A Study on the Dynamic Property of Expansive Soil Treated With Sugarcane Bagasse Ash

By:

Abyalew Bekele Ayele

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's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies
Adama Science and Technology University

Adama, Ethiopia

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DECLARATION

I, hereby declare that this Master Thesis entitled "**A Study on the Dynamic Property of Expansive Soil Treated With Sugarcane Bagasse Ash**" is my original work and has not been submitted for the award of any academic degree in any other university. All sources of materials that are used for this thesis have also been duly acknowledged.

Abyalew Bekele Ayele		
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 "**A Study on the Dynamic Property of Expansive Soil Treated With Sugarcane Bagasse Ash**" prepared under my guidance by Abyalew Bekele Ayele submitted in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering (Specialization in Geotechnical Engineering). Therefore, I recommend the submission of the revised version of the thesis to the department following the applicable procedures.

Dr. Yadeta Chemdesa		
Name of the Advisor	Signature	Date

Approval Sheet

I, the advisors of the thesis entitled "**A Study on the Dynamic Property of Expansive Soil Treated with Sugarcane Bagasse Ash**" and developed by Abyalew Bekele, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Dr. Yadeta Ch.		
Major Advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis by Abyalew Bekele have read and evaluated the thesis entitled **" Study on the Dynamic Property of Expansive Soil Treated With Sugarcane Bagasse Ash**" and examined the candidate during 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.

Chairperson	Signature	Date
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Final approval and acceptance of the thesis is contingent upon submission of its final copy to the Office of Postgraduate Studies (OPGS) through the Department Graduate Council (DGC) and School Graduate Committee (SGC)

Department Head	Signature	Date
Postgraduate Dean	Signature	Date
Office of Postgraduate Studies, Dean	Signature	Date

DEDICATION

It is with genuine gratitude and warm regard that I dedicate this work to my beloved mother who have meant and continue to mean so much to me, W/ro Meazashegenet Belaye Hailu.

Your love for me knows no boundary and you have been a constant source of support and encouragement throughout my life. Loosing you left me a void never to be filled in my live. Though you are not with me, I will make sure your memory lives as long as I shall live. I love you beyond words and miss you every single day of my life. May you find peace and happiness in paradise.

ACKNOWLEDGEMENT

I would like to thank God for granting me the courage, strength and health in order to finish this research. I also want to extend my genuine gratitude to my advisor Dr. Yadeta Chemdesa for his valuable advice, and critical comments from the beginning to the end of the thesis.

Table of Contents

List of Tables

List of Figures

List of Acronyms and Abbreviation

AASHTO	American Association of States Highways and Transport
ASTM	American Society for Testing Material;
ERA	Ethiopian Road Authority;
g	gram;
gm/cm^3 or g/cc	Gram per Centimeter Cube;
Gs	Specific Gravity;
Kpa	Kilopascal;
LL	Liquid Limit;
MDD	Maximum Dry Density;
mm	Millimeter;
NMC	Natural Moisture Content;
OMC	Optimum Moisture Content;
PAI	pozzolanic activity index
PI	Plasticity Index;
PL	Plastic Limit;
SCBA	sugarcane bagasse ash
UCS	Unconfined Compressive Strength;
USCS	Unified Soil Classification System;
USBR	Us bureau of reclamation
WAC	Water Absorption Capacity, and
Wt	Weight

Abstract

Expansive soil possesses high plasticity, and shrinkage & swelling characteristics that varies with moisture content. When used for construction, expansive soil poses a problem due to this characteristics. The recent advancement in geotechnical engineering is the use of additives that could avoid or reduce this situations. Besides, when exposed to dynamic excitations, expansive soil losses the small shear strength it poses and causes additional problem. The properties of the soil related to dynamic excitation are known as dynamic soil properties.

In this study, experimental program were conducted using cyclic simple shear test to better understand the dynamic properties of expansive soil treated with sugarcane bagasse ash (SCBA). The influencing factors, including strain amplitude, confining pressure, cycle number and the effect of SCBA treatment on the dynamic properties were discussed. The analysis results showed that SCBA considerably reduces the plasticity characteristics of the soil and increases the unconfined compressive strength (UCS) up to 9% addition by dry mass. The addition of SCBA not only improves the plasticity and UCS but also increases the dynamic shear modulus and the damping ratio. It is also observed that dynamic shear modulus increases with increment of confining pressure; while it decreases for the increase in number of cycles and strain. In the case of damping ratio, it decreases with number of cycles. However, it increases with confining pressure and shear strain rate.

Keyword: **Expansive soil, Sugarcane bagasse ash, Dynamic soil property, Shear modulus, Damping ratio,**

CHAPTER ONE

INTRODUCTION

1.1 Background

Expansive soils, which are also called black cotton soils, are found in most part of Ethiopia. These are the type of soils that are basically susceptible to damaging volumetric changes, with change in moisture content. To improve these properties, the common method followed is stabilization. Expansive soils are also known for covering some of the earthquake prone areas and numerous facilities on expansive soils imposes dynamic load due to operation of machinery, construction operations, traffic, mining, wind, and wave actions. During dynamic loading, the soft clay will lose the small shear strength it possesses which causes cracks and damage to the structures laying on it (G. Lanzo & M. Vucetic, 1999). Thus understanding of a treated expansive soil behavior under dynamic loading is of essential necessity.

Generally, it is also essential to know the ground response of soil deposits in addition to the requirement of strict structural earthquake design guidelines. In order to study problems involving soil -structure interactions, site responses of soil deposits, and prediction of the ground motion it is important to know the dynamic properties of soil deposits. In soil dynamics the fundamental parameters used to describe deformation characteristics of a soil are in terms of shear modulus and damping ratio (Kramer, 1996). These parameters are used to estimate the response of soil and soil-structure system to cyclic and dynamic loading, caused by earthquake, machine vibration, vehicular traffic, blasting and etc.

A wide variety of field and laboratory techniques are available, each with different advantages and limitation with respect to different problems. Laboratory tests generally provide the most direct means of evaluating dynamic soil parameters for seismic analyses (Abu, 2011). These laboratory tests are classified as High strain range, and Low strain range test. Since the soil is nonlinear material it could be tested under wide range of strain. It is possible to test the soils using cyclic simple shear test as it classified under high strain range (Abu, 2011).

Research had been done on Koka sand, Kolfe Keranio area of Addis Ababa on red clay soils, Gulelle area red brown clay soils (Natnael , 2020), Dessie town, Ziway town (Abraham, 2014), Adama town (Abu, 2011), Arbaminch town (Mengesha , 2013), Bishoftu town and Mojo town to determine the shear modulus and damping ratio of the natural soil in this areas and to assess the response behavior of these soil to cyclic loading. However, researches on the dynamic property of stabilized soil, particularly on a soil treated with bagasse ash are rare to find in Ethiopia.

In this study the behavior of expansive soil treated with sugarcane bagasse ash when they are subjected to dynamic loading have been investigated using a cyclic simple shear testing device. For the study a soil found from Gelan area of Akaki Kality sub city in Addis Ababa is used.

1.2 Statement of the problem

Engineering problems due to expansive soils have been reported in many countries all over the world costing millions of dollars due to several damage of structures (Tibebu, 2015). Since expansive soils usually exhibit high swelling-shrinkage behavior and compressibility, they are generally unsuitable for construction works. These behaviors are mainly governed by the amount and type of clay minerals present in the soil. Hence, it is a common practice to use chemical additives to alter these undesirable properties and made the clayey soil suitable in construction activities.

Expansive soils when exposed to dynamic loading will also lose the small shear strength it possess and damages will propagate in the form of crack, settlement, and lose of stability to the structure constructed on it (Vucetic, 1992). The loading parameters which govern the cyclic response of soil are those that govern the distortion or deformation of the soil skeleton (Vucetic, 1992). The distortion due to dynamic load is mainly expressed in terms of shear strain which is directly responsible for the breakage of particle bonds, slippage at the particle contacts, corresponding change of microstructural repulsion force and tendency towards volume change which cause pore pressure variations (Vucetic, 1992). Thus an understanding of expansive soil behavior under different dynamic loading parameters is of essential necessity for a broader range of use.

The chemical stabilization of expansive soils has been thoroughly studied and well understood. When soil treated with chemical additives such as lime, cement, etc. in presence of moisture, the swelling potential and plasticity decrease in the short-term, and mechanical strength increases in the long-term (Wesley G. Holtz, 1956). In contrast, the effect of chemical treatment on dynamic properties of expansive soils were not sufficiently explored yet, particularly in Ethiopia. In other word, there is a gap in studying the dynamic property of stabilized soil and the reaction of stabilized soil for cyclic loading which if studied will benefit a lot in the application of soil stabilization in different areas. This study, therefore, aimed to fill this gap.

1.3 Objective

The main objective of this research is to conduct a study on the effect of adding sugarcane bagasse ash in the dynamic properties (shear modulus and damping) of an expansive soil.

Specific objectives

- To determine the index properties and classify the soil.
- To determine the optimum percentile amount of sugarcane bagasse ash for the treatment of the soil obtained from the study area.
- To study the effect of adding sugarcane bagasse ash in the improvement of the shear modulus and damping ratio of the expansive soil
- To study the factors that influence the dynamic properties of soils on the sugarcane bagasse ash treated expansive soil of the study area.

1.4 Scope and Limitation of the Study

The scope of this study is focused on determining the optimum amount of sugarcane bagasse ash that could improve the index properties and strength capacity of the black cotton soil that is obtained from the study area, and evaluating the effect of the optimum amount of sugarcane bagasse ash that is added to the expansive soil on the effect of the dynamic properties of the soil. Even though there exist many factors that have contribution on the dynamic property of a soil only effective confining pressure, strain amplitude, and number of loading cycle are investigated in this research.

1.5 Organization of the Thesis

The thesis involves six chapters. Chapter one gives a brief description of the thesis background, statement of the problem, objectives, scope and limitations of the work that has been done. The second chapter discuses literatures regarding properties of expansive soil, the use of sugarcane bagasse ash for the treatment of expansive soil, soil dynamic properties, and previous researches that has been done on dynamic property analysis of chemically treated expansive soil. The third chapter gives a brief description of material and metrology. The fourth chapter presents results and discussions of the tests conducted during the course of the research. Chapter five presents the conclusions and the recommendations made regarding the study. The last chapter contains references that are used for this study purpose.

CHAPTER TWO

LITERATURE REVIEW

2.1 Expansive Soils

The physical properties of soil are mostly governed by size, shape, and chemical composition of the grains. Mechanical and chemical process of breaking down of rock to a smaller pieces is the cause for the production of solid phase of soil. Due to these weathering actions a wide range of size of individual grains will exist within the soil. Particle sizes that range from cobble, gravel, sand, silt, and up to clay. Gravel and sand are the coarse fractions and they are considered inert materials because of their insignificant activity. In contrast, clay and silty clays are particles of ultra-fine size in the form of platelets. They carry an unbalanced negative electric charge on their surface. This electric charge and large specific surface they possess render them highly active. They can absorb water as well as the positively-charged ions from the salts in water to neutralize the electric charge they carry on their surface (Tibebu, 2015).

Clay mineral structures are fundamentally formed from two building blocks. They are the tetrahedral unit and the octahedral units. The octahedral units forms a sheet structure bounding together with hydroxyl ion common to three octahedral units. The sheet that is formed is called gibbsite sheet (Murthy, 2009). Furthermore the combination of this gibbsite with two sheets of silicate in different arrangement and conditions lead to the formation of different clay minerals. These clay minerals can be grouped to Kaolinite Group, Montmorillonite Group, and Illite Group on the basis of their crystalline arrangements.

The basic structural unit in kaolinite group consists of an alumina sheet combined with silica sheet tips. The silica sheet and one base of the alumina sheet from a common interface, the structural units joined together by hydrogen bond, which develops between the oxygen of silica sheet and the hydroxyls of alumina sheet as the bond is fairly strong, the mineral is stable moreover, water cannot easily enter between the structural unit and cause expansion (Wesley G. Holtz, 1956). In montmorillonite group of minerals the basic structural unit consists of an alumina sheet sandwiched between two silica sheets, successive structural units are stacked one over another. The two successive structural units are joined together by a link between oxygen ions of the two-silica sheets. The inter sheet bonding is due to mainly to van der Waals force and thus, very week relative to hydrogen or other ion bonding and dipolar molecules of water can enter between the sheets causing them to expand readily (Wesley G. Holtz, 1956). The illite clay mineral group consists of an octahedral layer of gibbsite sandwiched between two layers of silica tetrahedral. This produces a 1: 2 mineral with additional difference that some of the silica positions are filled with aluminum atoms and potassium ions are attached between layers to make up the charge deficiency (Wesley G. Holtz, 1956). The bonding results in a less stable condition than for kaolinite, and thus activity of illite is greater than kaolinite but lower than montmorillonite.

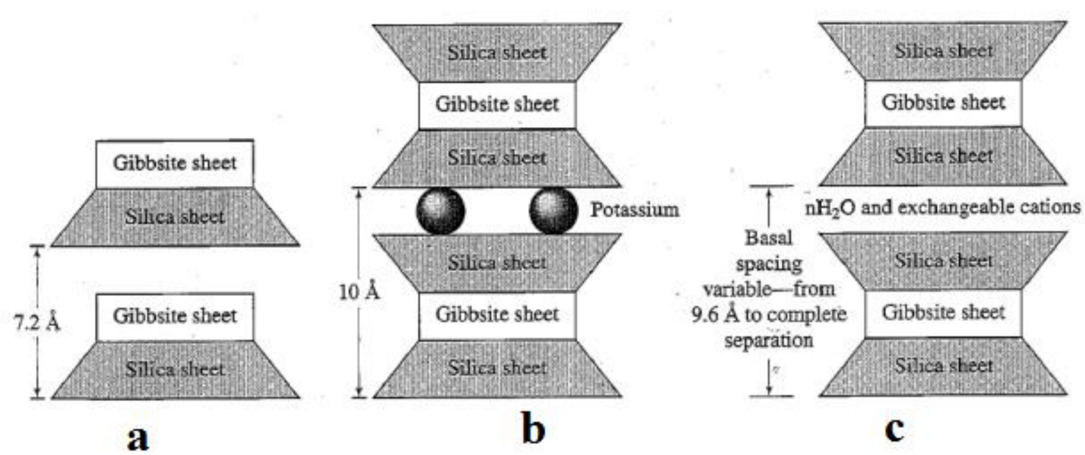


Figure 1 : Clay minerals (a) Kaolinite, (b) Illite, (c) Montmorillonite *(Das B. M., 2010)*

Swelling soil, which are clayey soil are also called expansive soils (Murthy, 2009). Commonly they are known as black clays or in some regions as *"black cotton"* soils. The name *black cotton* came from the fact that the soils are found favorable in some regions for growing cotton (Alemayehu Teffera, 1999). These soils possess a high plasticity index. Their color varies from dark grey to black. It is easy to recognize these soil in the field during either dry or wet seasons. Shrinkage cracks are visible on the ground surface during dry seasons. The maximum width of these cracks may be up to 20mm or more and they travel deep in to the ground up to 1.5m (Murthy, 2009).

The parent materials associated with expansive soils are either basic igneous rock or sedimentary rocks with high clay content. Basalt, dolerlite sills and dykes, gabros and norites are the basic parent igneous rock of expansive soil. Among sedimentary rocks, shales, marles and limestones are the basic parent materials of expansive soils (Hiwot, 2016). In order of increasing expansiveness clay minerals are listed as Kaolinite, Illite, and Montmorillonite. The clay mineral that is mostly responsible for expansiveness of a soil belongs to the Montmorillonite group.

Montmorillonite is most common of all the clay minerals and is well known for its swelling properties. This mineral is composed of two silica tetrahedral sheets with a central aluminal octahedral sheet. The alumina sheet is sandwiched between two silica sheets. The bond between the individual units is very weak and there exist a cleavage so that water is easily able to penetrate between the sheets and cause their separation, structure breaks, and hence swelling (Alemayehu Teffera, 1999).

2.1.1 Identification of Expansive soil

It was evident that expansive soil deposits can be recognized in the field through visual inspections (Dinke, 2015). A shiny surface was easily obtained when a partially dry piece of the soil have been polished with a smooth object such as the top of a finger nail. The wet samples of the soil are sticky and it will be relatively difficult to clean the soil from the hands. They usually have a color of black and gray. In the regions where there were seasonal moisture variation, open or closed fissures, slickenside (highly polished or glossy fissure surface) and presence of fissures forming granular fragments of clayey soils (Shattering or micro-shattering) occurs (Dinke, 2015).

Identification and classification of expansive soil in the laboratory is mainly done by mineralogical classification and index tests. For mineralogical identification, X-ray diffraction, Differential Thermal Analysis and Electron microscope resolution are used (Hiwot, 2016). The index property tests used for identification of expansive soil includes grain size analysis, consistency test, and free swell. Casagrande unified soil classification system (USCS) is based on the plasticity chart shown in the figure (2) and later modified by the US Bureau of Reclamation (USBR), Holtz & Gibbs table (3). Correspondingly many criteria are simultaneously used to identify and characterize expansive soil such as plasticity index, shrinkage limit, percentage smaller than 0.001mm (1µ), free swelling, and percent swell under a pressure of 1psi. (Alemayehu Teffera, 1999).

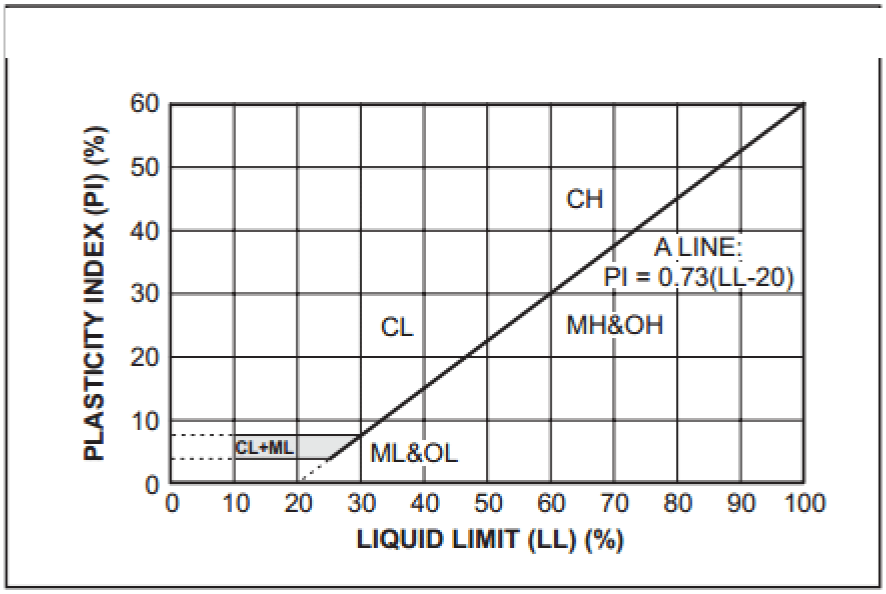


Figure 2: plasticity chart Unified Soil Classification System (Alemayehu Teffera, 1999)

Table 1 : Relation with degree of expansion and atterberg limits (Alemayehu Teffera, 1999)

No	Degree of expansion	Liquid Limit (LL) %	Shrinkage Limit (SL) %	Plastic Index (PI) %
1	Very high	60-70	>30	>35
2	High	40-60	20-30	20-35
3	Medium	30-40	10-20	10-20
4	Low	20-30	<10	<10

Index tests also consist of grain size analysis. Classification of soil based on AASHTO classification soil of which more than 35% pass through the No. 200 sieve are classified as group A-4, A-5, A-6, and A-7. This soils are mostly silt and clay type materials (Das B. M., 2010).

Table 2 : AASHTO Soil Classification System (Das B. M., 2010)

Table 5.1. AASHTO Classification System

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			

* If plasticity index is equal to or less than (liquid Limit—30), the soil is A—7—5 (*i.e.* PL > 30%)
If plasticity index is greater than (Liquid Limit—30), the soil is A—7—6 (*i.e.* PL < 30%)

Table 3 : U.S bureau of soil classification (Das B. M., 2010)

Clay (size)	Silt (size)	Sand				Fine Gravel	Gravel
		Very Fine	Fine	Medium	Coarse		
0.005		0.05	0.1	0.25	0.5	1.0	2.0

Free swell test consisting of placing a known volume of dry soil passing through the no 40 sieve into graduated cylinder filled with water and measuring the swells volume after it has completely settled, the free swell of a soil is determined as: Soils with free swell of 100% may cause damage to light structures when they become wet; soils with free swells less than 50% have been found to exhibit only small volume changes.

Another method is Skepton's method of clay soil classification which is based on their activity. Montmorilloniteic clay (expansive clay) is defined as active. Based on Skepton: Colloidal

Activity $(A) = {}^{I_p}/_{C}$ where C is percent of clay fraction finer than 2μ and I_p is the plasticity index.

Table 4 : Skepton's Method of clay Soil Classification based on activity

Degree of Activity	Activity
Inactive clay	< 0.75
Normal clay	0.75 – 1.25
Active clay	>1.25

2.1.2 Effect of expansive soils on civil engineering structures

In wet seasons, expansive soil will absorb water and swell up; as a result, the whole ground level rises. This increase in ground level is usually called free-field heave. However, if a structure is built on such a soil deposit, the foundations form an obstruction to the soil to freely move up and consequently, the soil applies an upward pressure on the foundation. This pressure that the soil applies on the foundation is called swell-pressure. If the footing transfers a downward stress which is smaller than the swelling pressure, the footing moves upward. These upward and downward movement of foundations become cyclic seasonal movements during the entire life span of the structure. Thus, these cyclic movement will tend to tear up the walls and eventually destabilize the whole structure. Light structures, such as single or double story residential buildings, pavements, etc. which generally transmit smaller stresses to the soil than the swell-pressure are those that suffer the damage most (Afewerq, 2004).

As different researches showed, this problematic soil causes a great deal of problem on different structures in Ethiopia. Afewerk Sisay (2004) on the research he have conducted on randomly selected lightweight buildings constructed on expansive soil of Addis Ababa he observed that: sixty-nine out of ninety-sex buildings are affected by the consequence of expansive soil. The problem observed varies from simple to large structural damages. Most of the damages are occurred on the walls of the structures with diagonal crack at the junction of openings and corner wall cracks than the floors and ceilings. Buildings with masonry foundation also exhibit the larger number of damage compared to footing and stiffened mat foundation.

In the study conducted by Dejenie Abera (2019) on the newly founded condominium sites in Addis ababa of Bole Arabsa, Koyefeche, Jemo, Yeka and Ayat where the plasticity index of the soil ranges from 20-35 and above shows high plasticity. And the researcher observed settlement of masonry wall below the grade beam, suspension of grade beam, column alignment problem due to grade beam suspension, cracking of masonry wall in different segments, cracking of walls that ranges from slight to moderate cracks and exterior wall diagonal cracking are seen.

Expansive soils predominantly black cotton and other clayey soils occur in various parts of Ethiopia, covering nearly 40% surface area of the country (Bantayehu, 2017). In central, western, north-western, south-western and same parts of southern and eastern Ethiopia are coved with black cotton soil (Afewerq, 2004). Predominantly these soils are observed in area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berihan, Addis Ababa - Gohatsion, and Addis Ababa - Mojo. Also they are observed in towns like Mekelle, Bahirdar, Gambela, Arba Minch and the most Southern, South-west and south-east part of the capital Addis Ababa area in which the most major recent construction are being carried out (Bantayehu, 2017). Black cotton soils often occur covering an extensive area in flat and gently sloping landscapes such as on the highland plateau, low land flood plains and valley flours (Afewerq, 2004).

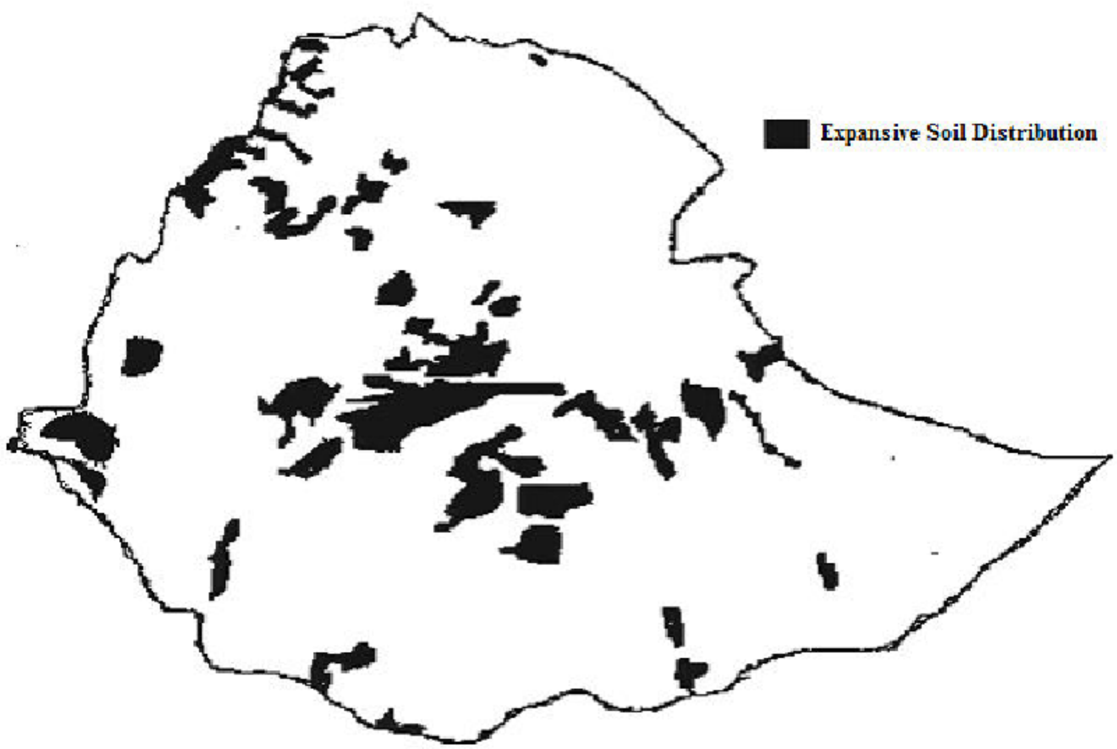


Figure 3 : Distribution of black cotton soil in Ethiopia *(Bantayehu, 2017)*

Whenever the soil for construction do not meet the requirements, a process called stabilization is employed to improve the properties of the soil. Soil stabilization may be broadly classified as mechanical stabilization and chemical stabilization. Under mechanical stabilization compaction, excavation and replacement and mixing of different soil are used. Under chemical stabilization the use of additives such as lime stabilization, cement stabilization, asphalt stabilization, and etc. are used. Besides with the increasing demand for the performance of soil stabilization materials in the fields of engineering application and environmental protection, scholars have made many achievements in exploring the strength improvement of different additives on traditional soil stabilizers (Lim Jing Jin et al, 2018). This includes fly ash, sugarcane bagasse ash, west ceramic dust, Granulated Blast Furnace Slag, Rice Husk Ash and others. For the purpose of this research sugarcane bagasse ash is used for the stabilization of expansive soil.

2.2 Properties of Sugarcane Bagasse Ash

Bagasse is the fibrous residue generated from sugar factory after the juice has been extracted from the sugarcane. It is currently used as a bio fuel, in the manufacture of pulp and paper products and building materials. About 40-45% of this fibrous residue is left which is reused in the same industry as fuel boiler for heat generation leaving behind 8-10% ash as waste. This ash is known as sugarcane bagasse ash (SCBA) (Melat, 2016). The older method to dispose SCBA is spreading as fertilizer in the field (Bhoi A.K et al., 2020). Even though this method is the most frequently used, scholars are recommending not using SCBA as fertilizer due to the heavy metals present in it which may lead to adverse effect on the yielding of the crop (Bhoi A.K et al., 2020). Many researches showed that the use of SCBA as a supplementary cementitious material would solve the environmental problems related to its disposal (Guilherme , Pryscila , & Luís , 2019)

Bagasse ash is a pozzolanic material which is very rich in the oxides of amorphous silica and aluminum and sometimes calcium (Bethlehem, 2015). High silica in the SCBA could be due to the absorption of silicic acid from the soil and deposit in amorphous state in the plant (Pritish et al., 2021). Besides the main oxides, SCBA also contain carbon and water expressed by loss on ignition (Pritish et al., 2021). As per ASTM C618 Standard Specification for Natural Pozzolan, the loss on ignition (LOI) content of maximum 10% is allowed for class N and 6% maximum for class C and F. Higher LOI content revealed the presence of unburnt carbon which could decrease the pozzolanic activity of the SCBA at a later stage (Pritish et al., 2021).

Table 5 : Chemical Requirements for Pozzolanic Property (ASTM C 618–05)

	Class		
	N	F	C
Silicon dioxide (SiO_2) + aluminum oxide (Al_2O_3) + iron oxide (Fe_2O_3),min, %	70.0	70.0	50.0
Sulfur trioxide (SO_3), max, %	4.0	5.0	5.0
Moisture content, max, %	3.0	3.0	3.0
Loss on ignition, max, %	10.0	6.0	6.0

It also exhibets dark coloration which is an indication of the strong presence of carbonaceous particles in the ash (Guilherme et al., 2019). The color may vary depending on the completeness of the combustion process and the structural transformation of silica in the ash.

Table 6 : Chemical Composition of Sugarcane Bagasse Ash *(Taresa , 2019)*

Oxides	Sio_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
SCBA	72.0	7.20	4.16	1.44	1.28	2.16	3.5	0.16	0.47	0.09	1.29	5.90

Pozzolanic reaction occur over long time scales. The main mechanism involves the transformation of calcium hydroxides via water within the soil to combine with the aluminate and /or silicate minerals (P.Sargent, 2015). The high surface area aluminate and silicate minerals in the presence of water and alkali like calcium produce cementitious materials, comprising calcium silicates and aluminium hydrates. Any dissolved Ca^{2+} ions within the soil react any dissolved SiO_2 and Al_2O_3 located in the clay particles to produce hydrated gels of C-S-H and C-A-H which cements soil particles together as:

$$Ca^{2+} + 2OH^{-} + SiO_2 \longrightarrow C - S - H$$

$$Ca^{2+} + 2OH^{-} + Al_2O_3 \longrightarrow C - A - H$$

And with curing which is the retention of moisture within a material it will gradually increase in strength and reduce the shrinkage of the material (Bethlehem, 2015).

Calcination and grinding can improve the pozzolanic characteristics of SCBA (Guilherme et al., 2019). Recalcination is helpful to adjust its chemical composition by reducing its carbon content. Additionally grinding is also required, given its ability to homogenize and control the particle size distribution of SCBA. This will also increase the specific surface area. As the particle size declined the SCBA will be highly reactive which infers more pozzolanity (Guilherme et al., 2019). Mansaneira, E. C et al., 2017 uses pozzolanic activity index (PAI) to check the pozzolanity of SCBA. For a material to be considered as pozzolanic, the PAI value should not be less than 75% of the compression strength of the reference mortar. Based on the test carried grinded SCBA present PAI greater than 75% which is failed to be attend by not grinded sample. It is also shown that the ash that is grinded for 2hours has the largest PAI value compared with the ash grinded for 1hour. This is a conformation that grinding makes the ash reactive.

Researches so far has been done in the utilization of SCBA as a replacement material and tries to stabilize expansive soil with sugarcane bagasse ash as standalone material and combining it with other cementious material like lime and cement. Some are discussed as below.

2.3 Sugarcane Bagasse Ash based Researches

Teshale (2019) evaluates the use of SCBA as cement replacement material to minimize the degradation of environment due to cement demand. He uses a SCBA that is grounded to a fineness similar to that of Portland cement with replacement ratio of 5%, 10%, and 15% by weight of cement. The impact to the mechanical and physical properties of the concrete and paste such as consistency, setting time, soundness, workability, compressive strength and durability are tested. From the investigation SCBA to cement mix proportion from 5% to 10% results in a better compressive strength than that of the controlled concrete with 100% ordinary Portland cement (OPC). However a reduction in strength is observed for a concrete mix of more than 10% SCBA. It is also observed that a reduction in unit weight of up to 1.72% occurred for 15% replacement.

Tsigehana (2019) uses SCBA sieved with 250μm sieve size and a range from 5% to 20% by weight of cement together with control mix was used. The results show that up to 5% SCBA replacement of OPC is possible based on concrete compressive strength which gives about 8% and 6% enhancement in the strength for 21 and 28 days of curing respectively. The water absorption of the concrete is also increased with increasing of SCBA.

Taresa (2019) uses sugar cane bagasse ash which is find out to be rich in SiO_2 to partially replace the quartz/silca sand in wall tiles to produce wall tile of a better quality and low cost. The results show that replacement of quartz by 35% SCBA and at 17MPa pressing pressure found out to be the best for ceramic wall tile production. Hence, it is proved that SCBA have a silica content that could replace the natural quartz. For the test a partial replacement of 20, 35, and 50% by weight was used.

Daniel (2020) tests the pozzolanic reactivity of bagasse ash while partially replacing 10% by volume of cement via compressive strength development. To attain this standard consistency test, characterization of hardened paste, measurement of the heat of hydration, determination of the amount of bound water, determination of the pozzolanic activity of SCBA and determination of compressive strength were made. From the tests it has been concluded that the water demand of the blinded sample was relatively higher than the reference sample due to the fineness of the ash which also increase the initial and final setting time. Compressive strength at early age (≤ 1day) was lower, however in the later days (3, 7, 28, 56, and 90 days) the compressive strength was found to be higher than the reference sample. Generally the results are showing that there exists a pozzolanic reactivity of bagasse as while replacing cement.

Dinke (2015) investigates the effect of applying bagasse ash alone and combining it with ordinary Portland cement (OPC) on the engineering properties of highly plastic expansive soil that is classified under A-7-6 according to AASHTO soil classification and CH under unified soil classification system. For the treatment 5, 10, 15, and 20% bagasse ash and 2, 4, 6, and 8% cement has been used. The results show that liquid limit, plastic limit and plasticity index decreases with increasing in bagasse ash content. A considerable reduction was also observed in the free swelling index value of the soil. The value drops from 118% to 54% in a 10% addition of SCBA. On the other hand the maximum dry density decreases and the optimum moisture content increases with addition of ash. For the compressive strength test a 7, 14, and 28days of curing sample was used and it has been observed that the sample increase in UCS up to 10% of SCBA addition and reduced for further addition. This happens to be the same for California bearing ratio test (CBR).

Mengistu et al. (2020) showed the usage of industrial waste product (sugarcane bagasse ash) as stabilizing material of expansive soil in replacement of cement or lime. The bagasse ash obtained from wonji sugar factory is used as standalone additive for the treatment of expansive soil collected from Bole Airport terminal expansion project. Four laboratory tests atterberg limit, unconfined compression strength (UCS), compaction, California bearing ratio (CBR) test has been conducted on the black cotton soil without blending it with bagasse ash and with 3%, 6%, 9% and 12% of bagasse ash mixture.

Improvement in the maximum dry density was observed for 3%, 6% and 9% of bagasse ash, but then after further increase in blend maximum dry density decreases. When the soil was treated with a bagasse ash its maximum dry density improved from 1.46 gr/cm^3 at OMC of 23.94% (pure black cotton soil) to 1.95 gr/cm^3 at OMC of 23.13% (at 9% of bagasse ash). It was also observed that, CBR values increase rapidly for 3%, 6% and 9% of bagasse ash, but then after suddenly dropped to lower values. Furthermore Plasticity index of black cotton soil decreases with increase percentage of bagasse ash replacement and Water taking capacity of Black cotton soil (expansive soil) decreases with increase in content of bagasse ash. Finally they have decided that the optimum bagasse ash amount for the testes that has been conducted to be 9%.

Melat Nesru (2016) Melat showed that the plasticity index slightly reduced with increased in bagasse ash content and curing has also insignificant effect on the plasticity of the expansive soil. It is also observed that optimum moisture content increased while the maximum dry density values decreased with increment of bagasse ash content. This is because of the partial replacement of comparatively heavy soils with the light weight bagasse ash; comparatively low specific gravity value (1.95) of bagasse ash than that of replaced soil (2.79). It may also be attributed to coating of the soil by the bagasse ash which result to large particles with larger voids and hence less density. The increase in the optimum moisture content was mainly due to bagasse ash is finer than the soil. The more fines the more surface area, so more water is required to provide well lubrication. It is also managed to decrease free swell, free swell index and free swell ratio of the stabilized samples with increasing bagasse ash content. CBR values slightly increased with the addition of bagasse ash.

For her research Melat N. uses 0 to 30% bagasse ash mixture with incremental of 5%. She recommended that bagasse ash will be more effective in stabilizing highly plastic expansive soil when it is used together with lime rather than as a standalone material.

Bethlehem (2015) on this thesis work the researcher conducted an experimental study to investigate the effect of adding sugarcane bagasse ash on the geotechnical characteristics of lateritic soil. Her results shows that the liquid limit and plasticity index showed a general increase with increasing amount of bagasse ash. The shrinkage limit and the free swell also showed an increase with increment of the bagasse ash. The maximum dry density and optimum moisture content decreased and increased respectively with the increment of SCBA. However, the changes observed were insignificant. Nevertheless, the bagasse ash had very little content of lime (CaO) and same chemical composition as the lateritic soil sample. As a result, the addition of the ash did not bring a significant change on the strength of the soil.

Other researches are summarized on the table below:

Table 7 : Summery on the use of SCBA for soil stabilization

Title	Journal	Discussion
Amit S. et al. "Waste product 'bagasse ash' can be used as stabilizing material for expansive soils"	International journal of research in engineering and Technology IJRET, vol. 3 issue 03, 2014	• 3%, 6%, 9%, and 17% of bagasse ash was used • Significant increase in UCS, CBR and MDD values obtained at 6% replacement of bagasse ash
Ahmed Batari et al. " Effect Of Bagasse Ash On The Properties Of Cement Stabilized Black Cotton Soil"	International journal of transportation engineering and technology vol. 3 No. 4, 2017, pp 67-73	• Uses 0, 1, 2, 3, 4, 5, & 6% of bagasse ash with 0,4,6, 8% of cement • The addition of bagasse ash alone does not improve the strength properties of the natural black cotton soil • Optimum blend was achieved with 5% bagasse ash as admixture to 8% cement stabilization
Hasen et al. "Remediation Of Expansive Soil Using Agricultural Waste Bagasse Ash"	International Conference On Transportation Geotechnics, ICTG 2016	• Bagasse ash and hydrated lime mix of 0%, 6%, 10%, 18% and 25% with 3:1 ratio of bagasse ash to lime is used • Addition of bagasse ash to expansive soil decreases its swelling, because bagasse ash is nonplastic material. The free swell ratio (FSR) decreases as

		percentage of bagasse ash increases. • Tests show that unconfined compressive strength increases up to 25% by 80% • The use of bagasse ash with lime is more effective than lime only in curtailing the swelling tendency of soil.
Meron Wubshet "Bagasse Ash As A Sub-Grade Soil Stabilizing Material"	School of Civil & Environmental Engineering, Addis Ababa Institute of Technology, AAU,2014	• Uses 0 to 30% of bagasse ash with increment of 5% for the stabilization of expansive soil • Observes a decrease in plasticity, free swelling and maximum dry density with increase in optimum moisture content. • Peak CBR value is attained in 30% of Bagasse ash content • Addition of Lime improves the soil at 15% bagasse ash.
K. S. Gandhi "Expansive Soil Stabilization Using Bagasse Ash"	International Journal of Engineering Research & Technology (IJERT), 2012	• Evaluate the effect of adding 0 % , 3% ,5%, 7% and 10% SCBA in index properties of expansive soil from Surat city, India • Free Swell Index and Swelling Pressure decreases as Percentage of bagasse Ash increases; and shows some linearity between them.

Niken Silmi Surjandari *et al* "Enhancing the engineering properties of expansive soil using bagasse ash"	Journal of Physics: Conf. Series 909, 2017	• Evaluate the enhancement of the engineering properties of expansive soil using bagasse ash treated with bagasse ash by weight (0, 5, 10, 15, and 20%) based on dry mass. • The plasticity index decreased from 53.18 to 47.70%. The maximum dry density of the specimen increased from 1.13 to 1.24 gr/cm3. The percentage swelling decreased from 5.48 to 3.29%
Srinivasa Reddy K. et al. "Stabilization of Expansive Soil Using Bagasse Ash"	International *Journal of Civil Engineering and Technology*, 8(4), 2017	• Different dosage of bagasse ash i.e. 2%, 4%, 6%, 8% and 10% were used to stabilize the taken soil sample, The performance of bagasse ash was evaluated using physical and strength performance tests • It was observed from above figures by the addition of 6% bagasse ash for the expansive soil had perfect effect compare to the other dosage, increasing UCC indicates a reduction in the settlement

Summary on Sugarcane Bagasse Ash Stabilization

Generally the pozzolanic reactions are dependent on both access to water, pozzolanic material and the content calcium ions. Since there is no enough calcium in bagasse ash, the soil-bagasse ash mixture is not successful in stabilizing for some of the researches. Therefore it is shown that certain amount of calcium from cement or lime is necessary. Sugarcane bagasse ash has the same geotechnical property as fly ash and Texas Department of Transportation manual typical proportions for the ratio of lime to fly ash that will yield the best engineering property is determined by using lime to fly ash ratios of 1:3, 1:4 and 1:5. (Bethlehem, 2015) Bagasse ash contents can also be used accordingly. On the contrary, adding of large amount of bagasse ash is also a cause for density reduction and increase in moisture content, because of the partial replacement of comparatively heavy soils with the light weight bagasse ash; comparatively low specific gravity value of bagasse ash than that of replaced soil. It may also be attributed to coating of the soil by the bagasse ash which result to large particles with larger voids and hence less density. Increase in the optimum moisture content might be due to bagasse ash is finer than the soil. (Melat, 2016)

Therefor considering this and other literatures a Bagasse ash amount of 0%, 6%, 9%, and 12% are used for this research.

2.4 Currently potential of Bagasse ash in Ethiopia

In order to assess the potential of bagasse ash production in Ethiopia, it is imperative to evaluate the sugarcane crop yield in the country. Ethiopia began investment in sugar industry in 1954 with one sugar factory which have been expanded to four small size sugar factories up to 1998. By recognizing the nature of the industry, the Ethiopian Government established Ethiopian Sugar Corporation (ESC) in November, 2011 in order to complete the projects commenced by the Agency and embark on the expansion of new sugar projects. The total area developed by these factories is 305,200 hectares. The area developed at Wonji Shewa is 17,000 hectares capable of producing 225,000tons of sugar per annum. The Metehara Sugar Factory, which was brought on stream in 1969 at Metehara, developed 10,200 hectares and has a capacity to process 130, 000tons of sugar annually. The Finchaa Sugar Factory (in East Wellega zone of the Oromia National Regional State) which was completed in 1998 developed 21,000 hectares and has a production capacity of 266,000tons of sugar per annum. Tendaho Sugar Factory developed total Cane Plantation Currently 18,900 Ha -50,000 Ha, First Phase Finalized On Sept. 2014/15, has a capacity to process (258,000tons of Sugar) and Second Phase (618,000ton of Sugar) annually.

This annual production is not sufficient to satisfy the local sugar demand forcing the government to annually import 1.5 million quintals from abroad [The Ethiopian Sugar Industry Current Status & Future Prospect on the 23rd ISO Seminar Nov.2014]. Fill the gap between supply and demand of sugar, Ethiopian Sugar Corporation (ESC) has set a strategic plan to build ten new sugar factories at different locations of the country. ESC started the development of additional 10 big size sugar projects by expanding the factory sites from 5 location to 9 location which is distributed all over the country. And the number of industries from 4 to more than 14. When these and the existing facilities are up and running, the ESC is predicting that the country will become one of the ten largest sugar producers by 2023, with production estimated 4.2 million metric tons. However, to crack the list of today's top 10 sugar (cane + beet sugar) producers, Ethiopia would have to produce more than 4.5 million metric tons. In comparison, when comparing Ethiopia against cane sugar producers, Ethiopia would be the 8th largest producer after Australia (4.8 MMT) and before the United States (3.3 MMT). Beside this the Ethiopia government is undertaking expansion projects on the existing factories to increase their production capacity. At the end of this expansion projects on Fincha, Methara and Wonji/Shoa sugar factories the additional (not including the current production) total aggregate production capacity is expected to be around 365,000 tons of sugar annually [Ethiopia plans to take 2.5pc of the world sugar market, 2010]. As can be seen from the above discussion the sugar production in the country is being boosted at a high rate, even planning to hold 2.5 percent of the world sugar market in the coming five years according to the strategic plan. Boosting sugar production will also result in high amount of bagasse and bagasse ash.

Eight factories are producing sugar currently in Ethiopia these includes Wonji-shewa, Metahara, Fincha, Kessem, Tendaho, Arjo, Omo kuraz II and Omo kuraz III. In addition five factories which are Omo kuraz I, Omo kuraz V, Tana abeles I, Tana abeles II and wolkaite are at their construction phase. According to Ethiopian Sugar Corporation, the countries sugar production has been increased from 2.5 million quintals in 2019 to 3.2 million quintals in 2020 (The Ethiopian Herald, 21 Feb. 2021 This boosting will also result in high amount of bagasse and bagasse ash potential. For each 10 tons sugarcane crushed, a sugar factory produces nearly 3 tons of wet bagasse. (Mengistu Mena, 2020)

2.5 Dynamic Property of Soil

In nature there exist all kids of Phenomena's which produce dynamic or cyclic loading environments affecting civil engineering systems such as earthquakes, water waves, storms, wind power, construction operations, traffic loads, vibrating machinery and many other (Sas W et al., 2013). The nature and distribution of damage due to dynamic loading is strongly influenced by the response of soil to the cyclic loading. To evaluate the effect of such loads it is necessary to understand, measure and quantitatively model the dynamic and cyclic properties of the soils involved (R. Dobry & M. vucetic, 1987). Soil response analysis is used to evaluate dynamic stress and strain. It is also used to determine earthquake induced forces that lead to instability of earth and earth-retaining structures.

To obtain the maximum benefit from the method of seismic analysis, an understanding of the dynamic response characteristics of soil is essential. Roads and runways are largely designed on the basis of knowledge obtained from soil tests under static loading such as UCS, plate loading, density, and CBR essentially dynamic nature. However, the use of static loading does not represent the actual loading due to traffic which is alternative and repetitive in nature. Thus, the material properties under such a load must be investigated in terms of dynamic response and behavior of pavement-soil system (Yung-Chieh & Yong S. , 1972). One of the major considerations in the design of offshore structures is the effect of wave action on the foundation. This requires a study of the effects of cyclic loading on the soil properties (Adel & Louise , 1981).

Expansive soils cover some of the earthquake prone areas, and numerous facilities on expansive soils are also subjected to dynamic loading. While liquefaction is not a problem for clays, but they suffer loss of strength when dynamically stressed. Researches reveals that the reversal of the cyclic stress resulted in large excess pore water pressure causing a decrease in the strength of a clay soil than one-directional stress (Adel & Louise , 1981). On the other hand the combination of static and dynamic loading on the clay soil increases the maximum permanent settlement associated with the loadings as the loading amplitude increases. As the loading cycle number increase the stress usually decreases and for stress-controlled tests increment in strain is observed as time is elapsed in the test and phenomena is known as degradation or decrease in strength (Adel & Louise , 1981).

The dynamic response of soil under cyclic loading is expressed in terms of important mechanical properties of soil like shear wave velocity, shear modulus, damping ratio, stress history, plasticity index and Poisson's ratio. Geotechnical engineering problems that involve dynamic loading of soil and soil-structure interaction system the knowledge of shear modulus and damping characteristics are important (Mengesha , 2013).

The behavior of soils subjected to cyclic loading is governed by what is known as dynamic soil properties (Kramer, 1996). Two of the most important dynamic soil property parameters required for dynamic analysis are the soil stiffness (Shear modulus) and material damping (Damping ratio). These properties are used to assess the dynamic response of soils at different strain levels. Thus understanding of expansive soil behavior under dynamic loading and being able to accurately model the change in a given soil dynamics properties as it is strained is critical to being able to predict how it will behave when subjected to dynamic loading.

2.5.1 Cyclic stress strain behavior of soil

The concept of dynamic loading of soil refers to the propagation of stress waves through soil layers. The term "Cyclic" refers to systems oscillating with constant amplitude and frequency. Cyclic loadings are loads that exhibit a degree of regularity both in their magnitude and frequency. One of the most recognizable forms of a soil in a cyclic loading scenario happens during an earthquake and machine vibration-induced loading on foundations.

The stress strain behavior of soil under dynamic loading depends on the nature of the soil, the environment of the soil (static stress state and water content); and the nature of the dynamic loading (strain magnitude, strain rate, and number of cycles of loading).

The effect of percentage of fines, material grading, and plasticity index (PI) and stress history are significant on dynamic property of soil for small strain level ($\gamma \leq 0.01\%$). However, for problems that involve medium to large strains ($\gamma > 0.01\%$) the effects of loading frequency (f) and the number of cycles (N) on stress strain property of soil are significant. For large strain level ($\gamma > 0.1\%$) the kind of behavior is termed degraded hysteresis type (Mengesha , 2013).

The manner in which the shear modulus and damping ratio change with cycles is considered to depend upon the manner of change in the effective confining stress during irregular time histories of shear stress application (Yohannes , 2015). The stress strain response of soil is non-linear for large strain but linear for small cyclic shear strain. Earthquake-induced stresses and strains that produce cyclic shearing of the soil are generally considered to be medium to large strain (≥ 0.01%) level (Mengesha , 2013).

Geotechnical earthquake engineering analysis assumes that earthquake ground motions are generated by vertically-propagated shear waves which cause cyclic shearing of the soil (Kramer, 1996). The stress-strain response of soil to this type of cyclic loading is nonlinear and hysteretic and commonly characterized by a hysteresis loop (Yohannes , 2015). The most common model used to represent the hysteretic behavior of soil in seismic analysis is the equivalent-linear model. The equivalent-linear model characterized non-linear hysteretic soil behavior using an equivalent shear modulus, and damping ratio (Yohannes , 2015).

A wide variety of procedures, including laboratory and field tests have been used to determine both shear modules and damping characteristics of soils. Previous Cyclic laboratory studies have shown that, cyclic response of saturated soils manifested by the deformation of the soil skeleton (Vucetic, 1992). These deformations are mainly due to breakage of particle bond, slippage at the particle contacts, and corresponding change of microstructural repulsion forces and can be expressed in terms of the shear strain of the soil (Vucetic, 1992). Hysteretic stress-strain relationships under moderate to relatively high strains may be determined in the laboratory by means of triaxial compression tests, simple shear tests or torsional shear tests conducted under cyclic loading conditions (Vucetic, 1992). In this study shear modulus and damping factors were determined using cyclic simple shear tests.

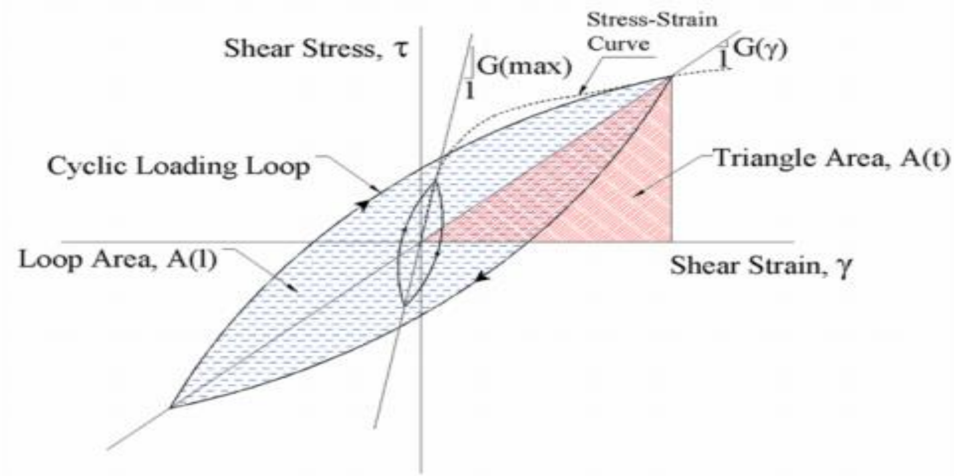


Figure 4: Representative Hysteretic Shear Stress- Shear Strain Relationship

2.5.2 Material Parameters Describing Dynamic properties of soil

Shear Modulus (G) The shear modulus is the measurement of stiffness of soils and it is the most basic parameter to evaluate soil behavior in dynamic analysis. The average shear modulus over one cycle is best described by the secant shear modulus G_{sec}, which is proportional to the slope of the line reaching the points of higher and lower shear strain over one cycle. The shear modulus conventionally expressed as the secant shear modulus in soil dynamics. The most reliable method of producing representative moduli has been to evaluate the difference in maximum and minimum stress and the difference in maximum and minimum strain developed in the hysteresis loop.

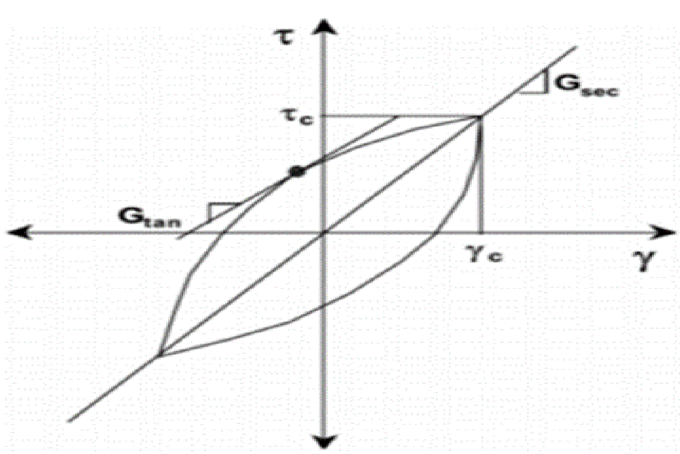


Figure 5 Hysteretic Loop Showing Secant and Tangential Modulus

$$\text{Shear Modulus, } G = \frac{\text{Amplitude of cyclic shear stress}}{\text{Amplitude of cyclic shear strain}} = \frac{\Delta\tau}{\Delta\gamma}$$

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 shear strain.

The maximum shear modulus (G_{max}) is determined by plotting a tangent line at the initial point of the curve. Secant shear modulus (G_{sec}) is the slope of the line that connects any point of γ to the origin.

Damping Ratio (D) Damping is the phenomenon of the dissipation of energy in a vibrating or cyclically loaded body of material or mechanical system. Usually in the form of heat, which cause the dampening of oscillation. A material has a large damping if it dissipate a lot of energy during cyclic loading or vibrations (M. Vucetic et al., 1999). In soil two types of damping are important: soil damping and radiation or geometric damping. The first one is the dissipation of energy within a soil element during cyclic loading. While the second one is transmission of energy away from an energy source by a mechanism of radiation (G. Lanzo & M. Vucetic, 1999).

The evaluation of damping properties of soils is important for the solution of many geotechnical engineering problems involving the transmission of waves through the soil, such as the seismic response of soil deposits and supported structures to earthquake loads and the machine foundation vibration.

The loss of energy is evident from the hysteretic behavior of the soil during cyclic loading the stress- strain behavior in a single cycle of loading is described by a hysteresis loop, similar to the idealized loop as shown in the figure below:

The damping ratio, D, is a measure of the dissipated energy (W_D) versus elastic strain energy (W_S) through one cycle of loading-unloading and it is defined by the equation as:

$$D = \frac{W_D}{4\pi . W_S} = \frac{1}{4\pi} . \frac{A(l)}{A(t)}$$

Form the figure A (l) – the loop area and A (t) – the triangular area

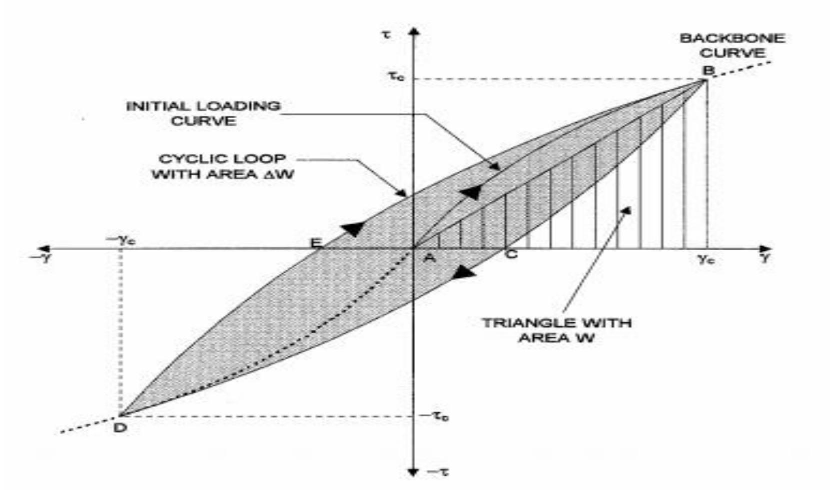


Figure 6 Idealized first cycle stress-strain loop

A_Δ represents the maximum strain stored energy and A_{loop} is the energy loose per cycle represented by the area enclosed inside the hysteresis loop.

$$A_{loop} = \frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$$

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

$$shear\ stress\ \tau = \frac{shear\ force}{area\ of\ sample} = \frac{F}{A}$$

$$shear\ strain\ \gamma = \frac{Lateral\ LVDT}{Height\ of\ sample}$$

Where LVDT is axial deformation measurement.

Initial Shear Modulus (G_{max}):- the initial or maximum shear modulus, G_{max} is widely regarded as one of the most essential parameters for earthquake engineering, traffic engineering, vibration machine foundation, vibration isolation measures and analysis of dynamic soil structure interactions (Kramer, 1996). In recent years, many studies were performed to investigate the behavior of soil at small strain level. It is a very important parameter not only for seismic ground response analysis but also for a variety of geotechnical applications. A considerable number of empirical relationships have been proposed for estimating initial shear modulus for different kind of soils. Since cyclic simple shear tests are high strain tests (0.01% to 5%), the maximum shear modulus, G_{max} that corresponds to very low strain levels cannot be determined from such tests. For this study the maximum shear modulus, G_{max} of each specimen was determined using equation below from Hardin B.O. and W. L. Black (Hardin & Drnevich, 1972). The calculated value of G_{max} was used to obtain the normalized shear modulus. Generally, the values of the initial shear modulus of soil ranges from 2,750kPa to 69,000kPa (Abu, 2011).

$$G_{max} = 3220 \times (\frac{(2.976 - e)^2}{(1 + e)} \times \ (OCR)^k \ \times \ (\sigma_o^-)^{k_o}$$

$$\sigma'_o = \frac{(\sigma'_v + 2k_o \ \times \ \sigma'_v)}{3}$$

Where

- ✓ e = void ratio
- ✓ OCR = over consolidation ratio = ${}^{P_c}/{}_{P_o}$
- ✓ P_c = pre consolidation pressure of a specimen
- ✓ P_o = present effective vertical pressure
- ✓ σ'_o = mean effective stress
- ✓ σ'_v = effective vertical stress
- ✓ K_o = coefficient of lateral earth pressure at rest
- ✓ K = dimensionless quantity which is function of PI, it can be obtained from a table

Table 8 Empirical values of exponential parameter, k

PI	0	20	40	60	80	>100
k	0	0.18	0.3	0.41	0.48	0.5

Degradation index (δ), soil behavior under cyclic loading is not the same as under monotonic loading. Under cyclic loading there exist excessive settlement and due to progressive generation of pore pressure, causes the reduction in the effective normal stress which is also a cause for the reduction in the strength. The cyclic degradation in the cyclic stress controlled mode is expressed with a degradation index, δ. Which describes the reduction of the secant shear modulus Gs with number of cycles, N as follows.

$$\delta = \frac{G_{SN}}{G_{S1}} = \frac{\tau_c/\gamma_{CN}}{\tau_c/\gamma_{C1}} = \frac{\gamma_{C1}}{\gamma_{CN}}$$

And the degradation parameter, t

$$t = \frac{-\log \delta}{\log N} \text{ or } \delta = N^{-t}$$

Where

τ_c – Cyclic shear stress which is constant while

γ_{CN} - Is cyclic shear strain amplitude varies with N.

γ_{C1} - Is the shear strain registered at first cycle and

γ_{CN} - Is the final shear strain at which the test is stopped.

The index δ decreases with N because γ_{CN} increases with N. lower value of δ implies higher degree of degradation.

Normalized Modulus (G/G_{max}), from the modulus reduction curve, normalization of the shear modulus (*G*) is carried out with respect to the maximum shear modulus (G_{max}), which is commonly referred to as the modulus ratio. Determining shear modulus (G) and material damping (D) for ground response analysis involves estimating or measuring the variation of G and D with the strain γ, primarily in the laboratory. It is a common practice to normalize G by dividing it by G_{max}. A plot of the variation of G/G_{max} with γ is called a normalized modulus reduction curve. A common approach to model *D* is to relate the damping ratio with G/G_{max} (Abraham, 2014)

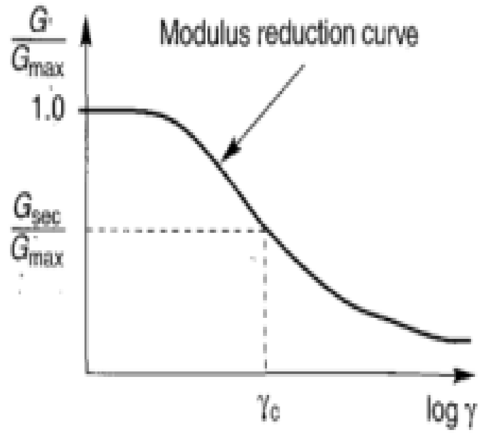


Figure 7: Modulus Reduction Curve

2.5.3 Method of determining shear modulus and damping ratio

A wide variety of procedures, including laboratory and field tests have been used to estimate both shear modulus and damping characteristics. The main procedures can be summarized as follows:

Direct Determination of Stress-Strain Relationships: Hysteretic stress-strain relationships can be determined in the laboratory by means of triaxial compression tests, simple shear tests or torsional shear tests conducted under cyclic loading conditions. In general these procedures are useful for measuring shear modules and damping factors under moderate to relatively high strains ($0.01 < \gamma < 5\%$). At low strain level Resonant Column Test, Ultrasonic Pulse Test, and Piezoelectric Bender Element laboratory tests are used to determine the property of the soil in the laboratory. High strain element tests also includes Cyclic Triaxial Test, Cyclic Direct Simple Shear Test and Cyclic Torsional Shear Test (Seed & Idriss, 1970).

Forced vibration tests: Forced vibration tests, involving the determination of resonant frequencies and measurement of response with different frequencies have been used to determine both shear modules and damping factors. Test conditions in the laboratory have included the application of longitudinal vibrations and tensional vibrations to cylindrical samples or shear vibrations to layers of soil placed on a shaking table. In general these procedures are useful for determining properties at relatively low to moderate strain levels ($10^{-4} < \gamma < 10^{-2}$ %) (Seed & Idriss, 1970).

Free Vibration Tests: Free vibration tests, in which measurements are made of the decay in response of a soil sample or soil deposit, have been used to measure both shear modules and damping factors for soils. Methods of excitation are essentially similar to those used for forced vibration tests, but the procedures can be used for measurement of soil characteristics at relatively low to moderately high strain levels ($10^{-3} < \gamma < 1\%$) (Seed & Idriss, 1970).

Field Measurement of Wave Velocities Field tests have been used to measure the velocity of propagation of compression waves, shear waves, and Rayleigh waves from which values of soil modulus can readily be determined for low strain ($\gamma \leq 10^{-4}$ %) conditions. These procedures have not provided values of damping factors (Seed & Idriss, 1970).

In this study, direct determination of stress-strain relationships was used for shear modulus and damping ratio determination from cyclic simple shear test.

2.5.4 Factors affecting shear modulus and damping ratio

The most important factors that affect the dynamic behavior of soils divided into two main categories (Yohannes , 2015). Firstly, external variables such as stress/strain path, stress/strain magnitude, stress/strain rate and stress/strain duration can affect the dynamic property of soils. Secondly, characteristics of the soil: such as soil type, size and shape of soil particles and void ratio can affect the response of soil to dynamic loadings. These factors are; method of sample preparation in laboratory, confining pressure, methods of loading, over consolidation ratio, shear strain levels, loading frequency, soil plasticity, percentage of fines, void ratio, relative density and soil type.

Out these factors: confining pressure, soil plasticity, number of cycles and shear strain levels were discussed in this study.

Shear strain amplitude: The deformation characteristics of soils are highly nonlinear. It manifested in the shear modulus and damping ratio, which vary significantly with the amplitude of shear strain under cyclic loading (Ishihara, 1996). Shear strain significantly affect the shear modulus and damping ratio of soils. The shear modulus decreases and damping ratio increases with an increasing shear strain amplitude.

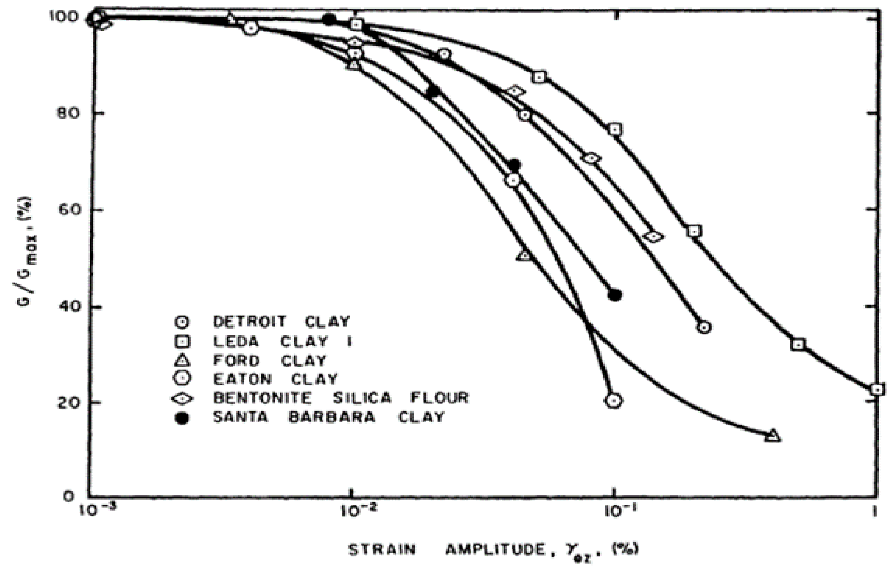


Figure 8 normalized modulus reduction relation for clay with strain amplitude (Seed et al., 1988)

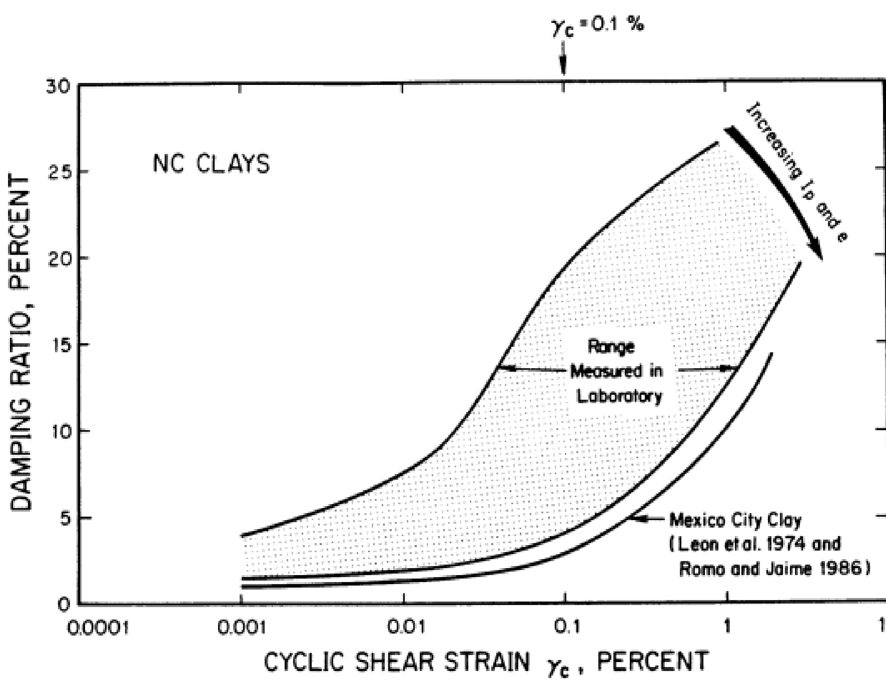


Figure 9 Damping ratio range and trend measured for clay in the laboratory (R. Dobry & M. vucetic, 1987)

Number of cycles: For a given value of mean effective stress and shear strain the shear modulus slightly increases and damping ratio decreases with the number of cycles. For most seismic events, the number of significant cycles is likely less than 20. The value determined at 5th cycles provides reasonable values for all practical purposes (Das 1993). Furthermore, the effect of number of cycles practically disappears when stress application is repeated for more than 10 cycles.

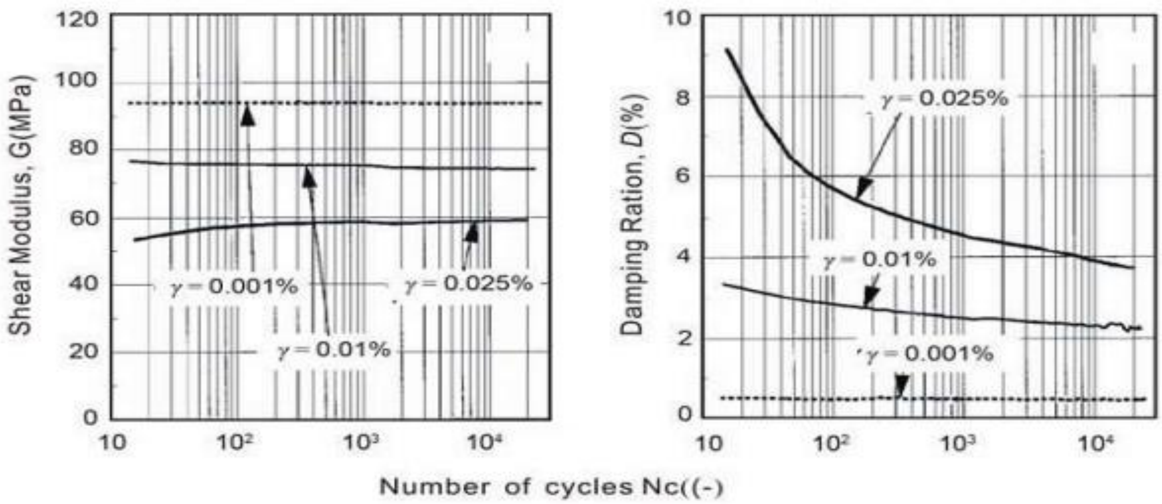


Figure 10 Effect of number of cycles on damping ratio (Min Wange, et al., 2012)

Plasticity index: Unlike other factors plasticity index dominantly and consistently affects the shear modulus value of cohesive soils (Seed et al., 1988). Vucetic and Dobry 1991 reported that, with an increase in PI, soils are more resistant to cyclic loading and modulus reduction takes places in a slower rate. Highly plastic soils have smaller damping and relatively large range of linearly elastic behavior, which are the prerequisites for a resonance. Therefore, when seismic waves propagate through a deposited composed of such soils a resonance of the deposit may take place which greatly amplify the incoming seismic motion and cause considerable damage. On the other hand soil failure due to cyclic degradation of stiffness and strength in highly plastic clay deposits is unlikely, because the degradation is small.

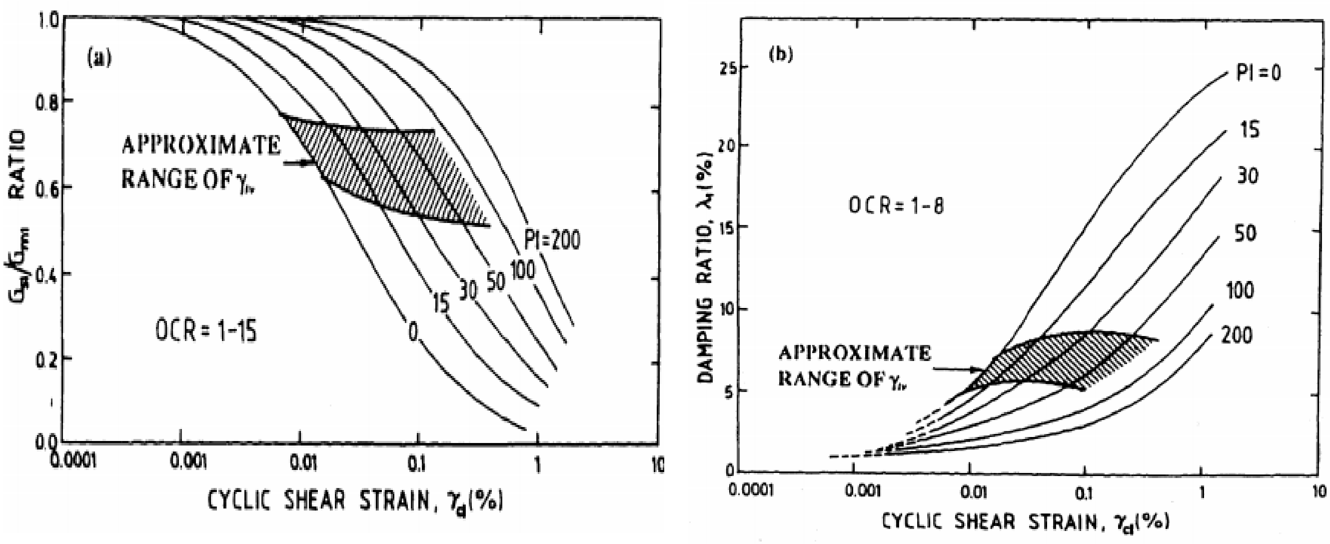


Table 9 Effect of PI on modulus reduction and damping ratio (M. Vucetic, 1992)

Confining pressure: With an increase of confinement, the shear modulus of soil increases and damping ratio decreases. This effect is predominately visible in case of cohesion less soils. However, confining pressure does not show 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 (Seed et al. 1988).

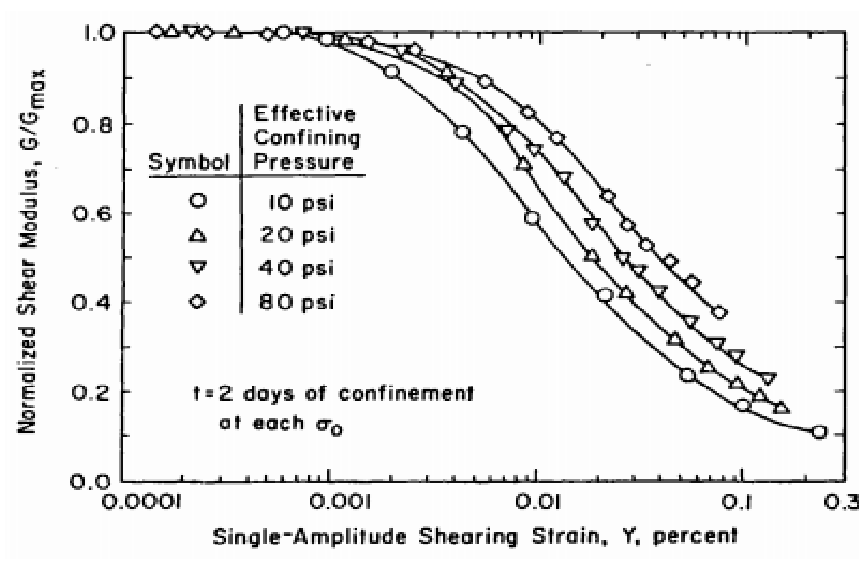


Figure 11 Effect of confining pressure on normalized modulus reduction *(M. Vucetic, 1992)*

Consolidation history: (Seed et al. 1988) summarized the effect of consolidation histories for undisturbed natural clay soils having medium to high plasticity index. As seen from Figure 12 , the shear modulus and damping ratio values are within narrow strand and do decrease or increase with the same proportion over a wide range of strain. Hence, neither shear modulus nor damping is significantly influenced whether a soil is normally consolidated or over consolidated.

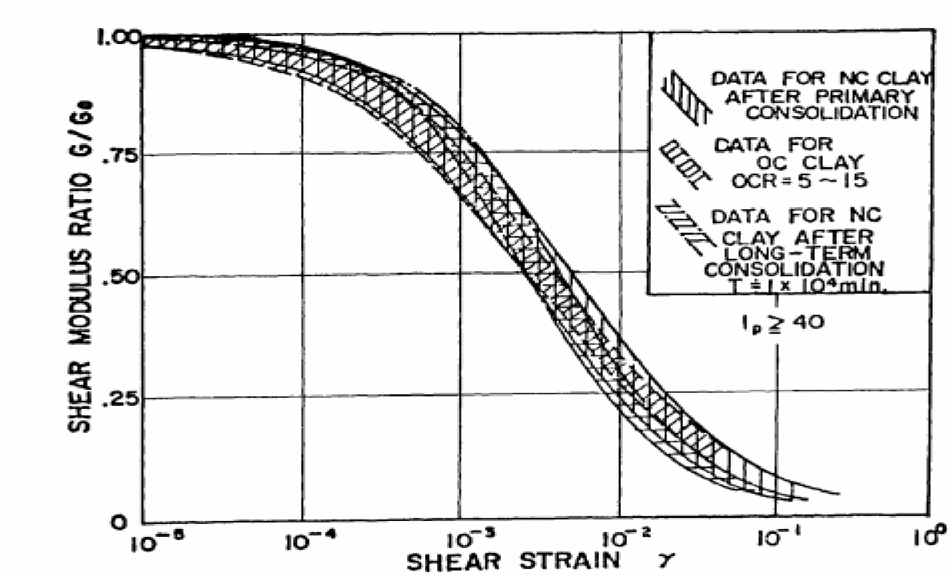


Figure 12 comparison of normalized modules reduction relationship of clay for deferent consolidation states

Void ratio: Void ratio is an important factor influencing shear moduli and damping ratio of sands. As studied by Hardin and Drenvich (1972), the shear modulus increases and damping ratio values decrease with a reduction in void ratio.

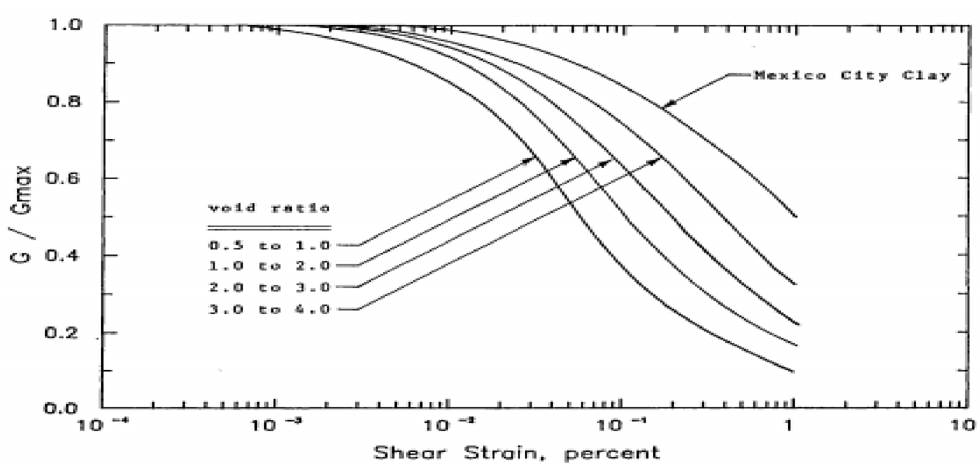


Figure 13 Normalized modulus reduction relation for clays with different void ratio

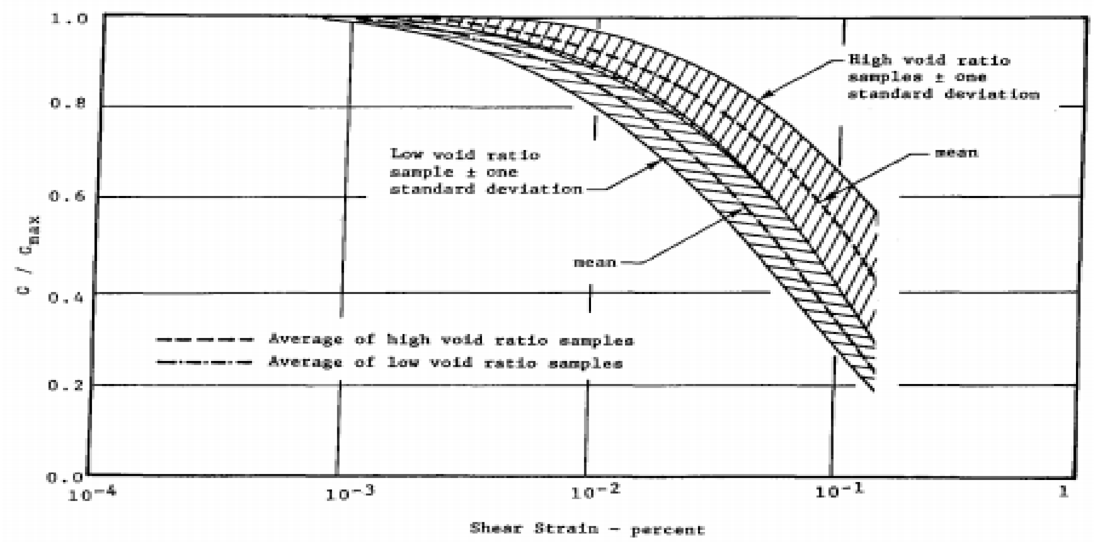


Figure 14 Effect of void ratio on normalized modulus reduction relationship

(Seed, Sun, & R. Golesorkhi, 1988)

The effect of percentage of fines, material grading, and plasticity index and stress history are significant on dynamic property of soil for small strain level ($\gamma \leq 0.001\%$). Whereas for problems that involve medium to large strain ($\gamma > 0.01\%$) the effect of loading frequency and number of cycle on stress strain property of soil are significant. (Mengistu Mena, 2020)

Table 10 : Summery on the effect of increase various factors on G, Gmax and damping ratio, λ of normally consolidated and moderately over consolidated clays (R. Dobry & M. vucetic, 1987)

Increasing Factors	G_{max}	G/G_{max}	λ
Confining pressure, $\overline{\sigma_c}$	Increases with $\overline{\sigma_c}$	Stay constant or increase with $\overline{\sigma_c}$	Stay constant or decrease with $\overline{\sigma_c}$
Void ratio, e	Decreases with e	Increase with e	Decrease with e
Geological age, t	Increases with t	May increase with t	Decrease with t
Cementation, c	Increases with c	May increase with c	May decrease with c
Overconsoliodation, OCR	Increases with OCR	Not affected	Not affected
Plasticity index, PI	• Increases with PI if OCR >1 • Stays about constant if OCR=1	Increase with PI	Decrease with PI
Cyclic strain, γ_c	-	Decrease with γ_c	Increase with γ_c

Stress rate (Frequency of cyclic loading), $\hat{\gamma}$	Increase with $\hat{\gamma}$	Not affected if G and G_{max} are measured at same $\hat{\gamma}$	Stay constant or may increase with $\hat{\gamma}$
Number of loading cycle, N	Decreases after N cycles of large with recovers later with time		Not significant for moderate γ_c and N

Many problems in civil engineering practice require the knowledge of the properties of soils subjected to dynamic loading. These problems includes the dynamic bearing capacity of foundations, response of machine foundations subjected to cyclic loading, soil-structure interaction during the propagation of stress wave generated due to an earthquake, earthquake resistance of dams and embankments, and fast transportation systems (Das B. M., 2011). Thus, many researcher have done researches on studying the behavior of soil under cyclic loading for both natural and stabilized soil. Same are discussed below:

2.6 Previous Researches on Dynamic Property of Stabilized Soil

On the paper presented at 12th World Conference on Earthquake Engineering, February 2000 by the title ***“Dynamic Properties of Untreated and Treated Cohesive Soils”*** by ***K K Chepkoit and M S Aggour*** uses four commercially produced clay and a natural clay obtained from a site in Virginia USA with a wide range of plasticity index ranging from 22 to 600%. A hydrated lime with specifications required for lime for soil stabilization were used (ASTM C977). The percentage of lime used for treatment varied from 2 to 8% by weight. A resonant column device was used to determine the dynamic property of the soil. All lime treated samples were prepared as per the procedure recommended in ASTM D3551.

Finally it is found that cohesive soils with low plasticity exhibit a high shear modulus at low strain levels. At high strain levels, all soils regardless of their plasticity tend to converge, because the shear modulus of low plasticity soils decreases rapidly with increases in shear strain, as compared to cohesive soils with high plasticity. Lime treatment has been shown to reduce the plasticity of cohesive soils, hence it showed a significant effect on the dynamic properties of the treated cohesive soils. High treatment levels of lime exhibit high shear modulus in all treated cohesive soils at low strain levels. Model as a functions of shear strain, and plasticity or treatment level was developed.

Argaw Asha and ***Nihar Ranjan*** present a paper on Seventh International Conference on Case Histories in Geotechnical Engineering 2013 by the title ***"Dynamic Properties of Stabilized Subgrade Clay Soil"*** the study was conducted a soil samples collected from Banda, Uttar Pradesh, India which has a risk of seismic damage. The soil was highly expansive in nature and used in road subgrade. The clay soil is treated with Rice Husk Ash and Portland cement slag of different proportion and the optimum content (82.5% Soil, 7.5% Portland slag cement and 10% Rice Husk Ash) was chosen for static as well as cyclic tests. The variation of dynamic properties of the stabilized clay soil such as shear modulus (G), damping ratio (D), and degradation index (δ) with number of cycles for different strain amplitude of γ= 0.4% to 1% at frequency of 0.5Hz or 30cycle/min and effective confining pressure of 100kPa have been studied.

The results show that the damping ratios increase with increase in amplitude of shear strain and vary from 7-9% to 14-19% for γ= 0.4% to 1% respectively. The shear modulus decreases with increase in amplitude of strain.

Min Wang et al. (2012) in this study the influence of water content, confining pressure. Vibration frequency, consolidation ratio and cycle number on the dynamic characteristics of clayey soil treated with 1% lime are investigated. The samples were cured for 28 days. They have concluded that dynamic modulus increases with increment of vibration frequency, consolidation ratio, and confining pressure, while it decreases from the increase in water content and vibration number. In the case of damping ratio, it gradually decreases with the growth of vibration frequency and water content. Keeping other factors constant however damping ratio increase with increasing of consolidation ratio, confining ratio, confining pressure, and vibration number. It is also showed that consolidation ratio has a strong impact on the dynamic properties of lime- treated clayey soil.

On the paper presented by **Yung-Chieh Chiang** and **Yong S. Chae** on the national academies of science engineering and medicine: high research board by the title "***Dynamic properties of cement treated soils***' uses cement to treat sand and clay subjected to dynamic loads. It is found that cement is very effective in increasing the rigidity of the soil and in reducing the deformation. They have concluded that when resonance is a problem in foundation design, it can be avoid by cement stabilization that produces changes in the elastic properties of the soil. In detail the research showed that dynamic shear moduli for both materials decreases with increasing shear-strain amplitude and the effect is most pronounced at small strain amplitude. It is also observed moisture content doesn't have significant effect on dynamic shear modulus for cohesionless soils. However it rapidly reduces the dynamic shear modulus of the clay soil as the moisture increased. In this research an experimental study was conducted to determine the dynamic shear modulus and damping characteristics of uniform sand and silty clay treated with Portland cement using the resonant column technique. For the test cement content, confining pressure, shear- strain amplitude, and moisture content were used as a test variables.

H. soltani-Jigheh and A. Soroush (2010) on the research by the title ***"Cyclic behavior of mixed clayey soils"*** uses cyclic triaxial test to evaluate the effect of number of loading cycles, cyclic strain amplitude, granular material content, grain sizes and effective confining pressure on the degradation of strength and deformability and pore water pressure behavior of a mixed clay. The clay that is used for the study is mixed with 0%, 20%, 40%, and 60% of sand or gravel. The results show that the addition of sand and gravel grain in to clay matrix increases the degree of degradation and pore water pressure building up during cyclic loading. On the other hand degradation index decreases as number of loading cycle and cyclic strain increases.

Braja M. Das et al. (1995) uses a laboratory model tests to conduct evaluation on the nature of settlement of a surface strip foundation supported on geogrid-reinforcement and without reinforcement saturated clay while being subjected to combination of static and cyclic loading of low frequency. They have observed that for a given static loading the maximum permanent settlement increases with the increase in amplitude of cyclic load intensity. It is also found that putting a full depth geogrid reinforcement may reduce the permanent settlement of a foundation by about 20% to 30% compared to one without reinforcement.

Pei-Hsun and Sheng-Huoo (2012) In this paper the dynamic property of cement stabilized soil, slang cement stabilized soil, and cement with 6% sodium silicate stabilized soil were studied using resonant column test. The magnitude of confining pressure, amount of cement admixture, and shearing stain amplitude were the parameters considered in the study. The engineering properties of this soil are: liquid limit LL=40%, plastic limit PL=22%, unit weight =17.2 kN/m^3, natural water content Wn=37%, and un-drained shear strength su=30 kN/m^2. Three cement contents of 5%, 15% and 25% by weight of soil were used in this study to investigate the effect of cement contents on cyclic deformation characteristics of cement stabilized soil. Test results show that the maximum shear modulus of cement stabilized soil increases with increasing confining pressure, the minimum damping ratio decreases with increasing confining pressure. However, the relationship between the maximum shear modulus and the confining pressure varies with the type of additive. The shear modulus of cement stabilized soil decreases with increasing shearing strain while the damping ratio increases with increasing shearing strain.

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Materials

3.1.1 Expansive soil

Expansive soil were collected from Addis Ababa, Akaki Kality sub city around Gelan Condominium. The area is found along the route of the road connecting Addis-Adama expressway with Lebu and Haile garment areas. As well the Addis-Djibouti railway also pass near to this area. At present there is dramatic increase urbanization in the area with the construction of Gelan condominium settlement and many privately owned houses. Addis-Djibouti railway Gelan station, the new federal penitentiary, privately owned industrial parks constructed by Kerchanshe group and Adama Korkoro plc., Akaki Kality deep water project and other facilities are also found in the area. Looking at only the industry park developed by Kerchanshe Group the “Gelan Assembly Plant and Commercial Park” lies on 150,000 m^2 of land with three car assembling factories in it. Besides it also contains commercial and recreational facilities.

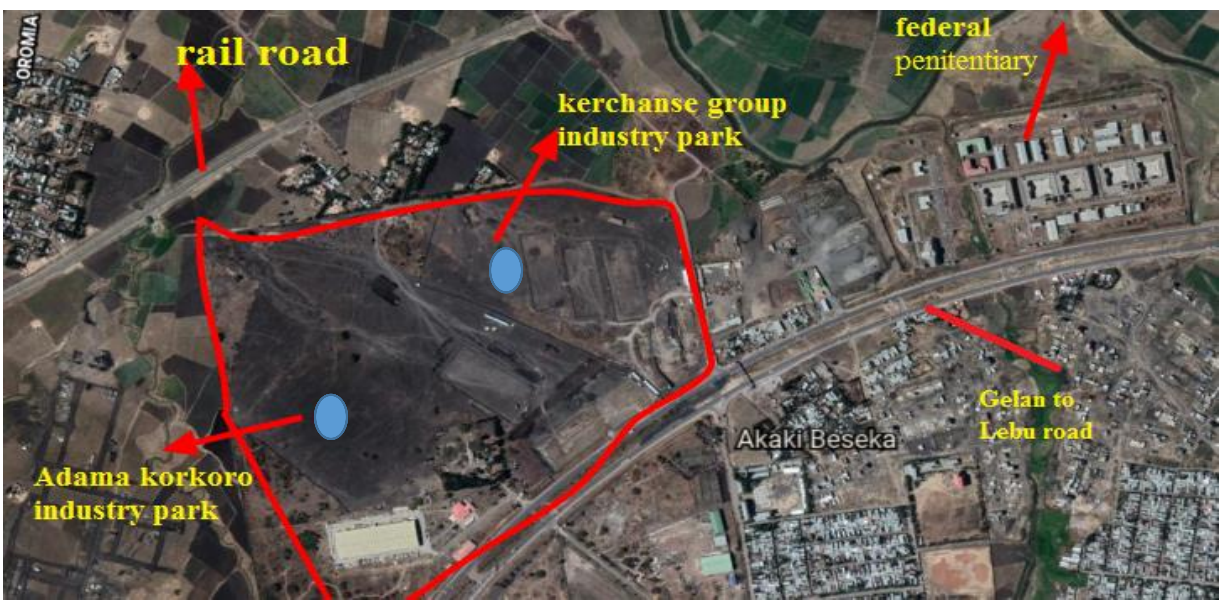


Figure 15 : Areal view of the study area

Based on the 1995 Ethiopian Building Code of Standards on seismicity the country is divided in five seismic zones approximately equal seismic risks depending on the known distribution of earthquakes. These zones are no damaging zone (0 zones), less damaging zones (zone 1 and 2) and zones of major damaging (zone 3 and 4). Based on this zoning the study area is found under region three of seismic zone based on the 1995 Ethiopian Building Code of Standards on seismicity.

Considering the industrial parks two disturbed soil samples out of two test pits with 1.5m depth have been collected from representative locations.

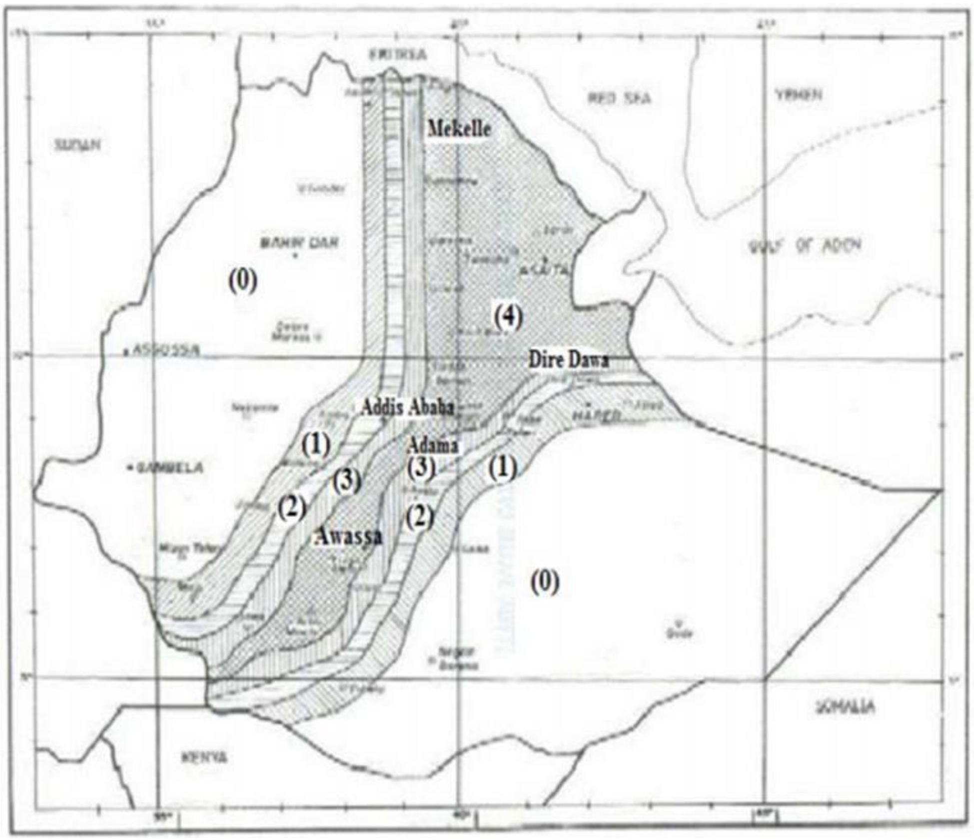


Figure 16: Seismic Map of Ethiopia from EBCS, 1995

3.1.2 Sugarcane Bagasse Ash

The bagasse ash used in this study was brought from the new Wonji sugar factory which is situated about 28Km from Adama to the south between the towns of Awash Melkasa and Dera. The fresh bagasse ash released from the furnace which was cooled in the air was collected. It was not treated with any chemicals. The bagasse ash was sieved and those grain sizes that pass through 150µm (No. 100) were used.

Base on the chemical composition that is given in table 5 for the SCBA that is obtained from Wenji shoa sugar factory for this research (SiO_2 +Al_2O_3 + FeO_3 is 83.36%, LOI amount is 5.9%, and H_2O amount is 1.29), the SCBA is grouped under N-class pozzolana, according to ASTM C 618–05 chemical requirements for pozzolanic property.

3.2 Experimental Program

The objective of this research is to determine the proper amount of SCBA that could properly improve the physical and engineering properties of expansive soil that is obtained from the study area and study the behavior of the treated soil when exposed to cyclic loading. Thus, test programs that focuses on the accurate characterization of the sample soil and evaluation of its strength and physical characteristics alteration with addition of SCBA have been prepared. Besides, mainly a test program that could help to study the treated soil behavior when exposed for cyclic loading have been prepared. The first program in the test was to identify, collect, characterize and classification of the soil sample. The laboratory tests conducted in this stage include: Sieve analysis, Atterberg limits, Free swell, Moisture content, Specific gravity, Proctor compaction test (Optimum moisture content & Maximum dry density), Swelling pressure and Unconfined compressive strength tests (UCS). The second test program comprises tests that have a duty to evaluate the physical and engineering property alteration of the expansive soil when mixed with different dry mass percentile amount of SCBA. Based on previous studies and literatures SCBA percentile amount of 6%, 9%, and 12% together with untreated sample were used. The laboratory tests conducted in this stage include: Atterberg limits, Free swell, Proctor compaction test (Optimum moisture content & Maximum dry density), Swelling pressure and Unconfined compressive strength tests (UCS). From this stage the optimum amount of SCBA that had better treat the expansive soil has been selected. The third program consist of studying the behavior of the expansive soil that is treated with the optimum amount of SCBA and compare the results with the dynamic behavior of the natural soil. For this stage a cyclic simple shear apparatus has been used. It closely replicates the field loading condition due to the vertical propagation of shear waves during seismic excitations. The experimental scheme for this program is shown below:

Number of cycle	Confining pressure	Shear Strain	Strain frequency
40	100kpa	0.01%	1HZ
	200kpa	0.1%	
	400kpa	1%	
		3%	

Table 11 : Experimental Scheme for Dynamic Loading Test

3.3 Test Procedures

Reviewing literatures, academic journals, research articles and any previous works that are relatable to the study topic have been conducted before conducting the tests. The procedure for each test is given in the following sections.

3.3.1 Grain Size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles. ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils is used as a standard reference. Grain size analysis provides the grain size distribution, and it is required in classifying the soil. The hydrometer method is used to determine the distribution of the finer particles that were passed sieve size 0.075mm (N0.200). Mixer (blender), 152H Hydrometer, Sedimentation cylinder, Control cylinder, Thermometer, Beaker, Timing device are used for hydrometer analysis. A hydrometer analysis was performed to measure the amount of silt and clay size particles.

3.3.2 Natural Moisture Content Test

This test was performed to determine the water (moisture) content of soils. The water content is the ratio, expressed as a percentage, of the mass of “pore” or “free” water in a given mass of soil to the mass of the dry soil solids. Equipment used for this test were mentioned as follows; Drying oven, Balance, Moisture can, Gloves, Spatula. By using ASTM D2216 standard test methods for laboratory determination of water (moisture) content of soil.

3.3.3 Specific Gravity Test

The specific gravity of a given material is defined as the ratio of the weight of a given volume of the material to the weight of an equal volume of distilled water. Specific gravity of the soil sample was determined using ASTM D 854 procedures. It is used to calculate parameters such as void ratio, porosity, particle size distribution using hydrometer, and degree of saturation. It is determined by using a soil sample passing No. 10 sieve and a pycnometer.

3.3.4 Atterberg Limit Test

For each percentage of SCBA, Atterberg limit tests have been conducted. Atterberg limits are carried out to determine the consistency of fine-grained soils. For the determination of Atterberg limits, procedures given in ASTM D 4318 were used for liquid limit and plastic limit respectively. An Atterberg limits device was used to determine the liquid limit of soil using the soil passing through a 475 µm (No. 40) sieve. Plastic limit of the soil was determined by using soil passing through a 475 µm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index were used to classify the soil.

3.3.5 Compaction Test

Compaction test is a laboratory test used to determine the relationship between the moisture content and the dry density of a soil. Knowledge of the optimum moisture content and the maximum dry unit weight of soils is very important for construction specifications of soil improvement by compaction. The moisture content at which the maximum dry unit weight is obtained is referred to as the optimum moisture content. Adding water beyond the optimum value will reduce density. After determining natural moisture content for the virgin expansive soil, the samples were compacted according to ASTM D-698 with standard Proctor test to find moisture-density relationships of untreated soil samples. Air dried soil passing no 4 sieve were used to determine moisture-density relation of the soil mixed with varying proportions of the additives. Moisture content versus dry density graph was produced and optimum moisture content (OMC) and maximum dry density (MDD) were determined from the graph.

3.3.6 Free Swell Index Test

To study the swelling property of the soil, the simplest test conducted is free swell test. This test was suggested by Holtz and Gibbs in 1995 to measure the expansive potential of cohesive soils. The test is performed by slowly pouring 10gm of dry soil which passed through a sieve opening of 0.425mm (No. 4 sieve) into a 100 cm^3 graduated cylinder filled with distilled water. In this case tap water was used because of the unavailability of distilled water. Final volume of suspension is read after 24hrs.

Free swell is calculated as,

$$free\ swell = \frac{final\ volume - initial\ volume}{initial\ volume} * 100$$

3.3.7 Unconfined Compressive Strength Test

Unconfined compressive strength was conducted according to ASTM D-2166 standard. The Unconfined compressive strength test samples were prepared with optimum moisture and maximum dry density with standard proctor test. The prepared sample were cured for 7days. Curing is done by wrapping the remolded specimen with plastic sheet and placed it in a container with water bath to prevent moisture change for 7 days. The test was conducted on sample having a dimension of 38mm diameter and 76mm height. Unconfined compressive strength testing was performed on all extracted specimens using a strain rate of approximately one percent per minute. A data acquisition system was used to record the applied load and deformation. Corrections to the cross-sectional area were applied prior to calculating the compressive stress on the specimen. Each specimen was loaded until peak stress was obtained.

3.3.8 Swelling Pressure Test

The swelling pressure is defined as the vertical pressure required to prevent volume change of laterally confined sample when it gets water. This test was done in accordance with ASTM D-2435 standard with one dimensional consolidation method. In this test the samples were placed in the consolidation ring height of 20mm then, subjected to under a 7 kPa applied load is wetted and allowed to fully swell. At this point a standard consolidation test is conducted by applying incremental loads starting with 50kPa and ending with 1600kPa. The pressure required to revert the specimen to its initial void ratio (height) is used to define the swelling pressure. The swelling pressure tests were conducted on the treated and untreated expansive soil samples.

3.3.9 Cyclic Simple Shear Test

Nowadays there are different types of cyclic simple shear apparatuses in use. In this research, the type of apparatus used was 31-WF7500 cyclic simple shear machine which is developed by the controls group. The complete system is controlled by the UTS004 software application program. The system is designed to allow a sample to be consolidated and sheared under drained conditions. Cyclic simple shear tests can be conducted for a wider range of strain amplitude (that is, 10^{-2} % to about 5 %). This range is the general range of strain encountered in the ground motion during seismic activities (Das B. M., 2011). In the cyclic simple shear device, the shear strain is induced by horizontal movement at the bottom of the sample relative to the top. The horizontal diameter of the sample remains constant.

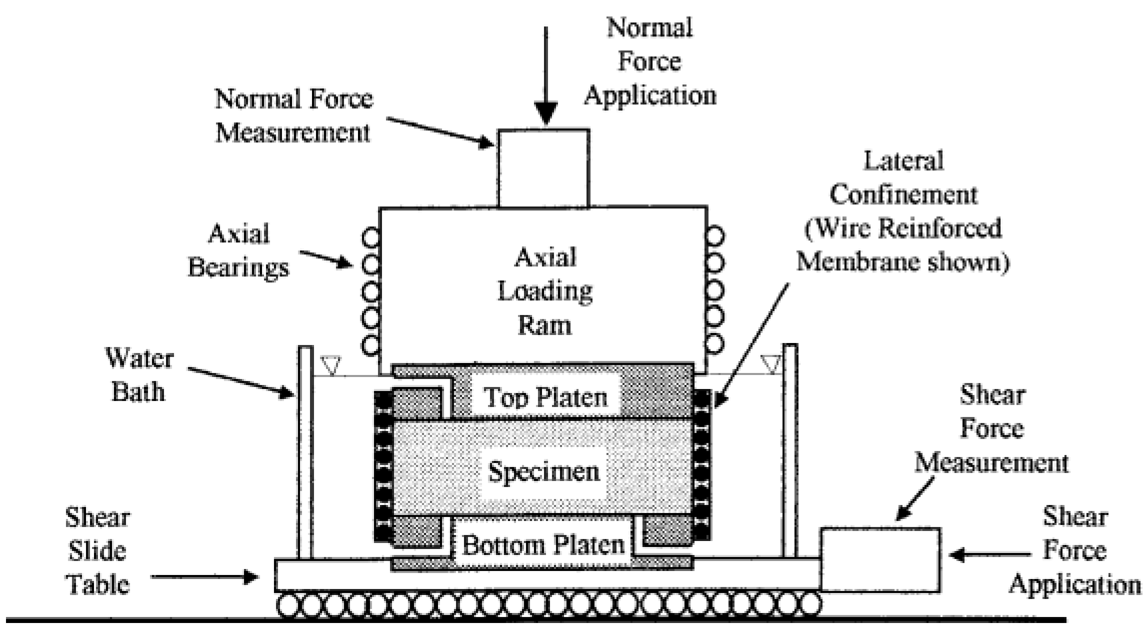


Figure 17 Schematic Diagram of Essential Direct Simple Shear Components

The sample is set up in the machine, which has a rigidly fixed top half and a moving bottom half. The top half houses the vertical ram. This is housed in a linear bearing to allow vertical movement and prevent horizontal movement. The bottom half is mounted on roller bearings as in a standard shear box. The sample is supported by a stack of close fitting-polished stainless steel rings. The rings provide confinement and produce a K_o condition during consolidation and also maintain a constant diameter throughout the test. During shear, the rings slide across each other as shown in figure18. During the shearing stage, of the test the vertical height of the sample is maintained at a constant height by the vertical actuator in a closed control loop with the vertical displacement transducer. The rings maintain a constant sample diameter.

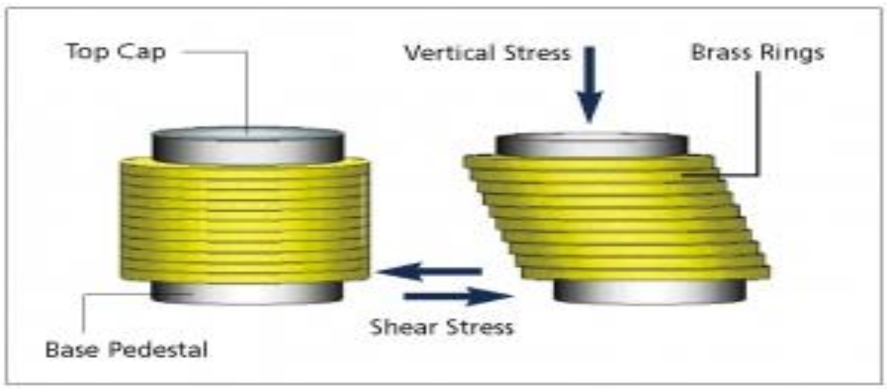


Figure 18 laterally constrained sample under cyclic simple shear conditions

The general procedures of ASTM D 6528 were used to prepare and test samples of soil in cyclic simple shear. Sample diameter is 70mm and height is varied from 20 mm to maintain height to diameter ratio less than 0.4, in order to fulfill the ASTM D 6528-07 criteria. The specimen testing begins with a consolidation stage which continues until testing is paused by the operator. Loading is then reinitiated for either a cyclic shear testing stage. While paused, the test loading control parameter may be altered and will take effect at the commencement of the next loading stage.

Consolidation Stage

The consolidation stage is simply the application of a 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.

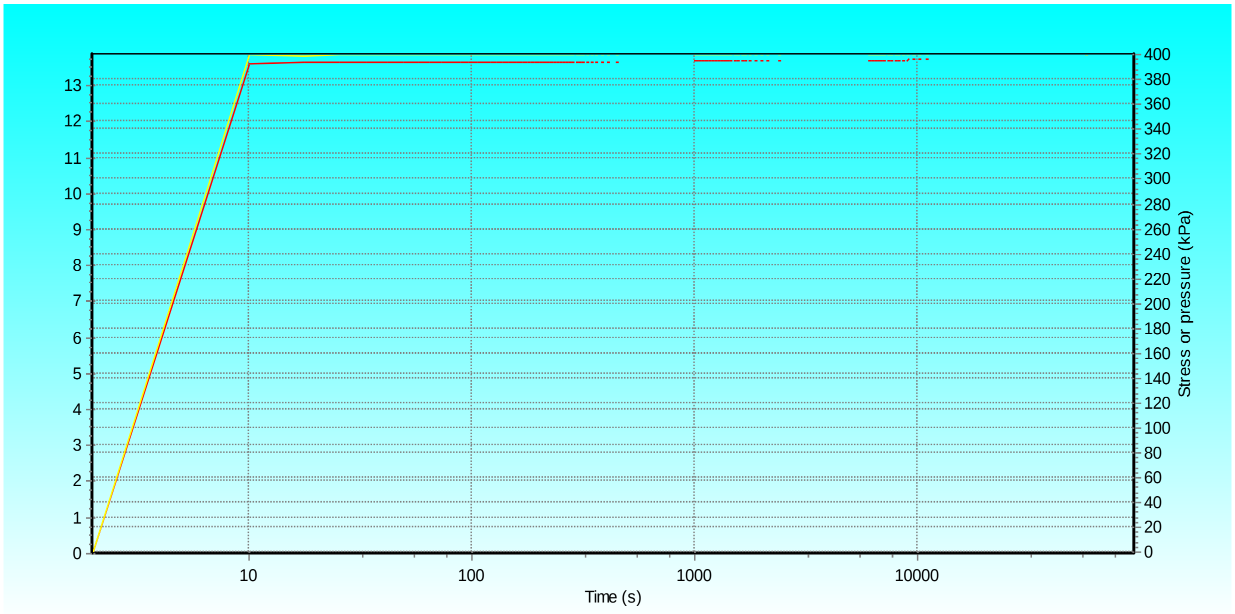


Figure 19: Consolidation stage at 400kPa axial stress and 1% shear strain amplitude

Cyclic Simple Shear Stage

This stage of the test applies a lateral cyclic shear force, or optionally a displacement, to the specimen, while the vertical force is either maintained at the specified stress, or optionally, the specimen height is maintained. Both axial and lateral force and specimen displacements are measured for each loading cycle. Measured data are obtained from 50 sample points captured over the cycle period. This data is displayed to the operator in the form of wave shapes, charts and tables and also logged by the system to an archive data file. The loading cycle shape is operator selectable from predefined functions but may also be a user generated shape.

The main advantage of simple shear tests is that the principle is straightforward: the shear stress applied in the horizontal direction. This helps cyclic simple shear test to simulate the in-situ conditions. The cyclic simple shear test represents more accurately the stress state (field condition) and most nearly simulates earthquake-loading condition. In addition to accurate representation of natural condition of stress state, it can measure dynamic soils properties over wide ranges of strain.

For this study, sinusoidal wave with 1Hz frequency, 40 numbers of cycles were used. A loading cycle shape has been selected to be sinusoidal from predefined shear shape options. Sinusoidal is the most common types of seismic wave shape for analysis (Yohannes , 2015). Axial/normal stresses of 100kPa, 200kPa and 400kPa to account the stress history of the soil (normally consolidated, and pre consolidated). It can also be conducted on different axial stress which enables one to see the effect of confining pressure on the value of shear modulus and damping ratio. 0.01%, 0.1%, 1%, and 3% shear strains were also selected. Which accounts for medium and large strain levels. Cyclic simple shear test gives simply raw data from 50 data points and 40 numbers of cycles for single cyclic loading in the form of Microsoft excel. Hence, for complete test of any selected pressure and amplitude, 2000 data points were recorded.

CHAPTER FOUR

TEST RESULTS AND DISCUSSIONS

In this chapter Laboratory test results are briefly discussed and the effect of sugarcane bagasse ash (SCBA) is evaluated for each laboratory tests. Revision on the physical properties of in-situ soil has also been summarized in this chapter, to make the comparison between treated and untreated soil samples easier. After the optimum amount of sugarcane bagasse ash mix is determined, the behavior of the soil and SCBA mix for dynamic loading and its dynamic soil properties are studied, as well as the results are briefly discussed.

4.1 Properties of Natural soils

The first laboratory tests were conducted on soil samples from test Pit 1 and Pit 2 to analyze the behavior of each soil sample. ASTM and IS standards are used for testing the expansive soil in the laboratory. Summary of the results are indicated in the table below:

No.	Tests	Test Methods	Test Results	
			Pit 1	Pit 2
1	**Grain Size Analysis**	ASTM D422		
	Clay, %		43.84	38.48
	Silt, %		50.28	58.65
	Sand, %		2.79	2.27
	Gravel, %		3.09	0.59
2	**Atterberg Limit**	ASTM D4318		
	Liquid Limit, %		94.25	82.25
	Plastic Limit, %		48.51	49.51
	Plastic Index, %		45.74	32.7
3	Free swelling, %	Is 2720	140	120
4	Specific Gravity	ASTM D854	2.57	2.55
5	Natural Moisture Content, %	ASTM D7263	28.33	28.67
6	UCS (kpa)	ASTM D2166	215.3	142.11

7	**Standard compaction test**			
	MDD (g/cc)	ASTM D698	1.259	1.242
	OMC, %		25.7	30.0
8	Swelling pressure test, (kpa)	ASTM D4546	156.29	162.83

Table 12 Properties of the Natural Soils

According to the laboratory test results of the natural soil sample obtained during the present study, the proportion of fines passing no 200 sieve 94%, liquid limit 94.25% & 82.25% and plasticity index 45.74% & 32.7% for soil from test pit 1 and test pit 2 respectively, the soil is classified in to A-7-5 as per the AASHTO and CH (high plastic clay) as per the USCS classification system.

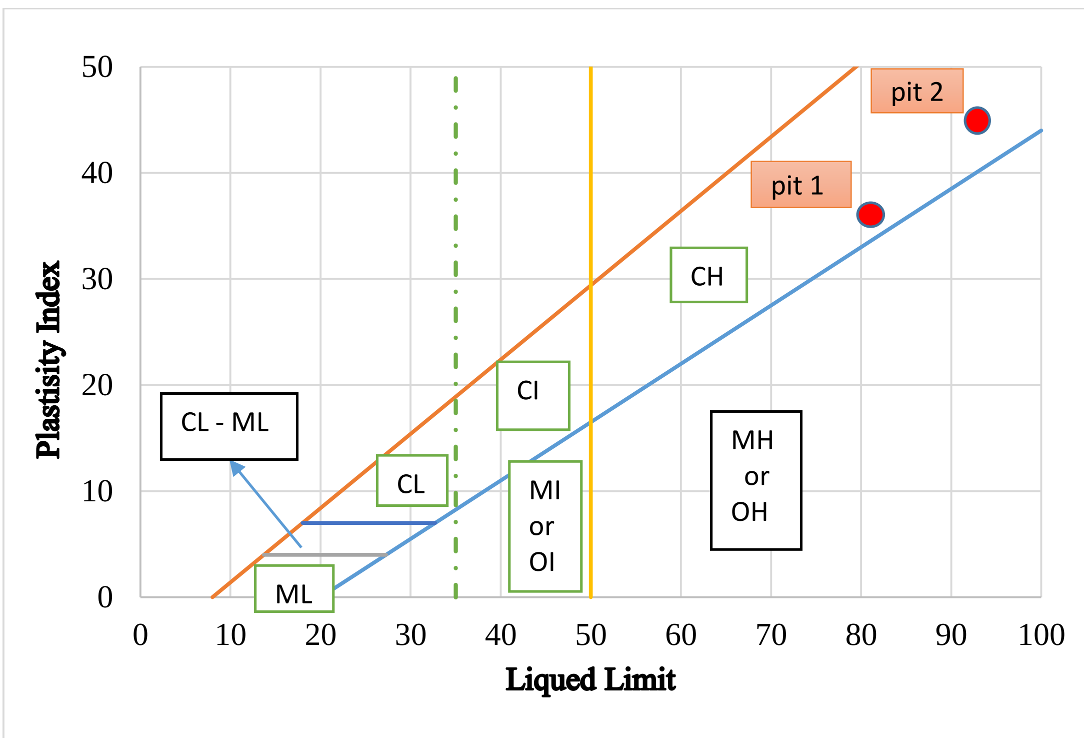


Figure 20 USCS plasticity chart

As far as the engineering performance of soils of this class is concerned, such soils are expansive soils which have high volume changing properties with variation in moisture content (Chen, 1988; McKeen, 1976).

The liquid limit and plasticity index values are very much greater than the Ethiopian Roads Authorities requirements, i.e., liquid limit less than 60% and plasticity index less than 30%

(ERA, 2002). According to ERA's design manual of 2002, soils with swelling potential and clay content greater than 35% and 28% respectively have high degree of expansiveness. Accordingly, sample show higher values in each parameter and the soil in general thus has very high expansive potential. The free swell index of 140% also revealed that the soil is very expansive soil, since its free swell index is greater than 100%.

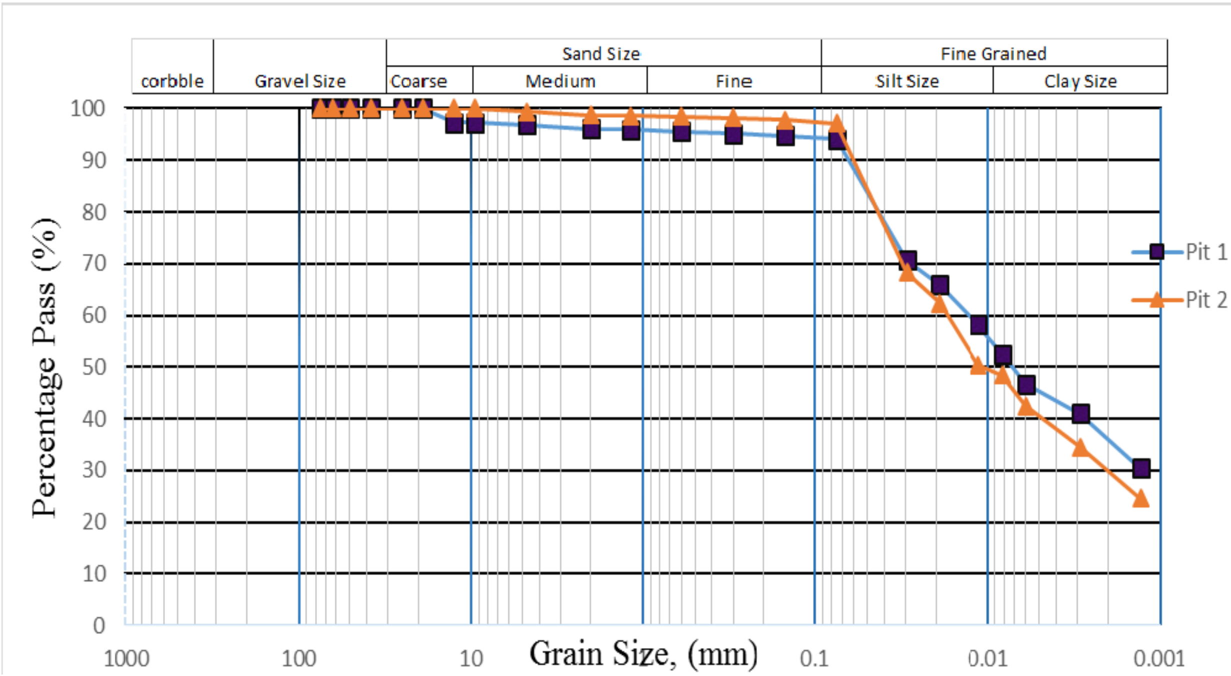


Figure 21 Grain size distribution curve for the natural soils

4.2 Effect of SCBA on Index Property of Soil

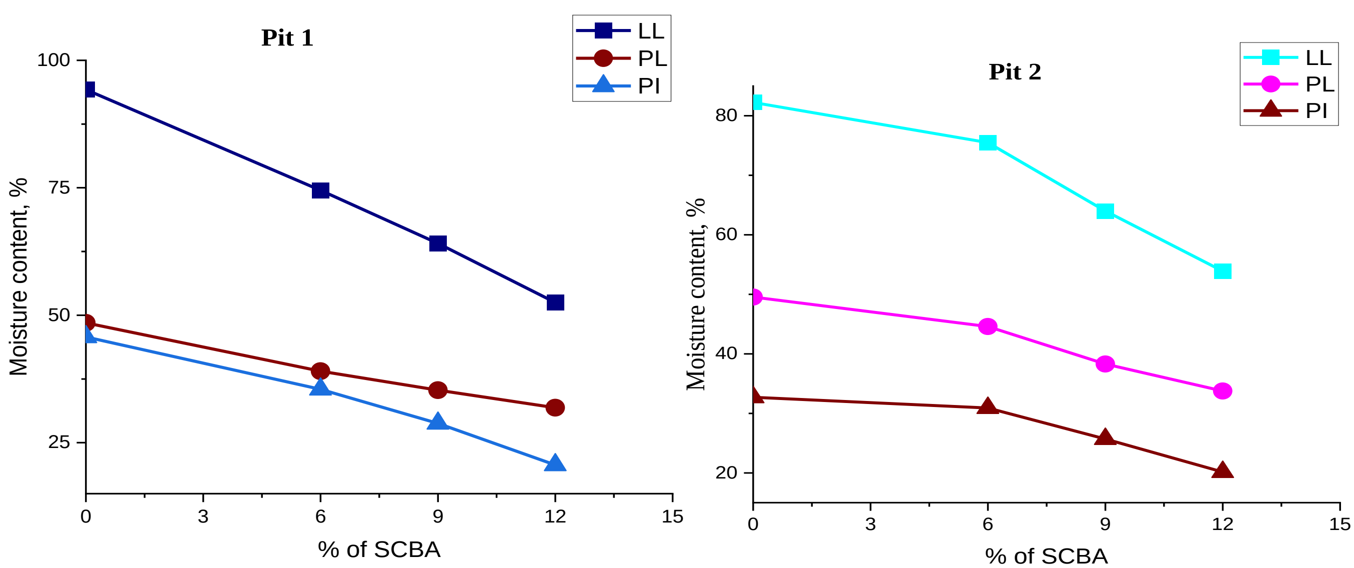


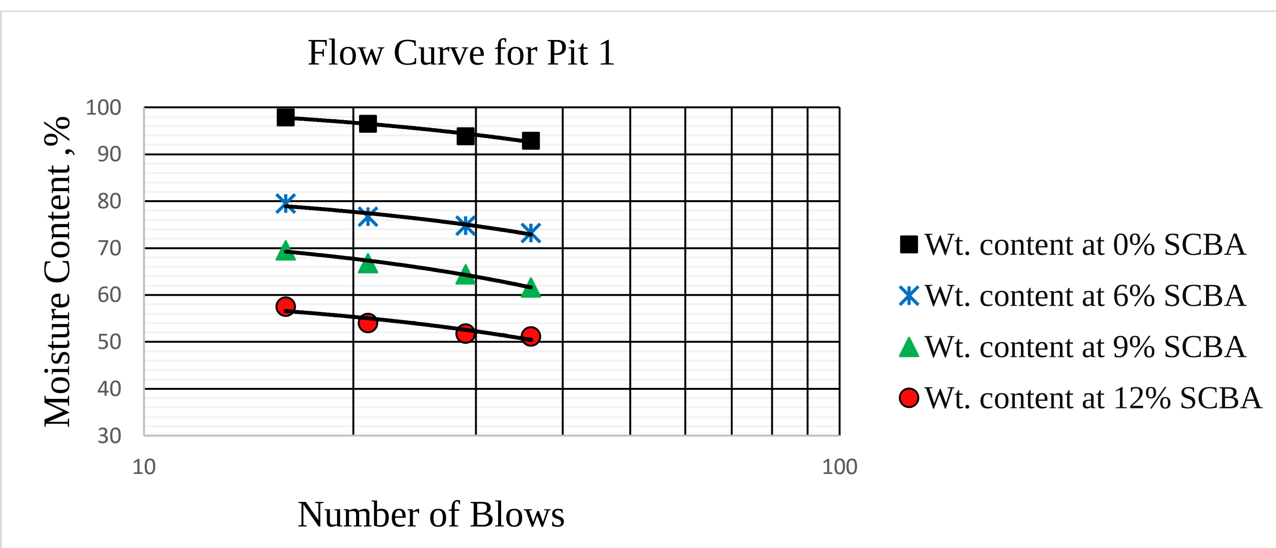
Figure 22 Effect of SCBA on LL, PL, and PI

The Atterberg limits are a basic measure of the nature of a fine-grained soil. Liquid limit results for soil and SCBA combinations at different percentages were shown in the above figure (22). The general decrease in liquid limit at all soil - bagasse ash combination was attributed to the fact that the bagasse ash reaction forms compounds possessing cementations properties with soil particles. This trend conforms to finding that the liquid limit reduces with increasing bagasse ash content.

Type of test	**Pit 1**				**Pit 2**			
Atterberg limits test	Amount of SCBA added				Amount of SCBA added			
	0%	**6%**	**9%**	**12%**	**0%**	**6%**	**9%**	**12%**
LL	94.25	74.5	64.1	52.5	82.25	75.5	64	53.9
PL	48.51	39.02	35.31	31.87	49.55	44.6	38.3	33.75
PI	45.74	35.48	28.79	20.6	32.7	30.9	25.7	20.15

Table 13**:** LL, PL, PI values for different amount of SCBA mix

For the soil- combinations at different percentage of bagasse ash, there was a general decrease in plastic limit with increasing SCBA content. The plasticity index decreases with increasing bagasse ash content. The reduction in plasticity index is mainly due to the formation of cementious compound and this effect could also be attributed to the combined action of partial replacement of plastic soil particles with non-plastic particles of SCBA. Which led to flocculation and agglomeration of clay particles. A reduction in plasticity index causes a significant decrease in swell potential.



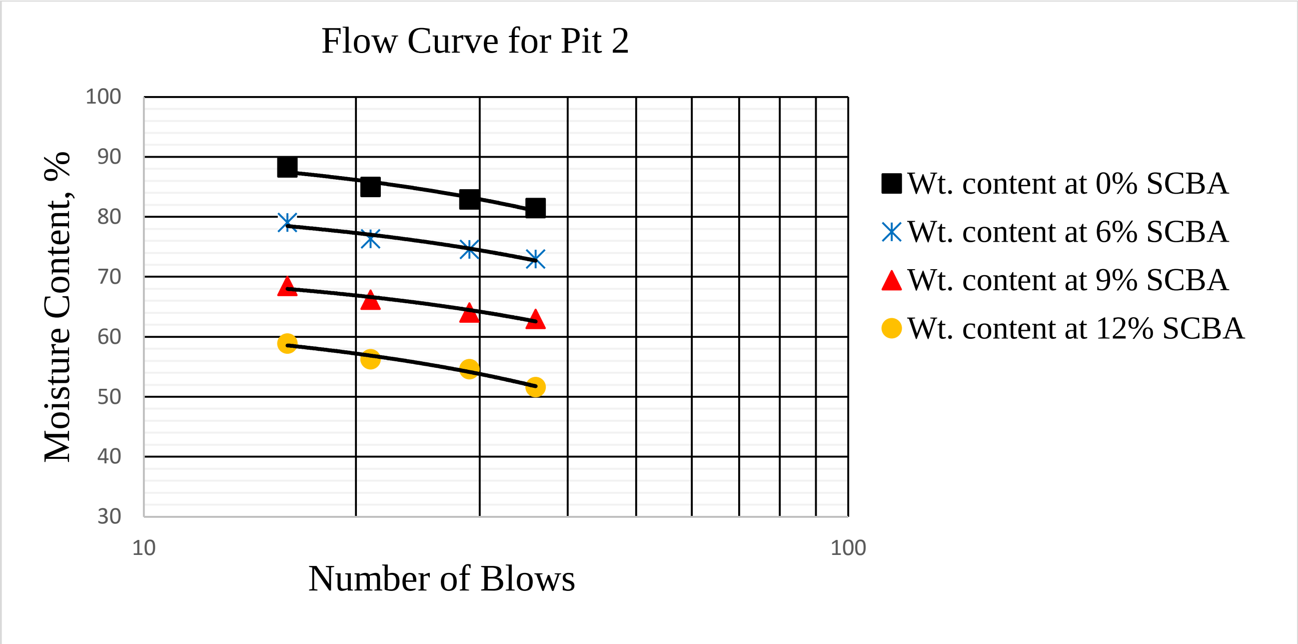


Figure 23 Flow Curve for liquid limit of expansive and treated soil

4.3 Effect of SCBA on Free Swell Index

The expansive soil samples have a free swell index value of maximum 140%. The result show that with the addition of SCBA there was a reduction in the free swell index values from 140% to 130%, 100% and 90% with the addition of bagasse ash for 6%, 9%, and 12% respectively for soil sample from test pit 1. For the test pit 2 with the same amount of SCBA addition the swelling index decreases from 120% for the natural soil to 110%, 90% and 85%.

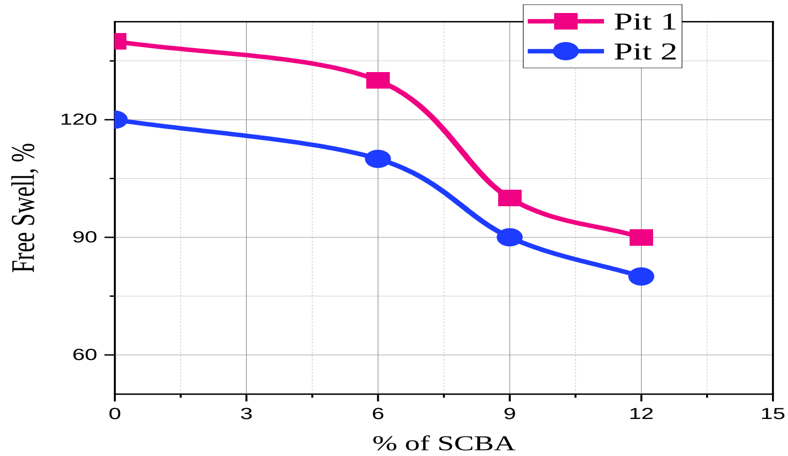


Figure 24 : Effect of SCBA on Free Swelling

4.4 Effect of SCBA on Moisture-Density Relations

The OMC of treated soil increases with increase in SCBA content but the MDD decreases with increase in SCBA. This could be due to the absorption of excess water by bagasse ash, the relatively lower specific gravity of bagasse ash and change in particle size compared to clay soil. The more fines the more surface area, so more water was required to provide well lubrication. The SCBA content also decrease the quantity of free silt and clay fraction, forming coarser materials, which occupy larger spaces for retaining water. The increase of water content was also attributed by the pozzalanic reaction of SCBA with the soil.

From Table 14 the maximum dry density of soil decreases gradually with an increase of SCBA content. The reduction in dry density was a result of flocculation and agglomeration of fine grained soil particles which occupies larger space leading to a corresponding drop in maximum dry density. It was also the result of initial coating of soils by SCBA to form larger aggregate, which consequently occupy larger spaces.

The advantage of the increase in OMC and corresponding decrease in MDD of the soil is that it allows compaction to be easily achieved with wet soil. Thus, there is less of a need for the soil to be dried to lower moisture content prior to compaction in the field (Meron et al., 2014).

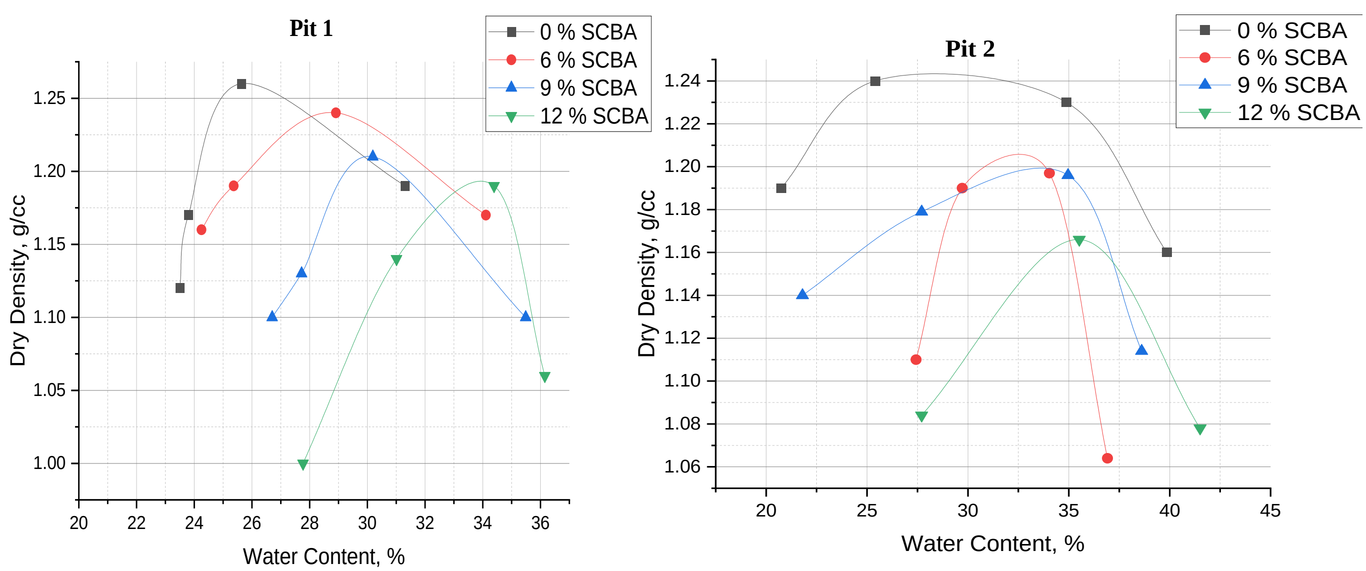


Figure 25 Effect of SCBA on Moisture - Density Relation

The summery test results are shown in below Table 13.

Table 14 Summery for Effect of SCBA on Moisture-Density Relations

Type of test	Pit 1				Pit 2			
Standard compaction test	Amount of SCBA added				Amount of SCBA added			
	0%	6%	9%	12%	0%	6%	9%	12%
Optimum moisture contain, OMC (%)	25.7	28.8	30.05	34.0	30	32.5	34.0	35.6
Maximum dry density, MDD (g/cc)	1.257	1.243	1.21	1.19	1.242	1.209	1.199	1.166

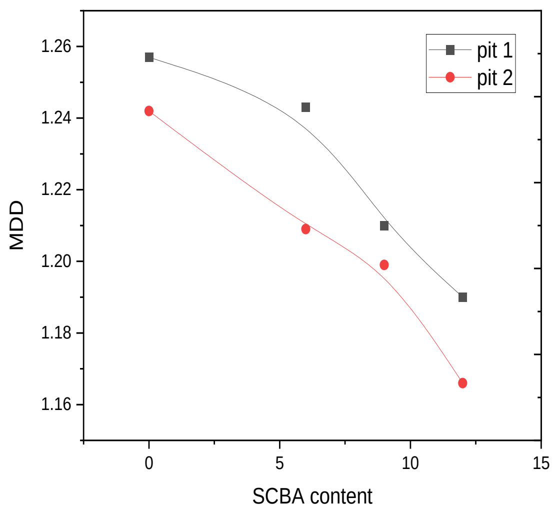


Figure 26 Relationship between MDD and treatment level

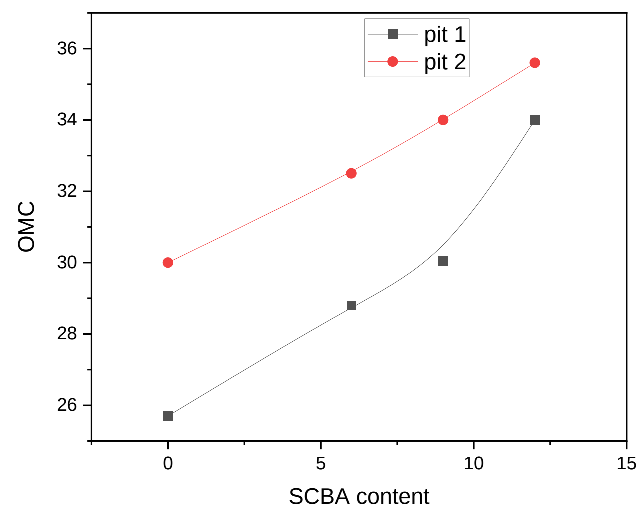


Figure 27 Relationship between OMC and treatment level

4.5 Effect of SCBA on Unconfined Compressive Strength (UCS)

Unconfined compressive strength was conducted according to ASTM D-2166 standard. This is the most common and adaptable method of evaluating the strength of stabilized soil. It is the main test recommended for the determination of the required amount of additive to be used in stabilization of soil. Each 7 days cured specimen was loaded until peak stress was obtained as shown in the figure (26):

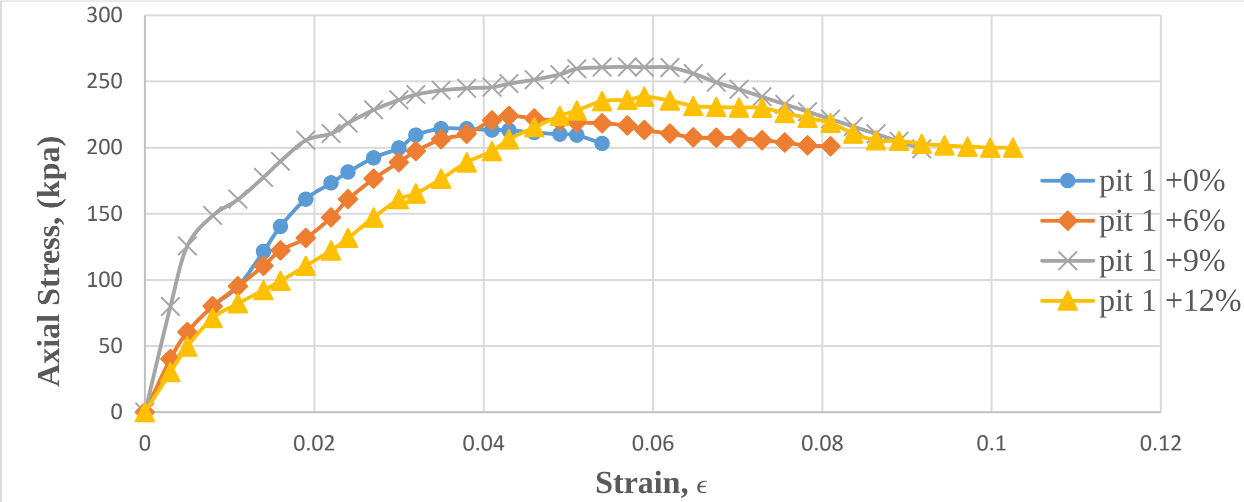


Figure 28 Stress Strain relation of UCS test for pit 1

The optimum amount of SCBA was selected from the specimen having the highest UCS values. Accordingly, from the mix it is found that the optimum amount is 9% SCBA. The laboratory test result of unconfined compressive strength has been shown in figure 25. The UCS value of expansive soil increase from 215.3kpa to 264.28kpa and from 142.11kpa to 239.77kpa for the addition of 9% SCBA for soil sample from test pit 1 and test pit 2 respectively. This could be the result of the increased pozzolanic reaction with increased amount of SCBA which results in the formation of calcium silicate hydrates and micro fabric changes which was responsible for increased strength. However further increase in SCBA amount didn't cause any improvement in the UCS but rather showed a decrease in strength.

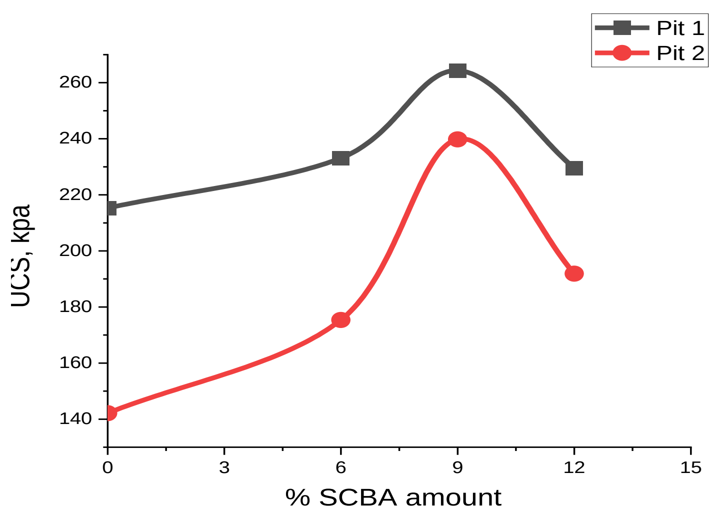


Figure 29 : Effect of SCBA on UCS values

4.6 Effect of SCBA on Swelling Pressure

The swelling pressure of natural soil was 156.29kpa for test soil from pit 1 and 162.83kpa for soil from test pit 2. Using the same procedure after treatment the swelling pressure decreases to 81.3kPa and 83.37kpa in addition of 12% SCBA. Appendix C shows the detail of the swell pressure test results. From both free swell and swell pressure tests, the treatment has a good effect in reducing the swelling properties of the expansive soil.

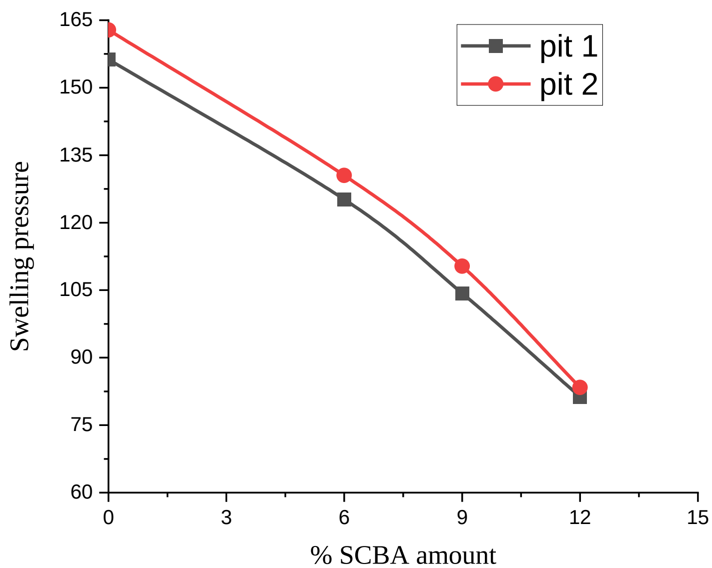


Figure 30 : Effect of SCBA on Swelling Pressure

Generally, considering the optimum amount 9% SCBA that is selected based on UCS test results; the reduction in swelling characteristics of the soil is not satisfactory. Thus methods that could elevates the pozzolanic characteristics of the SCBA should be adopted. One of the methods that could improve the results is the use of additional material like lime or cement which will be a source of calcium to react with silicate and aluminum ions present in the SCBA.

4.7 Laboratory Investigation of Dynamic Soil Properties

Computation of shear modulus and damping ratio is done using hysteresis loop of each cycles using computed shear stress and recorded shear strain. The lateral force and specimen displacement record of 50 points at the 5th cycle is used for the computation of shear stress and strain value.

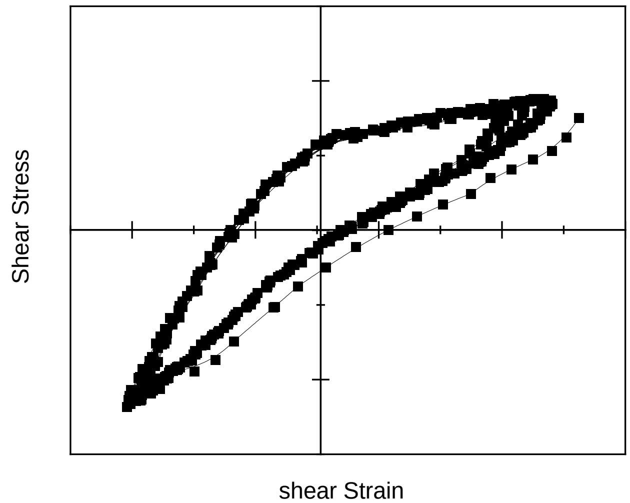


Figure 31: Hysteresis Loop for the first 10 cycles of untreated soil at 0.01 % strain and 100 kpa confinning pressure

Simulate repeated loading sinusoidal loading cycles has been used. In repeated loading like traffic, the number of significant cycles is likely to be more than 1000. For this research a cycle number of 40 were used.

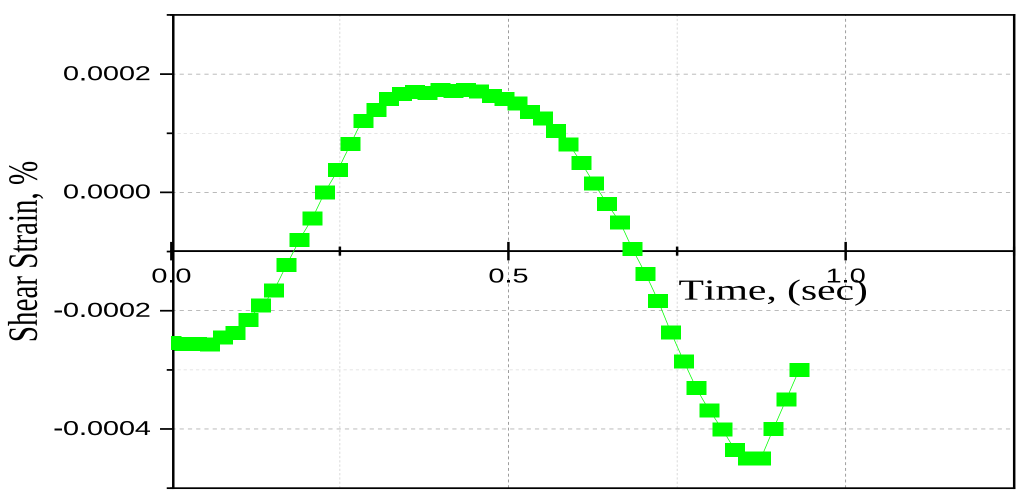


Figure 32: Sinusoidal wave shapes for 1 % strain of 100 kPa and 1 Hz for a single cycle

The use of different type and amount of material used for soil stabilization affect shear modulus and damping as it is observed from the literatures. The optimum amount of SCBA used for this part of the test was 9%, which is determined from the previous static load tests. The remolded sample were prepared based on the optimum moisture content and maximum dry density related to the 9% SCBA mixed soil and the natural soil. For this part of the laboratory investigation the soil sample with the lower UCS have been used. Which the soil from test pit 2 with UCS value of 239.77kpa is after treated. Curing of the specimens was done with the same procedure adapted for UCS test for 7 days. The method used for both treated and untreated soil samples is the same.

4.7.1 Determination of Shear Modulus and Damping Ratio of Untreated and Treated Soil

As compared with untreated soil, treatment results in an increase in dynamic shear modulus and damping ratio of the soil. Increase from 15% to 20% after stabilization is observed.

Shear modulus can be calculated using equation,

$$G = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}} \quad \text{(Das B. M., 2011)}$$

Typical calculation for shear modulus and damping ratio of natural soil. Is shown in the table below:

Table 15 : Typical calculation for shear modulus and damping ratio of natural soil.

Computation of shear modulus, G		Computation of damping ratio, D	
τ_{max}	0.03238	Area of the loop $A_{loop} = \frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$	0.00233
τ_{min}	-0.03467	Area of the triangle A(t) = 0.5*L*S L=$\gamma_{max} - \gamma_{center}$, S= $\tau_{max} - \tau_{center}$	0.000103
$\tau_{max} - \tau_{min}$	0.06705	Damping ratio D (%)	22.5103
γ_{max}	0.037105		
γ_{min}	-0.03814		
$\gamma_{max} - \gamma_{min}$	0.07524		
G (MPa)	0.8910		

Table 15 shows the shear modulus and damping ratio of natural soil with 1 % shear strain, at the 5th cycle, and 100 kPa axial stress.

Table 16 : Typical calculation for shear modulus and damping ratio for treated soil with 1 % shear strain, at the 5th cycle, and 100 kPa axial stress.

Computation of shear modulus, G		Computation of damping ratio, D	
τ_{max}	0.04563	Area of the loop $A_{loop} = \frac{1}{2}\sum_{i=1}^{n}(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$	0.001973
τ_{min}	-0.04321	Area of the triangle A(t) = 0.5*L*S	0.0000762
$\tau_{max} - \tau_{min}$	0.08884	Damping ratio D (%)	25.8922
γ_{max}	0.03721		
γ_{min}	-0.04977		
$\gamma_{max} - \gamma_{min}$	0.08698		
G (MPa)	1.0213		

The same procedure was used to compute the shear modulus and damping ratio for different shear strain amplitude for both natural and treated soil and it is given in the tables below:

Table 17 Shear Modulus and Damping Ratio Values for Untreated Soil

Untreated Soil								
	Axial Stress, 100kpa							
Strain, (%)	0.01	0.1	1	3	0.01	0.1	1	3
Cycle No.	G (Mpa)				D (%)			
5th	2.01	1.83	0.89	0.65	19.83	21.01	22.51	23.25
10th	4.88	3.75	1.25	0.97	18.24	20.01	21.30	22.53
20th	5.55	4.15	1.75	1.45	17.85	19.23	20.05	21.46
40th	6.08	5.33	3.26	2.03	13.98	15.65	18.35	20.56
Axial Stress, 200kpa								
Cycle No.	G (Mpa)				D (%)			
5th	2.56	1.95	0.98	0.83	16.83	18.35	21.09	22.52
10th	5.7	4.56	2.15	1.36	15.23	17.05	20.3	21.97
20th	8.01	7.26	5.21	3.07	13.85	15.53	19.75	20.55
40th	8.36	7.68	6.68	4.25	12.85	14.05	18.63	19.90
Axial Stress, 400kpa								

Cycle No.	G (Mpa)				D (%)			
5th	3.22	2.87	1.87	1.05	16.05	17.35	19.95	21.5
10th	6.11	5.35	2.53	1.69	14.23	16.19	18.3	20.35
20th	11.23	9.25	5.23	4.01	12.11	15.33	16.25	19.25
40th	12.36	10.12	8.36	5.11	11.25	14.26	15.37	17.05

Table 18 Shear Modulus and Damping Ratio Values for Treated Soil

Treated Soil								
	Axial Stress, 100kpa							
Strain, (%)	0.01	0.1	1	3	0.01	0.1	1	3
Cycle No.	G (Mpa)				D (%)			
5th	2.31	2.10	1.02	0.75	22.80	24.16	25.89	26.74
10th	5.61	4.31	1.44	1.12	20.98	23.01	24.50	25.91
20th	6.38	4.77	2.01	1.67	20.53	22.11	23.06	24.68
40th	6.99	6.13	3.75	2.33	16.08	18.00	21.10	23.64
Axial Stress, 200kpa								
Cycle No.	G (Mpa)				D (%)			
5th	2.92	2.22	1.12	0.95	18.51	20.19	22.33	24.77
10th	6.50	5.20	2.45	1.55	16.75	18.76	22.01	24.17
20th	9.13	8.28	5.34	3.50	15.24	17.08	21.73	22.61
40th	9.53	8.76	5.94	4.85	14.14	15.46	20.49	21.89
Axial Stress, 400kpa								
Cycle No.	G (Mpa)				D (%)			
5th	3.67	3.27	0.99	0.74	17.98	19.43	22.34	24.08
10th	6.97	6.10	2.88	1.93	15.94	18.13	20.50	22.79
20th	12.80	10.55	5.96	4.57	13.56	17.17	18.20	21.56
40th	14.09	11.54	9.53	5.83	12.60	15.97	17.21	19.10

4.7.2 Variation of shear modulus and damping ratio with shear strain

For different shear strain ranges, the variation of shear modulus with shear strain has been plotted for both natural and treated clay soil. It is observed that shear modulus decreases as the shear strain increases whereas the damping ratio increases with increasing shear strain. This is because at higher values of shear strain, the soil strength becomes smaller to resist deformation and more energy released to yield high deformation. The damping ratio increases with increase in shear strain due to energy dissipation during deformation of the soil.

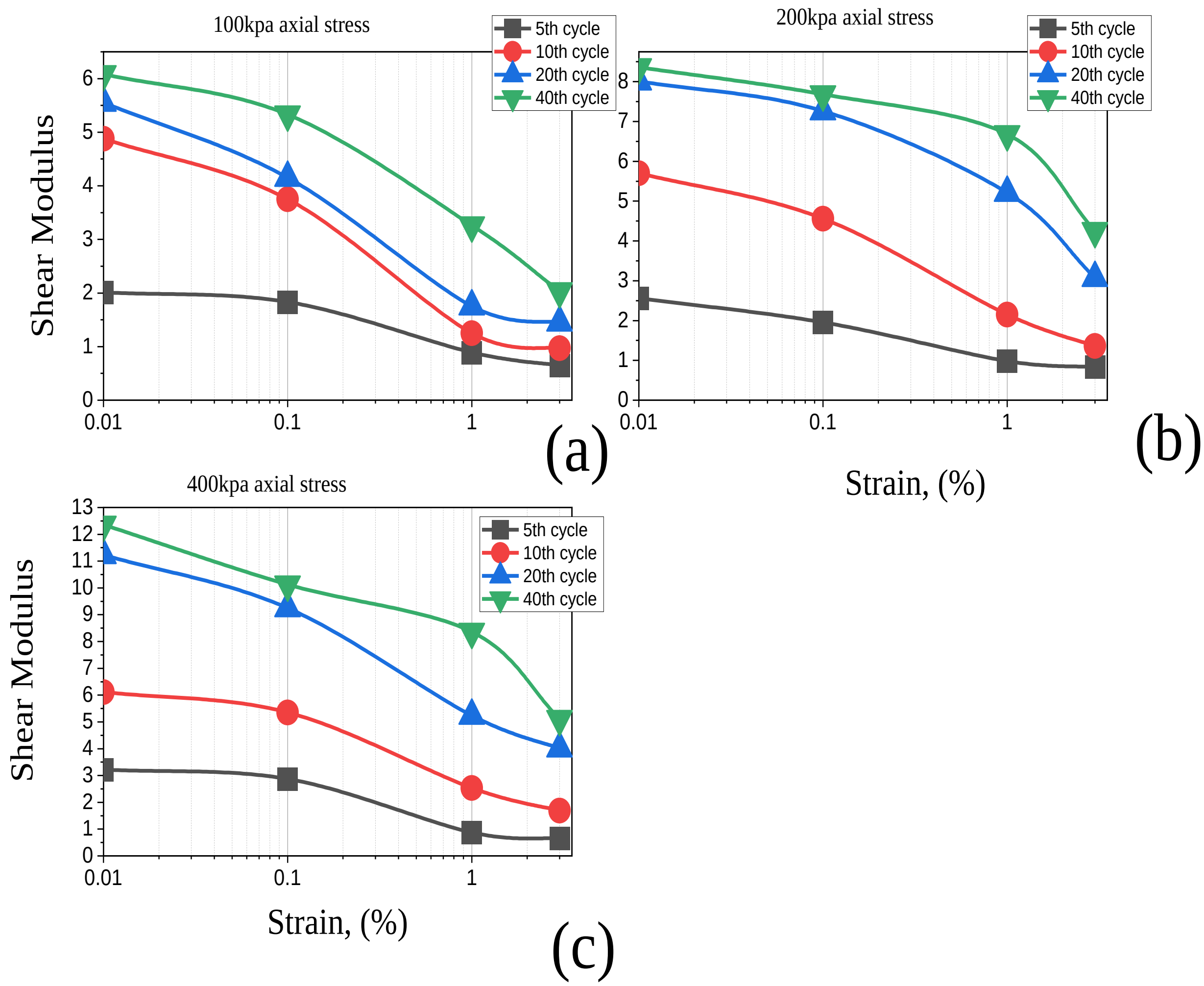


Figure 33 Variation of shear modulus with shear strain at (a) 100kpa, (b) 200kpa, and (c) 400kpa confining pressure for the untreated or natural soil

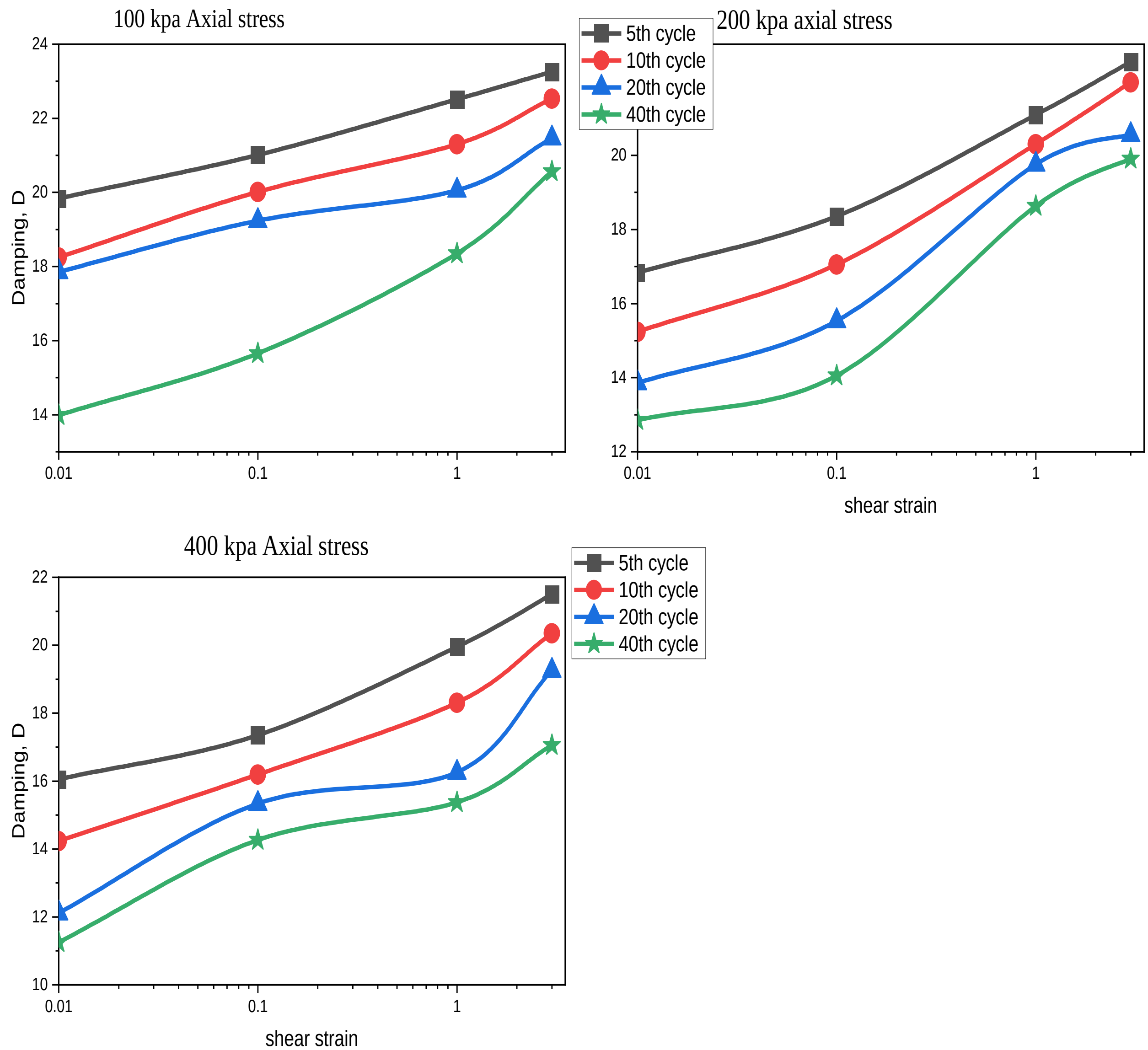


Figure 34 Variation of Damping with shear strain at 100kpa, 200kpa, and 400kpa confining pressure for the untreated or natural soil

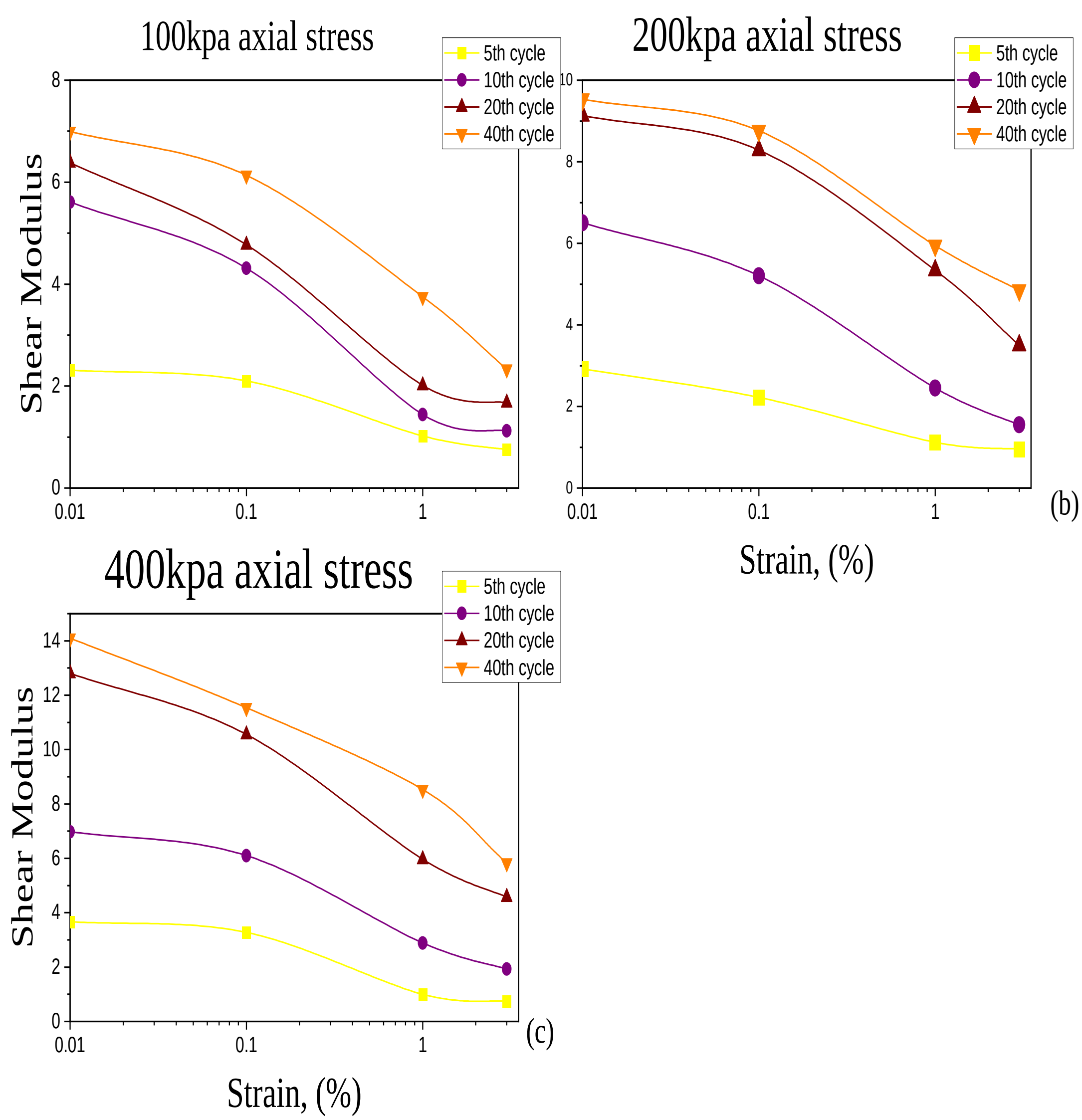


Figure 35 Variation of shear modulus with shear strain at 100kpa, 200kpa, and 400kpa confining pressure for the treated soil

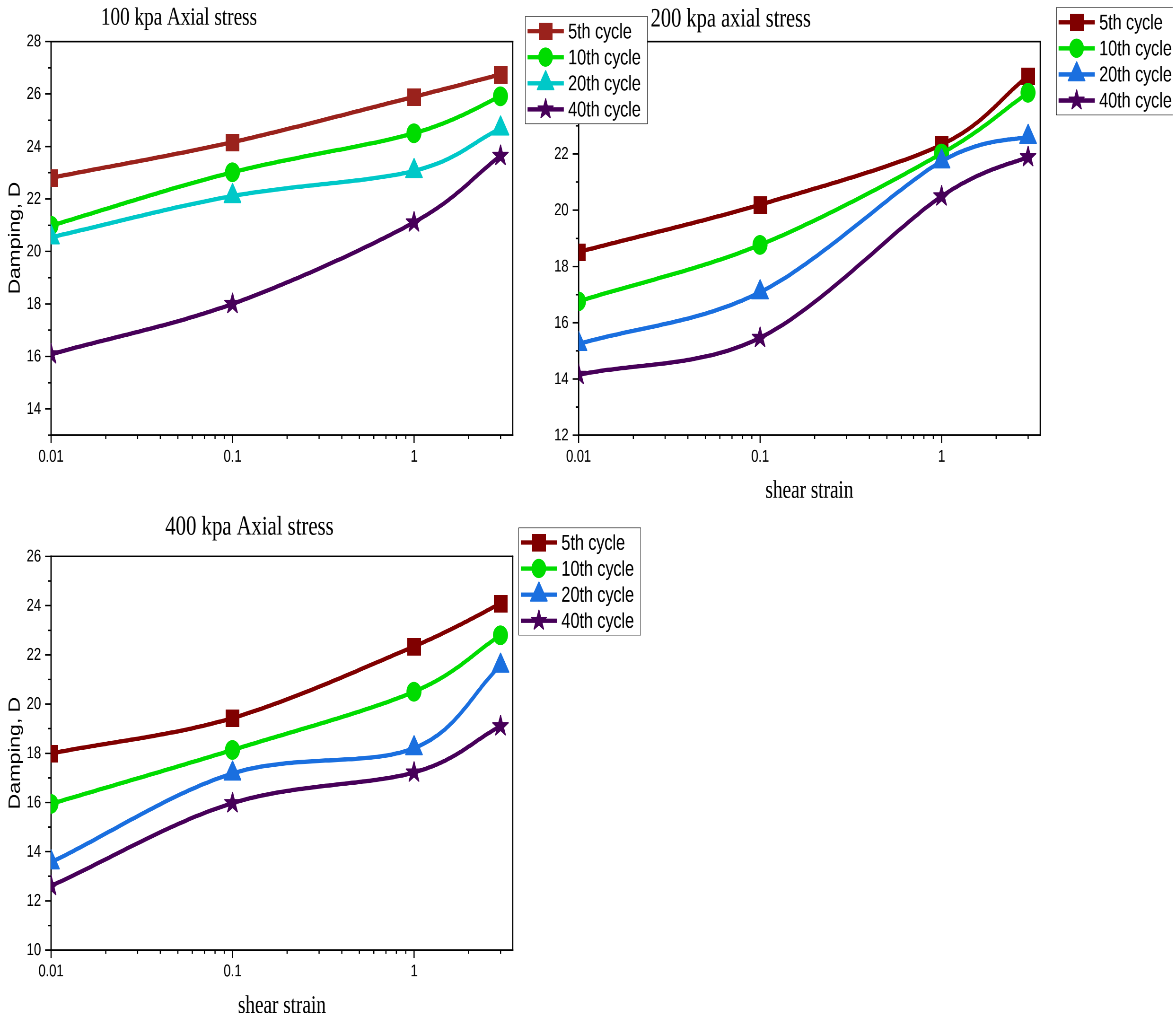


Figure 36 Variation of Damping with shear strain at 100kpa, 200kpa, and 400kpa confining pressure for the treated soil

4.7.3 Variation of shear modulus and damping ratio with number of cycles

Considering a single confining pressure which is 200Kpa and different shear strain amplitudes, the relationship between shear modulus and damping with cycle number is expressed on the graphs shown below.

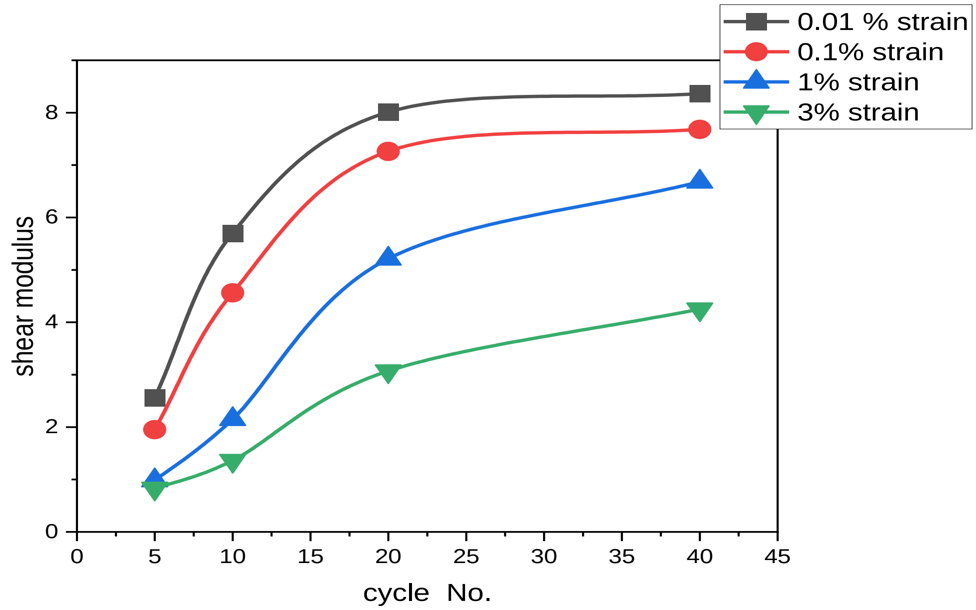


Figure 37 Variation of shear modulus with number of cycles for the natural soil at 200kpa confining pressure

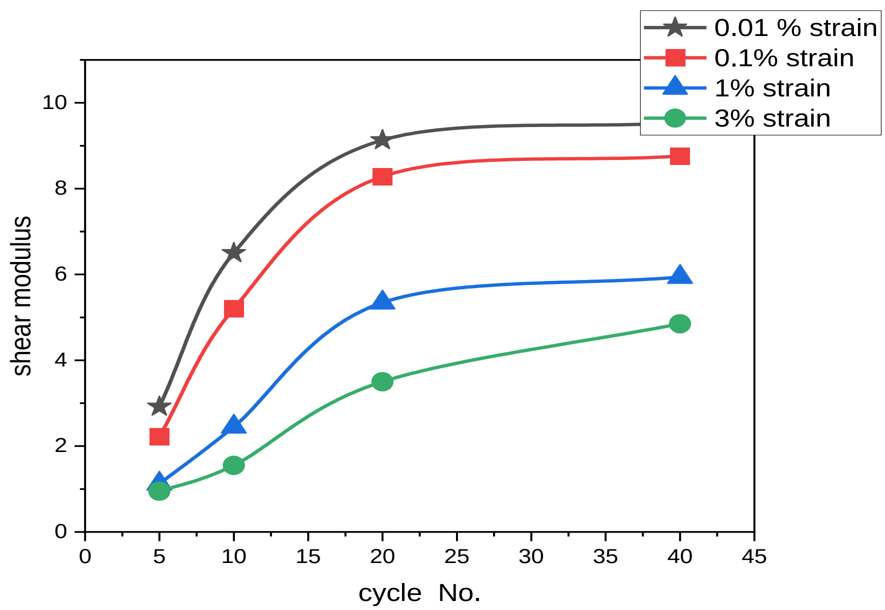


Figure 38 Variation of shear modulus with number of cycles for the treated soil at 200kpa confinning pressure

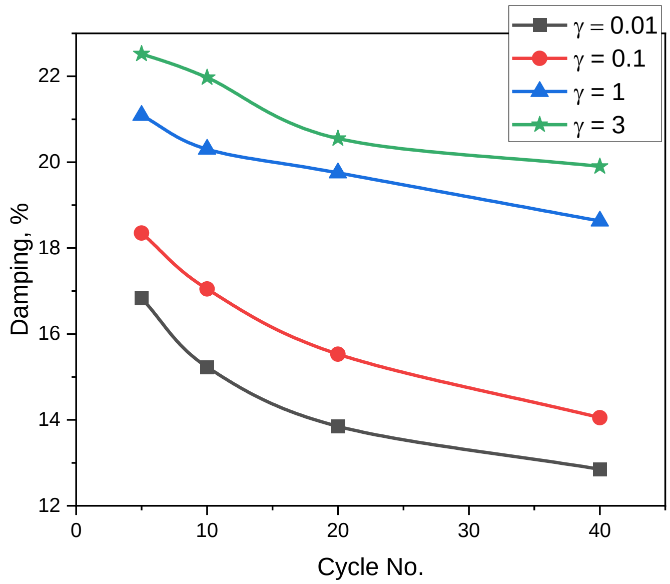


Figure 39 Effect of cycle no. on damping ratio for untreated soil at 200kpa confining stress

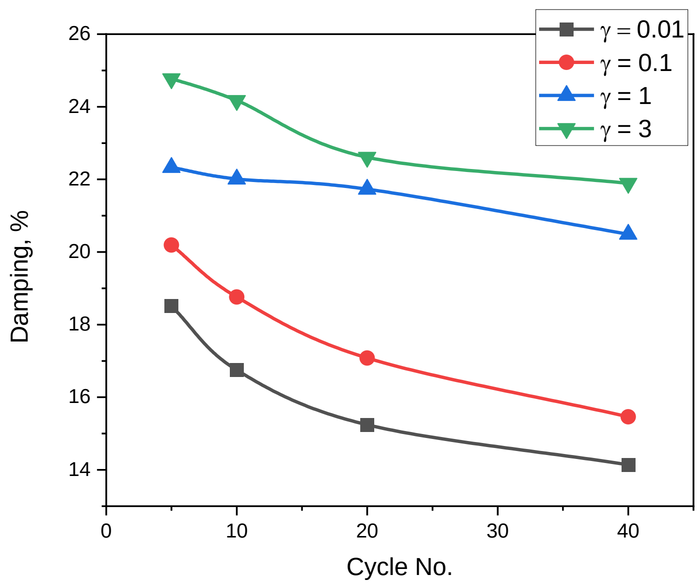


Figure 40 Effect of cycle no. on damping ratio of treated soil with 200kpa confining pressure

As indicated in figures 33 to 36 the number of cycles has noticeable effect on shear modulus and damping ratio values. The shear modulus increases and damping ratio decreases, with increasing number of cycles Since, strength degradation occur from cycle to cycle as cycling loading continues.

4.7.4 Effect of confining pressure on shear modulus and damping ratio with strain

The confining stress has a major influence on shear modulus and damping values. As the confining pressure increases, the soil particles become closer to each other resulting in compactness of soil. Compaction causes an increase in density that further results in the increment of stiffness which is described as shear modulus. Whereas, compactness reduces the deformation of soils, which leads to the reduction of energy loss or damping ratio.

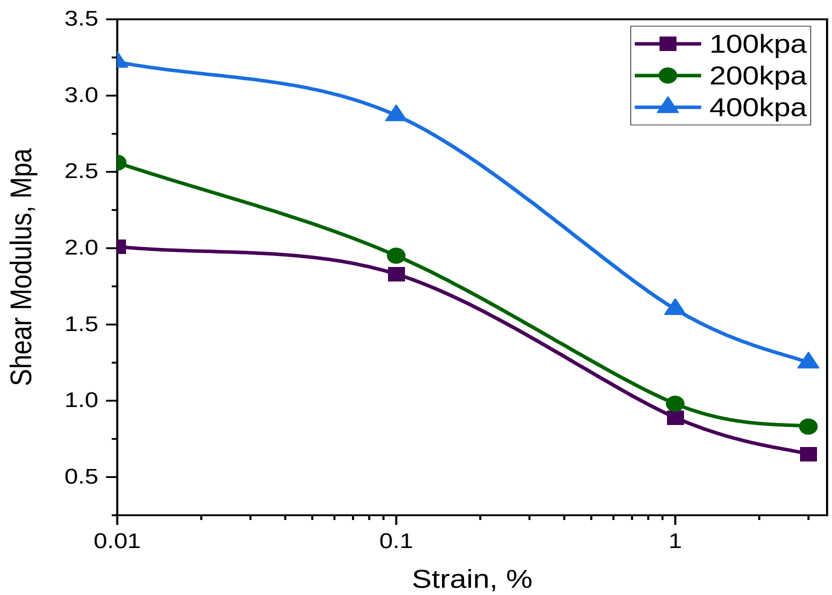


Figure 41 Effect of confining pressure on shear modulus for natural soil

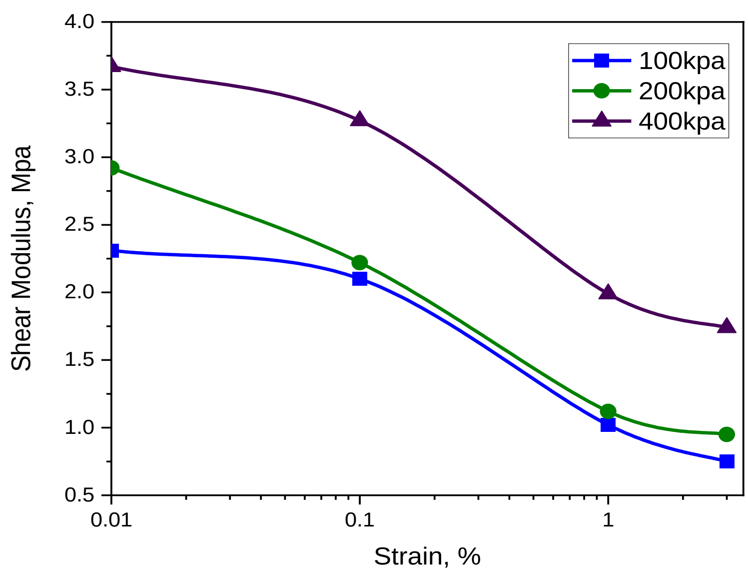


Figure 42 Effect of confining pressure on shear modulus for treated soil

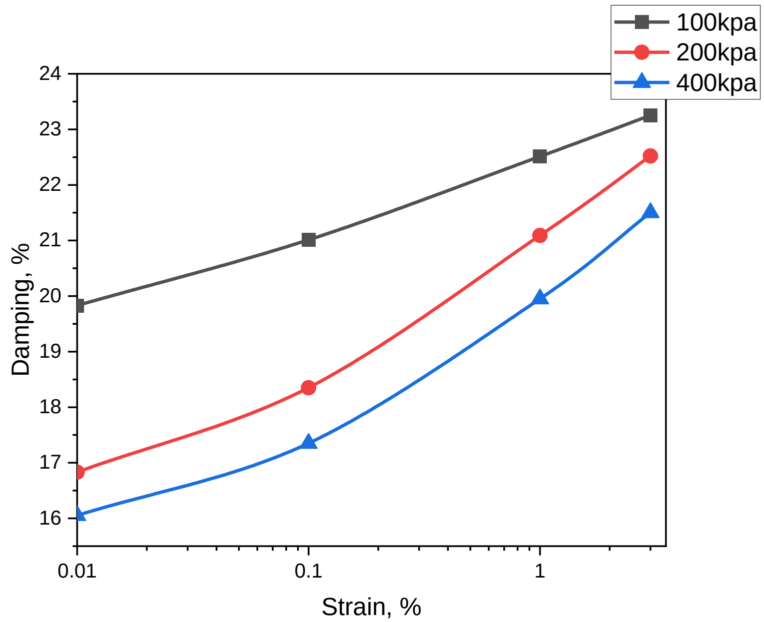


Figure 43 Effect of confining pressure on damping ratio for natural soil

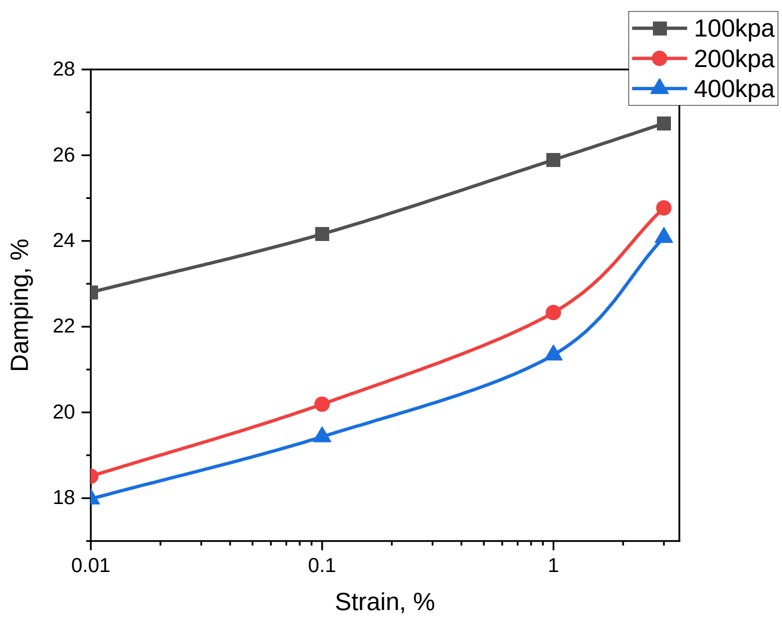


Figure 44 Effect of confining pressure on damping ratio for treated soil

Figure 37 to 40 shows effect of confining pressure on shear modulus and damping ratio with strain at the 5th cycle.

4.7.5 Estimation of maximum shear modulus and normalized shear modulus.

Cyclic simple shear test machine only operates in a strain range of 10^{-2} to 5%. Thus it cannot measure the shear modulus and damping values at a very low strain level. In this case the maximum shear modulus which should be computed from a very small strain range of amplitude in elastic range which is lower than 10^{-4} % will be calculated either from direct field test or from empirical equations. The Empirical equation stated in section 2.4.2 was used in this research for the calculation of maximum shear modulus.

$$G_{max} = 3220 \times (\frac{(2.976 - e)^2}{(1 + e)}) \times (OCR)^k \times (\sigma_o')^{k_o}$$

Typical calculation of G_{max} for untreated soil at 100kpa axial stress with PI value of 35.55, from table -7 using interpolation has k value of 0.29 and from one-dimensional consolidation test results given in appendix B has void ratio 1.07 and pre-consolidation pressure of 98.25kpa.

Over consolidation ratio = OCR = $^{P_c}/_{P_o}$ = $^{98.25}/_{100}$ = 0.98

Mean effective stress $= \sigma'_o = \frac{(\sigma'_v + 2k_o \times \sigma'_v)}{3} = \frac{(100+2*0.5\times100)}{3}$ = 66.67kpa

$$G_{max} = 3220 \times (\frac{(2.976 - 1.07)^2}{(1 + 1.07)}) \times (0.98)^{0.29} \times (66.67)^{0.5}$$

G_{max} = 45872.47 kpa

G_{max} = 45.87 Mpa

Likewise the maximum shear modulus is calculated for untreated and treated soil at 100kpa, 200kpa, and 400kpa axial stress and presented on the table below. Normalized shear modulus also calculated accordingly.

Dimensions	Untreated Soil			Treated soil		
	Axial stress (kpa)			Axial Stress (kPa)		
	100	200	400	100	200	400
Void ratio (e)	1.07	1.07	1.07	0.95	0.95	0.95
PI (%)	32.7	32.7	32.7	25.7	25.7	25.7
a	0.29	0.29	0.29	0.21	0.21	0.21
OCR	0.98	0.49	0.25	1.003	0.502	0.251
σ'_o	66.67	133.33	266.67	66.67	133.3	266.67
G_{max} (Mpa)	45.87	53.06	61.73	55.39	78.32	110.78

Table 19 Maximum Shear modulus values

Confining pressure	G_{max}	Shear Strain (%)	G/G_{max} Cycle No. 5	10	20	40
100kpa	45.87 Mpa	0.01	0.044	0.106	0.121	0.133
		0.1	0.040	0.082	0.090	0.116
		1	0.019	0.027	0.038	0.071
		3	0.014	0.021	0.032	0.044
200kpa	53.06Mpa	0.01	0.048	0.107	0.151	0.158
		0.1	0.037	0.086	0.137	0.145
		1	0.018	0.041	0.098	0.126
		3	0.016	0.026	0.058	0.080
400kpa	61.73Mpa	0.01	0.052	0.099	0.182	0.200
		0.1	0.046	0.087	0.150	0.164
		1	0.014	0.041	0.085	0.135
		3	0.011	0.027	0.065	0.083

Table 20 Normalized shear modules values for untreated soil

Confining pressure	G_{max}	Shear Strain (%)	G/G_{max} Cycle No. 5	10	20	40
100kpa	55.39 Mpa	0.01	0.042	0.101	0.115	0.126
		0.1	0.038	0.078	0.086	0.111
		1	0.018	0.026	0.036	0.068
		3	0.014	0.02	0.030	0.042
200kpa	78.32Mpa	0.01	0.037	0.083	0.117	0.122
		0.1	0.028	0.066	0.106	0.112
		1	0.014	0.031	0.068	0.076
		3	0.012	0.020	0.045	0.062
400kpa	110.78Mpa	0.01	0.033	0.063	0.116	0.127

		0.1	0.030	0.055	0.095	0.104
		1	0.009	0.026	0.054	0.086
		3	0.007	0.017	0.041	0.053

Table 21 Normalized shear modules values for treated soil

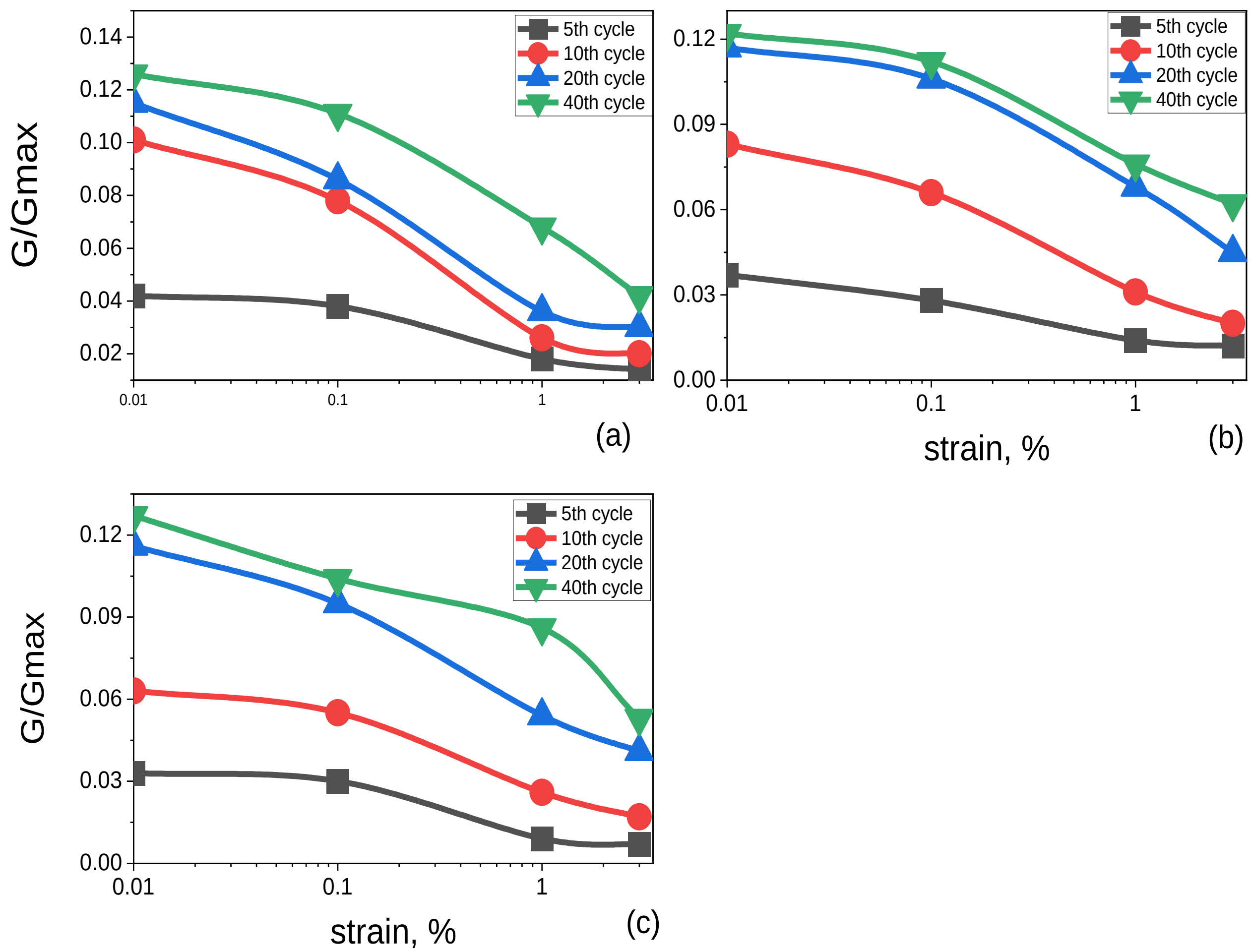


Figure 45 variation of normalized shear modulus (G/Gmax) with strain amplitude at (a) 100kpa, (b) 200kpa, and (c) 400kpa confining stress for the treated soil

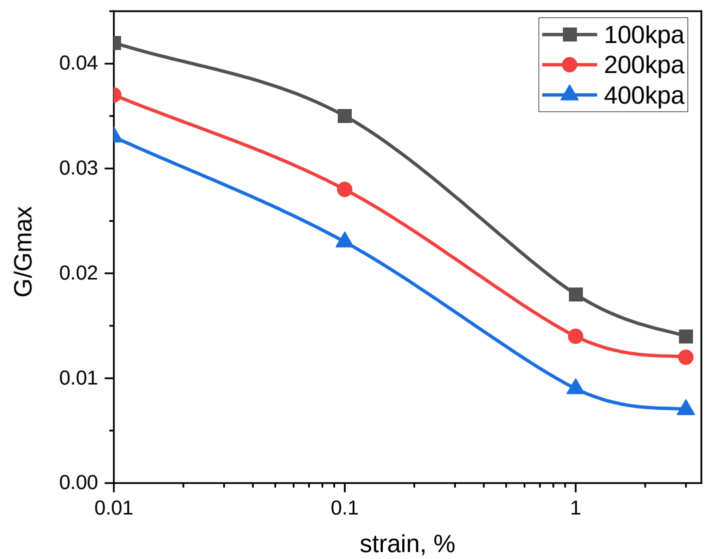


Figure 46 variation of normalized shear modulus (G/Gmax) with shear strain at the 5th cycle for treated soil with different confining pressures

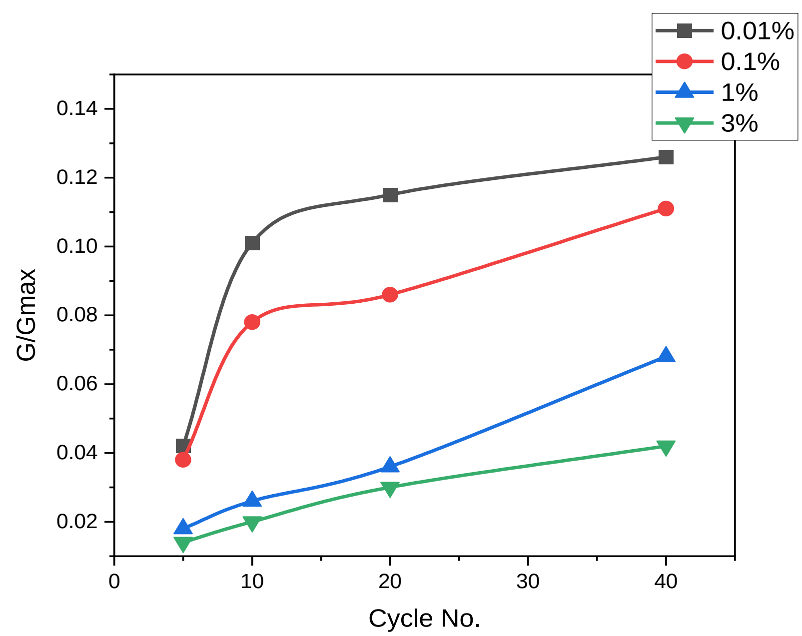


Figure 47 variation of normalized shear modulus (G/Gmax) with cycle no. at 100kpa confinning pressure for the treated soil

4.7.6 Effect of treatment on shear modulus and damping of the soil

With treatment the shear modulus and damping ratio of the soil increases as shown in the figure 44 to 47. The effect of treatment explained in terms of combined contribution of cohesion and internal friction to the stiffness and shear resistance of the soil. This is explained as, with the addition of SCBA cementation between the soil particles is increased. Thus increase in shear modulus at a given confining pressure resulting from SCBA treatment is due to a larger extent of increased value of stiffness due to cementation created as a result of treatment. Besides, the cementation also contributes for the increased in shear resistance of the soil and the requirement for a larger energy to break the particle bonding and the dissipation of more energy through the process. This phenomenon is observed in the increased value of damping ratio due to the treatment.

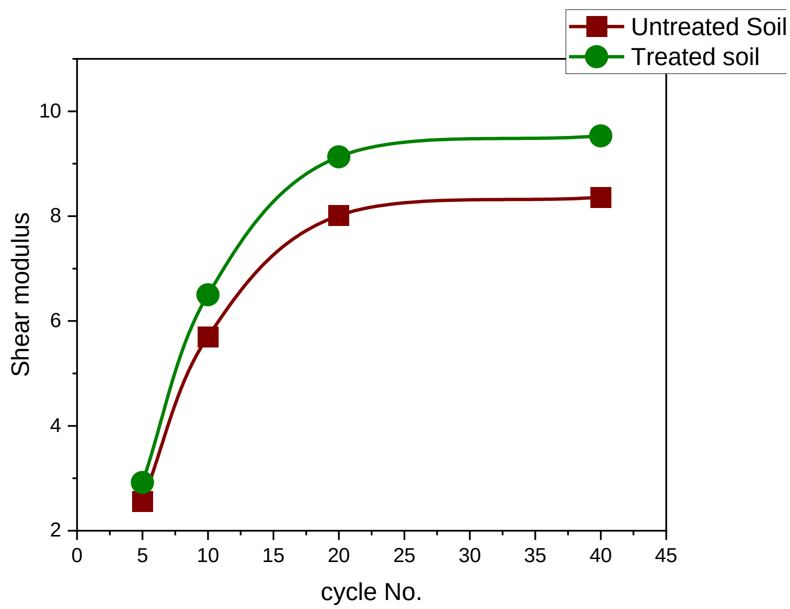


Figure 48 Variation of Shear Modulus with Cycle Number for treated and untreated soil

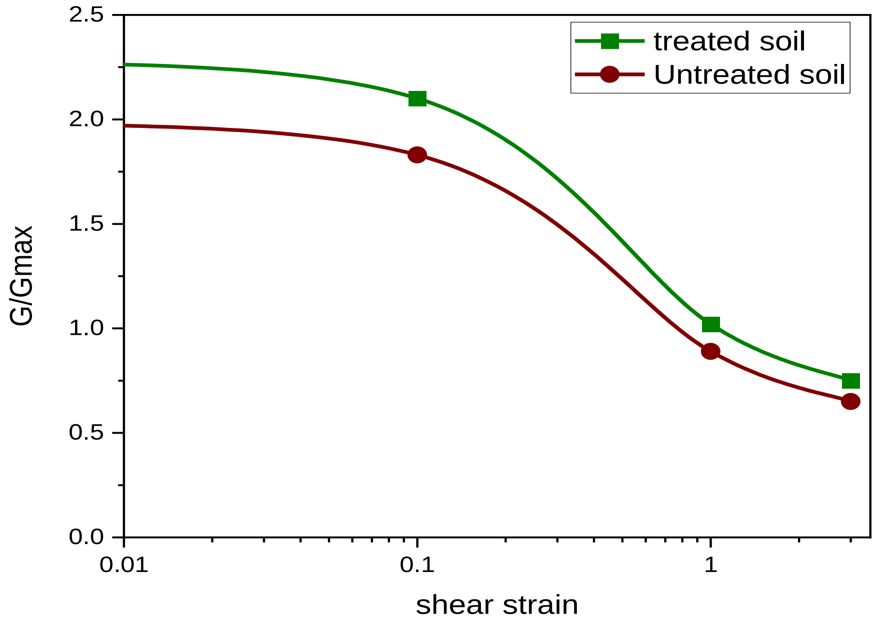


Figure 49 Effect of treatment on normalized shear modulus with shear strain

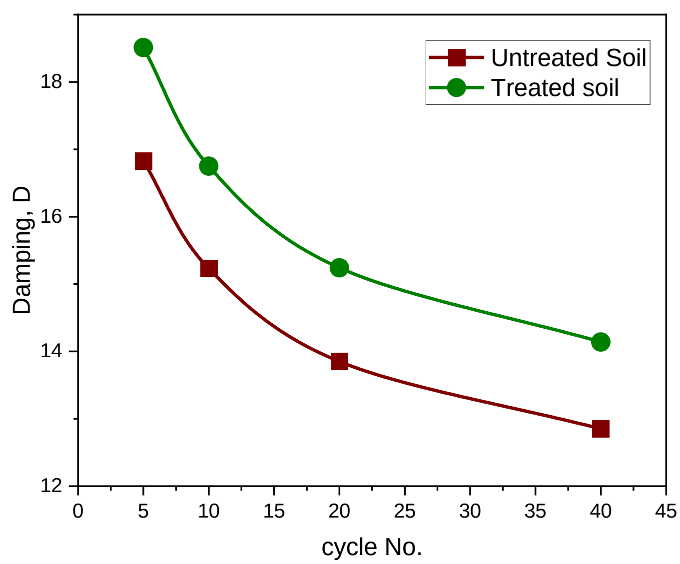


Figure 50 Variation of Damping with Cycle Number for treated and untreated soil

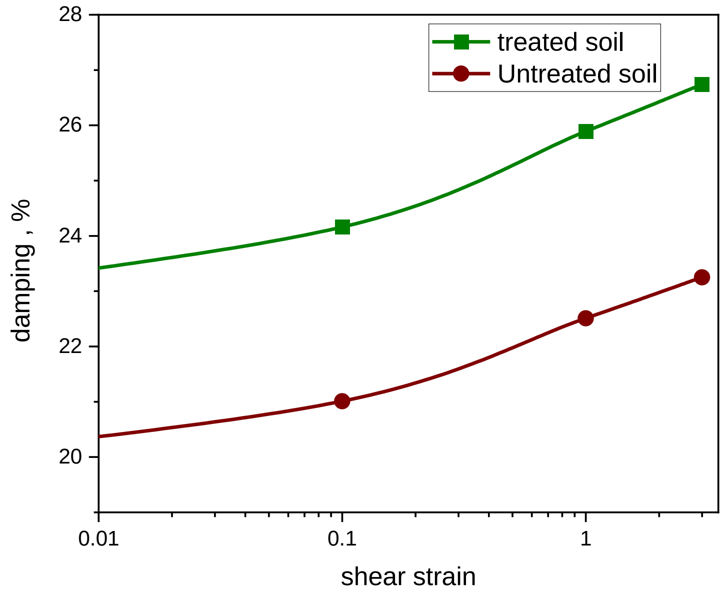


Figure 51 Effect of treatment on damping ratio with shear strain variation

4.7.7 Comparison of Shear Modulus Reduction and Damping Ratio with Previous Studies

The variation of normalized shear modulus with shear strain is described graphically by modulus reduction curve. Previously, some researchers developed different normalized shear modulus and damping ratio curves for different soil. The computed normalized shear modulus (G/Gmax) for an axial stress of 100 kPa were compared with the one developed for CH silty clay soil by Teshome (2019), Natnael (2020), and ferric chloride treated expansive soils by Wengelawit (2021).

It is illustrated in the figures below that the normalized shear modulus values have good agreement with curve developed by Teshome (2019) and Wengelawit (2021). However, the values at lower strain is lower than the curves developed by Natnael (2020), the variation may come from different reasons like over consolidation ratio and sampling technique.

It is also shown that the damping ratio of ferric chloride treated expansive soils is higher that SCBA treated expansive soil. This might be from the strength of ferric chloride to better treat the expansive soil than SCBA.

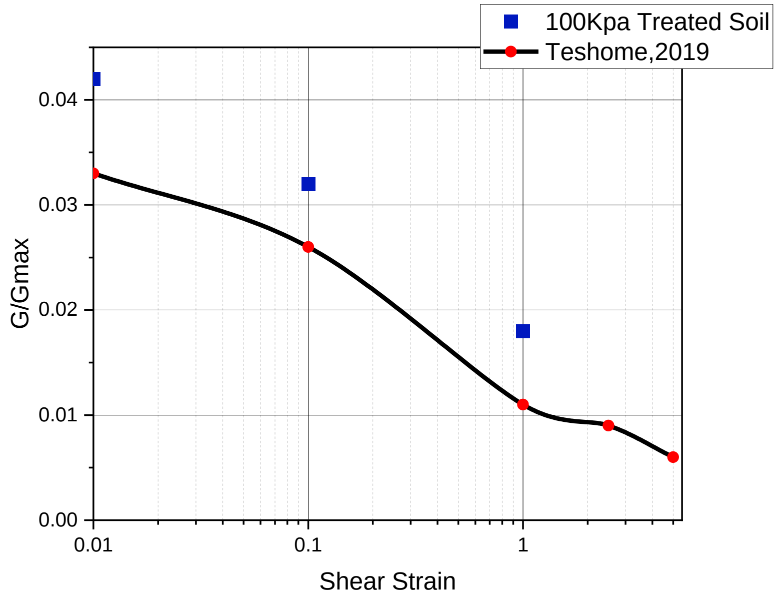


Figure 52 Comparison for modulus reduction values of treated soil with curves developed for Jimma town CH silty clay soil by Teshome (2019).

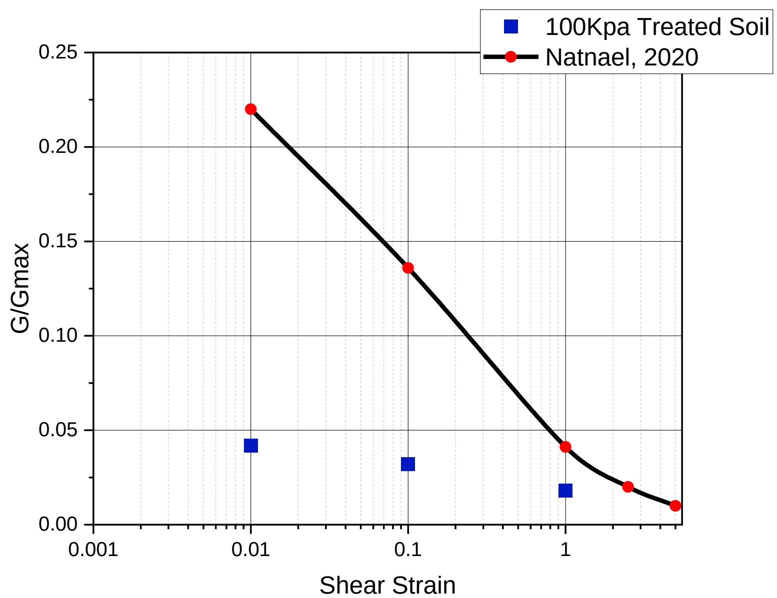


Figure 53 Comparison for modulus reduction values of treated soil with curves developed for Addis Ababa Yeka sub city CH silty clay soil by Natnael (2020).

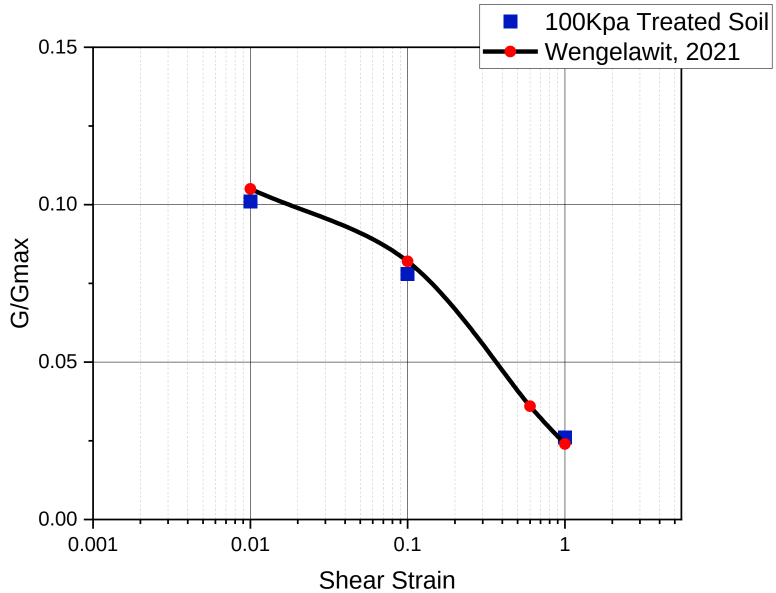


Figure 54 Comparison for modulus reduction values of treated soil with curves developed for Ferric Chloride Treated Expansive Soils by Wengelawit (2021).

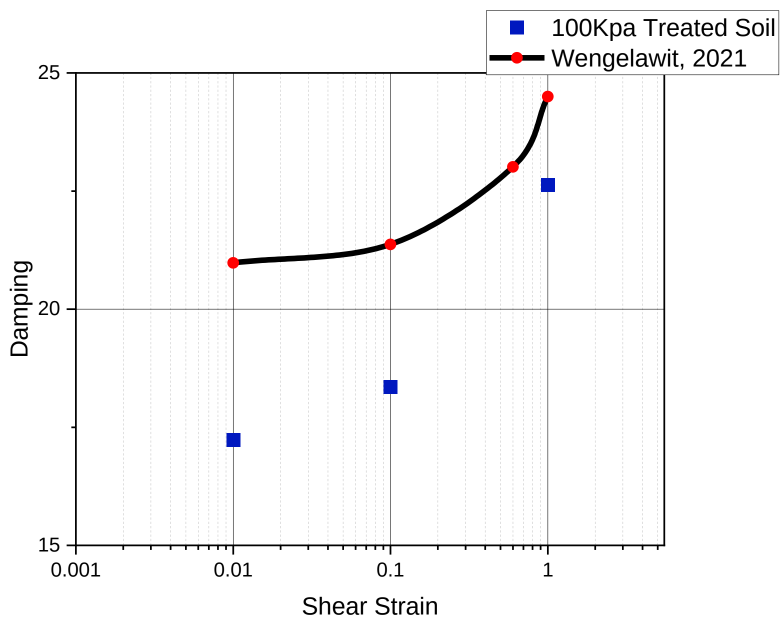


Figure 55 Comparison for damping ratio values of treated soil with curves developed for Ferric Chloride Treated Expansive Soils by Wengelawit (2021).

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

This study has investigated the dynamic property of expansive soil treated with 9% sugarcane bagasse ash. To determine the optimum amount of bagasse ash that is required for the treatment of expansive soil a serious of static geotechnical laboratory tests has been carried out. For the investigation of the dynamic properties a cyclic simple shear test has been used, and various factors affecting the dynamic property has been discussed. From the result of this investigations the following conclusions has been drawn:

- Stabilization of expansive soil with SCBA not only improves the static strength and swelling potential but also improves the dynamic properties of the expansive soil.
- The addition of SCBA managed to reduce the liquid limit for the soil sample obtained from pit 1 from 94.25% to 52.5% in addition of 12% SCBA. For pit 2 soil LL decreases from 82.25% to 53.9%.
- The plastic limit is reduced from 48.51% to 31.87% and 49.55% to 33.75% for pit 1 and pit 2 soil respectively.
- Likewise the PI also minimized from 45.74% to 20.6% and 32.7% to 20.15% for pit 1 and pit 2 soils respectively with addition of 12% SCBA amount by dry mass on the natural soil.
- Free swell index is reduced from 140% to 90% and 120% to 85% in the addition of 12% SCBA
- For moisture density relation the addition of SCBA keeps reducing the MDD and increases the OMC which helps to achieve compaction to be easily attend with wet soil.
- For pit 1 soil OMC increases from 25.7% at the natural state to 34.05% in addition of 12% SCBA. And from 30% to 35.6% for soil from pit 2
- MDD reduced from 1.26g/cc to 1.96g/cc and from 1.24g/cc to 1.17g/cc for samples from pit 1 and 2

- The swelling pressure for the natural soil from pit 1 which is 156.29kpa and from pit 2 which is 162.83kpa were reduced to 81.3kpa and 83.37kpa respectively in addition of 12% SCBA.
- For UCS test the values increased up to 90% SCBA and further increase reduce the UCS for both test pits.
- For test pit 1 the UCS increased fro 215.3kpa to 264.28kpa by 22.7% in addition of 9% SCBA. Whereas for pit 2 the value improves from 142.11kpa to 239.77kpa which is 68.7%.
- The shear modulus decreases from 2.01Mpa to 0.65Mpa as the shear strain increases for the untreated soil at the 5th cycle. Whereas for treated soil the shear modulus decreased from 2.31Mpa to 0.75Mpa as the strain increased at the 5th cycle.
- On the other hand the damping ratio increased from 19.83 to 23.25% for untreated soil and 22.8 to 26.74% for treated soil as the shear strain increased at the 5th cycle.
- The number of cycle have also shows a noticeable effect on shear modulus and damping ratio values.
- As the number of cycle increases the shear modulus also increases and damping ratio however decreases.
- With confining pressure the densification of the soil increased and the deformability decreases thus the shear modulus increased and damping ratio decreases.
- With treatment the shear modulus and damping ratio of the soil increases.

5.2 Recommendations

- The dynamic soil properties were studied under a limited factors, comprising a wide range of factors will help to have better understanding of the behavior of the soil.
- Studying the Dynamic properties of stabilized expansive soil with various stabilization techniques and materials will be a good area of study for the future.

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Appendix

Appendix: A

Cyclic Shear Test Result

A-1 Typical Tabulation of Shear Stress and Shear Strain Values of Selected Strain for Treated Soil

Shear stress and shear strain values of treated soil with 0.01 % strain at 200 kPa, for the 1st to 5th cycles.

Cycle Num	Time (sec)	Lateral Lvdt (mm)	Lateral Force (KPa)	shear stress(γ) = lateral Lvdt/height of specimen	shear strain (τ) = (Lateral Force / area of specimen) * 10^3 MPa
1	0	-0.31808	0.03603	0.009358	-0.004544
	0.0095	-0.31831	0.03602	0.009356	-0.004547
	0.019	-0.31837	0.03573	0.009281	-0.004548
	0.0285	-0.31828	0.03584	0.009309	-0.004547
	0.038	-0.31848	0.03535	0.009182	-0.004550
	0.0475	-0.31888	0.03409	0.008855	-0.004555
	0.057	-0.3196	0.03251	0.008444	-0.004566
	0.0665	-0.32006	0.03121	0.008106	-0.004572
	0.076	-0.32029	0.03056	0.007938	-0.004576
	0.0855	-0.3206	0.03046	0.007912	-0.004580
	0.095	-0.32054	0.0306	0.007948	-0.004579
	0.1045	-0.3206	0.03074	0.007984	-0.004580
	0.114	-0.32057	0.03093	0.008034	-0.004580
	0.1235	-0.32063	0.03117	0.008096	-0.004580
	0.133	-0.32049	0.03152	0.008187	-0.004578
	0.1425	-0.32049	0.03189	0.008283	-0.004578
	0.152	-0.32043	0.03232	0.008395	-0.004578
	0.1615	-0.32034	0.03262	0.008473	-0.004576
	0.171	-0.3204	0.033	0.008571	-0.004577
	0.1805	-0.32054	0.03315	0.008610	-0.004579
	0.19	-0.32069	0.03372	0.008758	-0.004581
	0.1995	-0.32066	0.034	0.008831	-0.004581
	0.209	-0.3206	0.03451	0.008964	-0.004580
	0.2185	-0.32066	0.03477	0.009031	-0.004581
	0.228	-0.32063	0.03553	0.009229	-0.004580
	0.2375	-0.32032	0.03562	0.009252	-0.004576
	0.247	-0.32066	0.03599	0.009348	-0.004581

	0.2565	-0.31991	0.03617	0.009395	-0.004570
	0.266	-0.32034	0.03645	0.009468	-0.004576
	0.2755	-0.32046	0.03646	0.009470	-0.004578
	0.285	-0.32017	0.03672	0.009538	-0.004574
	0.2945	-0.32043	0.03686	0.009574	-0.004578
	0.304	-0.3204	0.03707	0.009629	-0.004577
	0.3135	-0.32032	0.03734	0.009699	-0.004576
	0.323	-0.32011	0.03758	0.009761	-0.004573
	0.3325	-0.31986	0.03837	0.009966	-0.004569
	0.342	-0.3192	0.03971	0.010314	-0.004560
	0.3515	-0.31888	0.04073	0.010579	-0.004555
	0.361	-0.31871	0.04085	0.010610	-0.004553
	0.3705	-0.31877	0.04096	0.010639	-0.004554
	0.38	-0.31863	0.04083	0.010605	-0.004552
	0.3895	-0.31871	0.04088	0.010618	-0.004553
	0.399	-0.3184	0.04073	0.010579	-0.004549
	0.4085	-0.31874	0.04085	0.010610	-0.004553
	0.418	-0.31848	0.04069	0.010569	-0.004550
	0.4275	-0.31854	0.0411	0.010675	-0.004551
	0.437	-0.31834	0.0422	0.010961	-0.004548
	0.4465	-0.31765	0.04352	0.011304	-0.004538
	0.456	-0.31708	0.04454	0.011569	-0.004530
	0.4655	-0.31708	0.04528	0.011761	-0.004530
2	0	-0.317	0.04511	0.011717	-0.004529
	0.0095	-0.31748	0.0459	0.011922	-0.004535
	0.019	-0.31731	0.04753	0.012345	-0.004533
	0.0285	-0.31714	0.04806	0.012483	-0.004531
	0.038	-0.31697	0.04674	0.012140	-0.004528
	0.0475	-0.31705	0.04673	0.012138	-0.004529
	0.057	-0.31682	0.04718	0.012255	-0.004526
	0.0665	-0.31688	0.04786	0.012431	-0.004527
	0.076	-0.31688	0.04842	0.012577	-0.004527
	0.0855	-0.31657	0.04837	0.012564	-0.004522
	0.095	-0.31674	0.04775	0.012403	-0.004525
	0.1045	-0.31668	0.04734	0.012296	-0.004524
	0.114	-0.31688	0.04636	0.012042	-0.004527
	0.1235	-0.31682	0.04775	0.012403	-0.004526
	0.133	-0.31671	0.04793	0.012449	-0.004524
	0.1425	-0.31694	0.04808	0.012488	-0.004528
	0.152	-0.317	0.0486	0.012623	-0.004529

	0.1615	-0.31654	0.04873	0.012657	-0.004522
	0.171	-0.31665	0.05011	0.013016	-0.004524
	0.1805	-0.31648	0.04985	0.012948	-0.004521
	0.19	-0.31637	0.04961	0.012886	-0.004520
	0.1995	-0.31674	0.04954	0.012868	-0.004525
	0.209	-0.3166	0.04937	0.012823	-0.004523
	0.2185	-0.31645	0.04855	0.012610	-0.004521
	0.228	-0.31662	0.0451	0.011714	-0.004523
	0.2375	-0.31617	0.04165	0.010818	-0.004517
	0.247	-0.31534	0.04124	0.010712	-0.004505
	0.2565	-0.31471	0.04159	0.010803	-0.004496
	0.266	-0.31333	0.04137	0.010745	-0.004476
	0.2755	-0.31351	0.04143	0.010761	-0.004479
	0.285	-0.31336	0.04146	0.010769	-0.004477
	0.2945	-0.31382	0.04113	0.010683	-0.004483
	0.304	-0.31499	0.04068	0.010566	-0.004500
	0.3135	-0.3166	0.04016	0.010431	-0.004523
	0.323	-0.31768	0.03981	0.010340	-0.004538
	0.3325	-0.31826	0.0395	0.010260	-0.004547
	0.342	-0.31868	0.03973	0.010319	-0.004553
	0.3515	-0.31894	0.03993	0.010371	-0.004556
	0.361	-0.31871	0.03944	0.010244	-0.004553
	0.3705	-0.31934	0.03976	0.010327	-0.004562
	0.38	-0.32032	0.03894	0.010114	-0.004576
	0.3895	-0.3214	0.0387	0.010052	-0.004591
	0.399	-0.32309	0.03824	0.009932	-0.004616
	0.4085	-0.32444	0.03857	0.010018	-0.004635
	0.418	-0.32495	0.03738	0.009709	-0.004642
	0.4275	-0.32515	0.03926	0.010197	-0.004645
	0.437	-0.32444	0.04036	0.010483	-0.004635
	0.4465	-0.32389	0.04093	0.010631	-0.004627
	0.456	-0.32375	0.04079	0.010595	-0.004625
	0.4655	-0.32361	0.04031	0.010470	-0.004623
3	0	-0.32461	0.03785	0.009831	-0.004637
	0.0095	-0.32495	0.03848	0.009995	-0.004642
	0.019	-0.32415	0.03824	0.009932	-0.004631
	0.0285	-0.32343	0.03918	0.010177	-0.004620
	0.038	-0.322	0.03967	0.010304	-0.004600
	0.0475	-0.32112	0.03974	0.010322	-0.004587

	0.057	-0.32006	0.04003	0.010397	-0.004572
	0.0665	-0.32014	0.03985	0.010351	-0.004573
	0.076	-0.32014	0.03969	0.010309	-0.004573
	0.0855	-0.32011	0.03967	0.010304	-0.004573
	0.095	-0.3204	0.03931	0.010210	-0.004577
	0.1045	-0.32086	0.03915	0.010169	-0.004584
	0.114	-0.32052	0.03921	0.010184	-0.004579
	0.1235	-0.31906	0.03918	0.010177	-0.004558
	0.133	-0.318	0.03995	0.010377	-0.004543
	0.1425	-0.31631	0.03972	0.010317	-0.004519
	0.152	-0.31488	0.04042	0.010499	-0.004498
	0.1615	-0.31405	0.03992	0.010369	-0.004486
	0.171	-0.31368	0.03946	0.010249	-0.004481
	0.1805	-0.31439	0.03913	0.010164	-0.004491
	0.19	-0.31479	0.03836	0.009964	-0.004497
	0.1995	-0.31577	0.03808	0.009891	-0.004511
	0.209	-0.31588	0.03782	0.009823	-0.004513
	0.2185	-0.31574	0.03779	0.009816	-0.004511
	0.228	-0.31522	0.03808	0.009891	-0.004503
	0.2375	-0.31414	0.03796	0.009860	-0.004488
	0.247	-0.31385	0.03838	0.009969	-0.004484
	0.2565	-0.31325	0.03796	0.009860	-0.004475
	0.266	-0.31333	0.03748	0.009735	-0.004476
	0.2755	-0.31491	0.03675	0.009545	-0.004499
	0.285	-0.31654	0.03612	0.009382	-0.004522
	0.2945	-0.31771	0.03601	0.009353	-0.004539
	0.304	-0.31831	0.03591	0.009327	-0.004547
	0.3135	-0.31831	0.03629	0.009426	-0.004547
	0.323	-0.31826	0.03669	0.009530	-0.004547
	0.3325	-0.31823	0.0374	0.009714	-0.004546
	0.342	-0.31831	0.03818	0.009917	-0.004547
	0.3515	-0.319	0.03927	0.010200	-0.004557
	0.361	-0.3212	0.04069	0.010569	-0.004589
	0.3705	-0.32358	0.04231	0.010990	-0.004623
	0.38	-0.32598	0.04439	0.011530	-0.004657
	0.3895	-0.32752	0.04657	0.012096	-0.004679
	0.399	-0.32815	0.0491	0.012753	-0.004688
	0.4085	-0.32827	0.05153	0.013384	-0.004690
	0.418	-0.32807	0.05408	0.014047	-0.004687

	0.4275	-0.32861	0.05695	0.014792	-0.004694
	0.437	-0.32867	0.0563	0.014623	-0.004695
	0.4465	-0.32827	0.05346	0.013886	-0.004690
	0.456	-0.32109	0.03312	0.008603	-0.004587
	0.4655	-0.31934	0.04693	0.012190	-0.004562
4	0	-0.34091	0.06629	0.017218	-0.004870
	0.0095	-0.33659	0.05574	0.014478	-0.004808
	0.019	-0.32661	0.05597	0.014538	-0.004666
	0.0285	-0.32235	0.0571	0.014831	-0.004605
	0.038	-0.31883	0.05929	0.015400	-0.004555
	0.0475	-0.31951	0.05292	0.013745	-0.004564
	0.057	-0.32232	0.06275	0.016299	-0.004605
	0.0665	-0.32369	0.05855	0.015208	-0.004624
	0.076	-0.32518	0.06093	0.015826	-0.004645
	0.0855	-0.32481	0.05704	0.014816	-0.004640
	0.095	-0.32383	0.05842	0.015174	-0.004626
	0.1045	-0.32303	0.0583	0.015143	-0.004615
	0.114	-0.32129	0.05956	0.015470	-0.004590
	0.1235	-0.31903	0.05739	0.014906	-0.004558
	0.133	-0.31917	0.05817	0.015109	-0.004560
	0.1425	-0.3186	0.05836	0.015158	-0.004551
	0.152	-0.32014	0.05966	0.015496	-0.004573
	0.1615	-0.32014	0.0579	0.015039	-0.004573
	0.171	-0.32129	0.05783	0.015021	-0.004590
	0.1805	-0.32106	0.05855	0.015208	-0.004587
	0.19	-0.32072	0.05878	0.015268	-0.004582
	0.1995	-0.31943	0.05902	0.015330	-0.004563
	0.209	-0.31743	0.05824	0.015127	-0.004535
	0.2185	-0.31691	0.05852	0.015200	-0.004527
	0.228	-0.31614	0.05756	0.014951	-0.004516
	0.2375	-0.31702	0.0586	0.015221	-0.004529
	0.247	-0.3168	0.05854	0.015205	-0.004526
	0.2565	-0.31823	0.05909	0.015348	-0.004546
	0.266	-0.318	0.05856	0.015210	-0.004543
	0.2755	-0.31883	0.05838	0.015164	-0.004555
	0.285	-0.31854	0.05796	0.015055	-0.004551
	0.2945	-0.31837	0.05814	0.015101	-0.004548
	0.304	-0.31823	0.05874	0.015257	-0.004546
	0.3135	-0.31771	0.05866	0.015236	-0.004539

	0.323	-0.31797	0.05906	0.015340	-0.004542
	0.3325	-0.31783	0.05822	0.015122	-0.004540
	0.342	-0.31863	0.05801	0.015068	-0.004552
	0.3515	-0.3194	0.05824	0.015127	-0.004563
	0.361	-0.32086	0.05864	0.015231	-0.004584
	0.3705	-0.32089	0.05873	0.015255	-0.004584
	0.38	-0.32172	0.05848	0.015190	-0.004596
	0.3895	-0.32143	0.05899	0.015322	-0.004592
	0.399	-0.32112	0.05717	0.014849	-0.004587
	0.4085	-0.32103	0.05751	0.014938	-0.004586
	0.418	-0.32052	0.05829	0.015140	-0.004579
	0.4275	-0.32109	0.05823	0.015125	-0.004587
	0.437	-0.32077	0.05837	0.015161	-0.004582
	0.4465	-0.32177	0.05963	0.015488	-0.004597
	0.456	-0.32169	0.05949	0.015452	-0.004596
	0.4655	-0.32258	0.05803	0.015073	-0.004608
5	0	-0.322	0.05845	0.015182	-0.004600
	0.0095	-0.3214	0.05762	0.014966	-0.004591
	0.019	-0.3208	0.05825	0.015130	-0.004583
	0.0285	-0.32132	0.05876	0.015262	-0.004590
	0.038	-0.32054	0.05909	0.015348	-0.004579
	0.0475	-0.32152	0.05855	0.015208	-0.004593
	0.057	-0.321	0.05825	0.015130	-0.004586
	0.0665	-0.32163	0.05833	0.015151	-0.004595
	0.076	-0.32143	0.05842	0.015174	-0.004592
	0.0855	-0.32192	0.05829	0.015140	-0.004599
	0.095	-0.32177	0.05825	0.015130	-0.004597
	0.1045	-0.32149	0.05833	0.015151	-0.004593
	0.114	-0.32112	0.05879	0.015270	-0.004587
	0.1235	-0.32077	0.05848	0.015190	-0.004582
	0.133	-0.32089	0.05861	0.015223	-0.004584
	0.1425	-0.32014	0.0585	0.015195	-0.004573
	0.152	-0.32089	0.05821	0.015119	-0.004584
	0.1615	-0.32011	0.05859	0.015218	-0.004573
	0.171	-0.32106	0.05888	0.015294	-0.004587
	0.1805	-0.32126	0.05799	0.015062	-0.004589
	0.19	-0.3216	0.058	0.015065	-0.004594
	0.1995	-0.3214	0.05836	0.015158	-0.004591
	0.209	-0.32143	0.058	0.015065	-0.004592

	0.2185	-0.32132	0.0588	0.015273	-0.004590
	0.228	-0.3206	0.05899	0.015322	-0.004580
	0.2375	-0.32097	0.05823	0.015125	-0.004585
	0.247	-0.32086	0.05803	0.015073	-0.004584
	0.2565	-0.32089	0.05853	0.015203	-0.004584
	0.266	-0.32063	0.05831	0.015145	-0.004580
	0.2755	-0.32117	0.05853	0.015203	-0.004588
	0.285	-0.32123	0.05857	0.015213	-0.004589
	0.2945	-0.32163	0.05846	0.015184	-0.004595
	0.304	-0.32149	0.05859	0.015218	-0.004593
	0.3135	-0.3216	0.05861	0.015223	-0.004594
	0.323	-0.32132	0.05824	0.015127	-0.004590
	0.3325	-0.32077	0.05819	0.015114	-0.004582
	0.342	-0.32057	0.05845	0.015182	-0.004580
	0.3515	-0.32026	0.05874	0.015257	-0.004575
	0.361	-0.32054	0.05867	0.015239	-0.004579
	0.3705	-0.3204	0.05848	0.015190	-0.004577
	0.38	-0.32109	0.05835	0.015156	-0.004587
	0.3895	-0.32049	0.0582	0.015117	-0.004578
	0.399	-0.321	0.05832	0.015148	-0.004586
	0.4085	-0.3208	0.05836	0.015158	-0.004583
	0.418	-0.32054	0.05861	0.015223	-0.004579
	0.4275	-0.3208	0.05855	0.015208	-0.004583
	0.437	-0.31986	0.05854	0.015205	-0.004569
	0.4465	-0.31923	0.05827	0.015135	-0.004560
	0.456	-0.31917	0.05847	0.015187	-0.004560
	0.4655	-0.31917	0.0584	0.015169	-0.004560

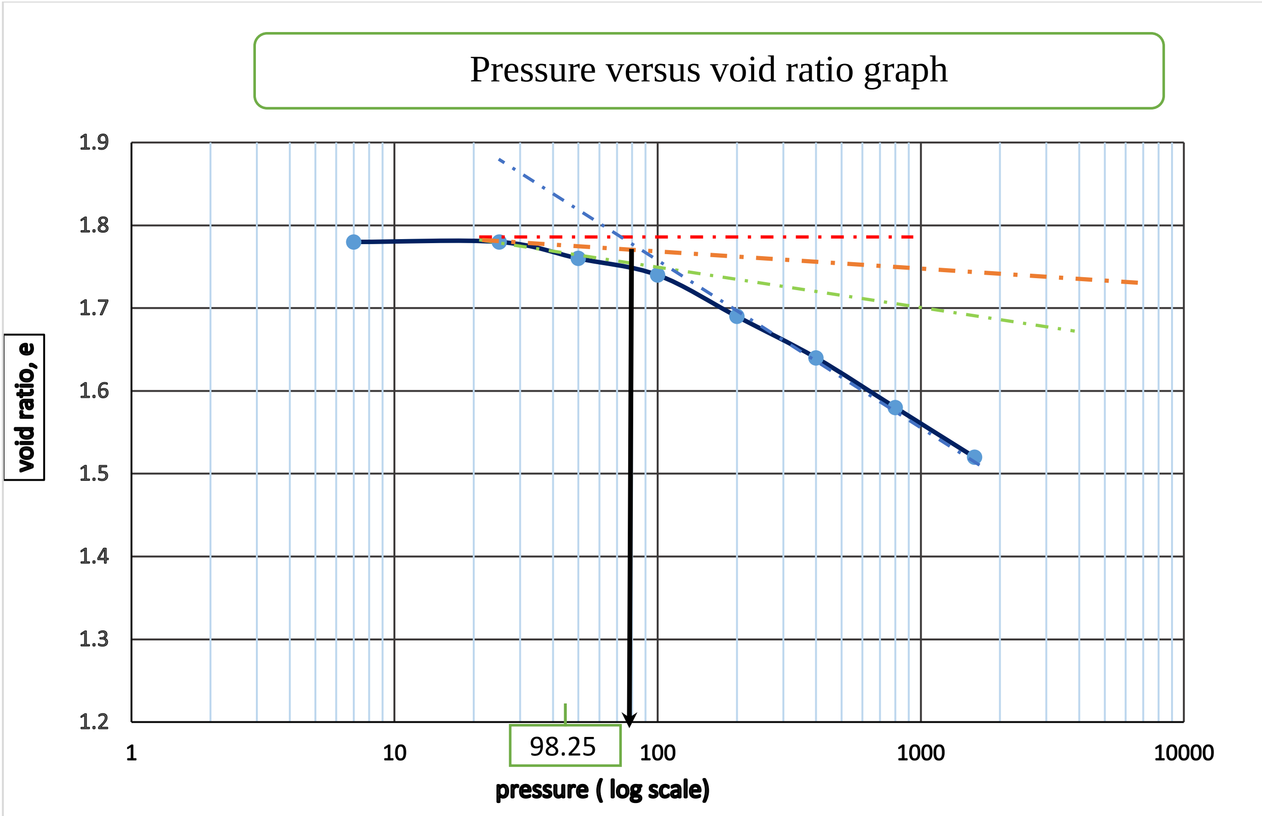
Appendix B

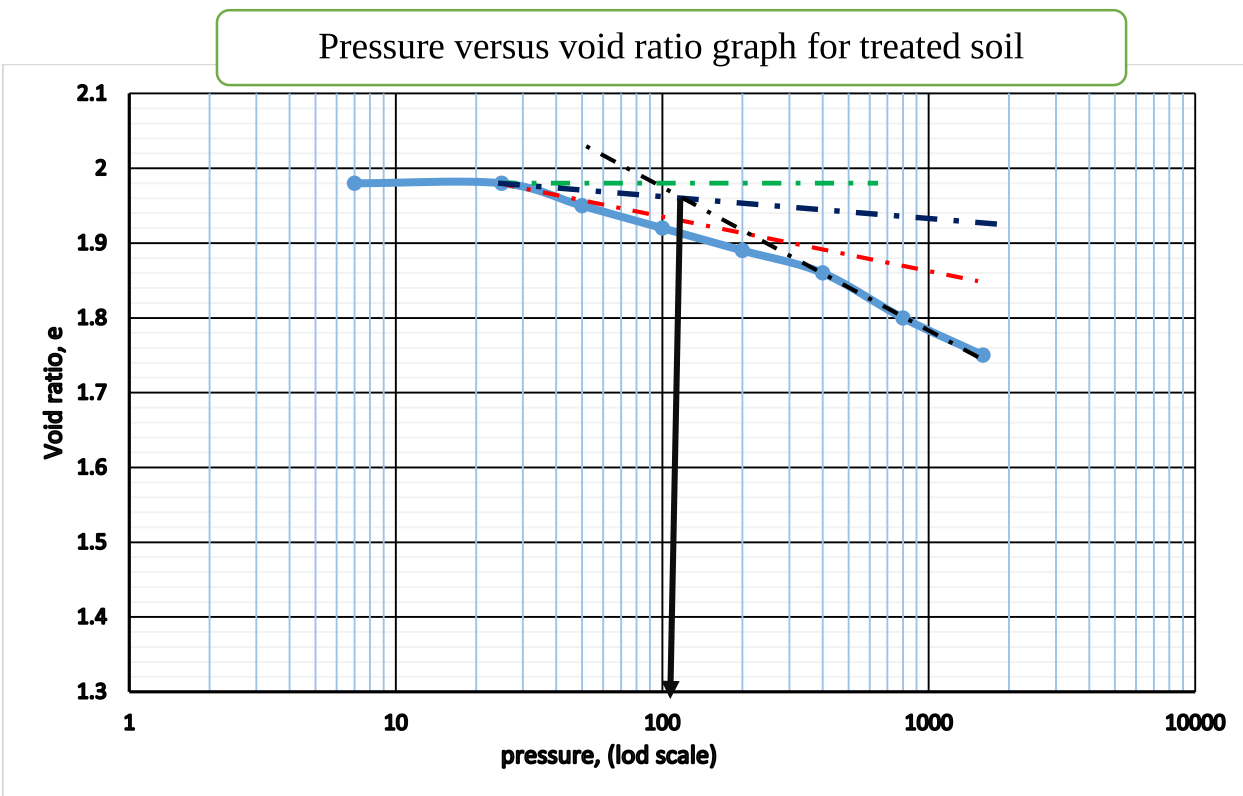
Determination of pre-consolidation Pressure

B-1 Determination of Pre-consolidation pressure for the untreated or natural soil sample from test pit 2

Untreated soil sample	
In the beginning of the test	
Sample type	Disturbed
Ring area, A (cm^2)	19.625
Height of sample, H_s(mm)	20
Seating load, kpa	7
Initial void ratio, e_o	1.78
Initial moisture content, wt. (%)	28.67
Specific gravity, G_s	2.55
Wet density, (g/cm^3)	
In the end of the test	
Final moisture content, (%)	33.5
Dry density (g/cm^3)	43.50
Height of solids, H_s(mm)	8.21
Final void ratio, e_f	1.07

Calculation table					
Applied pressure (kpa)	Final dial reading (mm)	Change in specimen height	Final specimen height	Void height, h_v (mm)	Void ratio, e
Loading					
7	6.514	0	20	12.8	1.78
25	6.519	0	20	12.8	1.78
50	6.624	0.11	19.89	12.69	1.76
100	6.814	0.3	19.7	12.5	1.74
200	7.114	0.6	19.4	12.2	1.69
400	7.503	0.99	19.01	11.81	1.64
800	7.921	1.41	18.59	11.39	1.58
1600	8.344	1.83	18.17	10.97	1.52


Pressure versus void ratio graph
void ratio, e
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1
10
100
1000
10000
98.25
pressure (log scale)


Pressure versus void ratio graph for treated soil
Void ratio, e
2.1
2
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1
10
100
1000
10000
pressure, (lod scale)

Appendix C

Determination of Free Swell and Swelling Pressure

C-1 Determination of free swell for the untreated or natural soil sample from test pit 1

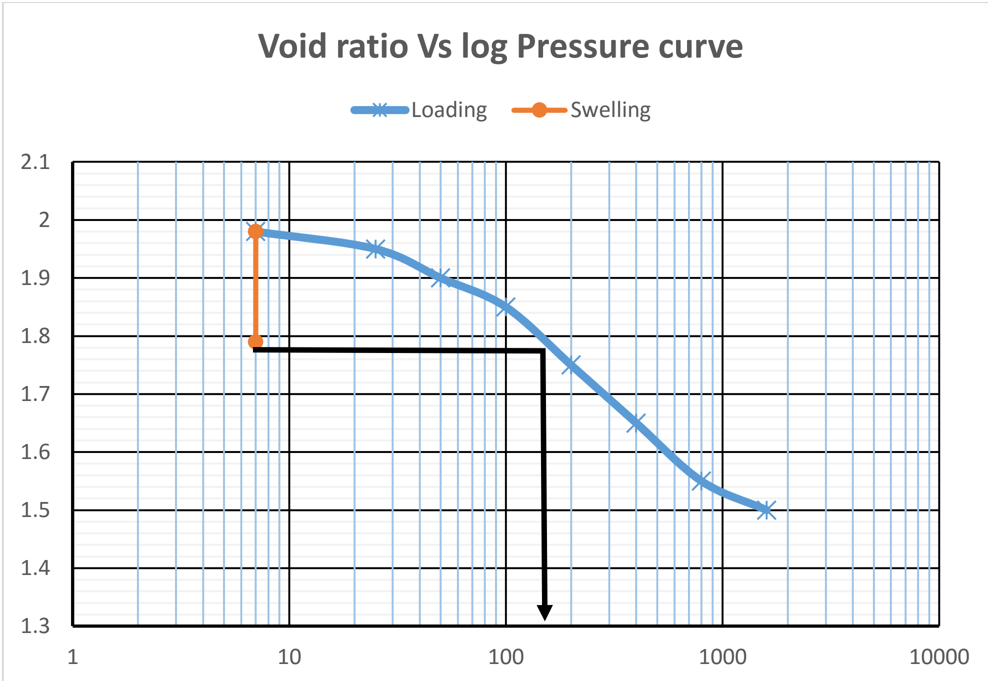
Natural soil			Pit 1 soil + 6% SCBA		
Trial	1	2	Trial	1	2
Initial volume, ml	10	10	Initial volume, ml	10	10
Final volume, ml	23.5	24.5	Final volume, ml	22	24
Free swell, %	135	145	Free swell, %	120	140
Avg. Free swell		140	Avg. Free swell		130
Pit 1 soil + 9% SCBA			**Pit 1 soil + 12% SCBA**		
Trial	1	2	Trial	1	2
Initial volume, ml	10	10	Initial volume, ml	10	10
Final volume, ml	19	21	Final volume, ml	18	20
Free swell, %	90	110	Free swell, %	90	100
Avg. Free swell		100	Avg. Free swell		90

C-2 Determination of free swell for the untreated or natural soil sample from test pit 2

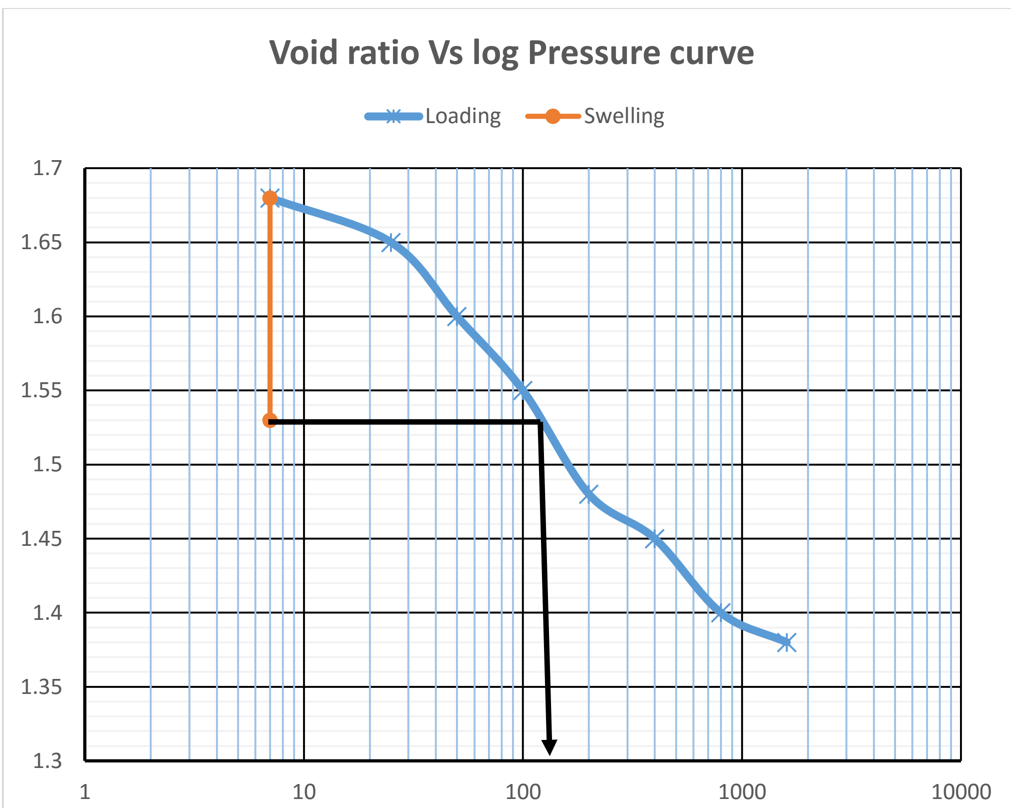
Natural soil			Pit 2 soil + 6% SCBA		
Trial	1	2	Trial	1	2
Initial volume, ml	10	10	Initial volume, ml	10	10
Final volume, ml	23.5	20.5	Final volume, ml	20	22
Free swell, %	135	105	Free swell, %	100	120
Avg. Free swell		120	Avg. Free swell		110
Pit 2 soil + 9% SCBA			**Pit 2soil + 12% SCBA**		
Trial	1	2	Trial	1	2
Initial volume, ml	10	10	Initial volume, ml	10	10
Final volume, ml	19	20	Final volume, ml	19	17
Free swell, %	90	100	Free swell, %	90	70
Avg. Free swell		90	Avg. Free swell		80

C-3 Determination of swelling pressure for the untreated or natural soil sample and treated soil sample with addition of SCBA for test pit 2

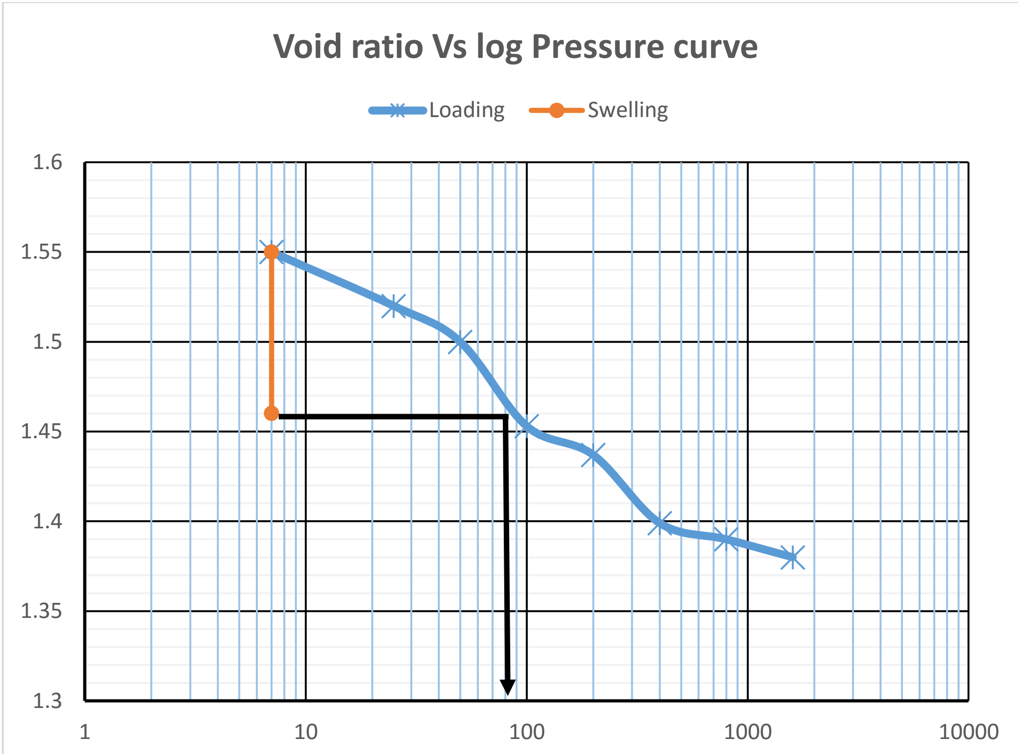
1. 0% SCBA



2. 6% SCBA



3. 12% SCBA


Void ratio Vs log Pressure curve
Loading
Swelling
1.6
1.55
1.5
1.45
1.4
1.35
1.3
1
10
100
1000
10000