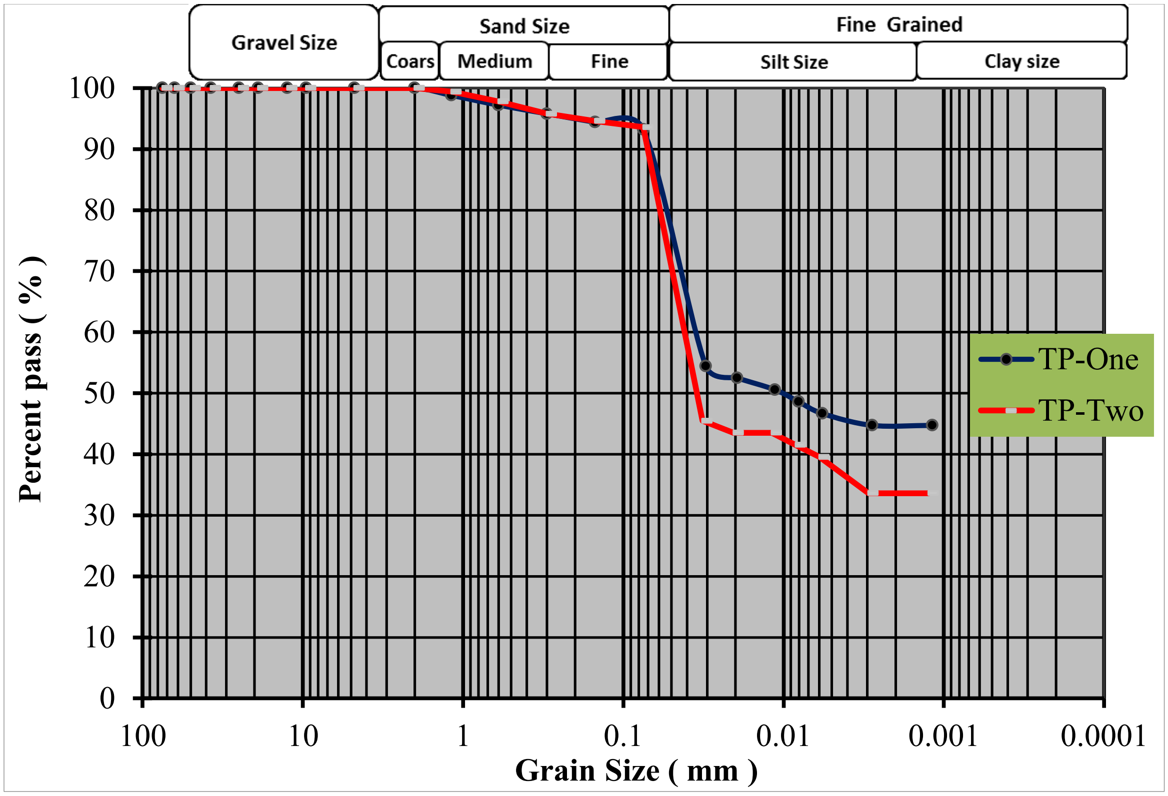


Figure 26: Gradation curve of soil from TP-1 and TP-2

4.1.6. Soil Classification

Unified soil classification plasticity chart, implies that the A-line separates clays from silts and the U-line indicates the upper limit of the relationship between PI and LL. Accordingly, the soil under study is plotted on the plasticity chart. The soil from TP-1 lies above the A-line which means the soil is clay with high plasticity (CH) and that from TP-2 below the A-line which means the soil is silty with high plasticity (MH) and both soils are conformed to their gradation test results. Classification of soils of study area according to USCS is shown in Figure 27. According to the AASTHO classification system, the TP-1 soil sample lies above A-line and TP-two soil sample falls in region A-7-5 (Figure 28). Such soils are generally considered the least suitable as a highway material or subgrade. As general from index properties of black cotton soils, soil from TP-1 is expected to have a higher impact on the construction of structures. As a result, soil from TP-1 is mentioned during sample preparation.

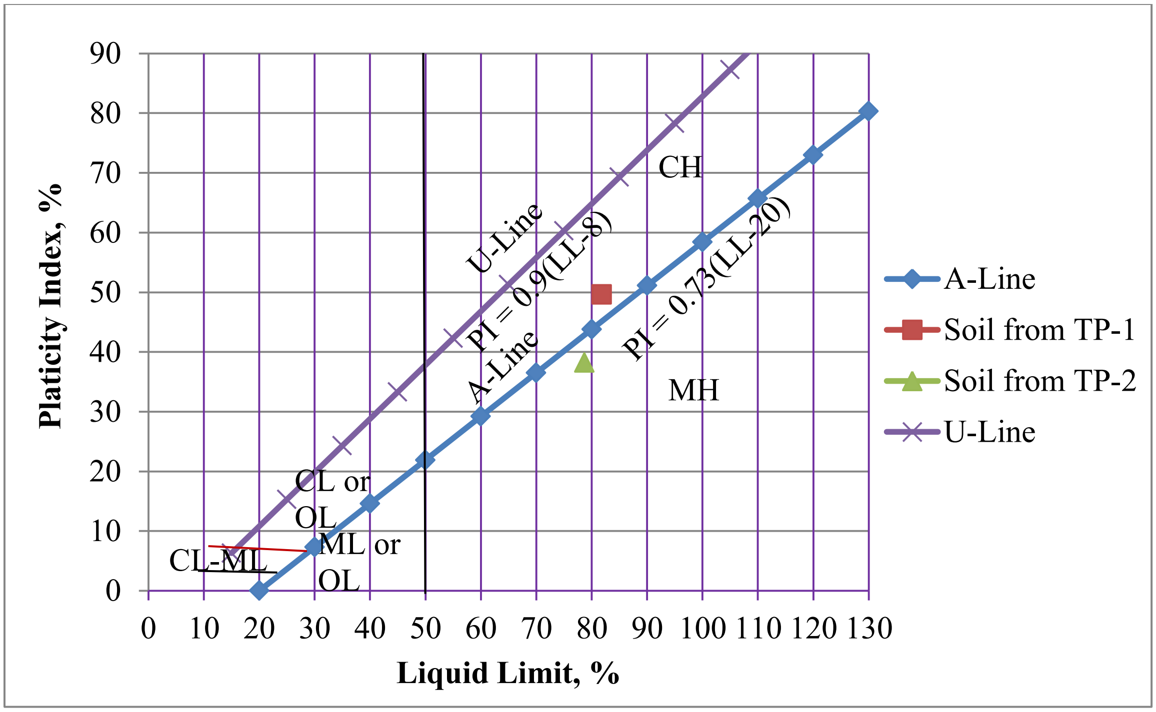


Figure 27: Plasticity chart of the study area according to USCS

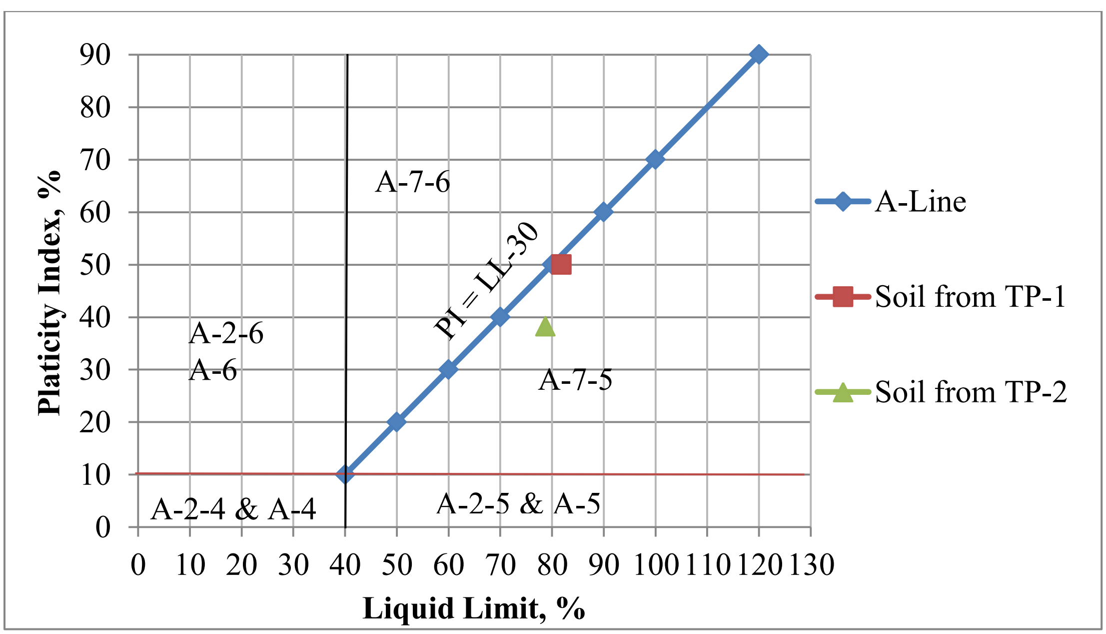


Figure 28: Plasticity chart of study area according to AASHTO

4.1.7. Compaction Characteristics

The maximum dry density (MDD) determined from the compaction curve (Figure 29) is for TP-1 and TP-2 are 1452 and 1484 Kg/m^3 respectively. While optimum moisture content (OMC) is 24% and 22.8% for TP-1 and TP-2 are respectively.

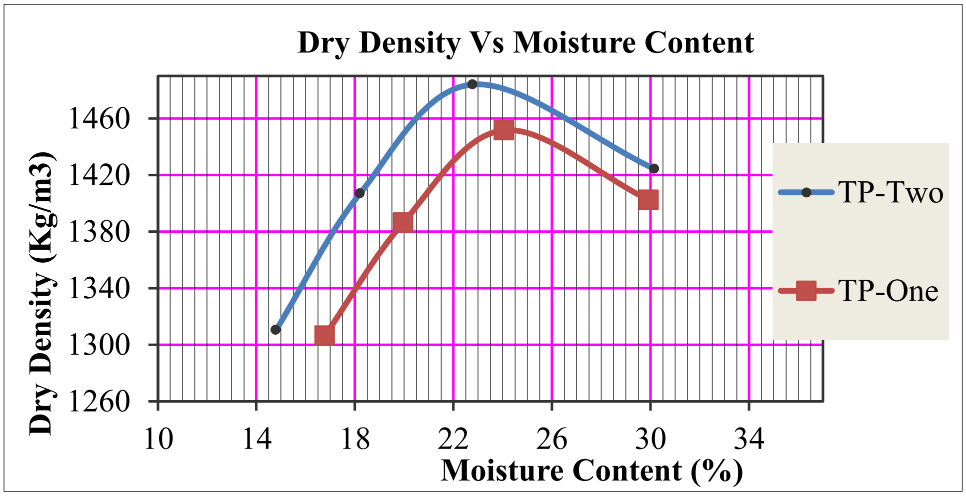


Figure 29: Compaction characteristics of soil from TP-1 and TP-2

Table 3: Summary of test result for soil properties

Sample ID	Soil from TP-1	Soil from TP-2
Specific gravity	2.78	2.7
Liquid Limit (%)	81.8	78.7
Plastic Limit (%)	32.18	40.48
Plasticity Index (%)	49.62	38.22
Free swell (%)	70	40
MDD(Kg/m^3)	1452	1484
OMC (%)	24.06	22.78
Natural Moisture Content (%)	40.31	39.95
AASHTO system	A-7-6	A-7-5
USCS	CH	MH

4.2. Physical Characteristics of Teff Straw Fiber

Water absorption of teff straw fiber is 60.7%, its' average length and diameter are 5cm and 0.4mm respectively. The angle of bending before the break was 260^o-280^o during dry conditions. Its' Color is also brown-reddish and white.

4.3. Effect of Teff Straw Fiber and Lime on Engineering Property

4.3.1. The Effect of Teff Straw Fiber on Compaction characteristics

A standard compaction test was done for soil reinforced with different percentages and different lengths of teff straw fiber. The compaction curves for different percentages of teff straw fiber of 40mm length are shown in the figure below and the remaining compaction curves are included in Appendix C (Figure C-1 and C-2).

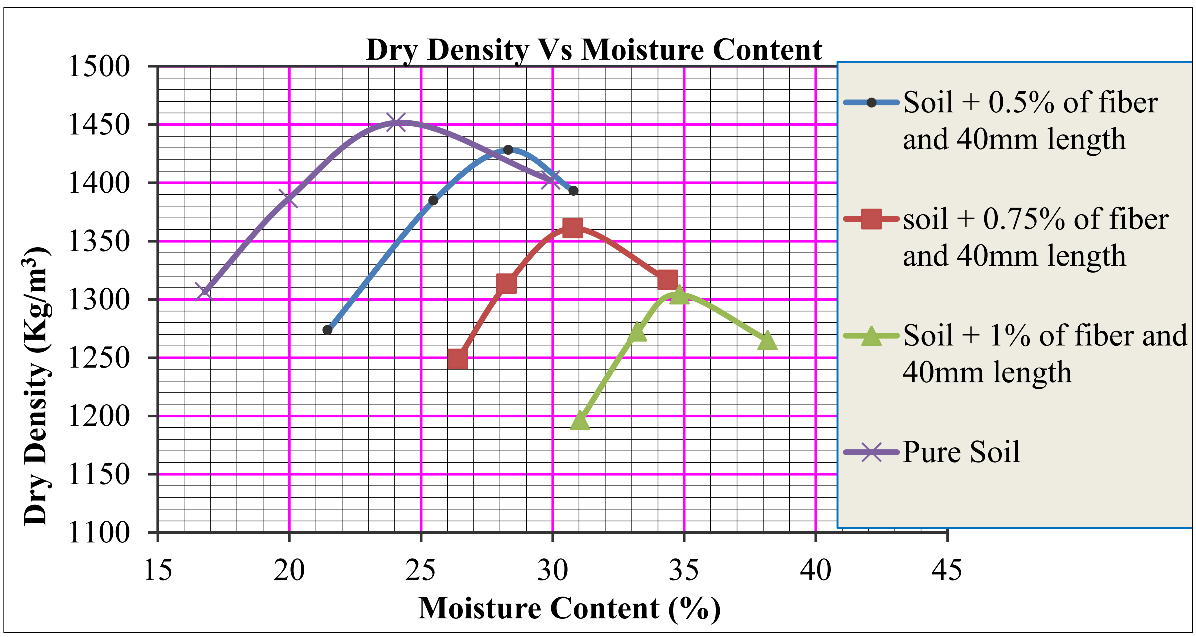


Figure 30: Compaction curve for soil reinforced with 40mm length of fiber

From the compaction curve, MDD of black cotton soil decreased and OMC increased as the percentage of fiber increased (Figure 31). Similar with (Maja, 2018) when lateritic soil reinforced with teff straw fiber, MDD decreased from 1478 to 1435 kg/m^2 and OMC increased from 23.8 to 26%.

In this study, black cotton soil reinforced with 60mm length of teff straw fiber with increased percentages of fiber (0%, 0.5%, 0.75% and 1%), MDD decreased from 1452Kg/m^3 to 1290Kg/m^3 and OMC increased from 24.06% to 33.37% (Table 4). This implies that the density of fiber was less than that of the density of soil means that specific gravity decreased as percentage of fiber increased (Appendix B, Table B-7) and its' water absorption was greater.

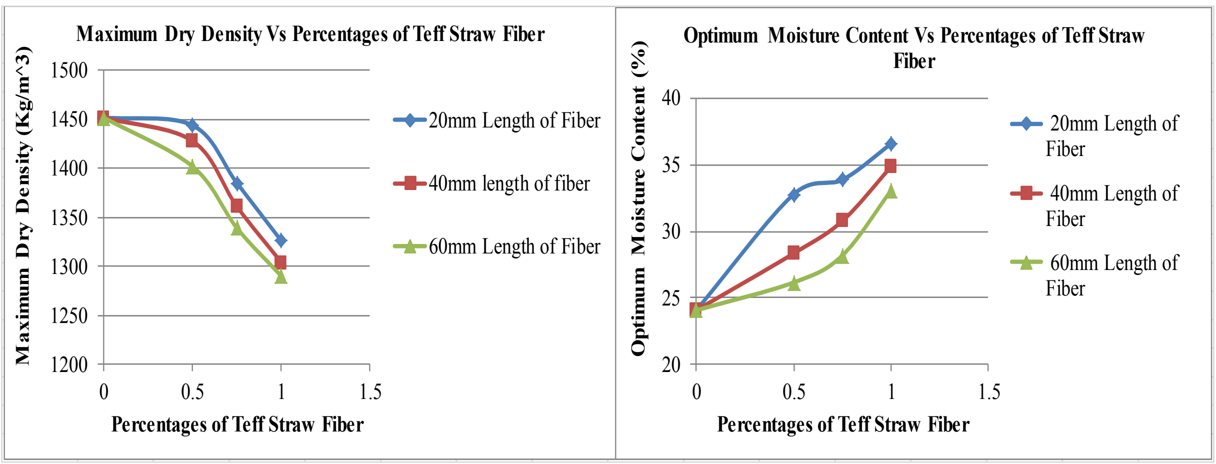


Figure 31: Variation of MDD and OMC of soil reinforced with different percentages of fiber

As the length of fiber increased OMC decreased and MDD decreased by little amount almost no change. For instance, soil reinforced with 0.5% of teff straw fiber content and increased length (20mm, 40mm, and 60mm), MDD decreased from 1444Kg/m^3 to 1402Kg/m^3 and OMC also decreased from 32.77% to 26.15% as the length of fiber increased (Table 4).

Table 4: Summary of MDD and OMC of soil reinforced with teff straw fiber

Percentage of Fiber (%)	Fiber Lengths					
	20mm Length		40mm Length		60mm Length	
	MDD(Kg/m^3)	OMC (%)	MDD(Kg/m^3)	OMC (%)	MDD(Kg/m^3)	OMC(%)
0	1452	24.06	1452	24.06	1452	24.06
0.5	1444	32.77	1428	28.33	1402	26.15
0.75	1385	33.86	1361	30.77	1339	28.18
1	1326	36.56	1304	34.84	1290	33.05

4.3.2. The Effect of Teff Straw Fiber on the Strength of the Soil

An unconfined compressive strength test was done on soil, soil reinforced with teff straw fiber by two conditions. Such as sample of soil reinforced as soon as it prepared (uncured) and after seven days cured. The stress-strain relationship, unconfined compressive strength, and undrained cohesion of each sample were determined from the test result. UCS test result for soil reinforced with 20mm length of teff straw fiber after 7days cured shown in (figure 32). From the below figure, the stiffness and peak stress of soil increased when the soil was reinforced with teff straw fiber up to 0.75% then decreased for further increase of teff straw fiber. The other test result is found in (Appendix D).

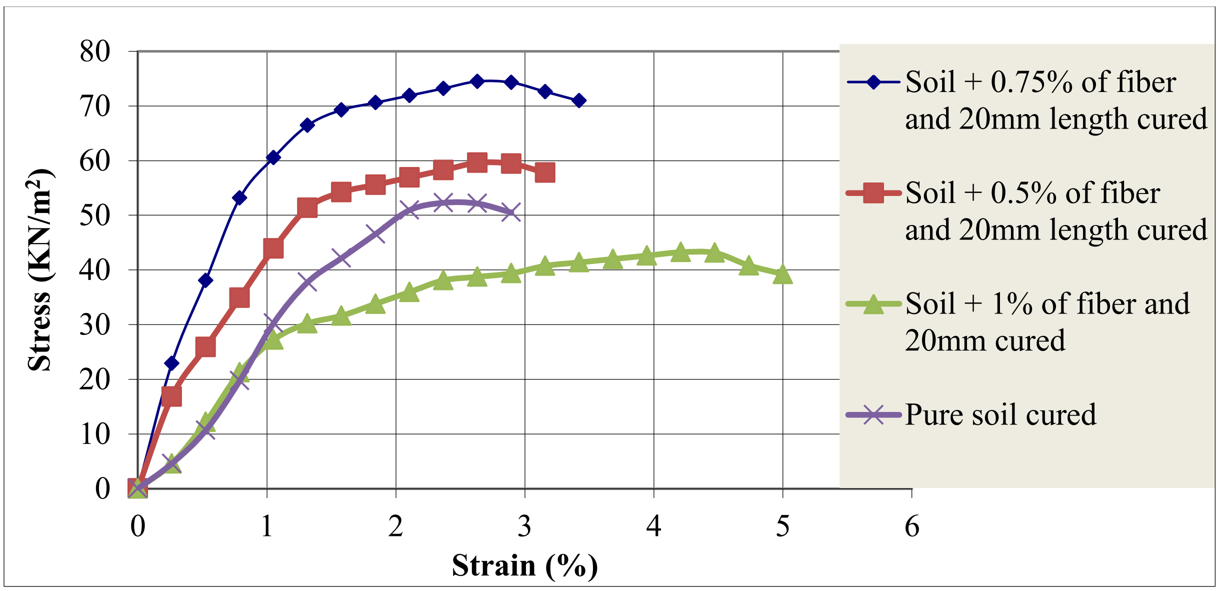


Figure 32: Stress-Strain relationship for soil reinforced with 20mm length of teff straw fiber

From the test result, as the percentage and length of teff straw fiber increased the unconfined compressive strength and cohesion of the soil increased for both uncured and cured conditions. Additionally, curing time also increased the strength of the soil and soil reinforced with teff straw fiber (Figure 33). This implies that as the soil is reinforced with teff straw fiber, the strength of the soil increased due to the fiber binding the soil particles together and the binding ability increased within a length of the fiber and curing time.

As the percentage of the fiber increase (0%, 0.5%, 0.75%, and 1% by weight), the unconfined compressive strength of the soil increased up to 0.75%, and then decreased. For instance, the unconfined compressive strength of soil reinforced with 60mm length of fiber is (40.8, 47.3, 64.5, and 42.1kpa) and (52.4, 65.2, 91.8, and 49.3kpa) uncured condition and seven days cured respectively. As a result, 0.75% of fiber was an optimum percentage of fiber for all length, and 60mm length of fiber was the optimum one which mentioned during samples preparation of soil reinforced/lime stabilized in next section and soil reinforced for consolidation test.

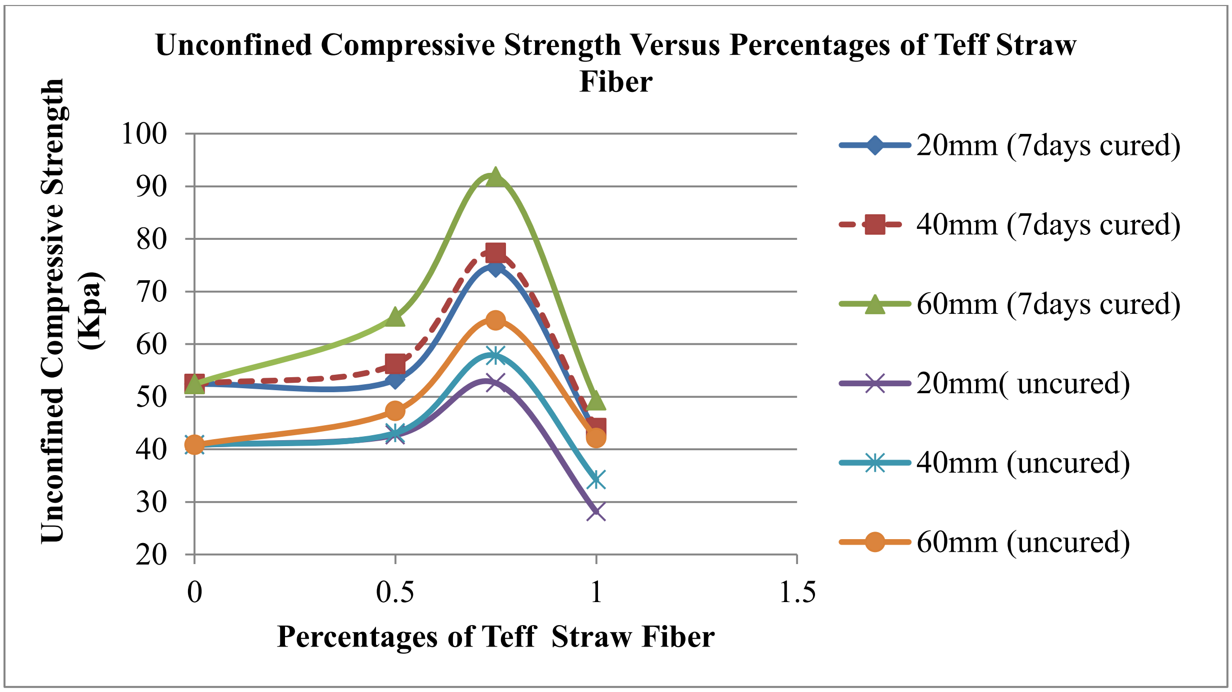


Figure 33: Variation of unconfined compressive strength with different percentages of teff straw fiber and curing time

4.3.3. The Effect of Lime on Compaction Characteristics of Reinforced Soil

A standard compaction test was done for soil reinforced with 0.75% teff straw fiber and optimum length (60mm) and stabilized with different percentages of lime (0%, 2%, 4%, and 6%).

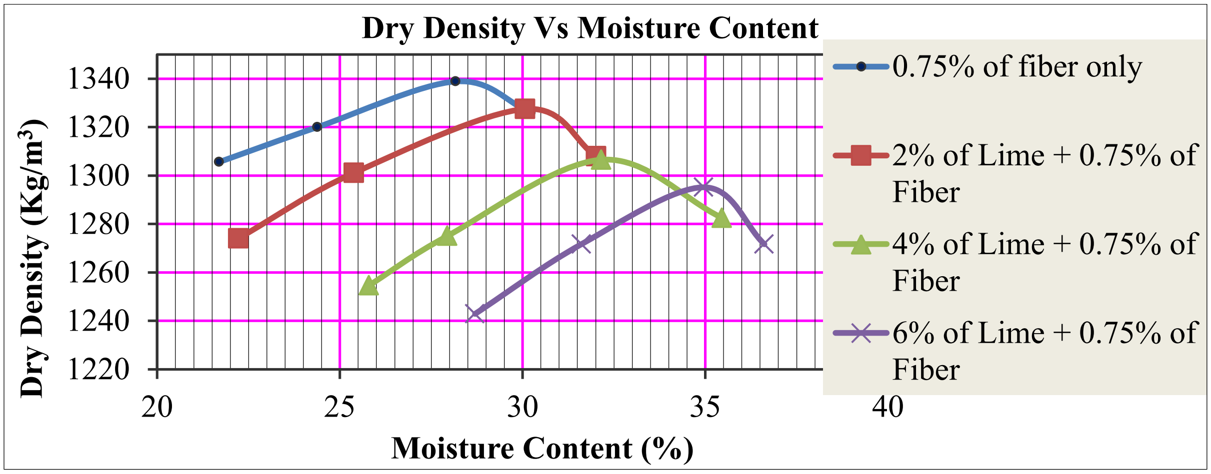


Figure 34: Compaction test for soil reinforced/lime stabilized with different percentages of lime

The result indicates, at fixed percentages of fiber as percentages of lime increase, MDD decreased and OMC increased. Similar with (Narasihma Rao, 2014) when black cotton soil stabilized with lime. As percentages of lime increase (0%, 2%, 4% and 6%) MDD of soil reinforced with fiber decreased from 1339kg/m^3 to1295kg/m^3 and OMC increased from 28.18% to 34.96% (Table 5). For the reason that the density of lime is less than the density of soil, and as soil replaces by lime MDD decreased and OMC increased due to requirement of additional water for pozzolanic reaction.

Table 5: Summary of MDD and OMC of soil reinforced and lime stabilized samples

Name of Samples	Percentages of Lime	Percentages of Fiber	MDD (Kg/m^3)	OMC (%)
Soil Reinforced with 0.75% teff straw Fiber and Lime stabilized with different percent	0	0.75	1339	28.18
	2	0.75	1327	30.08
	4	0.75	1307	32.16
	6	0.75	1295	34.96

4.3.4. The Effect of Lime on the Strength of Reinforced Soil

UCS test was conducted for soil samples reinforced with 0.75% of fiber and 60mm length (optimum percentage and length) and stabilized with different percentages of lime (0%, 2%, 4%, and 6%) uncured and after 7days cured. The test results are shown in Figure 35 and Table 6. The result demonstrates that the stiffness and strength of soil increased with increasing lime from 0% to 4% and decreased for a further increase to 6% both in the case of uncured and 7 days cured samples (Figure 35 and 36). For the reason that the cation exchange and attraction between the cations associated with the surfaces of soil particles and calcium cations of the lime causes soil particles to become close to each other and increase cohesion of the soil.

The effect is more pronounced for cured condition due to pozzolanic reaction consume part of the soil's water content and soil becomes stiffer and make cementations compounds. Therefore, lime content up to 4% enhanced the strength and is considered as the optimum lime percentage for further study (next section).

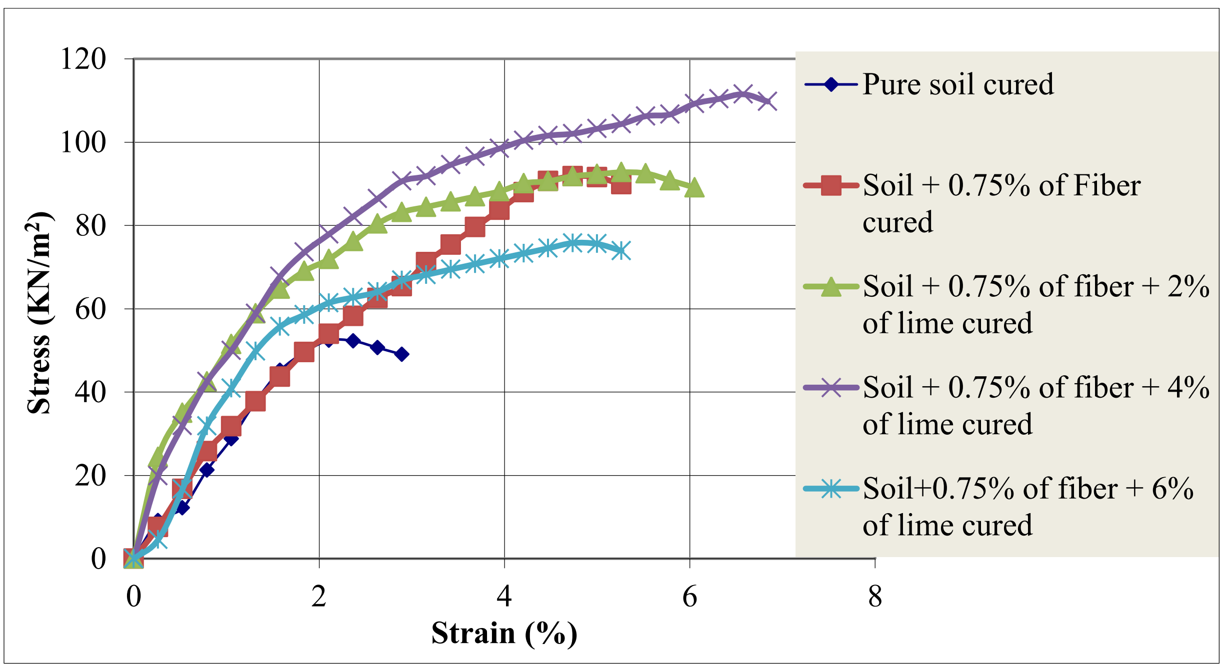


Figure 35: Stress-Strain relationship of soil reinforced and lime stabilized with different percentages of lime

It is also worth noticing that the strength increased by more than double when the soil is both reinforced and treated with the optimum percentage of fiber (0.75%) and lime (4%) compared to lime stabilized soil both for cured and uncured samples (Table 6). It indicates that a combination of teff straw fiber reinforcement and lime stabilization is more effective for ground improvement than lime stabilization alone.

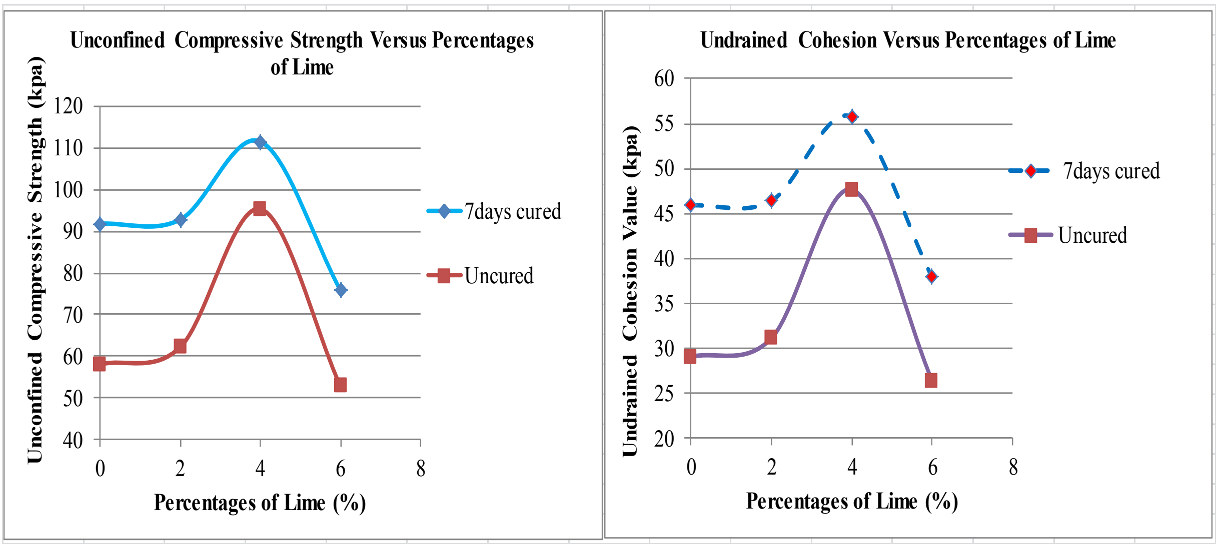


Figure 36: The effect of lime on UCS and cohesion of reinforced soil uncured and 7days cured

Table 6: Summary for UCS and cohesion of soil reinforced and lime stabilized samples

Percentages of Lime	Percentages of Fiber	Uncured UCS Test		7days cured UCS Test	
		Unconfined compressive strength(kpa)	Undrained cohesion(kpa)	Unconfined compressive strength(kpa)	Undrained cohesion(kpa
0	0.75	58.1	29.05	91.8	45.9
2	0.75	62.2	31.1	92.8	46.4
4	0.75	95.3	47.65	111.5	55.75
6	0.75	52.8	26.4	75.8	37.9
4	0	45.4	22.7	59.1	29.55

4.4. The Effect of Teff Straw and Lime on Consolidation Characteristics

The one-dimensional consolidation test was conducted for unreinforced/unstabilized soil, soil reinforced with different percentages of fiber (0.5%, 0.75%, and 1%) and optimum length (60mm), soil stabilized with optimum lime content (4%), and soil reinforced/stabilized by optimum lime content and different percentages of fiber. The coefficient of consolidation, compression index, coefficient of the volume of compressibility, and void ratio versus pressure was determined for each sample and discussed in this section.

4.4.1. The Effect of Teff Straw Fiber and Lime on Coefficient of Consolidation of Soils

The coefficient of consolidation of each sample was determined by using Taylor's square root of time fitting method. The coefficient of consolidation-pressure relationship for different conditions (pure soil, fiber-reinforced soil, lime stabilized, and fiber-reinforced/lime stabilized soil) is shown in the figure below.

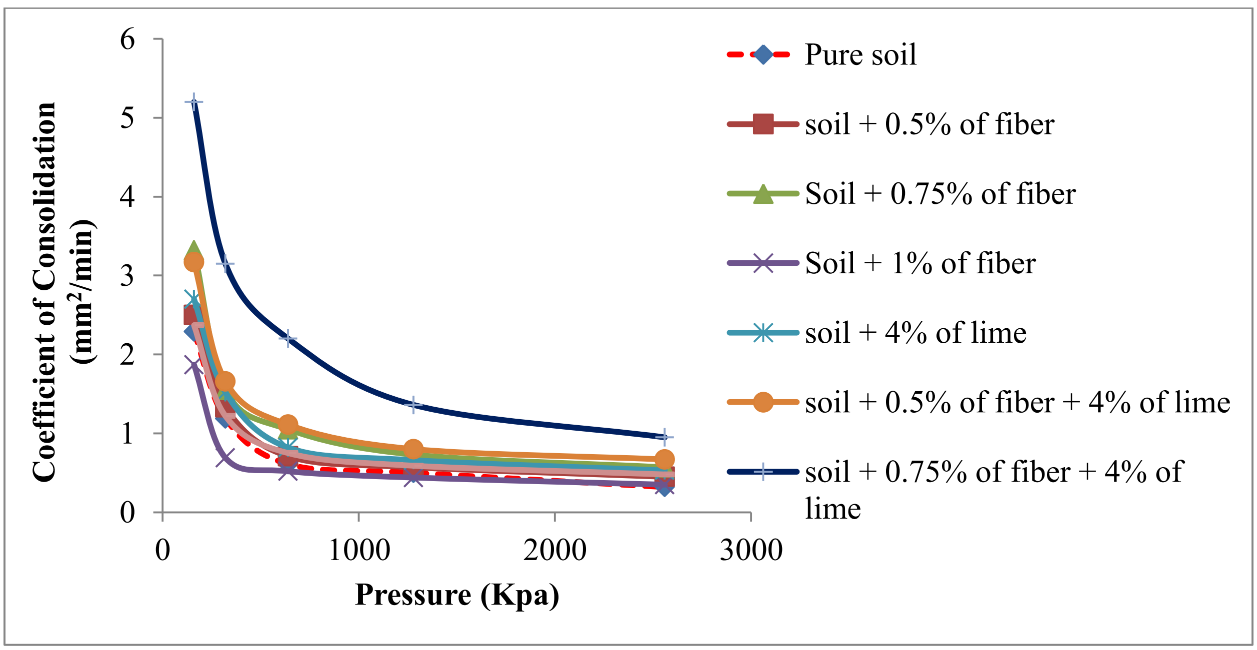


Figure 37: Coefficient of consolidation versus pressure for different conditions

The result indicates that the coefficient of consolidation of the black cotton soils increased when the soil is reinforced with different percentages of teff straw fiber. As percentages of teff straw fiber increased coefficient of consolidation increased up to 0.75%, then decreased (Figure 38).

The coefficient of consolidation increased more when the soil is reinforced and stabilized by teff straw fiber and lime, respectively due to aggregation particles of soil by cation exchange and flocculation process (Figure 39). Additionally, the increasing coefficient of consolidation is due to the soil have less affinity with water when reinforced with teff straw fiber and stabilized by lime. A higher coefficient of consolidation can reduce the duration of consolidation and leads to an improvement in the stability and bearing capacity of the soil. As a result, the combination of soil reinforced with 0.75% of teff straw fiber and stabilized by 4% lime is more significant to improve the stability and bearing capacity of the black cotton soil.

Figure 40 shows the effect of fiber reinforcement and lime stabilization-fiber reinforcement on time for 90% primary consolidation ($t_{90\%}$). The result shows that $t_{90\%}$ decreased when the soil is reinforced by teff straw fiber and stabilized by lime alone and combined. However, the effect is more pronounced for the reinforced and stabilized soil, which indicates the significant improvement of the rate of consolidation.

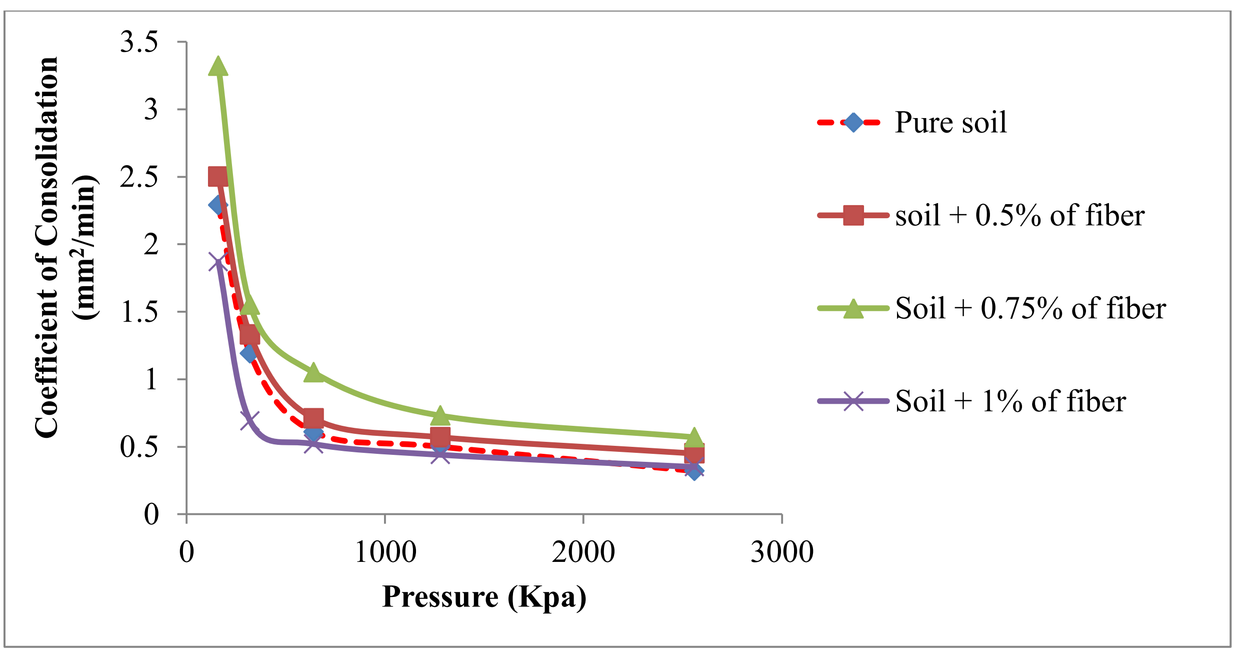


Figure 38: Coefficient of consolidation of soil reinforcd with teff straw fiber

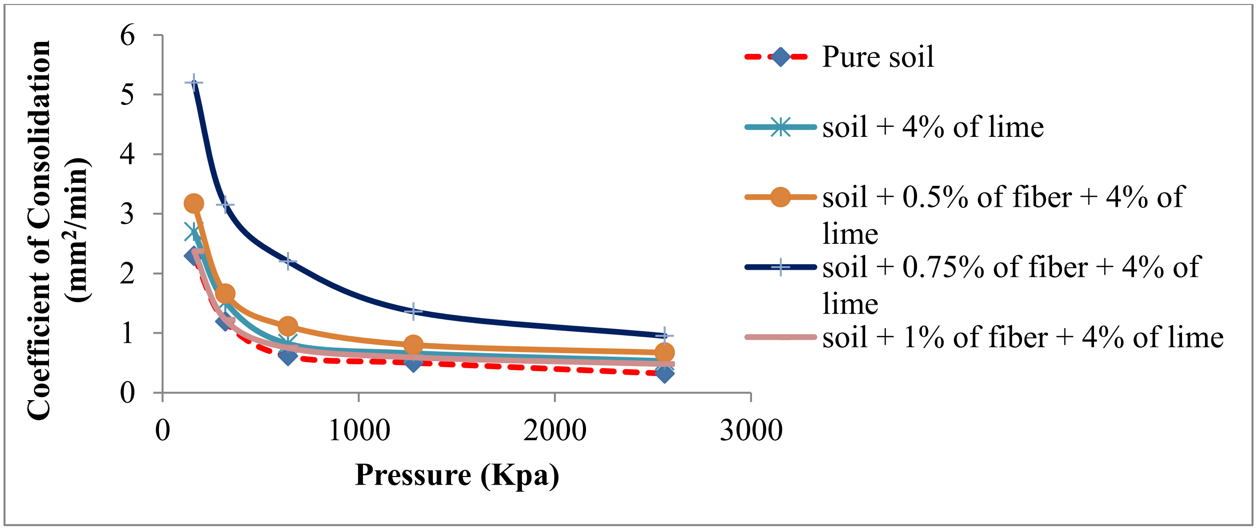


Figure 39: Coefficient of consolidation of soil reinforced and lime stabilized samples

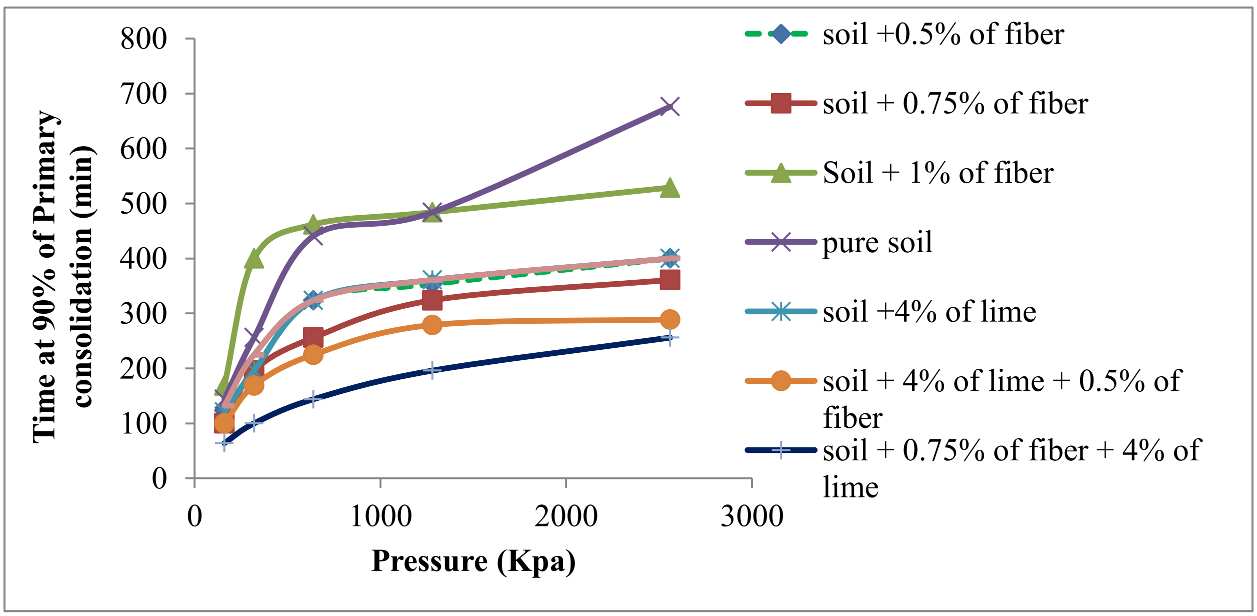


Figure 40: The effect of teff straw fiber and lime on t_{90} of soil

Table 7: Summary of the coefficient of consolidation of samples

Pressure (Kpa)	Coefficient of consolidation (mm^2/min)							
	Pure soil	Soil reinforced samples			Soil stabilized with 4% of lime	Soil reinforced/lime stabilized		
		0.5% of fiber	0.75% of fiber	1% of fiber		0.5% of fiber + 4% of lime	0.75% of fiber + 4% of lime	1% of fiber + 4% of lime
160	2.29	2.5	3.32	1.87	2.7	3.17	5.2	2.37
320	1.19	1.33	1.55	0.69	1.54	1.65	3.21	1.21
640	0.61	0.71	1.05	0.52	0.84	1.09	2.202	0.74
1280	0.5	0.57	0.74	0.44	0.67	0.78	1.37	0.59
2560	0.32	0.45	0.57	0.35	0.54	0.67	0.95	0.48

Table 8: Summary for time at 90% of primary consolidation of samples

Pressure (Kpa)	Time at 90% of primary consolidation (minute)							
	Pure soil	Soil reinforced samples			Soil stabilized with 4% of lime	Soil reinforced/lime stabilized		
		0.5% of fiber	0.75% of fiber	1% of fiber		0.5% of fiber + 4% of lime	0.75% of fiber + 4% of lime	1% of fiber + 4% of lime
160	144	123.21	100	169	121	100	64	132.25
320	256	196	196	400	196	169	100	225
640	441	324	256	462	324	225	144	324
1280	484	353.44	324	484	361	278.89	196	361
2560	676	400	361	529	400	289	256	400

4.5.2. The Effect of Teff Straw Fiber and Lime on Coefficient of Volume Compressibility

The coefficient of volume compressibility of each sample was determined from the plot of void ratio versus pressure (figure 41).

From the test result, the coefficient of volume compressibility of black cotton soil was not improved by soil reinforced with teff straw fiber (Figure 42). However, the combination of fiber reinforcement and lime stabilization decreased the compressibility of black cotton soil. For the reason that in a combination of reinforcement and lime stabilization pozzolanic action quickly undergoes with the soil, thus effectively decreasing the void ratio in soil reinforced and lime stabilized samples. As a result, decreasing coefficient of the volume of compressibility took place. The combination of 0.75% of teff straw fiber reinforced and 4% of lime stabilized soil more reduced the coefficient of volume compressibility of soil (Figure 43).

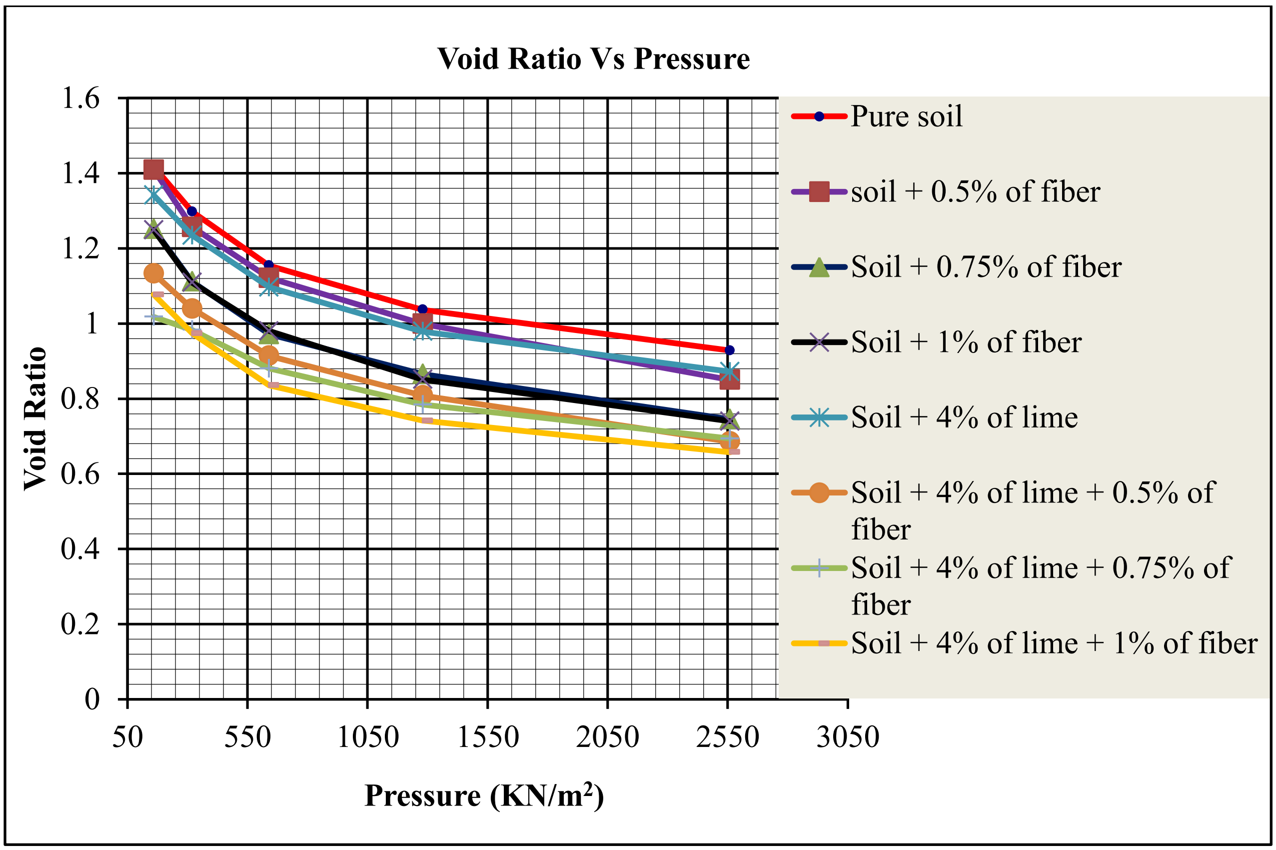


Figure 41: Void ratio versus pressure for different conditions

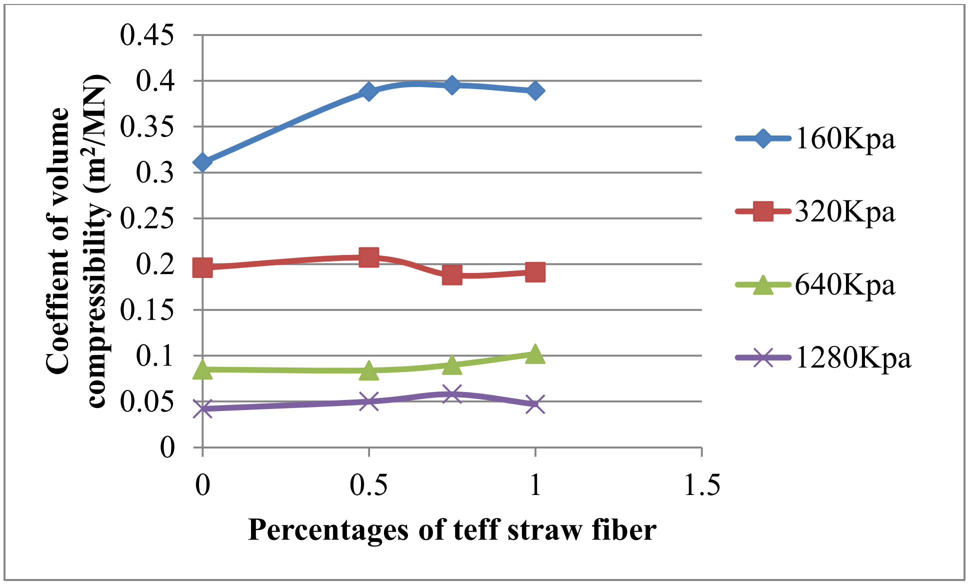


Figure 42: Coefficient of volume compressibility of soil reinforced under different pressure

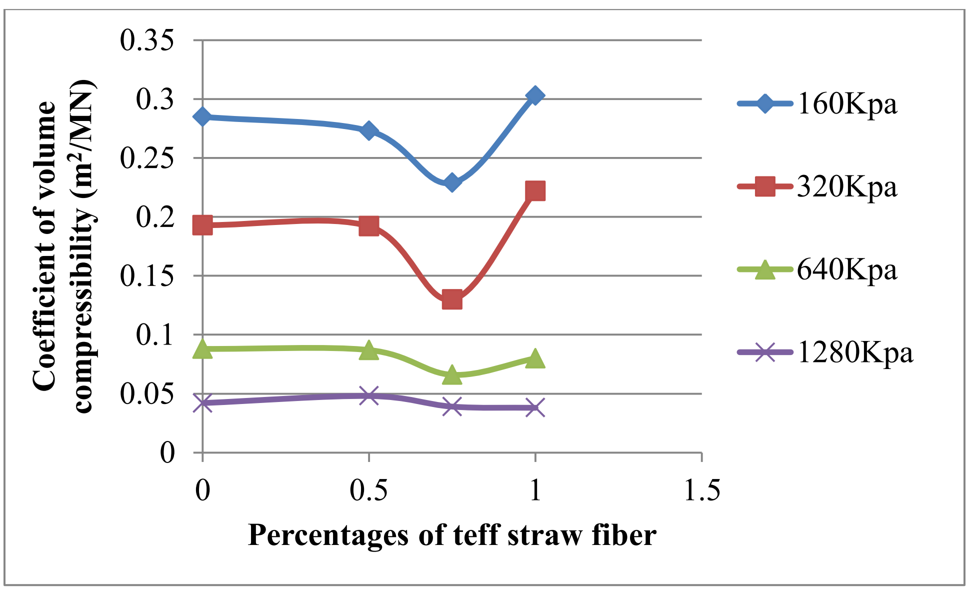


Figure 43: Coefficient of volume compressibility of soil reinforced and lime stabilized under different pressure

Table 9: Summary of the coefficient of volume compressibility

Pressure (Kpa)	Coefficient of volume compressibility (m^2/MN)							
	Pure soil	Soil reinforced samples			Soil stabilized with 4% of lime	Soil reinforced/lime stabilized		
		0.5% of fiber	0.75% of fiber	1% of fiber		0.5% of fiber + 4% of lime	0.75% of fiber + 4% of lime	1% of fiber + 4% of lime
160	0.311	0.388	0.395	0.389	0.285	0.273	0.229	0.303
320	0.196	0.207	0.188	0.191	0.193	0.192	0.130	0.222
640	0.085	0.084	0.090	0.102	0.088	0.087	0.066	0.080
1280	0.042	0.050	0.058	0.047	0.042	0.048	0.039	0.038

4.5.3. The Effect of Teff Straw Fiber and Lime on Compression and Re-compression Index

The compression index and recompression index of each sample were determined from void ratio-pressure curves (logarithm). At a given pressure level, the void ratio of samples decreases after reinforcement and lime stabilization (Figure 44). In other words, the compression index and re-compression index decreased when soil is reinforced with teff straw fiber and stabilized by lime. The effect compression is not significant for fiber reinforcement only. However, the recompression index appears to be decreased similar to fiber-reinforced and lime treatment (Figure 45). The reduction of recompression index is due to fiber binding soil particles together and prevent swelling potential.

The result suggests that fiber reinforcement and lime stabilization can reduce both the compressibility and swelling potential of soil (Figure 46). The reduction of compression index is due to bond developed between soil particles during cation exchange, flocculation reaction and pozzolanic action. The Decrease in swelling can be attributed to the fibers locking the soil particles together. Additionally, lime reduced the plasticity of soil which is preventing potential swelling.

The combination of 0.75% of teff straw fiber-reinforced and 4% of lime stabilized soil has significant approvements in compression index and re-compression index rather than lime stabilized alone (Figure 46 and Table 10). i.e The compression index of black cotton soil reduced from 0.407 to 0.39 and re-compression index reduced from 0.117 to 0.112 when soil stabilized with lime alone and when soil reinforced with 0.75% of teff straw fiber and stabilized with 4% of lime compression index, and re-compression index of soil reduced to 0.296 and 0.081 respectively.

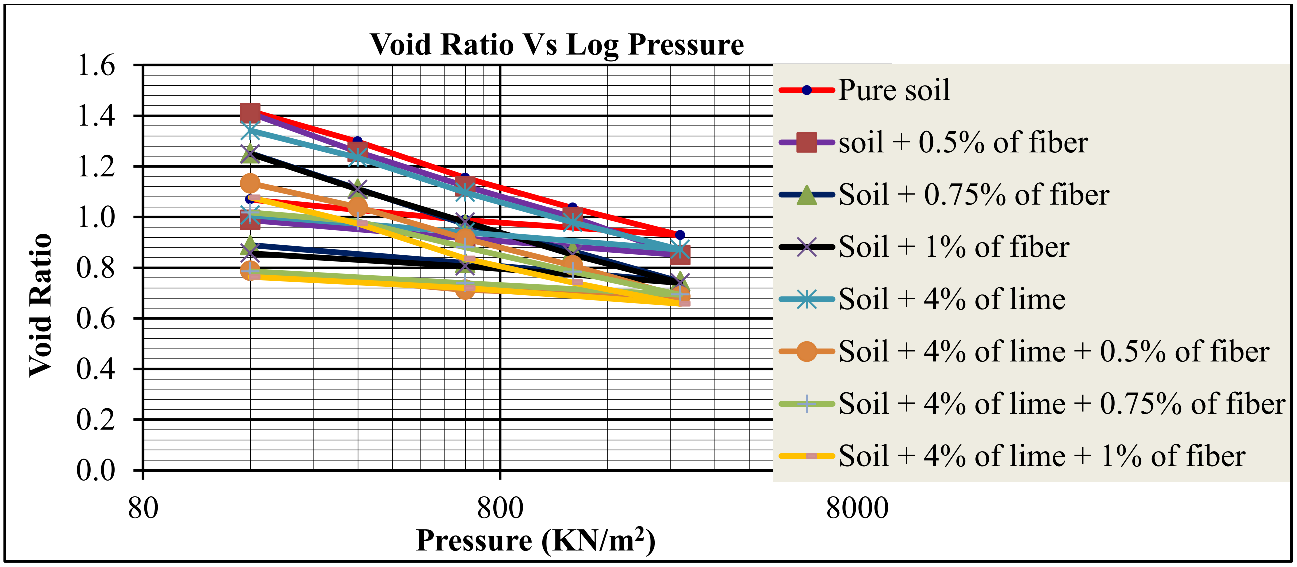


Figure 44: Graph of void ratio versus log pressure of samples

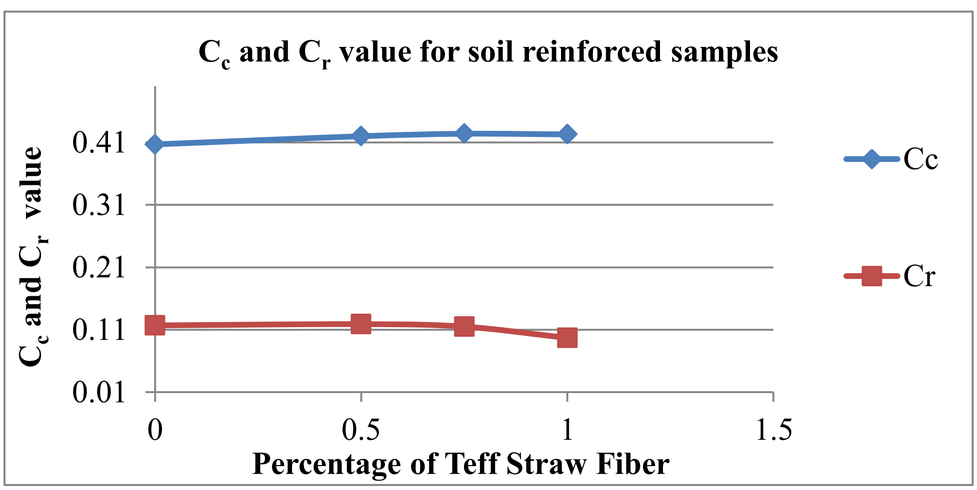


Figure 45: Compression index and re-compression index of soil reinforced samples

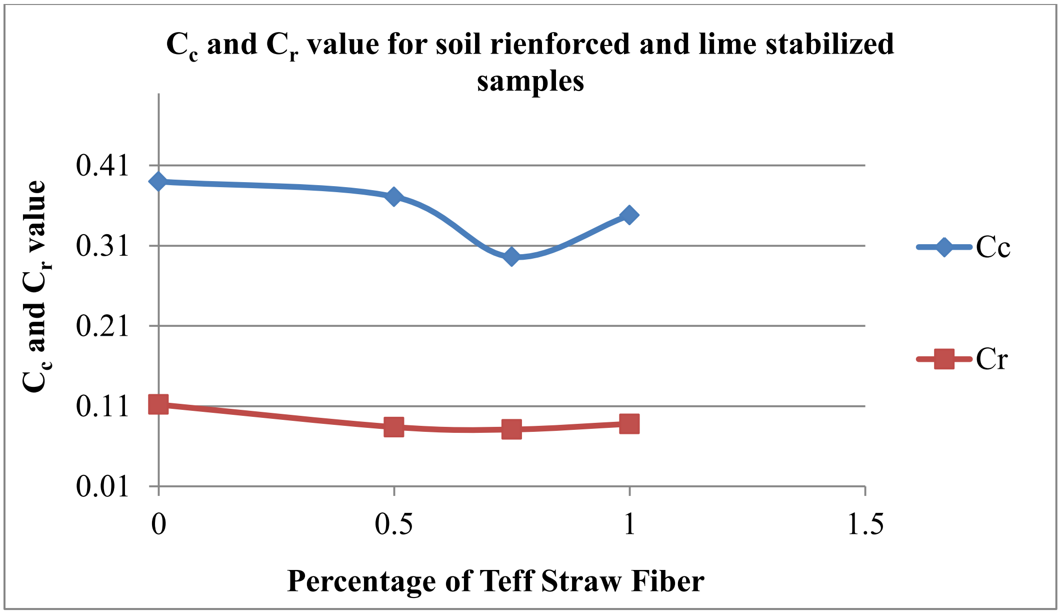


Figure 46: Compression index and re-compression index of soil reinforced and lime stabilized

Table 10: Summary for compression index and re-compression index of samples

Sample Name	Compression Index, C_c	Re-compression Index, C_r
Black cotton soil	0.407	0.117
Soil reinforced with 0.5% fiber	0.420	0.119
Soil reinforced with 0.75% fiber	0.424	0.115
Soil reinforced with 1% fiber	0.423	0.097
Lime stabilized only	0.390	0.112
Soil reinforced/lime stabilized with 0.5% fiber	0.371	0.084
Soil reinforced/lime stabilized with 0.75% fiber	0.296	0.081
Soil reinforced/lime stabilized with 1% fiber	0.348	0.088

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study, the effect of teff straw fiber and lime on the strength and compressibility behavior of black cotton soil was investigated. To determine their effects on strength and compressibility characteristics of black cotton soil different percentages and length of teff straw fiber and lime mixed with soil and conducted laboratory tests such as compaction test, unconfined compressive test, and consolidation test. From laboratory results, the following conclusions are drawn.

- ❖ MDD decreased and OMC increased as the percentage of fiber increased. The reduction in maximum dry unit weight is due to the lower specific gravity of teff straw fiber compare to that of the soils. The increase in optimum moisture content, on the other hand, is related to the high water absorption capacity of teff straw fiber.
- ❖ As the percentage and length of teff straw fiber increased, the unconfined compressive strength and cohesion of the soil increased for both uncured and cured conditions. The strength of the soil increased due to the fiber binding the soil particles together and the binding ability increased within the length of the fiber.
- ❖ As percentage of the fiber increased (0%, 0.5%, 0.75% and 1%) the unconfined compressive strength of the soil increased until 0.75%, and then decreased for fiber content >0.75% for all fiber length. Hence, the optimum teff straw fiber required for stabilizing the expansive soil was found as 0.75% by dry weight of the soil.
- ❖ The strength of soil increased with increasing lime from 0% to 4% and decreased for a further increase to 6% both the case of uncured and 7 days cured sample. Hence, 4% of lime is the optimum lime content to stabilize the given soil.
- ❖ The strength increased by more than double when the soil is both reinforced and treated with the optimum percentage of fiber and lime as compared to soil treated with lime only.
- ❖ Coefficient of consolidation increased both when the soil is reinforced with teff straw fiber only and reinforced with teff straw fiber and stabilized by lime. The effect is more pronounced for the reinforced and stabilized soil due to aggregation particles of soil by cation exchange and flocculation process.

- ❖ A higher coefficient of consolidation can reduce the duration of consolidation and leads to an improvement in the stability and bearing capacity of the soil. As a result, the combination of soil reinforced with 0.75% of teff straw fiber and stabilized by 4% lime is more significant to improve the stability and bearing capacity of the black cotton soil.
- ❖ The effect compression is not significant for fiber reinforcement only. Fiber reinforcement and lime stabilization can reduce the compressibility and swelling potential of soil. The reduction of compressibility of soil due to the increased formation of pozzolanic action that was increased within existing pores spaces and the decrease in swelling can be attributed to the fibers locking the soil particles together and preventing potential swelling.

Generally, a combination of teff straw fiber reinforcement and lime stabilization is more effective for ground improvement than lime stabilization alone.

5.2. Recommendation

Based on the experimental results obtained from this study, the usage of teff straw fiber in improving problematic soils, especially in combination with lime will be an alternative and economical method in highly active clayey zones. However, for the full-scale characterization of the soil behavior, further researches are needed.

- ➢ To understand the effect of teff straw fiber and lime on the microstructure of black cotton soil.
- ➢ To check the durability of the fiber under reinforcement soil.
- ➢ To understand the impact of the biological component of teff straw fiber on engineering properties of black cotton soil.

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APPENDICES

Appendix A: Values of Effective Depth and K in Hydrometer Analysis

Table A- 1: Values of effective depth based on hydrometer and sedimentation of cylinder

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective depth, L, cm	Actual Hydrometer Reading	Effective depth, L, cm	Actual Hydrometer Reading	Effective depth, L, cm
1.000	16.3	0	16.3	31	11.2
1.001	16.0	1	16.1	32	11.1
1.002	15.8	2	16.0	33	10.9
1.003	15.5	3	15.8	34	10.7
1.004	15.2	4	15.6	35	10.6
1.005	15.0	5	15.5		
1.006	14.7	6	15.5	36	10.4
1.007	14.4	7	15.3	37	10.2
1.008	14.2	8	15.2	38	10.1
1.009	13.9	9	15.0	39	9.9
1.010	13.7	10	14.8	40	9.7
1.011	13.4	11	14.7	41	9.6
1.012	13.1	12	14.5	42	9.4
1.013	12.9	13	14.3	43	9.2
1.014	12.6	14	14.2	44	9.1
1.015	12.3	15	14.0	45	8.9
1.016	12.1	16	13.8	46	8.8
1.017	11.8	17	13.7	47	8.6
1.018	11.5	18	13.5	48	8.4
1.019	11.3	19	13.2	49	8.3
1.020	11.0	20	13.0	50	8.1
1.021	10.7	21	12.9	51	7.9
1.022	10.5	22	12.7	52	7.8
1.023	10.2	23	12.5	53	7.6
1.024	10.0	24	12.4	54	7.4
1.025	9.7	25	12.2	55	7.3
1.026	9.4	26	12.0	56	7.1
1.027	9.2	27	11.9	57	7
1.028	8.9	28	11.7	58	6.8
1.029	8.6	29	11.5	59	6.6
1.030	8.4	30	11.4	60	6.5

Table A- 2: Value of K for use in equation for computing diameter of particle in hydrometer analysis

Temperature, (C^{o})	Specific Gravity, G_s			
	2.65	2.70	2.75	2.80
16	0.01435	0.01414	0.01394	0.01374
17	0.01417	0.01396	0.01376	0.01356
18	0.01399	0.01378	0.01359	0.01339
19	0.01382	0.01361	0.01342	0.01323
20	0.01365	0.01344	0.01325	0.01307
21	0.01348	0.01328	0.01309	0.01291
22	0.01332	0.01312	0.01294	0.01276
23	0.01317	0.01297	0.01279	0.01261
24	0.01301	0.01282	0.01264	0.01246
25	0.01286	0.01267	0.01249	0.01232

Appendix B: Index Properties Test Result

Table B- 1: Data for liquid limit and plastic limit test of soil from TP-1

	Liquid Limit Test				Plastic Limit Test	
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 1	Trial 2
No. of Blows, N	34	28	23	18	-------	-------
Mass of Can , M_c	49.2	47.7	48	47.8	38	37
Mass of moist Sample + Can, M_1	74	74.7	67.9	71.1	41.8	40.2
Mass of Dried Sample + Can, M_2	63	62.6	58.9	60.5	40.9	39.4
Mass of Moist , M_m=M_1-M_c	11	12.1	9	10.6	0.9	0.8
Mass of Dried Sample, M_d=M_2-M_c	13.80	14.90	10.90	12.70	2.90	2.40
Moisture, w = (M_m-M_d) / M_d x 100	79.71	81.21	82.57	83.46	31.03	33.33
Liquid limit and plastic limit	**LL = <u>81.8%</u>**				**PL = <u>32.18%</u>**	
Plasticity Index, PI = LL - PL	**PI = <u>49.62%</u>**					

Table B- 2: Data for liquid limit and plastic limit test of soil from TP-2

	Liquid Limit Test				Plastic Limit Test	
	Trial 1	Trial 2	Trial 3	Trial 4	Trial 1	Trial 2
No. of Blows, N	35	28	25	18	-------	-------
Mass of Can , M_c	47.8	49.6	47.3	47	24.6	24.4
Mass of moist Sample + Can, M_1	71.7	73.5	69.7	72.4	28	28.3
Mass of Dried Sample + Can, M_2	61.3	63	59.8	61.1	27	27.2
Mass of Moist , M_m=M_1-M_c	10.4	10.5	9.9	11.3	1	1.1
Mass of Dried Sample, M_d=M_2-M_c	13.50	13.40	12.50	14.10	2.40	2.80
Moisture, w = (M_m-M_d) / M_d x 100	77.04	78.36	79.20	80.14	41.67	39.29
Liquid limit and plastic limit	**LL = <u>78.7%</u>**				**PL = <u>40.48%</u>**	
Plasticity Index, PI = LL - PL	**PI = <u>38.22%</u>**					

Table B- 3: Data for free swell test of soil from TP-1 and TP-2

sample name	**TP-one**		**TP-Two**	
Test Number	1	2	1	2
Initial Volume (Vo)	10	10	10	10
Final Volume (Vf)	16.5	17.5	13.5	14.5
Free swell (%) = $\frac{Vf-Vo}{Vo} * 100$	65	75	35	45
Average	**70**		**40**	

Table B- 4: Wet sieve analysis for soil from TP-1 and TP-2

Wet Sieve Analysis for Soil from Pit-One				
Sieve size (mm)	Weight retained (g)	% retained	Cumulative % retained	% pass
12.5	0	0	0	100
10	1.1	0.314	0.314	99.686
5	3.8	1.086	1.4	98.6
2	4.7	1.343	2.743	97.257
0.425	9.6	2.743	5.486	94.514
0.075	8.7	2.486	7.972	92.028
Pan	322.1	92.03	100	0
Total	350			
Wet Sieve Analysis for Soil from Pit-Two				
19	0	0	0	100
12.5	10.7	3.057	3.057	96.943
10	2	0.57	3.627	96.373
5	1.9	0.543	4.17	95.83
2	3.5	1	5.17	94.83
0.425	6.7	1.9143	7.08	92.92
0.075	8.3	2.37	9.45	90.55
Pan	316.9	90.54	99.99	0
Total	350			

Table B- 5: Hydrometer analysis data for soil from TP-1

A. Sieve Analysis

Sieve size		Retained for 1st Sieving		Hydrometer Retained		Percent Passing
	(mm)	Weight(g)	% Rtd.	Weight	% Rtd.	(%)
3"	75	0	0			100
21/2"	63	0	0			100
2"	50	0	0			100
11/2"	37.5	0	0			100
1"	25	0	0			100
3/4"	19	0	0			100
1/2"	12.5	0	0			100
3/8"	9.5	0	0			100
N^o 4	4.75	0	0			100
N^o 10	2	0	0			100
N^o 16	1.18		0.00	0.60	1.20	98.80
N^o 30	0.6		0.00	0.80	1.60	97.20
N^o 50	0.3		0.00	0.70	1.40	95.80
N^o 100	0.15		0.00	0.70	1.40	94.40
N^o 200	0.075		0.00	0.60	1.20	93.20
Pan		50.00	100.00	46.60	93.20	
Total		50		50		

B. Hydrometer

Elapsed Time, t,min	Actual Hydrometer Reading R_h	Temp (^oC)	Effective Depth, L (mm) (from Table A-1)	K (From Table A-2)	Reading In Dispersant R_o	Modified Hydrometer Reading $R_d = R_h - R_o$	Particle Diameter, D (mm)	Percentage Finer Than D, P (%)
2	33.000	21	10.90	0.01309	5.000	28.000	0.031	54.46
5	32.000	21	11.10	0.01309	5.000	27.000	0.020	52.51
15	31.000	21	11.20	0.01309	5.000	26.000	0.011	50.57
30	30.000	21	11.40	0.01309	5.000	25.000	0.008	48.62
60	29.000	21	11.50	0.01309	5.000	24.000	0.006	46.68
250	28.000	22	11.70	0.01294	5.000	23.000	0.003	44.73
1440	28.000	21	11.70	0.01309	5.000	23.000	0.001	44.73

Calculation:

Hydrometer No = 152H , Specific Gravity, G_s = 2.78, Dry mass of Soil, m = 50g

$C = \frac{(100{,}000/m)*Gs}{Gs-1}$, $D = K* SQRT(L / t)$, $P = C* R_d$

Test result for soil from test pit-one

Gravel	Nil
Sand	6.80%
Silt	48.20%
Clay	45.00%

Table B- 6: Hydrometer analysis data for soil from TP-2

A. Sieve Analysis

Sieve size		Retained for 1st Sieving		Hydrometer Retained		Percent Passing
	(mm)	Weight(g)	% Rtd.	Weight	% Rtd.	(%)
3"	75	0	0			100
21/2"	63	0	0			100
2"	50	0	0			100
11/2"	37.5	0	0			100
1"	25	0	0			100
3/4"	19	0	0			100
1/2"	12.5	0	0			100
3/8"	9.5	0	0			100
N^o 4	4.75	0	0			100
N^o 10	2	0	0			100
N^o 16	1.18		0.00	0.30	0.60	99.40
N^o 30	0.6		0.00	0.80	1.60	97.80
N^o 50	0.3		0.00	1.00	2.00	95.80
N^o 100	0.15		0.00	0.60	1.20	94.60
N^o 200	0.075		0.00	0.50	1.00	93.60
Pan		50.00	100.00	46.80	93.60	
Total		50		50		

B. Hydrometer

Elapsed Time, t,min	Actual Hydrometer Reading R_h	Temp (^oC)	Effective Depth, L (mm) (from Table A-1)	K (From Table A-2)	Reading In Dispersant R_o	Modified Hydrometer Reading $R_d = R_h - R_o$	Particle Diameter, D (mm)	Percentage Finer Than D, P (%)
2	28.000	21	11.70	0.01328	5.000	23.000	0.032	45.49
5	27.000	21	11.90	0.01328	5.000	22.000	0.020	43.51
15	27.000	21	11.90	0.01328	5.000	22.000	0.012	43.51
30	26.000	21	12.00	0.01328	5.000	21.000	0.008	41.53
60	25.000	21	12.20	0.01328	5.000	20.000	0.006	39.56
250	22.000	22	12.70	0.01312	5.000	17.000	0.003	33.62
1440	22.000	21	12.70	0.01328	5.000	17.000	0.001	33.62

Calculation:

Hydrometer No = 152H , Specific Gravity, G_s = 2.7, Dry mass of Soil, m = 50g

$C = \frac{(100{,}000/m)*Gs}{Gs-1}$, D = K* SQRT(L / t), $P = C * R_d$

Test Result for soil from test pit- two

Gravel	Nil
Sand	6.40%
Silt	60.00%
Clay	33.60%

Table B- 7: Specific gravity test for soils and soil reinforced samples

Sample ID	Test Pit-One		Test Pit-Two		Soil with 0.5% fiber		Soil with 0.75% fiber		Soil with 1% fiber	
Test Number	1	2	1	2	1	2	1	2	1	2
Mass of dry soil (A) (g)	10	10	10	10	15	15	10	10	15	15
Mass of flask filled with water (B) (g)	141.1	140	137.5	137.4	159.6	159.5	137.6	137.5	157.3	155.4
Mass of flask filled with soil and water (g) (C)	147.4	146.5	143.9	143.6	168.9	168.8	143.8	143.7	166.5	164.7
Specific gravity= $\frac{A}{A+(B-C)}$	2.7	2.85	2.78	2.63	2.64	2.63	2.63	2.63	2.60	2.63
Average	2.78		2.7		2.64		2.63		2.62	

Table B- 8: Specific gravity test for soils and soil reinforced and lime stabilized samples

Sample ID	Soil with 4% of lime		Soil with 0.5% of fiber and 4% of lime		Soil with 0.75% of fiber and 4% of lime		Soil with 1% of fiber and 4% of lime	
Test Number	1	2	1	2	1	2	1	2
Mass of dry soil (A) (g)	10	10	15	15	10	10	15	15
Mass of flask filled with water (B) (g)	140.79	140.4	158.18	157.8	143.6	142.64	158.3	157.49
Mass of flask filled with soil and water (g) (C)	147	146.6	167.48	167.08	149.8	148.8	167.56	166.75
Specific gravity= $\frac{A}{A+(B-C)}$	2.64	2.63	2.63	2.62	2.63	2.60	2.61	2.61
Average	2.64		2.63		2.62		2.61	

Table B- 9: Test result for water observation of teff straw fiber

Sample name	Teff straw fiber	
Test Number	1	2
Mass of dry fiber (g) (A)	9.5	11.6
Mass of saturated surface dry (g) (B)	24.6	28.8
Water Observation (%) = $\frac{B-A}{B}$ *100	61.54	59.72
Average	60.7%	

Appendix C: Compaction Test Result

Table C- 1: Data for compaction test of soil from TP-1 and TP-2

	Soil from Test Pit- One				Soil from Test Pit-Two			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Weight of Soil + Mold(g)	4970	5100	5230	5250	4930	5100	5250	5280
Weight of Mold (g)	3530	3530	3530	3530	3530	3530	3530	3530
Soil of Sample (g)	1440	1570	1700	1720	1400	1570	1720	1750
Volume of Mold (cm^3)	944	944	944	944	944	944	944	944
Bulk density (kg/m^3)	1525	1663	1801	1822	1483	1663	1822	1854
Weight of wet soil + Can	168.3	169.80	172.60	178.30	160.4	169.30	170.10	175.00
Weight of dry Soil + Can	147.9	145.8	144.5	143.7	142.8	146.7	143.4	139
Weight of Can	26.3	25.6	27.7	28.1	26.2	25.2	26.2	27.4
Weight of moisture	20.4	24.00	28.10	34.60	17.6	22.60	26.70	36.00
Weight of dry soil	121.6	120.2	116.8	115.6	116.6	121.5	117.2	111.6
Moisture content (%)	16.78	19.97	24.06	29.93	15.09	18.60	22.78	32.26
Dry density (kg/m^3)	1306	1386	1452	1402	1289	1402	1484	1402
OMC (%)	**24.06**				**22.78**			
MDD (kg/m^3)	**1452**				**1484**			

Table C- 2: Standard compaction test for soil reinforced with 20mm length of fiber

	Soil + 0.5% of fiber				Soil + 0.75% of fiber				Soil + 1% of fiber			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Wt Soil+Mold	4740	4960	5080	5060	4770	4880	5020	5000	4780	4890	4980	4970
Wt of Mold	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270
Soil of Sample(g)	1470	1690	1810	1790	1500	1610	1750	1730	1510	1620	1710	1700
Volume of Mold (cm^3)	944	944	944	944	944	944	944	944	944	944	944	944
Bulk density(kg/m^3)	1557	1790	1917	1896	1589	1706	1854	1833	1600	1716	1811	1801
Wt of wet soil + Can	125.4	131.1	153.2	142.2	141.5	141.2	145.8	143.6	127.2	142	152.8	144.4
Wt Dry Soil + Can	107	107.8	122.2	111.5	123.9	115.2	115.9	112.9	105.1	114.3	119.2	111.4
Wt Can	25.6	27.3	27.6	28.8	55.5	25.6	27.6	29	26.9	28.5	27.3	27.4
Wt of moisture	18.40	23.3	31.0	30.7	17.6	26.0	29.9	30.7	22.1	27.7	33.6	33.0
Wt of dry soil	81.4	80.5	94.6	82.7	68.4	89.6	88.3	83.9	78.2	85.8	91.9	84
Moisture content (%)	22.60	28.94	32.77	37.12	25.73	29.02	33.86	36.59	28.26	32.28	36.56	39.29
Dry density (kg/m^3)	1270	1388	1444	1383	1264	1322	1385	1342	1247	1297	1326	1293
OMC (%)	**32.77**				**33.86**				**36.56**			
MDD(kg/m^3)	**1444**				**1385**				**1326**			

Table C- 3: Standard compaction test for soil reinforced with 40mm length of fiber

	Soil + 0.5% of fiber				Soil + 0.75% of fiber				Soil + 1% of fiber			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Wt Soil + Mold (g)	4720	4910	5000	4990	4760	4860	4950	4940	4750	4880	4930	4920
Wt of Mold (g)	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270
Wt of Soil (g)	1450	1640	1730	1720	1490	1590	1680	1670	1480	1610	1660	1650
Volume of Mold (cm^3)	944	944	944	944	944	944	944	944	944	944	944	944
Bulk density (kg/m^3)	1536	1737	1833	1822	1578	1684	1780	1769	1568	1706	1758	1748
Wt of wet soil + Can (g)	139.2	137.3	140.0	143.3	137.6	134.1	134.5	132.4	140.5	146.4	143.6	157.3
Wt Dry Soil + Can (g)	119.3	114.8	115.1	115.6	114.6	110.5	109.7	105.2	112.8	116.8	113.5	121.6
Wt Can (g)	26.6	26.5	27.2	25.7	27.5	27.0	29.1	26.1	23.6	27.7	27.1	28.1
Wt of dry Soil (g)	19.9	22.5	24.9	27.7	23.0	23.6	24.8	27.2	27.7	29.6	30.1	35.7
Wt of Moisture (g)	92.7	88.3	87.9	89.9	87.1	83.5	80.6	79.1	89.2	89.1	86.4	93.5
Moisture content (%)	21.5	25.5	28.3	30.8	26.4	28.3	30.8	34.4	31.1	33.2	34.8	38.2
Dry density (kg/m^3)	1273	1384	1428	1393	1249	1313	1361	1316	1196	1272	1304	1265
OMC (%)	**28.3**				**30.8**				**34.8**			
MDD (kg/m^3)	**1428**				**1361**				**1304**			

Table C- 4: Standard compaction test for soil reinforced with 60mm length of fiber

	Soil + 0.5% of fiber				Soil + 0.75% of fiber				Soil + 1% of fiber			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Wt Soil + Mold (g)	5070	5160	5220	5220	5010	5080	5170	5160	5010	5070	5170	5170
Wt of Mold (g)	3550	3550	3550	3550	3550	3550	3550	3550	3550	3550	3550	3550
Soil of Sample(g)	1520	1610	1670	1670	1460	1530	1620	1610	1460	1520	1620	1620
Volume of Mold(cm^3)	944	944	944	944	944	944	944	944	944	944	944	944
Bulk density(kg/m^3)	1610	1706	1769	1769	1547	1621	1716	1706	1547	1610	1716	1716
Wt of wet soil + Can	127.7	128.6	128.6	129.4	158.90	157.8	139.9	156.9	121.9	135	136.7	138.2
Wt Dry Soil + Can	109.3	109.1	107.6	106.7	134.1	131.1	115.5	126.3	102.6	111.4	109.2	108.8
Wt Can (g)	27.1	28.8	27.3	27.5	28.8	25.7	28.9	26.8	27.5	27.6	26	27.3
Wt of dry Soil (g)	18.4	19.5	21.0	22.7	24.8	26.7	24.4	30.6	19.3	23.6	27.5	29.4
Wt of Moisture (g)	82.2	80.3	80.3	79.2	105.3	105.4	86.6	99.5	75.1	83.8	83.2	81.5
Moisture content (%)	22.4	24.3	26.2	28.7	23.5	25.3	28.2	30.8	25.7	28.2	33.1	36.1
Dry density (kg/m^3)	1316	1372	1402	1375	1252	1293	1339	1304	1230	1256	1290	1261
OMC (%)	**26.2**				**28.2**				**33.1**			
MDD (kg/m^3)	**1402**				**1339**				**1290**			

Table C- 5: Standard compaction test for soil reinforced and lime stabilized with different percentages of lime

	Soil + 2% of lime + 0.75% of fiber				Soil + 4% of lime + 0.75% of fiber				Soil + 6% of lime + 0.75% of fiber			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Wt Soil + Mold (g)	4740	4810	4900	4900	4800	4850	4900	4920	4780	4850	4920	4910
Wt of Mold (g)	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270
Soil of Sample (g)	1470	1540	1630	1630	1530	1580	1630	1650	1510	1580	1650	1640
Volume of Mold(cm^3)	944	944	944	944	944	944	944	944	944	944	944	944
Bulk density (kg/m^3)	1557	1631	1727	1727	1621	1674	1727	1748	1600	1674	1748	1737
Wt of wet soil + Can	147	149.00	148.00	145.20	97.0	99.5	142.4	142.9	141.5	146.8	148.7	148.7
Wt Dry Soil + Can	125	124.2	120	116.1	81.5	82.7	114.1	112.4	115.7	118.0	117.1	116.0
Wt Can (g)	26	26.5	26.9	25.2	26.3	26.7	26.1	26.0	25.8	26.9	26.7	26.7
Wt of moisture (g)	22	24.80	28.00	29.10	15.5	16.8	28.3	30.5	25.8	28.8	31.6	32.7
Wt of dry soil (g)	99	97.7	93.1	90.9	55.2	56.0	88.0	86.4	89.9	91.1	90.4	89.3
Moisture content (%)	22.22	25.38	30.08	32.01	28.1	30.0	32.2	35.3	28.7	31.6	35.0	36.6
Dry density (kg/m^3)	1274	1301	1327	1308	1265	1288	1308	1293	1243	1272	1295	1273
OMC (%)	**30.08**				**32.2**				**35**			
MDD (kg/m^3)	**1327**				**1308**				**1295**			

Table C- 6: Standard compaction test for soil reinforced and lime stabilized with different percentages of fiber

	Soil + 4% of lime				Soil + 4% of lime + 0.5% of fiber				Soil + 4% of lime + 1% of fiber			
	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4	Trial 1	Trail 2	Trail 3	Trail 4
Wt Soil + Mold (g)	4840	4930	5010	5000	4840	4890	4950	4960	4800	4850	4890	4900
Wt of Mold (g)	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270	3270
Soil of Sample (g)	1570	1660	1740	1730	1570	1620	1680	1690	1530	1580	1620	1630
Volume of Mold (cm^3)	944	944	944	944	944	944	944	944	944	944	944	944
Bulk density (kg/m^3)	1663	1758	1843	1833	1663	1716	1780	1790	1621	1674	1716	1727
Wt of wet soil + Can	135.9	139.2	139.4	138.9	139.5	137.4	138.5	136.8	124.2	129.8	126.8	126.2
Wt Dry Soil + Can	114.4	114.9	113.4	110.7	116.2	112.9	111.7	109.1	101.8	104.9	101.6	99.5
Wt Can (g)	27.3	26.1	27.3	27.2	27.5	26.5	26.6	29	27.3	26.2	26.1	25.8
Wt of moisture (g)	21.5	24.3	26.0	28.2	23.3	24.5	26.8	27.7	22.4	24.9	25.2	26.7
Wt of dry soil (g)	87.1	88.8	86.1	83.5	88.7	86.4	85.1	80.1	74.5	78.7	75.5	73.7
Moisture content (%)	24.68	27.36	30.20	33.77	26.27	28.36	31.49	34.58	30.07	31.64	33.38	36.23
Dry density (kg/m^3)	1334	1381	1416	1370	1317	1337	1353	1330	1246	1271	1287	1268
OMC (%)	**30.02**				**31.49**				**33.38**			
MDD (kg/m^3)	**1416**				**1353**				**1287**			

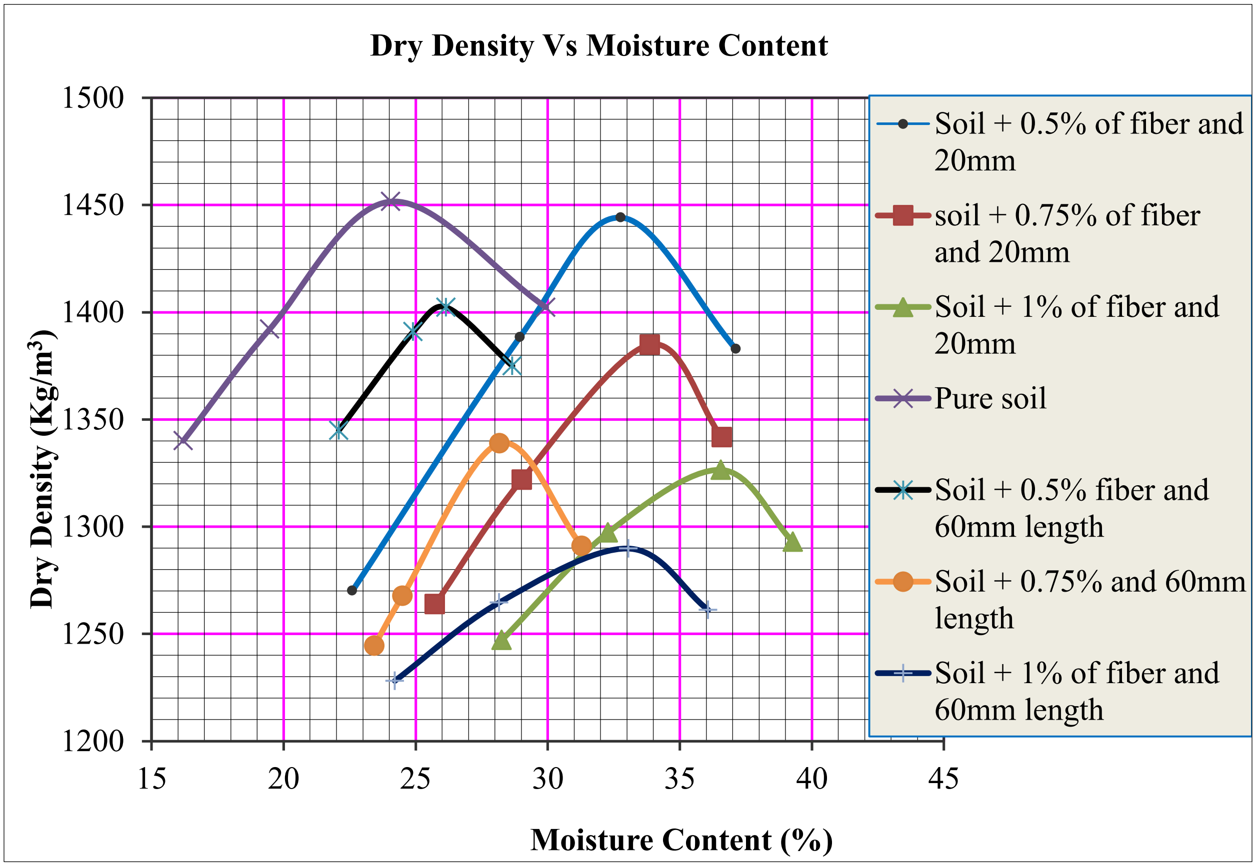

Figure C- 1: Compaction curve for soil reinforced with 20 and 60mm lengths of fiber

Appendix D: Unconfined Compressive Strength Test Result

For all UCS test samples

Initial Diameter (cm)	3.7
Initial Height H_o, (mm)	76
Initial Area A_o (m^2)	0.0010
Initial Volume (cm^3)	81.67
P.R.Factor (KN/div)	0.00153

Table D- 1: UCS test data for pure soil uncured and 7days cured condition

Sample Name	Deformation dial in [mm] ΔH	Unit strain $\boldsymbol{\varepsilon}$ = ΔH/Ho	Proving Ring Reading	Load in [KN]	Cross - sectional Area A = $A_o/(1-\boldsymbol{\varepsilon})$	Stress in [KN/m^2]
Pure Soil uncured condtion	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	4	0.0061	0.00100	6.1
	0.40	0.0053	8	0.0122	0.00101	12.2
	0.60	0.0079	15	0.0230	0.00101	22.8
	0.80	0.0105	23	0.0352	0.00101	34.8
	1.00	0.0132	27	0.0413	0.00101	**40.8**
	1.20	0.0158	27	0.0413	0.00102	40.7
	1.40	0.0184	26	0.0398	0.00102	39.0
	1.60	0.0211	25	0.0383	0.00102	37.4
Pure Soil with 7days cured	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	7	0.0107	0.00100	10.7
	0.40	0.0053	11	0.0168	0.00101	16.7
	0.60	0.0079	17	0.0260	0.00101	25.8
	0.80	0.0105	24	0.0367	0.00101	36.3
	1.00	0.0132	29	0.0444	0.00101	43.8
	1.20	0.0158	33	0.0505	0.00102	49.7
	1.40	0.0184	34	0.0520	0.00102	51.1
	1.60	0.0211	35	0.0536	0.00102	52.4
	1.80	0.0237	35	0.0536	0.00102	**52.3**
	2.00	0.0263	34	0.0520	0.00103	50.7
	2.20	0.0289	33	0.0505	0.00103	49.0

Table D- 2: UCS test data for soil reinforced with 20mm length of fiber uncured condition

Sample Name	Deformation dial in [mm] ΔH	Unit strain ε = ΔH/Ho	Proving Ring Reading	Load in [KN]	Cross - sectional Area A = $A_o/(1-\varepsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	4	0.0061	0.00100	6.1
	0.40	0.0053	10	0.0153	0.00101	15.2
	0.60	0.0079	16	0.0245	0.00101	24.3
	0.80	0.0105	21	0.0321	0.00101	31.8
	1.00	0.0132	24	0.0367	0.00101	36.2
	1.20	0.0158	27	0.0413	0.00102	40.7
	1.40	0.0184	28	0.0428	0.00102	42.1
	1.60	0.0211	28.5	0.0436	0.00102	**42.7**
	1.80	0.0237	28.5	0.0436	0.00102	42.6
	2.00	0.0263	27	0.0413	0.00103	40.2
	2.20	0.0289	26	0.0398	0.00103	38.6
Soil Reinforced with 1% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	2	0.0031	0.00100	3.1
	0.40	0.0053	5	0.0077	0.00101	7.6
	0.60	0.0079	7	0.0107	0.00101	10.6
	0.80	0.0105	8	0.0122	0.00101	12.1
	1.00	0.0132	9	0.0138	0.00101	13.6
	1.20	0.0158	11	0.0168	0.00102	16.6
	1.40	0.0184	12	0.0184	0.00102	18.0
	1.60	0.0211	14	0.0214	0.00102	21.0
	1.80	0.0237	15	0.0230	0.00102	22.4
	2.00	0.0263	16	0.0245	0.00103	23.8
	2.20	0.0289	17	0.0260	0.00103	25.3
	2.40	0.0316	18	0.0275	0.00103	26.7
	2.60	0.0342	19	0.0291	0.00104	**28.1**
	2.80	0.0368	19	0.0291	0.00104	28.0
	3.00	0.0395	18	0.0275	0.00104	26.5
	3.20	0.0421	17	0.0260	0.00104	24.9

Sample name	Deformation dial in [mm] ΔH	Unit strain ϵ = ΔH/ΔHo	Proving Ring Reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\epsilon$	Stress in [KN/m^2]
Soil Reinforced with 0.75% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	8	0.0122	0.00101	12.2
	0.60	0.0079	13	0.0199	0.00101	19.7
	0.80	0.0105	17	0.0260	0.00101	25.7
	1.00	0.0132	21	0.0321	0.00101	31.7
	1.20	0.0158	24	0.0367	0.00102	36.1
	1.40	0.0184	27	0.0413	0.00102	40.5
	1.60	0.0211	29	0.0444	0.00102	43.4
	1.80	0.0237	31	0.0474	0.00102	46.3
	2.00	0.0263	32	0.0490	0.00103	47.7
	2.20	0.0289	33	0.0505	0.00103	49.0
	2.40	0.0316	33.5	0.0513	0.00103	49.6
	2.60	0.0342	34	0.0520	0.00104	50.2
	2.80	0.0368	34.5	0.0528	0.00104	50.8
	3.00	0.0395	35	0.0536	0.00104	51.4
	3.20	0.0421	35.5	0.0543	0.00104	52.0
	3.40	0.0447	36	0.0551	0.00105	**52.6**
	3.60	0.0474	36	0.0551	0.00105	52.5
	3.80	0.0500	35	0.0536	0.00105	50.9
	4.00	0.0526	34.5	0.0528	0.00106	50.0

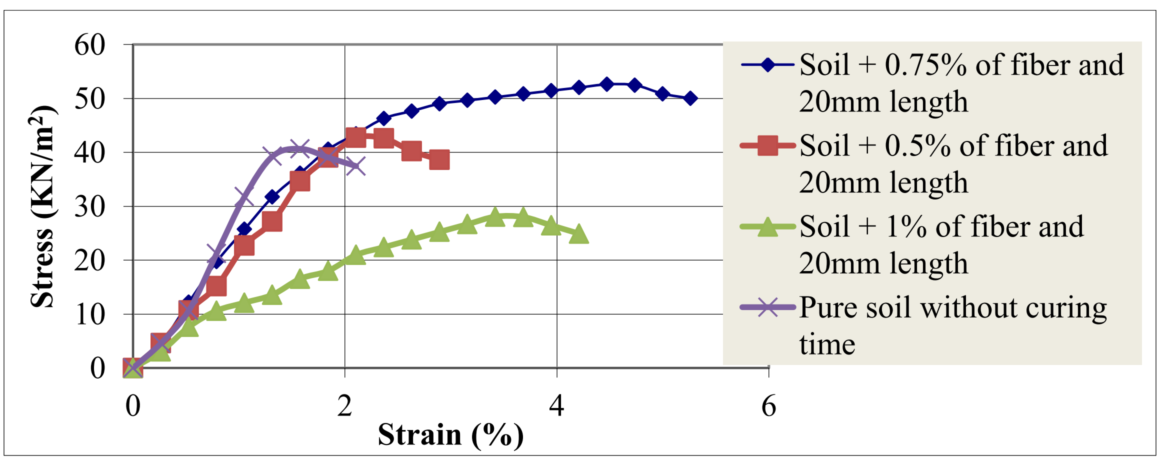

Figure D- 1: Stress-Strain relationship of soil reinforced with 20mm length uncured

Table D- 3: UCS test data for soil reinforced with 20mm length of fiber with 7days cured

Sample Name	Deformation dial in [mm] ΔH	Unit strain $\varepsilon = \Delta H/Ho$	Proving Ring Reading	Load in [KN]	Cross - sectional Area $A = A_o/(1-\varepsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	11	0.0168	0.00100	16.8
	0.40	0.0053	17	0.0260	0.00101	25.9
	0.60	0.0079	22	0.0337	0.00101	33.4
	0.80	0.0105	28	0.0428	0.00101	42.4
	1.00	0.0132	32	0.0490	0.00101	48.3
	1.20	0.0158	34	0.0520	0.00102	51.2
	1.40	0.0184	35	0.0536	0.00102	52.6
	1.60	0.0211	35.5	0.0543	0.00102	**53.2**
	1.80	0.0237	34	0.0520	0.00102	50.8
	2.00	0.0263	33	0.0505	0.00103	49.2
	2.20	0.0289	32	0.0490	0.00103	47.5
Soil Reinforced with 0.75% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	15	0.0230	0.00100	22.9
	0.40	0.0053	25	0.0383	0.00101	38.0
	0.60	0.0079	35	0.0536	0.00101	53.1
	0.80	0.0105	40	0.0612	0.00101	60.6
	1.00	0.0132	44	0.0673	0.00101	66.4
	1.20	0.0158	46	0.0704	0.00102	69.3
	1.40	0.0184	47	0.0719	0.00102	70.6
	1.60	0.0211	48	0.0734	0.00102	71.9
	1.80	0.0237	49	0.0750	0.00102	73.2
	2.00	0.0263	50	0.0765	0.00103	74.5
	2.20	0.0289	50	0.0765	0.00103	74.3
	2.40	0.0316	49	0.0750	0.00103	**72.6**
	2.60	0.0342	48	0.0734	0.00104	70.9

Sample Name	Deformation dial in [mm] ΔH	Unit strain $\epsilon = \Delta H/\Delta Ho$	Proving Ring reading	Load in [KN]	Cross-sectional Area $A = A_o/(1-\epsilon)$	Stress in [KN/m^2]
Soil Reinforced with 1% of fiber and 20mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	8	0.0122	0.00101	12.2
	0.60	0.0079	14	0.0214	0.00101	21.3
	0.80	0.0105	18	0.0275	0.00101	27.3
	1.00	0.0132	20	0.0306	0.00101	30.2
	1.20	0.0158	21	0.0321	0.00102	31.6
	1.40	0.0184	22.5	0.0344	0.00102	33.8
	1.60	0.0211	24	0.0367	0.00102	35.9
	1.80	0.0237	25.5	0.0390	0.00102	38.1
	2.00	0.0263	26	0.0398	0.00103	38.7
	2.20	0.0289	26.5	0.0405	0.00103	39.4
	2.40	0.0316	27.5	0.0421	0.00103	40.7
	2.60	0.0342	28	0.0428	0.00104	41.4
	2.80	0.0368	28.5	0.0436	0.00104	42.0
	3.00	0.0395	29	0.0444	0.00104	42.6
	3.20	0.0421	29.5	0.0451	0.00104	**43.2**
	3.40	0.0447	29.5	0.0451	0.00105	43.1
	3.60	0.0474	28	0.0428	0.00105	40.8
	3.80	0.0500	27	0.0413	0.00105	39.2

Table D- 4: UCS test data for soil reinforced with 40mm length of fiber uncured condition

Sample Name	Deformation dial in [mm] ΔH	Unit strain ε = ΔH/Ho	Proving Ring Reading	Load-in [KN]	Cross-sectional Area A = $A_o/(1-\varepsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	10	0.0153	0.00100	15.3
	0.40	0.0053	14	0.0214	0.00101	21.3
	0.60	0.0079	17	0.0260	0.00101	25.8
	0.80	0.0105	20	0.0306	0.00101	30.3
	1.00	0.0132	22	0.0337	0.00101	33.2
	1.20	0.0158	23	0.0352	0.00102	34.6
	1.40	0.0184	24	0.0367	0.00102	36.0
	1.60	0.0211	26	0.0398	0.00102	38.9
	1.80	0.0237	27	0.0413	0.00102	40.3
	2.00	0.0263	28	0.0428	0.00103	41.7
	2.20	0.0289	29	0.0444	0.00103	**43.1**
	2.40	0.0316	29	0.0444	0.00103	43.0
	2.60	0.0342	28	0.0428	0.00104	41.4
Soil Reinforced with 0.75% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	10	0.0153	0.00100	15.3
	0.40	0.0053	15	0.0230	0.00101	22.8
	0.60	0.0079	19	0.0291	0.00101	28.8
	0.80	0.0105	22	0.0337	0.00101	33.3
	1.00	0.0132	25	0.0383	0.00101	37.7
	1.20	0.0158	28	0.0428	0.00102	42.2
	1.40	0.0184	31	0.0474	0.00102	46.6
	1.60	0.0211	34	0.0520	0.00102	50.9
	1.80	0.0237	36	0.0551	0.00102	53.8
	2.00	0.0263	37.5	0.0574	0.00103	55.9
	2.20	0.0289	38.5	0.0589	0.00103	57.2
	2.40	0.0316	39	0.0597	0.00103	**57.8**
	2.60	0.0342	39	0.0597	0.00104	57.6
	2.80	0.0368	38	0.0581	0.00104	56.0
	3.00	0.0395	37	0.0566	0.00104	54.4

Sample Name	Deformation dial in [mm] ΔH	Unit strain ϵ = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross-sectional Area A = $A_o/(1-\epsilon)$	Stress in [KN/m^2]
Soil Reinforced with 1% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	7	0.0107	0.00101	10.7
	0.60	0.0079	11	0.0168	0.00101	16.7
	0.80	0.0105	13	0.0199	0.00101	19.7
	1.00	0.0132	15	0.0230	0.00101	22.6
	1.20	0.0158	16	0.0245	0.00102	24.1
	1.40	0.0184	17	0.0260	0.00102	25.5
	1.60	0.0211	19	0.0291	0.00102	28.5
	1.80	0.0237	21	0.0321	0.00102	31.4
	2.00	0.0263	22	0.0337	0.00103	32.8
	2.20	0.0289	23	0.0352	0.00103	**34.2**
	2.40	0.0316	23	0.0352	0.00103	34.1
	2.60	0.0342	22	0.0337	0.00104	32.5
	2.80	0.0368	21	0.0321	0.00104	30.9

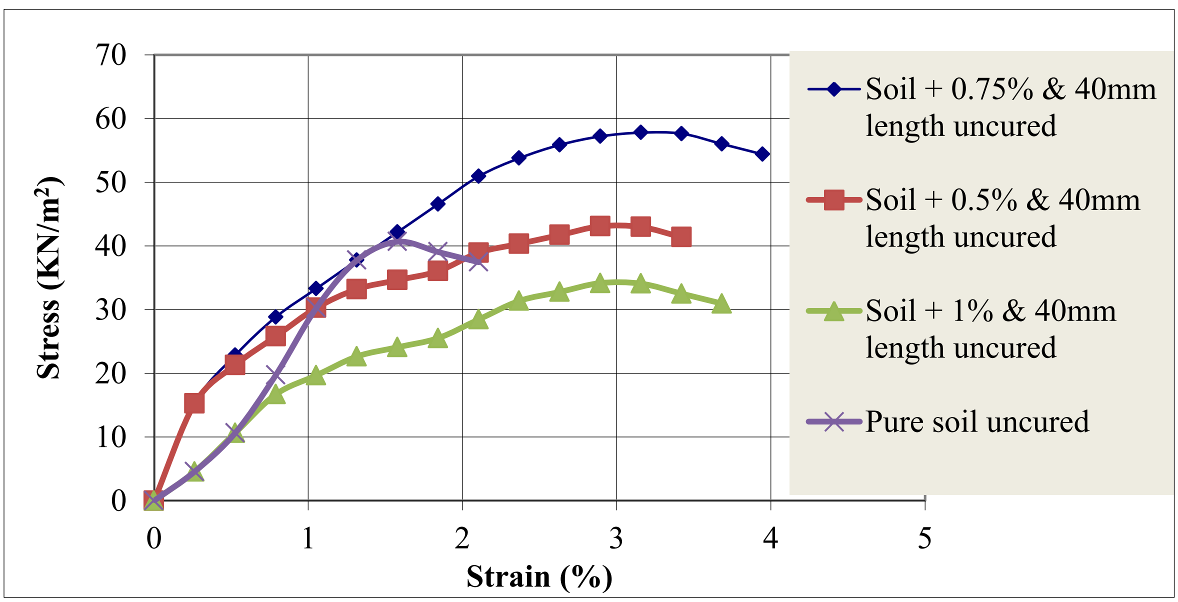

Figure D- 2: Stress-Strain relationship of soil reinforced with 40mm length uncured condition

Table D- 5: UCS test data for soil reinforced with 40mm length of fiber with 7days cured

Sample Name	Deformation dial in [mm] ΔH	Unit Strain ε = ΔH/Ho	Proving Ring reading	Load in [KN]	Cross-sectional Area A = $A_o/(1-\varepsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	11	0.0168	0.00100	16.8
	0.40	0.0053	16	0.0245	0.00101	24.4
	0.60	0.0079	21	0.0321	0.00101	31.9
	0.80	0.0105	25	0.0383	0.00101	37.8
	1.00	0.0132	28	0.0428	0.00101	42.3
	1.20	0.0158	31	0.0474	0.00102	46.7
	1.40	0.0184	33	0.0505	0.00102	49.6
	1.60	0.0211	34	0.0520	0.00102	50.9
	1.80	0.0237	35	0.0536	0.00102	52.3
	2.00	0.0263	36	0.0551	0.00103	53.6
	2.20	0.0289	37	0.0566	0.00103	55.0
	2.40	0.0316	37.5	0.0574	0.00103	55.6
	2.60	0.0342	38	0.0581	0.00104	**56.2**
	2.80	0.0368	38	0.0581	0.00104	56.0
	3.00	0.0395	37	0.0566	0.00104	54.4
Soil Reinforced with 0.75% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	13	0.0199	0.00100	19.8
	0.40	0.0053	19	0.0291	0.00101	28.9
	0.60	0.0079	25	0.0383	0.00101	37.9
	0.80	0.0105	29	0.0444	0.00101	43.9
	1.00	0.0132	35	0.0536	0.00101	52.8
	1.20	0.0158	39	0.0597	0.00102	58.7
	1.40	0.0184	43	0.0658	0.00102	64.6
	1.60	0.0211	48	0.0734	0.00102	71.9
	1.80	0.0237	50	0.0765	0.00102	74.7
	2.00	0.0263	51	0.0780	0.00103	76.0
	2.20	0.0289	52	0.0796	0.00103	**77.3**
	2.40	0.0316	52	0.0796	0.00103	77.0
	2.60	0.0342	51	0.0780	0.00104	75.4
	2.80	0.0368	50	0.0765	0.00104	73.7

Sample Name	Deformation dial in [mm] ΔH	Unit Strain ϵ = ΔH/ΔHo	Proving Ring Reading	Load in [KN]	Cross-sectional Area A = $A_o/(1-\epsilon)$	Stress in [KN/m^2]
Soil Reinforced with 1% of fiber and 40mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	5	0.0077	0.00100	7.6
	0.40	0.0053	8	0.0122	0.00101	12.2
	0.60	0.0079	11	0.0168	0.00101	16.7
	0.80	0.0105	13	0.0199	0.00101	19.7
	1.00	0.0132	16	0.0245	0.00101	24.2
	1.20	0.0158	18	0.0275	0.00102	27.1
	1.40	0.0184	20	0.0306	0.00102	30.0
	1.60	0.0211	22	0.0337	0.00102	33.0
	1.80	0.0237	23	0.0352	0.00102	34.4
	2.00	0.0263	25	0.0383	0.00103	37.2
	2.20	0.0289	26	0.0398	0.00103	38.6
	2.40	0.0316	27	0.0413	0.00103	40.0
	2.60	0.0342	28	0.0428	0.00104	41.4
	3.00	0.0395	29	0.0444	0.00104	42.6
	3.20	0.0421	30	0.0459	0.00104	**44.0**
	3.40	0.0447	30	0.0459	0.00105	43.8
	3.60	0.0474	29	0.0444	0.00105	42.3
	3.80	0.0500	28	0.0428	0.00105	40.7

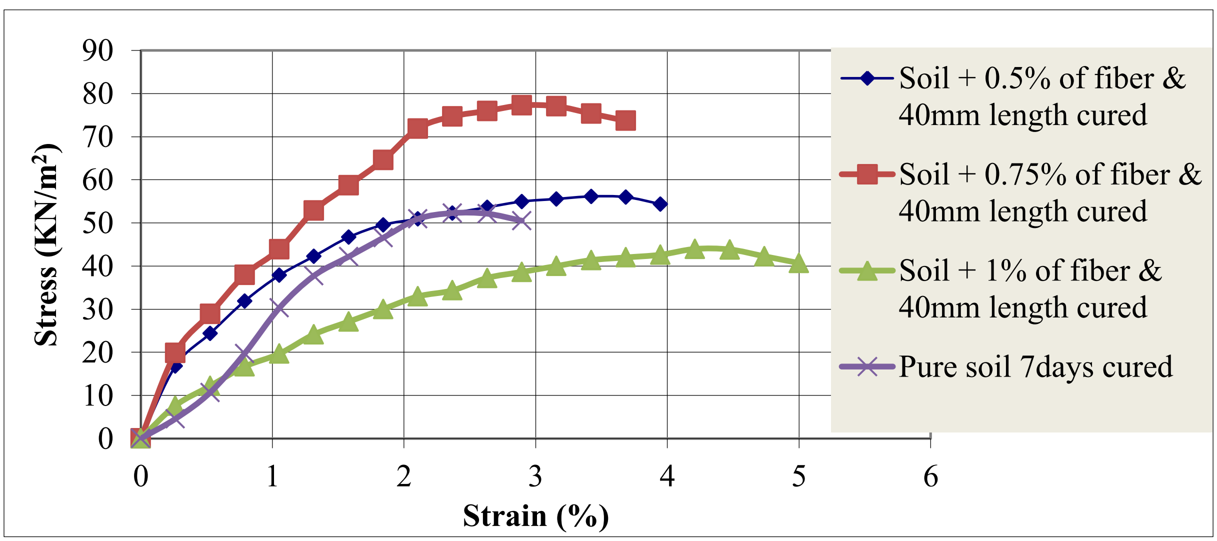

Figure D- 3: Stress-Strain relationship of soil reinforced with 40mm length with 7days cured

Table D- 6: UCS test data for soil reinforced with 60mm length of fiber uncured condition

Sample Name	Deformation dial in [mm] ΔH	Unit Strain ε = ΔH/Ho	Proving Ring Reading	Load in [KN]	Cross-sectional Area A = $A_o/(1-\varepsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	6	0.0092	0.00101	9.1
	0.60	0.0079	11	0.0168	0.00101	16.7
	0.80	0.0105	16	0.0245	0.00101	24.2
	1.00	0.0132	20	0.0306	0.00101	30.2
	1.20	0.0158	23	0.0352	0.00102	34.6
	1.40	0.0184	25	0.0383	0.00102	37.5
	1.60	0.0211	26	0.0398	0.00102	38.9
	1.80	0.0237	27	0.0413	0.00102	40.3
	2.00	0.0263	28	0.0428	0.00103	41.7
	2.20	0.0289	30	0.0459	0.00103	44.6
	2.40	0.0316	31	0.0474	0.00103	45.9
	2.60	0.0342	32	0.0490	0.00104	**47.3**
	3.00	0.0395	31	0.0474	0.00104	45.6
	3.20	0.0421	30	0.0459	0.00104	44.0
Soil Reinforced with 1% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	4	0.0061	0.00100	6.1
	0.40	0.0053	7	0.0107	0.00101	10.7
	0.60	0.0079	9	0.0138	0.00101	13.7
	0.80	0.0105	13	0.0199	0.00101	19.7
	1.00	0.0132	15	0.0230	0.00101	22.6
	1.20	0.0158	18	0.0275	0.00102	27.1
	1.40	0.0184	19	0.0291	0.00102	28.5
	1.60	0.0211	21	0.0321	0.00102	31.5
	1.80	0.0237	23	0.0352	0.00102	34.4
	2.00	0.0263	25	0.0383	0.00103	37.2
	2.20	0.0289	26	0.0398	0.00103	38.6
	2.40	0.0316	27	0.0413	0.00103	40.0
	2.60	0.0342	28	0.0428	0.00104	**41.4**
	3.00	0.0395	27	0.0413	0.00104	39.7

Sample Name	Deformation dial in [mm] ΔH	Unit Strain ϵ = ΔH/ΔHo	Proving Ring Reading	Load in [KN]	Cross-sectional Area A = $A_o/(1-\epsilon)$	Stress in [KN/m^2]
Soil Reinforced with 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	9	0.0138	0.00100	13.7
	0.40	0.0053	13	0.0199	0.00101	19.8
	0.60	0.0079	16	0.0245	0.00101	24.3
	0.80	0.0105	20	0.0306	0.00101	30.3
	1.00	0.0132	23	0.0352	0.00101	34.7
	1.20	0.0158	25	0.0383	0.00102	37.6
	1.40	0.0184	26	0.0398	0.00102	39.0
	1.60	0.0211	28	0.0428	0.00102	41.9
	1.80	0.0237	29	0.0444	0.00102	43.3
	2.00	0.0263	30	0.0459	0.00103	44.7
	2.20	0.0289	32	0.0490	0.00103	47.5
	2.40	0.0316	33	0.0505	0.00103	48.9
	2.60	0.0342	34	0.0520	0.00104	50.2
	2.80	0.0368	35	0.0536	0.00104	51.6
	3.00	0.0395	36	0.0551	0.00104	52.9
	3.20	0.0421	37	0.0566	0.00104	54.2
	3.40	0.0447	38	0.0581	0.00105	55.5
	3.60	0.0474	39	0.0597	0.00105	56.8
	3.80	0.0500	40	0.0612	0.00105	58.1
	4.00	0.0526	41	0.0627	0.00106	59.4
	4.20	0.0553	42	0.0643	0.00106	60.7
	4.40	0.0579	43	0.0658	0.00106	62.0
	4.60	0.0605	44	0.0673	0.00106	63.2
	4.80	0.0632	45	0.0689	0.00107	**64.5**
	5.00	0.0658	45	0.0689	0.00107	64.3
	5.20	0.0684	44	0.0673	0.00107	62.7

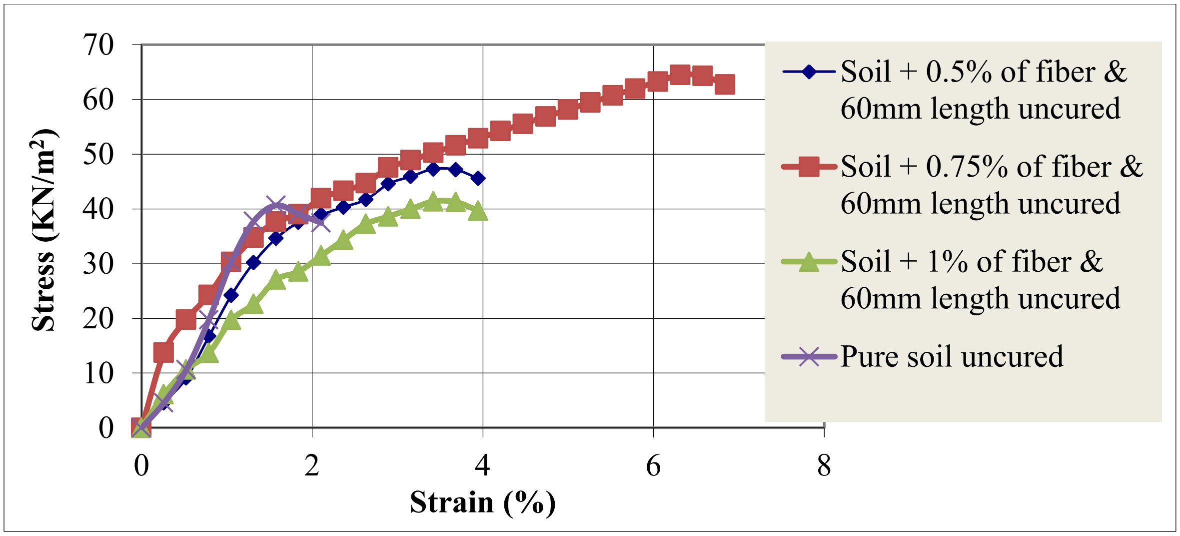

Figure D- 4: Stress-Strain relationship of soil reinforced with 60mm length uncured

Table D- 7: UCS test data for soil reinforced with 60mm length of fiber with 7days cured

Sample Name	Deformation dial in [mm] ΔH	Unit strain $\boldsymbol{\varepsilon} = \Delta H/Ho$	Proving Ring reading	Load in [KN]	Cross-sectional Area $A = A_o/1-\boldsymbol{\varepsilon}$	Stress in [KN/m^2]
Soil Reinforced with 0.5% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	5	0.0077	0.00100	7.6
	0.40	0.0053	8	0.0122	0.00101	12.2
	0.60	0.0079	11	0.0168	0.00101	16.7
	0.80	0.0105	14	0.0214	0.00101	21.2
	1.00	0.0132	17	0.0260	0.00101	25.7
	1.20	0.0158	20	0.0306	0.00102	30.1
	1.40	0.0184	24	0.0367	0.00102	36.0
	1.60	0.0211	31	0.0474	0.00102	46.4
	1.80	0.0237	36	0.0551	0.00102	53.8
	2.00	0.0263	39	0.0597	0.00103	58.1
	2.20	0.0289	42	0.0643	0.00103	62.4
	2.40	0.0316	44	0.0673	0.00103	**65.2**
	2.60	0.0342	44	0.0673	0.00104	65.0
	2.80	0.0368	43	0.0658	0.00104	63.4
	3.00	0.0395	42	0.0643	0.00104	61.7

Sample name	Deformation dial in [mm] ΔH	Unit strain ϵ = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross-sectional Area $A = A_o/1\text{-}\epsilon$	Stress in [KN/m²]
Soil Reinforced with 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	5	0.0077	0.00100	7.6
	0.40	0.0053	11	0.0168	0.00101	16.7
	0.60	0.0079	17	0.0260	0.00101	25.8
	0.80	0.0105	21	0.0321	0.00101	31.8
	1.00	0.0132	25	0.0383	0.00101	37.7
	1.20	0.0158	29	0.0444	0.00102	43.7
	1.40	0.0184	33	0.0505	0.00102	49.6
	1.60	0.0211	36	0.0551	0.00102	53.9
	1.80	0.0237	39	0.0597	0.00102	58.3
	2.00	0.0263	42	0.0643	0.00103	62.6
	2.20	0.0289	44	0.0673	0.00103	65.4
	2.40	0.0316	48	0.0734	0.00103	71.1
	2.60	0.0342	51	0.0780	0.00104	75.4
	2.80	0.0368	54	0.0826	0.00104	79.6
	3.00	0.0395	57	0.0872	0.00104	83.8
	3.20	0.0421	60	0.0918	0.00104	87.9
	3.40	0.0447	62	0.0949	0.00105	90.6
	3.60	0.0474	63	0.0964	0.00105	**91.8**
	4.00	0.0526	62	0.0949	0.00106	89.9
Soil Reinforced with 1% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	4	0.0061	0.00100	6.1
	0.40	0.0053	7	0.0107	0.00101	10.7
	0.60	0.0079	9	0.0138	0.00101	13.7
	0.80	0.0105	11	0.0168	0.00101	16.7
	1.00	0.0132	15	0.0230	0.00101	22.6
	1.20	0.0158	18	0.0275	0.00102	27.1
	1.40	0.0184	21	0.0321	0.00102	31.5
	1.60	0.0211	27	0.0413	0.00102	40.4
	1.80	0.0237	31	0.0474	0.00102	46.3
	2.00	0.0263	35	0.0536	0.00103	52.1
	2.20	0.0289	36	0.0551	0.00103	53.5
	2.40	0.0316	37	0.0566	0.00103	**54.8**
	2.80	0.0368	37	0.0566	0.00104	54.5
	3.00	0.0395	36	0.0551	0.00104	52.9

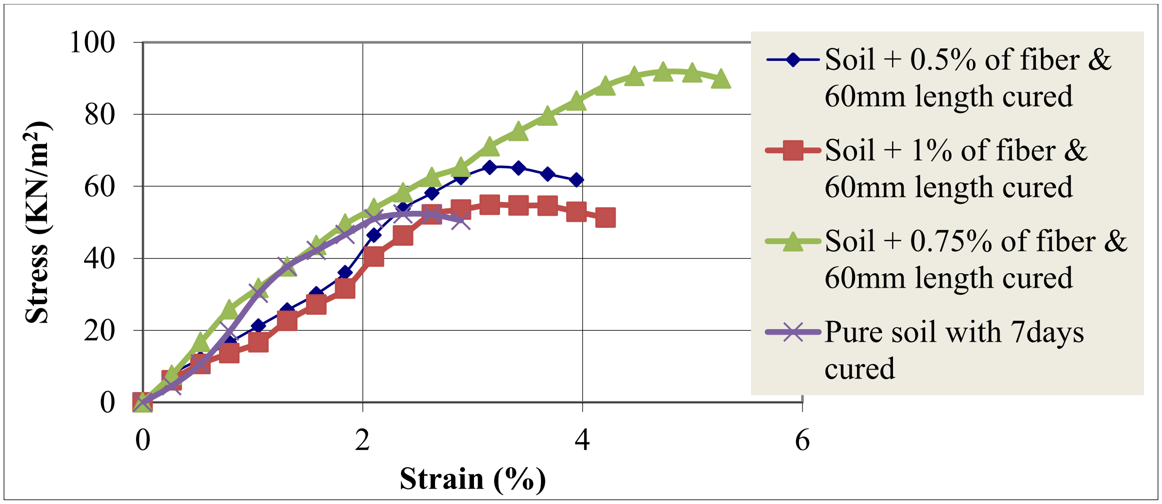

Figure D- 5: Stress-Strain relationship of soil reinforced with 60mm length with 7days cured

Table D- 8: UCS test data for soil reinforced and lime stabilized samples in case of uncured condition

Sample Name	Deformation dial in [mm] ΔH	Unit strain ε = ΔH/Ho	Proving Ring reading	Load in [KN]	Cross-sectional Area $A = A_o/1-\varepsilon$	Stress in [KN/m^2]
Soil + 6% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	8	0.0122	0.00100	12.2
	0.40	0.0053	11	0.0168	0.00101	16.7
	0.60	0.0079	14	0.0214	0.00101	21.3
	0.80	0.0105	17	0.0260	0.00101	25.7
	1.00	0.0132	21	0.0321	0.00101	31.7
	1.20	0.0158	23	0.0352	0.00102	34.6
	1.40	0.0184	25	0.0383	0.00102	37.5
	1.60	0.0211	26	0.0398	0.00102	38.9
	1.80	0.0237	28	0.0428	0.00102	41.8
	2.00	0.0263	29	0.0444	0.00103	43.2
	2.20	0.0289	30	0.0459	0.00103	44.6
	2.40	0.0316	31	0.0474	0.00103	45.9
	2.60	0.0342	33	0.0505	0.00104	48.8
	2.80	0.0368	34	0.0520	0.00104	50.1
	3.00	0.0395	35	0.0536	0.00104	51.4
	3.20	0.0421	36	0.0551	0.00104	**52.8**
	3.40	0.0447	36	0.0551	0.00105	52.6
	3.60	0.0474	35	0.0536	0.00105	51.0
	3.80	0.0500	34	0.0520	0.00105	49.4

Sample name	Deformation dial in [mm] ΔH	Unit strain ϵ = ΔH/ΔHo	Proving Ring reading	Load in [KN]	Cross-sectional Area A = A_o/1-ϵ	stress in [KN/m^2]
Soil + 2% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	9	0.0138	0.00100	13.7
	0.40	0.0053	11	0.0168	0.00101	16.7
	0.60	0.0079	13	0.0199	0.00101	19.7
	0.80	0.0105	15	0.0230	0.00101	22.7
	1.00	0.0132	17	0.0260	0.00101	25.7
	1.20	0.0158	19	0.0291	0.00102	28.6
	1.40	0.0184	20	0.0306	0.00102	30.0
	1.60	0.0211	21	0.0321	0.00102	31.5
	1.80	0.0237	23	0.0352	0.00102	34.4
	2.00	0.0263	25	0.0383	0.00103	37.2
	2.20	0.0289	28	0.0428	0.00103	41.6
	2.40	0.0316	30	0.0459	0.00103	44.5
	2.60	0.0342	32	0.0490	0.00104	47.3
	3.00	0.0395	36	0.0551	0.00104	52.9
	3.20	0.0421	38	0.0581	0.00104	55.7
	3.40	0.0447	39	0.0597	0.00105	57.0
	3.60	0.0474	40	0.0612	0.00105	58.3
	3.80	0.0500	41	0.0627	0.00105	59.6
	4.00	0.0526	42	0.0643	0.00106	60.9
	4.20	0.0553	43	0.0658	0.00106	**62.2**
	4.60	0.0605	42	0.0643	0.00106	60.4
Soil + 4% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	13	0.0199	0.00100	19.8
	0.40	0.0053	18	0.0275	0.00101	27.4
	0.60	0.0079	23	0.0352	0.00101	34.9
	0.80	0.0105	28	0.0428	0.00101	42.4
	1.00	0.0132	32	0.0490	0.00101	48.3
	1.20	0.0158	38	0.0581	0.00102	57.2
	1.40	0.0184	41	0.0627	0.00102	61.6
	1.60	0.0211	45	0.0689	0.00102	67.4
	1.80	0.0237	50	0.0765	0.00102	74.7
	2.00	0.0263	55	0.0842	0.00103	81.9
	2.20	0.0289	57	0.0872	0.00103	84.7
	2.40	0.0316	59	0.0903	0.00103	87.4
	2.60	0.0342	61	0.0933	0.00104	90.1
	2.80	0.0368	62	0.0949	0.00104	91.4
	3.00	0.0395	63	0.0964	0.00104	92.6
	3.20	0.0421	65	0.0995	0.00104	**95.3**
	3.60	0.0474	64	0.0979	0.00105	93.3

Table D- 9: UCS test data for soil reinforced and lime stabilized samples in case of 7days cured

Sample Name	Deformation dial in [mm] ΔH	Unit strain ε = ΔH/Ho	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\varepsilon$	Stress in $[KN/m^2]$
Soil + 6% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	11	0.0168	0.00101	16.7
	0.60	0.0079	21	0.0321	0.00101	31.9
	0.80	0.0105	27	0.0413	0.00101	40.9
	1.00	0.0132	33	0.0505	0.00101	49.8
	1.20	0.0158	37	0.0566	0.00102	55.7
	1.40	0.0184	39	0.0597	0.00102	58.6
	1.60	0.0211	41	0.0627	0.00102	61.4
	1.80	0.0237	42	0.0643	0.00102	62.7
	2.00	0.0263	43	0.0658	0.00103	64.1
	2.20	0.0289	45	0.0689	0.00103	66.9
	2.40	0.0316	46	0.0704	0.00103	68.2
	2.60	0.0342	47	0.0719	0.00104	69.4
	2.80	0.0368	48	0.0734	0.00104	70.7
	3.00	0.0395	49	0.0750	0.00104	72.0
	3.20	0.0421	50	0.0765	0.00104	73.3
	3.40	0.0447	51	0.0780	0.00105	74.5
	3.60	0.0474	52	0.0796	0.00105	**75.8**
	3.80	0.0500	51	0.0780	0.00105	74.1

Sample Name	Deformation dial in [mm] ΔH	Unit Strain $\epsilon = \Delta H/\Delta Ho$	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\epsilon$	Stress in [KN/m^2]
Soil + 2% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	16	0.0245	0.00100	24.4
	0.40	0.0053	23	0.0352	0.00101	35.0
	0.60	0.0079	28	0.0428	0.00101	42.5
	0.80	0.0105	34	0.0520	0.00101	51.5
	1.00	0.0132	39	0.0597	0.00101	58.9
	1.20	0.0158	43	0.0658	0.00102	64.8
	1.40	0.0184	46	0.0704	0.00102	69.1
	1.60	0.0211	48	0.0734	0.00102	71.9
	1.80	0.0237	51	0.0780	0.00102	76.2
	2.00	0.0263	54	0.0826	0.00103	80.4
	2.20	0.0289	56	0.0857	0.00103	83.2
	2.40	0.0316	57	0.0872	0.00103	84.5
	2.60	0.0342	58	0.0887	0.00104	85.7
	2.80	0.0368	59	0.0903	0.00104	86.9
	3.00	0.0395	60	0.0918	0.00104	88.2
	3.20	0.0421	61.5	0.0941	0.00104	90.1
	3.40	0.0447	62	0.0949	0.00105	90.6
	3.60	0.0474	63	0.0964	0.00105	91.8
	3.80	0.0500	63.5	0.0972	0.00105	92.3
	4.00	0.0526	64	0.0979	0.00106	**92.8**
	4.20	0.0553	64	0.0979	0.00106	92.5
	4.40	0.0579	63	0.0964	0.00106	90.8
	4.60	0.0605	62	0.0949	0.00106	89.1

Sample Name	Deformation dial in [mm] ΔH	Unit Strain ϵ = ΔH/ΔHo	Proving Ring Reading	Load in [KN]	Cross - sectional Area A = A_o/1-ϵ	Stress in [KN/M^2]
Soil + 4% of lime + 0.75% of fiber and 60mm length	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	13	0.0199	0.00100	19.8
	0.40	0.0053	21	0.0321	0.00101	32.0
	0.60	0.0079	28	0.0428	0.00101	42.5
	0.80	0.0105	33	0.0505	0.00101	50.0
	1.00	0.0132	39	0.0597	0.00101	58.9
	1.20	0.0158	45	0.0689	0.00102	67.8
	1.40	0.0184	49	0.0750	0.00102	73.6
	1.60	0.0211	52	0.0796	0.00102	77.9
	1.80	0.0237	55	0.0842	0.00102	82.2
	2.00	0.0263	58	0.0887	0.00103	86.4
	2.20	0.0289	61	0.0933	0.00103	90.6
	2.40	0.0316	62	0.0949	0.00103	91.9
	2.60	0.0342	64	0.0979	0.00104	94.6
	2.80	0.0368	65.5	0.1002	0.00104	96.5
	3.00	0.0395	67	0.1025	0.00104	98.5
	3.20	0.0421	68.5	0.1048	0.00104	100.4
	3.40	0.0447	69.5	0.1063	0.00105	101.6
	3.60	0.0474	70	0.1071	0.00105	102.0
	3.80	0.0500	71	0.1086	0.00105	103.2
	4.00	0.0526	72	0.1102	0.00106	104.4
	4.20	0.0553	73.5	0.1125	0.00106	106.2
	4.40	0.0579	74	0.1132	0.00106	106.7
	4.60	0.0605	76	0.1163	0.00106	109.2
	4.80	0.0632	77	0.1178	0.00107	110.4
	5.00	0.0658	78	0.1193	0.00107	**111.5**
	5.20	0.0684	77	0.1178	0.00107	109.7

Table D- 10: UCS test data for soil stabilized with optimum percentage of lime uncured and 7days cured conditions

Sample name	Deformation dial in [mm] ΔH	Unit strain $\epsilon = \Delta H/\Delta H_o$	Proving Ring reading	Load in [KN]	Cross - sectional Area $A = A_o/1-\epsilon$	Stress in [KN/m^2]
Uncured condition Soil + 4% of lime	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	3	0.0046	0.00100	4.6
	0.40	0.0053	5	0.0077	0.00101	7.6
	0.60	0.0079	9	0.0138	0.00101	13.7
	0.80	0.0105	12	0.0184	0.00101	18.2
	1.00	0.0132	16	0.0245	0.00101	24.2
	1.20	0.0158	20	0.0306	0.00102	30.1
	1.40	0.0184	22	0.0337	0.00102	33.0
	1.60	0.0211	25	0.0383	0.00102	37.4
	1.80	0.0237	26	0.0398	0.00102	38.8
	2.20	0.0289	28	0.0428	0.00103	41.6
	2.40	0.0316	29	0.0444	0.00103	43.0
	2.80	0.0368	30	0.0459	0.00104	44.2
	3.20	0.0421	31	0.0474	0.00104	**45.4**
	3.60	0.0474	30	0.0459	0.00105	43.7
	3.80	0.0500	29	0.0444	0.00105	42.2
Cured condition Soil + 4% of lime	0.00	0.0000	0.0	0.0000	0.00100	0.0
	0.20	0.0026	8	0.0122	0.00100	12.2
	0.40	0.0053	15	0.0230	0.00101	22.8
	0.60	0.0079	22	0.0337	0.00101	33.4
	0.80	0.0105	27	0.0413	0.00101	40.9
	1.00	0.0132	29	0.0444	0.00101	43.8
	1.20	0.0158	31	0.0474	0.00102	46.7
	1.40	0.0184	33	0.0505	0.00102	49.6
	1.60	0.0211	35	0.0536	0.00102	52.4
	1.80	0.0237	36	0.0551	0.00102	53.8
	2.00	0.0263	37	0.0566	0.00103	55.1
	2.20	0.0289	38	0.0581	0.00103	56.5
	2.40	0.0316	39	0.0597	0.00103	57.8
	2.60	0.0342	40	0.0612	0.00104	**59.1**
	2.80	0.0368	40	0.0612	0.00104	58.9
	3.00	0.0395	39	0.0597	0.00104	57.3
	3.20	0.0421	38	0.0581	0.00104	55.7

Appendix E: One-Dimensional Consolidation Test Result

Table E- 1: Specimen data before test and after test for pure soil

Before Test			
Weight of sample & Ring (gm)	261.50	Specific Gravity (G_s)	2.78
Weight of Ring (gm)	111.50	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	150.00	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	98.60	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	51.40	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	52.13	Bulk Density (r_b) (gm/cm^3)	1.697
		Dry Density (r_d) (gm/cm^3)	1.116
Initial Void Ratio : $e_o = (G_s/r_d)-1$		1.492	
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)		8.027	
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)		97.16	

After Test			
Weight of Sample + Ring +Can (gm)	301.40	Overall settlement (mm)	3.391
Weight of Dry Sample + Ring +Can (gm)	259.20	Volume change (cm^3)	14.98
Weight of Ring + Can (gm)	160.60	Final Volume (cm^3)	73.39
Weight of Wet Sample (gm)	140.80	Final Bulk Density (gm/cm^3)	1.919
Weight of Dry Sample (gm)	98.60	Final Dry Density (gm/cm^3)	1.344
Weight of Final Moisture (gm)	42.20	Final Void Ratio:(e_f)	1.069
Final Moisture Content (M_f) (%)	42.80	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	111.3

Table E- 2: One-Dimensional consolidation test result for pure soil

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.027mm			Initial Void Ratio, e_o			1.492		
Voids Ratio					Compressibility			Coefficient of consolidation		
Increment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)*(1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1+H_2)$	$Cv = (0.428*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.492				-		-
					0.590	160	0.184			
1	160	0.590	19.410	1.418				144	19.7	2.29
					0.966	160	0.311			
2	320	1.556	18.444	1.298				256	18.93	1.19
					1.154	320	0.196			
3	640	2.710	17.290	1.154				441	17.87	0.61
					0.944	640	0.085			
4	1280	3.654	16.346	1.036				484	16.82	0.5
					0.869	1280	0.042			
5	2560	4.523	15.477	0.928				676	15.91	0.32
					0.483	1920	0.016			
6	640	4.040	15.960	0.988						
					0.649	480	0.085			
7	160	3.391	16.609	1.069						

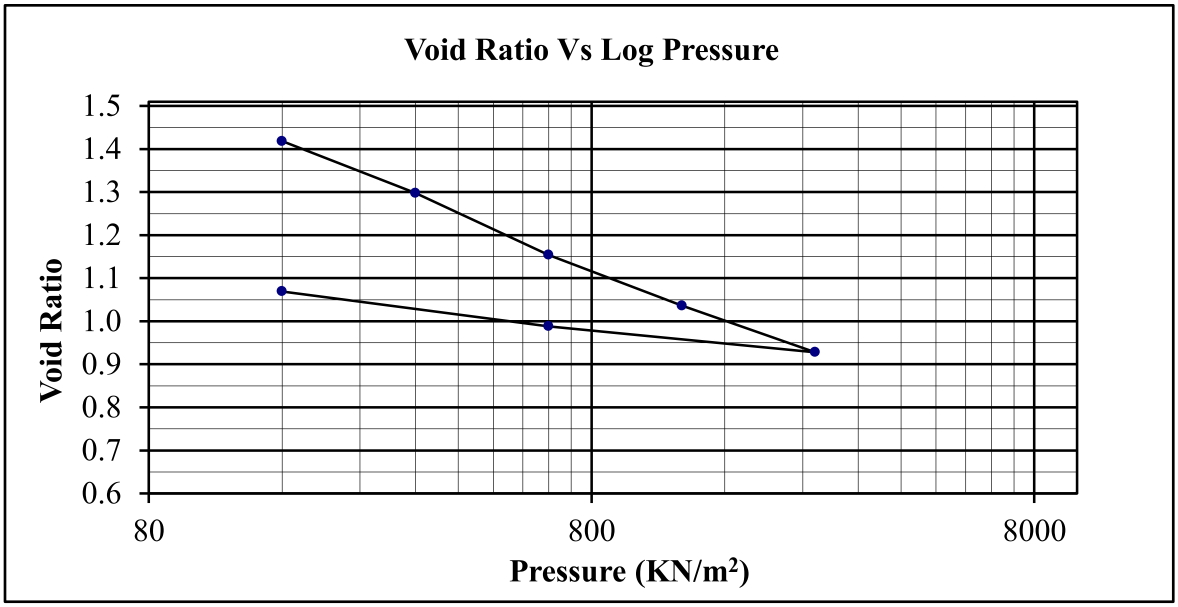

Figure E- 1: Void ratio versus log pressure for pure soil

a) Compression index of pure soil:

$P_1 = 160$ Kpa $\quad\quad e_1 = 1.418$

$P_2 = 2560$ Kpa $\quad\quad e_2 = 0.928$

$$C_c = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.407$$

b) Re-compression index of pure soil:

$P_1 = 160$ Kpa $\quad\quad e_1 = 1.069$

$P_2 = 2560$ Kpa $\quad\quad e_2 = 0.928$

$$C_r = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.117$$

Table E- 3: Specimen data before test and after test for soil reinforced with 0.5% of fiber

Before Test			
Weight of sample & Ring (gm)	233.90	Specific Gravity (G_s)	2.64
Weight of Ring (gm)	108.90	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	125.00	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	93.90	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	31.10	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	33.12	Bulk Density (r_b) (gm/cm^3)	1.415
		Dry Density (r_d) (gm/cm^3)	1.063
Initial Void Ratio : $e_o = (G_s/r_d)-1$		1.484	
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)		8.050	
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)		58.90	

After Test			
Weight of Sample + Ring +Can (gm)	236.50	Overall settlement (mm)	4.797
Weight of Dry Sample + Ring +Can (gm)	202.80	Volume change (cm^3)	21.20
Weight of Ring + Can (gm)	108.90	Final Volume (cm^3)	67.17
Weight of Wet Sample (gm)	127.60	Final Bulk Density (gm/cm^3)	1.900
Weight of Dry Sample (gm)	93.90	Final Dry Density (gm/cm^3)	1.398
Weight of Final Moisture (gm)	33.70	Final Void Ratio:(e_f)	0.889
Final Moisture Content (M_f) (%)	35.89	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	106.63

Table E- 4: One-Dimensional consolidation test for soil reinforced with 0.5% of fiber

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.050mm			Initial Void Ratio, e_o			1.484		
Voids Ratio					Compressibility			Coefficient of consolidation		
Incre ment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)^* (1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1 + H_2)$	$Cv = (0.428^*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.484				-		-
					1.878	160	0.587			
1	160	1.878	18.122	1.251				123 .21	19.06	2.5
					1.126	160	0.388			
2	320	3.004	16.996	1.111				196	17.56	1.33
					1.128	320	0.207			
3	640	4.132	15.868	0.971				324	16.43	0.71
					0.850	640	0.084			
4	1280	4.982	15.018	0.866				353 .44	15.44	0.57
					0.968	1280	0.050			
5	2560	5.950	14.050	0.745				400	14.53	0.45
					0.586	1920	0.022			
6	640	5.654	14.636	0.818						
					0.567	480	0.081			
7	160	4.751	15.203	0.889						

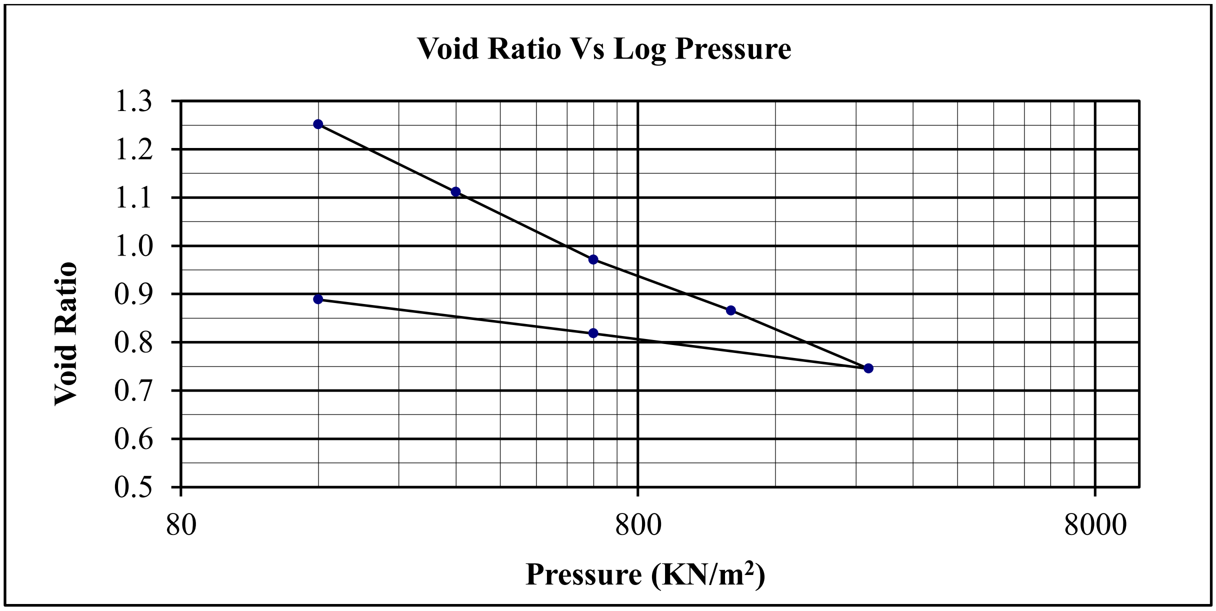

Figure E- 2: Void ratio versus log pressure for soil reinforced with 0.5% of fiber

1. Compression index of soil reinforced with 0.5% of fiber:

P_1 = 160 Kpa e_1 = 1.251

P_2 = 2560 Kpa e_2 = 0.745

$$C_c = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.42$$

2. Re-compression index of soil reinforced with 0.5% of fiber:

P_1 = 160 Kpa e_1 = 0.889

P_2 = 2560 Kpa e_2 = 0.745

$$C_r = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.119$$

Table E- 5: Specimen data before test and after test for soil reinforced with 0.75% of fiber

Before Test			
Weight of sample & Ring (gm)	239.80	Specific Gravity (G_s)	2.63
Weight of Ring (gm)	111.50	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	128.30	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	93.20	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	35.10	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	37.66	Bulk Density (r_b) (gm/cm^3)	1.452
		Dry Density (r_d) (gm/cm^3)	1.067
Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.465		
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	8.115		
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)	64.75		

After Test			
Weight of Sample + Ring +Can (gm)	276.55	Overall settlement (mm)	3.865
Weight of Dry Sample + Ring +Can (gm)	254.90	Volume change (cm^3)	17.08
Weight of Ring + Can (gm)	160.60	Final Volume (cm^3)	71.29
Weight of Wet Sample (gm)	115.95	Final Bulk Density (gm/cm^3)	1.626
Weight of Dry Sample (gm)	94.30	Final Dry Density (gm/cm^3)	1.323
Weight of Final Moisture (gm)	21.65	Final Void Ratio:(e_f)	0.988
Final Moisture Content (M_f) (%)	22.96	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	61.1

Table E- 6: One-Dimensional consolidation test for soil reinforced with 0.75% of fiber

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.115mm			Initial Void Ratio, e_o			1.465		
Voids Ratio					Compressibility			Coefficient of consolidation		
Incre ment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)*(1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1 + H_2)$	$Cv = (0.428*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.465				-		-
					0.450	160	0.141			
1	160	0.450	19.550	1.409				100	19.775	3.32
					1.236	160	0.395			
2	320	1.686	18.314	1.257				196	18.932	1.55
					1.100	320	0.171			
3	640	2.786	17.214	1.121				256	17.814	1.05
					1.992	640	0.090			
4	1280	3.778	16.222	0.999				324	16.768	0.74
					1.210	1280	0.058			
5	2560	4.988	15.012	0.850				361	15.617	0.57
					0.537	1920	0.019			
6	640	4.451	15.549	0.916						
					0.586	640	0.059			
7	160	3.865	16.135	0.988						

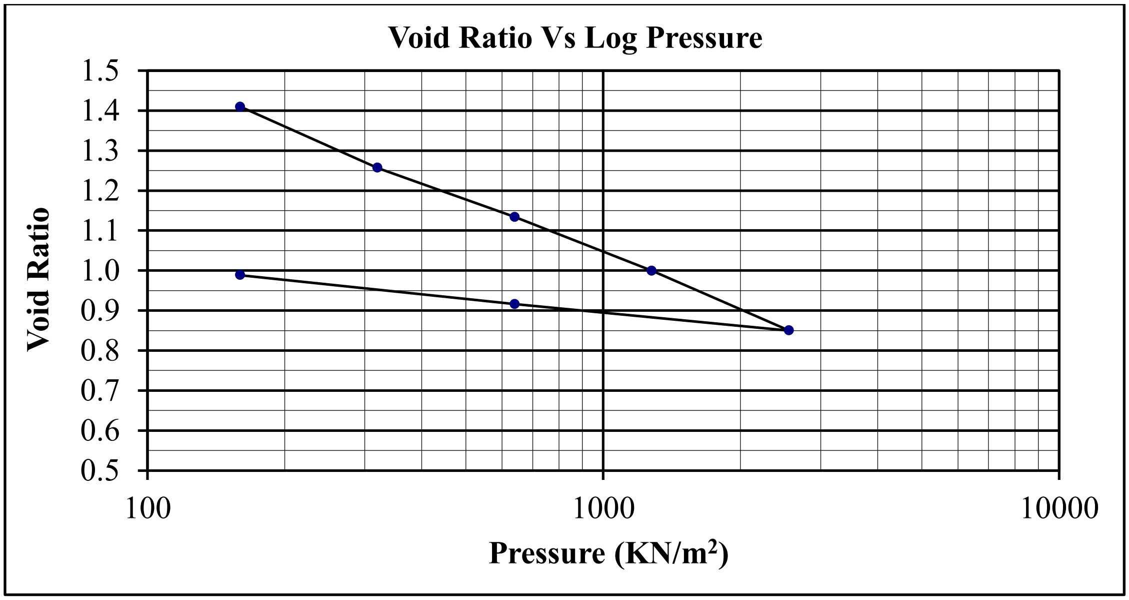

Figure E- 3: Void ratio versus log pressure for soil reinforced with 0.75% of fiber

1) Compression index of soil reinforced with 0.75% of fiber:

P_1 = 160 Kpa e_1 = 1.409

P_2 = 2560 Kpa e_2 = 0.85

$$C_c = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.424$$

2) Re-compression index of soil reinforced with 0.75% of fiber:

P_1 = 160 Kpa e_1 = 0.988

P_2 = 2560 Kpa e_2 = 0.85

$$C_r = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.115$$

Table E- 7: Specimen data before test and after test for soil reinforced with 1% of fiber

Before Test			
Weight of sample & Ring (gm)	239.90	Specific Gravity (G_s)	2.62
Weight of Ring (gm)	109.00	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	130.90	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	95.70	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	35.20	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	36.78	Bulk Density (r_b) (gm/cm^3)	1.481
		Dry Density (r_d) (gm/cm^3)	1.083
Initial Void Ratio : $e_o = (G_s/r_d)-1$		1.419	
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)		8.267	
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)		67.9	

After Test			
Weight of Sample + Ring +Can (gm)	233.40	Overall settlement (mm)	4.650
Weight of Dry Sample + Ring +Can (gm)	204.70	Volume change (cm^3)	20.55
Weight of Ring + Can (gm)	109.00	Final Volume (cm^3)	67.82
Weight of Wet Sample (gm)	124.40	Final Bulk Density (gm/cm^3)	1.834
Weight of Dry Sample (gm)	95.70	Final Dry Density (gm/cm^3)	1.411
Weight of Final Moisture (gm)	28.70	Final Void Ratio:(e_f)	0.857
Final Moisture Content (M_f) (%)	29.99	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	91.7

Table E- 8: One-Dimensional consolidation test result for soil reinforced with 1% of fiber

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.267mm			Initial Void Ratio, e_o			1.419		
Voids Ratio					Compressibility			Coefficient of consolidation		
Incre ment N°	Pressure P (Kpa)	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change (δH) mm	Incremental Pressure change (δp) Kpa	$m_v = (\delta H/H_1)*(1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1 + H_2)$	$Cv = (0.428*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.419				-		-
					1.408	160	0.440			
1	160	1.408	18.592	1.249				169	19.296	1.87
					1.158	160	0.389			
2	320	2.566	17.434	1.109				400	18.013	0.69
					1.068	320	0.191			
3	640	3.634	16.366	0.980				462	16.900	0.52
					1.066	640	0.102			
4	1280	4.700	15.300	0.851				484	15.833	0.44
					0.916	1280	0.047			
5	2560	5.616	14.384	0.740				529	14.842	0.35
					0.548	1920	0.020			
6	640	5.068	14.932	0.806						
					0.418	480	0.058			
7	160	4.650	15.350	0.857						

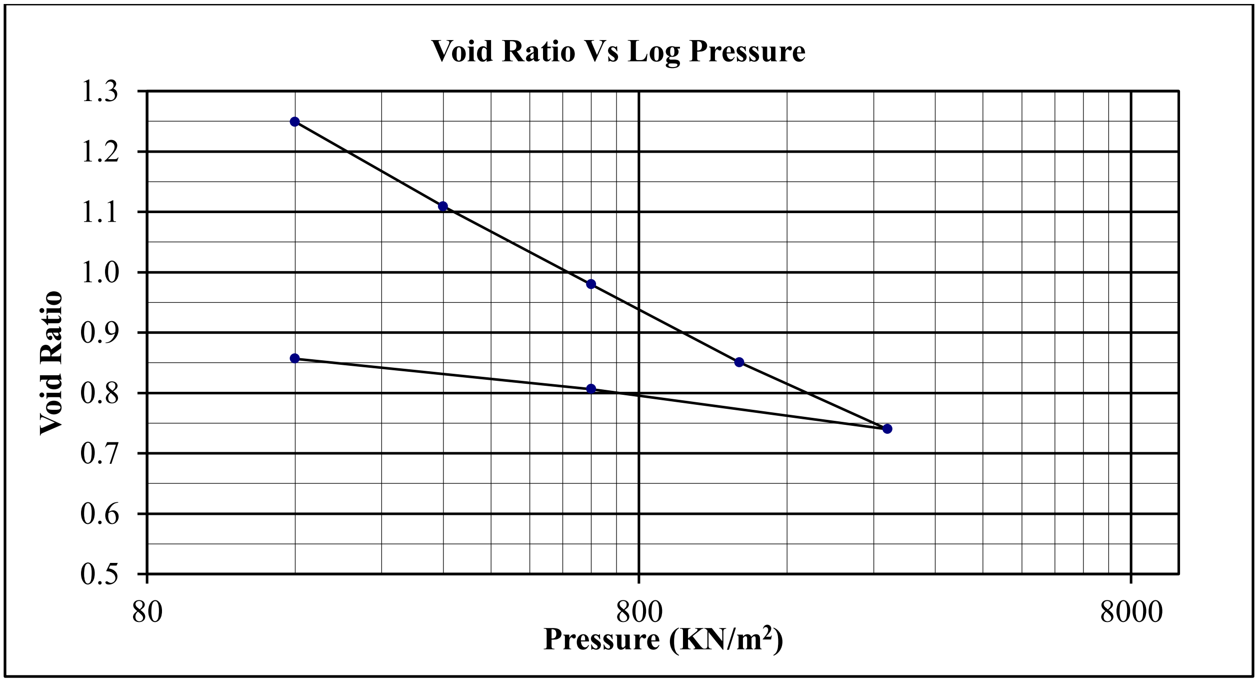

Figure E- 4: Void ratio versus log pressure for soil reinforced with 1% of fiber

1. Compression index of soil reinforced with 1% of fiber:

P_1 = 160 Kpa $e_1 = 1.249$

P_2 = 2560 Kpa $e_2 = 0.740$

$$C_c = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.423$$

2. Re-compression index of soil reinforced with 1% of fiber:

P_1 = 160 Kpa $e_1 = 0.857$

P_2 = 2560 Kpa $e_2 = 0.740$

$$C_r = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.097$$

Table E- 9: Specimen data before test and after test for soil stabilized with 4% of lime

Before Test			
Weight of sample & Ring (gm)	261.60	Specific Gravity (G_s)	2.64
Weight of Ring (gm)	111.50	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	150.10	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	95.90	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	54.20	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	56.52	Bulk Density (r_b) (gm/cm^3)	1.699
		Dry Density (r_d) (gm/cm^3)	1.085

Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.433
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	8.221
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)	104.14

After Test			
Weight of Sample + Ring +Can (gm)	288.50	Overall settlement (mm)	3.498
Weight of Dry Sample + Ring +Can (gm)	256.50	Volume change (cm^3)	15.46
Weight of Ring + Can (gm)	160.60	Final Volume (cm^3)	72.91
Weight of Wet Sample (gm)	127.90	Final Bulk Density (gm/cm^3)	1.754
Weight of Dry Sample (gm)	95.90	Final Dry Density (gm/cm^3)	1.315
Weight of Final Moisture (gm)	32.00	Final Void Ratio:(e_f)	1.007
Final Moisture Content (M_f) (%)	33.37	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	87.46

Table E- 10: One-Dimensional consolidation test for soil stabilized with 4% of lime

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.221mm			Initial Void Ratio, e_o			1.433		
Voids Ratio					Compressibility			Coefficient of consolidation		
Increment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e = H - H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)^* (1000/\delta p)$ m^2/MN	t_{90} min	$H = 1/2(H_1 + H_2)$	$Cv = (0.428^* H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.433				-		-
					0.752	160	0.235			
1	160	0.752	19.248	1.341				121	19.624	2.70
					0.878	160	0.285			
2	320	1.630	18.370	1.234				196	18.809	1.53
					1.133	320	0.193			
3	640	2.763	17.237	1.097				324	17.804	0.83
					0.969	640	0.088			
4	1280	3.732	16.268	0.979				361	16.753	0.66
					0.880	1280	0.042			
5	2560	4.612	15.388	0.872				400	15.828	0.53
					0.558	1920	0.019			
6	640	4.054	15.946	0.940						
					0.556	480	0.073			
7	160	3.498	16.502	1.007						

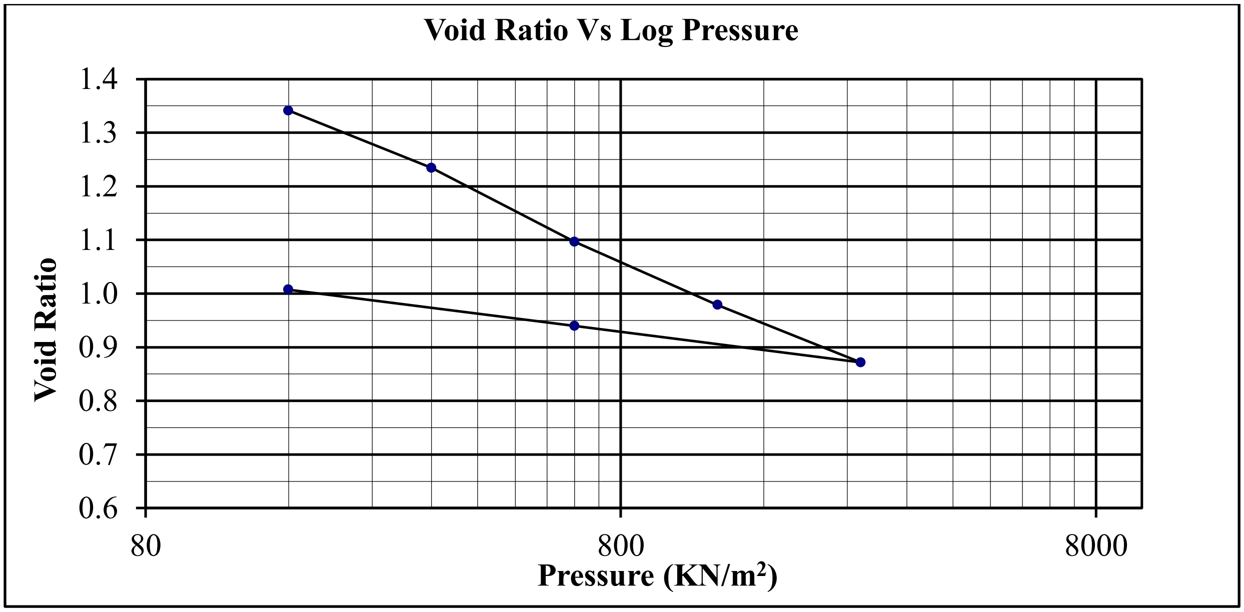

Figure E- 5: Void ratio versus log pressure for soil stabilized with 4% of lime

1. Compression index of soil stabilized with 4% of lime:

P_1 = 160 Kpa $\quad$ $e_1 = 1.341$

P_2 = 2560 Kpa $\quad$ $e_2 = 0.872$

$$C_c = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.39$$

2. Re-compression index of soil stabilized with 4% of lime:

P_1 = 160 Kpa $\quad$ $e_1 = 1.007$

P_2 = 2560 Kpa $\quad$ $e_2 = 0.872$

$$C_r = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.112$$

Table E- 11: Specimen data before test and after test for soil reinforced with 0.5% of fiber and lime stabilized

Before Test			
Weight of sample & Ring (gm)	254.80	Specific Gravity (G_s)	2.63
Weight of Ring (gm)	115.00	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	139.80	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	101.70	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	38.10	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	37.46	Bulk Density (r_b) (gm/cm^3)	1.582
		Dry Density (r_d) (gm/cm^3)	1.151

Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.285
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	8.752
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)	76.66

After Test			
Weight of Sample + Ring +Can (gm)	254.40	Overall settlement (mm)	4.361
Weight of Dry Sample + Ring +Can (gm)	216.70	Volume change (cm^3)	19.27
Weight of Ring + Can (gm)	115.00	Final Volume (cm^3)	69.10
Weight of Wet Sample (gm)	139.40	Final Bulk Density (gm/cm^3)	2.017
Weight of Dry Sample (gm)	101.70	Final Dry Density (gm/cm^3)	1.472
Weight of Final Moisture (gm)	37.70	Final Void Ratio:(e_f)	0.787
Final Moisture Content (M_f) (%)	37.07	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	123.89

Table E- 12: One-Dimensional Consolidation Test for soil reinforced with 0.5% of fiber and lime stabilized

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.752mm			Initial Void Ratio, e_o			1.285		
Voids Ratio					Compressibility			Coefficient of consolidation		
Increment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height H = H_O-ΔH mm	Voids Ratio e=H-H_S/H_S	Incremental Height change δH mm	Incremental Pressure change δp Kpa	m_v =(δH/H_1)*(1000/ δp) m^2/MN	t_{90} min	H=1/2(H_1 + H_2)	Cv = (0.428*H^2)/t_{90} (mm^2/min)
0	0	0.000	20.000	1.285				-		-
					1.336	160	0.418			
1	160	1.336	18.664	1.133				100	19.332	3.17
					0.816	160	0.273			
2	320	2.152	17.848	1.039				169	18.256	1.67
					1.098	320	0.192			
3	640	3.250	16.750	0.914				225	17.299	1.13
					0.922	640	0.086			
4	1280	4.172	15.828	0.809				278.89	16.289	0.81
					1.074	1280	0.053			
5	2560	5.246	14.754	0.686				289	15.291	0.69
					0.234	1920	0.008			
6	640	5.012	14.988	0.713						
					0.651	480	0.090			
7	160	4.361	15.639	0.787						

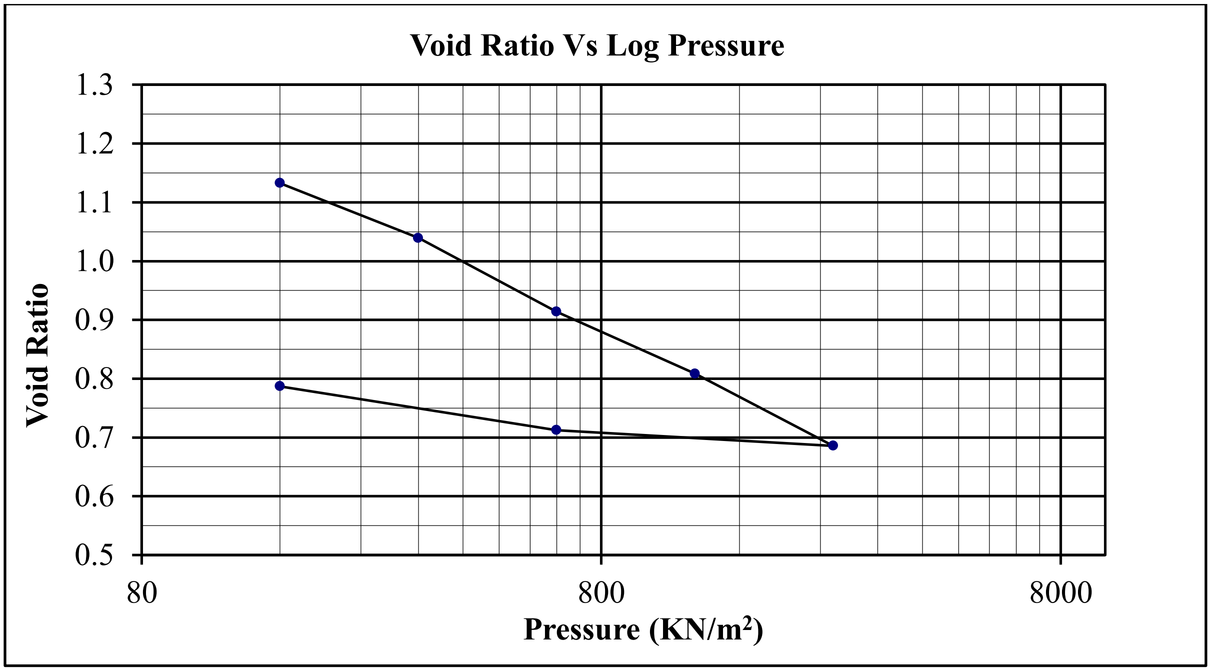

Figure E- 6: Void ratio versus log pressure for soil reinforced with 0.5% of fiber and lime stabilized

1. Compression index of soil reinforced with 0.5% of fiber and lime stabilized:

P_1 = 160 Kpa e_1 = 1.133

P_2 = 2560 Kpa e_2 = 0.686

$$C_c = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.371$$

2. Re-compression index of soil reinforced with 0.5% of fiber and lime stabilized:

P_1 = 160 Kpa e_1 = 0.787

P_2 = 2560 Kpa e_2 = 0.686

$$C_r = \frac{e1 - e2}{\log(\frac{p2}{P1})} = 0.084$$

Table E- 13: Specimen data before test and after test for soil reinforced with 0.75% of fiber and lime stabilized

Before Test			
Weight of sample & Ring (gm)	260.10	Specific Gravity (G_s)	2.62
Weight of Ring (gm)	111.50	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	148.60	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	102.40	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	46.20	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	45.12	Bulk Density (r_b) (gm/cm^3)	1.682
		Dry Density (r_d) (gm/cm^3)	1.159

Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.261
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	8.846
Initial Saturation: $S_o = (M_o*G_s)/e_o$ (%)	93.74

After Test			
Weight of Sample + Ring +Can (gm)	298.70	Overall settlement (mm)	2.656
Weight of Dry Sample + Ring +Can (gm)	263.00	Volume change (cm^3)	11.74
Weight of Ring + Can (gm)	160.60	Final Volume (cm^3)	76.63
Weight of Wet Sample (gm)	138.10	Final Bulk Density (gm/cm^3)	1.802
Weight of Dry Sample (gm)	102.40	Final Dry Density (gm/cm^3)	1.336
Weight of Final Moisture (gm)	35.70	Final Void Ratio:(e_f)	0.961
Final Moisture Content (M_f) (%)	34.86	Final Saturation: $S_f = (M_f*G_s)/e_f$ (%)	95.07

Table E- 14: One-Dimensional Consolidation Test for soil reinforced with 075% of fiber and lime stabilized

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.846mm			Initial Void Ratio, e_o			1.261		
Voids Ratio					Compressibility			Coefficient of consolidation		
Increment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)*(1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1 + H_2)$	$Cv = (0.428*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.261				-		-
					0.374	160	0.117			
1	160	0.374	19.626	1.219				64	19.813	5.20
					0.718	160	0.229			
2	320	1.092	18.908	1.138				100	19.267	3.15
					0.788	320	0.130			
3	640	1.880	18.120	1.048				144	18.514	2.02
					0.768	640	0.066			
4	1280	2.648	17.352	0.962				196	17.736	1.36
					0.876	1280	0.039			
5	2560	3.524	16.476	0.863				256	16.914	0.95
					0.440	1920	0.014			
6	640	3.084	16.916	0.912						
					0.428	480	0.053			
7	160	2.656	17.344	0.961						

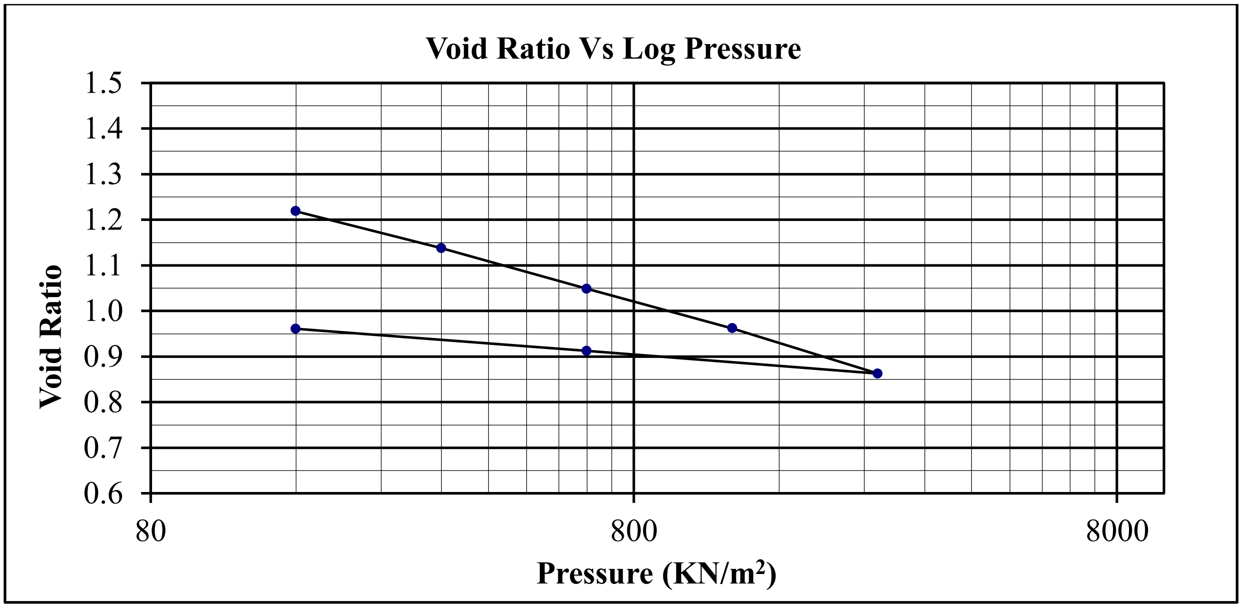

Figure E- 7: Void ratio versus log pressure for soil reinforced with 0.75% of fiber and lime stabilized

1. Compression index of soil reinforced with 0.75% of fiber and lime stabilized:

P_1 = 160 Kpa e_1 = 1.219

P_2 = 2560 Kpa e_2 = 0.863

$$C_c = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.296$$

2. Re-compression index of soil reinforced with 0.75% of fiber and lime stabilized:

P_1 = 160 Kpa e_1 = 0.889

P_2 = 2560 Kpa e_2 = 0.863

$$C_r = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.081$$

Table E- 15: Specimen data before test and after test for soil reinforced with 1% of fiber and lime stabilized

Before Test			
Weight of sample & Ring (gm)	245.80	Specific Gravity (G_s)	2.61
Weight of Ring (gm)	109.00	Sample Diameter (D) (mm)	75.00
Weight of sample (gm)	136.80	Sample Thickness (H_o) (mm)	20.00
Weight of Dry Sample (gm)	102.50	Area (A) (cm^2)	44.18
Weight of Initial Moisture (gm)	34.30	Volume (V) (cm^3)	88.37
Initial Moisture content (M_o) (%)	33.46	Bulk Density (r_b) (gm/cm^3)	1.548
		Dry Density (r_d) (gm/cm^3)	1.16

Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.25
Height of Solids : $H_s = H_o/(1+e_o)$ (mm)	8.888
Initial Saturation: $S_o = (M_o{*}G_s)/e_o$ (%)	69.86

After Test			
Weight of Sample + Ring +Can (gm)	240.70	Overall settlement (mm)	4.322
Weight of Dry Sample + Ring +Can (gm)	211.50	Volume change (cm^3)	19.10
Weight of Ring + Can (gm)	109.00	Final Volume (cm^3)	69.27
Weight of Wet Sample (gm)	131.70	Final Bulk Density (gm/cm^3)	1.901
Weight of Dry Sample (gm)	102.50	Final Dry Density (gm/cm^3)	1.480
Weight of Final Moisture (gm)	29.20	Final Void Ratio:(e_f)	0.764
Final Moisture Content (M_f) (%)	28.49	Final Saturation: $S_f = (M_f{*}G_s)/e_f$ (%)	97.33

Table E- 16: One-Dimensional Consolidation Test for soil reinforcedwith 1% of fiber and lime stabilized

Sample Diameter (D)		75.0mm			Initial Height, H_o			20.0mm		
Height of Solid (H_S)		8.888mm			Initial Void Ratio, e_o			1.250		
Voids Ratio					Compressibility			Coefficient of consolidation		
Increment N^o	Pressure p Kpa	Cumulative compression (ΔH) mm	Consolidated height $H = H_O - \Delta H$ mm	Voids Ratio $e=H-H_S/H_S$	Incremental Height change δH mm	Incremental Pressure change δp Kpa	$m_v = (\delta H/H_1)*(1000/\delta p)$ m^2/MN	t_{90} min	$H=1/2(H_1 + H_2)$	$Cv = (0.428*H^2)/t_{90}$ (mm^2/min)
0	0	0.000	20.000	1.250				-		-
					1.540	160	0.481			
1	160	1.540	18.460	1.077				132.25	19.230	2.37
					0.916	160	0.310			
2	320	2.456	17.544	0.974				225	18.002	1.22
					1.228	320	0.219			
3	640	3.684	16.316	0.836				324	16.930	0.75
					0.836	640	0.080			
4	1280	4.520	15.480	0.742				361	15.898	0.59
					0.746	1280	0.038			
5	2560	5.266	14.734	0.658				400	15.107	0.48
					0.550	1920	0.019			
6	640	4.716	15.284	0.720						
					0.394	480	0.054			
7	160	4.322	15.678	0.764						

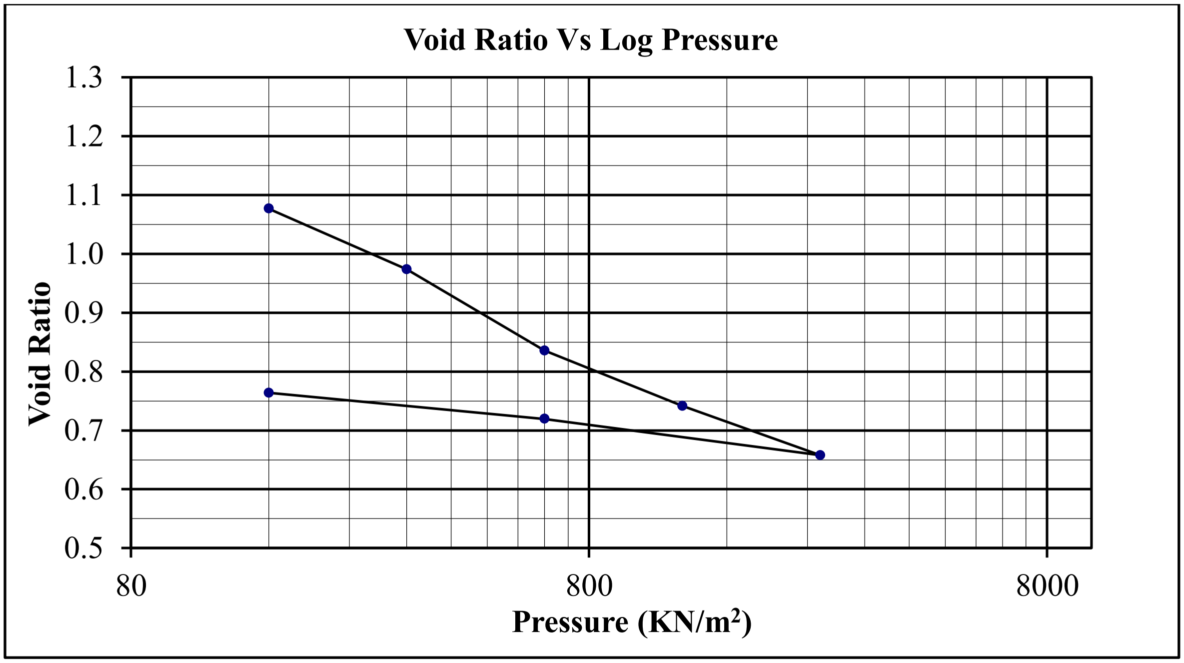

Figure E- 8: Void ratio versus log pressure for soil reinforced with 1% of fiber and lime stabilized

1. Compression index of soil reinforced with 1% of fiber and lime stabilized:

P_1 = 160 Kpa $e_1 = 1.077$

P_2 = 2560 Kpa $e_2 = 0.658$

$$C_c = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.348$$

2. Re-compression index of soil reinforced with 1% of fiber and lime stabilized:

P_1 = 160 Kpa $e_1 = 0.764$

P_2 = 2560 Kpa $e_2 = 0.658$

$$C_r = \frac{e1-e2}{\log(\frac{p2}{P1})} = 0.088$$