Addis Ababa University
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Study on Dynamic Properties of Treated Expansive Soils Using Ferric Chloride

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Declaration

I hereby declare that research work titled "***Study on Dynamic Properties of Treated Expansive Soils Using Ferric Chloride***" is my original work. This work has not been presented elsewhere and all sources of materials used for the thesis have been properly acknowledged.

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Abstract

Expansive soils have characteristics of swelling and shrinkage with an increase and reduction in water content. This variation within the property poses a problem when used in the construction. Recent advancements within the field of geotechnical engineering dictate that alteration of soil characteristics by adding chemical admixtures will cut back or eliminate this situation. During this work, an effort has been done to analyze the effect of stabilization on the dynamic properties of expansive soil concerning the essential influencing factors. These include the shear modulus, damping magnitude relation characteristics, and resilient modulus for each of the untreated and chloride stabilized expansive soils. For this purpose, a cyclic simple shear test under the condition of sustained traffic load is used.

The findings of the situations show that: for strain vary of 0.01 to 1 percent; the decreasing amplitude of shear modulus of treated soil with strain decreases around 1.2 times that of the untreated soil. Whereas the damping ratio of treated soil increases around 30 percent on average. Thus measured resilient modulus of treated soil is in the allowable range; it satisfies the need of operating performance of treated expansive soil. Moreover, the take a look at results showed that stabilization using chloride compounds encompass a nice impact on altering and improving static still as dynamic properties of expansive soil. So the theoretical reference for the promotion and application of chloride-stabilized expansive soil is going to be used as an alternate suggests that in sensible comes wherever such soil prevails.

Key words: ***Expansive, stabilization, chloride, shear modulus, damping, resilient modulus, amplitude.***

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Symbols and Abbreviation

a	Parameter related to PI
A_{loop}	Area of a hysteresis loop
ASTM	American Society for Testing and Materials
AASHTO	American Association of State Highway and Transportation Officials
$CaCl_2$	Calcium Chloride
CH	High plastic clay
e	Void ratio of soils
D	Damping ratio
DCP	Dynamic cone penetration
$FeCl_3$	Ferric Chloride
FHWA	Federal Highway Administration Offices of Research & Development
G	Dynamic shear modulus
Gmax	Maximum Dynamic shear modulus
Gs	Specific gravity of soil
KCl	Potassium Chloride
MDD	Maximum dry density
M_R	Resilient modulus
NaCl	Sodium Chloride
OCR	Over consolidation ratio
OMC	Optimum moisture content
PI	Plastic index
Qu	Ultimate compressive strength
UCS	Unconfined compressive strength
γ	Shear stain
τ	Shear stress
σ'_m	Mean principal effective stress
σ'c	Pre-consolidation pressure of specimen
σ'p	Present effective vertical stress
σ_d	Applied deviator stress
ε_r	Resilient strain

Chapter One
Introduction

1.1 Background

Expansive soil is one of the problematic soils on earth. This soil is characterized by the dominant mineral referred to as montmorillonite. Expansive soil will cause harm to the structures hosted on that thanks to the wetting-drying method caused by moisture fluctuation. At present, with the prosperous development of infrastructures like highways, high-speed railways, and canals are constructed in the areas of expansive clay. During this case, ground improvement is critical, the soil stabilization chemically is one leading to fairly effective and economic possibility. Alteration of soil characteristics by adding chemical admixture within the soil may be a very fashionable technique for the stabilization of expansive soil. Chemical stabilization improves the soil mechanical property by reducing swelling and shrinkage characteristic of soil at the side of reduction in soil malleability. This alteration of soil characteristics might have a bearing on the dynamic strength and characteristics of the stabilized soil because of a hundred million times of perennial traffic load action (Wang et al., 2012; Yonghui et al., 2017).

Evaluation of the consequences of native soil conditions needs investigation of soil properties below dynamic loading. The behavior of soil subjected to dynamic loading is ruled by what has returned to be called 'dynamic soil properties' (Kramer, 1996).

Determination of dynamic properties of soil like shear modulus and damping is important in the analysis of dynamically loaded foundations, and alternative earth structures like embankments. Issues related to expansive soil are recognized and investigated worldwide. As a result of expansive soil will cowl earthquake-prone areas, varied facilities on expansive soil subjected to dynamic loading are often affected, and thus below standing of expansive soil behavior under dynamic loading is of essential necessitates (Lu et al., 2019; Okur & Akinci, 2018; Zhou et al., 2019).

This thesis work presents the results of dynamic options of expansive soil incorporation with the constant impact of strain amplitude, axial stress, and treatment level on dynamic properties.

1.2 Statement of the Problem

Pavement structures like subgrade are vulnerable to frequent vibration due to repeated traffic load. Traffic loads are dynamic loads and have effects on the strength and deformation of the under-laying soil layer, so to resist these vibratory loads the subgrade needs to satisfy the strength and deformation requirements under the repeated loading action.

If the route of the road passes on expansive soil, one has to make the subgrade workable and improve its strength by stabilization and also control the moisture variation. Dynamic traffic load incorporation with expansive soil has a seriously detrimental effect on the performance of the subgrade soil. Swelling of this soil must be identified so that they can be either removed or stabilized in the pavement design. Remove and replacement with other material has extra cost therefore stabilization improves its strength with minimum cost and effort. The behavior of chemically stabilized expansive soil is varied because of the type and amount of chemicals used.

Design of subgrade soil was done using static load which is not accurately representing the field condition. Therefore using the real scenario of filed condition the design of stabilized subgrade soil input parameters can be gained by using vibratory traffic load rather than static load. And also the influence of chloride compounds on the improvement of expansive soil will be investigated in whether it affect and improve the dynamic strength and related dynamic properties or not.

1.3 Objectives

1.3.1 General Objective

The main objectives of this research are to study dynamic properties of treated and untreated expansive soil and to study the influence of adding chemical stabilizers on the dynamic properties of expansive soil.

1.3.2 Specific Objectives

- To determine the dynamic soil properties including shear modulus, modulus reduction, and damping variations with strains and assess the parametric effect on treated and un-treated expansive soil.
- To determine the resilient modulus of untreated soil and the effect of treatment on it.
- To investigate the effect of chloride on the important soil dynamic parameters.

- To compare the findings of this work with results from different literatures and previous related works used in practice.

1.4 Significance of the Study

The main contribution of this research is that it explores the dynamic characteristics of treated expansive soil under dynamic traffic load. Also, it demonstrates how stabilization improves both the static and dynamic properties of expansive soil.

1.5 Scope of the Study

The scope of this research is limited to identifying the expansive soil property, ferric chloride stabilization, and investigating the effect of this chloride on dynamic properties. It also includes the estimation of resilient modulus for both treated and untreated soils. The authors' experimental results were used to verify the findings. The findings in this document are limited to the soil sample of a specific site.

1.6 Research Methodology

To achieve the above objectives the following methodologies were followed:

I. Literature review: work of literature on expansive soils, soil stabilizations, dynamic properties, and other relevant materials were collected and reviewed.

II. Sampling and testing: The material sampling and testing methods were undertaken as per ASTM and AASHTO standards. A disturbed expansive soil sample was collected from the active site of Addis Ababa around Dembel city center and the chemical additive, ferric chloride was collected from LABKEMICAL PLC. Identification and classification of expansive soil were made first then the engineering properties of the soil samples after stabilization was tested at different percentages of ferric chloride. i.e., 0.5 %, 1.0 %, 1.5 %, 2.0 %, and 4 % by weight of dry soil. To conduct a cyclic simple shear test remolded samples were used.

III. Analysis and discussion of test results.

IV. Conclusions were drawn and recommendations were made based on the results obtained.

1.7 Organization of the Thesis

This research is organized into five chapters. The thesis background, objectives, scope, and methodology are all briefly defined in the first chapter. The second chapter of this research summarizes the literature reviews on expansive soil, stabilization methods of expansive soil, introduction to the concept of resilient modulus, and dynamic soil characteristics of soils. The materials, methods, and experimental setup used in the analysis are presented in the third chapter. The fourth chapter explains the results of basic geotechnical properties of expansive soil before and after treatment and results obtained from cyclic simple shear tests and discussion on test results. The final chapter presents the conclusions and recommendations with the possibility of future extensions of this research.

Chapter Two
Literature Review

2.1 General Background

The current prosperous development of infrastructures such as highways, railways, and canals would involve areas with expansive clay. Roads constructed on these soil areas are known for deterioration and unpredictable behavior. Since the foundation of roads and highways soil; involves earthwork in different forms of subgrade material which is undisturbed, transported, and rework able base or embankment. A good road needs adequate support and material stability otherwise it will rapidly deteriorate. Stable material has no or little volume change and it resists deformation under the repeated traffic load in a cyclic volume change of water (Wang et al., 2012; Al-Rawas & Goosen, 2006).

The subgrade is the lowest supporting layer in the pavement structure under laying the base layer which is expected to have basic desirable characteristics related to strength, stiffness, and swelling. Most of the failures of subgrade soil of pavement caused by the expansive soil in the subgrade. If these properties are not fulfilled, engineers are expected to come up with ground improvement methods (Bantayehu, 2017; Wang et al., 2012; Yuan et al., 2019).

Expansive clay is unsuitable subgrade material, covering about 40% of the area of Ethiopia. This soil needs a cost-effective and environmentally friendly technique for improvement. To eliminate the problem from such soils, a technique of soil stabilization needs to be taken out to enhance some of its properties. There are many techniques of soil stabilization like mechanical, chemical and physical stabilization. Soil stabilization is a collective term for any physical, chemical, or biological method or any combination of such methods employed to improve or change certain properties of natural soil to make it serve adequately an intended engineering purpose. Stabilization improves its strength and workability of expansive soil by reducing volume change due to moisture variation (Bantayehu, 2017; Kim & Zia Siddiki, 2006).

2.2 Reviews on Expansive Soils

2.2.1 Introduction

The characteristics and engineering behavior of soils depend on their geological processes, soil formation mechanics, and local climatic conditions. The problem related to expansive soils covered 3 % of the world's land area. In Ethiopia, coverage of these soils is 40 % and is mostly found in central and southern areas. Cyclic volume changes associated with shrinkage and swelling processes result severe distress and damage in many engineering structures and economic loss (Bantayehu, 2017).

Expansive soil is problematic geo-materials, and it can cause damage to the structures hosted by the wetting-drying cyclic process. And term generally referred to a soil that has the potential for shirking or swelling under changing moisture conditions (Ramana, 1993; Wang et al., 2012). When exposed to a high quantity of water, clay has a property of difficulty to make construction due to its low strength and stiffness. This has a serious problem in geotechnical engineering because they may cause damage and cracks both on the foundation and on the building and also has damage on road pavement.

2.2.3 Identification and Classification of Expansive Soils

2.2.3.1 Identification of Expansive Soils

Expansive soils show high swelling potential in both field investigation and laboratory tests (Chen, 1988; Ramana, 1993). Different tests and methods have been developed for estimating shrink-swell potential and identifying expansive soil. These can be mainly classified into three categories namely; field identification, mineralogical identification, and inferential testing methods (Abu-Farsakh et al., 2015; A. Lemi, 2015; Hamza, 2016; Ravi et al., 2015).

- Field identification method that indicates the potential of expansiveness by exhibiting black and gray color, desiccated surfaces with open or closed fissures, the heave of the ground due to seasonal moisture variation, highly polished or shiny fissure surface, cracking appear in nearby structures of buildings, longitudinal cracks near road shoulders and around the center line and transversal crack emerge in minor drainage of high ways are observed (Burmister, 1949; Khan et al., 2016).

- Mineralogical Identification Methods: X-ray diffraction analysis, differential thermal analysis, dye adsorption, chemical analysis, and scanning electron microscopy are under this method (Asuri & Keshavamurthy, 2016; Chen, 1988).
- Inferential Testing Methods: These methods relate some of the index properties of fine-grained soils with the clay mineralogical composition to estimate their swell potential. Indirect methods and direct methods are classified under this method. Direct methods give exact physical measurements of swelling whereas indirect methods involve the use of basic geotechnical properties and classification schemes to estimate shrink-swell potential (Asuri & Keshavamurthy, 2016).

Indirect Methods: this method is related with the evaluation of some physical properties of soil such as consistency indices and activity. Liquid limit, shrinkage limit, percent clay size composition, plasticity index, and the like estimate the swell potential of soils.

Direct Methods: The methods coming under this category measure the swell potential of a soil directly which provides actual physical measurements of swelling. Laboratory tests include free swell, index of expansion, 1D consolidation-swelling, California Bearing Ratio, and potential volume change have been evolved to directly determine swelling of soil along with moisture content changes.

2.2.3.2 Classification to Expansive Soils

Soils are classified in the general schemes: United Soil classification systems and the American Association of State High Way and Transportation Officials method according to index properties. Soils rated CM or CH by the USCS, and A-6 or A-7 by AASHTO can be considered potentially expansive (Sridharan & Prakash, 2000).

Researchers planned strategies for predicting the potential of swell and distinguishing and classifying the expansive soil. Prediction of the potential of swell exploitation consistency limits square measures the foremost wide used approach. several of the procedure conjointly embody the clay content (Hamza, 2016; Ravi et al., 2015).

The common criteria used for identification and classification of the expansive soils mentioned as below:

1. Method of Chen (1988), and Holtz and Gibbs (1956)

The plasticity index is the best parameter for finding the swelling behavior of all types of clays.

Furthermore, it's been confirmed that the sole take look at which might be used as an initial indication for swelling behavior of most clays is that the physical property index. These researchers showed the effects of the plasticity index of clays on the swelling potential as summarized and indicated in Table 2.1 (Al-Rawas & Goosen, 2006).

Table-2-1: Expansive soil classification based on plasticity index.

Swelling Potential	**Plasticity Index**	
	Chen (1988)	Holtz and Gibbs (1956)
Low	0-15	<18
Medium	10-35	15-28
High	20-55	25-41
Very High	35 and above	>35

2. A method based on shrinkage limit by Altmeyer (1955) and Chen,(1975)

Shrinkage properties of clay are also one method to predict the potential of swell, Table 2.2.

It was Altmeyer (1955) as the first investigator who suggested that it can be possible to predict the potential of swell by using different values of shrinkage limits and linear shrinkage (FHWA, 1977).

Table-2-2 Relationship between shrinkage limit, linear shrinkage, and degree of expansion (Chen, 1975).

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

3. The method suggested by Woodward and Lundgren, (1962)

The swelling potential is another criterion to identify the expansiveness of soil using different values (Woodward, and Lundgren, 1962). The potential of swell can be correlated with swell percentages from oedometer tests in the laboratory and compacted samples with OMC and MDD. This has been indicated in Table 2.3 (FHWA, 1977).

Table-2-3: Relationship between swelling potential and degree of expansion. (Woodward, & Lundgren, 1962)

Swelling potential (%)	**Degree of expansion**
0-1.5	Low
1.5-5	Medium
5-25	High
>25	Very High

2.3 Reviews on Soil Stabilization

2.3.1 General

Soil stabilization is one of the methods in which special soil binding material or any other chemical additives added to problematic or less desirable soils to upgrade or improve their properties. These stabilizing additives can decrease moisture content, increase the cohesion of particles and serve as binding and waterproofing materials (Chen, 1988).

Modification of bearing capacity of less desirable soil by chemical admixture is a common method for stabilizing the swell-shrink tendency of expansive soils. Chemical stabilization decreases the swelling and shrinkage properties of soil and increases its strength, durability, and stuffiness (Ashango & Patra, 2013).

2.3.2 Methods of Soil Stabilization

Stabilizing materials are used to maintain optimum moisture and increase the cohesion between soil particles to produce that act as a waterproofing material.

Stabilized soil can serve as foundation material for a high-quality pavement; which will serve a heavy volume of traffic. Soil stabilization can be made by several methods (Lim et al., 2014; Makusa, 2012).

All these methods summarized by:

- Mechanical stabilization
- Chemical stabilization
- Salt stabilization

Mechanical stabilization is a process of changing natural soil to the desired engineering property by mixing it with different particle size gradations and then compact the new soil to the required dry density using regular methods. This can be achieved by a physical entity by changing the physical properties of natural soil particles by either compaction or by incorporating barriers and nailing. The central idea of mechanical stabilization of expansive soils is the production of a soil or soil mass that will not or cannot change in volume and has sufficient strength to safely sustain the loads applied to it or causes no destruction to transportation facilities as its volume changes (Hasan & Nikraz, 2015; Ikeagwuani & Nwonu, 2019; Makusa, 2012).

Chemical stabilization: This method mainly depends on reactions between stabilizing agents and minerals of soil chemically to attain the desired results. The most widely used chemical additives are cement and lime which are used for pavement treatment and foundation engineering widely. They mainly involve reducing plasticity and increase in bearing capacity (Ikeagwuani & Nwonu, 2019; Lim et al., 2014; Obianigwe & Ngene, 2018).

- **Lime stabilization**: in this method quicklime or hydrated lime is used to improve plastic clay and react chemically with complex silicates of clay or other cementing materials. The lime lowers the water content and plasticity of the soil and improves workability.
  Also used to reduce the shrink/swell potential of clay soils. Lime-stabilized subgrade and sub-base materials are used for very low to high traffic volume applications. However, the amount used in subgrade treatment is generally from 3 to 6 percent.
- **Cement stabilization**: cement creates a hard, bound, and impermeable layer. Cement-treated soils are most frequently used as treated subgrade or road base in roads with very low to high traffic volume applications.

Salt Stabilization: For this stabilization method salt, soil and water are mixed. Before compaction, the salt is dissolved by water and mix with soil. After the mixture dried, the surface becomes dense and hard.

Salt holds together the smaller and large soil particles hold together by salt i.e. “cementing” them together. The most commonly used salt products are chlorides and applied either in a solution or solid-state (Lim et al., 2014; Obianigwe & Ngene, 2018).

For unbound road surfacing with higher traffic volumes, chlorides are mostly used. Stabilization using chloride compound is the basic in this review and, therefore, this document focused on stabilization with ferric chloride.

2.4 Dynamic Soil Properties of Stabilized Expansive Soil

2.4.1 General Background

The economic development of the one country desires different infrastructures like highways, railways, and airfields runway so on. These structures have high speed, significant masses, and recurrent actions. Traffic load has repetitive nature, the dynamic response of foundation soil underneath these masses got to be studied to grasp the behavior of pavement.

The utilization of static loading doesn't represent the particular loading so, a lot of attention ought to be paid to dynamic and creep characteristics. During this context, static style ways can't be ready to meet the increasing necessities of subgrade and pavement materials for strength and deformation (Chiang & Chae, 1972; Okur & Akinci, 2018; Yonghui et al., 2017).

In observe there's a limitation to account for real properties of the subgrade soils subjected to repeated traffic loads. It's essential to get a lot of data relating to with dynamic properties of most road-making materials underneath moving load. This document describes the experimental investigation conducted to see the dynamic properties of chloride stabilized soils.

2.4.2 Resilient Modulus

A resilient modulus is one of the most parameters characterizing the mechanical properties of subgrade soils and the stability of subgrade soil. Now a day's resilient modulus evolved into thought for a style of subgrade. Most pavement engineering has a similar definition for resilient modulus. Typically it is represented because of the magnitude relation of applied deviatoric stress to redeemable or resilient strain, therefore it represents the stiffness of the subgrade soil. I.e. resilient modulus represents elastic behavior and cargo-carrying capability of subgrade soil underneath dynamic traffic loading. Therefore it helps to characterize the soil in a mechanistic-empirical pavement style guide (Liu & Zhou, 2012; Kim et al., 2005; Yuan et al., 2019).

Resilient modulus expressed as the magnitude relation of applied deviator stress to recoverable strain in equation 2.1.

$$M_R = \frac{\sigma_d}{\varepsilon_r} \qquad (2.1)$$

Where, M_R = Resilient modulus

σ_d = Applied deviator stress

ε_r = Resilient strain

The use of M_R for characterization of subgrade soil indicates resilient modulus is a basic material property that might be utilized in the mechanistic analysis of multi-layered systems and for characterizing pavement materials.

2.4.3 Dynamic Shear Modulus and Damping Ratio

The basics principles adopted within the simulation of a typical moving load is in a sinusoidal form. The best purpose of the loading is analogous to the loading condition of traffic forthwith higher than the subgrade. The response of soils to the present loading condition is controlled principally by the physical soil properties. Properties connected with dynamic loading area unit shear wave rate (Vs), shear modulus (G), and damping ratio (D). The customary names for this type of properties are “dyanamic soil properties”. Once soils are subjected to dynamic loading it causes stability problem then, large strains are induced (Luna & Jadi, 2000; Kim & Zia Siddiki., 2006).

The cyclic shear stress-shear strain properties of soil is a key to an understanding to how the soil reply to the applied shear loads such of these are created by different dynamic loads like repeated traffic load, earthquake, blasting and so on. This can be nonlinear and hysteretic (Buzacott, 1987).

The hysteresis loop is made and represented by two parameters. These parameters are shear modulus and damping that is inclination and breadth of the hysteresis loop respectively. Figure 2.1 could be a simplified schematic showing one loop of cruciform cyclic loading and its corresponding parameters.

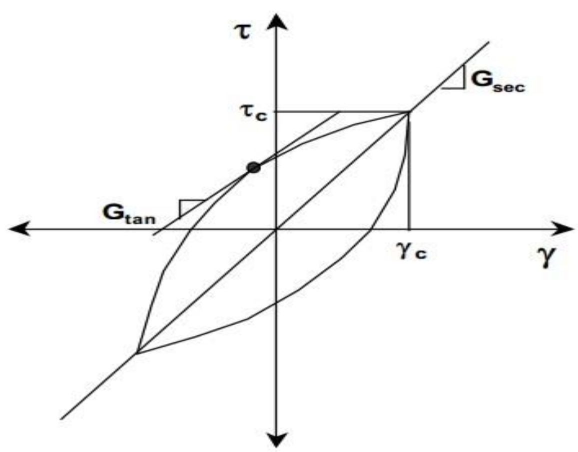

Figure-2-1: Hysteresis Loop Showing Secant and Tangent Shear Modulus.

As the values of strain amplitude varied, completely different size loops become developed. The line joining the locus of points corresponding to the tips of the loop is called backbone curve (or skeleton). Secant shear modulus can decrease once shear strain becomes increases.

Therefore, at a very small shear strain maximum shear modulus was developed. A different way of representing this shear modulus degradation with cyclic strain is by means that of the modulus reduction curve (Luna & Jadi, 2000).

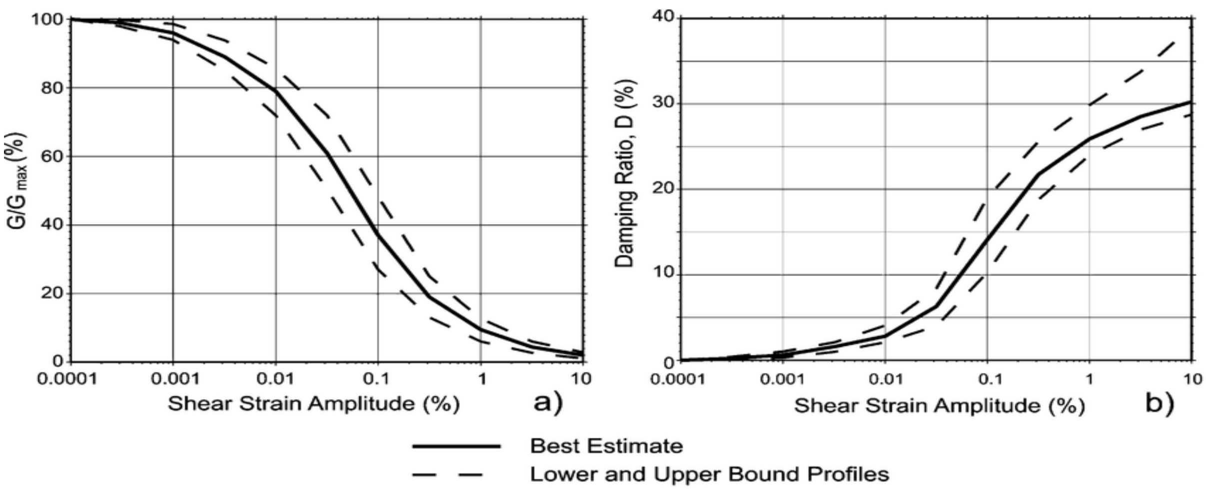

Figure-2-2: Modulus reduction and Damping ratio Curves (Győri et al., 2011).

As shown in Figure 2.2, the modulus reduction curve and damping ratio vartiation with shear stain show reciprocally relationship. Damping is usually expressed in terms of damping ratio (D) and obtained by dividing the world of the loop by the world triangle from the physical phenomenon loop. Damping magnitude relation represents the power of a fabric to dissipate dynamic load or dampen the system. It ought to be noted that several factors contribute to the stiffness of soils throughout cyclic loadings, such as malleability wet content, quantity and kind of treatment, cyclic load, and soil structures.

2.4.4 Factors Affecting Dynamic Characteristics of Treated and Untreated Expansive Soils

For chemically stabilized expansive soil the major important influencing factors are confining pressure, shear strain amplitude, moisture content, loading frequency, treatment level type of additives, and soil structures. Those factors affecting the shear modulus and damping ratio of soils have been studied by Hardin and Drnevich from these the strain level, the plasticity index and the mean effective stress are discussed in detail (Chiang, Y. C., & Chae, Y. S., 1972; Hardin & Drnevich, 1972).

On the other hand, the most influencing factors on resilient modulus are deviator stress, compaction water content, and dry density. Another factor that affects the resilient modulus is the seasonal variation of moisture content. Decrease in moisture content and an increase in density the resilient modulus of soil become increases and as deviator stress increases it comes to decrement (George, 2004).

Since moisture variation is the major cause of cyclic volume change of expansive soil. Therefore it can be taken as a main influencing factor for estimating resilient modulus by altering the treatment level.

2.5 Previous Related Research Works on Both Static and Dynamic Loading

Farid & Wahdan, n.d. studied the behavior of expansive soil treated by using different electrolyte Substances. The electrolytes used as stabilizing agents are Sodium Chloride (NaCl), Potassium Chloride (KCl), Calcium Chloride ($CaCl_2$), and Ferric Chloride ($FeCl_3$). Different percentages of the chemicals substances mixed with soil for testing. From the laboratory studies, it was observed that values of liquid limit and plasticity index reduced with adding the different chemical substances up to 1.5% percentages of any of the electrolytes. More increase of the chemical percentage than 1.5% will not give a significant reduction.

The Shrinkage Limit values are increased. Using 1.5% percentage of NaCl, KCl, $CaCl_2$ and $FeCl_3$ treatment, UCS values are increased by 137 %, 150 %, 167 % and 187 % for 7 days curing period respectively. The CBR values was increased by 155 %, 180 %, 200 and 215 % respectively for 1.5 % of NaCl, KCl, $CaCl_2$ and $FeCl_3$ treatments.

Durga, (2017) studied the stabilization of soil using chemical methods. The experimentation was conducted on highly problematic soil of India which is categorized under highly expansive soil. This document investigates the effect of potassium chloride, calcium chloride, and ferric chloride with the addition of 0.5, 1, and 1.5 % by dry weight on the properties of expansive soil under a controlled laboratory. After the experiment, it is observed that the liquid limit values are decreased by 57 %, 63%, and 70% respectively for 1 % of KCl, $CaCl_{2,}$ and $FeCl_3$ chemicals added to the expansive clay. The shrinkage limit is increasing with 1.5 % chemical addition; i.e. the shrinkage limit of stabilized expansive clay is increased from 12 % to 15% for KCl and $CaCl_2$ and 16 % for KCl, $FeCl_3$.

The D.F.S values are decreased by 40 %, 43 % and 47 % for 1 % of KCl, $CaCl_2$ and $FeCl_3$ treatments respectively. UCS values after 14 days curing were increased by 133 %, 171 % and 230 % respectively for 1 % of KCl, $CaCl_2$ and $FeCl_3$.

Yonghui et al., (2017) conducted a laboratory experiment on the dynamic properties of cement stabilized subgrade used in heavy haul railway. A dynamic triaxial test under the condition of sustained vibration was conducted under different dynamic stress amplitudes, confining pressures, and vibration frequencies in this study. Silicate cement was used to improve soil samples, and cement contents were 3% and 5%. Considering features of train load action, single-width vibration sine waves were used for loading, and frequencies were taken as 1 and 5 HZ. The decrement of dynamic modulus reaches 70 %, whereas the increment of damping ratio is 80% with stain. The working performance of cement-stabilized expansive soil used as subgrade satisfies requirements.

Wang et al., (2012) studied characteristics of expansive soil stabilized by lime and subjected to dynamic load. During this study, the dynamic behavior of lime-treated expansive clay was investigated and mentioned with prestigious factors, together with water content, confining pressure, vibration frequency, over consolidation ratio, and cycle variety on the dynamic characteristics. Dynamic modulus will increase with the expansion of vibration frequency, over consolidation ratio, and confining pressure; whereas it was opposite for the water content.

It's conjointly found that dynamic modulus initially declines step by step then reaches a leveling-off with the increase of vibration variety. While increasing vibration frequency and water content, the damping ratio decreases step by step.

These previous studies showed chemical additives considerably improve the static properties of expansive soil. When the treatment reaches optimum quantity, it will satisfy the need to be used as a construction material. Hence more previous studies were done on dynamic characteristics of treated expansive soil with lime and cement; these studies show stabilization improves the dynamic properties, but they have basic disadvantage related environmental pollution and needs more cost. However dynamic properties of stabilized expansive soils treated with chloride were not studied widely and it's cost effective method. Thus this paper shows the impact of chloride compound in basic engineering properties and dynamic parameters of expansive soil.

Chapter Three

Materials and Methods

3.1 Introduction

Expansive soil cannot be used as construction materials unless it can be replaced or treated. The workability of this soil depends on the method of treatment. Proper treatment techniques should be adopted for successful improvement. In this chapter material used and method adopted are discussed briefly.

3.2 Materials

The testing program and procedure have 3 processes. These processes are soil sample choice, improvement method of soil, and dynamic loading system.

3.2.1 Soil Sample Selection

The soil sample employed in this analysis was collected from active sites of Addis Ababa around the Dembel city center. The sample was collected in the dry season. Disturbed soil samples were taken at a depth of 1.5 m from natural ground level and its black color.

3.2.2 Chemical Used for Improvement

Commercially grade ferrous chloride chemical agent is employed for this study as stabilizing electrolytes. The advantages of using ferric chloride over other stabilizing agents are the following: It is a non-combustible, commercially available, and cheaper additive. Due to its sparingly soluble in water, free cations are available for substantial cation exchange reaction.

The purity of this chloride compound is about 99.0 % whereas the rest 1.0 % is other impurities which are summarized in Table 3.1.

Table-3-1: Chemical specification of ferric chloride (Manufacturer Manual).

Specifications	**Purities (%)**
Minimum Assay of ferric chloride ($FeCl_3$)	99.0%
Maximum Limits of Impurities	%
Arsenic (As)	0.0005
Copper (Cu)	0.04
Magnesium (Mg)	0.15
Lead (Pb)	0.02
Zinc (Zn)	0.04

3.3 Methods

3.3.1 Sample Preparation and Stabilization

The collected soil sample was air-dried, then pulverized by a rubber hummer. Laboratory tests are conducted as per ASTM and AASHTO manuals to seek out the kind and properties of virgin soil to see the essential physical and mechanical soil parameters. For every test distilled water is used to remold the sample.

Chloride based chemical was accustomed for improving the soil sample. Soil composites were ready by combining soil with 0.5 %, increment of chloride (in powder form) by weight of dry soil. i.e. 0.5, 1, 1.5 and 2 percent of $FeCl_3$% become added.

In the laboratory mixing was done manually with correct care to form an identical combine and left for someday then confirm physical and mechanical parameters for every composite.

The basic geotechnical properties employed for each untreated and treated soil to classify the soil and conjointly to outline the optimum chemical. Table 3.2 describes the laboratory tests done to outline the properties of the sample.

Table-3-2: Laboratory tests for determining basic geotechnical properties.

Standard References	Laboratory tests
AASHTO DESIGNATION T 088-93	Standard method of test for particle size analysis
AASHTO DESIGNATION T 089-94	Standard method of test for Liquid Limit
ASTM D 4318	Standard Test Method for Plastic Limit, and Plasticity Index of Soils
ASTM D 854-00	Specific Gravity by Pycnometer
ASTM D 2435	One dimensional consolidation
ASTM D 698	Standard test methods for laboratory compaction
ASTM D 2166	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil

3.3.2 Dynamic Loading System of the Test

From different reasons for cyclic loading or vibration, traffics and rail tracks area units are one amongst them. This traffic still as rail tracks have continual loading on the underlying soil (Ishihara, 1997; Yonghui et al., 2017).

To study the cyclic loading impact on the subgrade soil cyclic simple shear test is employed to live dynamic properties of the soil at different strain values.

3.3.2.1 Summary of cyclic Simple Shear Testing System

In this analysis, the kind of equipment used is a 31-WF7500 cyclic simple shear machine that is controlled by the UTS004 package applications programmer as shown in Figure 3.1.

Generally, cyclic simple shear equipment is employed in the dynamic field of soil behavior and may simulate quite simply many various field loading conditions. The cyclic simple shear test may be a plane strain device. The shear strain is elicited by horizontal movement at the rock bottom of the sample relative to the top.

The horizontal diameter of the sample remains constant; thus any modification in volume shall be a result of vertical movement of the top platen (Seinwill, 1984). The system is meant to permit a sample to be consolidated, drained then sheared.

Figure-3-1: 31-WF7500 cyclic simple shear machine.(Seinwill, 1984)

- **Sample preparation**

To conduct the cyclic simple shear test, either undisturbed or remolded soil samples to field condition (at field density and water content) are used to represent the natural state. The sample has a 70 mm diameter with a 20 mm height.

Specimen sample placed on a pedestal and supported by a rubber membrane and stationed with O-rings. To keep a constant diameter throughout the test, the sample is supported by a series of slip rings (Seinwill, 1984).

- **Consolidation stage**

In this stage, the specimen sample is subjected to consolidation forces. It is carried out after the sample is prepared and placed for testing. By keeping the lateral loading constant apply static axial loading stress to the specimen. Then the system measured and logged axial stress and lateral displacements of specimen data over time. At the time of proceeding to this stage, data are a displayed operator in the form of charts and tables to the operator. When consolidation of the specimen sample was complete, the operator manually terminates the process. At this stage the sample is not saturated (Seinwill, 1984).

- **Cyclic simple shear stage**

This stage shows the dynamic forces/displacements being applied to the sample. Therefore test applies a lateral cyclic shear force or displacement to the specimen. Since the axial axis is maintained at the specified stress then specimen height is maintained. For each loading cycle axial force, lateral force, and specimen displacements are measured. Fifty sample points of measured data are obtained then displayed in the form of wave shapes, charts, and tables. The loading cycle shape is selectable from predefined functions by the operator, but may also be a user-generated shape (Seinwill, 1984).

3.3.2.2 Cyclic Simple Shear Test Contents and Procedure

In this research, the cyclic shear test conducted is strain-dependent which allows a lateral cyclic displacement to be selected to the specimen.

Dynamic loading tests to be conducted by using remolded soil samples. The samples were prepared with maximum dry density and optimum moisture content for both untreated and treated soils.

The following procedure was used for the cyclic simple shear test.

- The sample of the specimen is cylindrical with a height of 20 mm and a diameter of 70 mm. Traffic load actions were considered as a sinusoidal wave to simulate loading action.
- After sample preparation, the cyclic simple shear test has two stages. The first stage is the consolidation and the second is the cyclic shear stage.
- For the consolidation stage application of static axial loading, stress is varied from 50 kPa to 200 kPa by keeping the lateral loading stationary then axial stress and lateral displacement of the tested specimen recorded over time and logged. These logged data are presented in the form of a chart. This stage continues until the rate of vertical strain becomes less than 0.05 %/ hr. After all, this is completed terminate the consolidation of the specimen manually. After the consolidation stage was completed immediately cyclic shearing stage was continued.

- In the cyclic shear stage lateral cyclic displacement is selected to the specimen while keeping specimen height constant then lateral force and displacements of the specimen were measured for each cycle with time then calculate shear stress and shear stain values. The hysteresis loop of each cycle can be computed and plotted from the results of shear stress and shear strain. Shear stress and shear strain values in a cycle have been analyzed for the determination of shear modulus and damping ratio values of the hysteresis loop.

The shear modulus and damping ratio values are determined for certain cycles. Thus, in this research, the 10th cycle is selected for analysis purposes.

The machine enables to conduct the cyclic simple shear test with strain level 0.01 to 5 %, however, strain rate for subgrade soil varies from 0.003-0.6 % and for sub-base vary from 0.1 to 1 % since the overall strain rate for pavement base, sub base, and subgrade layers involves at high level i.e. the typical strain rate value of pavement ranges from 0.01 to 1 %, therefore, applied shearing strain values ranges from 0.01 to 1 % were used in this study. The uniform sinusoidal loading frequency is 2 HZ (Ishihara, 1997; Picchio et al., 2020; Sawangsuriya et al., 2006). For this research, frequency is considered as a basic influencing factor of the dynamic properties of the stabilized subgrade. Table.3. 3 summarizes the basic input values of axial stress, cyclic shear stain, and frequencies used in this study.

Table-3-3: Test values of axial stress, shear strain, and frequency.

Sample type	**Axial stress (kPa)**	**Shear strain (%)**				**Frequency (HZ)**
Remolded to field condition	50	0.01	0.1	0.6	1	2
	100	0.01	0.1	0.6	1	2
	200	0.01	0.1	0.6	1	2
Shear strain amplitude		0.002	0.02	0.12	0.2	-

The cyclic amplitude used for this paper is calculated from applied shear strain and shown in Table.3. 4.

$$Shear\ strain\ (\%) = \frac{\text{Amplitude}(\text{A})}{\text{Height of sample}} \tag{3.1}$$

Table-3-4: Applied shear strain corresponding with cyclic amplitude.

Shear stain (%)	**Amplitude(A)**
0.01	0.002
0.1	0.02
0.6	0.12
1	0.2

Chapter Four

Data Analysis and Discussion of Test Results

4.1 Introduction

In this chapter laboratory, results and discussion are described in detail. The basic geotechnical properties of the treated, as well as un-treated soil samples, were evaluated. For the most practical purposes, the specimen was cured for 7 days for the consistency tests and strength tests For the cyclic simple shear test 1 day cured specimen is used, but the other laboratory tests are conducted by an uncured specimen.

4.2 Properties of Natural (untreated) Soil

The laboratory tests were carried out according to ASTM and AASHTO standard manuals. The basic geotechnical properties of the soil sample are given in Table 4.1.

Table-4-1: Basic geotechnical properties of natural (untreated) Soil.

Soil Properties	**Values**
Grain size distribution	
Percentage passing sieve No. 200(%)	95.43
Specific Gravity	2.73
Atterberg Limit	
Liquid Limit (%)	94
Plastic Limit (%)	40
Plasticity Index (%)	54
Proctor Compaction	
Maximum Dry Density (kN/m^3)	12.85
Optimum Moisture Content (%)	34.75
Free Swell (%)	140

According to Table 4.1 a soil with a percentage passing No. 200 sieve (0.075 mm) of 95.43 % exhibits liquid limit 94 %, plastic limit 40 %, and plasticity index 54 % is classified as A-7-5 according to AASHTO. It also has poor engineering property; therefore it cannot be used as a construction material unless it's treated properly.

As Chen (1988) and Holtz and Gibbs (1956) suggested, Atterberg limits were used to predict the swelling potential. Hence the soil used in this research has a plasticity index of 54 %; it can be classified as having very high swelling potential.

The soil generally falls below the standard recommendations for most geotechnical construction works. Therefore, the soil requires initial modification and/or stabilization to improve its workability and engineering property. There are different techniques used to treat expansive soil; chemical stabilization among the ones used and makes it workable. Details of test results of grain size analysis, Atterberg limits, standard compactions, unconfined compression, and swelling pressures are given in Appendix A.

4.2.1 One-dimensional Consolidation

Consolidation characteristics of the soils were determined to understand their stress history. To determine the pre-consolidation pressure oedometer test was conducted. Figure 4.1 below shows the e-log p curve; where e is plotted to a natural scale and P is plotted to a logarithmic scale. The Pre-consolidation pressure obtained was 200 kPa. Detailed calculations of test results are shown in Appendix B.

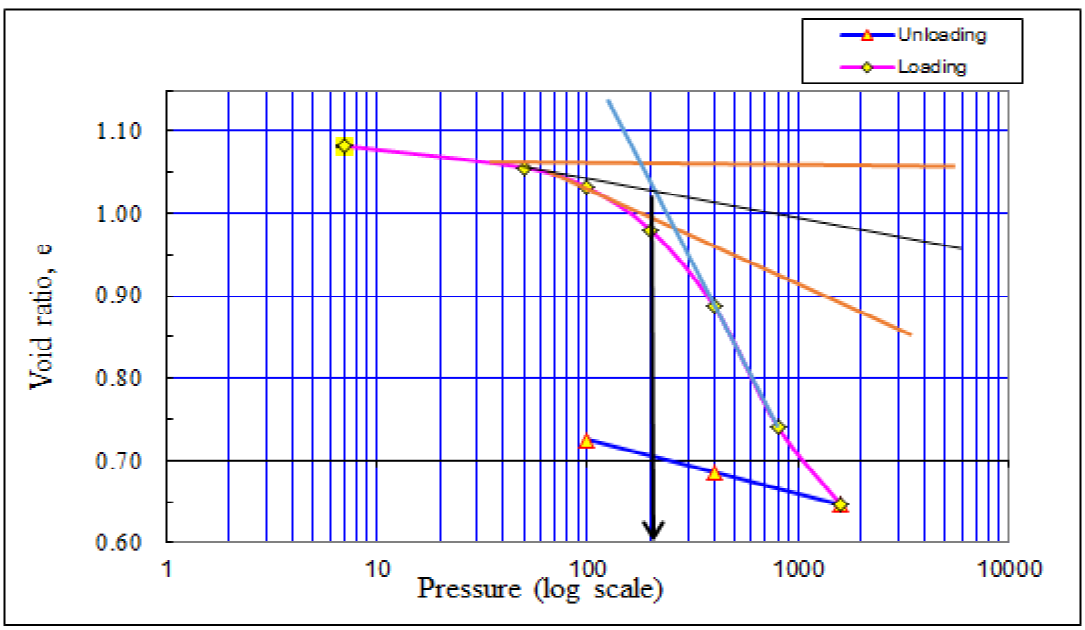

Figure-4- 1: Consolidation pressure versus void ratio.

4.3 Properties of Treated Soil with Ferric Chloride ($FeCl_3$)

4.3.1 Effect of Ferric Chloride on Atterberg Limits

The addition of a different percentage of Ferric chloride to the expansive soil affects the Atterberg limits. Liquid limit values were reduced with the addition of chloride on expansive soil and also plastic limit was significantly increased with adding ferric chloride up to 1.5 percentages. Furthermore, the addition of chloride more than 1.5 % will not show a significant increment. The summary of the test results of Atterberg limits for varying percentages of ferric chloride is shown in Table 4.2.

Table-4-2: Summary of the test results of Atterberg limits for varying percentages of ferric chloride.

Ferric Chloride ($FeCl_3$) %	Time of test conducted	Liquid Limit Value (%)	Plastic Limit Value (%)	Plasticity Index value (%)
0	-	94	40	54
0.5	No curing	80	49	31
1		79	50	29
1.5		72	52	20
2		71	53	18
4		70	48	22
0.5	7 days Cured	80	49	31
1		78	52	26
1.5		71	52	19
2		72	50	22

In general, use of chlorine compounds as a stabilizing agent has an effect on index properties. The increase in plastic limit and simultaneous decrease in liquid limit cause net reduction of plasticity index with the addition of chloride to the expansive clay soil. The reason for the reduction in plasticity index with chloride could be the solubility of chloride in water and reduction of double diffusible layer thickness due to cation exchange. Curing of specimens was done with the help of desiccator by putting the mix of soil and powdered ferric chloride which were not moulded. Hence the mixture is dry; it had not as such significant effect on the Atterberg limits. Figure 4.2 shows the net reduction in plasticity index of different percentages of ferric chloride.

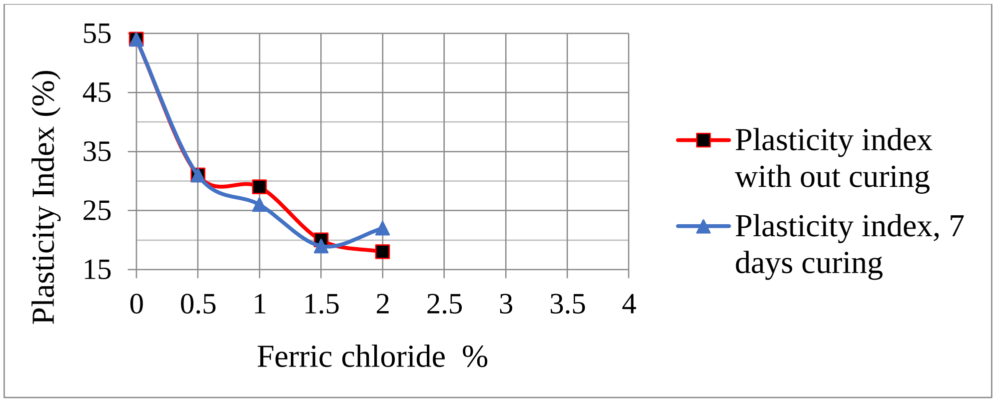

Figure-4- 2: Reduction in plasticity index for different percentages of ferric chloride.

4.3.2 Effect of Ferric Chloride on Soil Swell

I. **Free swell**: The method was suggested by Holtz and Gibbs, (1956) to measure the expansive potential of cohesive soils. It gives a fair approximation of the degree of the expansiveness of a given soil sample. Freeswell is determined as the ratio of change in volume to the initial volume, expressed as a percentage. The summarized free swell of untreated and treated soil was given in Table.4.3. below.

Table-4-3: Effects of Ferric Chloride on Free Swell of soil.

Amount of Ferric Chloride (%)	**Initial Volume (ml)**	**Final Volume (ml)**	**Free Swell** $= \frac{\text{Initial Volume} - \text{Final Volume}}{\text{Initial volume}}$ **(%)**
0	10	22	120
0.5	10	20	100
1	10	19	90
1.5	10	17	80
2	10	16	60

II. **Swell Pressure test**: This test can be done by consolidation test apparatus. Using swell – consolidation method, swelling pressure tests are performed for natural and treated soils. Place a soil sample in an oedometer and apply a normal load equal to 1psi (7kN/m2) then inundate the sample and allow it to swell. After the swelling is complete, load the sample in increments until the soil returns to its original volume. The pressure that corresponds to the original volume is the swelling pressure. Figure 4.3 shows how swelling pressure is measured from the swell pressure test.

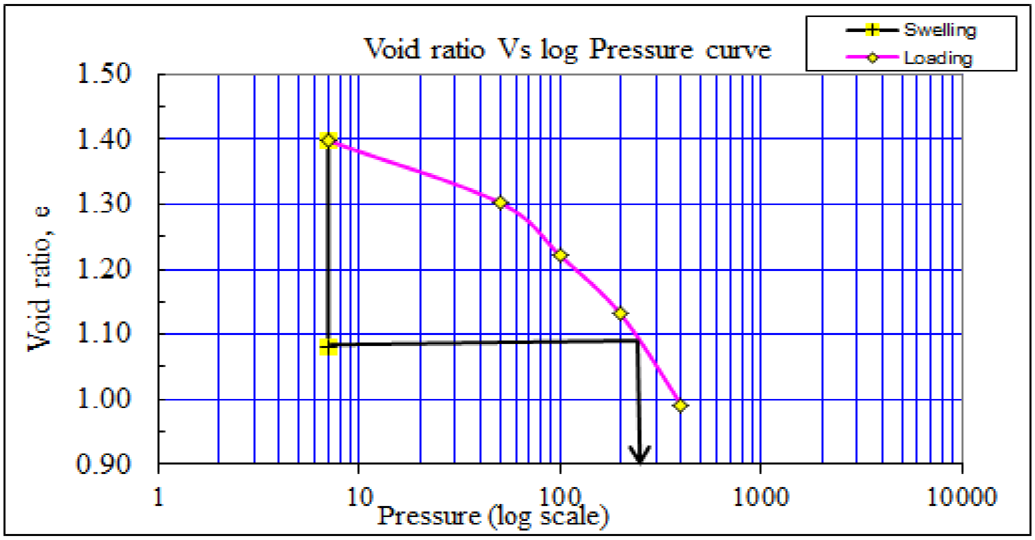

Figure-4- 3: Swell – Consolidation versus void ratio to predict swelling pressure of natural soil

From Figure 4.3 above the swelling pressure of natural soil was 280Kpa. Using the same procedure after treatment the swelling pressure decreases to 70 kPa. Appendix B 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. The addition of 1.5 -2 % of chloride can be used as the optimum amount to regulate the swelling properties of this expansive soil.

4.3.3 Effect of Ferric Chloride on Moisture Density Relations of the Soil

All the treated soil samples' compaction curves fall on the dried side of the untreated soil compaction curve. The addition of ferric chloride affects decreasing the moisture content and increasing the dry density of the untreated soil. For all percentages of chloride used (0.5 % - 2 %), the maximum dry density is higher than that of the untreated soil whereas the optimum moisture content is reduced. Table.4.4 shows the effect of the addition of ferric chloride on moisture density relations of the soil. Figures 4.4 and 4.5 show summarized results for OMC and MDD.

Table-4-4: Effect of ferric chloride on moisture density relations of the soil.

Ferric Chloride ($FeCl_3$) %	0	0.50	1	1.50	2
Optimum water Content, %	34.75	32.5	31.5	25	27.5
Maximum dry Density, g/cm^3	1.29	1.31	1.33	1.42	1.41
Wet unit Weight, g/cm^3	1.73	1.74	1.75	1.77	1.79

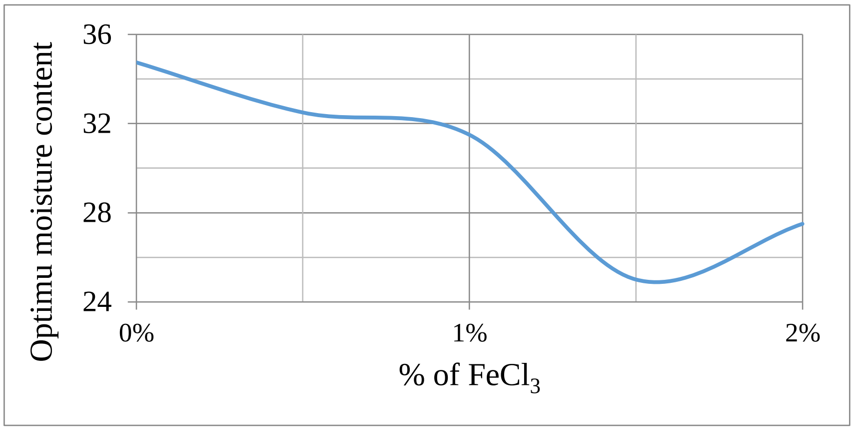

Figure-4- 4: Optimum moisture content for varying percentages of ferric chloride.

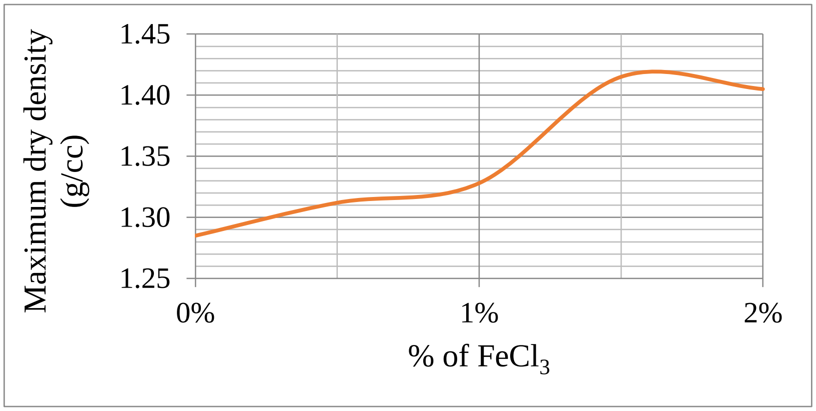

Figure-4- 5: Maximum Dry density for varying percentages of ferric chloride.

4.3.4 Effect of Ferric Chloride on Shear Strength Properties

The unconfined compressive strength was conducted on a remolded sample and the prepared samples are tested without curing and 7 days curing. It's observed that unconfined compressive strength increases with time of curing it may be due to the soil mineral layers and the ferric ions can get enough time for reaction and bonding. As the percentage of chloride added increases up to 2 % the unconfined compressive strength also increases.

Beyond 2 % the strength properties increase slightly due to the high water absorption properties of the chloride. Table.4.5 summarizes the effect of ferric chloride added to expansive soil on UCS and Figure 4.6 shows the effect of varied percentages of ferric chloride on the unconfined compressive strength of the soil.

Table-4- 5: Variation of unconfined compressive strength of treated expansive clay soil.

Chemical added to the soil	**$FeCl_3$ %**	**Unconfined Compressive Strength (kPa)**	
		Without curing	7 days curing
Untreated	-	115	-
Ferric Chloride ($FeCl_3$)	0.5	122	136
	1	143	179
	1.5	216	265
	2	214	292

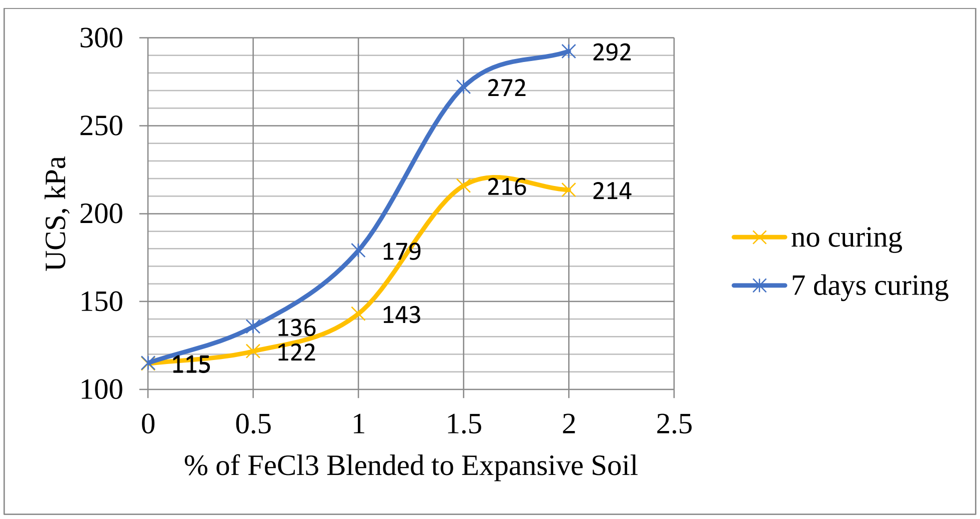

Figure-4- 6: Effect of the varied percentage of ferric chloride on unconfined compressive strength of the soil.

4.4 Laboratory Investigation of Dynamic Soil Properties

4.4.1 Determination of Shear Modulus and Damping Ratio of Untreated Soil

4.4.1.1 Shear Stress and Strain Parameters

From Cyclic simple shear test, for a single cycle loading lateral lvdt. (Specimen displacement) and lateral force was measured and displayed to the operator on Microsoft Excel Spreadsheet. This raw data had 50 data points for each cycle of loadings. To calculate shear strain and shear stress equation.4.1 and 4.2 are used respectively.

$$\gamma = \frac{Lateral\ Lvdt.\left(Displacement\right)}{Height\ after\ consolidation < 20mm} \tag{4.1}$$

$$\tau = \frac{Shear\ Force}{Area\ of\ sample} * 10^3 (MPa) \tag{4.2}$$

$$Area\ of\ sample = \pi * \frac{d^2}{4} \tag{4.3}$$

Where: γ = Shear stain

τ = Shear stress

d= Diameter of the ring

$$Area\ of\ sample = \pi * \frac{70^2}{4} = 3848.5\ mm^2$$

In repeated loadings like traffic, the number of significant cycles is likely to be more than 1000. In this study, the specimen was cyclically loaded through 1000 cycles (Ishihara, 1997). For all practical purposes, the values determined at one hundred cycles are likely to provide reasonable values. To simulate repeated traffic loading sinusoidal loading cycle has been selected. The sinusoidal wave for a 1 % strain and 2 Hz and a single cycle is shown in Figure 4.7.

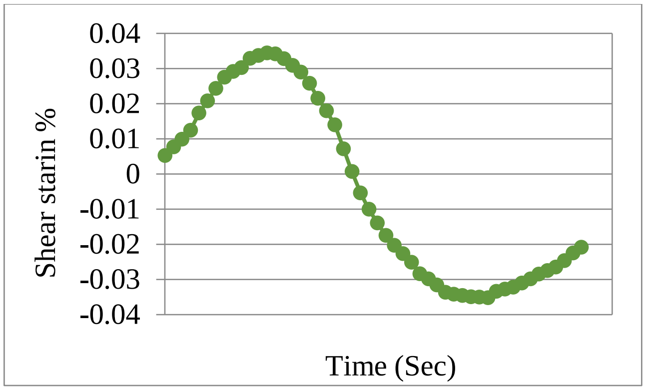

Figure-4- 7: Sinusoidal wave shapes for 1 % strain of 50 kPa and 2 Hz for a single cycle.

4.4.1.2 Computation of Shear Modulus and Damping Ratio

To compute the shear modulus and damping ratio the hysteresis loop of each cycle is needed. The hysteresis loop of every cycle loading can be plotted using the computed shear tress and recorded shear strain values. The lateral force and specimen displacement record of 50 points at the 10th cycle is used for the typical computation of shear stress and strain values.

Shear modulus and damping ratio can be computed using equations 4.4 and 4.5 respectively. (B. Das & G.V. Ramana, 2010)

$$Shear\ modulus,\ G = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}} \qquad (4.4)$$

$$Damping\ ratio,\ D = \frac{Area\ of\ the\ loop}{4 * \pi * area\ of \Delta\ )} \qquad (4.5)$$

The area of the loop can be computed by the trapezoidal rule. The trapezoidal rule allows estimating the values area of a given curve using a definite integral. The formula for the area of the curve drove from the definite integral of two consecutive points from the hysteresis loop. Therefore the area of the loop is the summation of the area of each curve.

$$Area\ of\ the\ loop = 0.5 * \sum(\ \tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1}) \qquad (4.6)$$

$$Area\ of\ \Delta = 0.5 * A * B \qquad (4.7)$$

Figure 4.8 below illustrates the hysteresis loop of 50 sample points. These are used to compute the shear modulus and damping ratio. First, draw a compatible right angle triangle on the half side of the hysteresis loop then measure the height (B) and the side adjacent to the right angle (A). Using these two measurements compute shear modulus and damping ratio.

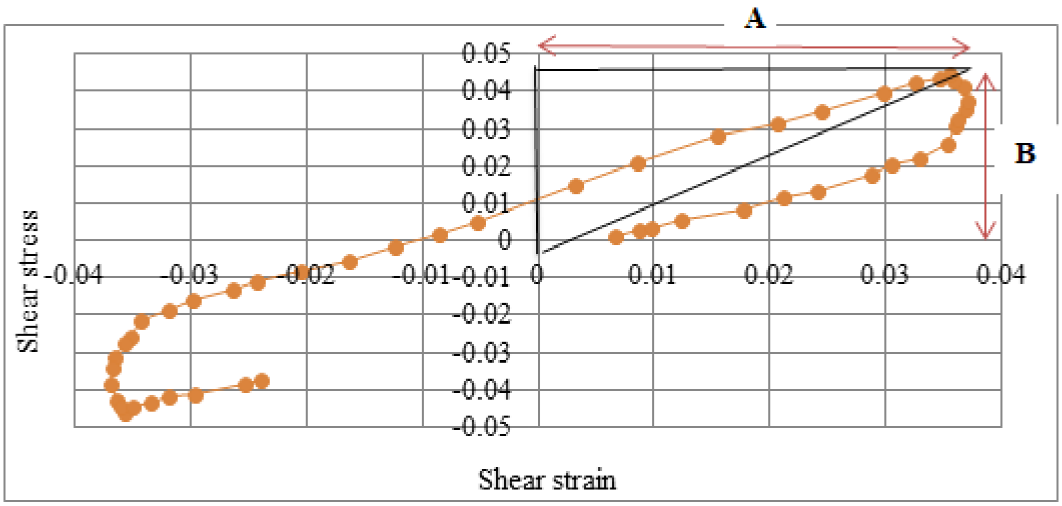

Figure-4- 8: Shear stress and shear strain of natural soil to show hysteresis (1 % strain, 100 kPa, and 10th cycle).

Using Figure.4.8, the computation of shear modulus, G, and damping ratio, D is presented in Table 4.6.

Table-4-6: Typical calculation for shear modulus and damping ratio of natural soil.

Computation of Shear modulus, G		**Computation of Damping Ratio, D**	
τ_{max}	0.044563	Area of loop	0.002343
τ_{min}	-0.04622	Area of Δ	0.00084
$\tau_{max} - \tau_{min} = 2A$	0.090781	Damping ratio, D (%)	22.21
γ_{max}	0.037147		
γ_{min}	-0.03687		
$\gamma_{max} - \gamma_{min} = 2B$	0.074021		
G (MPa)	1.23		

Table 4.6 shows the shear modulus and damping ratio of natural soil with 1 % shear strain, 10^{th} cycle, and 100 k Pa axial stress. The same procedure was used to compute the shear modulus and damping ratio for different shear strain amplitude and given in Appendix C:2.

In this analysis, the shear modulus decreases as shear strain increases whereas the damping ratio increases with increasing shear strain.

4.4.2 Determination of Shear Modulus and Damping Ratio of Treated Soil

The type and amount of chemicals used as stabilizing agents affect shear modulus and damping ratio. The optimum percentage of chloride to stabilize the natural soil was gained from its static properties. In this document 1.5-2 % of ferric chloride was used as an optimum amount. Using this optimum amount remolded specimens were prepared and a cyclic shear test was conducted. The computation method of shear modulus and damping ratio of treated soil was the same as untreated soil. Figure 4-9 shows a typical hysteresis loop of treated soil for 1% strain and 100 kPa axial stress. Using these loop shear modulus and damping ratio of treated soil were computed and given in Table 4.7. The shear modulus and damping ratio for the rest of the cycles are calculated similarly and summarized in a table given in Appendix C: 2.

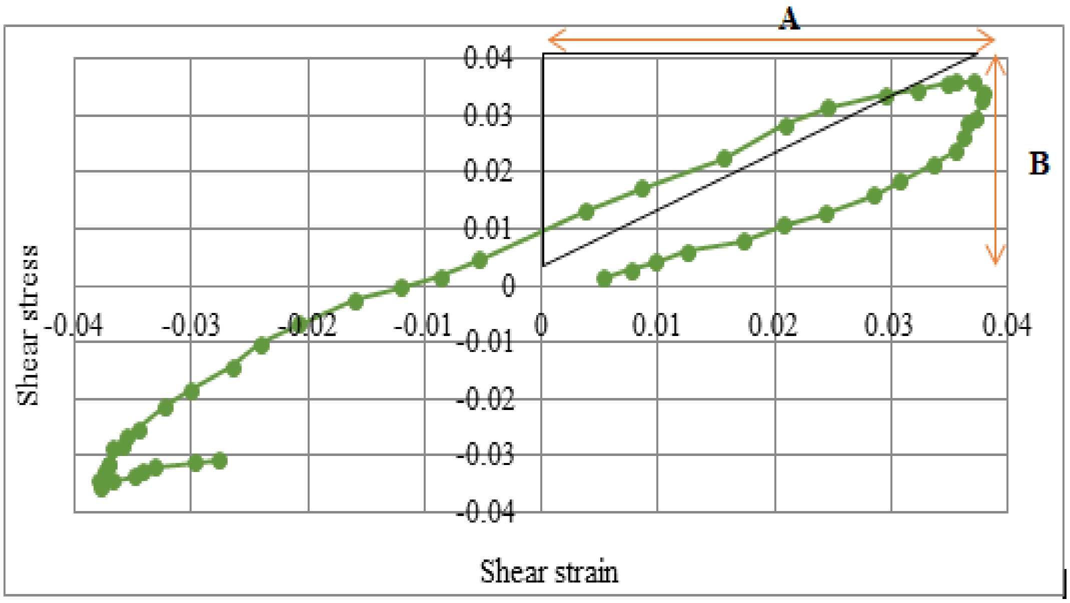

Figure-4- 9: Shear stress and shear strain of treated soil to show hysteresis (1 % strain, 100 kPa, and 10th cycle).

Table-4-7: Typical calculation for shear modulus and damping ratio of treated soil.

Computation of Shear modulus, G		Computation of Damping Ratio, D	
τ_{max}	0.035884	Area of loop	0.001961
τ_{min}	-0.035471	Area of Δ	0.000678
$\tau_{max} - \tau_{min} = 2A$	0.071355	Damping ratio, D (%)	23.05
γ_{max}	0.038015		
γ_{min}	-0.037944		
$\gamma_{max} - \gamma_{min} = 2B$	0.075959		
G (MPa)	0.939		

4.4.3 Variation of Shear Modulus and Damping Ratio with Strain

For different shear strain ranges, the variation of shear modulus with shear stain has been estimated and plotted for both natural and treated clay soil. Shear strain ranges from 0.01 to 1% for a number of cycles of 1 to 100, and applied axial stress ranges from 50 to 200 kPa: natural soil's shear modulus and damping ratio are nearly 0.6 to 11 MPa and 10 to 20%, respectively. For 1.5- 2% addition of ferric chloride, the shear modulus and damping ratio of treated soil vary in the range of 0.9 to 8 MPa and 15 to 25 % respectively.

Typical diagrams of variation of shear modulus with the shear strain for axial stress of 100 kPa of different shear strain values are shown in Figure.4.10 and 4.11.

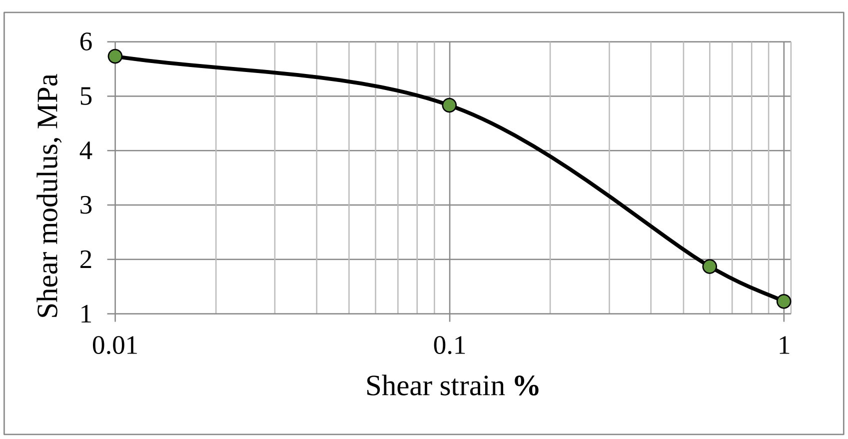

Figure-4- 10: Variation of shear modulus with the shear strain of natural (untreated) soil for axial stress 100 kPa and cycle number 10.

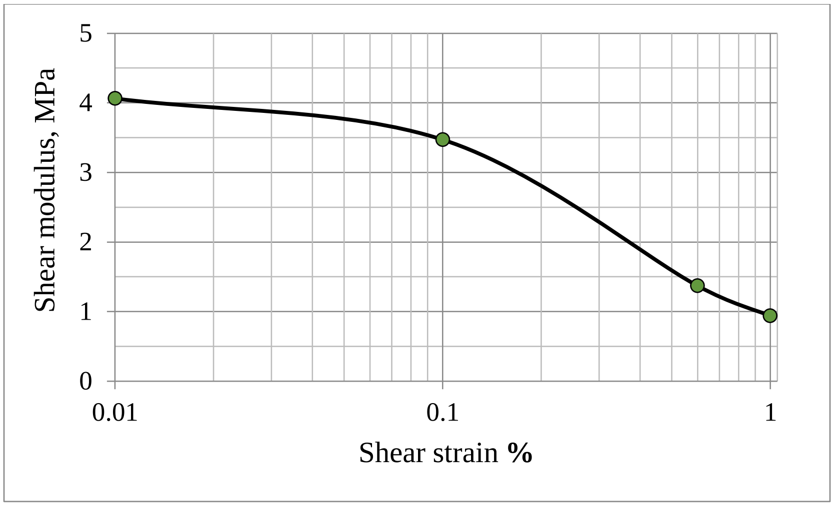

Figure-4- 11: Variation of shear modulus with the shear strain of treated soil for axial stress 100 kPa and cycle number 10.

The shear modulus decreases as the shear strain values increase. Therefore according to Figure 4.10 and 4.11; for natural soil shear modulus of decreases nearly from 8010 kPa to 3700 kPa and treated soil; it decreases around 5400 kPa to 1740 kPa.

From Figure 4.10 and 4.11, the variation of the shear modulus with the shear strain of the natural soils shows decrement with increasing shear strain. Again the shear modulus of treated soil decreases as shear strain increases. However, as compared to the untreated soil the shear modulus of treated soil was smaller; it could be due to the direct impact of treatment on the soil plasticity. I.e. plasticity of natural soil is greater than treated soil. Therefore when the plasticity of the soil is lesser the shear modulus corresponding to that plasticity becomes smaller.

The variation of damping ratio with shear strain for both natural and treated clay soil has been plotted for different shear strain ranges. Typical diagrams for axial stress of 100 kPa of different strains are shown in Figures 4.12 and 4.13.

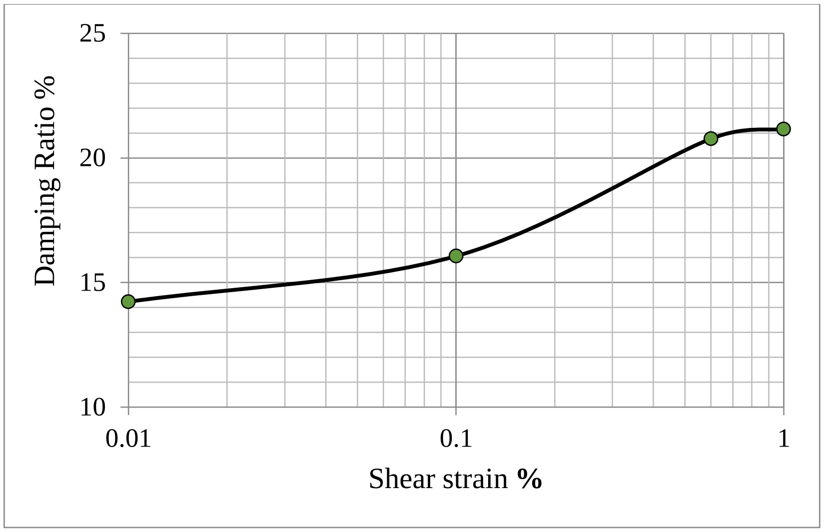

Figure-4- 12: Variation of damping ratio with the shear strain of natural (untreated) soil for cycle number 10 and axial stress 100 kPa.

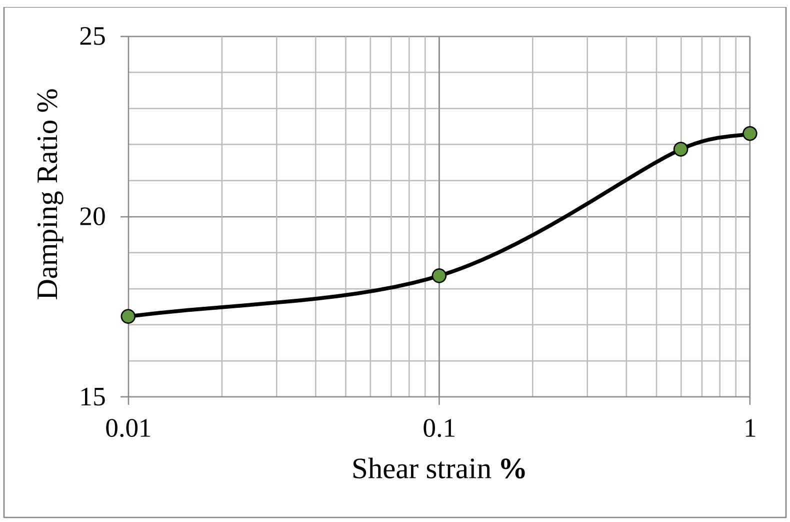

Figure-4- 13: Variation of damping ratio with the shear strain of treated soil for cycle number 10 and axial stress 100 kPa.

The variation of damping ratio with strain for both soils shows increment with increasing shear strain, as seen in Figures 4.12 and 4.13. As the shear strain increase from 0.01% to 1 %, the damping ratio also increases in the range of 15 to 20 % after stabilization. Hence the strain range of the cyclic simple shear test is limited to $10^{\wedge -2}$ to 5 %, it can consider smaller strain below 0.01%.

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

The variation of shear modulus and damping ratio with the number of cycles of stabilized clay soil has been plotted for a different strain of 100 kPa axial stress shown in Figures 4.14 and 4.15.

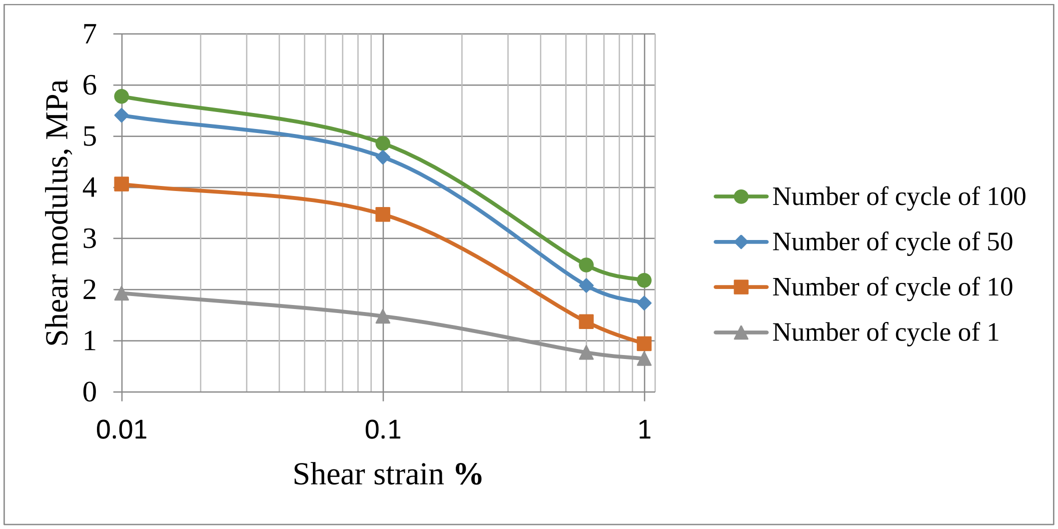

Figure-4- 14: Variation of shear modulus with the number of cycles for stabilized clay soil.

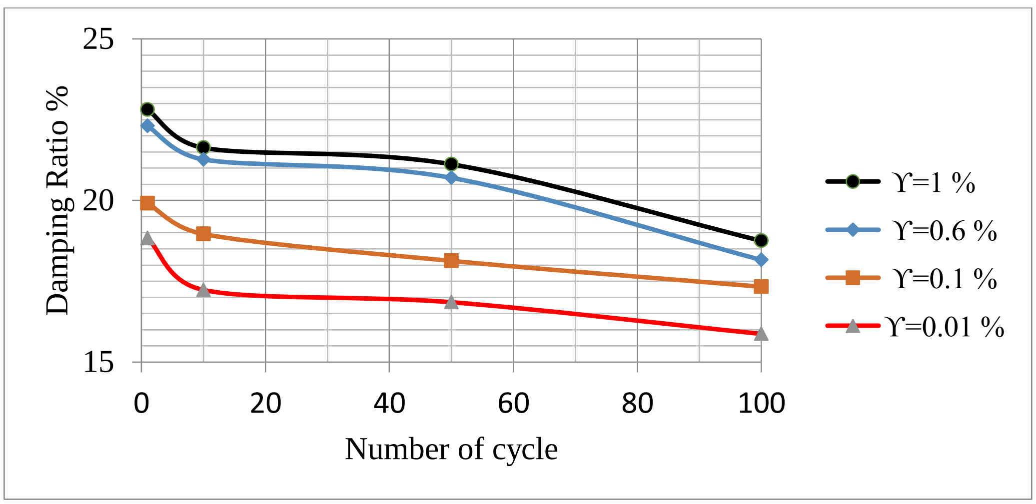

Figure-4- 15: Variation of damping ratio with the number of cycles stabilized clay soil.

4.4.5 Estimation of Maximum Shear Modulus and Normalized Shear Modulus

The cyclic simple shear test has a strain range of 10^$^{-2}$ to 5 % therefore it cannot measure shear modulus and damping at a very low strain level. Maximum shear modulus is computed from a very small strain range of amplitude in elastic range (typically lower than γc of 10^{-4}%) either from direct field tests or from strain method using laboratory testing and also empirical equations.

Hence maximum shear modulus is the peak value of secant shear modulus it cannot be measured from the cyclic simple shear test. (Liu Hong-shuai and et.al. 2008)

Empirical Equation 4.9 suggested by Hardin (1972) was used for estimation of maximum shear modulus.

$$Gmax = 14760 * \frac{(2.973 - e)^2}{1 + e} * OCR^a * \sigma'_m{}^{0.5} \tag{4.8}$$

Where G_{max} - is the maximum shear modulus in psf

e - Void ratio,

OCR- over consolidation ratio

a - an exponent of over-consolidation ratio depends on the plasticity of soil

σ'_m-the mean principal effective stress in psf

At rest condition $\sigma'_2 = \sigma'_3$, with $K_o = 0.5$ for cohesive soil then $\sigma'_m = \frac{2}{3} * \sigma'_1$

Table-4-8: Value of "a" with respect to plasticity index (Hardin, 1972).

PI	**a**
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

The over-consolidation ratio can be calculated using Equation 4.9 and the OCR values used in the document were in Table 4.9 below.

$$OCR = \frac{\sigma'c}{\sigma'p} \tag{4.9}$$

Where σ'_c = Pre-consolidation pressure of specimen

σ'_p = present effective vertical stress

Table 4. 9: Over consolidation ratio values of tested soil.

Pre-consolidation pressure, σ'$_c$	**Effective vertical stress, σ'$_p$**		**OCR= (σ'$_c$)/(σ'$_p$)**	
	Untreated soil	**Treated soil**	**Untreated soil**	**Treated soil**
200 kPa	26.1 kPa	26.7 kPa	7.66	7.49
200 kPa	26.1 kPa	26.7 kPa	7.66	7.49
200 kPa	26.1 kPa	26.7 kPa	7.66	7.49

From Table 4.8, the value of 'a' for a PI value 0-60 falls in the range of 0-0.41. Therefore linear interpolation is used to get the value of a. By using equation 4.8 the values of G_{max} can be calculated and presented in table 4.10.

Table-4-10: Summarized values of maximum shear modulus (G_{max}).

Parameters	**Untreated soil**			**Treated soil**		
	Axial Stress (kPa)			Axial Stress (kPa)		
	50	100	200	50	100	200
Void Ratio (e)	1.08	1.08	1.08	0.97	0.97	0.97
PI (%)	54	54	54	20	20	20
a	0.377	0.377	0.377	0.18	0.18	0.18
(OCR)	7.66	7.66	7.66	7.49	7.49	7.49
σ'$_m$ (lb/ft^2)	696.33	1392.67	2785.33	696.33	1392.67	2785.33
G_{max}(lb/ft^2)	763722.2	1080070.2	1527447.2	569140.7	804889.4	1138283.5
G_{max}(kPa)	36567.2	51714.04	73134.6	27250.6	38538.3	54501.3

For each specimen typical calculation of normalized shear modulus (G/G_{max}) was computed for both natural and treated soil and tabulated in Table 4.11 and 4.12.

Table-4-11: Typical calculation of normalized shear modulus (G/Gmax) for natural soil.

Effective vertical stress	**Strain, %**	**0.01**	**0.1**	**0.6**	**1**
	Cycle number	**G/Gmax**			
50 kPa	1	0.060	0.046	0.019	0.018
	10	0.112	0.085	0.032	0.024
	50	0.146	0.113	0.048	0.034
	100	0.160	0.127	0.064	0.057
100 kPa	1	0.045	0.040	0.016	0.013
	10	0.111	0.093	0.042	0.024
	50	0.155	0.138	0.081	0.072
	100	0.161	0.149	0.090	0.080
200 kPa	1	0.044	0.040	0.013	0.011
	10	0.084	0.073	0.035	0.023
	50	0.142	0.120	0.071	0.044
	100	0.147	0.125	0.078	0.052

Table-4- 12: Typical calculation of normalized shear modulus (G/Gmax) for treated soil.

Effective vertical stress	**Strain, %**	**0.01**	**0.1**	**0.6**	**1**
	Cycle number	**G/Gmax**			
50 kPa	1	0.067	0.047	0.026	0.022
	10	0.109	0.085	0.036	0.031
	50	0.121	0.109	0.043	0.036
	100	0.145	0.120	0.066	0.053
	1	0.050	0.038	0.020	0.017
	10	0.105	0.082	0.036	0.024

100 kPa	50	0.140	0.119	0.054	0.045
	100	0.150	0.126	0.064	0.057
200 kPa	1	0.041	0.026	0.015	0.014
	10	0.111	0.094	0.027	0.016
	50	0.141	0.109	0.069	0.057
	100	0.146	0.119	0.075	0.066

4.4.6 Effect of Ferric Chloride on Shear Modulus and Damping Ratio

Ferric chloride improves the static property of the soil. It has also some effects on the dynamic properties. As the percentage of chloride compound increases plasticity index values decrease from 54 to 20. The effect of improvement of the dynamic property due to ferric chloride is shown in Figures 4.16 and 4.17 for axial stress of 100 kPa and the number of cycles of 10.

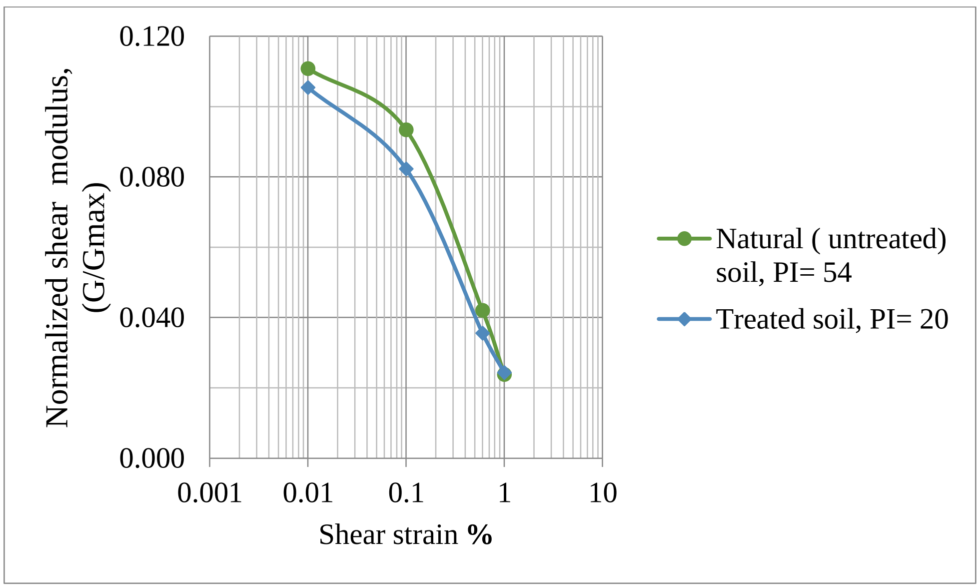

Figure-4- 16: Effect of Ferric Chloride on the variation of normalized shear modulus with shear strain.

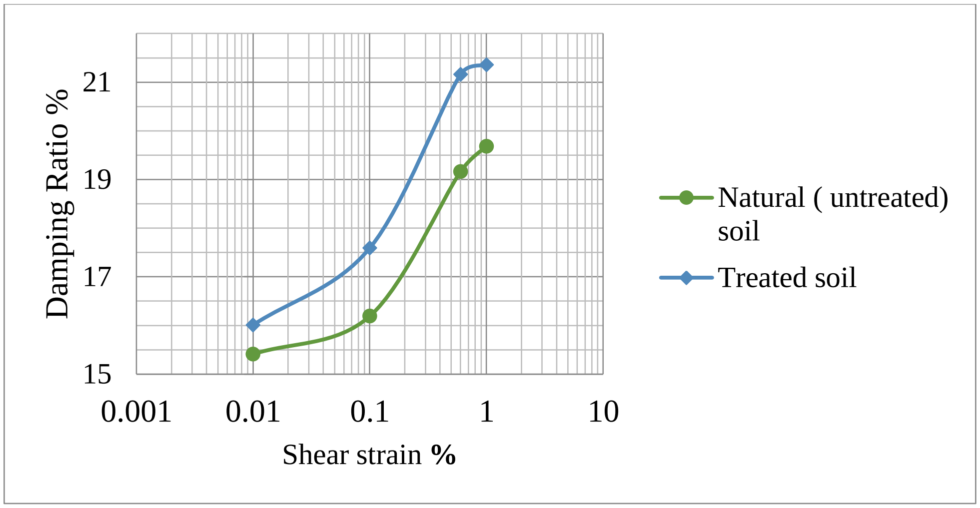

Figure-4- 17: Effect of Ferric Chloride on the variation of damping ratio with shear strain.

Figure 4.16 illustrates the normalized shear modulus of treated soil and untreated soil. When the shear strain range is in between 0.01- 0.1 % there becomes a little decrement due to treatment, but after 0.6 % shear strain the shear modulus is nearly the same this shows degradation is not changed because of improvement. On the other hand in the case of the damping ratio shown in Figure 4.17 the treated soil has a better damping ratio than the natural soil.

4.5 Determination of Resilient Modulus

The resilient modulus Mr, defined as the unloading modulus of the hysteresis loop after many cycles of repeating loads, is a stiffness measurement of pavement layers. (Sas et al., 2017) Resilient modulus can be computed directly from repeated loading triaxial test or indirectly from soil index properties and numerical models like the k-*q* model called the "Uzan-Witczak model".(Sas et al., 2017)

In this case, it is difficult to conduct a dynamic triaxial test due to the unavailability of the testing machine. Therefore the model of unconfined compressive strength and soil index properties used for the prediction of resilient modulus of fine-grained soils. (Hossain & Kim, 2013)

$$M_R = 7884.2 + 99.7 * qu + 193.1 * PI - 47.9 * P_{200} \qquad (4.10)$$

Where model parameters: M_R = resilient modulus (psi)

qu = ultimate compressive strength (psi)

PI = plasticity index (non-plastic soil, PI = 0)

P_{200} = % passing No. 200 sieve

Using equation 4.10 resilient modulus for both natural and stabilized clay soil was computed and shown in Table 4.13 below.

Table-4-13: Measured resilient modulus of treated and untreated soil.

Model parameters	**Natural soil**	**Treated soil**	
		Without curing	7 days curing
Ultimate compressive strength (psi)	16.6793	31.3282	42.351
Plasticity index,%	54	20	20
% passing No. 200 sieve	95.43	95.43	95.43
Resilient modulus, M_R (psi)	9605.67	11000.5	12099.5
Resilient modulus, M_R (MPa)	66.3	75.9	83.5

4.6 Comparison of Test Results with Previous Findings

Reduction modulus and damping ratio curves were come about by researchers for different soil types. Experimental results presented in chapter four were compared with previous publication data. Vucetic & Dobry (1991) developed reduction modulus and damping ratio curves for both normally consolidated and over consolidated clay soil. Quezon & Takele (2017) and Teshome (2019) performed the cyclic simple shear test and developed modulus reduction and damping ratio curves for local soil.

4.6.1 Comparison of Shear Modulus Reduction with Previous Studies

The computed normalized shear modulus (G/Gmax) for an axial stress of 100 kPa and 200 kPa and number of cycles of 10 from Table 4.12 are plotted against shear strain. The plotted curves were compared with the one developed by Vucetic and Dobry (1991), Quezon & Takele (2017), and Teshome (2019) as shown in Figures 4.18, 4.19, and 4.20 below respectively.

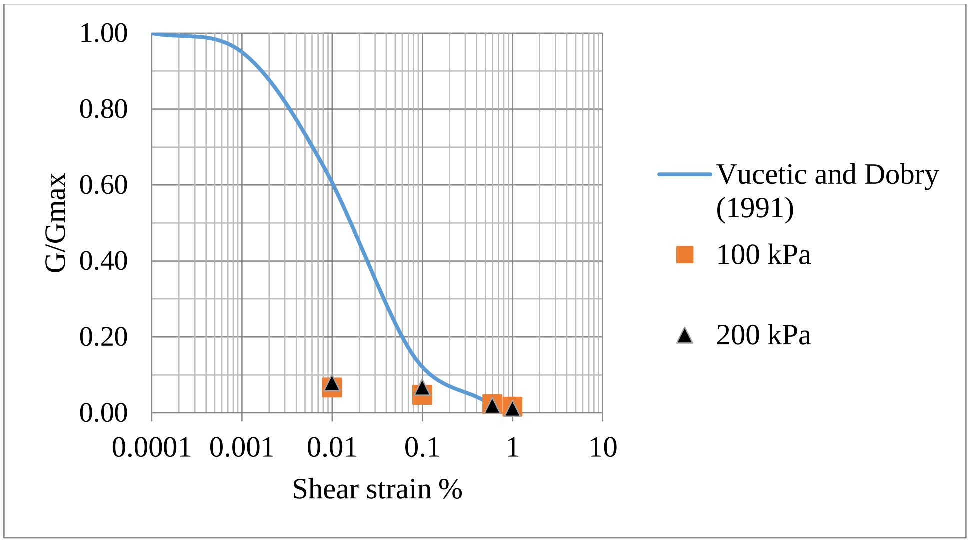

Figure-4- 18: Comparison of modulus reduction values of stabilized clay soil with curves developed for plastic soils by Vucetic and Dobry (1991).

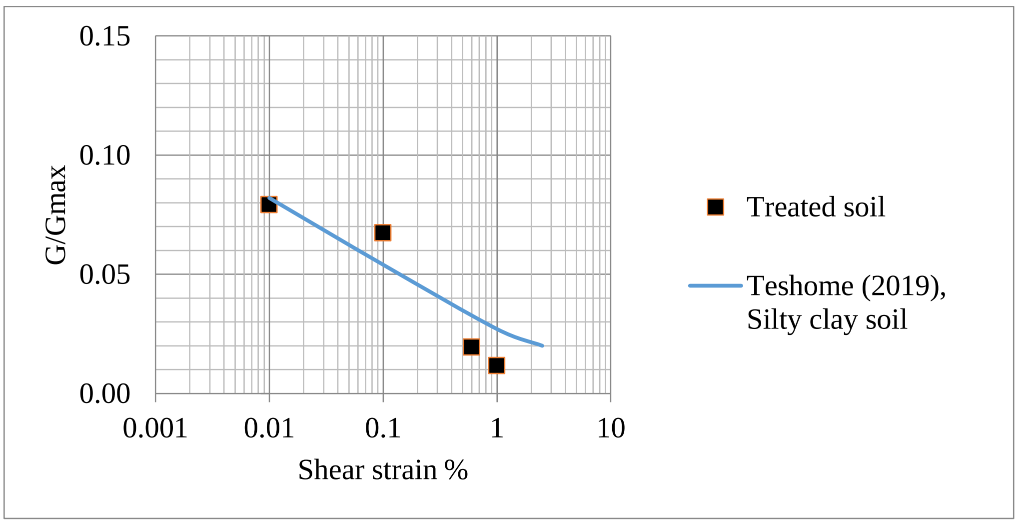

Figure-4- 19: Comparison of modulus reduction values of stabilized clay soil with curves developed for CH silty clay soil by Teshome (2019).

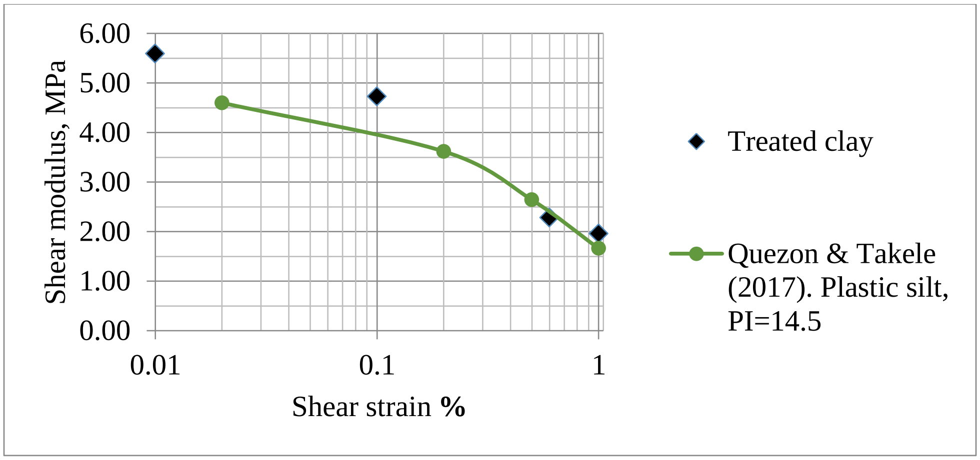

Figure-4- 20: Comparison of shear modulus values of stabilized clay soil with curves developed for plastic silt soil by Quezon & Takele (2017).

Figure 4.18 indicates that for larger shear strains, the normalized shear modulus coincides well with Vucetic and Dobry (1991) while the G/Gmax varies for shear strains smaller than 0.1 %. It's possible that the discrepancy is attributable to the sample preparation and testing machine utilized to determine dynamic properties. Figures 4.19 and 4.20 show that the normalized shear modulus and shear modulus of treated soil is compatible with local plastic silt soils in Addis Ababa, as well as silty clay soil curves established by Quezon & Takele (2017) and Teshome (2019).

4.6.2 Comparison of Damping Ratio with Previous Studies

Comparison of damping ratio was made with local soil curve developed by Quezon & Takele (2017) in Figure 4.21 and with kinburu clay (sensitive Leda clay around Ottawa – Quebec area) by Thirugnansampanther (2016) in Figure 4.22.

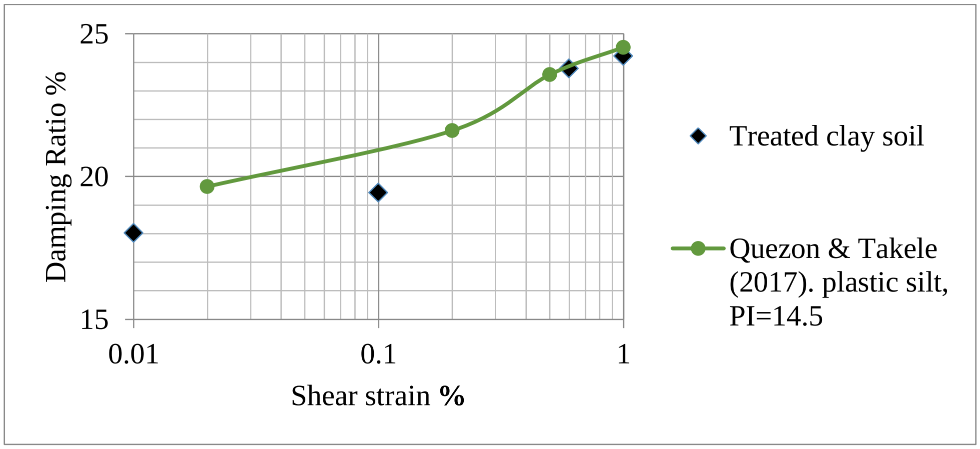

Figure-4- 21: Comparison of damping ratio values of treated soil (PI=20) with curves developed for plastic silt (PI=14.5) by Quezon & Takele (2017).

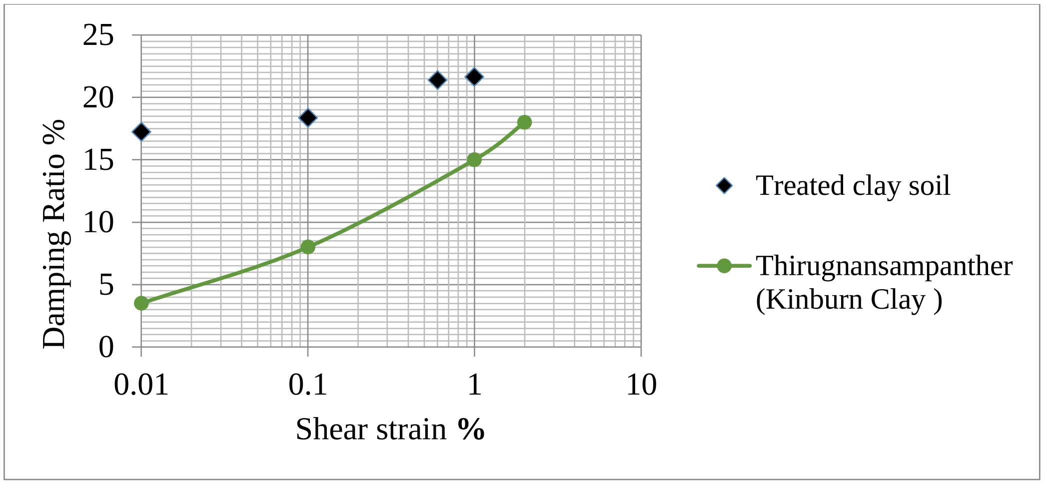

Figure-4- 22: Comparison of damping ratio values of treated soil (PI=20) with curves developed for Kinburu clay (PI=20) by Thirugnansampanther (2016).

Comparison between Quezon & Takele (2017)'s local plastic silt soil with the treated soil has shown in Figure 4.21 above. The damping ratio of these soils is in good agreement as to the shear strain increases. In Figure 4.22 both treated and kinburn clay soil have the same plasticity index, but the treated soil shows a higher damping ratio. This demonstrates that, even if the soil has the same plasticity index, treatment has a considerable impact on the soil's damping ratio.

4.6.3 Comparison of Resilient Modulus with Predicted Resilient Modulus

The predicted values of resilient modulus of natural and ferric chloride treated soil are compared with predicted values of LA347: C and US171: B subgrade soil types gained from the DCP direct model. Where, LA347: C is heavy clay subgrade soil and US171: B is lime treated subgrade soil (Mohammad et al., 2007).

The predicted resilient modulus of natural soil used in this document has 66.3 MPa and LA347: C has a predicted M_R value of 50.05 MPa. Hence 66.3 MPa is greater than 50.5 MPa, both are in the range of resilient modulus of CH or A-7-5 clay soil.

Comparison between US171: B having a predicted M_R of 46.33 MPa and ferric chloride treated soil with M_R of 75.3 MPa. From these observations stabilization using chloride is an effective alternative treatment technique to conventional lime/ cement treatment.

In general, the resilient modulus values were increased with an increase in density, treatment content, and curing period. However, M_R decreases with an increase in the water content of the soil.

Chapter Five
Conclusions and Recommendations

5.1 Conclusions

A study was undertaken to investigate the effect of ferric chloride on the dynamic properties of expansive soil. The following conclusions had been drawn.

1. The stabilization of the expansive soil treated with the chemical like ferric chloride gave a good improvement in the strength and decrease in the swelling property for both static and dynamic properties.
2. The plasticity index of the soil decreased as the amount of ferric chloride in the soil increased, up to 2% ferric chloride.
3. Free swell and swelling also showed decreasing with increasing addition of ferric chloride up to 1 % by replacing the interlayer of metallic cations.
4. OMC values decrease while MDD showed a slight increment. UCS of the treated soil decreased with increasing ferric chloride. This revealed that ferric chloride does not exhibit any pozzolanic property. Therefore treating expansive soil using ferric chloride is promising and one of the good alternative ways to stabilize and make a soil workable and good for construction.
5. The addition of ferric chloride reduces the plasticity of the soil. The shear modulus and damping ratio are affected by the decrease in plasticity. For the number of cycles 1 to 100, shear strain ranges from 0.01 to 1 % and axial stress 50 to 200 kPa, the shear modulus of treated soil decreases approximately 1.2 times that of natural soil, and damping ratio values increase around 1.4 times that of natural soil.
6. The addition of ferric chloride improves the soil's resilient modulus as well. On average, the soil's resilient modulus increases nearly by 14-25 %.
7. The dynamic properties of ferric chloride-treated expansive soil improved more than those of lime/cement-treated expansive soil. It's because ferric chloride is sparingly soluble in water and provides sufficient cations for the exchange reaction, as well as its efficiency in reducing swelling in high plastic soils.

8. By observing the overall effects of ferric chloride both in static and dynamic properties soil as well as on resilient modulus we can use it as an effective alternative treatment technique to conventional lime/ cement treatment.

5.2 Recommendations for Future work

I. Dynamic properties of stabilized expansive soil for local soil conditions with various stabilization techniques would be computed for repeated loading.

II. Estimating the resilient modulus of subgrade soil from the cyclic triaxial test will give a better result and good to know the in-situ behavior directly rather than computing from an indirect method like index properties.

III. Cyclic simple shear test equipment has its limitation on the simulated frequency of repeated traffic loading it's better to use cyclic triaxial test equipment.

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Appendix: A

A: 1- Grain size Analysis: this test consists **of** wet sieve analysis and hydrometer analysis.

Table A.1 Shows gran size variation with percentage passing.

Table- A- 1: Gran size variation with percentage passing.

Sieve Openings (mm)	Percentage Passing (%)
4.750	100.00
2.360	99.83
2.000	99.61
1.180	99.09
0.425	97.87
0.150	96.78
0.075	95.43
0.052	93.17
0.037	91.65
0.026	90.14
0.019	87.12
0.014	84.09
0.010	79.55
0.005	68.97
0.004	59.89
0.003	52.33
0.002	43.26
0.001	40.23

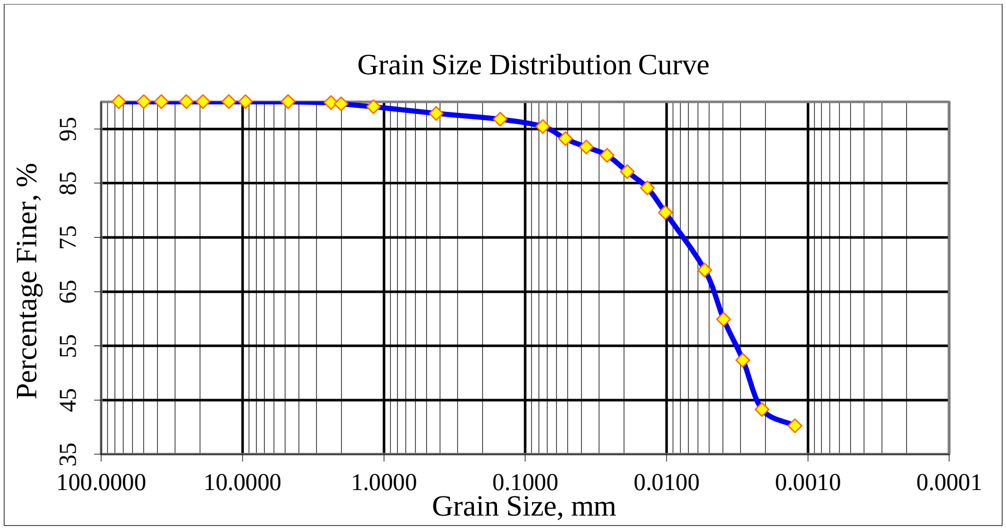

Figure-A- 1: Grain size distribution curve.

A: 2- Atterberg Limit Test Results

Table- A- 2: Summary of test results for Atterberg limits.

No.	Sample Type	Consistency limits with no curing (%)			Consistency limits 7- Day curing (%)		
		Liquid Limit	Plastic Limit	Plasticity Index	Liquid Limit	Plastic Limit	Plasticity Index
1	ES	94	40	54	94	40	54
2	ES+0.5 %	80	49	31	80	49	31
3	ES+1%	79	50	29	78	52	26
4	ES+ 1.5%	72	52	20	71	52	19
5	ES+2%	71	53	18	72	50	22

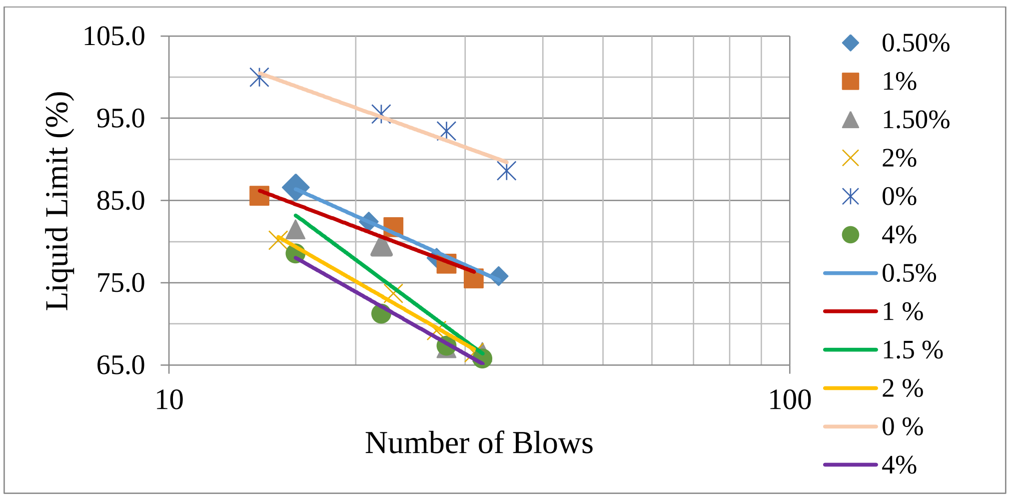

Figure-A- 2: Liquid limit values of natural soil and natural soil with addition of FeCl3 (No curing).

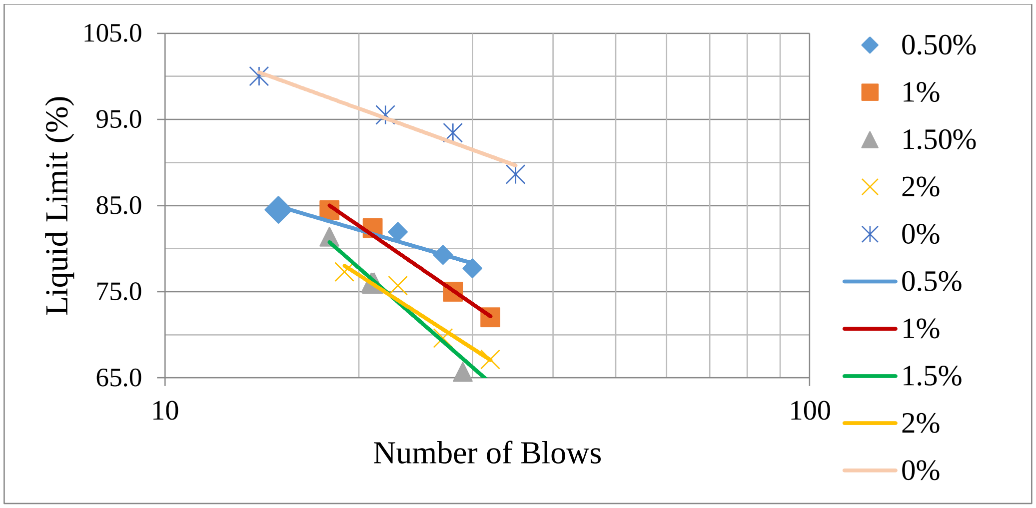

Figure-A- 3: Liquid limit values of natural soil and natural soil with addition of FeCl3 (7 days curing).

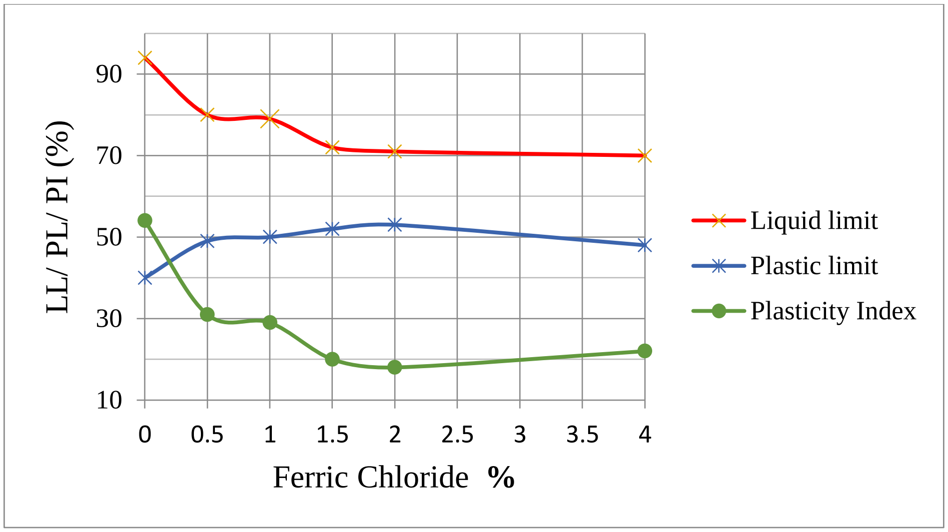

Figure-A- 4: Summarized values of variation of Atterberg limits with FeCl3 percentage (no curing).

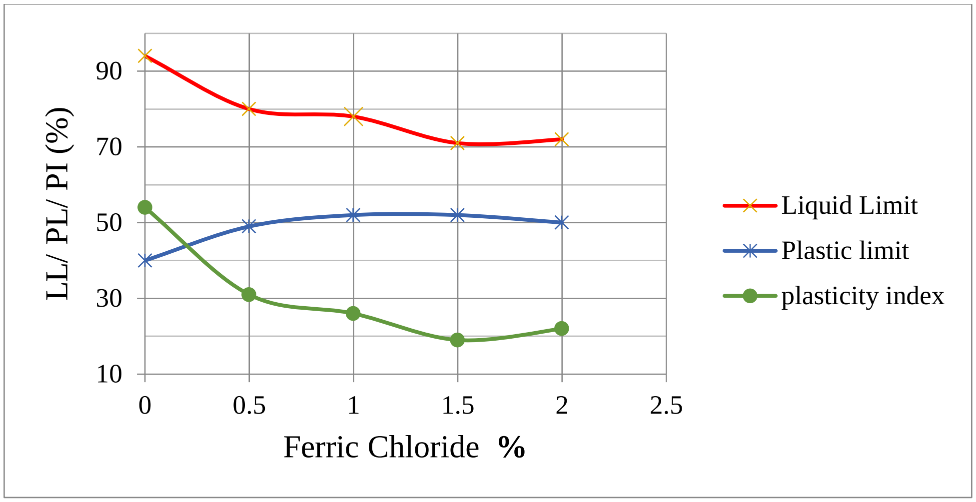

Figure-A- 5: Summarized values of variation of Atterberg limits with FeCl3 percentage (7 days curing).

A: 2- Summary of Values for Strength tests

Table- A- 3: Summarized of Values for Strength tests.

No.	Sample Type	Moisture Density Relations		UCS (KPa)	
		OMC (%)	MDD (g/cc)	Without curing	7-days curing
1	ES	34.75	1.29	85	85
2	ES + 0.5%	32.5	1.31	112	105
3	ES + 1%	31.5	1.33	143	179
4	ES + 1.5%	25	1.42	189	220
5	ES + 2%	27.5	1.41	170	213

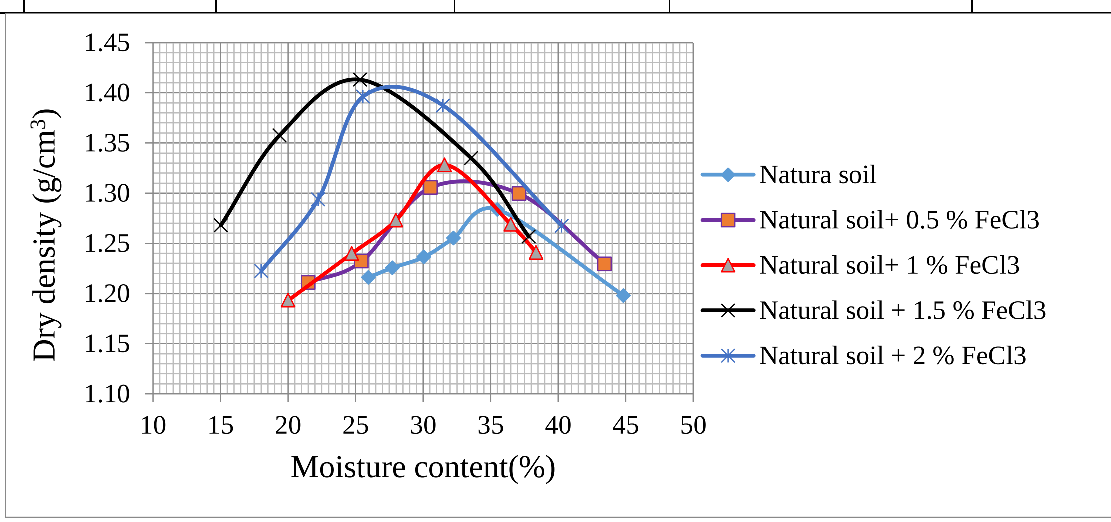

Figure-A- 6: Summarized values of Moisture Density Relations with no curing.

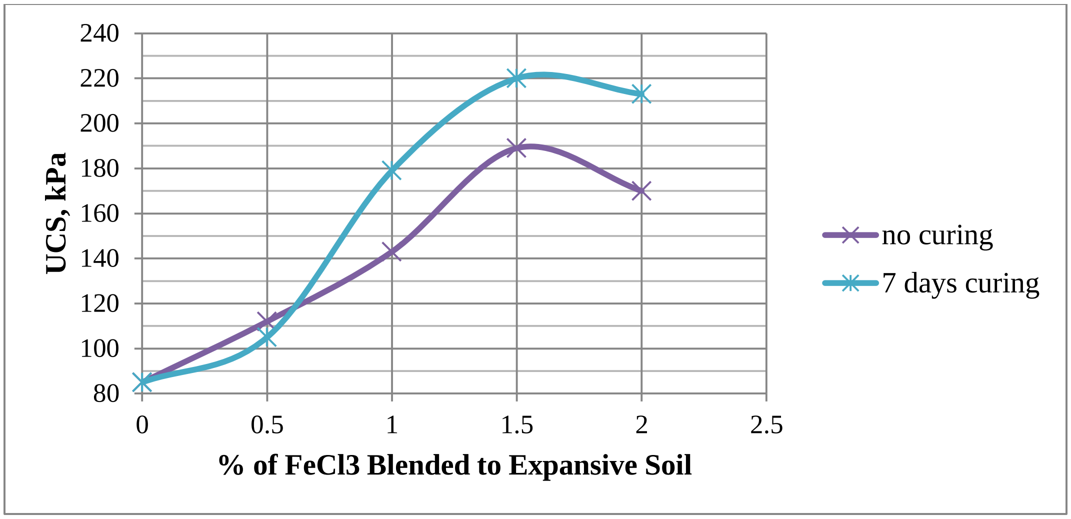

Figure-A- 7: Summarized values of variation of unconfined compressive strength with different percentages of FeCl3.

Appendix: B

Consolidation Test Results

B: 1 – Swell Pressure Test Results

Table-B-1: Swell pressure test result of natural (untreated) soil.

[A] At the beginning of the test	
Sample type	Remolded
Ring Area,cm^2:	19.63
Height of sample, mm:	20
Seating Load, kPa	7
Initial Void Ratio, e_o:	1.08
Initial moisture content,%	18.43
Specific Gravity:	2.73
Wet density, g/cm3	1.73
[B] At the end of the test	
Final Moisture Content,%	20.66
Dry specimen weight (m_s), gm	51.51
Dry density, g/cm3	1.73
Height of Solids(H_s), mm	9.61
Final Void Ratio, e_f:	0.99

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, e
Loading					
7	10.000	0.00	20.00	10.39	1.08
7	13.046	3.05	23.05	13.43	1.40
50	12.132	2.13	22.13	12.52	1.30
100	11.346	1.35	21.35	11.73	1.22
200	10.482	0.48	20.48	10.87	1.13
400	9.142	-0.86	19.14	9.53	0.99
Swelling pressure = 280 kPa					

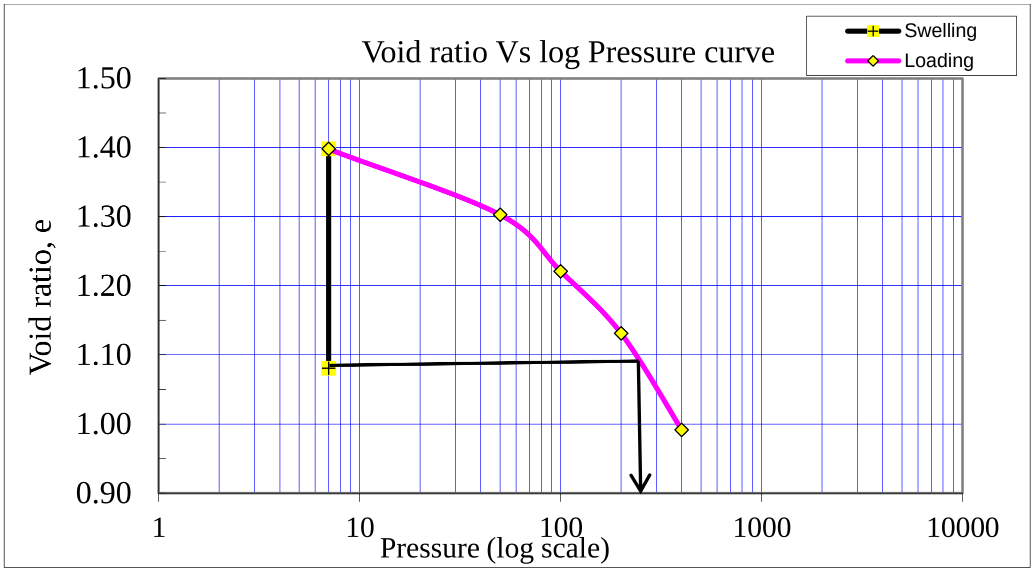

Figure-B-1: Consolidation pressure versus the void ratio of Natural soil.

Table- B-2: Swell pressure test result of natural Soil + 1% $FeCl_3$.

[A] At the beginning of the test	
Sample type :	Remolded
Ring Area,cm^2:	19.63
Height of sample, mm:	20
Seating Load, KPa	7
Initial Void Ratio, e_o:	1.13
Initial moisture content,%	17.62
Specific Gravity:	2.73
Wet density, g/cm3	1.73
[B] At the end of the test	
Final Moisture Content,%	19.34
Dry specimen weight (m_s), gm	53.27
Dry density, g/cm3	1.73
Height of Solids(H_s), mm	9.94
Final Void Ratio, e_f:	0.95

[C]Calculation table:					
Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, e
Loading					
7	2.020	0.00	20.00	10.06	1.01
7	3.220	1.20	21.20	11.26	1.13
50	2.654	0.63	20.63	10.69	1.08
100	2.084	0.06	20.06	10.12	1.02
200	1.420	-0.60	19.40	9.46	0.95
Swelling pressure = 110 kPa					

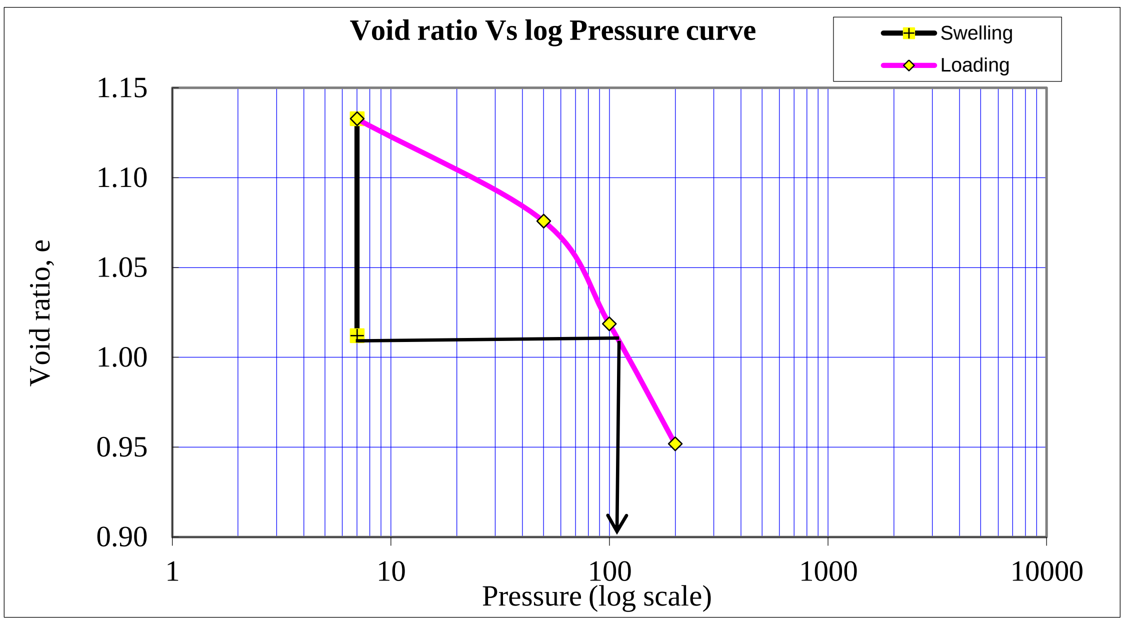

Figure-B-2: Consolidation pressure versus void ratio of Natural soil + 1% $FeCl_3$.

Table-B-3: Swell pressure test result of natural Soil + 2 % $FeCl_3$.

[A] At the beginning of the test		
Sample type :		Remolded
Ring Area,cm^2:		19.63
Height of sample, mm:		20
Seating Load, KPa		7
Initial Void Ratio, e_o:		1.05

Initial moisture content,%	15.93
Specific Gravity:	2.73
Wet density, g/cm3	1.73
[B] At the end of the test	
Final Moisture Content,%	19.67
Dry specimen weight (m_s), gm	55.17
Dry density, g/cm3	1.73
Height of Solids(H_s), mm	10.29
Final Void Ratio, e_f:	0.87

[C]Calculation table:

Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, E
Loading					
7	10.000	0.00	20.00	9.71	0.94
7	11.132	1.13	21.13	10.84	1.05
50	10.212	0.21	20.21	9.92	0.96
100	9.246	-0.75	19.25	8.95	0.87
Swelling pressure = 70 kPa					

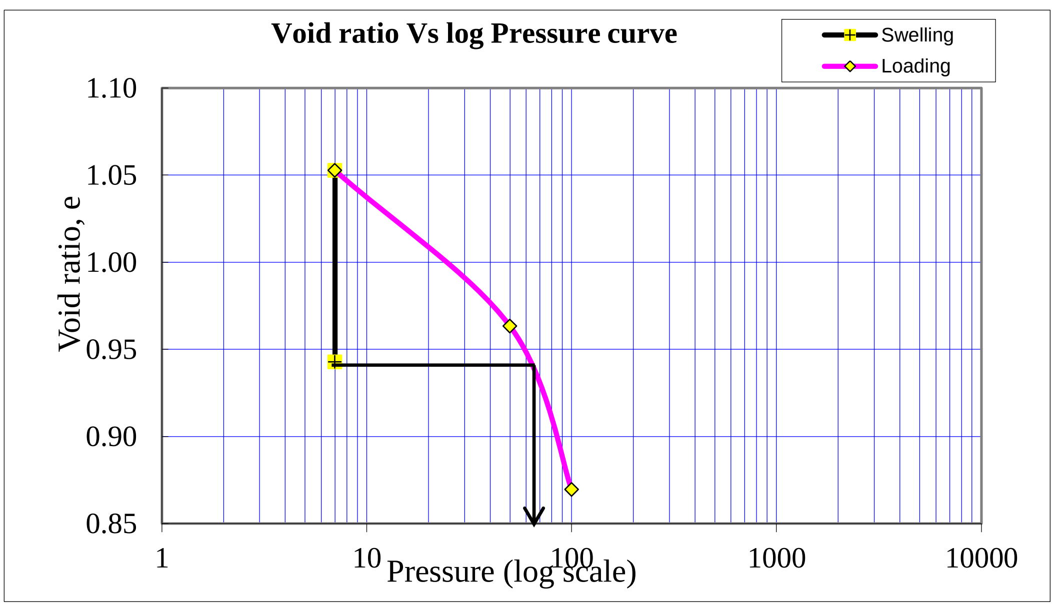

Figure-B-3: Consolidation pressure versus void ratio of Natural soil + 2 % FeCl3.

B: 2- Determination of pre-consolidation Pressure

Table-B-4: Determination of Pre-consolidation pressure.

[A] At the beginning of the test		
Sample type :		Remolded
Ring Area,cm^2:		19.63
Height of sample, mm:		20
Seating Load, KPa		7
Initial Void Ratio, e_o:		1.08
Initial moisture content,%		18.43
Specific Gravity:		2.73
Wet density, g/cm3		1.73
[B] At the end of the test		
Final Moisture Content,%		20.68
Dry specimen weight (m_s), gm		51.51
Dry density, g/cm3		1.73
Height of Solids(H_s), mm		9.61
Final Void Ratio, e_f:		0.65

[C]Calculation table:					
Applied pressure P (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height (mm)	Void Height,H_v (mm)	Void Ratio, e
Loading					
7	6.654	0.00	20.00	10.39	1.08
7	6.668	0.01	20.01	10.40	1.08
50	6.414	-0.24	19.76	10.15	1.06
100	6.180	-0.47	19.53	9.91	1.03
200	5.684	-0.97	19.03	9.42	0.98
400	4.794	-1.86	18.14	8.53	0.89
800	3.392	-3.26	16.74	7.13	0.74
1600	2.480	-4.17	15.83	6.21	0.65
Unloading					
1600	2.480	-4.17	15.83	6.21	0.65
400	2.862	-3.79	16.21	6.60	0.69
100	3.242	-3.41	16.59	6.98	0.73

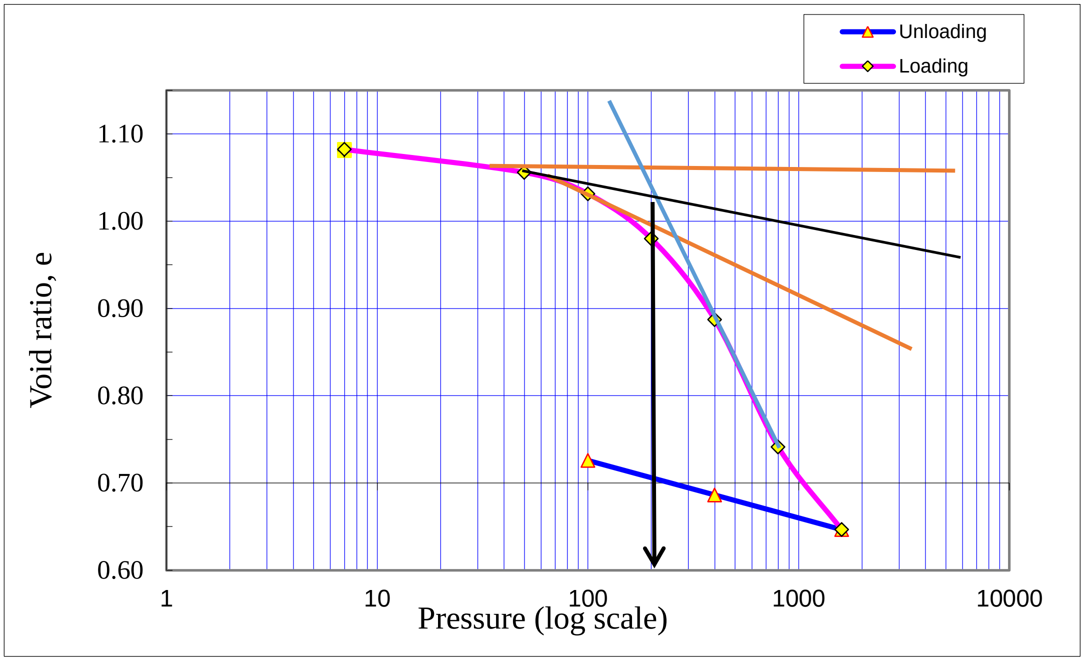

Figure-B-4: Consolidation pressure versus void ratio.

Appendix: C

Cyclic Shear Test Result

C: 1-Typical Tabulation of Shear Stress and Shear Strain Values of Selected Strain for Untreated and Treated Soil

Table-C-1: Shear stress and shear strain values of natural (untreated) soil with 1 % strain a 100 kPa, 10th cycle.

Cycle No.	Time (Sec)	Lateral Lvdt	Lateral Force	γ= Disp/19.55	τ= (Shear force/3845.5) *10^3 (Mpa)	$(\tau_i-\tau_{i+1})$ $*(\gamma_i+\gamma_{i+1})$
100	0	0.1304	0.0051	0.006619	0.001325	-2.2E-05
	0.010	0.17093	0.0107	0.008677	0.00278	-1E-05
	0.019	0.19183	0.01283	0.009738	0.003334	-4.8E-05
	0.029	0.24249	0.02128	0.012309	0.005529	-8.8E-05
	0.038	0.34884	0.0326	0.017708	0.008471	-0.00012
	0.048	0.41616	0.04466	0.021125	0.011605	-8.5E-05
	0.057	0.47638	0.05185	0.024182	0.013473	-0.00023
	0.067	0.56825	0.06867	0.028845	0.017843	-0.00014
	0.076	0.60084	0.07763	0.030499	0.020171	-0.00014
	0.086	0.64924	0.08584	0.032956	0.022305	-0.00025
	0.095	0.69507	0.0998	0.035283	0.025932	-0.00034
	0.105	0.70933	0.1183	0.036007	0.030739	-0.00015
	0.114	0.71425	0.12629	0.036256	0.032815	-0.00017
	0.124	0.72666	0.13533	0.036886	0.035164	-0.00016
	0.133	0.7318	0.14372	0.037147	0.037344	-0.00028
	0.143	0.7232	0.15811	0.036711	0.041084	-0.00016
	0.152	0.7064	0.1665	0.035858	0.043264	-9.3E-05
	0.162	0.6996	0.1715	0.035513	0.044563	5.61E-05
	0.171	0.6814	0.16842	0.034589	0.043763	8.76E-05
	0.181	0.6418	0.1634	0.032579	0.042458	0.00018
	0.190	0.58731	0.15231	0.029813	0.039576	0.000248
	0.200	0.4801	0.13472	0.024371	0.035006	0.000158

	0.209	0.4064	0.1212	0.020629	0.031493	0.000118
	0.219	0.3054	0.10865	0.015503	0.028234	0.000178
	0.228	0.1689	0.08027	0.008574	0.020857	7.11E-05
	0.238	0.0625	0.05698	0.003173	0.014806	-2.2E-05
	0.247	-0.1051	0.01855	-0.00534	0.00482	-4.2E-05
	0.257	-0.1685	0.00704	-0.00855	0.001829	-6.7E-05
	0.266	-0.2439	-0.00534	-0.01238	-0.00139	-0.00011
	0.276	-0.3207	-0.02037	-0.01628	-0.00529	-9.8E-05
	0.285	-0.40489	-0.03062	-0.02055	-0.00796	-0.00014
	0.295	-0.47816	-0.04291	-0.02427	-0.01115	-0.00011
	0.304	-0.51803	-0.05148	-0.0263	-0.01338	-0.00013
	0.314	-0.5871	-0.06029	-0.0298	-0.01567	-0.00018
	0.323	-0.62977	-0.0715	-0.03197	-0.01858	-0.0002
	0.333	-0.67687	-0.08296	-0.03436	-0.02156	-0.00029
	0.342	-0.69415	-0.09915	-0.03524	-0.02576	-0.00012
	0.352	-0.70151	-0.1054	-0.03561	-0.02739	-0.00029
	0.361	-0.71995	-0.12093	-0.03655	-0.03142	-0.00019
	0.371	-0.72428	-0.13101	-0.03677	-0.03404	-0.00031
	0.380	-0.72642	-0.14711	-0.03687	-0.03823	-0.00033
	0.390	-0.71628	-0.1647	-0.03636	-0.0428	-0.00025
	0.399	-0.7018	-0.17787	-0.03562	-0.04622	0.000107
	0.409	-0.7081	-0.1721	-0.03594	-0.04472	2.93E-05
	0.418	-0.69033	-0.17051	-0.03504	-0.04431	6.05E-05
	0.428	-0.65805	-0.16711	-0.0334	-0.04342	0.000114
	0.437	-0.62813	-0.16041	-0.03188	-0.04168	3.66E-05
	0.447	-0.5825	-0.15812	-0.02957	-0.04109	0.000151
	0.456	-0.5002	-0.14756	-0.02539	-0.03834	5.33E-05
	0.466	-0.4985	-0.14351	-0.0253	-0.03729	-0.00097

Table-C-2: Shear stress and shear strain values of treated soil with 1 % strain at 100 kPa, 10th cycle.

Cycle No.	Time (sec)	Lateral Lvdt	Lateral Force	Shear strain= Disp/19.7	Shear stress= (lat.force/3848.5)*10^3 Mpa	(τi-τi+1)* (γi+γi+1)
100	0.010	0.10304	0.0055	0.00527	0.00143	-0.00001654
	0.019	0.15093	0.0104	0.00772	0.00270	-0.00002696
	0.029	0.19283	0.0163	0.00986	0.00424	-0.00004036
	0.038	0.24349	0.02326	0.01245	0.00604	-0.00005395
	0.048	0.33884	0.03023	0.01733	0.00786	-0.00010833
	0.057	0.40616	0.04117	0.02078	0.01070	-0.00009009
	0.067	0.47638	0.04885	0.02437	0.01269	-0.00017492
	0.076	0.55825	0.06157	0.02855	0.01600	-0.00015128
	0.086	0.60084	0.07139	0.03073	0.01855	-0.00019009
	0.095	0.65924	0.08274	0.03372	0.02150	-0.00016033
	0.105	0.69307	0.09166	0.03545	0.02382	-0.00017838
	0.114	0.70933	0.10123	0.03628	0.02630	-0.00018284
	0.124	0.71325	0.1109	0.03648	0.02882	-0.00004596
	0.133	0.72766	0.1133	0.03722	0.02944	-0.00025150
	0.143	0.73918	0.1262	0.03781	0.03279	-0.00009654
	0.152	0.7432	0.1311	0.03802	0.03407	-0.00013673
	0.162	0.7264	0.1381	0.03716	0.03588	0.00000038
	0.171	0.69496	0.13808	0.03555	0.03588	0.00000841
	0.181	0.6814	0.13762	0.03485	0.03576	0.00007645
	0.190	0.6318	0.13324	0.03232	0.03462	0.00005480
	0.200	0.57731	0.12983	0.02953	0.03374	0.00012382
	0.209	0.4801	0.12102	0.02456	0.03145	0.00014053
	0.219	0.4084	0.10912	0.02089	0.02835	0.00021394
	0.228	0.3054	0.08657	0.01562	0.02249	0.00012797
	0.238	0.1689	0.06627	0.00864	0.01722	0.00005101
	0.247	0.0725	0.05037	0.00371	0.01309	-0.00001335
	0.257	-0.1031	0.01754	-0.00527	0.00456	-0.00004014
	0.266	-0.168	0.0064	-0.00859	0.00166	-0.00004113

	0.276	-0.2339	-0.0013	-0.01196	-0.00034	-0.00005863
	0.285	-0.3107	-0.0094	-0.01589	-0.00244	-0.00015979
	0.295	-0.4049	-0.0262	-0.02071	-0.00681	-0.00015028
	0.304	-0.4716	-0.0391	-0.02412	-0.01016	-0.00020650
	0.314	-0.518	-0.0548	-0.02650	-0.01424	-0.00022766
	0.323	-0.5871	-0.0703	-0.03003	-0.01827	-0.00018115
	0.333	-0.6298	-0.0815	-0.03221	-0.02118	-0.00026897
	0.342	-0.6758	-0.097	-0.03457	-0.02520	-0.00009195
	0.352	-0.6941	-0.10205	-0.03550	-0.02652	-0.00012038
	0.361	-0.7015	-0.10854	-0.03588	-0.02820	-0.00004515
	0.371	-0.7199	-0.11093	-0.03682	-0.02882	-0.00020096
	0.380	-0.7242	-0.1214	-0.03704	-0.03154	0.00000578
	0.390	-0.7264	-0.1211	-0.03716	-0.03147	-0.00009833
	0.399	-0.7328	-0.12617	-0.03748	-0.03278	-0.00010054
	0.409	-0.7418	-0.1313	-0.03794	-0.03412	-0.00010248
	0.418	-0.7381	-0.13651	-0.03775	-0.03547	0.00008437
	0.428	-0.7179	-0.13215	-0.03672	-0.03434	0.00001933
	0.437	-0.6805	-0.13111	-0.03481	-0.03407	0.00003352
	0.447	-0.64681	-0.12921	-0.03308	-0.03357	0.00001558
	0.456	-0.61482	-0.12828	-0.03145	-0.03333	0.00001956
	0.466	-0.558	-0.12703	-0.02854	-0.03301	0.00001632
	0.476	-0.48608	-0.12585	-0.02486	-0.03270	-0.00090203

C: 2- Shear Modulus and Damping Ratio Values of Selected Cycles

Table-C-3: Shear modulus and damping ratio value of Un-treated soil.

Axial stress, 50 kPa								
Strain (%)	Untreated soil							
	0.01	0.1	0.6	1	0.01	0.1	0.6	1
Cycle no.	G (MPa)				D (%)			
1	2.21	1.67	0.704	0.65	18.83	20.91	22.61	22.81
10	4.08	3.11	1.17	0.875	18.23	19.76	22.04	22.21
50	5.33	4.15	1.74	1.33	17.85	19.16	19.92	22.1
100	5.864	4.65	2.358	2.078	14.47	15.3	19.64	20.32
Axial stress, 100 kPa								
Cycle No.	G (MPa)				D (%)			
1	2.33	2.08	0.84	0.67	15.83	17.91	21.6	21.81
10	5.73	4.83	2.17	1.23	15.23	16.06	20.77	21.16
50	8.01	7.16	4.21	3.7	13.85	15.13	19.91	20.12
100	8.35	7.69	4.68	4.15	12.47	13.93	18.64	19.19
Axial stress, 200 kPa								
Cycle No.	G (MPa)				D (%)			
1	3.25	2.89	0.976	0.78	15.8	18.37	20.5	20.81
10	6.11	5.33	2.528	1.69	15.41	16.19	19.16	19.683
50	10.36	8.81	5.17	3.2	12.2	16.33	18.78	19.1
100	10.74	9.16	5.71	3.83	11.33	12.47	17.1	17.8

Table-C-4: Shear modulus and damping ratio values of treated soil.

Axial stress = 50 kPa								
Strain (%)	0.01	0.1	0.6	1	0.01	0.1	0.6	1
Cycle no.	G (MPa)				D (%)			
1	1.827	1.27	0.716	0.612	19.34	21.1	23.71	24.41
10	2.98	2.31	0.987	0.85	18.43	19.76	22.71	23.05
50	3.305	2.975	1.184	0.973	17.85	18.76	19.2	20.18
100	3.96	3.265	1.8	1.448	16.57	17.97	18.07	18.21

Cycle No.	Axial stress = 100 kPa							
	G (MPa)				D (%)			
1	1.931	1.48	0.77	0.65	18.83	19.91	22.3	22.81
10	4.063	3.171	1.37	0.939	17.23	18.36	21.37	22.63
50	5.41	4.59	2.08	1.736	16.85	18.13	20.7	21.12
100	5.78	4.86	2.48	2.18	15.87	17.33	18.16	18.75
Cycle No.	Axial stress = 200 kPa							
	G (MPa)				D (%)			
1	2.25	1.43	0.836	0.78	17.58	20.22	21.45	21.961
10	4.53	3.63	1.48	0.989	16.01	17.59	21.16	21.353
50	7.71	5.96	3.77	3.1	15.42	18.03	19.78	20.11
100	7.94	6.51	4.1	3.58	14.83	15.47	17.51	17.8

C: 3 Modulus Reduction Values for both Natural and Treated Soil

Table-C-5: Modulus reduction values for natural and treated soil.

Strain (%)	Axial stress (kPa)	Cycle no.	0.01	0.1	0.6	1	G_{max}, (MPa)	0.01	0.1	0.6	1
			G (MPa)					G/G_{max}			
Natural soil	50	1	2.21	1.67	0.704	0.65	36.567	0.060	0.046	0.019	0.018
		10	4.08	3.11	1.17	0.875	36.567	0.112	0.085	0.032	0.024
		50	5.33	4.15	1.74	1.23	36.567	0.146	0.113	0.048	0.034
		100	5.864	4.65	2.358	2.078	36.567	0.160	0.127	0.064	0.057
	100	1	2.33	2.08	0.84	0.67	51.714	0.045	0.040	0.016	0.013
		10	5.73	4.83	2.17	1.23	51.714	0.111	0.093	0.042	0.024
		50	8.01	7.16	4.21	3.7	51.714	0.155	0.138	0.081	0.072
		100	8.35	7.69	4.68	4.15	51.714	0.161	0.149	0.090	0.080
	200	1	3.25	2.89	0.976	0.78	73.135	0.044	0.040	0.013	0.011
		10	6.11	5.33	2.528	1.69	73.135	0.084	0.073	0.035	0.023
		50	10.36	8.81	5.17	3.2	73.135	0.142	0.120	0.071	0.044
		100	10.74	9.16	5.71	3.83	73.135	0.147	0.125	0.078	0.052

Natural soil +1.5 % FeCl3	50	1	1.827	1.27	0.716	0.612	27.251	0.067	0.047	0.026	0.022
		10	2.98	2.31	0.987	0.85	27.251	0.109	0.085	0.036	0.031
		50	3.305	2.975	1.184	0.973	27.251	0.121	0.109	0.043	0.036
		100	3.96	3.265	1.8	1.448	27.251	0.145	0.120	0.066	0.053
	100	1	1.931	1.48	0.77	0.65	38.538	0.050	0.038	0.020	0.017
		10	4.063	3.171	1.37	0.939	38.538	0.105	0.082	0.036	0.024
		50	5.41	4.59	2.08	1.736	38.538	0.140	0.119	0.054	0.045
		100	5.78	4.86	2.48	2.18	38.538	0.150	0.126	0.064	0.057
	200	1	2.25	1.43	0.836	0.78	54.501	0.041	0.026	0.015	0.014
		10	4.53	3.63	1.48	0.989	54.501	0.083	0.067	0.027	0.018
		50	7.71	5.96	3.77	3.1	54.501	0.141	0.109	0.069	0.057
		100	7.94	6.51	4.1	3.58	54.501	0.146	0.119	0.075	0.066