# EXPERIMENTAL STUDY ON DYNAMIC PROPERTIES OF EXPANSIVE SUBGRADE SOIL STABILIZED WITH LIME

Dagmawit Tsegaye Kebede

A Thesis Submitted to the Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

September, 2022

Adama, Ethiopia

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# DECLARATION

I hereby declare that this Master Thesis entitled "Experimental Study on Dynamic Properties of Expansive Subgrade Soil Stabilized With Lime" 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.

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| 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 "Experimental Study on Dynamic Properties of Expansive Subgrade Soil Stabilized With Lime" prepared under my guidance by Dagmawit Tsegaye Kebede submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (Specialization in Geotechnical Engineering). Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

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| Advisor | Signature | Date |

# Approval Sheet

I, the advisor of the thesis entitled "Experimental Study on Dynamic Properties of Expansive Subgrade Soil Stabilized With Lime" and developed by Dagmawit Tsegaye Kebede, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of Examiners of the thesis by Dagmawit Tsegaye Kebede have read and evaluated the thesis entitled "Experimental Study on Dynamic Properties of Expansive Subgrade Soil Stabilized With Lime" 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 Geotechnical Engineering.

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# SYMBOLS AND ABBREVIATIONS

AAIT: Addis Ababa Institute of Technology

AASHTO: American Association of State Highway and Transportation Officials

ASTM: American Society for Testing and Materials

CH: Highly plastic clay

CBR: California Bearing Ratio

E: Easting

ERA: Ethiopian Road Authority

LL: Liquid Limit

LVDT: Linear Variable Differential Transformer

MDD: Maximum dry density

N: Northing

OCR: Over consolidation ratio

OMC: Optimum moisture content

PI: Plasticity Index

PL: Plastic Limit

UCS: Unconfined compressive strength

USCS: Unified Soil Classification System

$A_{loop}$: Area of the stress - strain loop

$A_{\Delta}$: Area of triangle

B: Width of ellipse

D: Damping ratio

e: Void ratio

Eq.: Equation

f: Frequency of cyclic loading

g : Gram

$gm/cm^3$ : Gram per Centimeter Cube

| | |
|---|---|
| G: | Shear modulus |
| $G_{max}$: | Maximum shear modulus |
| $G/G_{max}$: | Normalized shear modulus |
| $G_s$: | Specific gravity |
| $G_{sec}$: | Secant shear modulus |
| Hz: | Hertz |
| K: | Dimensionless quantity which is a function of PI |
| $K_o$: | Coefficient of lateral earth pressure at rest |
| Kpa: | Kilopascal |
| No: | Number |
| $P_c$: | Pre-consolidation pressure |
| $P_o$: | Present effective vertical pressure |
| W: | Maximum strain energy |
| $\Delta W$: | Dissipated energy |
| $\sigma'_o$: | Effective confining stress |
| $\sigma'_v$: | Effective vertical stress |
| $\tau$: | Cyclic shear stress |
| $\tau_{max}$: | Maximum cyclic shear stress |
| $\tau_{min}$: | Minimum cyclic shear stress |
| $\gamma$: | Cyclic shear strain |
| $\gamma_{max}$: | Maximum cyclic shear strain |
| $\gamma_{min}$: | Minimum cyclic shear strain |

# ABSTRACT

*Expansive clay soils are poor subgrade materials and their engineering properties are primarily influenced by intense moisture fluctuations. These soils exhibit swelling and shrinkage behavior which causes damage to structures, particularly lightweight buildings and pavements. Soil stabilization using different admixtures is one of the most common methods of treatment for enhancing the qualities of expansive soil. Dynamic characteristics of soils are important in designing and analyzing geotechnical structures subjected to dynamic loading. During this loading, expansive soil loses the small shear strength it possesses, resulting in additional damage. This study investigated the dynamic properties namely shear modulus and damping ratio of the untreated and lime-treated expansive subgrade soils for different strain amplitudes ranging from 0.01 to 1%. The strain amplitude, confining pressure, cycle number, and the effect of lime treatment were the parameters considered in this study. The study is carried out on expansive subgrade soil samples collected from Addis Ababa, Summit- Figa road project. The experimental study first performed Atterberg limits, free swell, compaction, unconfined compressive strength (UCS), and California bearing ratio (CBR) test on the mixtures prepared by adding different percentages of lime (0,3,6, and 9%). Laboratory cyclic simple shear tests were then performed to determine dynamic properties on the natural soil samples and remolded soil samples treated with lime prepared by compaction. Test results show that the plasticity characteristics and the swelling potential of the expansive clayey soil decrease with increased lime content and UCS of soil increases up to 6% addition of lime by dry weight of the soil. It was found that for both lime-treated and untreated soil samples the dynamic shear modulus decreases as the strain rate increases while increasing with the number of cycles and confining pressure. Conversely, the damping ratio value increases as the shear strain rate increases whereas, it decreases as the number of cycles and confining pressure increase. Furthermore, results indicate that treated soil has a higher shear modulus and lower damping ratio than untreated soil. Generally, the study shows the use of lime to stabilize expansive subgrade soil under both static and dynamic loading is effective in increasing the stiffness of the soil and reducing deformation.*

**Keywords**: Expansive soil, Stabilization, Lime, Dynamic soil property, Shear modulus, Damping ratio

# CHAPTER ONE

# INTRODUCTION

## 1.1. Background

Expansive soils are characterized by high plastic clays that are prevalent around the world and undergo significant volumetric changes in terms of swelling or shrinkage as moisture content variation. This is due to the presence of montmorillonite, a clay mineral that swells and shrinks a lot in response to changing moisture conditions. The process of swelling produces pressures that can cause structures to lift or heave, while shrinkage causes differential settlement (Hashem et al., 2016). These soils usually have undesirable engineering characteristics, such as low bearing capacity, low stability, and excessive swelling. The nature of these soils poses significant challenges to civil engineering structures, particularly road pavements (Zumrawi and Hamza, 2014).

The quality and service life of the pavement is significantly affected by the subgrade materials type. One of the challenges in the design and construction of pavements is the availability of adequate subgrade materials that meet the criteria of international standards (Aga, 2021).In Ethiopia, the road construction industry is facing challenges related to unstable and problematic soils. It has been observed that areas, where various construction activities are going on, are covered with expansive soil.

The occurrence of expansive subgrades is a challenge during pavement construction due to its substantial volume changes. The common procedure in this situation is to remove the natural expansive soil and replace it with non-expansive soil. However, this technique becomes unsuitable for deep and long extending formations of natural expansive soils. As a result, attempts have been made to improve the qualities of expansive soil by employing various types of stabilizers (Ahmed et al., 2020).

Chemical soil stabilization methods have been employed in tackling the challenges associated with expansive subgrade soils (Jalal et al., 2020). It involves adding various types of admixtures such as cement, lime, and other material. Stabilization alters soil properties such as strength, compressibility, swelling potential, and volume change properties. This change in soil properties may have an impact on the dynamic strength and features of the stabilized soil due to a hundred million times of cyclical traffic load action on the subgrade (Wang and Mei, 2012; Yonghui et al., 2017).

The importance of dynamic characteristics of soils in analyzing and designing geotechnical structures subjected to dynamic loadings such as earthquakes, vehicular traffic, machine vibration, pile-driving, blasting, and so on has led to a lot of research. However, a limited investigation is carried out to evaluate the dynamic properties of untreated and treated expansive soil. The soil responds nonlinearly to cyclic loads such as earthquakes and high traffic, which is important for the design of civil engineering structures such as embankments, road pavements, and railroads.

Any analysis of dynamic engineering problems requires the determination of two critical parameters, namely the shear modulus and damping ratio. Typically, these two parameters are measured in the laboratory or the field. The selection of testing methodologies for identifying dynamic soil characteristics necessitates a detailed analysis and understanding of the situation. One of the laboratory techniques for determining the dynamic properties of soils is using the cyclic simple shear test. It is possible to test stiffness and damping properties over a wide range of strain levels by applying cyclic shear stress to horizontal planes (Wilson, 2016).

In this study, the dynamic properties of expansive soil treated with lime have been investigated using a cyclic simple shear testing device. For the study, the soil sample was collected from Addis Ababa, Summit-Figa road project in Lemi-Kura sub-city.

## 1.2. Statement of the Problem

In the construction of pavements, the subgrade serves as the foundation for the pavement and must satisfy strength and deformation criteria under repeated cyclic loading. The loads imposed must be transmitted to soil layers that are capable of sustaining them without failing in bearing capacity, and soil layer deformations must not result in excessive permanent pavement settlement. However, pavements that lay on expansive subgrades soil can be subjected to early distress, resulting in the premature failure of the road pavement structure and requiring some type of modification and reengineering to improve its load-bearing capability (Amakye and Abbey, 2021). Expansive soil, when subjected to dynamic loading, loses the small shear strength it possesses and results in causing road cracks and movement (Ashango and Patra, 2013).

Chemical stabilization utilizing various admixtures such as cement, lime, fly ash, and other materials have been used effectively to improve the performance of problematic subgrade soils. Several studies of these stabilized soils have been carried out under static loads. However, limited study has been conducted to determine the dynamic properties of stabilized expansive soil, particularly in Ethiopia. The subgrade is typically subjected to different dynamic vibration loads induced by traffic, earthquakes, pile driving, and other vibration machines (Cherian and Kumar, 2017). Consequently, the material qualities under such a load must be examined to comprehend the response and dynamic behavior of stabilized expansive subgrade soil.

Therefore, the purpose of this study will be to fill the gap by studying the dynamic properties of lime-treated expansive soil and the effect of lime stabilization on the dynamic properties of expansive subgrade soil.

## 1.3. The Objective of the Study

### 1.3.1. General Objective

The main objective of this research is to investigate the effect of lime stabilization on the dynamic properties of expansive soil.

### 1.3.2. Specific Objective

- ✓ To characterize the natural subgrade soil.
- ✓ To determine the proportion of lime required to obtain maximum strength for stabilization of the expansive subgrade soil.
- ✓ To determine the dynamic soil properties (shear modulus and damping ratio values) of lime treated and untreated expansive soil using a cyclic simple shear test.
- ✓ To investigate the effect of factors on the dynamic properties of lime-treated and untreated expansive soils.

## 1.4. Research Question

At the end of this study, the research will answer the following question.

- ✓ What are the properties of the natural subgrade soil?
- ✓ In which proportion of lime maximum strength is obtained?
- ✓ What are the dynamic properties (shear modulus and damping ratio values) of lime-treated and untreated expansive soil?
- ✓ What is the effect of factors on the dynamic properties of lime-treated and untreated expansive soils?

## 1.5. Scope of the Study and Its Limitation

The scope of this study is to investigate the properties of natural expansive soil, analyze the effect of the addition of lime on the characteristics of the soil, and determine the optimum amount of lime that could enhance the strength capacity of the expansive soil collected from the study area, and evaluating the influence of the addition of lime on dynamic properties (shear modulus and damping ratio) of the soil using a cyclic simple shear test. Despite the fact that numerous elements influence a soil's dynamic properties, only effective confining pressure, strain amplitude, the number of loading cycles, and effect of treatetment were examined in this study. Finally, a comparison of the results of the study with those of other previous studies was made.

## 1.6. Significance of the Study

When solving geotechnical problems involving dynamic loading, the variation of soil dynamic properties is a critical input. Weak subgrade soils, such as those found on expansive soil, must always be improved in order to control stability and settlement. Chemical stabilization can be a method to improve the engineering properties of this subgrade soil. The dynamic property of this stabilized soil should be studied by taking into account the vibration caused due to repetitive traffic loads,earthquake and machine vibration.

Therefore the study has valuable significance to explore the dynamic properties of treated expansive soils under dynamic loads. The outcome of this study contributes to a better understanding of how stabilization enhances the static and dynamic characteristics of expansive soils and the factor that influence the dynamic properties of soil. Furthermore, as literature, the study can be utilized as input for other researchers or institutions.

## 1.7. Organization of the Thesis

This thesis work is organized into five chapters. The first chapter gives a brief description of the thesis background, problem statement, objectives, research questions, the significance of the study, and the scope and limitations of the work. The second chapter discusses the literature reviews on the properties of expansive soil, soil stabilization method, and dynamic properties of soil, and previous research works that have been done on both lime stabilization and dynamic property analysis of stabilized soil. The third chapter explains the material and methodologies used for the study. The fourth chapter summarizes the results of various laboratory tests and discusses the results, as well as makes some comparisons with previous research. The final chapter presents the conclusions and recommendations of the study.

# CHAPTER TWO

# LITERATURE REVIEW

## 2.1. General

Expansive soils are problematic soils that typically exhibit evident volume change as moisture content changes, posing major structural and geotechnical difficulties around the world. These soils usually have undesirable engineering properties, thereby low bearing capacity, low stability, and excessive swelling. Expansive soil causes more damage to buildings and infrastructure systems every year than the damage caused by floods, hurricanes, tornadoes, and earthquakes combined (Wu et al., 2019).

Subgrade materials are the ground or soil underneath a road pavement and should have sufficient capacity to support the weight of the road pavement and the traffic loads imposed. However, Pavements built on expansive subgrade soils experience early distress, resulting in premature pavement failure. It is well known that as the volume of material changes, larger stresses can be produced. These stresses react in the form of cracking, heaving, and settlement of highway pavements. As a result, road authorities would face a significant increase in the costs of routine maintenance, rehabilitation, and reconstruction of deteriorated pavements (Kim & Buttlar, 2009).

Many researchers are conducting substantial research on this problem and its remedial measures. Among various approaches for the remedial to the problem caused by expansive soils, the stabilization of such soils is the most practical alternative. In general, Soil stabilization appears to be an effective option for enhancing soil properties (Zumrawi & Hamza, 2014).

## 2.2. Review of Expansive Soils

### 2.2.1. Origin and Mineralogy of Expansive Soils

The formation of expansive soils is related to a combination of conditions and processes that result in the creation of clay minerals having a particular chemical composition that expands when in contact with water. The conditions or processes, which determine the clay mineralogy, include the composition of the parent material and the degree of physical and chemical weathering to which the materials are subjected (Bililign, 2019).

The parent materials that can be associated with expansive soils are either the basic igneous rocks or sedimentary rocks that contain montmorillonite as a constituent. The basic igneous rocks have a low silica content, ranging from 45 to 52 percent. This group includes rocks having a high metallic base, such as pyroxenes, amphiboles, biotite, and olivine. Gabbros, basalts, and volcanic glasses are examples of such rocks (Chen, 2012). The sedimentary rocks that contain montmorillonite as a constituent include shale and clay stones. The weathering process by which clay is formed includes the physical, biological, and chemical processes. The chemical weathering of parent rock minerals is the most important weathering process that leads to the formation of montmorillonite.

The clay mineral basic crystalline structural unit consists of silica tetrahedron blocks and aluminum octahedron blocks. Aluminum octahedron block may have Aluminum ($Al^{3+}$) or magnesium ($Mg^{2+}$). Gibbsite [$Al_2$ $(OH)_6$] is formed when only aluminum is present; brucite [$Mg_3(OH)_6$] is formed when only magnesium is present. These clay minerals have three important structural groups which are the kaolinite group, illite group, and montmorillonite group.

The clay mineral Kaolinitete is a two-layer mineral with a single tetrahedral sheet that is joined by a single octahedral sheet to produce a 2 to 1 lattice structure. The bonding combination of hydrogen and Vander Waals forces results in considerable strength and stability with little tendency for interlayers to take on water and swell. The bonding is sufficiently strong that there is no interlayer swelling in the presence of water (Mitchell and Soga, 2005). Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. There is a partial substitution of aluminum by magnesium and iron in the octahedral sheet and a partial substitution of silicon by aluminum in the tetrahedral sheet. Because of the (non-exchangeable) potassium ions held between them, the combined sheets have a relatively weak bonding. Montmorillonite clay minerals are characterized by weekly bonded layers. Each layer is made up of two silica sheets with an aluminum (gibbsite) sheet sandwiched between them. Water and exchangeable cations can enter the layers and separate them; as a result, the combined sheets have a very weak bond. Considerable swelling of montmorillonite can occur due to additional water absorbed between the combined sheets (Craig, 2013).

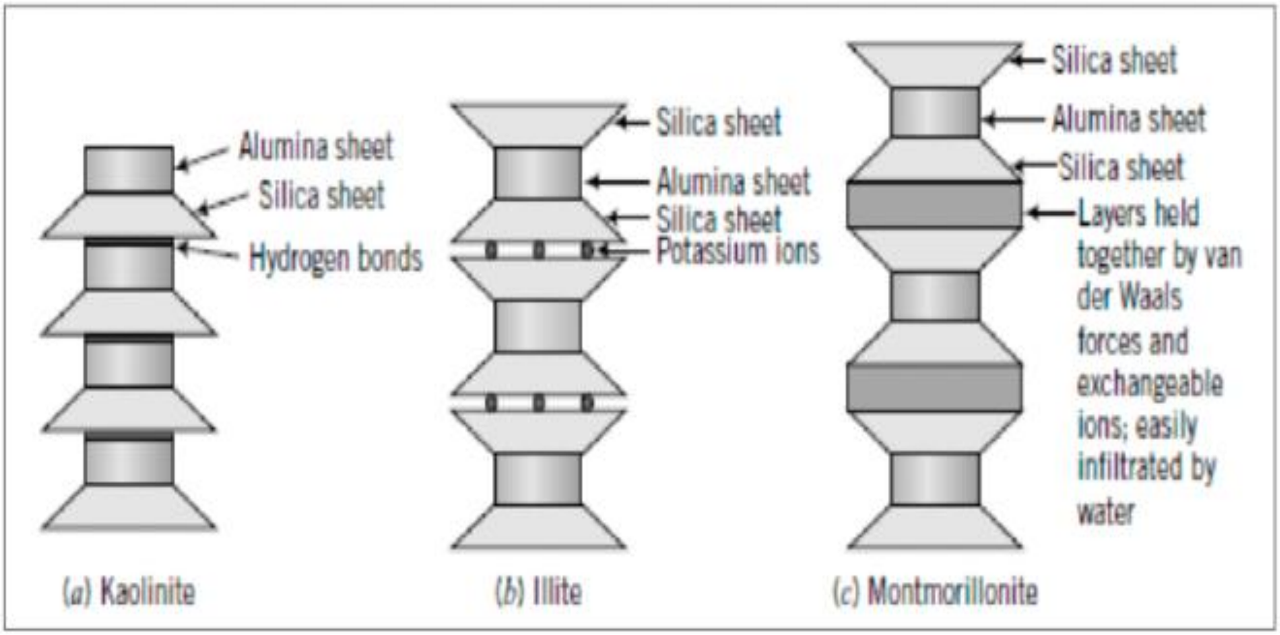


Figure 2.1: Structure of kaolinite, illite, and montmorillonite (Muni Budhu, 2007)

### 2.2.2. Identification of Expansive Soils

Expansive soils can be recognized both in the field by visual identification and through a series of laboratory tests.

#### I. Field Identification

Expansive soils can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include (Nelson & Miller, 1997).

- ✓ They usually have a black or grey color
- ✓ Wide and deep shrinkage cracks occur during dry periods
- ✓ Soil is rock-hard during the dry season, but very sticky and soft when wet
- ✓ A shiny surface when a partly dry piece of the soil is polished with fingers
- ✓ Damages on the surrounding structures due to the expansion of soils

## II. Laboratory Identification

There are three methods of identifying expansive soils in the laboratory test method. These are mineralogical identification, indirect methods, and direct measurement methods (Chen, 2012; Nelson and Miller, 1997).

**Mineralogical Identification-**mineralogy is a key aspect in determining how expansive soil behaves. The various methods of mineralogical identification are important in a research laboratory in exploring the basic properties of clays but are impractical and uneconomical for practicing engineers. A variety of techniques can be used to identify clay minerals, These are X-ray diffraction, Differential thermal analysis, Dye adsorption, Chemical analysis, and Electron microscope resolution.

**Indirect Methods-**Simple soil property tests can be performed in this approach to evaluate the swelling potential of expansive soils using the swelling test. These tests are simple to do and should be included as part of any assessment of expansive soils. Such tests include Atterberg limits and free swell Tests.

**Direct Methods** -Direct measurement is the most accurate and simple way to determine the swelling potential and swelling pressure of expansive clay. The conventional one-dimensional consolidation test can be used to take direct measurements of expansive soils. Such a device allows for the quick and reliable evaluation of clay's swelling potential under various conditions.

### 2.2.3. Classification of Expansive Soils

A range of classification methods has been developed using parameters derived from extensive soil identification testing. American Association of State High Way and Transportation Official (AASHTO) and United Soil classification systems (USCS) systems are widely known classification systems that use index properties of soils determined by simple laboratory tests for soil classification. AASHTO system classifies soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. Soil rated A-6 or A-7 by AASHTO can be considered potentially expansive (Nelson and Miller, 1997).

Table 2.1: AASHTO soil classification system (B. M. Das, 2010)

| *General Classification* | *Granular materials (35% or less passing No. 200 Sieve (0.075 mm)* | | | | | | | *Silt-clay Materials More than 35% passing No. 200 Sieve (0.075 mm)* | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Group Classification* | *A—1* | | *A—3* | *A—2* | | | | *A—4* | *A—5* | *A—6* | *A—7* |
| | *A—1—a* | *A—1—b* | | *A—2—4* | *A—2—5* | *A—2—6* | *A—2—7* | | | | *A—7—5* *A—7—6* |
| (*a*) Sieve Analysis: Percent Passing | | | | | | | | | | | |
| (*i*) 2.00 mm (No. 10) | 50 max | | | | | | | | | | |
| (*ii*) 0.425 mm (No. 40) | 30 max | 50 max | 51 min | | | | | | | | |
| (*iii*) 0.075 mm (No. 200) | 15 max | 25 max | 10 max | 35 max | 35 max | 35 max | 35 max | 36 min | 36 min | 36 min | 36 min |
| (*b*) Characteristics of fraction passing 0.425 mm (No. 40) | | | | | | | | | | | |
| (*i*) Liquid limit | | | | 40 max | 41 min | 40 max | 41 min | 40 max | 41 min | 40 max | 41 min |
| (*ii*) Plasticity index | 6 max | | N.P. | 10 max | 10 max | 11 min | 11 min | 10 max | 10 max | 11 min | 11 min* |
| (*c*) Usual types of significant Constituent materials | Stone Fragments Gravel and sand | | Fine Sand | Silty or Clayey Gravel Sand | | | | Silty Soils | | Clayey Soils | |
| (*d*) General rating as subgrade. | Excellent to Good | | | | | | | Fair to Poor | | | |

In the Unified soil classification system, a correlation is made between swell potential and unified soil classification. In this system, the soils are classified into fifteen groups.

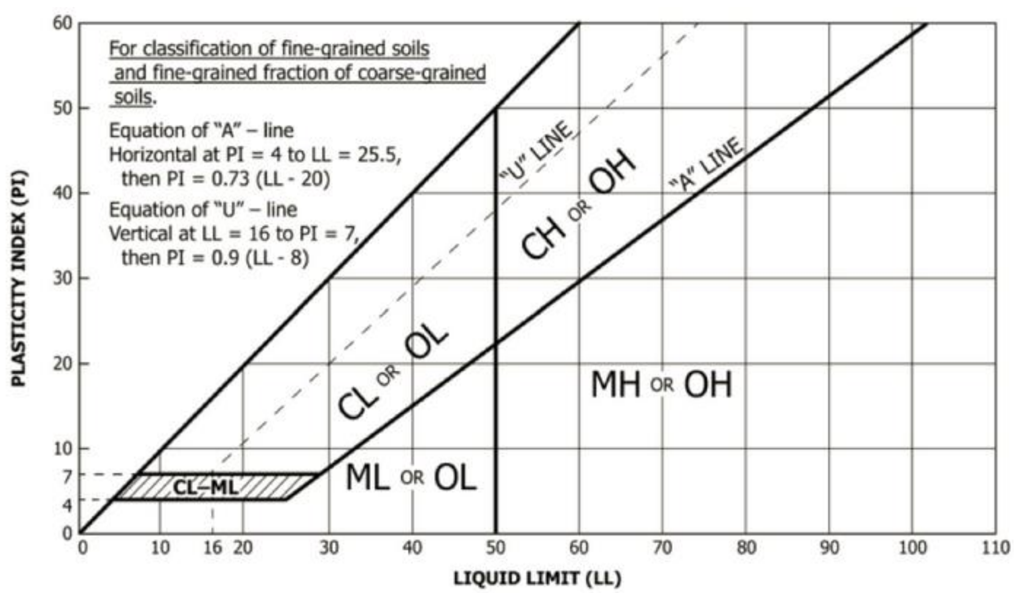


Figure 2.2: USCS soil classification system (B. M. Das, 2010)

The common criteria used for identification and classification of the expansive soils are by employing indirect prediction of swell potential including correlations based on index properties, swell, physical indicators, and a combination of them. Some of such classification systems are mentioned below:

### I. Method of Chen

In this method, a correlation is made between swell data and percent less than No. 200 sieves, liquid limit, and standard penetration resistance. The classification is as given in Table 2.2 (Chen, 1988)

Table 2.2: Chen method of expansive soil classification

| <No.200sieve, % | LL, % | Standard penetration | Probable Expansion | Degree of Expansion |
|---|---|---|---|---|
| <30 | <30 | <10 | <1 | Low |
| 30-60 | 30-40 | 10-20 | 1-5 | Medium |
| 60-95 | 60 | 20-30 | 3-10 | High |
| >95 | >60 | >30 | >10 | Very high |

### II. USBR Method

This method is developed by Holtz and Gibbs; it is based on a direct correlation of observed volume change with colloid content, plastic index, and shrinkage limit. The classification is as given in Table 2.3.

Table 2.3:Classification based on the bureau of reclamation method (Chen, 2012; Craig, 2013)

| Colloid Content (%) | Plasticity index (%) | Shrinkage Limit (%) | Probable Expansion (%) | Degree of Expansion |
|---|---|---|---|---|
| <18 | >15 | >15 | <10 | Low |
| 15-28 | 15-28 | 10-20 | 10-20 | Medium |
| 25-41 | 25-41 | 20-30 | 20-30 | High |
| >35 | >35 | >30 | >30 | Very high |

**III. Method of Daksanamurthy and Raman (1973)**

This method presented a single index method for identifying expansive soils using only liquid limits. They suggested four classes of clays according to their liquid limits as shown in Table 2.4.

Table 2.4: Relation between the swelling potential of clays and the liquid limit (Chen, 2012) Al-Rawas,2006)

| Swelling Potential | Liquid Limit |
|---|---|
| Low | 20<LL ≤ 35 |
| Medium | 35<LL≤50 |
| High | 50<LL≤70 |
| Very high | LL>70 |

## 2.3. Soil Stabilization

Soil stabilization is the process of alteration of one or more soil properties to create an improved soil material possessing the desired properties to meet specified Engineering requirements. By lowering permeability and improving overall strength, the stabilization process ensures that the soil is stable. As a result, the soil becomes strengthened and improved its bearing capacity.

Pavement design is based on the assumption that each layer of material in the pavement system will achieve a minimum specified structural quality. Each layer must be resistant to shearing, prevent excessive deflections that induce fatigue cracking within the layer or in the overlying layer, and avoid excessive permanent deformation. Stabilization is commonly used to improve quality, such as lowering the plasticity index or swelling potential, as well as increasing durability and strength with better soil gradation. As the quality of a soil layer is improved, the ability of that layer to distribute the load over a greater area generally increases so that a reduction in the required thickness of the soil and surface layers may be permitted (Guyer, 2018).

Mechanical and chemical stabilization is the most common methods for soil stabilization. Mechanical stabilization is a process of improving the stability and shear strength properties of the soil without changing the chemical properties of the soil (Molenaar, 2005). Compaction, excavation and replacement, and the mixing of different soils, are the method of mechanical stabilization. In chemical stabilization, the soil is stabilized by adding different chemicals or other materials. In this category, soil stabilization depends mainly on chemical reactions between the stabilizer and soil mineral to achieve the desired effect (Makusa, 2013). Chemical stabilization is commonly regarded as the most effective method of expansive soil improvement when the right additive is used; it changes the nature of the soil, strengthens it, and removes its sensitivity both to water and subsequent stress (Batari et al., 2017).

### 2.3.1. Lime Stabilization

Lime is an inorganic chemical compound that is produced by burning limestone in kilns. Lime is one of the oldest and still popular additives used to improve fine-grained soils. The two primary types of lime used in construction are quick lime (calcium oxide) and hydrated lime (calcium hydroxide) (Feysel, 2017). Calcium hydroxide (hydrated lime) is a fine, dry powder formed by 'slaking' quicklime (calcium oxide, CaO) with water; quicklime is produced by heating natural limestone (calcium carbonate, $Ca(CO)_3$) in a kiln until carbon dioxide is driven out. The following chemical equations show the reaction from which quick lime and hydrated lime are produced.

$$CaCO_3 + Heat \longrightarrow CaO + CO_2$$

$$CaO + H_2O \longrightarrow Ca(OH)_2 + Heat$$

Hydrated lime in the form of lime is used in the majority of lime stabilization work. Other forms of lime sometimes used in lime stabilization work are dehydrated dolomite lime, monohydrated dolomite lime, and dolomite quick lime. Quick lime is more effective as a stabilizer than hydrated lime, but not usually used for stabilization because it is caustic hence dangerous to handle, susceptible to moisture uptake in storage, and gives off much heat during hydration (Brook, 2015). Generally, hydrated lime is used in this study and is also known as slacked lime. The higher the magnesium content of the lime, the less affinity for water and the less heat generated during mixing.

Physically, quicklime is white of varying degrees of intensity, depending on its chemical purity. The color of lime depends on the occurrence of impurities within the lime and results in a grayish or yellowish appearance. Hydrated limes also have a similar relationship between purity and whiteness (Reshid, 2014). The chemical composition of hydrated Lime used for the study is shown in Table 2.5 below.

Table 2.5: Chemical composition of Senkele hydrated Lime (Negussie et. al, 2014 )

| Constituent | CaO | $SiO_2$ | MgO | $Fe_2O_3$ | $Al_2O_3$ | $K_2O$ | $Na_2O$ | $SO_3$ | $TiO_2$ | MnO | $P_2O_5$ | LOI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Percentage (%) | 59.47 | 6.21 | 3.91 | 3.57 | 2.18 | 0.79 | 0.61 | 0.58 | 0.33 | 0.28 | 0.21 | 17.04 |

The mechanisms by which lime stabilization occurs due to the chemical reactions between the lime and soil particles can broadly be grouped into initial (short-term) and longer-term. The initial reaction involved is attributed to cation exchange by replacing the exchangeable $Na^+$ or $K^+$ ion in the clay with the $Ca^{+2}$ ion of the lime and flocculation–agglomeration reactions (Feven, 2017). By the replacement of ions, the diffused water layer around the clay particle will decrease in thickness resulting in a significant change in the plasticity characteristic of the soil. The short-term reaction which occurs within minutes to hours of mixing leads to an immediate improvement in soil plasticity and workability due to fundamental change in the clay particle chemistry. The short-term effect includes, the LL (liquid limit) of the soil decreasing and the PL (plasticity limit) increasing, thus reducing the PI (plasticity index) of the soil; the finer clay-sized particles agglomerate to form larger particles; the large particles (clay clods) disintegrate to form smaller particles, and a drying effect takes place due to the absorption of moisture for hydration (Feysel, 2017).

Longer-term reactions involve interaction between free lime $Ca(OH)_2$ and soil particles these interactions are referred to as pozzolanic as they involve pozzolans (materials that consist predominantly of silica and alumina, such as clay minerals, and fly ash). The pozzolanic reaction is a slow process and occurs in the longer term for days and weeks and leads to the formation of cementing products that crystallize in the pore space between soil grains (Feysel, 2017). Long-term lime stabilization of soils, unlike the immediate effect, is a cementation process that bound individual grains together and significantly increases the mechanical strength of the treated material. The gain in strength associated with the formation of pozzolanic reaction is accelerated by heat, an advantage when using lime stabilization in the hot climate.

Extensive research has been done on the stabilization of soils using lime alone or in combination with other admixtures. Some of the studies are mentioned below.

## 2.4. Related Research Works on Lime Stabilization

**Gizachew, (2019)**, investigated the geotechnical property of expansive soil stabilized with marble dust and lime for road construction projects. Atterberg limits, free swell, compaction, and CBR tests were carried out by fixing the lime percentage (1-3) % and increasing the percentage of marble dust by 0%, 5%, 10%, 15%, 20%, 25%, and 30%. The results showed that adding marble dust with a small percentage of lime improved the geotechnical properties of expansive soil samples. It reduces plasticity index, swelling, and OMC with an increase in MDD and CBR values with all combinations. Therefore, using lime and marble dust improved the geotechnical property of soil and the materials can be used as a stabilizing agent for expansive soils.

**Negawo et al., (2019),** evaluate the efficiency of lime treatment to improve the mechanical properties of highly expansive clay soils from the Highlands of Ethiopia for road subgrades. Soils treated with quick lime at 5, 7, and 9% by dry weight of the soil were cured for seven days under a controlled temperature of 40 ± 2 °C and geomechanical laboratory tests were conducted to evaluate its impact on the engineering properties of the soil. Test results show substantial improvements in the properties of the soil after lime treatment. The addition of lime significantly reduces the plasticity index and swelling potential of the soil. Similarly, despite the reduction of optimum proctor dry density due to lime treatment, the unconfined compressive strength and the California bearing ratio show considerable improvements. Based on the study, expansive soils of the studied area can be effectively stabilized for road subgrade works with the addition of 7% quick lime by the dry weight of the soil.

**Feysel, (2017),** evaluate the effect of using hydrated lime to alter undesirable characteristics of high plastic soil to be used as core material for embankment dams. Different engineering laboratory tests were carried out on the CH soil in their natural states and when the hydrated lime was added to the soil at varying proportions of (2%, 4%, and 6%,) by weight of soil. From the test result, 6% of lime appears to be the minimum lime percentage required to make CH soil desirable core fill material. When 6% lime was added: The plasticity index was reduced by 53.51%, the maximum dry density improved by 7.34%, the optimum moisture content was reduced by 42.75%, and the compression index decreased by 27.08 %. The permeability also decreased by more than one order of magnitude from 4.47 * 106cm/set to 3.63 * 10-7cm/set. Therefore the study concluded that hydrated lime (Ca (OH) 2) can be a potential alternative to alter the undesirable characteristics of CH soil and make it a suitable core material for embankment dams.

**Indiramma et al., (2020),** investigated the individual and combined effect of lime and fly ash on the geotechnical properties of expansive soil. According to the study, expansive soil is first treated with 4% and 8% lime then, expansive soil is treated with 10% Fly ash + 4% Lime and 10% Fly ash + 8% Lime. The results show that the addition of lime alone or lime and fly ash in various percentages to expansive soil results in a flocculated structure with voids filled by admixtures, resulting in a decrease in plasticity characteristics, optimum moisture content, and differential free swell index, as well as an increase in maximum dry unit weight and thus strength of soil-admixtures, compared to soil alone.

## 2.5. Dynamic Soil Properties

The appropriate evaluation of the dynamic properties of soil is required to design a structure exposed to different dynamic loads such as machine vibration, traffic loads, earthquakes, and construction loading. Understanding, measuring, and quantitatively modeling the dynamic and cyclic features of the soils involved is required to evaluate the influence of such loads (Dobry and vucetic, 1987). The response of soil deposits influences dynamic loading, and in turn, the soil response is determined by dynamic soil characteristics.

The determination of two critical parameters, the shear modulus and the damping ratio value of the soils, is required for geotechnical engineering problems involving dynamic loading of soils and soil-structure interaction systems (Seed and Idriss, 1970). These two parameters are typically determined in the laboratory or field using various techniques. They are the reason that soils are not considered linear-elastic materials (Amir-Faryar and Aggour, 2016).

### 2.5.1. Shear Modulus and Damping Ratio

**Shear Modulus (G)**: Shear modulus is the measurement of a material's resistance to applied dynamic loading (Qiao et al., 2020). Shear modulus is usually expressed as the secant modulus, G determined by the extreme points on the hysteresis loop as shown in Figure 2.3. This hysteresis loop is produced from stress-strain values which are the results of laboratory tests such as triaxial and cyclic simple shear tests. The most reliable way of obtaining representative moduli has been to evaluate the ratio of cyclic shear stress ($\tau$) and corresponding cyclic strain ($\gamma$). Where, $\Delta\tau$ is the difference between the maximum and the minimum values of shear stress and $\Delta\gamma$ is the difference between the maximum and the minimum values of shear strain.

$$\text{Shear Modulus}, G = \frac{\Delta\tau}{\Delta\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.1)}$$

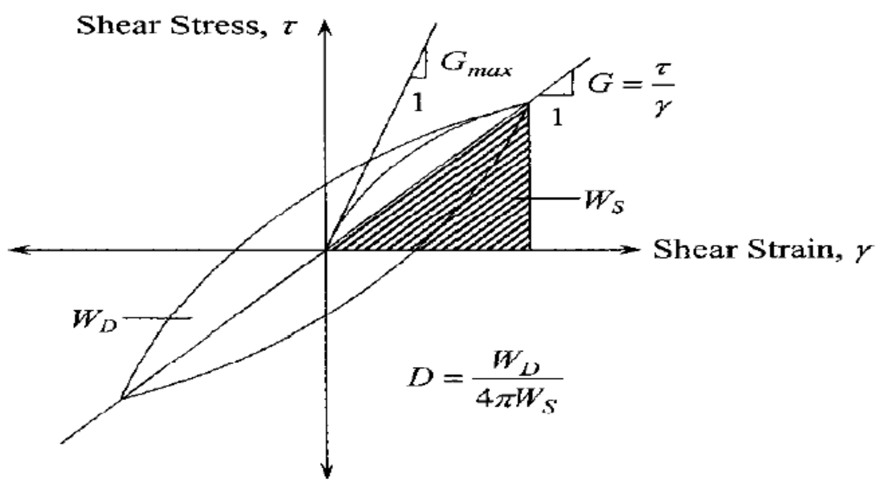


Figure 2.3:Hysteretic loop for one cycle of loading showing Gmax, G, and D (Kumar et al., 2013)

**Damping Ratio (D):** The dissipation of strain energy in the soil under cyclic loading is known as damping, and the damping ratio is defined as the dissipated energy divided by the maximum stored (i.e. elastic) energy per cycle. The damping ratio was determined experimentally, from the geometry of stress-strain loops using equation 2.2. The energy dissipation of cyclically loaded soils is frictional in nature, and the system deformation is mostly due to soil matrix rearrangement during both the loading and unloading stages of the cycle. The Friction at interparticle contact causes grain sliding or slipping, which is related to the hysteretic damping ratio.

$$\text{Damping ratio, D} = \frac{\text{Area of the hysteresis loop}}{4\pi * \text{Area of triangle}} = \frac{A_{loop}}{4\pi * A_{\Delta}} \text{ …………………… (2.2)}$$

Where, $A_{\Delta}$ is the maximum strain stored energy and $A_{loop}$ is the energy loose per cycle represented by the area enclosed inside the hysteresis loop as shown in Figure 2.3

$$\text{Area of loop} = \frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1}) \text{ ………………………… (2.3)}$$

The shear stress and shear strain were evaluated from lateral force and lateral LVDT test results.

$$\tau = \frac{\text{Shear Force}}{\text{Area of sample}} = \frac{F}{A} \text{ ………………………………………(2.4)}$$

$$\gamma = \frac{\text{Lateral Lvdt (Displacement)}}{\text{Height after consolidation}(<20\ mm)} \text{………………………………….(2.5)}$$

Where, LVDT is the linear variable, axial deformation measurement.

### 2.5.2. Determination of Initial (Maximum) Shear Modulus

**Initial Shear Modulus ($G_{max}$):** initial shear modulus is also known as the maximum shear modulus. $G_{max}$ is commonly regarded as one of the most important parameters in earthquake engineering, traffic engineering, vibration machine foundations, vibration isolation measures, and dynamic soil-structure interactions analysis (Kramer, 1996). For determining the initial shear modulus for different types of soils, a large number of empirical correlations have been developed. The maximum shear modulus, $G_{max}$, that corresponds to very low strain levels cannot be determined through cyclic simple shear tests since they are high strain tests (0.01% to 5%). The maximum shear modulus, $G_{max}$, of each specimen was determined for this study using Hardin B.O. and W. L. Black's equation (Hardin and Drnevich, 1972).

$$G_{max} = 3220 * \frac{(2.973-e)^2}{1+e} * OCR^a * \sigma'_O{}^{k_o} \quad \text{......(2.6)}$$

$$\sigma'_O = \frac{\sigma'_V + 2K_O * \sigma'_V}{3} \quad \text{......(2.7)}$$

Where ; e = void ratio

OCR = overconsolidation ratio = $\frac{P_C}{P_O}$

Pc = pre-consolidation pressure of a specimen (kPa)

Po = present effective vertical pressure (kPa)

σ'o = mean effective stress

Ko = coefficient of lateral earth pressure at rest ≈ 0.5

K = dimensionless quantity which is the function of PI

Table 2.6: Over-consolidation ratio exponent K varied with PI (B. Das and Ramana, 2011)

| Plasticity Index, PI(%) | K |
|---|---|
| 0 | 0 |
| 20 | 0.18 |
| 40 | 0.3 |
| 60 | 0.41 |
| 80 | 0.48 |
| ≥100 | 0.5 |

The results of the shear modulus of the cyclic simple shear tests were normalized to compare with the results of other tests. The normalized shear modulus ($G/G_{max}$) was computed using the calculated value of $G_{max}$. It's the ratio of shear modulus (G) to maximum shear modulus ($G_{max}$). The relation between normalized shear modulus and cyclic shear strain is shown in Figure 2.4.

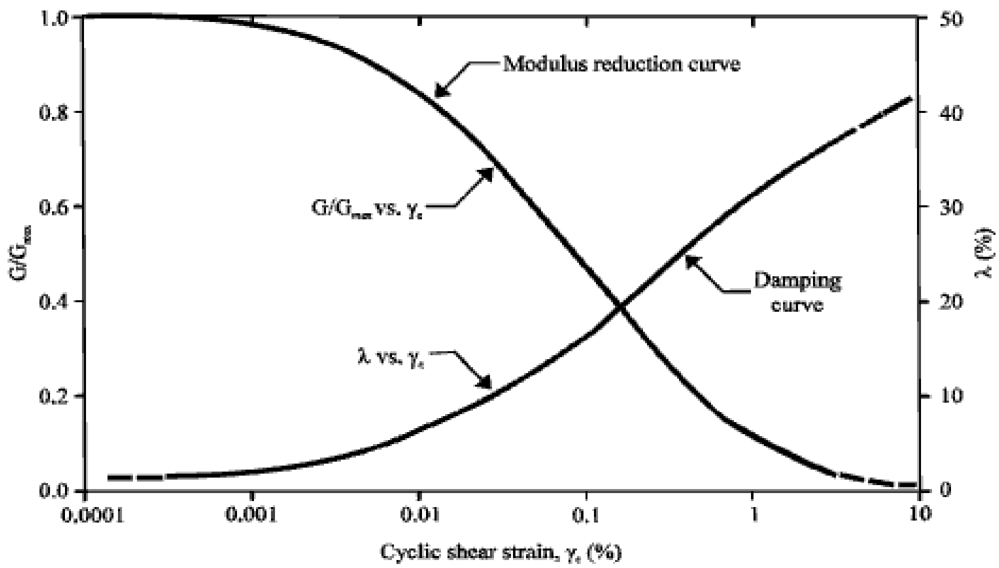


Figure 2.4: Shear modulus reduction and damping ratio curves (Khari et al., 2014)

### 2.5.3. Method of Determining Shear Modulus and Damping Ratio

Both shear modulus and damping properties have been estimated using several approaches, including laboratory and field experiments. Test procedures for measuring shear moduli and damping ratio were tabulated in Table 2.6 below.

Table 2.7: Test procedures for measuring shear moduli and damping ratio (Das 1993)

| General Procedure | Test condition | Approximate Strain range (γ) | Properties that can be determined |
|---|---|---|---|
| Determination of Hysteresis Stress-Strain relationships | Triaxial compression | 10-2 to 5 % | Modulus; Damping |
| | Simple shear | 10-2 to 5 % | Modulus; Damping |
| | Torsional shear | 10-2 to 5 % | Modulus; Damping |
| Forced vibration | Longitudinal vibrations | 10-4 to 10-2 % | Modulus; Damping |
| | Torsional vibrations | 10-4 to 10-2 % | Modulus; Damping |
| | Shear vibration-lab | 10-4 to 10-2 % | Modulus; Damping |
| | Shear vibration –field | 10-4 to 10-2 % | Modulus; Damping |

| | | | |
|---|---|---|---|
| Free vibration tests | Longitudinal vibrations | 10-3 to 1 % | Modulus; Damping |
| | Torsional vibrations | 10-3 to 1 % | Modulus; Damping |
| | Shear vibration-lab | 10-3 to 1 % | Modulus; Damping |
| | Shear vibration –field | 10-3 to 1 % | Modulus |
| Field wave velocity Measurements | Compression waves | $5x10{-}4\%$ | Modulus |
| | Shear waves | $5x10{-}4\%$ | Modulus |

#### 2.5.4. Factors Affecting Dynamic Properties of Soils

Soils, unlike many other structural materials, are highly nonlinear even at low strains. With increasing shear strain amplitude, this nonlinearity leads soil stiffness (represented by the shear modulus reduction curve) to decrease and the damping ratio to increase (Tigistu 2021).

Various researchers have been investigating the factors that influence soil dynamic behavior. These factors include the method of sample preparation in the laboratory( (whether intact or reconstituted samples), confining pressure, void ratio, water content, methods of loading, overconsolidation ratio, loading frequency, shearing strain amplitude, number of loading cycles, soil plasticity and percentage of fines (Jia, 2018). Out of these factors confining pressure, soil plasticity, the number of cycles, and shear strain levels were discussed in this study.

- ✓ **Effect of Shear Strain Amplitude**

Shear strain levels have a significant impact on the shear modulus and damping ratio of cyclically loaded soils. A particular soil has a high shear modulus and low damping ratio at low strain levels (<0.001%), and the soil stress-strain relationship is relatively linear. The stress-strain behavior of soil exhibits nonlinearity at higher strains (>0.01%), resulting in high damping ratio values. As a result, as the strain amplitude increases, the shear modulus of the soil decreases, and the damping ratio increases (Yohannes, 2015).

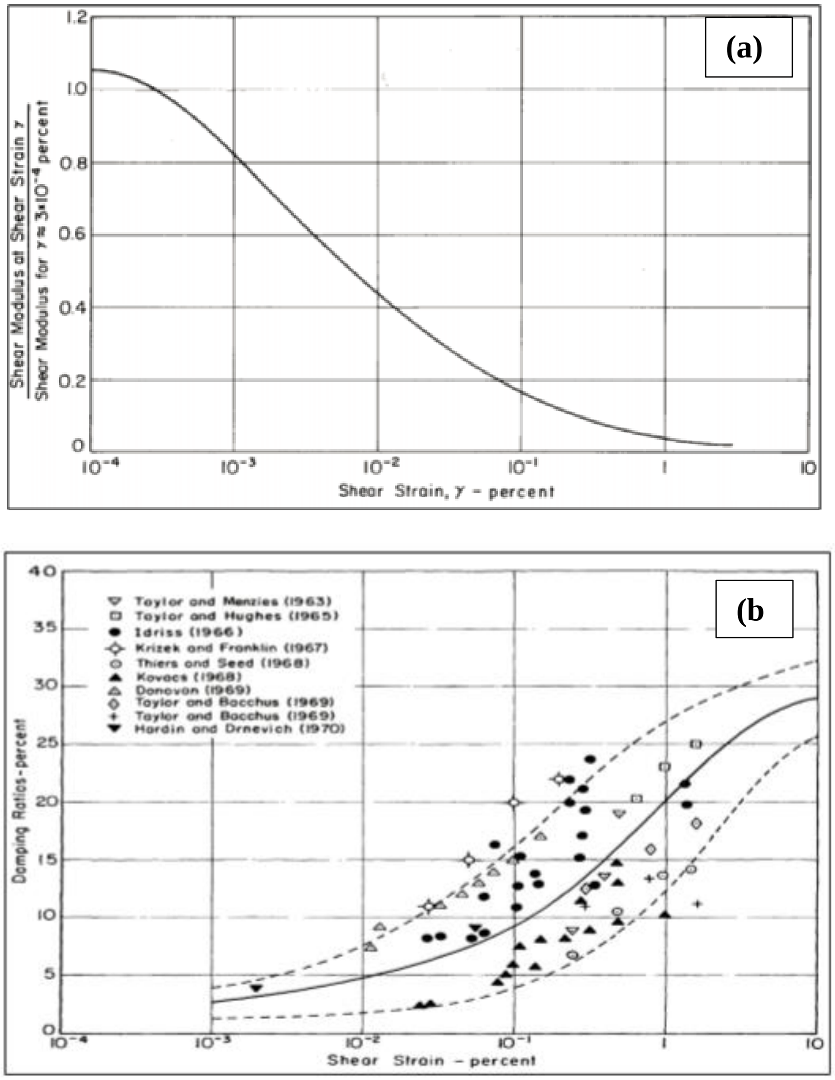


Figure 2.5:Variation of (a) shear modulus and (b) damping ratio with strain amplitude for saturated clay soil (Seed and Idriss, 1970)

- ✓ **Effect of Confining Pressure**

The shear modulus of soil increases as confinement increases, while the damping ratio decreases. This impact is most prominent with cohesion-less soils. However, confining pressure does not show a significant effect on normalized shear modulus values of cohesive soils. Various researchers concluded that the effect of confining pressure on the normalized shear modulus gradually diminishes as the plasticity of the soil increases (Sun et al., 1988).

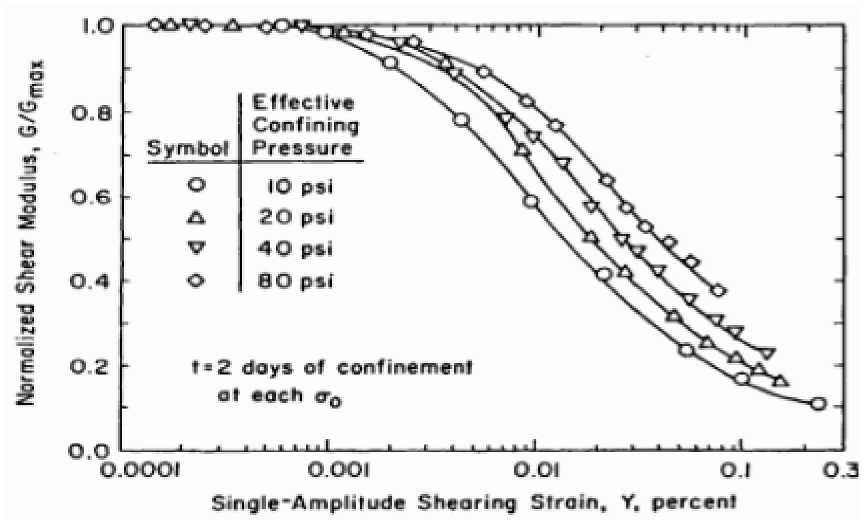


Figure 2.6:Effect of confining pressure on normalized modulus reduction (M. Vucetic, 1992)

- ✓ **Effect of Number of Cycles**

In general, the shear modulus slightly increased and the damping ratio significantly decreased as the number of loading cycles increased as shown in Figure 2.7. The primary increase in shear modulus and decrease in damping ratio occurred within the first 10 cycles of loading.

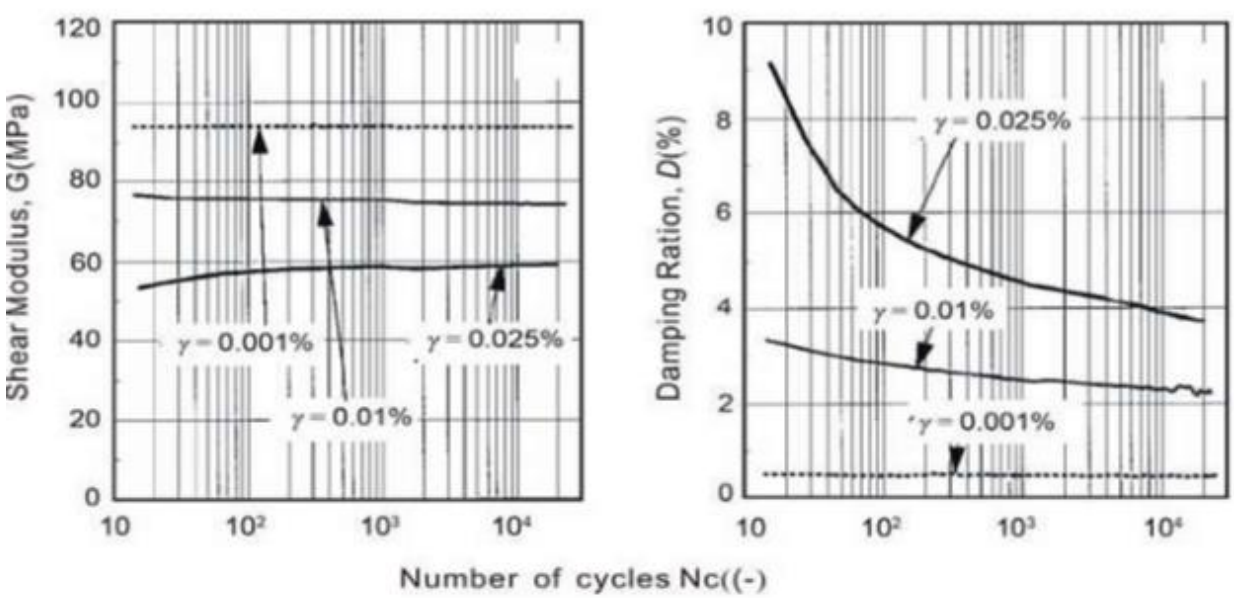


Figure 2.7: Effect of number of cycles on shear modulus and damping ratio (Min Wange, et al., 2012)

- ✓ **Effect of Void Ratio**

The void ratio is considered a critical factor that influences the soil's small-strain shear modulus. Soil particles get closer to each other when the void ratio decreases under load, resulting in the densification of the soil sample. As a result, the shear modulus increases, and damping ratio values decrease with a reduction in the void ratio.

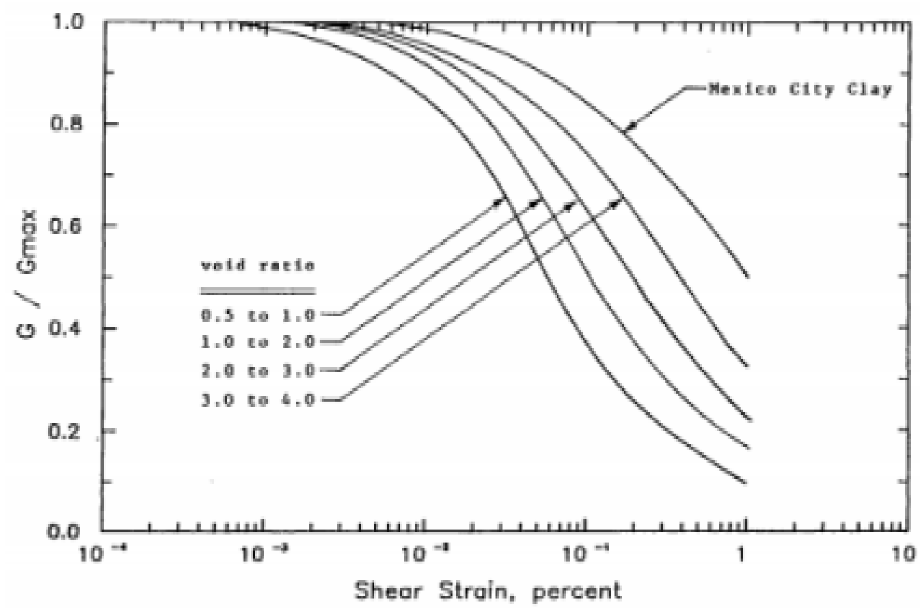


Figure 2.8: Normalized modulus reduction relation for clays with different void ratios (Hardin & Drnevich, 1972)

- ✓ **Effect of Soil Plasticity**

Unlike other variables, the plasticity index has a significant and consistent influence on the shear modulus of cohesive soils (Sun et al., 1988). Soils with higher plasticity generally exhibit a more linear stress-strain behavior. For the same cyclic strain value, the shear modulus of soil increases with PI, but the damping ratio decreases with PI (Kumar et al., 2013). The variation of shear modulus and damping ratio with soil plasticity is shown in Figure .2.9.

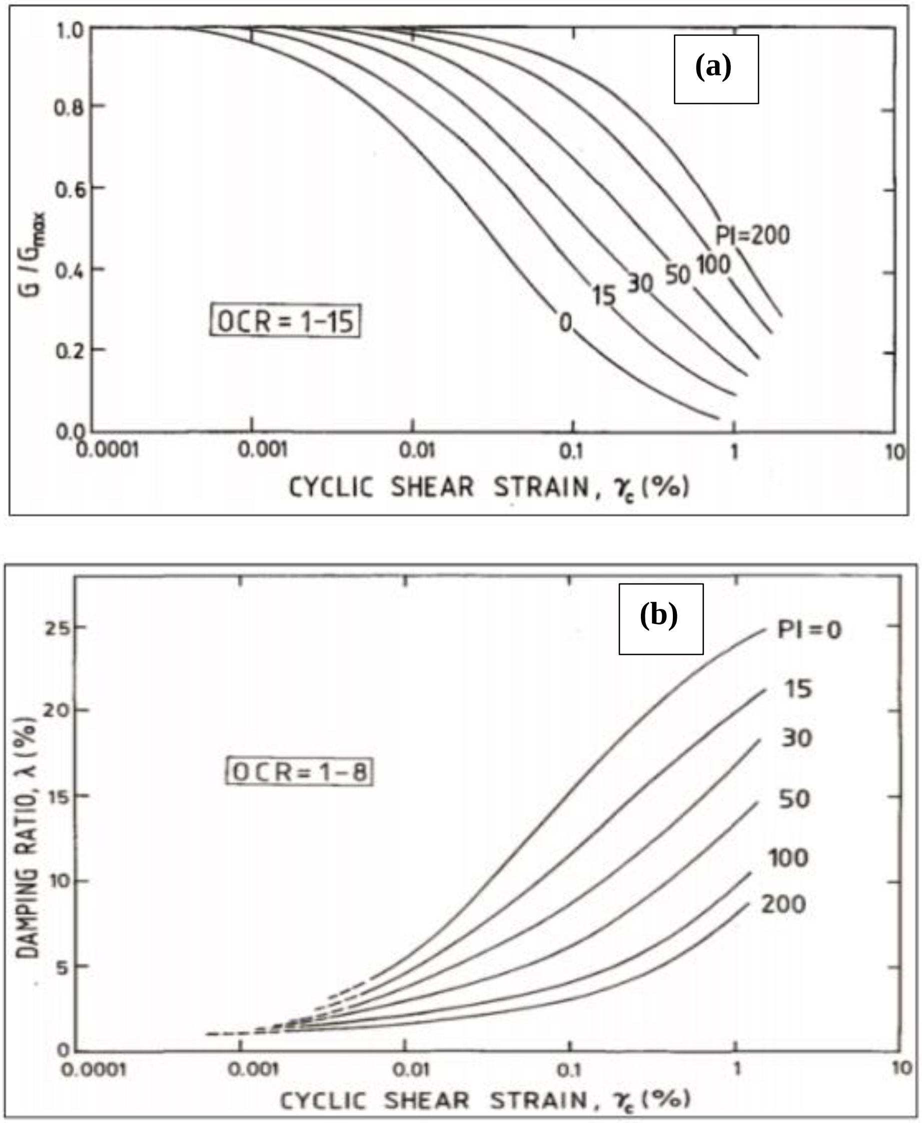


Figure 2.9 Variation of (a) modulus reduction ($G/G_{max}$ ) and (b) damping ratio values with PI (Kumar et al., 2013)

## 2.6. Related Research Works on Dynamic Properties of Stabilized Soil

**Ashango and Patra, (2013),** investigated the dynamic Properties of Stabilized Subgrade Clay Soil with Rice Husk Ash and Portland cement slag. According to the study the optimum mix was found to be 82.5 % soil+ 7.5 % Portland Slag Cement+10% Rice Husk Ash. Cyclic triaxial tests were conducted to investigate the variation of dynamic characteristics of stabilized clay soils such as shear modulus, G, damping ratio, D, and degradation index, with the number of cycles for different strain amplitudes ranging from 0.4 to 1% at a frequency of

0.5Hz and effective confining pressure of 100kPa.From the investigation damping ratios increase as the amplitude of shear strain increases, ranging from 7-9% to 14-19% for 0.4% to 1%, respectively and the shear modulus decreases with increasing strain amplitude. For the first 50 cycles, the modulus of the degradation index decreases at a rapid rate. They concluded that variation of dynamic properties of stabilized clay with strain amplitudes may be helpful for the design of subgrade clay soil.

**Dutta and Saride, (2016),** Investigated the influence of the addition of fly ash on the dynamic characteristics of expansive soils. A class C fly ash content of 5 to 20% with 5% intervals was used to treat moderately expansive soil having a free swell index of 50 %. A series of resonant column tests were performed to determine the small strain dynamic properties, shear modulus, and damping ratio, of the untreated and fly ash treated expansive soils at different curing periods. Results indicate that with an increase in fly ash content, the shear modulus increases, and the damping ratio of the expansive soil decreases. In addition, with the increase in the curing period, there is an increase in shear modulus and a decrease in damping ratio.

**Pei-Hsun and Sheng-Huoo,(2012)**, studied the dynamic properties of cement stabilized soil, slag cement stabilized soil, and cement with sodium silicate stabilized soil using a resonant column test. In this investigation, the effect of the amount of cement admixed, confining pressure magnitude, and shearing strain amplitude was considered. Test results show that the maximum shear modulus of cement stabilized soil increases as confining pressure increases, while the minimum damping ratio decreases. The relationship between maximum shear modulus and confining pressure, on the other hand, changes depending on the type of additive. The shear modulus of cement stabilized soil decreases as shearing strain increases, while the damping ratio increases. The findings also show that the cement with sodium silicate stabilized soil can withstand more shearing strain before stiffness degradation than other additives.

**Chiang and Chae, (1972),** in this study the dynamic shear modulus and damping parameters of 2 soils, uniform sand, and silty clay, treated with Type 1 portland cement are determined by the resonant column method. Cement content, confining pressure, shear-strain amplitude, and moisture content were the test variables investigated. According to the study, the use of cement to stabilize sand and clay under dynamic loading increases the stiffness of the soils and reduces deformation. In cement-treated cohesionless soils, the effect of confining pressure, which increases shear modulus and damping, is more apparent than in cohesive soils. The dynamic shear modulus is reduced and the damping is increased as the shear-strain amplitude increases. The moisture content of soils has a major influence on the dynamic properties of cement-treated cohesive soils but has not much influence on cement-treated cohesionless soils.

**Abyalew, (2022),** investigated the effect of sugarcane bagasse ash (SCBA) stabilization on the dynamic properties of expansive soil using a cyclic simple shear test. The investigation was conducted for the number of 40 cycles, with shear strain ranging from 0.01 to 3 %, and under axial stress of 100 to 400 kPa at a frequency of 1Hz. According to the study, the addition of SCBA improves both the static and dynamic characteristics of the soil. It is observed that the addition of the optimum amount of SCBA increases dynamic shear modulus and the damping ratio. The dynamic shear modulus increases as confining pressure increases, but decreases as the number of cycles and strain increases. On the other hand, the damping ratio decreases as the number of cycles increases. However, it increases with confining pressure and shear strain rate.

Table 2. 8: Summary of some related work on stabilization by Lime and dynamic properties of stabilized soil.

| Author | Title of the study | Types of material | Major findings |
|---|---|---|---|
| Gizachew, (2019) | Improving the GeotechnicalProperty of Expansive Soil through Marble Dust and Lime for Road Construction Projects. | Marble dust with (0%, 5%, 10%, 15%, 20%, 25%, and, 30%) and fixing lime content (1-3) % | ✓ Reduces PI, swelling, and OMC with an increase in MDD and CBR values.<br>✓ Combinations of 3% of lime and 30% of marble dust were found to be a percentage proportion that yielded maximum strength as well as minimum swelling potential and plasticity in the soil sample. |
| Feysel, (2017) | Stabilization of Highly Plastic Clay | Hydrated Lime (2%, 4%, and 6%,) | ✓ The maximum decrease in plasticity index, compression index, and |

| | | | |
|---|---|---|---|
| | Soil for the Impervious Core of Embankment Dam using Hydrated Lime (The Case of Gidabo Dam) | | permeability of soil with the addition of 6% lime.<br>✓ Concluded that hydrated lime can be a potential alternative to alter the undesirable characteristics of CH soil and make it a suitable core material for embankment dams. |
| Ashango and Patra, (2013) | Dynamic Properties of Stabilized Subgrade Clay Soil | Rice Husk Ash and Portland cement slag | ✓ The damping ratios increase as the amplitude of shear strain increases, and the shear modulus decreases with increasing strain amplitude.<br>✓ The variation of dynamic properties of stabilized clay with strain amplitudes may be helpful for the design of subgrade clay soil. |
| Dutta and Saride, (2016) | Dynamic properties of moderately expansive soil stabilized with class C fly ash | class C fly ash (0%, 5%, 10%, 15% and, 20%) | ✓ The shear modulus increases, and the damping ratio of the stabilized expansive soil decreases.<br>✓ With the increase in the curing period, there is an increase in shear modulus and a decrease in damping ratio. |
| Wengelawit, (2021) | Study on Dynamic Properties of Treated Expansive Soils | Ferric Chloride (0%, 0.5%, 1%, 1.5%, and 2%) | ✓ The shear modulus of treated soil decreases around 1.2 times that of untreated soil.<br>✓ The damping ratio of treated soil increases by around 30%.<br>✓ The resilient modulus of the soil specimen increases nearly by 14-25 % |

# CHAPTER THREE

# MATERIALS AND METHODS

## 3.1. Introduction

In this chapter, the materials used and methods adopted for the research are described along with their sources and their physical properties. The laboratory investigations on materials are carried out in AAiT geotechnical and material laboratory and Ethiopian Construction Design and Supervision Works Corporation (ECDSWC) geotechnical laboratory.

## 3.2. Description of the Study Area

Addis Ababa, the capital of Ethiopia, is located in the central highlands of Ethiopia, with an elevation of 2355 meters above means sea level. The geographical location of the city is approximately 8°58'50.2" N latitude and 38°45'27.9" E longitude. The city has a subtropical highland climate with a rainy season from June to September and a dry season from October through May. The mean annual rainfall over 33 years was found to be 1038.66 mm and the highest and lowest rainfall was recorded in the years 1977 and 2001 as 1487.3 mm and 771.3 mm respectively (Degife et al., 2018). In the 2007 census, the population was estimated to be 2,739,551 inhabitants.

Most of Addis Ababa's subgrade material is heavy clay and silty clay in nature, and a substantial area of the city subsoil is occupied by black cotton soil. The Summit-Figa road project is located in Addis Ababa in the Lemi Kura sub-city. From the previous literature, this area was covered by the extensive formation of expansive soil. The plasticity index (PI) of a different Addis Ababa city subgrade soil area is very high, ranging from 11.9 to 52.5%, and high liquid limit (LL) ranging from 45.5 to 97.5% (Tsion and Mindaye, 2021).

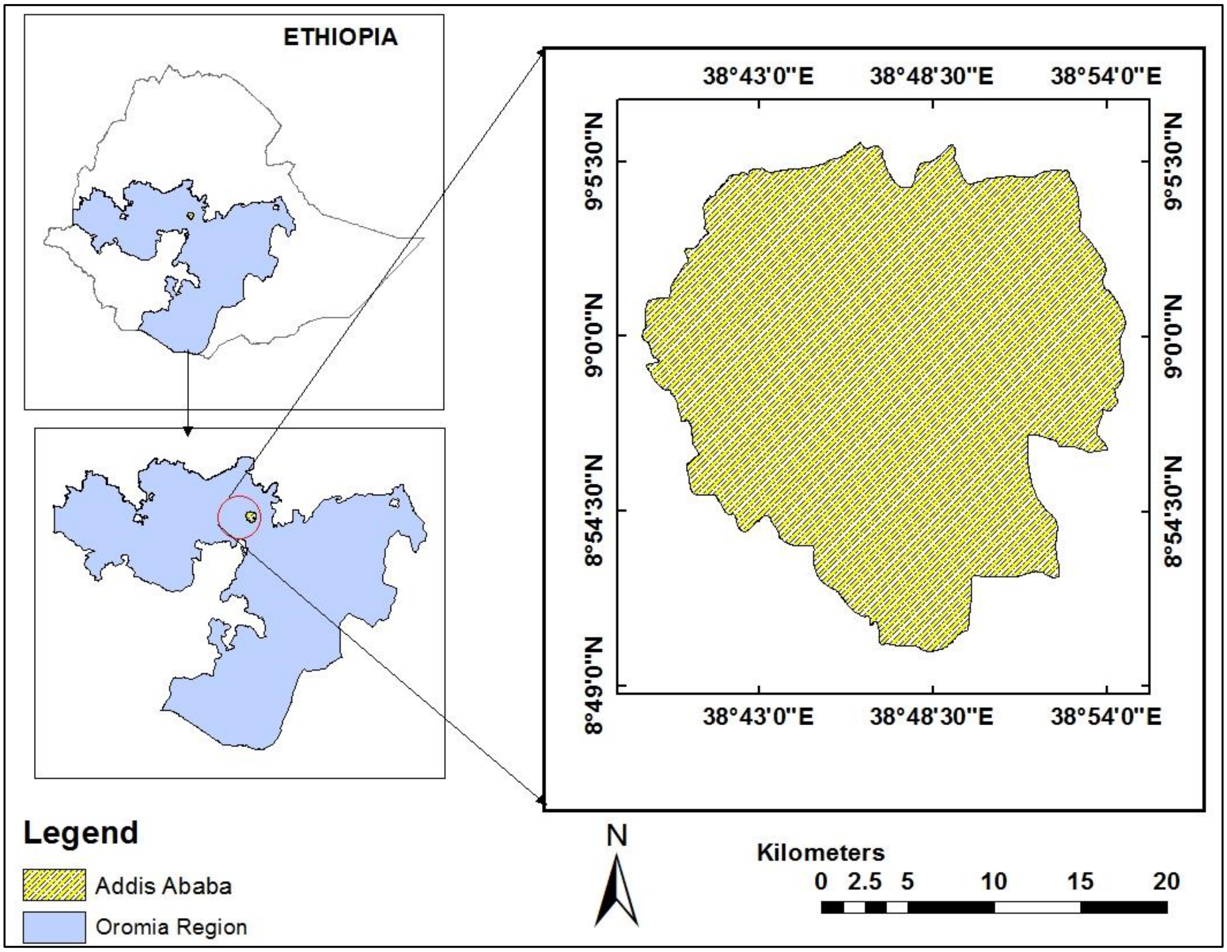


Figure 3.1:The location map of the study area

## 3.3. Materials

### 3.3.1. Expansive Subgrade Soil

The subgrade soil sample used for this research work was collected from Addis Ababa, Lemi Kura Sub-City, at the Summit-Figa road project. Disturbed soil samples were collected from two stations, at a depth of 1.5 m below the natural ground level. The soil is grayish-black in color and highly plastic clay.

### 3.3.2. Hydrated Lime

The hydrated lime used in this study was obtained from the Senkele lime factory, which is located in Ambo, Oromia. The Lime was stored in a cool and dry place away from weathering effects.

## 3.4. Test Programs

To meet the objectives of the research, field identification and laboratory tests were conducted to characterize the natural subgrade soil and to evaluate the effect of lime on static as well as dynamic properties of soil. The experimental program was divided into three sections.

i. The first test program was to identify, collect, characterize, and classification of the soil sample. Sieve analysis, Atterberg limits, Free swell, Moisture content, Specific gravity, Proctor compaction test, Unconfined compressive strength (UCS),and California Bearing Ratio (CBR) tests are among the laboratory experiments performed during this stage.
ii. In the second test program, the effect of lime on engineering properties of expansive soil sample was investigated by adding different percentages of lime i.e. 3%, 6%, and 9%, and altering different parameters such as the consistency limit of the soil, strength, and bearing capacity of the soil were analyzed. From this, the optimum amount of lime used to get maximum strength was determined.
iii. The third program includes investigating the dynamic properties of expansive subgrade soil treated with the lime. The results were compared with the dynamic properties of the natural soil to investigate the effect of lime treatment on the dynamic properties of expansive soils. To study the cyclic loading impact on the subgrade soil cyclic simple shear test is conducted at different strain values.

## 3.5. Method

### 3.5.1. Sample Preparation

The soil sample was air-dried and soil aggregates are broken up by a rubber-covered mallet. Then, the soil and hydrated lime are mixed manually to get a uniform mix ratio for each test. In this study, the percentage of lime required was determined by using the PH test method. According to ASTM D-6276, the PH test methods provide criteria for establishing the minimal soil-lime percentage, a minimum PH of 12.4 signifies the requirement for lime for soil stabilization. A series of tests were carried out by varying the lime percentage with soil and the lowest dosage of lime contents that gives soil lime mix a PH of 12.4 was achieved at 6% of lime was taken as the lime fixation. The investigation was carried out below, at, and above the lime fixation i.e 3%,6%, and 9% to determine the changes in the soil properties.

### 3.5.2. Grain Size Analysis Test

The grain-size analysis is carried out to determine the relative proportions of different grain sizes of soil that makes up a given soil mass. Wet sieve analysis is performed to determine the distribution of the coarser, larger-sized particles larger than 75 µm (retained on the No.200 sieve). The hydrometer analysis method is carried out to determine the distribution of the finer particle size smaller than 75 µ m (usually silt and clay). Grain size analysis was conducted for native soils according to ASTM D 422.

### 3.5.3. Moisture Content

The moisture content of the soil is defined as the ratio between the mass of water to the mass of the soil solid. The moisture or water content of a soil sample is an indicator of the amount of water present in the soil. The moisture content test was conducted in accordance with ASTM D2216 standard method.

### 3.5.4. Atterberg Limits Test

Atterberg limits tests were conducted for both natural soil and lime-treated expansive soil samples according to the procedure in ASTM D 4318. The test includes the determination of the liquid limits, plastic limits, and the plasticity index soil using the soil passing through a 425 µm (No. 40) sieve size. The liquid limit (LL) is the water content at which soil in a standard cup cut by a groove of standard dimensions will flow together for a distance of 13 mm when subjected to 25 shocks dropped from 10mm at a rate of two blows per second. The plastic limit (PL) is the water content, at which a soil can no longer be deformed by rolling into 3.2 mm diameter threads without crumbling.

### 3.5.5. Specific Gravity

Specific gravity is defined as the ratio of the mass of the unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of the samples was conducted according to ASTM D 854-83. The specific gravity of soils indicates how porous the soil is, how many voids it contains, as well as how saturated it is with water. This test was done employing a water pycnometer for oven-dried soil specimens that pass the 4.75mm (No.4) sieve.

### 3.5.6. Free Swell Index Test

The free swell test is one of the most frequently used simple tests to estimate the swelling Potential of expansive clay. As per IS 2720 testing procedure, the test is performed by pouring, 10gm of oven-dry soil passing a sieve size of 0.425mm (No. 40), into a 100 cm3 graduated jar filled with water. Samples are left undisturbed for 24 hours. Then the swelled Volume of the soil after the material settles (24hr) is measured. The volume difference between the final and initial volumes was calculated as a percentage of the initial volume.

### 3.5.7. Compaction Test

The compaction test includes the determination of the maximum dry density and the optimum moisture content for the natural soil and lime-treated soil. The test is conducted according to ASTM D-698 using soil samples that pass the 4.75mm (No.4) sieve. The maximum dry density is determined for both the natural and lime-treated mixture of about 2.5kg soil sample, by varying the moisture content. The sample is then compacted into 944 cubic centimeters in three layers of approximately equal mass with each layer receiving 25 blows. The moisture content of compacted soil is then determined by taking one small representative sample from the top, middle, and bottom. The values of the dry densities as obtained are plotted against their respective moisture contents. Then the optimum moisture content is determined as the corresponding value of moisture contents at maximum dry densities.

### 3.5.8. Unconfined Compressive Strength

ASTM D 2166-00 was used to conduct the unconfined compressive strength test. The sample was remolded with a diameter of 38mm and a height of 76mm, with optimum moisture content and dry density obtained from the standard compaction test for both untreated and lime-treated soil samples. After the samples were extracted from the mold they were wrapped with a plastic membrane and cured for seven days. A strain-controlled axial load was applied until maximum stress was obtained to get an approximate value of the strength of sample soils in terms of total stresses.

### 3.5.9. California Bearing Ratio Test

The CBR test measures the penetration resistance of soil under controlled moisture and density conditions. One-point CBR tests were carried out according to ASTM-D1883 for natural soil and lime-treated soil samples. 6kg of sample was prepared using optimum moisture content determined from standard compaction test for both treated untreated samples. The samples are then compacted in 2124 cubic centimeters mold in five layers with 56 blows from the standard compactor. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value of the soil sample.

### 3.5.10. One Dimensional Consolidation Test

Consolidation characteristics of the soils were determined to understand their stress history. The test was conducted according to the ASTM D2435 standard test method for onedimensional consolidation properties of soils using odometer apparatus. The soil sample was remolded at the optimum moisture content determined from the standard compaction test of size 50 mm in diameter and 20 mm high for both natural and lime-treated soil samples. . In this test the samples were placed in the consolidation ring then, the sample was allowed to swell under 7kpa setting pressure. Reading was taken immediately and the final reading was taken after the specimen stopped swelling. At this point, a standard consolidation test is conducted by applying incremental loads starting with 25kPa and ending with 1600kPa, and reading is taken for 24 hours.

### 3.5.11. Cyclic Simple Shear Test

The cyclic simple shear apparatus model 31-WF7500 was employed in this study. The apparatus uses an electronic reading system. An electronic reading system records lateral force and displacement, axial force and displacement. This electronic reading system is governed by the UTS004 software application program which has the functionality to perform consolidation and cyclic simple shear (Wykeham Farrance, 2003).

The cyclic simple shear is a plane strain machine. The shear strain is induced by horizontal movement at the bottom of the sample relative to the top. The width (or diameter) of the sample remains constant; therefore, any volume change could only be due to the vertical movement of the top platen. The system is designed to permit a sample to be consolidated, drained, and then sheared under constant volume conditions.

The cyclic simple shear tests can be conducted for a wider range of strain amplitude ($10^{-2}$ % to 5 %). The simple shear device is considered to be appropriate to evaluate the dynamic property of soils in the laboratory because it closely replicates the field loading condition due to the vertical propagation of shear waves during earthquake shaking (Yohannes,2015).

Figure 3.2: The cyclic shear testing apparatus

**Sample Preparation**

Pulverized soil samples that were sieved on sieve No.4 (4.75mm) were remolded at the field density and natural moisture. Then cylindrical 20mm height and 70mm diameter samples were prepared and placed in a rubber membrane, which was mounted on the bottom plate of a cyclic simple shear machine and secured with brass rings. These rings restrain the sample through the consolidation stage and permit lateral displacement during a cyclic shear stage.

### Consolidation Stage

The consolidation stage is the application of static axial loading stress to the specimen while the lateral loading (shear) axis is held stationary. Axial stress and specimen displacements (axial and lateral) are measured over time and logged by the system. Logged data is also displayed to the operator in the form of charts and tables as the test stage proceeds. The consolidation stage is manually terminated by the operator once the rate of vertical strain becomes less than 0.05% per hour. The axial stresses (pressures) during consolidation is varied from 50 kPa to 200 kPa by keeping the lateral loading constant.

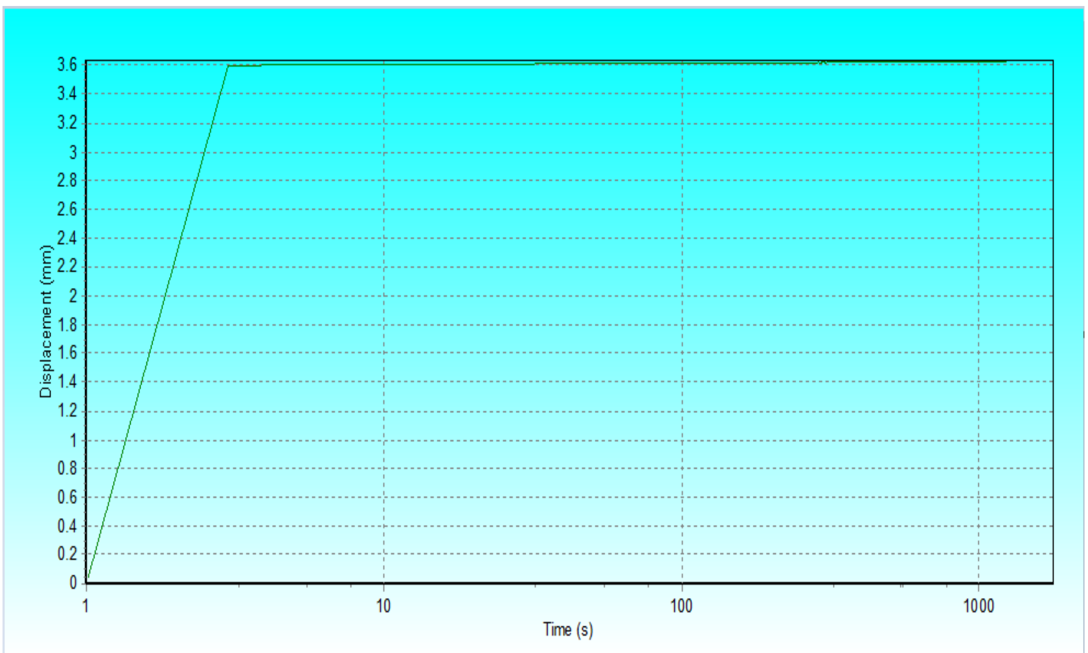


Figure 3.3: Consolidation stage at 100kPa axial stress and 1% shear strain amplitude

### Cyclic Simple Shear Stage

The cyclic shearing stage took place after consolidation was completed, by applying a lateral cyclic shear force of specified amplitude to the specimen, while the vertical height of the specimen is maintained as specimen displacements are measured for each loading cycle. Both axial and lateral force and specimen displacement are measured for each loading cycle. The lateral cyclic shear force tends to slide the rings over each other. However, the volume of the specimen remains unchanged. 50 sample points of measured data are obtained and then displayed in the form of wave shapes, charts, and tables. The loading cycle shape is selectable from predefined functions by the operator, but may also be a user-generated shape (Seinwill, 1984).

For this study during the consolidation phase, axial stresses (pressures) of 50 kPa,100 kPa, and 200 kPa were used, to produce in-situ stress conditions by applying vertical stresses, which simulate the loads from overburdened material located over the soil sample. The strain rate value ranging from 0.01% to 1 % was used. The cyclic behavior of the soil is relatively insensitive to the frequency of the applied cyclic load within the range of 0.5 to 2 Hz (Yohannes,2015). Thus for this study, a sinusoidal wave with a 1Hz frequency was used. The selected shear strain and corresponding amplitude are shown in Table 3.1.

The shear strain and the corresponding amplitudes are related according to the following Equation 3.1.

$$\text{Shear strain}(\%) = \frac{\text{Amplitude (A)}}{\text{Height of sample(H)}} * 100(\%) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

Table 3.1:Applied shear strain with corresponding cyclic amplitude.

| Shear strain (%) | Amplitude (mm) |
|---|---|
| 0.01 | 0.002 |
| 0.1 | 0.02 |
| 0.4 | 0.08 |
| 1 | 0.2 |

# CHAPTER FOUR

# RESULTS AND DISCUSSIONS

## 4.1. Introduction

In this chapter, laboratory test results are presented and their analysis is briefly discussed. The relevant engineering characteristics of the soil are analyzed both for natural and lime-treated soil samples separately. The tests include sieve analysis, Atterberg limits, specific gravity determination, free swell, moisture density relationship (compaction), unconfined compressive strength test (UCS), California bearing ratio (CBR), one- dimensional consolidation, and Cyclic Simple Shear Test. The specimen was cured for 7 days for compressive strength and cyclic simple shear test.

## 4.2. Properties of Natural soils

The results of tests carried out for the determination of natural soil properties and for identifying weaker soil for pit 1 and pit 2 are presented in Table 4.1.

Table 4.1: Properties of natural soils

| Test Type | Soil Test Results | |
|---|---|---|
| | Pit 1 | Pit 2 |
| **Sieve and Hydrometer Analysis** | | |
| Clay% | 72.61 | 66.78 |
| Silt% | 21.99 | 23.72 |
| Sand % | 5.3 | 9.4 |
| Gravel % | 0.1 | 0.1 |
| Percent Passing No. 200 sieve | 94.60 | 90.50 |
| **Atterberg Limit** | | |
| Liquid Limit% | 95.10 | 88.00 |
| Plastic Limit% | 37.60 | 36.15 |
| Plastic Index% | 58.00 | 52.00 |
| **Natural Moisture Content (%)** | 31.75 | 28.31 |
| **Specific gravity** | 2.70 | 2.72 |
| **Free swell %** | 110.00 | 80.00 |

| **standard Compaction** | | |
|---|---|---|
| MDD (g/cm$^3$) | 1.30 | 1.33 |
| OMC % | 36.17 | 32.02 |
| **Unconfined compressive strength(KPa)** | 92.23 | 113.50 |
| **CBR %** | 1.74 | 1.85 |

According to (ASTM-D422, 1998) the grain size distribution of the soils larger than 0.075 mm (No. 200 sieves) is determined by wet sieving and particle sizes smaller than 0.075 mm are determined by the hydrometer test. The results from the two methods were integrated, and the particle size distribution curves are illustrated in Figure 4.1. According to the particle size distribution curves for soil samples from Pit 1 and Pit 2, about 94.60 percent and 90.50 percent of the soil passes through the No. 200 sieve, respectively. The values of liquid limit, plastic limit, and plasticity index of the soil sample were 95.10 %,37.60%, and 58 % for pit 1, and 88.00%,36.15%, and 52% for pit 2 respectively.

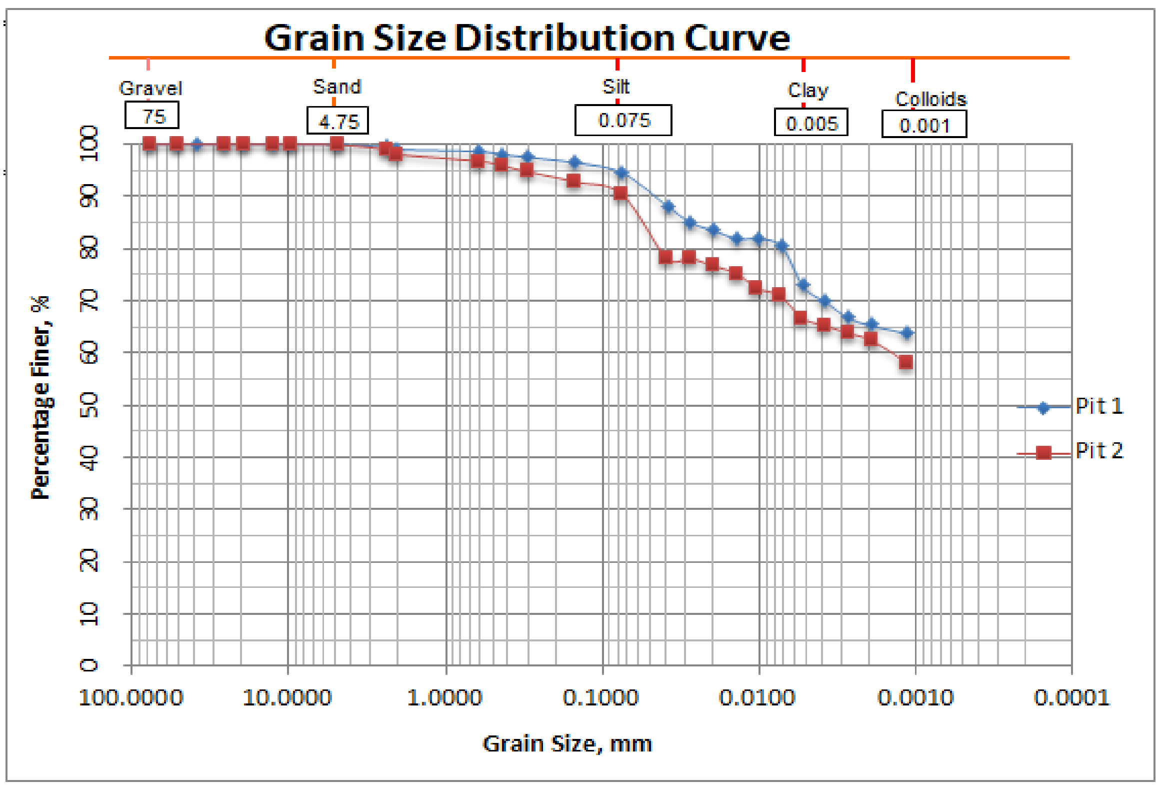


Figure 4.1: Grain size distribution curve for pit 1 and pit 2

As per the AASHTO soil classification system, both the soil samples from Pit 1 and pit 2 lays under group A-7-5, which have fair to poor subgrade material properties and CH (high plastic clay) as per the USCS classification system. The plastic index values of pit 1 and pit 2 are 58 % and 52% respectively which is higher than the ERA, 2013 standard requirement; i.e. a plasticity index of less than 30%. Therefore, soil samples show an expansive property. The 110% and 80% free swell index and the CBR value of 1.74% and 1.85 % for pit 1 and pit 2 soil samples respectively have indicated that the soils are expansive soil since their free swell index is greater than 50% and the CBR value less than 5% when compared with ERA requirements. Thus without any proper treatment methods, these soils are unsuitable for construction due to their low load-bearing capacity and significant swelling potential.

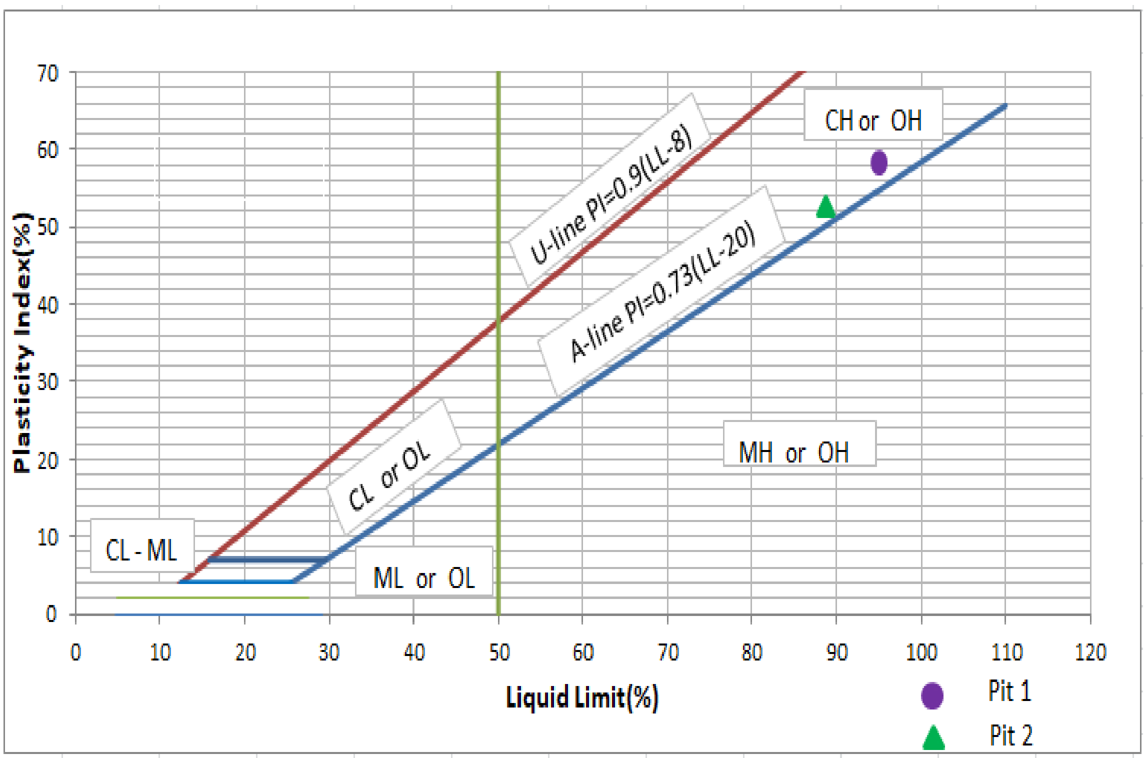


Figure 4.2:Soils classification according to USCS system

From the above-discussed test result, since pit 1 is weaker and more expansive than test pit 2, a soil sample taken from pit 1 was used to evaluate the effect of lime on the strength and dynamic properties of expansive subgrade soil.

## 4.3. Effect of Lime on Engineering Property

### 4.3.1. Effect of Lime on Atterberg Limits

The test results of different percentages of lime effect on Atterberg limit (LL, PL, PI) are shown in Table 4.2. The general trend shows that with lime treatment, LL and PI decrease while PL increases. For 0 % to 9 % of lime, the liquid limit decreased from 95.10% to 62.86 %, while the plastic limit increased slightly from 37.60 % to 42.66 %, resulting in a reduction in the plasticity index from 58.00 % to 20.20%. A similar trend was observed by Keybondori and Abdi (2021), Reshid (2014), and Agarwal and Jain, (2016).

Table 4.2: Test results of Atterberg limits for varying percentages of Lime

| Percentage of lime (%) | LL (%) | PL (%) | PI (%) |
|---|---|---|---|
| 0 | 95.10 | 37.60 | 58.00 |
| 3 | 81.26 | 39.14 | 42.12 |
| 6 | 68.58 | 41.40 | 27.18 |
| 9 | 62.86 | 42.66 | 20.20 |

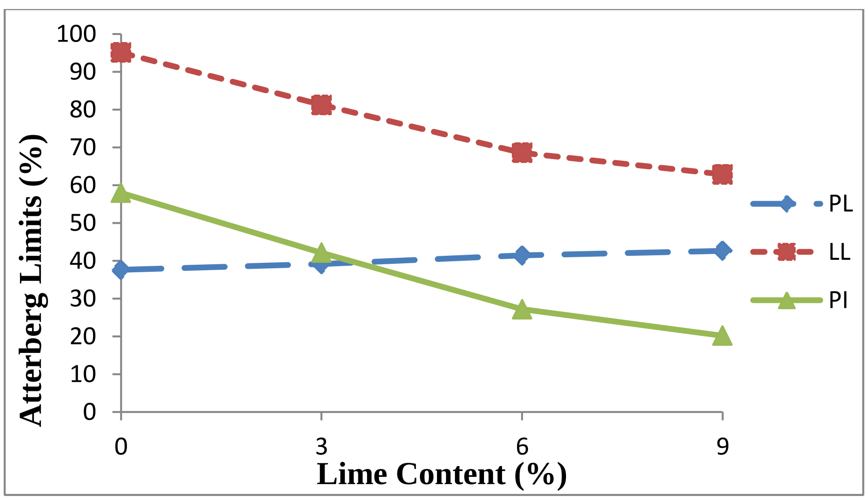


Figure 4.3: Variation of LL, PL, and, PI for different percentages of Lime

The reduction in plasticity index is related to the initial reaction involving cation exchange by replacement of ions, leading to a decrease in the thickness of the double water layer surrounding the clay particle resulting in a significant change in plasticity characteristic of the soil. The reduction in plastic index improves soil workability by reducing moisture absorption.

### 4.3.2. Effect of Lime on Free Swell Index

The result shows that with the addition of lime, the free swell index values were reduced from 110 % to 84 %, 48%, and 40 %, with the addition of lime for 3%, 6%, and 9% respectively, indicating a change in the surface chemistry of the clay particles which in turn reduces the water absorption capacity.

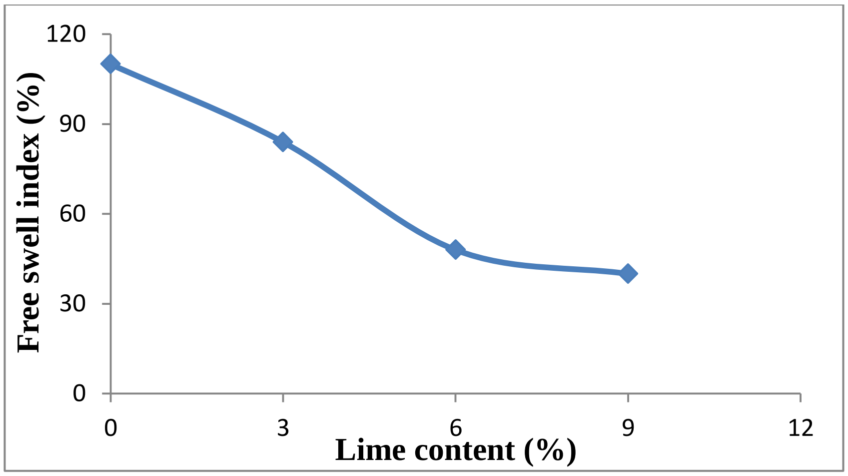


Figure 4.4:Reduction in the free swell index for different percentages of Lime

### 4.3.3. The Effect of Lime on Moisture-Density Relationship

The moisture density relationship of the soil samples was determined using a standard proctor compaction test. The effect of lime on the maximum dry density and optimum moisture content of the natural subgrade soil is shown in Figure 4.5. The results of the compaction tests showed that the addition of lime resulted in an improvement in the characteristics of the natural soil. MDD decreased from 1.30 g/cm$^3$ to 1.19 g/cm$^3$ and the OMC increased from 36.17% to 43.5% with an increase in lime content from 0% to 9%. Similar behavior was observed in other experimental work on the stabilization of soil with hydrated lime (Rashid 2014;Leite et.al. 2016).

The reduction in maximum dry unit weight may be the immediate reaction between soil and lime, which is represented by flocculation and agglomeration. Furthermore, the higher pH environment in the treated soil altered the surface charge distribution in clay soil particles, leading to an increase in particle layer repulsion. This, along with changes in the particle size distribution, caused a decrease in MDD. The increases in OMC with increasing lime content may be due to an increase in fine fraction and the hydration of lime (Amadi and Okeiyi, 2017). The increase in OMC and associated decrease in MDD of the soil has the advantage of allowing compaction with wet soil much easier. As a result, there is less of a need to dry the soil before compaction in the field to reduce moisture content (Meron and Samuel 2014).

Table 4.3: Effect of Lime on Moisture-Density Relations

| Percentage of lime(%) | 0 | 3 | 6 | 9 |
|---|---|---|---|---|
| Maximum dry density (g/cm3) | 1.30 | 1.27 | 1.23 | 1.19 |
| Optimum moisture content (%) | 36.17 | 37.80 | 40.10 | 43.50 |

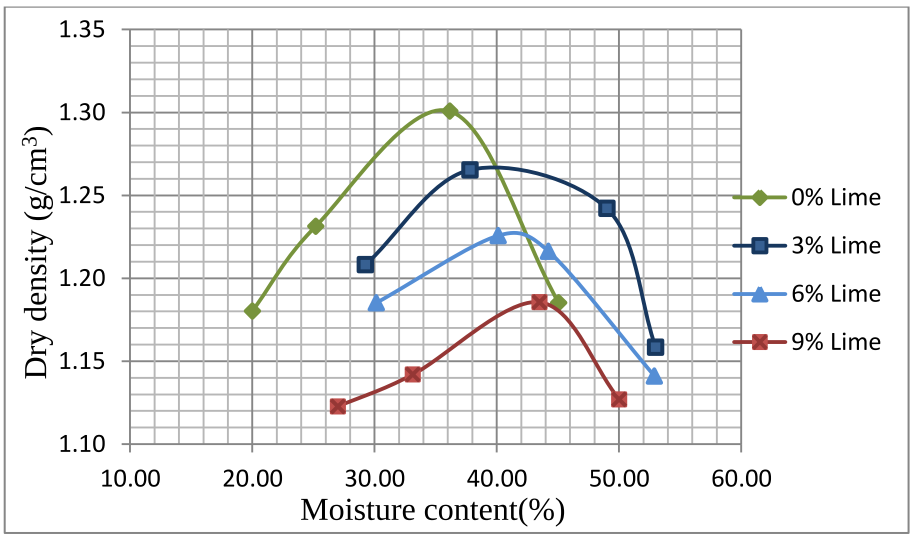


Figure 4.5: Effect of Lime on Moisture-Density relationship

### 4.3.4. The Effect of Lime on Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) test is an unconsolidated and undrained load test performed for determining the unconfined compressive strength of cohesive soil samples. UCS is taken as the maximum load attained per unit area during loading. The UCS test was conducted on the natural soil and on the seven days of cured treated soil samples, which were blended with different ratios of lime. The tests were carried out according to ASTM Standard D2166. For all combinations of lime and soil mixtures, initially, the stress gradually increases with an increase in axial strain. And after reaching the maximum axial stress, it decreases as the axial strain increases. The effect of lime on the unconfined compressive strength values of the soil lime mixtures is shown in Figure 4.6.

The unconfined compressive strength of the soil generally increases with the increase of lime content up to 6%, after 6% of lime stabilized specimen causes a reduction in UCS. Thus, the UCS value of the soil increased from 92.23 Kpa for the untreated soil to 274.75 Kpa for the specimens mixed with 6% lime, at which the maximum value of strength was attained. Bell (1996) suggested that since lime has neither considerable friction nor cohesion, an excess amount acts as a lubricant to the soil particles and hence decreases the strength. A similar trend was observed in previous literature which study soil stabilization using lime (Harish, 2017).

The increased shear strength of the stabilized soil may be due to the phenomenon of cation exchange between the negatively charged clay particles and the lime, as well as the flocculation–agglomeration mechanism which produces immediate improvements in soil properties. Furthermore, pozzolanic reactions occur between calcium ions and the clay minerals silica and alumina resulting in the formation of cementitious products, which are also responsible for the increased strength (Amadi and Okeiyi, 2017).

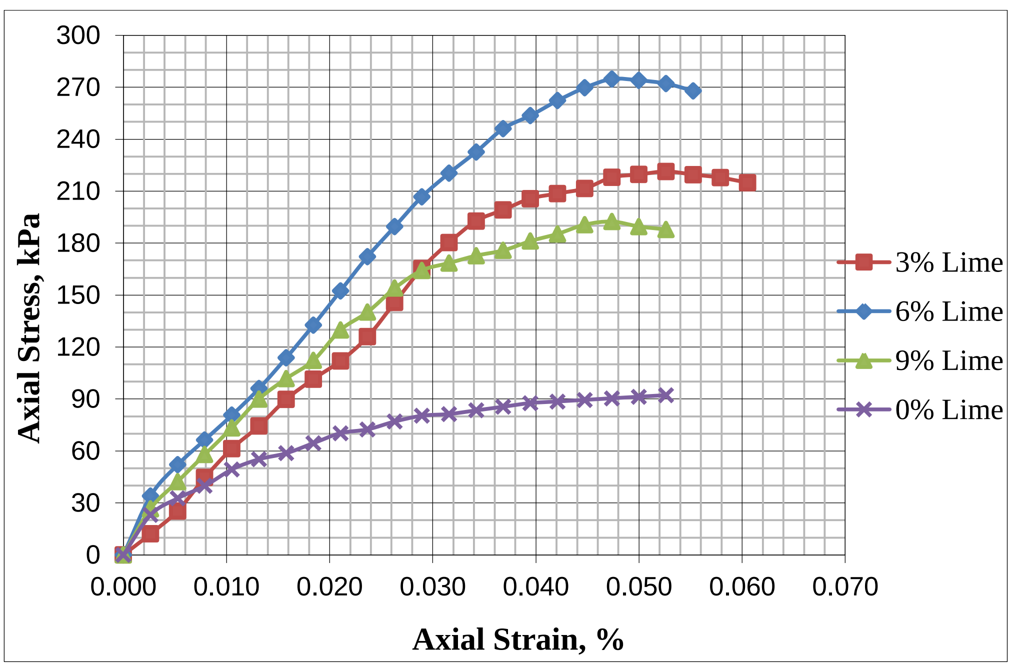


Figure 4.6:Variation of axial stress and axial strain with different content of lime

#### 4.3.5. The Effect of Lime on CBR

The California Bearing Ratio Test (CBR Test) is a penetration test for evaluating the bearing capacity of subgrade soil for the design of flexible pavement. The results of the CBR value for the 96-hour soaked condition of the soil-lime mix are shown in Figure 4.7.

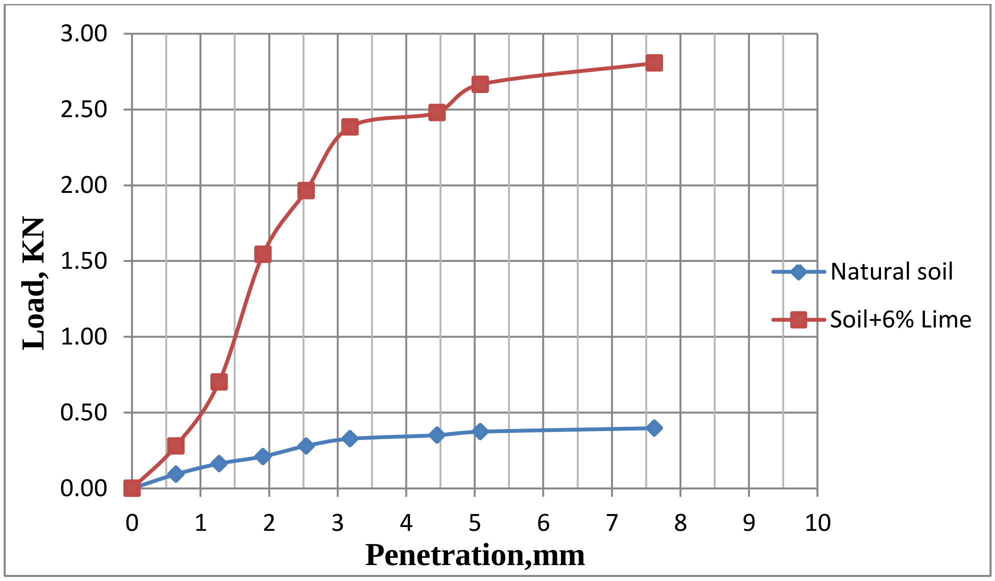


Figure 4.7: Load versus Penetration curve of natural and stabilized soil

The CBR value of the natural soil was 1.74% for soaked conditions. When 6% lime was added to the soil, the soaked CBR values increased to 14.88%. The percentage swell of the soil also reduced from 5.86% to 1.87% for the addition of 6% lime. The pozzolanic reactions between silica and alumina from the soil and lime to form different types of cementing agents are the main cause of the increase in strength of lime-stabilized soil. Therefore, according to ERA Pavement Design Manuals,(2002) the lime-treated soil changed the quality of the subgrade material from very poor (0-3%) to fair (7-20%) CBR value and full filled the specification as subgrade material. Detailed test results were given in Appendix B.

## 4.4. Laboratory Investigation of Dynamic Soil Properties

The data obtained from the Cyclic simple shear test gives raw data from 50 data points for single cyclic loading in the form of Microsoft excel. This raw data include lateral displacement and lateral force from which shear stresses and shear strains are evaluated, and then dynamic soil parameters are calculated.

The hysteresis loop is produced by plotting shear stress against shear strain. It is used for the calculation of the shear modulus and damping ratio for each cycle. The hysteresis loop of untreated soil at 1 % strain and under 100 kPa axial pressures of the first 50 cycles is shown in Figure 4.8. The shear stress is calculated simply by dividing the shear force by the area of the specimen base (3848.45 $mm^2$), and the shear strain is calculated by dividing the shear displacement by the specimen height after consolidation (<20mm).

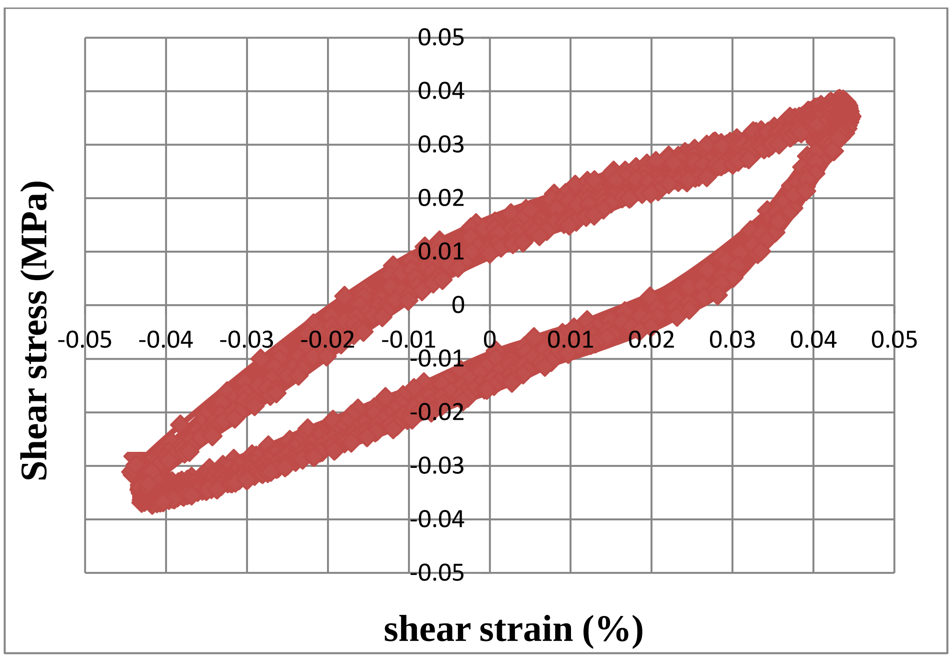


Figure 4.8: Hysteresis loop untreated soil at 1 % strain and under 100 kPa axial pressure of the first 50 cycles.

To simulate repeated loading sinusoidal loading cycle has been selected. In repeated loadings like traffic, the number of significant cycles is likely to be more than 1000 (Ishihara, 1997). The sinusoidal wave for a 1 % strain of 100 kPa and 1Hz for a single cycle is shown in Figure

4.9. For all practical purposes, the values determined at the $5^{th}$ cycle are likely to provide reasonable values (Das 2006).

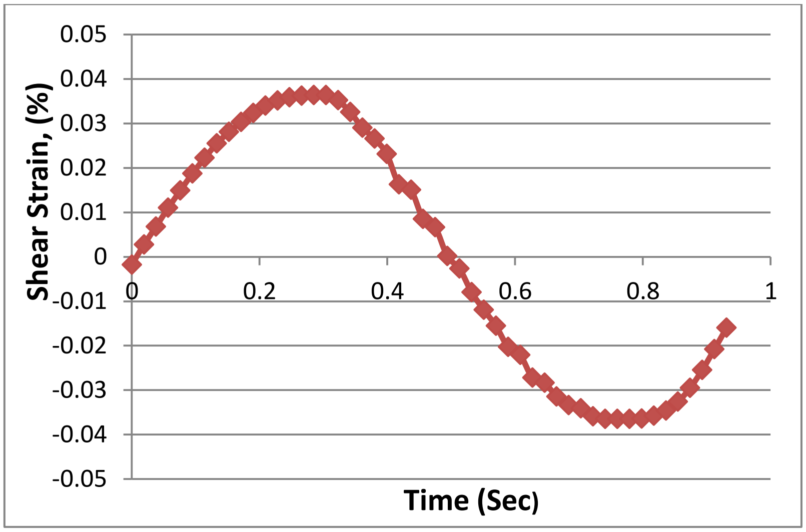


Figure 4.9: Sinusoidal wave shapes for 1 % strain of 100 kPa and 1 Hz for a single cycle

### 4.4.1. Determination of Shear Modulus and Damping Ratio of Untreated Soil

The values of shear modulus and damping ratio were computed from a hysteresis loop using the raw data obtained from the cyclic simple shear test for each cycle loading. The hysteresis loop shear stress and shear strain are used to calculate the shear modulus and damping ratio. The shear modulus and Damping ratio of the hysteresis loop can be calculated as shown in Table 4.4.

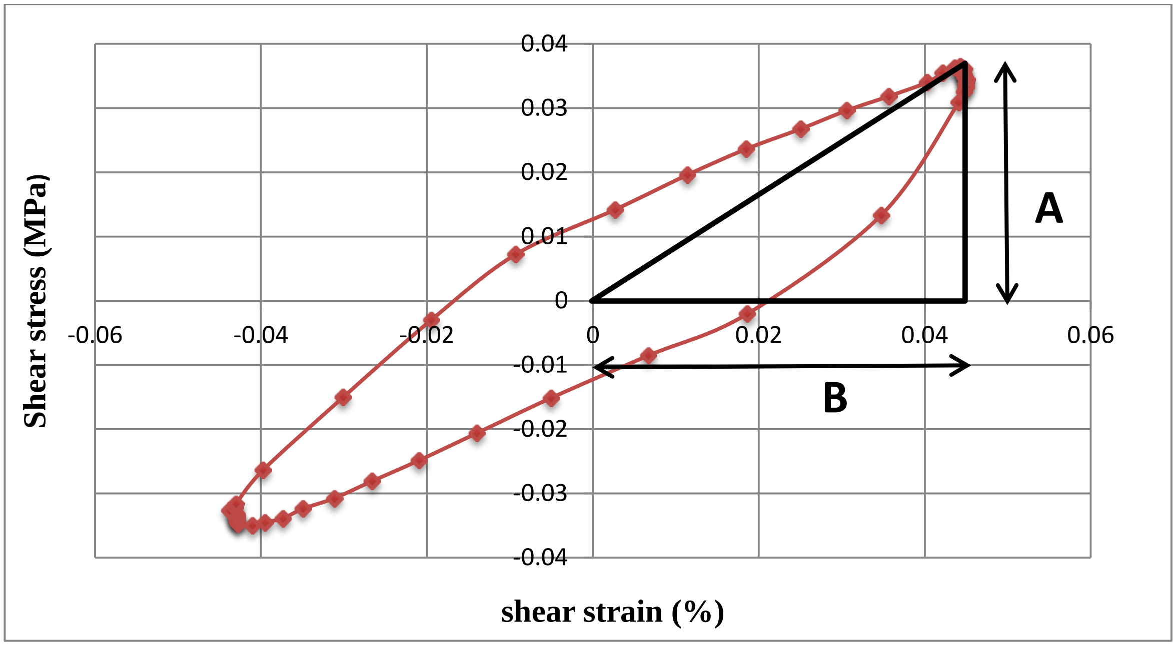


Figure 4.10: Hysteresis loops of natural soil under 100 kPa axial pressure and 1% cyclic shear strain of 5th cycle

Table 4.4: Typical calculation for shear modulus and damping ratio of natural soil under 1 % shear strain and 100 kPa axial stress at 5th cycle

| Computation of Shear modulus, G | | Computation of Damping Ratio, D | |
|---|---|---|---|
| $\tau_{max}$=A | 0.036517 | Area of loop = $\frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$ | 0.002249 |
| $\tau_{min}$ | -0.034760 | Area of $\Delta = \frac{1}{2} A * B$ | 0.000823 |
| $\tau_{max}$ - $\tau_{min}$ | 0.071277 | Damping ratio, D (%)= $\frac{\text{Area of loop}}{4*\pi*\text{Area of }\Delta}$ | 21.76 |
| $\gamma_{max}$ | 0.045049 | | |
| $\gamma_{min}$=B | -0.043760 | | |
| $\gamma_{max} - \gamma_{min}$ | 0.088809 | | |
| G (MPa) $= \frac{\tau_{max}-\tau_{min}}{\gamma_{max}-\gamma_{min}}$ | 0.80 | | |

Table 4.4 shows the shear modulus and damping ratio of natural soil with 1 % shear strain, and 100 kPa axial stresses at the $5^{th}$ cycle. The shear modulus and damping ratio for various shear strain amplitudes were calculated using the same procedure.

### 4.4.2. Determination of Shear Modulus and Damping Ratio of Treated Soil

According to the literature, different types and amounts of admixture used for soil stabilization affect the shear modulus and damping ratio of the soil. The optimum amount of hydrated lime to stabilize the expansive subgrade soil was determined from the static load test which is 6% for this study. The remolded sample was prepared using the optimum moisture content and maximum dry density for the 6 % lime-treated soil and natural soil sample. Cyclic shear test for treated soil was conducted on a 7-day cured specimen. The shear modulus and damping ratio computation methods for treated and untreated soil were the same. Table 4.5 shows the typical calculation for shear modulus and damping ratio for treated soil.

Table 4.5:Typical calculation for shear modulus and damping ratio of treated soil under 1 % shear strain and 100 kPa axial stress at 5th cycle

| Computation of Shear modulus, G | | Computation of Damping Ratio, D | |
|---|---|---|---|
| $\tau_{\max} = A$ | 0.045745 | Area of loop = $\frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$ | 0.002023 |
| $\tau_{\min}$ | -0.043797 | Area of Δ = $\frac{1}{2}A * B$ | 0.000792 |
| $\tau_{\max}$ - $\tau_{\min}$ | 0.089542 | Damping ratio, D (%) = $\frac{\text{Area of loop}}{4*\pi*\text{Area of }\Delta}$ | 20.33 |
| $\gamma_{max}$= B | 0.034612 | | |
| $\gamma_{min}$ | -0.032188 | | |
| $\gamma_{max} - \gamma_{min}$ | 0.066800 | | |
| G (MPa) $= \frac{\tau_{\max}-\tau_{\min}}{\gamma_{\max}-\gamma_{\min}}$ | 1.34 | | |

The same procedure was used to compute the shear modulus and damping ratio of both natural and treated soil for different shear strain amplitudes and is given in Tables 4.6 and 4.7.

Table 4.6: Shear modulus and damping ratio values for untreated soil

| **Untreated Soil** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Axial Stress, 50 kPa | | | | | | | | |
| Strain, (%) | 0.01 | 0.1 | 0.4 | 1 | 0.01 | 0.1 | 0.4 | 1 |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 2.54 | 1.8 | 0.96 | 0.75 | 19.23 | 21.02 | 22.21 | 22.80 |
| 10 th | 4.12 | 2.87 | 1.34 | 1.08 | 18.67 | 20.68 | 21.64 | 22.33 |
| 50 th | 5.62 | 4.25 | 2.05 | 1.73 | 16.72 | 18.64 | 20.35 | 20.74 |
| 100 th | 6.1 | 4.92 | 2.74 | 2.11 | 14.65 | 16.81 | 19.27 | 19.69 |
| Axial Stress, 100 kPa | | | | | | | | |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 3.05 | 2.25 | 1.12 | 0.8 | 16.72 | 18.98 | 21.02 | 21.76 |
| 10 th | 4.62 | 3.63 | 1.89 | 1.26 | 14.61 | 17.25 | 20.01 | 21.07 |
| 50 th | 6.32 | 5.65 | 4.15 | 3.52 | 12.12 | 15.14 | 18.54 | 19.91 |
| 100 th | 7.18 | 6.26 | 5.38 | 4.27 | 11.07 | 13.83 | 15.98 | 18.00 |
| Axial Stress, 200 kPa | | | | | | | | |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 4.54 | 3.21 | 1.64 | 0.98 | 14.08 | 17.21 | 19.00 | 20.54 |
| 10 th | 5.83 | 4.36 | 2.31 | 1.39 | 13.83 | 16.78 | 18.71 | 20.03 |
| 50 th | 8.75 | 7.78 | 6.12 | 4.74 | 12.1 | 15.10 | 17.55 | 18.14 |
| 100 th | 9.46 | 8.66 | 6.85 | 4.98 | 11.04 | 13.20 | 15.80 | 17.68 |

Table 4.7: Shear modulus and damping ratio values for treated soil

| **Treated Soil** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Axial Stress, 50 kPa | | | | | | | | |
| Strain, (%) | 0.01 | 0.1 | 0.4 | 1 | 0.01 | 0.1 | 0.4 | 1 |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 3.91 | 3.11 | 1.78 | 1.23 | 17.00 | 18.86 | 20.93 | 21.72 |
| 10 th | 6.38 | 4.78 | 2.27 | 1.91 | 14.23 | 16.59 | 18.33 | 19.9 |
| 50 th | 7.83 | 5.91 | 4.14 | 3.49 | 13.79 | 15.76 | 16.84 | 17.56 |
| 100 th | 8.56 | 6.78 | 5.13 | 3.82 | 11.93 | 13.90 | 14.39 | 15.15 |
| Axial Stress, 100 kPa | | | | | | | | |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 4.81 | 3.57 | 1.92 | 1.34 | 13.56 | 16.83 | 19.75 | 20.33 |
| 10 th | 7.50 | 5.61 | 3.13 | 2.27 | 12.07 | 14.59 | 18.28 | 19.02 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 50 th | 9.59 | 7.28 | 5.18 | 4.63 | 9.84 | 11.29 | 14.34 | 17.01 |
| 100 th | 10.35 | 8.14 | 6.04 | 5.42 | 8.28 | 9.85 | 13.14 | 14.53 |
| Axial Stress, 200 kPa | | | | | | | | |
| Cycle No. | G(Mpa) | | | | D(%) | | | |
| 5th | 6.26 | 4.93 | 2.93 | 2.39 | 11.24 | 14.68 | 17.04 | 17.96 |
| 10 th | 8.56 | 6.53 | 3.85 | 3.14 | 10.88 | 13.45 | 15.98 | 16.64 |
| 50 th | 11.82 | 8.83 | 6.9 | 6.53 | 9.81 | 11.27 | 13.4 | 13.98 |
| 100 th | 12.27 | 10.15 | 7.64 | 6.84 | 8.13 | 9.53 | 12.27 | 12.96 |

### 4.4.3. Variation of Shear Modulus and Damping Ratio with Strain

The shear modulus and damping ratio of cyclically loaded soils are strongly influenced by the shear strain levels. The variation of shear modulus and damping ratio with shear strain has been plotted for both natural and lime-treated clay soil for various shear strain ranges. From the figure below, it is observed that for both natural and treated soils the shear modulus decrease as the shear strain increase, while the damping ratio increases as the shear strain increase, which is the predicted behavior of soil under dynamic loading because of the nonlinear properties of soils. This is due to as shear strain increases, the soil's ability to resist deformation decreases, and more energy are dissipated, resulting in high deformation. Due to energy dissipation during soil deformation, the damping ratio increases as shear strain increases.

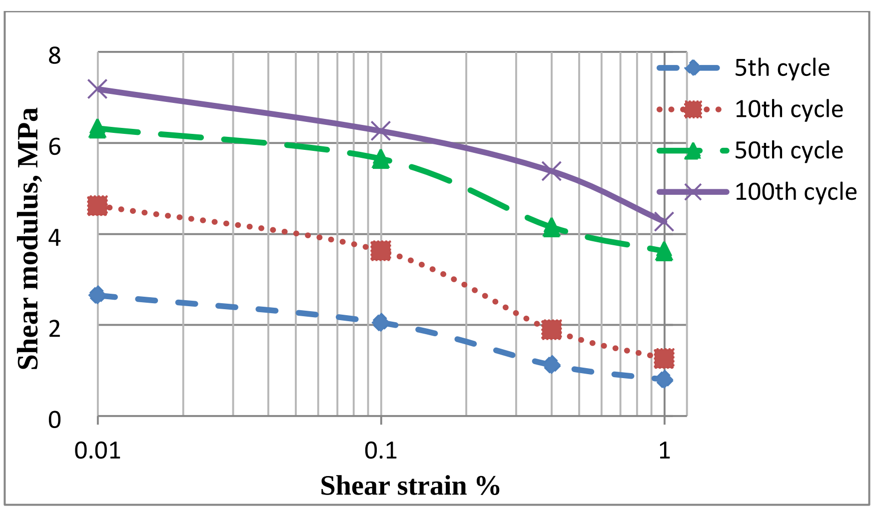


Figure 4.11: Variation of shear modulus with the shear strain of natural soil under 100 kPa axial stress

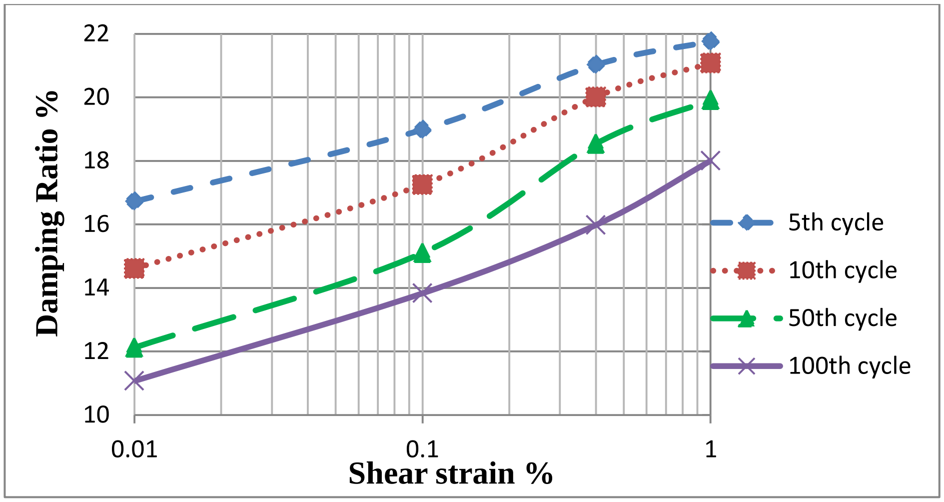


Figure 4.12: Variation of damping ratio with the shear strain of natural soil under 100 kPa axial stress

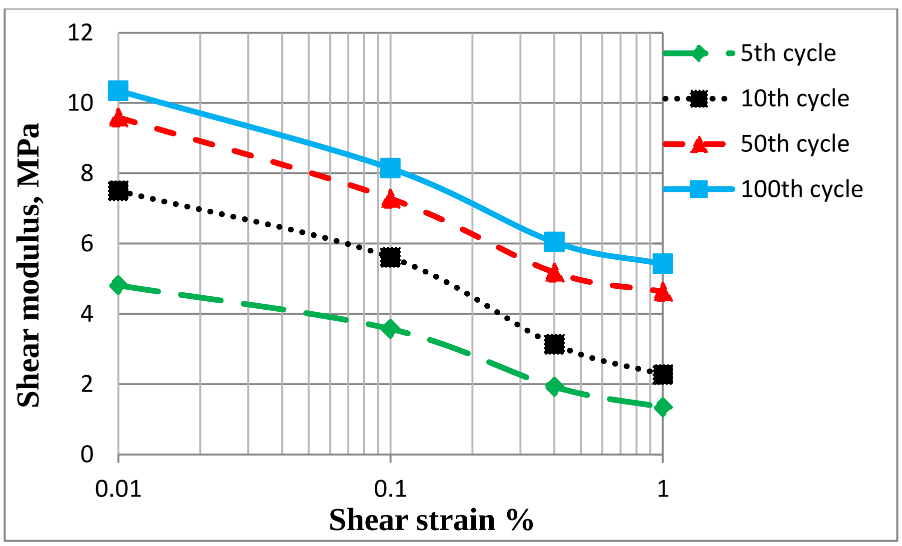


Figure 4.13: Variation of shear modulus with the shear strain of treated soil under 100 kPa axial stress

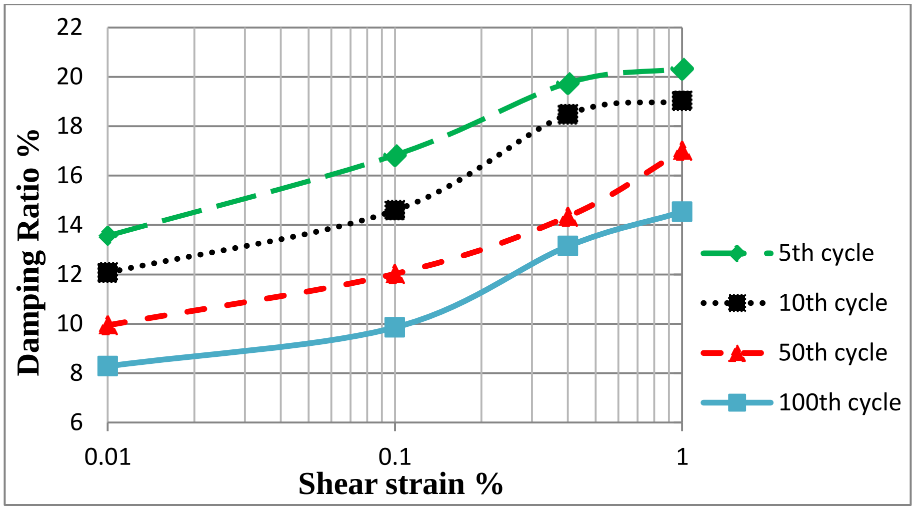


Figure 4.14: Variation of damping ratio with the shear strain of treated soil under 100 kPa axial stress.

### 4.4.4. Variation of Shear Modulus and Damping Ratio with Number of Cycle

The relationship between shear modulus and damping ratio with the number of cycles of natural and lime-treated soil is plotted for a different strain of 100 kPa axial stress shown in Figures 4.15to 4.18. From the graphs, it was observed that the shear modulus increases, and the damping ratio decrease with an increasing number of cycles.

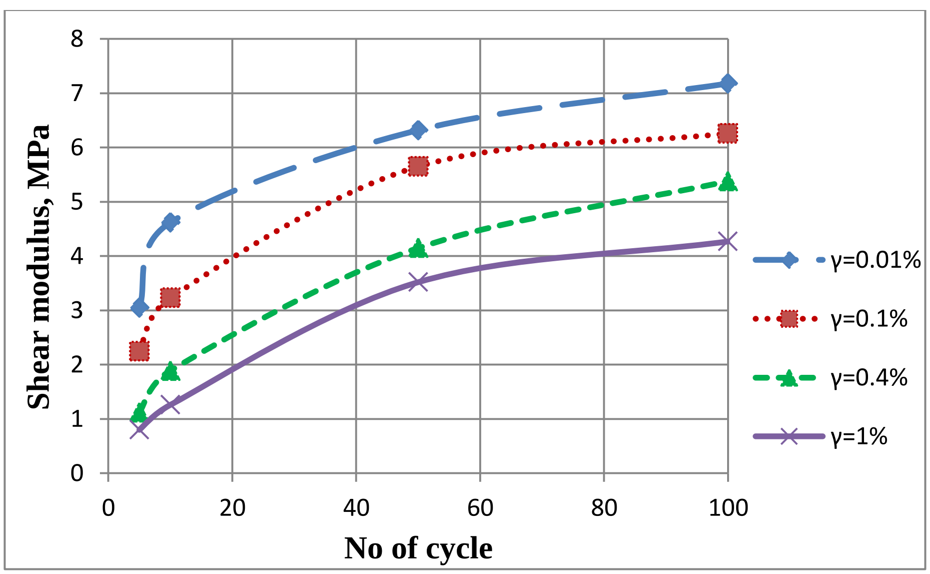


Figure 4.15:Variation of shear modulus with number of cycles for the natural soil under 100kPa axial stress

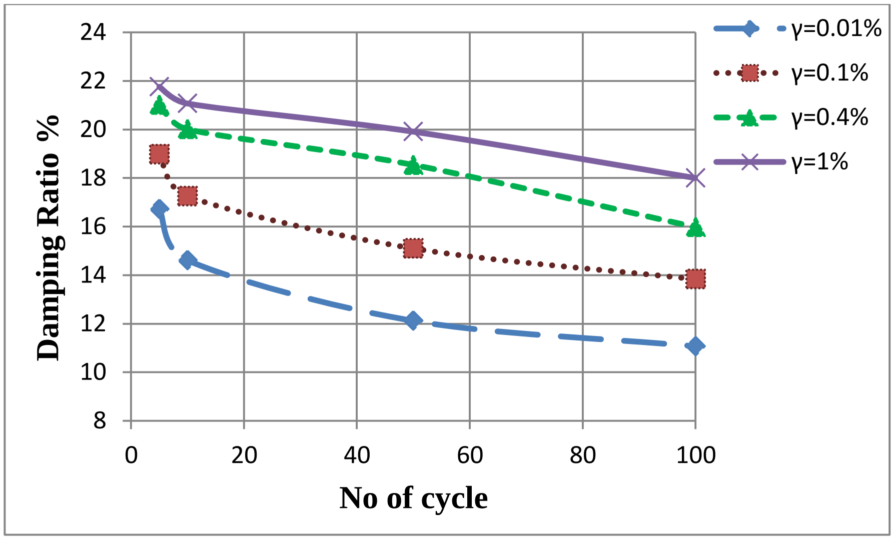


Figure 4.16:Variation of damping ratio with the number of cycles for the natural soil under 100 kPa axial stress

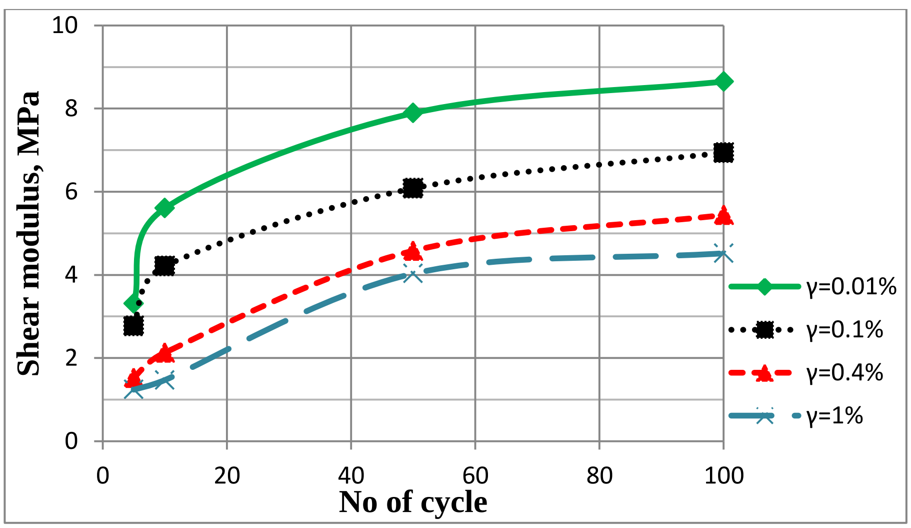


Figure 4.17:Variation of shear modulus with number of cycles for the treated soil under 100kPa axial stress

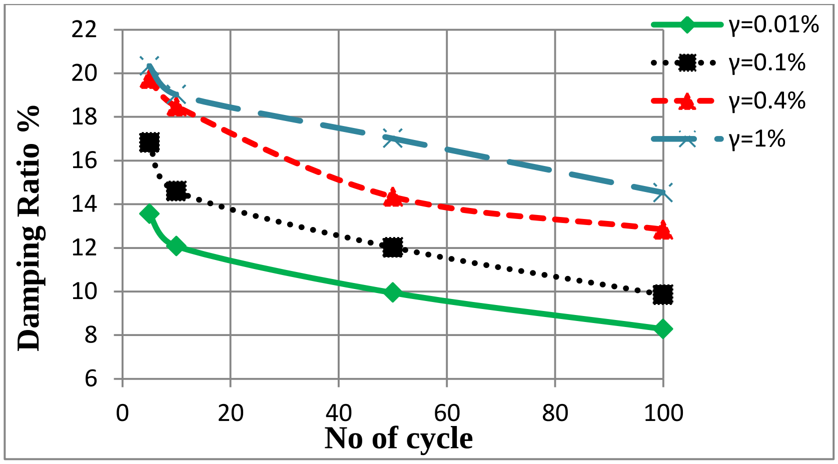


Figure 4.18: Variation of damping ratio with the number of cycles for the treated soil under 100kpa axial stress

### 4.4.5. Variation of Shear Modulus and Damping Ratio with Confining Pressure

Shear modulus and damping ratio values of soil are highly influenced by confining stress. Variation of shear modulus and damping ratio with shear strain is shown in the figure below for confining pressures of 50kPa, 100kPa, and 200kPa. As the confining pressure increases, the shear modulus increase, and the damping ratio decreases. This is due to the reason that, With the increase in confining pressure, there are increased numbers of particle-particle bonds which provide resistance to the specimen to deformation (Mitchell 1976). This means that there is an increased stiffness of the soil with an increase in the confining pressure. Whereas, the bonding reduces the deformation of soils, which leads to the reduction of energy loss or damping ratio.

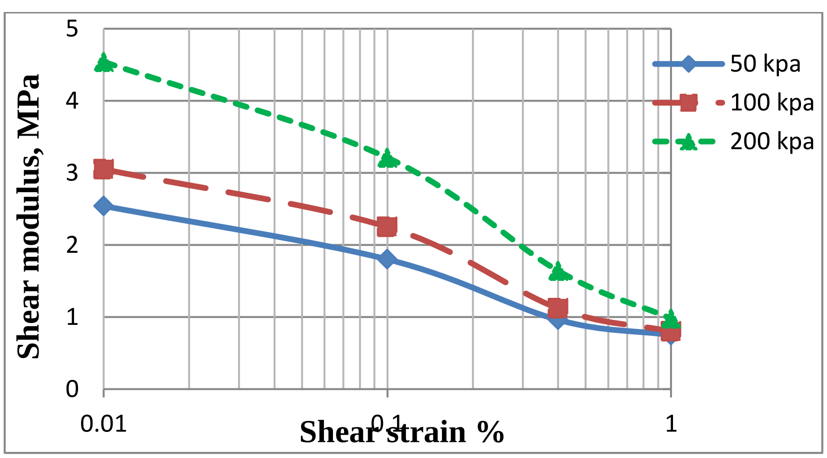


Figure 4.19: Effect of confining pressure on shear modulus for natural soil

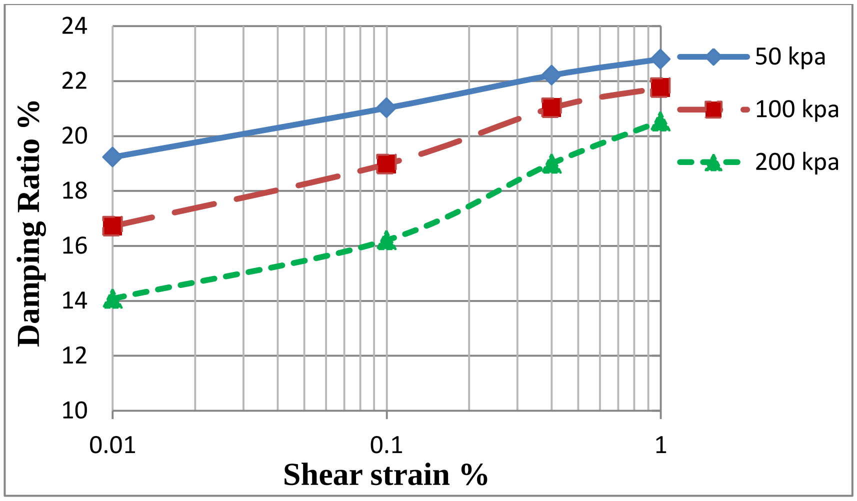


Figure 4.20: Effect of confining pressure on damping ratio for natural soil

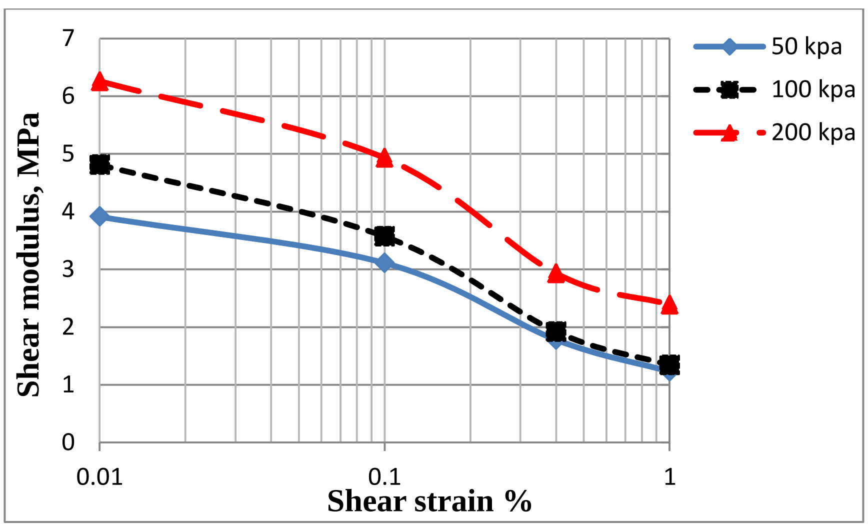


Figure 4.21:Effect of confining pressure on shear modulus for treated soil

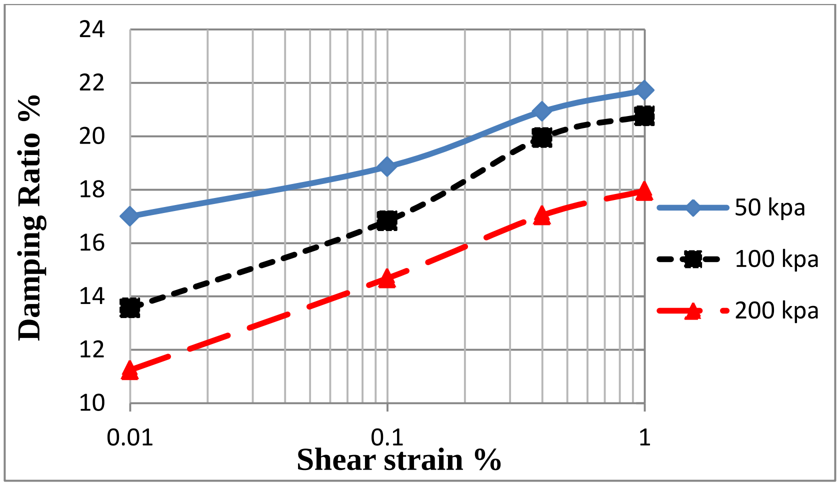


Figure 4.22:Effect of confining pressure on damping ratio for treated soil

### 4.4.6. Estimation of Maximum Shear Modulus and Normalized Shear Modulus

Maximum shear modulus is estimated from a very small strain range of amplitude in the elastic range usually less than 10 $^{-4}$ percent using either direct field measurements or a strain approach that includes laboratory testing and empirical equations. Because the Cyclic simple shear test machine runs in a strain range of $10^{-2}$ to 5%, it is unable to measure shear modulus and damping values at low strain levels. In this study, the equation stated in section 2.5.2 was used for the calculation of the maximum shear modulus.

$$G_{max} = 3220 * \frac{(2.973 - e)^2}{1 + e} * OCR^a * \sigma'_o{}^{k_o}$$

Typical calculation of maximum shear modulus ($G_{max}$) for untreated soil under 100kpa axial stress is shown below. From table 2.5 the value of K for a PI value of 58 by interpolation is 0.4, the void ratio is 1.05, and the pre-consolidation pressure is 90kpa, according to the one-dimensional consolidation test results in Appendix C.

$$OCR = \frac{P_C}{P_O} = \frac{90.0}{100} = 0.9$$

$$\sigma'_O = \frac{\sigma'_V + 2K_O * \sigma'_V}{3} = \frac{(100 + 2 * 0.5 * 100)}{3} = 66.67$$

$$G_{max} = 3220 * \frac{(2.973 - 1.05)^2}{(1 + 1.05)} * (0.9)^{0.40} * (66.67)^{0.5}$$

$G_{max}$ =45611.76Kpa = 45.61Mpa

By using the above equation the values of $G_{max}$ for untreated and treated soil is calculated and presented in Table 4.8.

Table 4.8: Maximum shear modulus ($G_{max}$)values

| Parameter | Untreated soil | | | Treated soil | | |
|---|---|---|---|---|---|---|
| | Axial Stress (kPa) | | | Axial Stress (kPa) | | |
| | 50 | 100 | 200 | 50 | 100 | 200 |
| Void Ratio (e) | 1.05 | 1.05 | 1.05 | 0.86 | 0.86 | 0.86 |
| PI (%) | 58 | 58 | 58 | 27.18 | 27.18 | 27.18 |
| A | 0.399 | 0.399 | 0.399 | 0.223 | 0.223 | 0.223 |
| (OCR) | 1.8 | 0.9 | 0.45 | 2.12 | 1.06 | 0.531 |
| σ'o | 33.33 | 66.67 | 133.33 | 33.33 | 66.67 | 133.33 |
| Gmax (MPa) | 42.55 | 45.61 | 48.88 | 52.94 | 64.15 | 77.72 |

The typical calculation of normalized shear modulus (G/Gmax) for untreated and lime-treated soil samples is shown below in Tables 4.9 and 4.10.

Table 4.9:Typical calculation of normalized shear modulus (G/Gmax) for natural soil

| Confining pressure | Strain, % | 0.01 | 0.1 | 0.4 | 1 |
|---|---|---|---|---|---|
| | Cycle number | G/Gmax | | | |
| 50 | 5 | 0.060 | 0.042 | 0.023 | 0.018 |
| | 10 | 0.097 | 0.067 | 0.031 | 0.025 |
| | 50 | 0.132 | 0.100 | 0.048 | 0.041 |
| | 100 | 0.143 | 0.116 | 0.064 | 0.050 |
| 100 | 5 | 0.066 | 0.049 | 0.025 | 0.018 |
| | 10 | 0.101 | 0.080 | 0.041 | 0.028 |

| | | | | | |
|---|---|---|---|---|---|
| | 50 | 0.139 | 0.124 | 0.091 | 0.077 |
| | 100 | 0.157 | 0.137 | 0.118 | 0.094 |
| 200 | 5 | 0.093 | 0.066 | 0.034 | 0.020 |
| | 10 | 0.119 | 0.089 | 0.047 | 0.028 |
| | 50 | 0.179 | 0.159 | 0.125 | 0.097 |
| | 100 | 0.194 | 0.177 | 0.140 | 0.102 |

Table 4.10: Typical calculation of normalized shear modulus (G/Gmax) for treated soil

| Confining pressure | Strain, % | 0.01 | 0.1 | 0.4 | 1 |
|---|---|---|---|---|---|
| | Cycle number | G/Gmax | | | |
| 50 | 5 | 0.074 | 0.059 | 0.034 | 0.023 |
| | 10 | 0.121 | 0.090 | 0.043 | 0.036 |
| | 50 | 0.148 | 0.112 | 0.078 | 0.066 |
| | 100 | 0.162 | 0.128 | 0.097 | 0.072 |
| 100 | 5 | 0.075 | 0.056 | 0.030 | 0.021 |
| | 10 | 0.117 | 0.087 | 0.049 | 0.035 |
| | 50 | 0.150 | 0.123 | 0.081 | 0.072 |
| | 100 | 0.161 | 0.127 | 0.094 | 0.084 |
| 200 | 5 | 0.081 | 0.063 | 0.038 | 0.031 |
| | 10 | 0.110 | 0.084 | 0.050 | 0.040 |
| | 50 | 0.152 | 0.114 | 0.089 | 0.084 |
| | 100 | 0.158 | 0.131 | 0.098 | 0.088 |

### 4.4.7. Effect of Lime on Shear Modulus and Damping of the Soil

The variation of shear modulus and damping ratio of the soil with treatment under axial stress of 100Kpa for the 5$^{th}$ cycle is shown in Figures 4.23 to 25. As illustrated in Figure 4.23, treated soil has a higher shear modulus value than untreated soil, This is due to the cementing effect with lime on the treated soil specimen caused by pozzolanic reactions. Since the treated soil exhibited more rigid behavior than the untreated soils, larger values of G resulted. On the other hand, the damping value of treated soil is lower than untreated soils as illustrated in Figure 4.25. The soil's Damping represents the quantity of energy dissipated under cyclic loading. As the degree of particle slippage and particle rearrangement increases, the damping ratio of the soil increases (Fahoum et al., 1996). The decrease in D can be explained by the reason that the treated materials were found to be more rigid than the untreated soil, less slippage and particle rearrangement would be anticipated. Therefore, damping would be less in treated soils than in untreated soils. This may be the reason for the probable decrease in damping ratio with an increase in cementation as stated by (Dobry, 1987).

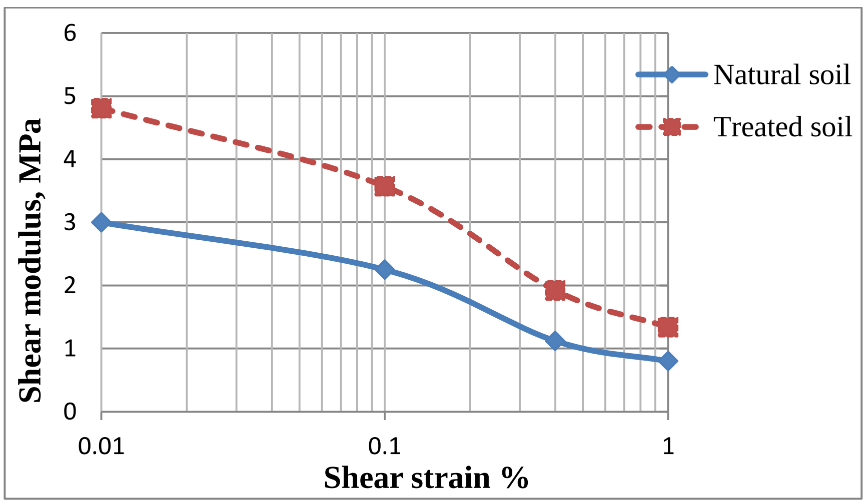


Figure 4.23: Effect of Lime on the variation of shear modulus with the shear strain

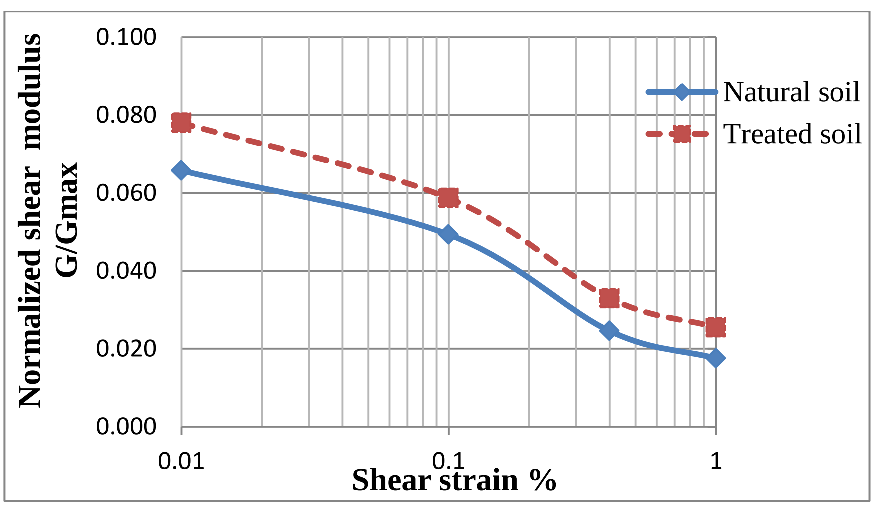


Figure 4.24:Effect of Lime on the variation of normalized shear modulus with shear strain

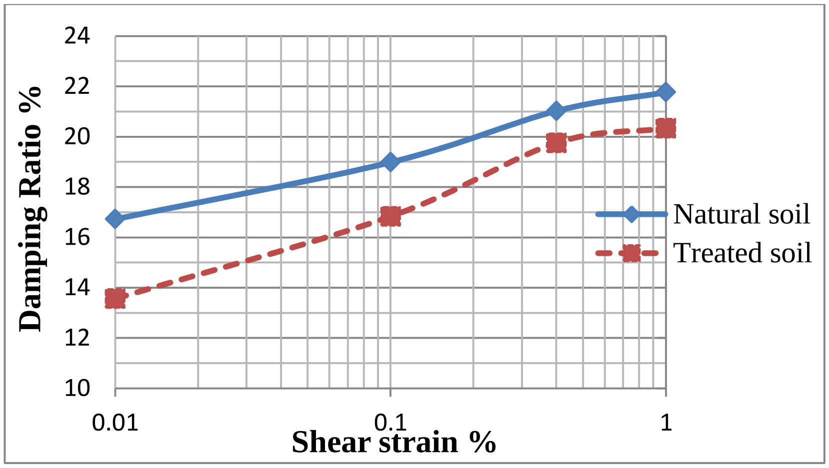


Figure 4.25: Effect of Lime on the variation of damping ratio with shear strain

### 4.4.8. Comparison of Shear Modulus Reduction and Damping Ratio with Previous Studies

Different researchers previously developed various modulus reduction (Normalized shear modulus) and damping ratio charts for various soils. The study's experimental results were compared to data from previously published literature. The normalized shear modulus (G/Gmax) and damping ratio curves computed were compared to the curve developed by Vucetic and Dobry (1991) for plastic soils, Teshome (2019) for local CH silty clay soil, Abyalew(2022) for Sugarcane Bagasse Ash treated soils, and Ferric Chloride Stabilized Soils by Wengelawit (2021)

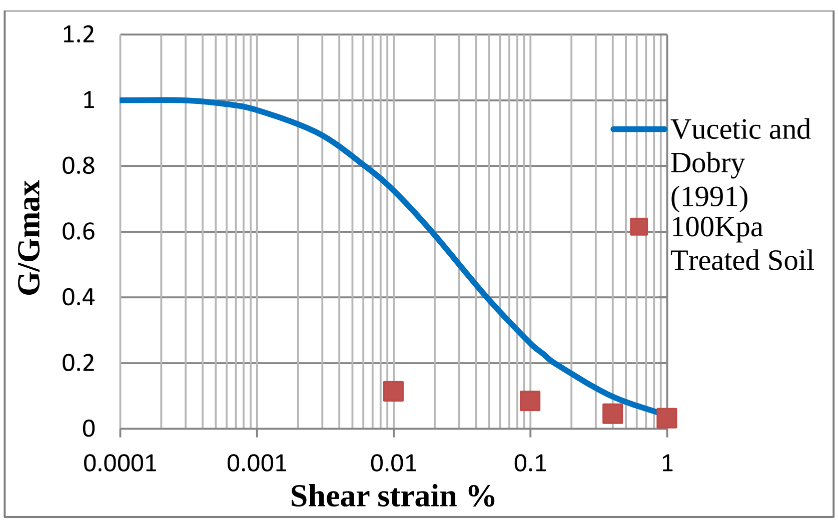


Figure 4.26:Comparison of modulus reduction values of stabilized clay soil with curves developed for plastic soils by Vucetic and Dobry (1991)

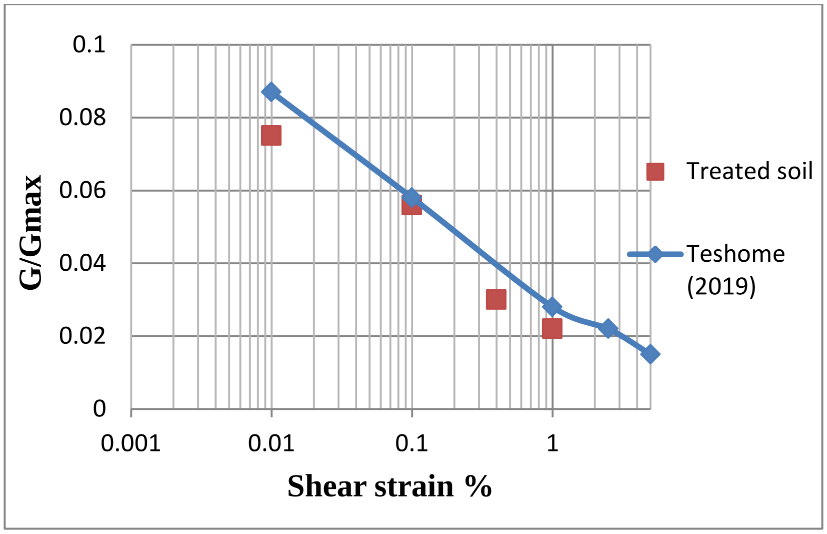


Figure 4.27:Comparison of modulus reduction values of treated soil with curves developed for Jimma town CH silty clay soil by Teshome (2019)

As illustrated in Figure 4.26 the computed normalized shear modulus value was comparable with the curves developed for saturated clays by Vucetic and Dobry (1991) for high shear strains (> 0.1%), however, the G/Gmax varies for lower shear strains(<0.1 %), This variation occurred may be due to differences in sampling, sample preparation, and utilization of different types of testing machines. From Figure 4.27, it can be observed that the modulus reduction curve of the treated soil sample has good agreement with the local plastic silt soil curve developed by Teshome (2019).

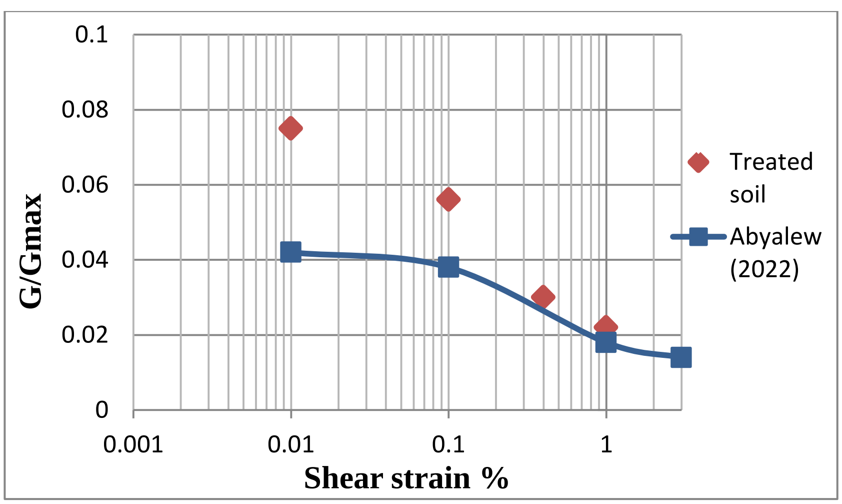


Figure 4.28: Comparison of modulus reduction values of treated soil with curves developed for Sugarcane Bagasse Ash treated expansive soils by Abyalew (2022)

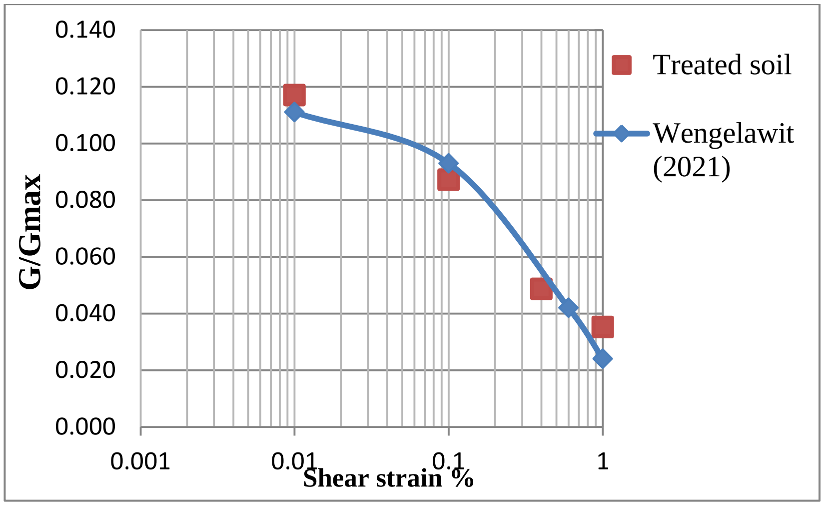


Figure 4.29: Comparison of modulus reduction values of treated soil with curves developed for Ferric Chloride stabilized expansive soils by Wengelawit (2021)

From Figures 4.28 and 4.29, it can be seen that the treated soil modulus reduction curve has compatible with the curve developed by Wengelawit (2021) for ferric chloride treated expansive soils and Abyalew (2022) for Sugarcane Bagasse Ash treated expansive soil; but the computed value of G/Gmax lie above SBCA treated expansive soils. This shows that the lime treatment has more impact on the soil's shear modulus than SBCA.

Figures 4.30 and 4.31 shows a comparison of the damping ratio value with the local soil curve developed for silty clay soil by Teshome (2019) and ferric chloride-treated soil by Wengelawit (2021). The computed damping ratio values were in good agreement with the curves but were higher than the silty clay soil curves. This may be due to the effect of treatment and testing conditions. And from Figure 4.31, the computed damping value is lower than for ferric chloride-treated soil this is due to the effect of the strength of ferric chloride.

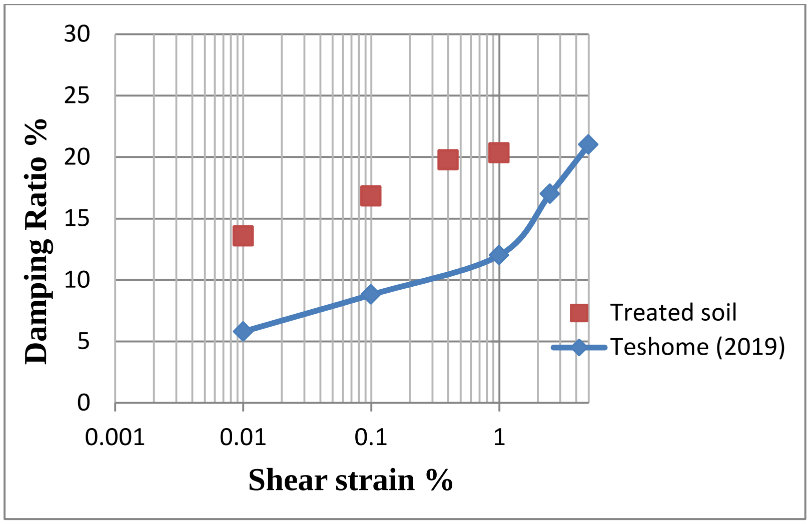


Figure 4.30:Comparison of damping ratio values of treated soil with curves developed for Jimma town CH silty clay soil by Teshome (2019).

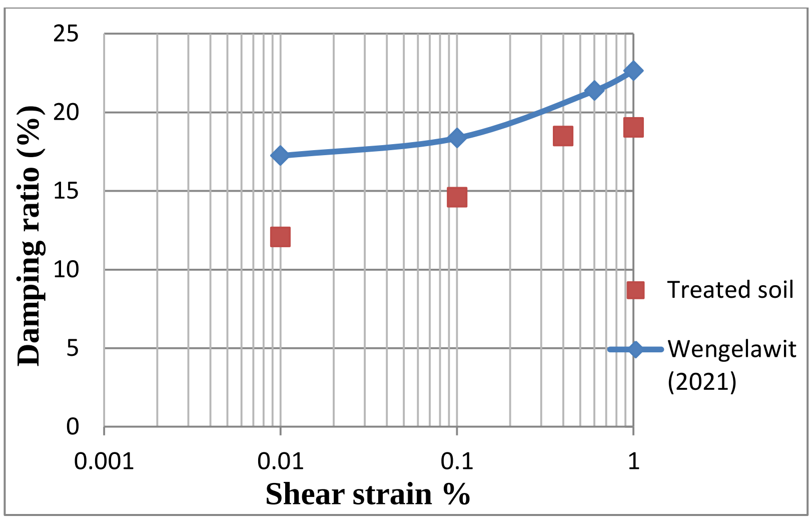


Figure 4.31: Comparison of damping ratio values of treated soil with Ferric Chloride stabilized expansive soils by Wengelawit (2021)

# CHAPTER FIVE

# CONCLUSIONS AND RECOMMENDATIONS

## 5.1. Conclusion

This study investigated the dynamic properties of expansive subgrade soil treated with lime. Laboratory index and static tests were conducted for the classification purpose and determination of the optimum amount of lime required to stabilize the soil. To determine the pertinent dynamic soil properties namely the shear modulus (G) and damping ratio (D), cyclic simple shear tests were performed on natural and lime-treated soil samples. Strain controlled cyclic simple shear tests were conducted for strain values of 0.01 %, 0.1 %, 0.4 %, and 1 % under axial stress of 50, 100 and 200 kPa.From the result of these investigations the following conclusions have been drawn:

- ✓ It is observed that with the addition of lime to expansive soil, swelling potential reduces and improves the static strength as well as dynamic properties of soil.
- ✓ The PI of the soil decreased from 58% to 20.20% with the amount of lime in the soil increasing from 0% to 9%.
- ✓ The MDD decrease and OMC increased as the lime content increased within the soil mass. Thus, MDD decreases from 1.30g/cm3 to 1.19g/cm3, and OMC increased from 36.17% to 43.50% as lime content increased from 0% to 9%.
- ✓ The UCS of the soil increased as the percentage of the lime increased up to 6%, and then decreased. The values of UCS of a soil increased from 92.23 kPa to 274.75 kPa by the addition of 6% lime by the dry weight of the soil.
- ✓ The CBR value of the expansive soil increases from 1.74% to 14.88% with the addition of 6% lime and the percent of swell decreases from 5.86% to 1.87%.
- ✓ The value of shear modulus is in the range of 0.75 to 9.46 MPa for untreated soil and 1.23 to 12.27 MPa for lime-treated soil specimens. On the otherhand, the damping ratio values range between 11.04 to 22.8 % for untreated soil and 8.13 to 21.72 % for lime-treated soil for strain amplitude ranging from 0.01% to 1%.

- ✓ The shear modulus value decrease as the strain amplitude increases from 0.01% to 1% for both lime-treated and untreated soil specimens while increasing with the number of cycles.
- ✓ Conversely, the damping ratio value increases as shear strain amplitude increases from 0.01% to 1% whereas, it decreases as the number of cycles increases.
- ✓ Additionally with an increase in confining pressure the shear modulus increases while the damping ratio decreases.
- ✓ The shear modulus of treated soil increases, and the damping ratio values decrease than that of natural soil for the strain amplitude varying from 0.01% to 1%. This is due to the increase in rigidity of the treated soil caused as a result of pozzolanic reactions.
- ✓ The study shows the use of lime to stabilize expansive subgrade soil under both static and dynamic loading is effective in increasing the stiffness of the soil and reducing deformation.

## 5.2. Recommendations

- ✓ The dynamic test was done by using cyclic simple shear apparatus. For comparisons and a better understanding of test results, the study should be conducted using the cyclic triaxial testing machine.
- ✓ To evaluate the influence of other parameters and comprehend the behavior of the soil further under dynamic loading, the test should be conducted while considering other parameters.
- ✓ Dynamic properties of stabilized soil with various stabilization techniques should be investigated for future study.

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# APPENDICES

# Appendix A: Test Results of Natural Soils

Table A.1:Natural moisture content for pits 1 and 2

| | Pit 1 | | | Pit 2 | | |
|---|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 | 3 |
| Container No | 12 | 1 | 48 | A3 | 5 | C12 |
| Mass of container, g | 15.89 | 15.49 | 15.19 | 15.48 | 15.89 | 15.58 |
| Mass of container + Wet soil, g | 56.56 | 56.62 | 56.75 | 39.09 | 38.62 | 35.97 |
| Mass of container + Dry soil, g | 47.25 | 47.02 | 45.95 | 34.42 | 33.41 | 31.20 |
| Mass of water, g | 9.31 | 9.60 | 10.80 | 4.67 | 5.21 | 4.77 |
| Mass of dry soil, g | 31.36 | 31.53 | 30.76 | 18.94 | 17.52 | 15.62 |
| Water content, % | 29.69 | 30.45 | 35.11 | 24.66 | 29.74 | 30.54 |
| Ave. moisture content,% | 31.75 | | | 28.31 | | |

Table A.2:Wet sieve analysis test results for pit 1

| Sieve No | Sieve Opening (mm) | Mass of Sieve (g) | Mass of sieve + Retained soil (g) | Mass of Retained soil (g) | Percentage Retained (%) | Cum. Percentage Retained (%) | Perc. Passing (%) |
|---|---|---|---|---|---|---|---|
| 3.8" | 9.5 | 586.0 | 586.0 | 0.0 | 0.0 | 0.0 | 100.0 |
| No 4 | 4.75 | 567.0 | 567.5 | 0.5 | 0.1 | 0.1 | 99.9 |
| No 8 | 2.36 | 396.5 | 400.6 | 4.1 | 0.4 | 0.5 | 99.5 |
| No 10 | 2 | 354.0 | 359.3 | 5.3 | 0.5 | 1.0 | 99.0 |
| No 16 | 0.6 | 323.9 | 327.4 | 3.5 | 0.4 | 1.3 | 98.7 |
| No 30 | 0.425 | 447.8 | 453.9 | 6.1 | 0.6 | 2.0 | 98.1 |
| No 50 | 0.3 | 308.6 | 313.8 | 5.2 | 0.5 | 2.5 | 97.5 |
| No 100 | 0.15 | 423.2 | 433.4 | 10.2 | 1.0 | 3.5 | 96.5 |
| No 200 | 0.075 | 417.1 | 436.0 | 18.9 | 1.9 | 5.4 | 94.6 |
| pan | ------- | 240.3 | 1186.5 | 946.2 | 94.6 | 100.0 | ------- |

Table A.3:Hydrometer analysis test for pit 1

| Elapsed Time (min) | Actual Hydrometer Reading | Composite Correction | Corrected Hydrometer Reading | Effective Depth (cm) | Coefficient K | Grain Size (mm) | Perc. Finer (%) | Perc. Finer Combined (%) |
|---|---|---|---|---|---|---|---|---|
| 3/4 | 1.0320 | -0.0027 | 1.0293 | 7.84 | 0.01344 | 0.0376 | 93.07 | 88.06 |
| 1 | 1.0310 | -0.0027 | 1.0283 | 8.10 | 0.01344 | 0.0270 | 89.89 | 85.06 |
| 2 | 1.0305 | -0.0027 | 1.0278 | 8.23 | 0.01344 | 0.0193 | 88.31 | 83.56 |
| 4 | 1.0300 | -0.0027 | 1.0273 | 8.36 | 0.01344 | 0.0137 | 86.72 | 82.05 |
| 8 | 1.0300 | -0.0027 | 1.0273 | 8.36 | 0.01344 | 0.0100 | 86.72 | 82.05 |
| 15 | 1.0295 | -0.0027 | 1.0268 | 8.50 | 0.01344 | 0.0072 | 85.13 | 80.55 |
| 30 | 1.0280 | -0.0027 | 1.0253 | 8.89 | 0.01344 | 0.0053 | 80.36 | 76.04 |
| 60 | 1.0270 | -0.0027 | 1.0243 | 9.16 | 0.01344 | 0.0052 | 77.19 | 73.04 |
| 120 | 1.0260 | -0.0027 | 1.0233 | 9.42 | 0.01344 | 0.0038 | 74.01 | 70.03 |
| 240 | 1.0250 | -0.0027 | 1.0223 | 9.69 | 0.01344 | 0.0027 | 70.84 | 67.02 |
| 480 | 1.0245 | -0.0027 | 1.0218 | 9.82 | 0.01344 | 0.0019 | 69.25 | 65.52 |
| 1440 | 1.0240 | -0.0027 | 1.0213 | 9.95 | 0.01344 | 0.0011 | 67.66 | 64.02 |

Table A.4: Wet sieve analysis test results for pit 2

| Sieve No | Sieve Opening (mm) | Mass of Sieve (g) | Mass of sieve + Retained soil (g) | Mass of Retained soil (g) | Percentage Retained (%) | Cum. Percentage Retained (%) | Perc. Passing (%) |
|---|---|---|---|---|---|---|---|
| 3.8" | 9.5 | 586.0 | 586.0 | 0.0 | 0.0 | 0.0 | 100.0 |
| No 4 | 4.75 | 567.0 | 567.5 | 1.1 | 0.1 | 0.1 | 99.9 |
| No 8 | 2.36 | 396.5 | 400.6 | 8.4 | 0.8 | 1.0 | 99.1 |
| No 10 | 2 | 354.0 | 359.3 | 9.5 | 1.0 | 1.9 | 98.1 |
| No 16 | 0.6 | 323.9 | 327.4 | 13.2 | 1.3 | 3.2 | 96.8 |
| No 30 | 0.425 | 447.8 | 453.9 | 8.9 | 0.9 | 4.1 | 95.9 |
| No 50 | 0.3 | 308.6 | 313.8 | 11.1 | 1.1 | 5.2 | 94.8 |
| No 100 | 0.15 | 423.2 | 433.4 | 19.3 | 1.9 | 7.2 | 92.9 |
| No 200 | 0.075 | 417.1 | 436.0 | 23.7 | 2.4 | 9.5 | 90.5 |
| pan | ------- | 240.3 | 1145.1 | 904.8 | 90.5 | 100.0 | ------- |

Table A.5:Hydrometer analysis test for pit 2

| Elapsed Time (min) | Actual Hydrometer Reading | Composite Correction | Corrected Hydrometer Reading | Effective Depth (cm) | Coefficient K | Grain Size (mm) | Perc. Finer (%) | Perc. Finer Combined (%) |
|---|---|---|---|---|---|---|---|---|
| 3/4 | 1.0300 | -0.0027 | 1.0273 | 8.36 | 0.01336 | 0.0387 | 86.34 | 78.12 |
| 1 | 1.0300 | -0.0027 | 1.0273 | 8.36 | 0.01336 | 0.0273 | 86.34 | 78.12 |
| 2 | 1.0295 | -0.0027 | 1.0268 | 8.50 | 0.01336 | 0.0195 | 84.76 | 76.69 |
| 4 | 1.0290 | -0.0027 | 1.0263 | 8.63 | 0.01336 | 0.0139 | 83.18 | 75.26 |
| 8 | 1.0280 | -0.0027 | 1.0253 | 8.89 | 0.01336 | 0.0103 | 80.02 | 72.40 |
| 15 | 1.0275 | -0.0027 | 1.0248 | 9.03 | 0.01336 | 0.0073 | 78.44 | 70.97 |
| 30 | 1.0270 | -0.0027 | 1.0243 | 9.16 | 0.01336 | 0.0053 | 76.86 | 69.54 |
| 60 | 1.0260 | -0.0027 | 1.0233 | 9.42 | 0.01336 | 0.0052 | 73.69 | 66.68 |
| 120 | 1.0255 | -0.0027 | 1.0228 | 9.55 | 0.01336 | 0.0038 | 72.11 | 65.25 |
| 240 | 1.0250 | -0.0027 | 1.0223 | 9.69 | 0.01336 | 0.0027 | 70.53 | 63.82 |
| 480 | 1.0245 | -0.0027 | 1.0218 | 9.82 | 0.01336 | 0.0019 | 68.95 | 62.38 |
| 1440 | 1.0230 | -0.0027 | 1.0203 | 10.22 | 0.01336 | 0.0011 | 64.20 | 58.09 |

Table A.6: Atterberg limit test result for pit 1

| | Liquid Limit(%) | | | Plastic Limit(%) | |
|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 |
| Container No | 26 | A36 | C31 | C34 | 71 |
| Mass of container, g | 16.02 | 14.44 | 14.60 | 15.47 | 15.14 |
| Mass of container + Wet soil, g | 30.98 | 29.96 | 32.61 | 63.54 | 65.88 |
| Mass of container + Dry soil, g | 23.78 | 22.43 | 23.68 | 50.20 | 52.25 |
| Mass of water, g | 7.20 | 7.53 | 8.93 | 13.34 | 13.63 |
| Mass of dry soil, g | 7.76 | 7.99 | 9.08 | 34.73 | 37.11 |
| Water content, % | 92.78 | 94.24 | 98.35 | 38.41 | 36.73 |
| No of blows | 32 | 24 | 18 | ------ | ------ |
| Average | 95.1 | | | 37.6 | |
| Plasticity Index (%) | 58 | | | | |

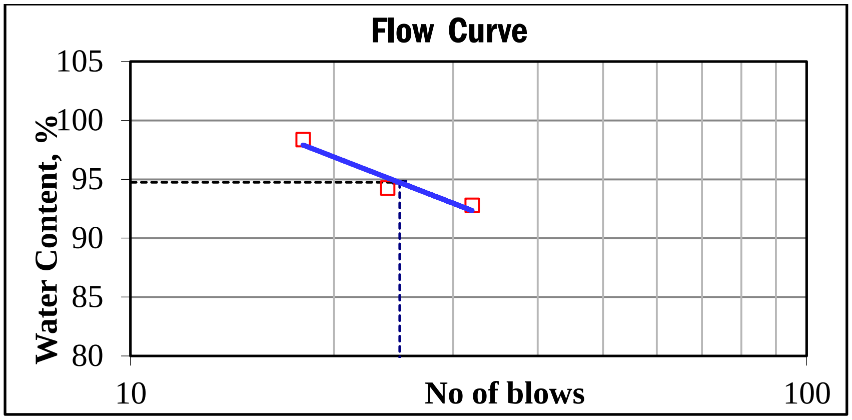



Figure A.1: Liquid limit determination for pit 1

Table A.7: Atterberg limit test result for pit 2

| | Liquid Limit(%) | | | Plastic Limit(%) | |
|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 |
| Container No | 26 | A36 | C31 | C34 | 71 |
| Mass of container, g | 15.63 | 12.96 | 15.89 | 13.79 | 15.49 |
| Mass of container + Wet soil, g | 32.55 | 28.86 | 34.05 | 67.52 | 63.23 |
| Mass of container + Dry soil, g | 24.78 | 21.46 | 25.35 | 52.66 | 51.10 |
| Mass of water, g | 7.77 | 7.40 | 8.70 | 14.86 | 12.13 |
| Mass of dry soil, g | 9.15 | 8.50 | 9.46 | 38.87 | 35.61 |
| Water content, % | 84.92 | 87.06 | 91.97 | 38.23 | 34.06 |
| No of blows | 33 | 28 | 17 | ------ | ------ |
| Average | 88.00 | | | 36.15 | |
| Plasticity Index (%) | 52 | | | | |

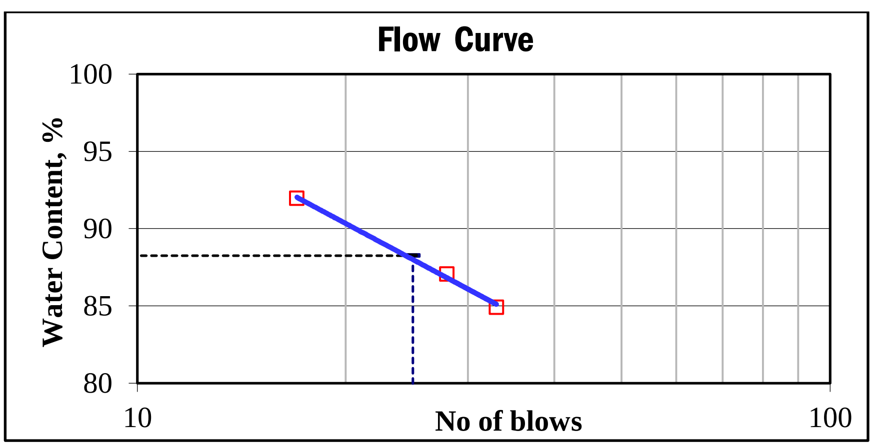



Figure A.2:Liquid limit determination for pit 1

Table A.8:Specific gravity test result for pit 1

| Determination No. | 1 | 2 |
|---|---|---|
| Pycnometer No. | P1 | P2 |
| W1= Mass of empty, clean pycnometer (g) | 111.9 | 107.2 |
| W2 = Mass of empty pycnometer + dry soil (g) | 149.4 | 144.3 |
| W3= Mass of pycnometer + dry soil + water (g) | 386.6 | 379.3 |
| W4 = Mass of pycnometer + water (g) | 363.0 | 355.9 |
| Specific Gravity (GS) | 2.694 | 2.714 |
| Average Specific gravity | 2.70 | |

Table A.9: Specific gravity test result for pit 2

| Determination No. | 1 | 2 |
|---|---|---|
| Pycnometer No. | P2 | P1 |
| W1= Mass of empty, clean pycnometer (g) | 107.3 | 111.7 |
| W2 = Mass of empty pycnometer + dry soil (g) | 147.3 | 152.0 |
| W3= Mass of pycnometer + dry soil + water (g) | 380.9 | 388.4 |
| W4 = Mass of pycnometer + water (g) | 355.5 | 363.0 |
| Specific Gravity (GS) | 2.734 | 2.706 |
| Average Specific gravity | 2.72 | |

Table A.10: Free swell test result for pit 1 and 2

| Soil sample | Initial Volume (g) | Final Volume | | Average Final Volume (g) | Free Swell Index (%) |
|---|---|---|---|---|---|
| | | Sample No.1 (g) | Sample No.2 (g) | | |
| pit-1 | 10.0 | 21.0 | 21.0 | 21.0 | 110 |
| pit-2 | 10.0 | 18.0 | 18.0 | 18.0 | 80 |

Table A.11: Compaction property of pit 1

| Determination No. | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Mass of Mold, g | 4374 | | 4374 | | 4374 | | 4374 | |
| Mass of mold+ Compacted Soil, g | 5711 | | 5829 | | 6046 | | 5997 | |
| Mass of Compacted soil, g | 1337 | | 1455 | | 1672 | | 1623 | |
| Volume of Mold,cm3 | 944 | | 944 | | 944 | | 944 | |
| Bulk density, g/cm3 | 1.42 | | 1.54 | | 1.77 | | 1.72 | |
| MoistureContent Determination | | | | | | | | |
| Weight of can | 15.64 | 15.52 | 15.72 | 15.52 | 14.44 | 12.69 | 14.60 | 15.99 |
| weight of can +wet soil | 55.01 | 54.21 | 45.14 | 49.55 | 46.48 | 43.53 | 48.27 | 46.78 |
| weight of can +dry soil | 48.29 | 47.92 | 39.34 | 42.57 | 38.25 | 35.15 | 38.11 | 36.95 |
| Moisture content (%) | 20.58 | 19.41 | 24.56 | 25.80 | 34.57 | 37.76 | 43.22 | 46.90 |
| Average Water Contenet, % | 20.00 | | 25.18 | | 36.17 | | 45.06 | |
| Dry density, g/cm3 | 1.18 | | 1.23 | | 1.30 | | 1.19 | |

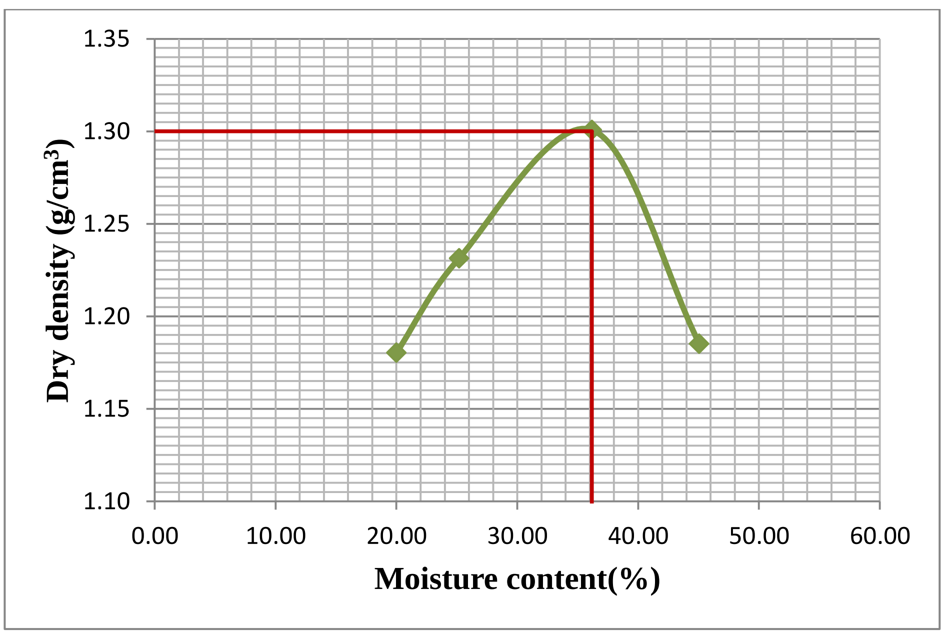



Figure A.3:Compaction curve of pit 1

Table A.12: Compaction property of pit 2

| Determination No. | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Mass of Mold, g | 4374 | | 4374 | | 4374 | | 4374 | |
| Mass of mold+ Compacted Soil, g | 5738 | | 5811 | | 5983 | | 5967 | |
| Mass of Compacted soil, g | 1364 | | 1437 | | 1609 | | 1593 | |
| Volume of Mold,cm3 | 944 | | 944 | | 944 | | 944 | |
| Bulk density, g/cm3 | 1.44 | | 1.52 | | 1.70 | | 1.69 | |
| Moisture Content Determination | | | | | | | | |
| Weight of can | 15.62 | 14.32 | 15.84 | 13.78 | 15.42 | 13.78 | 16.17 | 14.06 |
| weight of can +wet soil | 58.00 | 60.01 | 57.06 | 56.05 | 60.76 | 59.36 | 54.81 | 58.64 |
| weight of can +dry soil | 52.55 | 53.83 | 50.73 | 49.31 | 50.89 | 49.55 | 43.98 | 47.34 |
| Moisture content (%) | 14.76 | 15.64 | 18.14 | 18.97 | 27.83 | 28.81 | 38.94 | 33.95 |
| Average Water Contenet, % | 15.20 | | 18.56 | | 28.32 | | 36.45 | |
| Dry density, g/cm3 | 1.25 | | 1.28 | | 1.33 | | 1.24 | |

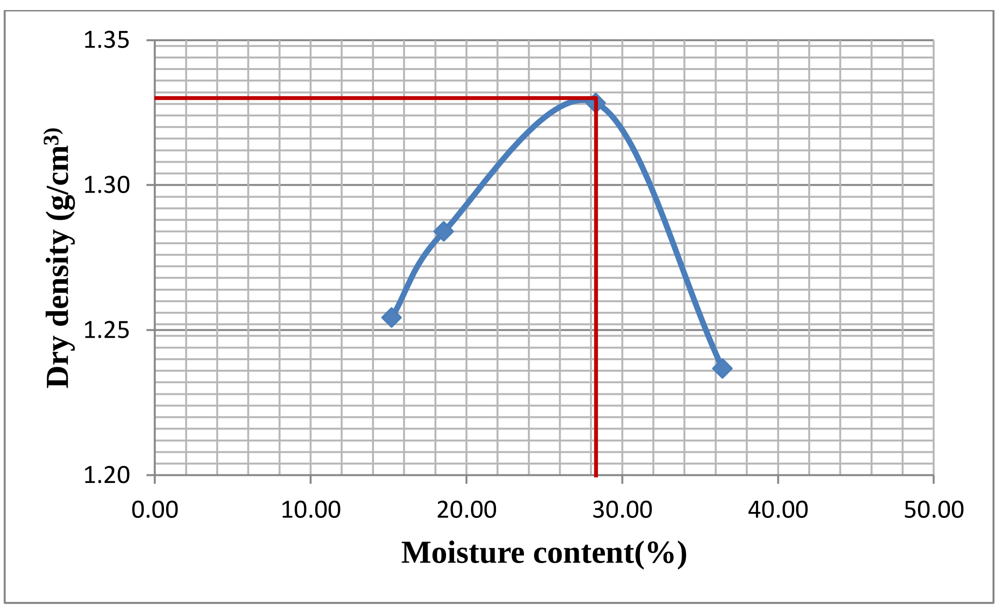



Figure A.4:Compaction curve of pit 2

Table A.13:Unconfined compression strength test result for pit 1

| **Test Pit No** | **1** | Ring Calibration Factor, kN/div | | | 0.00138 |
|---|---|---|---|---|---|
| Diameter of the sample, mm | 38 | | | | |
| Length of the sample, mm | 76 | | | | |
| Axial Deformation [mm] | Axial Strain [%] | Proving Ring Reading [div] | Axial Load [kN] | Corrected Area [m2] | Axial Stress [kPa] |
| 0 | 0.000 | 0 | 0.0000 | 0.00113 | 0 |
| 0.2 | 0.003 | 19 | 0.0262 | 0.00114 | 23.06 |
| 0.4 | 0.005 | 27 | 0.0373 | 0.00114 | 32.68 |
| 0.6 | 0.008 | 33 | 0.0455 | 0.00114 | 39.84 |
| 0.8 | 0.011 | 41 | 0.0566 | 0.00115 | 49.37 |
| 1 | 0.013 | 46 | 0.0635 | 0.00115 | 55.24 |
| 1.2 | 0.016 | 49 | 0.0676 | 0.00115 | 58.69 |
| 1.4 | 0.018 | 54 | 0.0745 | 0.00116 | 64.50 |
| 1.6 | 0.021 | 59 | 0.0814 | 0.00116 | 70.29 |
| 1.8 | 0.024 | 61 | 0.0842 | 0.00116 | 72.47 |
| 2 | 0.026 | 65 | 0.0897 | 0.00116 | 77.02 |
| 2.2 | 0.029 | 68 | 0.0938 | 0.00117 | 80.36 |
| 2.4 | 0.032 | 69 | 0.0952 | 0.00117 | 81.32 |
| 2.6 | 0.034 | 71 | 0.0980 | 0.00117 | 83.45 |
| 2.8 | 0.037 | 73 | 0.1007 | 0.00118 | 85.56 |
| 3 | 0.039 | 75 | 0.1035 | 0.00118 | 87.67 |
| 3.2 | 0.042 | 76 | 0.1049 | 0.00118 | 88.59 |
| 3.4 | 0.045 | 77 | 0.1063 | 0.00119 | 89.51 |
| 3.6 | 0.047 | 78 | 0.1076 | 0.00119 | 90.42 |
| 3.8 | 0.050 | 79 | 0.1090 | 0.00119 | 91.33 |
| 4 | 0.053 | 80 | 0.1104 | 0.00120 | 92.23 |

Table A.14:Unconfined compression strength test result for pit 2

| **Test Pit No** | **2** | Ring Calibration Factor, kN/div | | 0.00138 | |
|---|---|---|---|---|---|
| Diameter of the sample, mm | 38 | | | | |
| Length of the sample, mm | 76 | | | | |
| Axial Deformation [mm] | Axial Strain [%] | Proving Ring Reading [div] | Axial Load [kN] | Corrected Area [m2] | Axial Stress [kPa] |
| 0 | 0.000 | 0 | 0.0000 | 0.00113 | 0 |
| 0.2 | 0.003 | 11 | 0.0152 | 0.00114 | 13.35 |
| 0.4 | 0.005 | 16 | 0.0221 | 0.00114 | 19.37 |
| 0.6 | 0.008 | 18 | 0.0248 | 0.00114 | 21.73 |
| 0.8 | 0.011 | 21 | 0.0290 | 0.00115 | 25.29 |
| 1 | 0.013 | 25 | 0.0345 | 0.00115 | 30.02 |
| 1.2 | 0.016 | 29 | 0.0400 | 0.00115 | 34.73 |
| 1.4 | 0.018 | 36 | 0.0497 | 0.00116 | 43.00 |
| 1.6 | 0.021 | 40 | 0.0552 | 0.00116 | 47.65 |
| 1.8 | 0.024 | 47 | 0.0649 | 0.00116 | 55.84 |
| 2 | 0.026 | 55 | 0.0759 | 0.00116 | 65.17 |
| 2.2 | 0.029 | 59 | 0.0814 | 0.00117 | 69.72 |
| 2.4 | 0.032 | 64 | 0.0883 | 0.00117 | 75.42 |
| 2.6 | 0.034 | 70 | 0.0966 | 0.00117 | 82.27 |
| 2.8 | 0.037 | 73 | 0.1007 | 0.00118 | 85.56 |
| 3 | 0.039 | 77 | 0.1063 | 0.00118 | 90.00 |
| 3.2 | 0.042 | 80 | 0.1104 | 0.00118 | 93.26 |
| 3.4 | 0.045 | 84 | 0.1159 | 0.00119 | 97.65 |
| 3.6 | 0.047 | 90 | 0.1242 | 0.00119 | 104.34 |
| 3.8 | 0.050 | 93 | 0.1283 | 0.00119 | 107.52 |
| 4 | 0.053 | 94 | 0.1297 | 0.00120 | 108.37 |
| 4.2 | 0.055 | 97 | 0.1339 | 0.00120 | 111.52 |
| 4.4 | 0.058 | 99 | 0.1366 | 0.00120 | 113.50 |
| 4.6 | 0.061 | 98 | 0.1352 | 0.00121 | 112.04 |
| 4.6 | 0.061 | 97 | 0.1339 | 0.00121 | 110.90 |

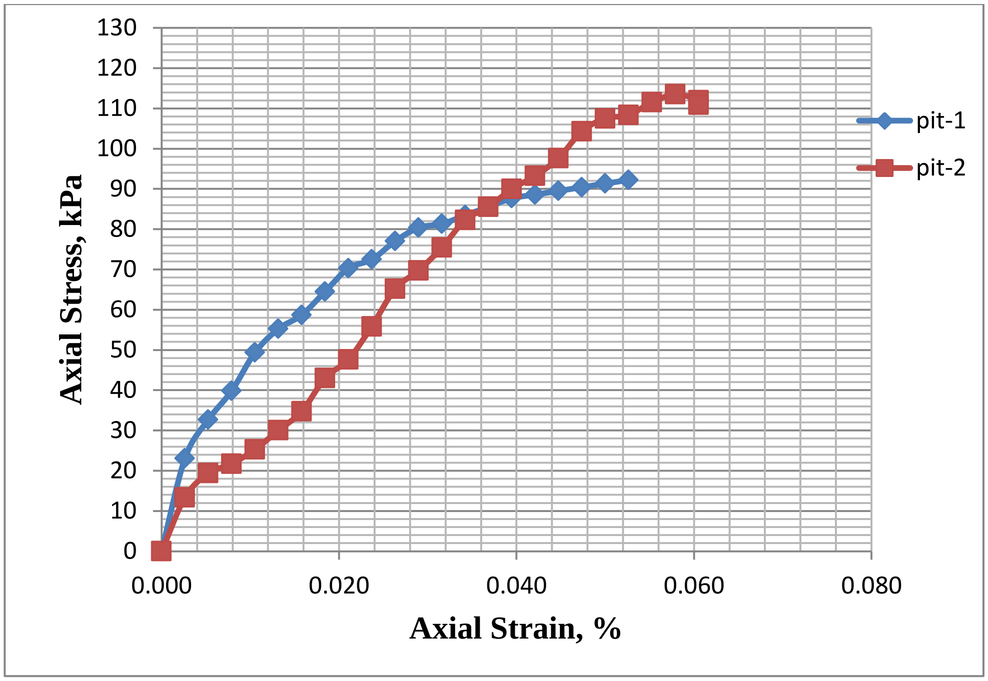



Figure A.5:Variation of axial stress and axial strain for pits 1 and 2

# Appendix B: Test Results of Lime Stabilized Soil

Table B.1:Free swell index test result for different percentages of Lime

| Soil sample | Initial Volume (g) | Final Volume | | Average Final Volume (g) | Free Swell Index (%) |
|---|---|---|---|---|---|
| | | Sample No.1 (g) | Sample No.2 (g) | | |
| Soil+3% Lime | 10 | 18.8 | 18 | 18.4 | 84 |
| Soil+6% Lime | 10 | 15 | 14.6 | 14.8 | 48 |
| Soil+9% Lime | 10 | 14 | 14 | 14 | 40 |

Table B.2:Atterberg limit test result of pit 1 soil with 3% Lime

| | Liquid Limit (%) | | | Plastic Limit (%) | |
|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 |
| Container No | 1 | A4 | 71 | 5 | MD |
| Mass of container, g | 15.74 | 15.32 | 13.32 | 15.78 | 14.44 |
| Mass of container + Wet soil, g | 37.17 | 29.28 | 30.01 | 64.23 | 68.54 |
| Mass of container + Dry soil, g | 27.97 | 23.05 | 22.20 | 50.95 | 52.94 |
| Mass of water, g | 9.20 | 6.23 | 7.81 | 13.28 | 15.60 |
| Mass of dry soil, g | 12.23 | 7.73 | 8.81 | 35.17 | 38.50 |
| Water content, % | 75.22 | 80.60 | 87.95 | 37.76 | 40.52 |
| No of blows | 31 | 25 | 19 | ------ | ------ |
| Average | 81.26 | | | 39.14 | |
| Plasticity Index (%) | 42.12 | | | | |

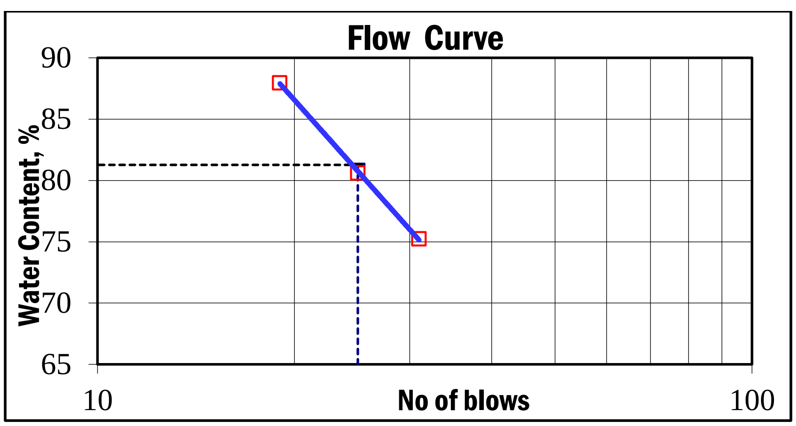



Figure B.1:Liquid limit determination of pit 1 soil with 3% Lime

Table B.3:Atterberg limit test result of pit 1 soil with 6% Lime

| | Liquid Limit (%) | | | Plastic Limit (%) | |
|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 |
| Container No | 5 | A4 | 3 | 5 | MD |
| Mass of container, g | 14.42 | 15.78 | 15.60 | 15.89 | 14.14 |
| Mass of container + Wet soil, g | 30.32 | 36.06 | 29.36 | 64.57 | 65.83 |
| Mass of container + Dry soil, g | 24.03 | 27.74 | 23.66 | 50.54 | 50.46 |
| Mass of water, g | 6.29 | 8.32 | 5.70 | 14.03 | 15.37 |
| Mass of dry soil, g | 9.61 | 11.96 | 8.06 | 34.65 | 36.32 |
| Water content, % | 65.45 | 69.57 | 70.72 | 40.49 | 42.32 |
| No of blows | 32 | 27 | 15 | ------ | ------ |
| Average | 68.58 | | | 41.40 | |
| Plasticity Index (%) | 27.18 | | | | |

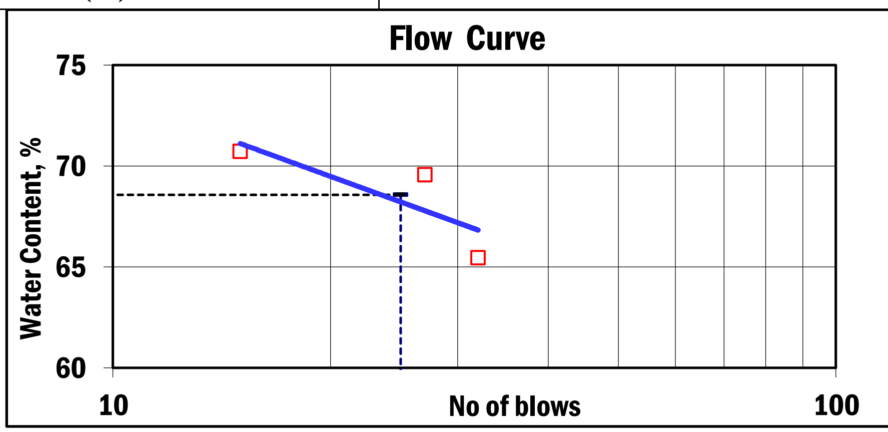



Figure B.2: Liquid limit determination of test pit 1soil with 6% Lime

Table B.4: Atterberg limit test result of pit 1 soil with 9% Lime

| | Liquid Limit (%) | | | Plastic Limit (%) | |
|---|---|---|---|---|---|
| Trial No | 1 | 2 | 3 | 1 | 2 |
| Container No | 6 | K | S2 | 2 | DR |
| Mass of container, g | 15.16 | 15.17 | 15.25 | 15.27 | 15.54 |
| Mass of container + Wet soil, g | 28.50 | 33.80 | 31.34 | 64.00 | 68.19 |
| Mass of container + Dry soil, g | 23.76 | 26.65 | 24.65 | 50.15 | 51.70 |
| Mass of water, g | 4.74 | 7.15 | 6.69 | 13.85 | 16.49 |
| Mass of dry soil, g | 8.60 | 11.48 | 9.40 | 34.88 | 36.16 |
| Water content, % | 55.12 | 62.28 | 71.17 | 39.71 | 45.60 |
| No of blows | 35 | 27 | 18 | ------ | ------ |
| Average | 62.86 | | | 42.66 | |
| Plasticity Index (%) | 20.20 | | | | |

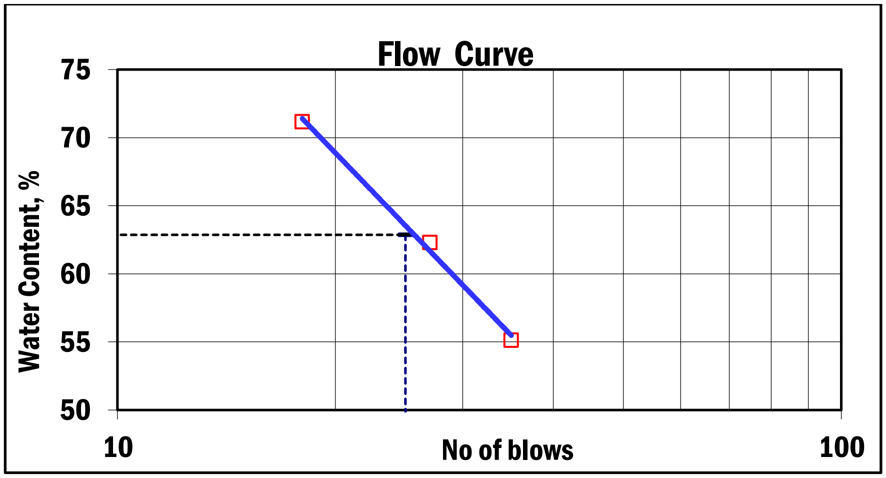



Figure B.3:Liquid limit determination of test pit 1soil with 9% Lime

Table B.5:Compaction property of pit 1 soil with 3% Lime

| Determination No. | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Mass of Mold, g | 4374 | | 4374 | | 4374 | | 4374 | |
| Mass of mold+ Compacted Soil, g | 5848 | | 6020 | | 6121 | | 6047 | |
| Mass of Compacted soil, g | 1474 | | 1646 | | 1747 | | 1673 | |
| Volume of Mold,cm3 | 944 | | 944 | | 944 | | 944 | |
| Bulk density, g/cm3 | 1.56 | | 1.74 | | 1.85 | | 1.77 | |
| MoistureContent Determination | | | | | | | | |
| Weight of can | 15.43 | 12.97 | 14.61 | 15.61 | 15.8 | 15.90 | 15.95 | 15.88 |
| weight of can +wet soil | 41.53 | 39.23 | 41.76 | 38.68 | 44.51 | 45.32 | 41.98 | 41.1 |
| weight of can +dry soil | 35.62 | 33.29 | 34.05 | 32.58 | 34.77 | 35.96 | 33.04 | 32.29 |
| Moisture content (%) | 29.27 | 29.23 | 39.66 | 35.95 | 51.34 | 46.66 | 52.31 | 53.69 |
| Average Water Content, % | 29.25 | | 37.80 | | 49.00 | | 53.00 | |
| Dry density, g/cm3 | 1.21 | | 1.27 | | 1.24 | | 1.16 | |

Table B.6:Compaction property of pit 1 soil with 6% Lime

| Determination No. | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mold, g | 4374 | 4374 | 4374 | 4374 |
| Mass of mold+ Compacted Soil, g | 5830 | 5995 | 6030 | 6021 |
| Mass of Compacted soil, g | 1456 | 1621 | 1656 | 1647 |
| Volume of Mold,cm3 | 944 | 944 | 944 | 944 |
| Bulk density, g/cm3 | 1.54 | 1.72 | 1.75 | 1.74 |
| Moisture Content Determination | | | | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Weight of can | 15.89 | 15.58 | 15.76 | 14.99 | 15.24 | 15.80 | 15.85 | 16.01 |
| weight of can +wet soil | 38.62 | 35.97 | 40.1 | 35.62 | 40.01 | 41.04 | 40.09 | 40.12 |
| weight of can +dry soil | 33.41 | 31.20 | 33.47 | 29.44 | 32.73 | 32.99 | 31.42 | 32.07 |
| Moisture content (%) | 29.74 | 30.54 | 37.44 | 42.77 | 41.62 | 46.83 | 55.68 | 50.12 |
| Average Water Content, % | 30.14 | | 40.10 | | 44.23 | | 52.90 | |
| Dry density, g/cm3 | 1.19 | | 1.23 | | 1.22 | | 1.14 | |

Table B.7: Compaction property of pit 1 soil with 9% Lime

| Determination No. | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Mass of Mold, g | 4374 | | 4374 | | 4374 | | 4374 | |
| Mass of mold+ Compacted Soil, g | 5720 | | 5809 | | 5980 | | 5970 | |
| Mass of Compacted soil, g | 1346 | | 1435 | | 1606 | | 1596 | |
| Volume of Mold,cm3 | 944 | | 944 | | 944 | | 944 | |
| Bulk density, g/cm3 | 1.43 | | 1.52 | | 1.70 | | 1.69 | |
| Moisture Content Determination | | | | | | | | |
| Weight of can | 16.45 | 15.51 | 15.57 | 15.58 | 15.78 | 14.50 | 15.43 | 15.73 |
| weight of can +wet soil | 45.18 | 49.01 | 34.77 | 40.82 | 43.86 | 47.07 | 38.61 | 38.35 |
| weight of can +dry soil | 39.28 | 41.65 | 30.92 | 33.46 | 33.85 | 39.25 | 30.50 | 31.20 |
| Moisture content (%) | 25.84 | 28.16 | 25.08 | 41.16 | 55.40 | 31.60 | 53.82 | 46.22 |
| Average Water Content, % | 27.00 | | 33.12 | | 43.50 | | 50.02 | |
| Dry density, g/cm3 | 1.12 | | 1.14 | | 1.19 | | 1.13 | |

Table B.8:Unconfined compression strength test result of pit 1 soil with 3% Lime

| **Test Pit No** | **1** | Ring Calibration Factor, kN/div | | | 0.00138 |
|---|---|---|---|---|---|
| Diameter of the sample, mm | 38 | | | | |
| Length of the sample, mm | 76 | | | | |
| Axial Deformation [mm] | Axial Strain [%] | Proving Ring Reading [div] | Axial Load [kN] | Corrected Area [m2] | Axial Stress [kPa] |
| 0 | 0.000 | 0 | 0.0000 | 0.00113 | 0 |
| 0.2 | 0.003 | 10 | 0.0138 | 0.00114 | 12.14 |
| 0.4 | 0.005 | 21 | 0.0290 | 0.00114 | 25.42 |
| 0.6 | 0.008 | 37 | 0.0511 | 0.00114 | 44.67 |
| 0.8 | 0.011 | 51 | 0.0704 | 0.00115 | 61.41 |
| 1 | 0.013 | 62 | 0.0856 | 0.00115 | 74.46 |
| 1.2 | 0.016 | 75 | 0.1035 | 0.00115 | 89.83 |
| 1.4 | 0.018 | 85 | 0.1173 | 0.00116 | 101.53 |

| | | | | | |
|---|---|---|---|---|---|
| 1.6 | 0.021 | 94 | 0.1297 | 0.00116 | 111.98 |
| 1.8 | 0.024 | 106 | 0.1463 | 0.00116 | 125.94 |
| 2 | 0.026 | 123 | 0.1697 | 0.00116 | 145.74 |
| 2.2 | 0.029 | 140 | 0.1932 | 0.00117 | 165.44 |
| 2.4 | 0.032 | 153 | 0.2111 | 0.00117 | 180.31 |
| 2.6 | 0.034 | 164 | 0.2263 | 0.00117 | 192.75 |
| 2.8 | 0.037 | 170 | 0.2346 | 0.00118 | 199.26 |
| 3 | 0.039 | 176 | 0.2429 | 0.00118 | 205.73 |
| 3.2 | 0.042 | 179 | 0.2470 | 0.00118 | 208.66 |
| 3.4 | 0.045 | 182 | 0.2512 | 0.00119 | 211.57 |
| 3.6 | 0.047 | 188 | 0.2594 | 0.00119 | 217.95 |
| 3.8 | 0.050 | 190 | 0.2622 | 0.00119 | 219.66 |
| 4 | 0.053 | 192 | 0.2650 | 0.00120 | 221.35 |
| 4.2 | 0.055 | 191 | 0.2636 | 0.00120 | 219.59 |
| 4.4 | 0.058 | 190 | 0.2622 | 0.00120 | 217.83 |
| 4.6 | 0.061 | 188 | 0.2594 | 0.00121 | 214.94 |

Table B.9:Unconfined compression strength test result of pit 1 soil with 6% Lime

| **Test Pit No** | **1** | Ring Calibration Factor, kN/div | | | 0.00138 |
|---|---|---|---|---|---|
| Diameter of the sample, mm | 38 | | | | |
| Length of the sample, mm | 76 | | | | |
| Axial Deformation [mm] | Axial Strain [%] | Proving Ring Reading [div] | Axial Load [kN] | Corrected Area [m2] | Axial Stress [kPa] |
| 0 | 0.000 | 0 | 0.0000 | 0.00113 | 0 |
| 0.2 | 0.003 | 28 | 0.0386 | 0.00114 | 33.98 |
| 0.4 | 0.005 | 43 | 0.0593 | 0.00114 | 52.05 |
| 0.6 | 0.008 | 55 | 0.0759 | 0.00114 | 66.40 |
| 0.8 | 0.011 | 67 | 0.0925 | 0.00115 | 80.68 |
| 1 | 0.013 | 80 | 0.1104 | 0.00115 | 96.07 |
| 1.2 | 0.016 | 95 | 0.1311 | 0.00115 | 113.78 |
| 1.4 | 0.018 | 111 | 0.1532 | 0.00116 | 132.59 |
| 1.6 | 0.021 | 128 | 0.1766 | 0.00116 | 152.49 |
| 1.8 | 0.024 | 145 | 0.2001 | 0.00116 | 172.28 |
| 2 | 0.026 | 160 | 0.2208 | 0.00116 | 189.59 |
| 2.2 | 0.029 | 175 | 0.2415 | 0.00117 | 206.80 |

| | | | | | |
|---|---|---|---|---|---|
| 2.4 | 0.032 | 187 | 0.2581 | 0.00117 | 220.38 |
| 2.6 | 0.034 | 198 | 0.2732 | 0.00117 | 232.71 |
| 2.8 | 0.037 | 210 | 0.2898 | 0.00118 | 246.14 |
| 3 | 0.039 | 217 | 0.2995 | 0.00118 | 253.65 |
| 3.2 | 0.042 | 225 | 0.3105 | 0.00118 | 262.28 |
| 3.4 | 0.045 | 232 | 0.3202 | 0.00119 | 269.70 |
| 3.6 | 0.047 | 237 | 0.3271 | 0.00119 | 274.75 |
| 3.8 | 0.050 | 237 | 0.3271 | 0.00119 | 273.99 |
| 4 | 0.053 | 236 | 0.3257 | 0.00120 | 272.08 |
| 4.2 | 0.055 | 233 | 0.3215 | 0.00120 | 267.88 |

Table B.10: Unconfined compression strength test result of pit 1 soil with 9% Lime

| **Test Pit No** | **1** | Ring Calibration Factor, kN/div | | | 0.00138 |
|---|---|---|---|---|---|
| Diameter of the sample, mm | 38 | | | | |
| Length of the sample, mm | 76 | | | | |
| Axial Deformation [mm] | Axial Strain [%] | Proving Ring Reading [div] | Axial Load [kN] | Corrected Area [m2] | Axial Stress [kPa] |
| 0 | 0.000 | 0 | 0.0000 | 0.00113 | 0 |
| 0.2 | 0.003 | 22 | 0.0304 | 0.00114 | 26.70 |
| 0.4 | 0.005 | 35 | 0.0483 | 0.00114 | 42.37 |
| 0.6 | 0.008 | 48 | 0.0662 | 0.00114 | 57.95 |
| 0.8 | 0.011 | 61 | 0.0842 | 0.00115 | 73.45 |
| 1 | 0.013 | 75 | 0.1035 | 0.00115 | 90.07 |
| 1.2 | 0.016 | 85 | 0.1173 | 0.00115 | 101.81 |
| 1.4 | 0.018 | 94 | 0.1297 | 0.00116 | 112.28 |
| 1.6 | 0.021 | 109 | 0.1504 | 0.00116 | 129.85 |
| 1.8 | 0.024 | 118 | 0.1628 | 0.00116 | 140.20 |
| 2 | 0.026 | 130 | 0.1794 | 0.00116 | 154.04 |
| 2.2 | 0.029 | 139 | 0.1918 | 0.00117 | 164.26 |
| 2.4 | 0.032 | 143 | 0.1973 | 0.00117 | 168.53 |
| 2.6 | 0.034 | 147 | 0.2029 | 0.00117 | 172.77 |
| 2.8 | 0.037 | 150 | 0.2070 | 0.00118 | 175.81 |
| 3 | 0.039 | 155 | 0.2139 | 0.00118 | 181.18 |
| 3.2 | 0.042 | 159 | 0.2194 | 0.00118 | 185.35 |
| 3.4 | 0.045 | 164 | 0.2263 | 0.00119 | 190.65 |
| 3.6 | 0.047 | 166 | 0.2291 | 0.00119 | 192.44 |
| 3.8 | 0.050 | 164 | 0.2263 | 0.00119 | 189.60 |
| 4 | 0.053 | 163 | 0.2249 | 0.00120 | 187.92 |

Table B.11:CBR and swell test result of pit 1 soil

| Sample type | Disturbed | | | |
|---|---|---|---|---|
| Penetration, mm | Load dial divs | Ring Factor N/div | Load (KN) | |
| 0 | 0 | 46.76 | 0.00 | |
| 0.64 | 2 | " | 0.09 | |
| 1.27 | 3.5 | " | 0.16 | |
| 1.91 | 4.5 | " | 0.21 | |
| 2.54 | 6 | " | 0.28 | |
| 3.18 | 7 | " | 0.33 | |
| 4.45 | 7.5 | " | 0.35 | |
| 5.08 | 8 | " | 0.37 | |
| 7.62 | 8.5 | " | 0.40 | |
| Penetration, mm | Load (KN) | Standard load,KN | CBR % | % Swell |
| 2.54 | 0.28 | 13.2 | 1.74 | 5.86 |
| 5.08 | 0.37 | 20 | 1.65 | |

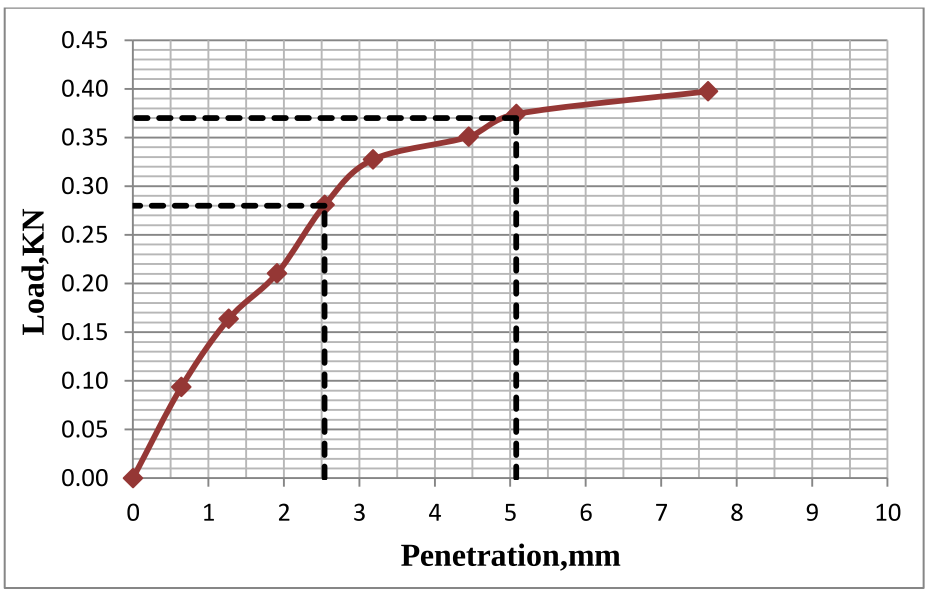



Figure B.4:Stress Vs penetration curve of soaked CBR for pit 1 Soil

Table B.12:CBR and swell test result of pit 1 soil with 6% lime

| Sample type | Disturbed | | | |
|---|---|---|---|---|
| Penetration, mm | Load dial divs | Ring Factor N/div | Load (KN) | |
| 0 | 0 | 46.76 | 0.00 | |
| 0.64 | 6 | " | 0.28 | |
| 1.27 | 15 | " | 0.70 | |
| 1.91 | 33 | " | 1.54 | |
| 2.54 | 42 | " | 1.96 | |
| 3.18 | 51 | " | 2.38 | |
| 4.45 | 53 | " | 2.48 | |
| 5.08 | 57 | " | 2.67 | |
| 7.62 | 60 | " | 2.81 | |
| Penetration,mm | Load (KN) | Standared load,KN | CBR % | % Swell |
| 2.54 | 1.96 | 13.2 | 14.88 | 1.87 |
| 5.08 | 2.67 | 20 | 13.33 | |

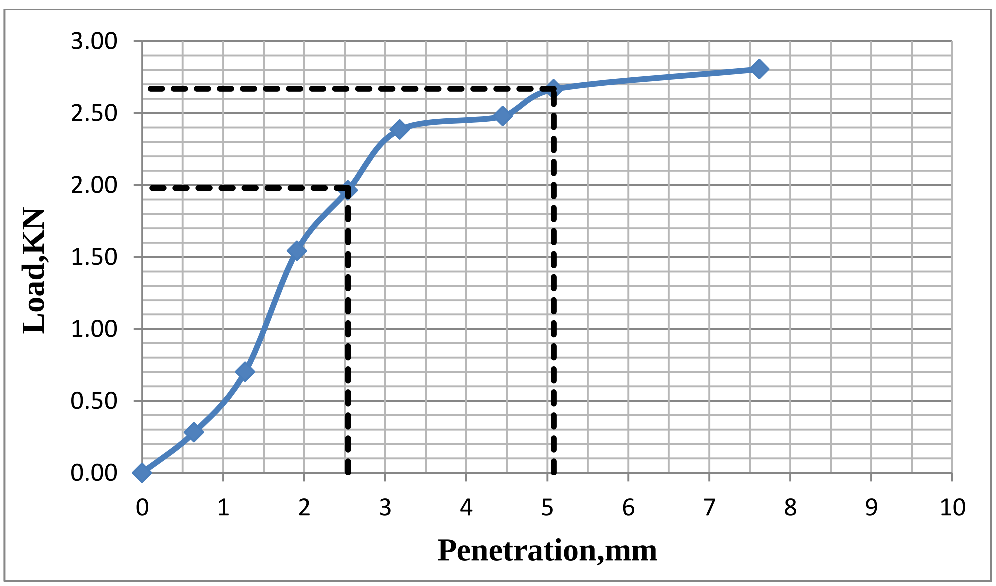



Figure B.5:Stress Vs penetration curve of soaked CBR for pit 1 soil with 6% lime

# Appendix C: Consolidation Test Results

Table C.1:Determination of pre-consolidation pressure for pit 1 natural soil

| At the beginning of the test | |
|---|---|
| Sample type : | Remolded |
| Ring Area,cm2: | 19.63 |
| Height of sample, mm: | 20 |
| Seating Load, KPa | 7 |
| Initial Void Ratio, eo: | 1.05 |
| Initial moisture content,% | 22.12 |
| Specific Gravity: | 2.70 |
| Wet density, g/cm3 | 1.61 |
| **At the end of the test** | |
| Final Moisture Content,% | 27.41 |
| Dry specimen weight (ms), gm | 106.86 |
| Dry density, g/cm3 | 1.32 |
| Height of Solids(Hs), mm | 9.756 |
| Final Void Ratio, ef: | 0.63 |

**Calculation table**

| Applied pressure (kpa) | Final dial reading (mm) | Change in specimen height | Final specimen height | Void height, hv (mm) | Void ratio, e |
|---|---|---|---|---|---|
| 7 | 8.101 | 0.00 | 20.00 | 10.24 | 1.05 |
| 25 | 7.878 | -0.22 | 19.78 | 10.02 | 1.03 |
| 50 | 7.558 | -0.54 | 19.46 | 9.70 | 0.99 |
| 100 | 7.101 | -1.00 | 19.00 | 9.24 | 0.95 |
| 200 | 6.461 | -1.64 | 18.36 | 8.60 | 0.88 |
| 400 | 5.751 | -2.35 | 17.65 | 7.89 | 0.81 |
| 800 | 4.951 | -3.15 | 16.85 | 7.09 | 0.73 |
| 1600 | 4.051 | -4.05 | 15.95 | 6.19 | 0.63 |

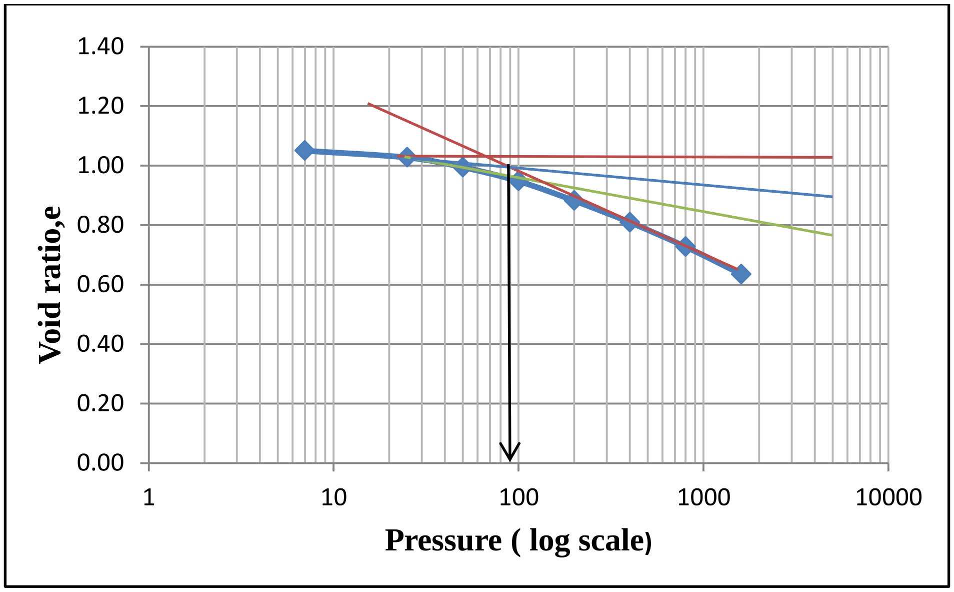



Figure C.1:Consolidation Pressure Versus void ratio curve of pit 1 natural soil.

Table C.2:Determination of pre-consolidation pressure of pit 1 soil + 6% Lime

| **At the beginning of the test** | |
|---|---|
| Sample type : | Remolded |
| Ring Area,cm2: | 19.63 |
| Height of sample, mm: | 20 |
| Seating Load, KPa | 7 |
| Initial Void Ratio, eo: | 0.86 |
| Initial moisture content,% | 17.84 |
| Specific Gravity: | 2.70 |
| Wet density, g/cm3 | 1.71 |
| **At the end of the test** | |
| Final Moisture Content,% | 20.17 |
| Dry specimen weight (ms), gm | 110.37 |
| Dry density, g/cm3 | 1.45 |
| Height of Solids(Hs), mm | 10.753 |
| Final Void Ratio, ef: | 0.32 |

**Calculation table**

| Applied pressure (kpa) | Final dial reading (mm) | Change in specimen height | Final specimen height | Void height, hv (mm) | Void ratio, e |
|---|---|---|---|---|---|
| 7 | 8.801 | 0.00 | 20.00 | 9.25 | 0.86 |
| 25 | 8.678 | -0.12 | 19.88 | 9.12 | 0.85 |
| 50 | 8.176 | -0.63 | 19.38 | 8.62 | 0.80 |
| 100 | 7.427 | -1.37 | 18.63 | 7.87 | 0.73 |
| 200 | 6.401 | -2.40 | 17.60 | 6.85 | 0.64 |
| 400 | 5.351 | -3.45 | 16.55 | 5.80 | 0.54 |
| 800 | 4.251 | -4.55 | 15.45 | 4.70 | 0.44 |
| 1600 | 2.981 | -5.82 | 14.18 | 3.43 | 0.32 |

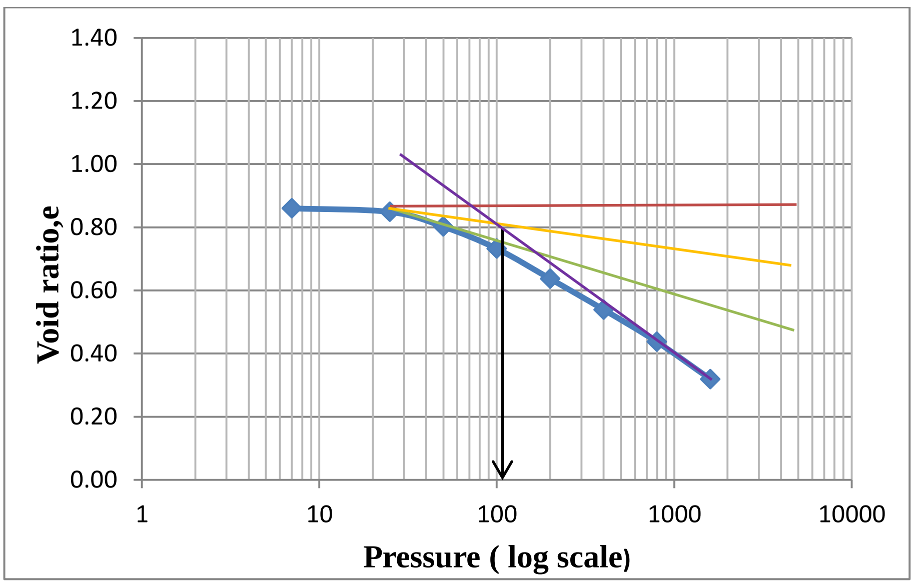



Figure C.2:Consolidation Pressure Versus Void ratio curve of pit 1 natural soil

# Appendix D: Cyclic Simple Shear Test Result

Table D.1: Typical tabulation of Shear stress and shear strain values of natural soil for 1 % strain under 100 kPa, at the 5th cycle

| Cycle No | Time (sec) | Lateral Lvdt | Lateral Force | sample area | shear stress(γ) =lateral Lvdt/height of the specimen | Shear strain τ = (Lateral Force / area of specimen ) * $10^3$ MPa |
|---|---|---|---|---|---|---|
| 5 | 0 | -0.82118 | -0.13377 | 3848.45 | -0.03476 | -0.04278 |
| | 0.019 | -0.82373 | -0.13236 | 3848.45 | -0.03439 | -0.04291 |
| | 0.038 | -0.82305 | -0.13149 | 3848.45 | -0.03417 | -0.04288 |
| | 0.057 | -0.82478 | -0.13026 | 3848.45 | -0.03385 | -0.04297 |
| | 0.076 | -0.82350 | -0.12956 | 3848.45 | -0.03366 | -0.04290 |
| | 0.095 | -0.82328 | -0.12922 | 3848.45 | -0.03358 | -0.04289 |
| | 0.114 | -0.82463 | -0.12790 | 3848.45 | -0.03324 | -0.04296 |
| | 0.133 | -0.82718 | -0.12706 | 3848.45 | -0.03302 | -0.04309 |
| | 0.152 | -0.83998 | -0.12566 | 3848.45 | -0.03265 | -0.04376 |
| | 0.171 | -0.82845 | -0.12393 | 3848.45 | -0.03220 | -0.04316 |
| | 0.19 | -0.82545 | -0.12160 | 3848.45 | -0.03160 | -0.04300 |
| | 0.209 | -0.76343 | -0.10153 | 3848.45 | -0.02638 | -0.03977 |
| | 0.228 | -0.57825 | -0.05793 | 3848.45 | -0.01505 | -0.03013 |
| | 0.247 | -0.37380 | -0.01148 | 3848.45 | -0.00298 | -0.01947 |
| | 0.266 | -0.17873 | 0.02778 | 3848.45 | 0.00722 | -0.00931 |
| | 0.285 | 0.05166 | 0.05466 | 3848.45 | 0.01420 | 0.00269 |
| | 0.304 | 0.21868 | 0.07546 | 3848.45 | 0.01961 | 0.01139 |
| | 0.323 | 0.35490 | 0.09097 | 3848.45 | 0.02364 | 0.01849 |
| | 0.342 | 0.48104 | 0.10304 | 3848.45 | 0.02677 | 0.02506 |
| | 0.361 | 0.58737 | 0.11404 | 3848.45 | 0.02963 | 0.03060 |
| | 0.38 | 0.68453 | 0.12256 | 3848.45 | 0.03185 | 0.03566 |
| | 0.399 | 0.77336 | 0.13084 | 3848.45 | 0.03400 | 0.04029 |
| | 0.418 | 0.80927 | 0.13658 | 3848.45 | 0.03549 | 0.04216 |
| | 0.437 | 0.83706 | 0.13972 | 3848.45 | 0.03631 | 0.04361 |
| | 0.456 | 0.85029 | 0.14053 | 3848.45 | 0.03652 | 0.04430 |
| | 0.475 | 0.85470 | 0.13952 | 3848.45 | 0.03625 | 0.04453 |
| | 0.494 | 0.85932 | 0.13888 | 3848.45 | 0.03609 | 0.04477 |
| | 0.513 | 0.85652 | 0.13737 | 3848.45 | 0.03569 | 0.04462 |
| | 0.532 | 0.85673 | 0.13656 | 3848.45 | 0.03548 | 0.04463 |
| | 0.551 | 0.85813 | 0.13530 | 3848.45 | 0.03516 | 0.04471 |
| | 0.57 | 0.86072 | 0.13404 | 3848.45 | 0.03483 | 0.04484 |
| | 0.589 | 0.86128 | 0.13364 | 3848.45 | 0.03473 | 0.04487 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | 0.608 | 0.86471 | 0.13261 | 3848.45 | 0.03446 | 0.04505 |
| | 0.627 | 0.86373 | 0.13115 | 3848.45 | 0.03408 | 0.04500 |
| | 0.646 | 0.86268 | 0.12981 | 3848.45 | 0.03373 | 0.04494 |
| | 0.665 | 0.86352 | 0.12813 | 3848.45 | 0.03329 | 0.04499 |
| | 0.684 | 0.85911 | 0.12519 | 3848.45 | 0.03253 | 0.04476 |
| | 0.703 | 0.84686 | 0.11900 | 3848.45 | 0.03092 | 0.04412 |
| | 0.722 | 0.78710 | 0.05127 | 3848.45 | 0.01332 | 0.04101 |
| | 0.741 | 0.76549 | -0.00784 | 3848.45 | -0.00204 | 0.03988 |
| | 0.76 | 0.69838 | -0.03284 | 3848.45 | -0.00853 | 0.03638 |
| | 0.779 | -0.09630 | -0.05821 | 3848.45 | -0.01513 | -0.00502 |
| | 0.798 | -0.26843 | -0.07932 | 3848.45 | -0.02061 | -0.01398 |
| | 0.817 | -0.40208 | -0.09565 | 3848.45 | -0.02485 | -0.02095 |
| | 0.836 | -0.51090 | -0.10814 | 3848.45 | -0.02810 | -0.02662 |
| | 0.855 | -0.59843 | -0.11838 | 3848.45 | -0.03076 | -0.03118 |
| | 0.874 | -0.67050 | -0.12460 | 3848.45 | -0.03238 | -0.03493 |
| | 0.893 | -0.71708 | -0.13040 | 3848.45 | -0.03388 | -0.03736 |
| | 0.912 | -0.75788 | -0.13280 | 3848.45 | -0.03451 | -0.03948 |
| | 0.931 | -0.78788 | -0.13476 | 3848.45 | -0.03502 | -0.04105 |

Table D.2: Typical tabulation of Shear stress and shear strain values of treated soil for 1 % strain under 100 kPa, at the 5th cycle

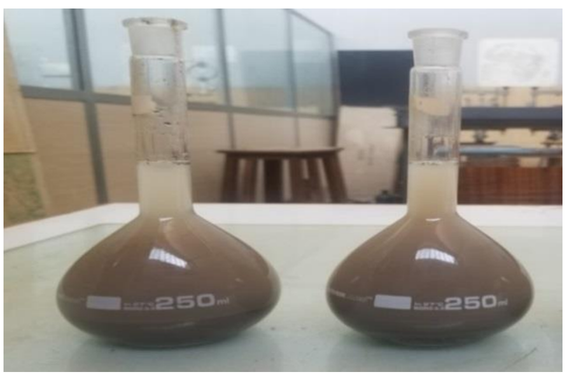
| **Cycle No** | **Time** | **Lateral Lvdt** | **Lateral Force** | **sample area** | **shear stress(γ) =lateral Lvdt/height of the specimen** | **Shear strain τ =(Lateral Force/ area of the specimen)* $10^3$ MPa** |
|---|---|---|---|---|---|---|
| 5 | 0 | -0.57483 | -0.16855 | 3848.45 | -0.04380 | -0.02995 |
| | 0.019 | -0.57661 | -0.16677 | 3848.45 | -0.04334 | -0.03004 |
| | 0.038 | -0.57614 | -0.16568 | 3848.45 | -0.04305 | -0.03001 |
| | 0.057 | -0.57735 | -0.16413 | 3848.45 | -0.04265 | -0.03008 |
| | 0.076 | -0.57645 | -0.16325 | 3848.45 | -0.04242 | -0.03003 |
| | 0.095 | -0.57630 | -0.16282 | 3848.45 | -0.04231 | -0.03002 |
| | 0.114 | -0.57724 | -0.16115 | 3848.45 | -0.04188 | -0.03007 |
| | 0.133 | -0.57903 | -0.16010 | 3848.45 | -0.04160 | -0.03017 |
| | 0.152 | -0.59499 | -0.15833 | 3848.45 | -0.04114 | -0.03100 |
| | 0.171 | -0.60792 | -0.15615 | 3848.45 | -0.04058 | -0.03167 |
| | 0.19 | -0.61786 | -0.15322 | 3848.45 | -0.03981 | -0.03219 |
| | 0.209 | -0.53440 | -0.12793 | 3848.45 | -0.03324 | -0.02784 |
| | 0.228 | -0.40478 | -0.07299 | 3848.45 | -0.01897 | -0.02109 |
| | 0.247 | -0.29596 | -0.01446 | 3848.45 | -0.00376 | -0.01542 |

| | | | | | | |
|---|---|---|---|---|---|---|
| | 0.266 | -0.12511 | 0.03500 | 3848.45 | 0.00910 | -0.00652 |
| | 0.285 | 0.03616 | 0.06887 | 3848.45 | 0.01790 | 0.00188 |
| | 0.304 | 0.15308 | 0.09508 | 3848.45 | 0.02471 | 0.00797 |
| | 0.323 | 0.24843 | 0.11462 | 3848.45 | 0.02978 | 0.01294 |
| | 0.342 | 0.33673 | 0.12983 | 3848.45 | 0.03374 | 0.01754 |
| | 0.361 | 0.41116 | 0.14369 | 3848.45 | 0.03734 | 0.02142 |
| | 0.38 | 0.47917 | 0.15443 | 3848.45 | 0.04013 | 0.02496 |
| | 0.399 | 0.54135 | 0.16486 | 3848.45 | 0.04284 | 0.02820 |
| | 0.418 | 0.56649 | 0.17209 | 3848.45 | 0.04472 | 0.02951 |
| | 0.437 | 0.58594 | 0.17605 | 3848.45 | 0.04574 | 0.03053 |
| | 0.456 | 0.59520 | 0.16447 | 3848.45 | 0.04274 | 0.03101 |
| | 0.475 | 0.59829 | 0.17580 | 3848.45 | 0.04568 | 0.03117 |
| | 0.494 | 0.60152 | 0.17499 | 3848.45 | 0.04547 | 0.03134 |
| | 0.513 | 0.59956 | 0.17309 | 3848.45 | 0.04498 | 0.03124 |
| | 0.532 | 0.59971 | 0.17207 | 3848.45 | 0.04471 | 0.03124 |
| | 0.551 | 0.60069 | 0.17048 | 3848.45 | 0.04430 | 0.03129 |
| | 0.57 | 0.60250 | 0.16889 | 3848.45 | 0.04389 | 0.03139 |
| | 0.589 | 0.60290 | 0.16839 | 3848.45 | 0.04375 | 0.03141 |
| | 0.608 | 0.60530 | 0.16709 | 3848.45 | 0.04342 | 0.03153 |
| | 0.627 | 0.61161 | 0.16525 | 3848.45 | 0.04294 | 0.03186 |
| | 0.646 | 0.61788 | 0.16356 | 3848.45 | 0.04250 | 0.03219 |
| | 0.665 | 0.62546 | 0.16144 | 3848.45 | 0.04195 | 0.03258 |
| | 0.684 | 0.66438 | 0.15774 | 3848.45 | 0.04099 | 0.03461 |
| | 0.703 | 0.59280 | 0.14994 | 3848.45 | 0.03896 | 0.03088 |
| | 0.722 | 0.55097 | 0.06460 | 3848.45 | 0.01679 | 0.02870 |
| | 0.741 | 0.53584 | -0.00988 | 3848.45 | -0.00257 | 0.02792 |
| | 0.76 | 0.48887 | -0.04138 | 3848.45 | -0.01075 | 0.02547 |
| | 0.779 | -0.06741 | -0.07334 | 3848.45 | -0.01906 | -0.00351 |
| | 0.798 | -0.18790 | -0.09994 | 3848.45 | -0.02597 | -0.00979 |
| | 0.817 | -0.28146 | -0.12052 | 3848.45 | -0.03132 | -0.01466 |
| | 0.836 | -0.35763 | -0.13626 | 3848.45 | -0.03541 | -0.01863 |
| | 0.855 | -0.41890 | -0.14916 | 3848.45 | -0.03876 | -0.02182 |
| | 0.874 | -0.46935 | -0.15700 | 3848.45 | -0.04079 | -0.02445 |
| | 0.893 | -0.50196 | -0.16430 | 3848.45 | -0.04269 | -0.02615 |
| | 0.912 | -0.53052 | -0.16733 | 3848.45 | -0.04348 | -0.02764 |
| | 0.931 | -0.55152 | -0.16980 | 3848.45 | -0.04412 | -0.02873 |

# Appendix E: Photos taken during the study

E.1: Soil Sample

E.2: Index Property Test



E.3: Compaction Test

E.4: Unconfined Compressive Strength Tests

E.5: CBR Tests


E.6: Consolidation Test


E.7: Cyclic Simple Shear Test