STABILIZATION OF EXPANSIVE SOIL USING LIME AND SAW DUST ASH (IN CASE OF AKAKI KALITY AREA)

By

Bakale Leta Tufa

A Thesis Submitted to the department of Civil Engineering

School of Civil Engineering and Architecture

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

Office of Graduate Studies

Adama Science and Technology University

June, 2023

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DECLARATION AND RECOMMENDATION

DECLARATION

I hereby declare that this Master Thesis entitled "stabilization of expansive soil using lime and saw dust ash in case of Akaki Kality area" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

Bakale Leta Tufa	___________________	_______________
Name of the student	Signature	Date

RECOMMENDATION

I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled "**Stabilization of expansive soil using lime and saw dust ash in case if Akaki Kality area**" prepared under my/our guidance by Bakale Leta submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Civil Engineering.

Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

Srikanth Vadlamudi (PhD)	______________	_______________
Major Advisor	Signature	Date
____________________	_______________	________________
Co-advisor	Signature	Date

APPROVAL

I/we, the advisors of the thesis entitled "Stabilization of expansive soil using lime and saw dust ash in case of Akaki Kality area" and developed by Bakale Leta, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Srikanth Vadlamudi (PhD)	____________________	________________
Major Advisor	Signature	Date
________________________	____________________	________________
Co-advisor	Signature	Date

APPROVAL

We, the undersigned, members of the Board of Examiners of the thesis by Bakale Leta have read and evaluated the thesis entitled "Stabilization of expansive soil using lime and saw dust ash in case of Akaki Kality area" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Civil Engineering.

Chairperson	Signature	Date
Internal Examiner	Signature	Date
Dr. Tezera Firew Azmatch		August 17, 2023
External Examiner	Signature	Date

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

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School Dean	Signature	Date
Office of Postgraduate Studies, Dean	Signature	Date

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials;
AASTU	Addis Ababa Science and Technology University
ASTM	American Society for Testing and Materials;
ASTU	Adama Science and Technology University
Cc	Compression index;
CEC	Cation Exchange Capacity
C_r	Recompression index;
Cu	Undrained shear strength;
C_v	Coefficient of consolidation;
MDD	Maximum dry density;
ES	Expansive soil;
FSI	Free Swell Index;
FSR	Free swell ratio;
LL	Liquid Limit;
MDD	Maximum Dry Density;
OMC	Optimum Moisture Content;
PI	Plastic Index;
PL	Plastic Limit;
SDA	Saw dust ash
UCS	Unconfined compressive strength;
USCS	Unified Soil Classification System;

ABSTRACT

Expansive soils are highly problematic because they undergo uncontrolled volumetric changes corresponding to variations in moisture content. The degree of expansiveness depends on whether the soil mass contain active clay minerals or not. Because of this alternate swelling and shrinkage, structures founded on them are severally damaged. The presence of the smectite group of minerals, such as montmorillonite, illite, vermiculite is mainly responsible for the swell and shrinkage behavior of expansive soils upon varying the moisture content.   They are poor materials to be utilized in the construction of highways, air fields, and light weight structures, because they contain a large percentage of plastic clay and often expansive, swelling as moisture is absorbed. Stabilization of expansive soils with stabilizers controls the potential of soils for a change in volume, and improves the strength of soils. The utilization of locally available material is essential for the sustainable and economical design of construction projects such as roads, railways and buildings. In this study, the combined effect of lime and industrial waste by-product of sawdust ash on expansive soil properties is investigated to control the problems posed by expansive soil. The Expansive soil sample used for this research work is collected from Addis Ababa,Akaki Kality Sub City around Tullu Dimtu condominium area. The tests was performed on the natural black cotton soil, black cotton soil mixed with lime and on black cotton soil mixed with sawdust ash and optimum amount of lime. Then optimum dosage of lime was obtained from the laboratory test result was chosen based on the maximum value unconfined compressive strength test result by mixing expansive soil with lime of 2%,3%,4% and 5%by mass of soil. Since the trial mixes showed that the use of 5 % lime gave better results, then the expansive soil added with optimum lime content for further improve the properties of the expansive soil with 4%, 8 %, 12%, and 16 % saw dust ash were analyzed. From the analysis of the test result, the liquid limit, plastic limit, plasticity index, and free swell and maximum dry density of the soil were decreased while optimum moisture content and unconfined compressive strength of treated soil increased upon addition of additives. From the study it is concluded that treating black cotton soil with lime and sawdust ash and improves the properties of the expansive soil.

Key words: **Expansive soils, Lime, saw dust ash, soil stabilization**

CHAPTER ONE

1.INTRODUCTION

1.1 Background

Expansive soils are clay soils that change their volume when the water contents of soils change. An increase in water content causes the soil to swell. It is characterized by a significant volume shift with changes in water content, and it can seriously harm and deform structures, especially light buildings and pavements(Zha & Du, 2015).

Expansive soil is commonly known as black cotton soil because of its color and its suitability for growing cotton. It starts to swell or shrinks excessively due to changes in moisture content(Asha et al., 2022). All the black cotton soils are not expansive soils and all the expansive soil are not black in color. These soils possessed high strength in the dry season and decreased rapidly in the wet season. Swelling and shrinkage of expansive soil cause deferential settlement resulting in severe damage to the foundation, buildings, roads, retaining structures, and canal linings(U.G.Fulzele & 3D.D.Parkhe, 2016).

Depending on the stress level and the soil swelling pressure, an engineering structure connected to black cotton soil will either settle or heave (Bhavsar & Patel, 2016a). The Engineers have faced difficulties with the black cotton soil due to its strong swelling and shrinking characteristics. The presence of the smectite group of minerals, such as montmorillonite, illite, vermiculite, etc. is mainly responsible for the swell and shrinkage behavior of expansive soils upon varying the moisture content(Jain et al., 2020). Due to its problematic nature, many industrial waste materials and chemical additives, such as lime, cement, gypsum, quarry dust, fly ash, bitumen, rice husk and non–traditional materials (sulfonated oils, potassium compounds, ammonium chloride, enzymes and polymers), are used to improve the physical, and engineering properties of expansive soils for highways, runways, and other constructions, such as embankments, slopes and foundation layer(Jain et al., 2020).

Because they experience uncontrolled volumetric fluctuations in response to changes in moisture content, expansive soils are quite problematic. These soils swell when they absorb water and shrink when water evaporates(Ahmed et al., 2020).

Structures built on them sustain several damages as a result of this alternate swelling and shrinkage. Problems associated with expansive soils are widespread throughout the five continents(Tripathy & Subba, 2009). Potentially, expansive soils can be found anywhere in the world but it is abundantly confined to semi-arid and arid regions.

Expansive soils have been linked to engineering issues in numerous nations all over the world. They severely damage structures, costing millions of dollars. The annual cost of damage to the civil engineering structures is estimated at £150 million in the UK, $1000 million in the USA and many billions of pounds worldwide. These damages are most common especially in the arid and semi-arid regions. Design and construction of civil engineering structures on and with expansive soils is a challenging task for geotechnical engineers(Bhavsar & Patel, 2016).

Either the existing soil must be removed and replaced with a non-expanding soil, or stabilization must be used to enhance the present soil's engineering capabilities. Replacing the existing soil might not be a feasible option therefore, the best available approach is to stabilize the soil with suitable stabilizers. Different kinds of soil stabilizers, including as fly ash, cement kiln dust, and lime, as well as locally accessible materials, such as slate dust and rice husk ash, are utilized to stabilize the soil. However, the selection of a specific kind of stabilizer depends upon the type of sub grade soil and availability of stabilizers(Bhavsar et al., 2014). Several researchers have reported the benefits of stabilizers for modifying the engineering properties of soil. Soil stabilization is a technique of increasing or maintaining the stability of soil mass and chemical alteration of soil to enhance their engineering properties. It is required when the soil available for construction is not suitable for the intended purpose. However, the term stabilization is generally restricted to the process which alter the soil material itself for improvement of its properties (Singh & Arora, 2017). Stabilization of expansive soils with admixtures controls the potential of soils for a change in volume, and improves the strength of soils.

The utilization of locally available materials is essential for the sustainable and economical design of construction projects such as roads, railways, and buildings. This study is aimed at investigating the combined effect of lime and saw dust ash to improve the geotechnical characteristics of black cotton soils. It uses lime and waste materials of saw dust ash for the improvement of black cotton soil.

1.2 Statement of Problem

Black cotton soils are the major problematic soils of the world. It is not suitable for heavy-stress designs; black cotton soil is the most prevalent cause of damage to building construction. They are poor materials to be utilized in the construction of highways, airfields, and lightweight structures, because they contain a large percentage of plastic clay and are often expansive, swelling as moisture is absorbed. The inherent factor ascribed to the expansiveness of black cotton soil is the high content of clay mineral, montmorillonite because of structural arrangement and weak bond between units. Thus, soils containing montmorillonite minerals are vulnerable to significant volume changes(Fentaw et al., 2021). They swell as the moisture gets entered into the lattice structure and shrinks if the water is removed. In a moist state, montmorillonite is highly plastic and has little internal friction. Its high swelling capacity could potentially jeopardize the stability of overlying buildings and pavements on roads. A large part of Addis Ababa, Akaki Kality sub-city is covered by expansive soils(Lemi A,2015). However, so many lightweight, residential buildings, and industrial park buildings were constructed over this problematic soil. When it comes in contact with water excessive swelling is caused and when the water content decreases shrinkage occurs(Ahmed et al., 2020). Because of this movement include lightly weighted constructions like residential buildings, pavements, canal beds, and linings that are found in this area are severely damaged.

Therefore, to minimize this problem different measures can be taken, one of those methods is stabilization. Chemical stabilizers are commonly used to improve the performance of soils with high plasticity, poor workability, and low strength and stiffness. Lime stabilization is one of the most commonly adopted techniques for stabilization of expansive soils that have always posed problems to civil and geotechnical engineers all over the world. However, in recent times, the focus is on the utilization of industrial waste materials along with lime in order to achieve improved performance and waste management. Stabilization of expansive soil using local solid waste material is preferable in terms of cost and environmental assessment. In this study, the combined effect of lime and an industrial waste by-product of sawdust ash on expansive soil properties is investigated aimed to control the problems posed by expansive soil and offers another way to dispose of the sawdust.

1.3 Objectives of the Study

1.3.1 General Objectives

The general objective of this study work is to investigate the effect of lime and saw dust ash mixture to improve the engineering properties of expansive soils around the Akaki Kality area.

1.3.2 Specific Objectives

- ✓ To evaluate the effect of lime and saw dust ash on the properties of the expansive soil using Atterberg limits, free swell, compaction, UCS and as measuring parameters.
- ✓ To compare the changes in properties of expansive soil untreated and treated with lime and saw dust ash stabilized soil.
- ✓ To investigate the effect of lime and saw dust ash on the consolidation test of the soils of the Akaki Kality area.

1.4 Scope of the Study

This study has been supported by secondary resources and a series of laboratory experiments. It focuses on the stabilization of black cotton soils of two different test pit locations in Addis Ababa, Akaki Kality sub city using lime and sawdust ash. However, the findings of the research were limited to one soil sample considered in this which is expansive clay soil. The relevant laboratory tests will be Grain size distribution, specific gravity, Free swell test, Atterberg limit, compaction, one-consolidation test and unconfined compressive strength test analysis of natural expansive soil, and expansive soil that has been lime and sawdust ash stabilized.

1.5 Significance of the Study

This study will be help in showing an alternate and locally available material for soil stabilization by reducing the cost of the removal of expansive soil and replacing it with other is non-expansive materials and protecting the environment by reducing of waste materials.

- To study the usefulness of lime and locally available industrial wastes mixture enhancing the engineering capabilities of problematic soil to make it suitable for foundation material.
- Utilizing local waste material as stabilizers and avoiding mass replacement methods.
- To keep the environment clean by using those wasted materials as soil stabilizers.

1.6 Limitations of the Study

The mineralogy of the soil samples could not be conducted due budget constraints and only the physical properties of soils and the stabilizers were determined. The study is conducted on selected section of a particular area and samples were taken from limited test pits for laboratory studies and limited number of tests were conducted to achieve the objectives of this study.

1.6 Organization of the thesis

The presentation of this thesis work is organized into five chapters. The first chapter gives a brief description of the thesis background, statement of the problem, objectives, scope, and significance of the study. The second chapter is a literature review that explains the nature and properties of expansive soil, the genesis of expansive soils, the mineralogy of expansive soils, the classification and distribution of expansive soils, the mechanism of swelling, the importance of stabilization, and its significance, and the conceptual background of the additives used for the research, and the important procedure in the stabilization process. The third chapter briefly presents the characterization of materials used for the study such as expansive soil, lime, and saw dust ash, experimental design, the methods adopted to collect data, and standard testing procedures followed. The fourth chapter presents the test results obtained; analysis of results and discussion of results for natural expansive soil, the addition of lime on expansive soil and combined effect of lime, and saw dust ash on expansive soil. Finally, conclusions and recommendations drawn from the study were presented in chapter five.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Origin of Expansive Soil

The word "expansive soil" generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture conditions (Sabat, 2016). Subsequent swelling and shrinkage of this soil due to change in moisture cause damages to different structures, particularly lightweight buildings and pavements. The degree of expansiveness depends on whether the soil mass contains active clay minerals or not. The most popular active clay mineral is Montmorillonite. The origin of Expansive soil is related to a combination of conditions and processes that results in the creation of clay minerals having a particular chemical and mineralogical makeup, which, when in contact with water expands. Variations in the conditions and processes may also form other clay minerals, most of which are non-expansive. Due to the disintegration of black lava i.e., Basalt Rock by the Sun, wind, and rain formation of black cotton soil occurs(U.G.Fulzele & 3D.D.Parkhe, 2016). The conditions or processes that determine the clay mineralogy include the composition of the parent material and the degree of physical and chemical weathering to which the materials are subjected (Tamrat, 2013).

2.1.1 Parent Material

The parent materials that can be associated with Expansive soils are classified into two groups(Ferdawek, 2019). The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montmorillonite as a constituent. The basic igneous rocks are comparatively low in silica and rich in a metallic base such as the pyroxenes, amphiboles, biotite, and olivine fall within this category. Such rocks including the gabbros, basalts and volcanic glass have decomposed to form montmorillonite. Shale and clay stones are the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils. Limestone and marl rich in magnesium can also be weathered to clay. These constituents of the shales and clay stones contain varying amounts of volcanic ash and glass, which are subsequently weathered to montmorillonite.

The montmorillonite was probably formed from two separate origins. The products of weathering and erosion of the rocks in the highlands were carried by streams to the coastal plains. The fine-grained soils eventually became shale accumulating in the ocean basin. Meanwhile, volcanic eruptions, sending up clouds of ash, fell on the plains and the seas. These ashes were altered to montmorillonite(Fu Hua Chen, 1975).

2.1.2 Weathering and Climate

The weathering process by which clay is formed includes the physical, biological and chemical processes. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering of the parent rock minerals. The parent material generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks, and volcanic ash. The formation is aided an alkaline environment, the presence of magnesium ions, and a lack of leaching. Such condition is favorable in semi-arid regions with relatively low rainfall or seasonal moderate rainfall, particularly where evaporation exceeds precipitation. Under these conditions, enough water is available for the alteration process but the accumulated cations will not be removed by rainwater(B Firdawek, 2019).

Climate is the principal factor governing the rate and type of soil formation. The two important components of climate are the amount and distribution of precipitation and temperature. The amount and distribution of precipitation affect the availability of moisture and the relative humidity of the soil atmosphere; it influences the concentration or chemical activity of solutions in the system. A warm climate with alternate dry and wet seasons is favorable for the formation of montmorillonite(Tamrat A, 2013).

2.2 Clay Mineralogy

Expansivity of soils is due to the presence of clay minerals. The term clay can refer both to size and a class of minerals. As a size term, it refers to all constituents of soil smaller than a particular size, usually 0.002 mm in engineering classifications. As a mineral term, it refers to Specific clay minerals that are distinguished by small particle size, a net Electrical charge, plastic when mixed with water, and high weathering resistance(B Firdawek, 2019). Minerals are crystalline and make up the solid’s constituent of soil. Minerals are classified according to chemical composition and structure.

Most minerals of interest to geotechnical engineers are composed of oxygen and silicon. Silicates are a group of minerals with a structural unit called the Silica Tetrahedral.

A central silica cation (positively charged ion) is surrounded by four oxygen anions (negatively charged ions), one at each corner of the tetrahedron. Silicate minerals are formed by the addition of cations and the interaction of tetrahedrons. Silica tetrahedrons combine to form sheets, called silicate sheets, which are thin layers of silica tetrahedrons in which three oxygen ions are shared between adjacent tetrahedrons. Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by a combination of alumina minerals, which consists of an aluminum ion surrounded by six oxygen or hydroxyl atoms in an octahedral(B Firdawek, 2019).

Three structural groups of clay minerals that are important for engineering purposes.

These three groups are: -

Kaolinite group - usually non-expansive.

Mika-like group - includes illites and vermiculites, which can be expansive but generally do not pose significant problems.

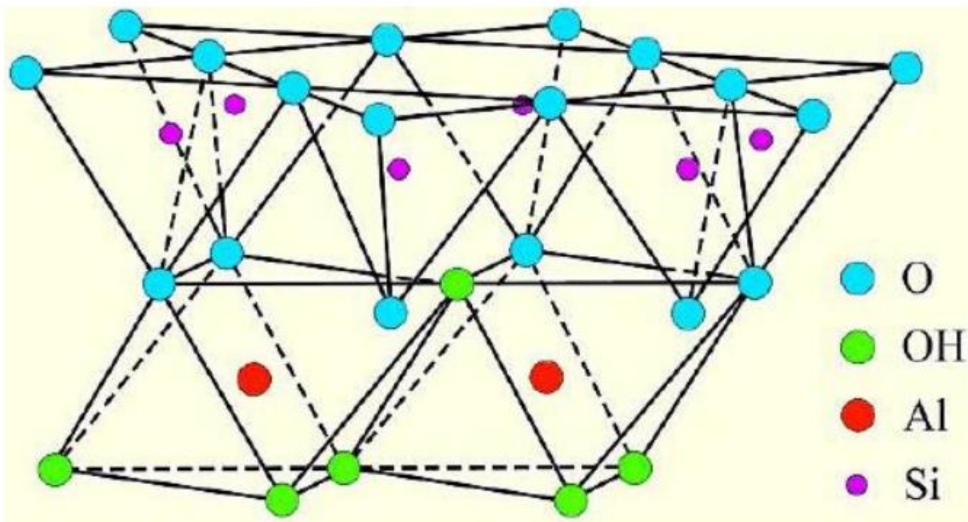
Smectite group - includes montmorillonites, which are highly expansive and are the most troublesome clay mineral

2.2.1 Kaolinite Group

The kaolin minerals are a group of clay minerals consisting of hydrous aluminum silicates. A common kaolin mineral is a kaolinite. Kaolinite is a typical two-layer mineral having a single tetrahedral sheet joined by a single octahedral sheet to form what is called a 2-to-1 lattice structure. The bonding combination of hydrogen and Vander Waals forces results in considerable strength and stability with little tendency for interlayers to take on water and swell. There is no interlayer swelling when there is water present because of the bonding's strength. Kaolinite is the least active of clay minerals. Kaolinite can be produced by weathering certain of the more active clay minerals as well as being directly formed as a by-product of rock weathering. Kaolinite tends to be found in regions of heavier rainfall(B Firdawek, 2019).

As a result, kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently, kaolinite has a low degree of expansiveness(Tamrat et al., 2013).

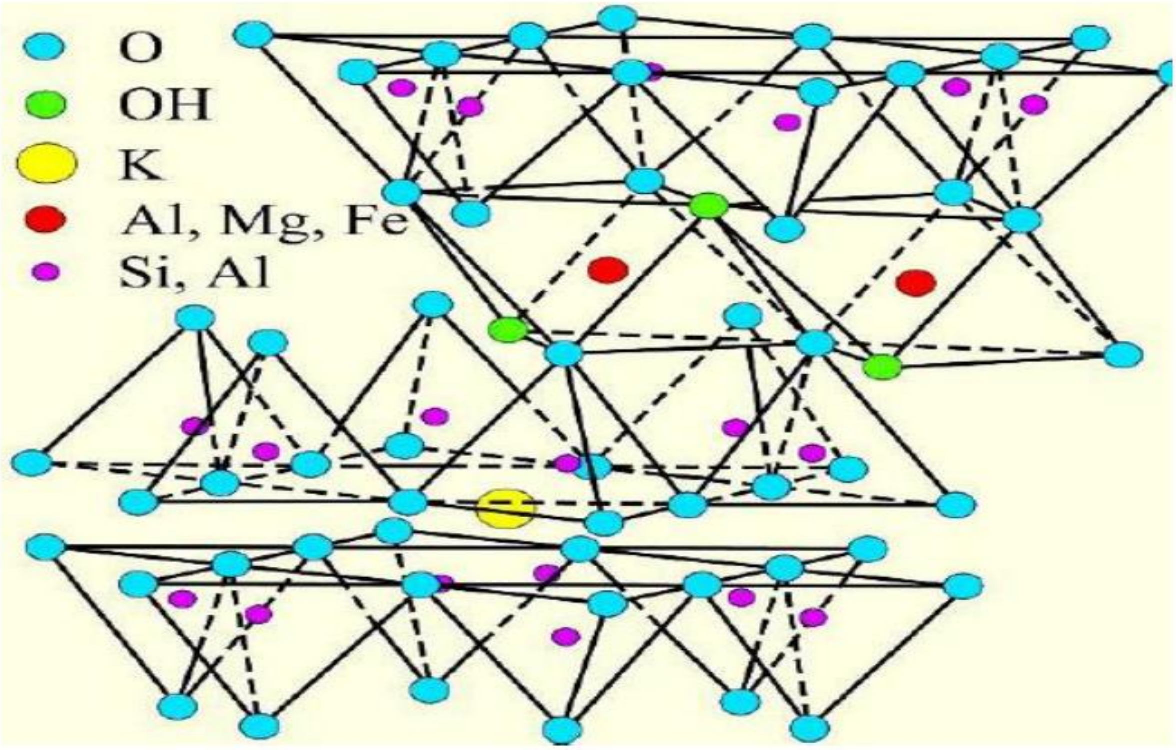
Figure 2. 1 Diagrammatic Sketch of the Kaolinite((B FiIrdawek, 2019))



2.2.2Illite Group

Illite has a basic structure consisting of a sheet of alumina octahedrons between and combined with two sheets of silica tetrahedrons. This clay mineral has a structure similar to montmorillonite, but the layers are more strongly bonded together. In terms of cation exchange capacity, inability to absorb and retain water, and physical characteristics. Illite is intermediate in activity between clays of kaolin and montmorillonite groups. Within the tetrahedral sheet, there is a partial substitution of aluminum for magnesium and iron. And within the tetrahedral sheet, there is a partial substitution of silicon by aluminum due to very weak bonding between the combined sheets as a result of (non-exchangeable) potassium ions held between them(B Firdawek, 2019). However, the basic illite units are joined by potassium ions w hich are non-exchangeable. Because of this, the illite units are reasonably stable and so the mineral swells much less than montmorillonite. Hence, illite has a moderate degree of expansiveness (Tamrat et al., 2013).

Figure 2. 2 Diagrammatic Sketch of the Illite(B Firdawek, 2019)



2.2.3 Smectite Group

Montmorillonite is a group of clay minerals that are characterized by weekly bonded layers. It is formed from weathering of volcanic ash in areas with poor drainage or in marine waters. Each layer consists of two silica sheets with an aluminum (gibbsite) sheet in the middle. The basic montmorillonite units are piled one on top of the other but, the bond between the individual units is relatively weak and water is easily able to penetrate between the sheets and cause their separation and hence swelling. Water and exchangeable cations can enter and separate the layers, creating a very small crystal that has a strong attraction to water. Montmorillonite has the highest activity and it can have the highest water content, greatest compressibility, and lowest shear strength of all clay minerals(B Firdawek, 2019).

The space between the combined sheets is occupied by water molecules and exchangeable cations. There is a very weak bond between the sheets that are joined due to these ions. Considerable swelling of montmorillonite being can occur due to additional water absorbed between the combined sheets(Umer, 2014). Therefore, montmorillonite has a very high degree of expansiveness (Tamrat et al., 2013).

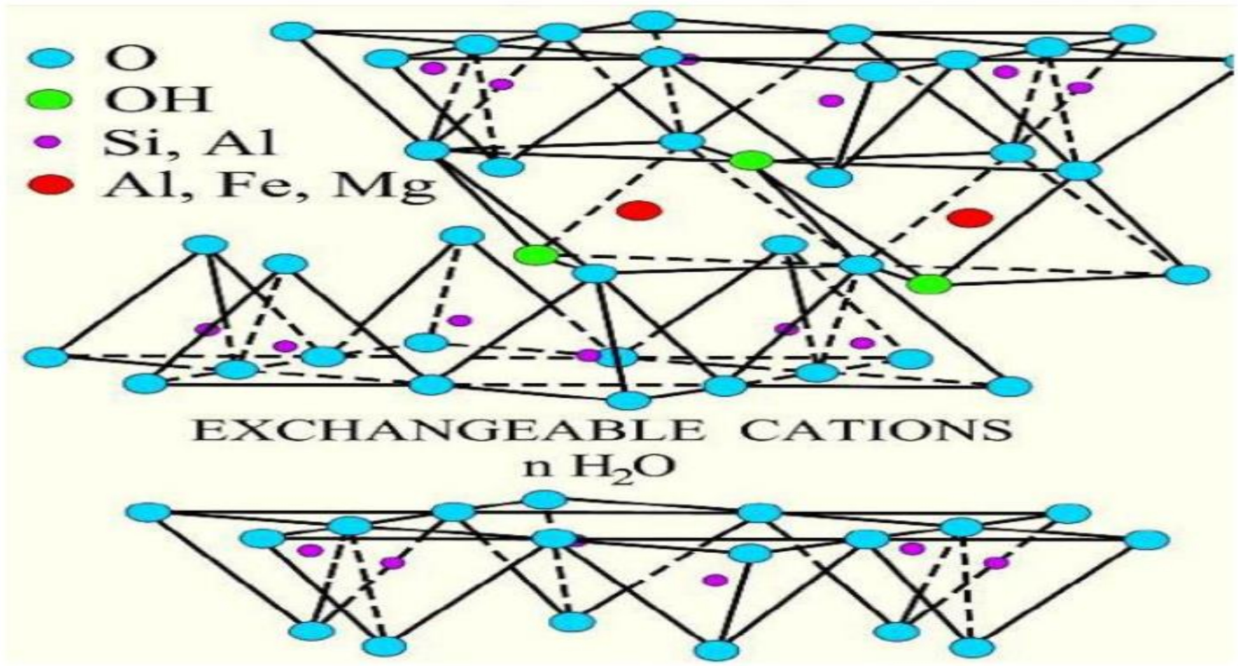


Figure 2.3 Diagrammatic Sketch of the Montmorillonite

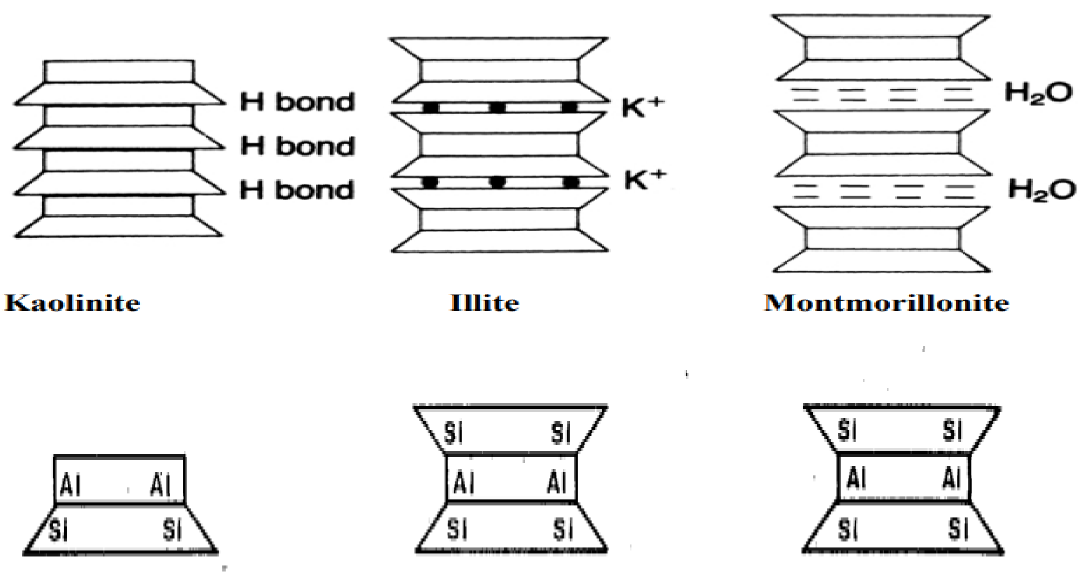


Figure 2. 3 Schematic Representations of Clay Minerals(Baser, 2009)

2.3 Distribution of Expansive

Expansive soils are widespread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria, etc. In other parts of the world case of expansive soils have been widely reported in countries like the USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela, etc.(B, Firdawek 2019). It is estimated that Ethiopia has 24.7 million acres of expansive soils as cited by(B, Firdawek 2019). They are widely spread in the central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis–Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. Also, areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure Wubshet M & 201.)

Figure 2. 4 Distribution of expansive soil in Ethiopia(Firdawek B, 2019)

2.4 Identification of Expansive Soils

Expansive soils that exhibit high swelling potential can be recognized by both field observation and laboratory tests. Investigation of expansive soils generally consists of two important phases. The first is the visual identification and recognition of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss different ways that are commonly used to identify expansive soils.

2.4.1 Field Identification

Expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Great potential swell is indicated by large and more frequent polygon arrangements of cracks while the network of small and thin cracks indicates low shrinks. Soils containing expansive clays are characterized by surface cracks or a "popcorn" texture when dry.

Some of the important field identification methods that indicates the potential for the expansiveness of soil is the following : (Tamrat A, 2013)

A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail.

The wet sample of the soil is sticky and it is relatively difficult to clean the soil from the hands.

The appearance of cracking in nearby structures.

They usually have a color black and/or gray

In the regions where there is seasonal moisture variation in open or closed fissures (a joint or similar discontinuity), Slickenside (highly polished or glossy fissure surface) and shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils) may be observed.

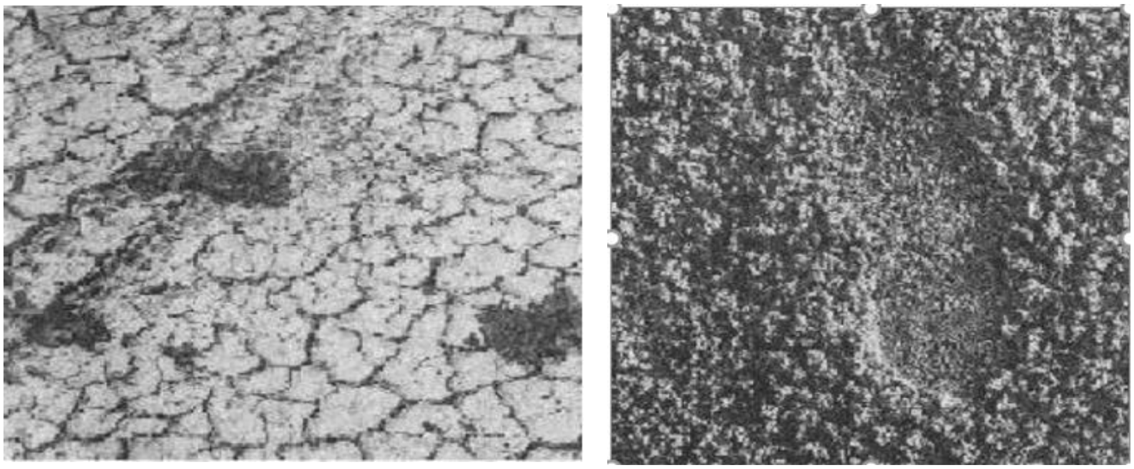


Figure 2. 5 (*a) Expansive soil showing cracks (b) Expansive soil showing popcorn (Mokhtari, 2012)*

2.4.2 Laboratory Identification

A key technique for choosing the best solution is identifying potential subsurface issues, such as swelling or shrinkage design and methods of construction. In general, there are three different methods of laboratory identification of expansive soil namely; mineralogical identification, indirect and direct methods.

2.4.3 Mineralogical Identification

The basis of the method is that the expanding potential of any clay can be evaluated by the identification of the constituent mineral of that clay. This technique is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size, and shape of clay particles, and surface activity of clay particles.

These properties are the primary element governing expanding soil behavior. The several methods under these methods are:

- X-ray diffraction
- Differential thermal analysis
- Dye absorption
- Chemical analysis and Base exchange capacity, etc

The various methods listed above should generally be used in combination. Using combinations of the methods, the different types of clay minerals present in a given soil can be evaluated quantitatively. But these methods are not suitable for routine tests because,

- they are time-consuming,
- require expensive test equipment and
  the results are interpreted by specially trained technicians

2.4.3.1 X-Ray Diffraction Method.

The X-ray diffraction method used in determining the proportion of the various minerals present in a colloidal clay consists essentially of comparing the ratios of the intensities of diffraction lines from the different minerals with the intensities of lines from the standard substance.

(FU HUA CHEN, 1975)claimed that the use of self-recording counter spectrometers instead of photographic techniques increases considerably both the accuracy and the convenience of the X-ray method. Brindley also believes that the X-ray method for quantitative determinations should be applied with considerable circumspection and that in favorable cases the possibility of identifying species by X-ray analysis can be regarded with restrained optimism.

2.4.3.2 Differential Thermal Analysis.

Differential thermal analysis when used in conjunction with X-ray diffraction and chemical analysis enables the identification of otherwise difficult materials. It is well established as a technique for the control of materials that undergo characteristic changes on heating. The use of differential thermal analysis techniques in identifying expansive soil is not always accurate(FU HUA CHEN, 1975).

2.4.3.3 Dye Adsorption.

Dyestuffs and other reagents which show characteristic colors when adsorbed by clay have been used to identify clay. When a clay sample has been pretreated with acid, the color assumed by the adsorbed dye depends on the base exchange capacity of the various clay minerals present. The presence of montmorillonite can be detected if its amount is greater than about 5 to 10 percent. The relatively simple testing procedure and speed of dye staining tests compared with X-ray diffraction and differential thermal analysis justify the wider application of the color method.

2.4.3.4 Chemical Analysis.

Chemical analysis can be a valuable supplement to other methods such as X-ray analysis in identifying clays. In the montmorillonite group of clay minerals, chemical analysis can be used to determine the nature of isomorphism and to show the origin and location of the charge on the lattice. According to (FU HUA CHEN, 1975), the isomorphous character of the montmorillonite group can probably be shown in no other way. The isomorphism involves three basic variations in the substitution: the substitution for Al for Si in tetrahedral positions in the lattice; the substitution of Fe for Al in the octahedral coordination; and the substitution of Mg for Al in the octahedral positions.

2.4.3.5 Electron Microscope Resolution. Microscopic examination of clay minerals offers a direct observation of the material. Two clays may give the same X-ray pattern and the same differential thermal curve but will show up distinct morphological characteristics under electron microscope resolution. The main purpose of the microscopic examination is to determine the mineralogical composition, texture, and internal structure.

(Fu Hua Chen, 1975) made an extensive study of the mineralogical composition of expansive clays by the use of the scanning electron microscope. It showed that the non-swelling clays appear as flat, relatively thick plates while montmorillonites have a crinkly, ridged, honeycomb-like texture. It might be possible to evaluate some properties of the expansive soil by observing the degree of crinkling and inter-particle bonding by scanning an electron microscope.

2.4.4 Indirect Methods

In this method, simple soil property tests can be used for the evaluation of the swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include

i. Atterberg Limits

In this method, measurement of the atterberg limits of the soil is conducted for identification of all soils and provides a wide acceptable means of rating. Especially when they are combined with other tests, they can be used to classify expansive soils. The relation between the swelling potential of clays and the plasticity index is shown in the table below.

Table 2. 1 Relation between the swelling potential of clays and the plasticity index(Wubshet M, 2013)

Swelling potential	Plasticity index
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

ii. Free Swell Tests

The free swell test may be considered a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. Experiments indicated that a good grade of high-swelling commercial bentonite will have a free swell of from 1200 to 2000 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings. The free swell percentage can be computed using Equation (2.1) from the relationship between initial and swelled volume(Wubshet M 2013).

Free swell (%) = $\frac{Vf-Vi}{Vi}$ *100 ………………………………………………………… (2.1)

Where:

V_i=initial volume

V_f=final volume

iii. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The procedure involves taking two oven-dried soil samples passing through a 425µm sieve, 10cc each was placed separately in two 100ml graduated soil samples. Distilled water was filled in one cylinder and kerosene within the second cylinder up to the 100ml mark.

The final soil volume is computed after 24 hours to calculate the free swell index. The free swell index is then calculated using Equation (2.2)(Wubshet M, 2013).

$$\text{Free swell index}_{(\%)} = \frac{Vw - Vk}{Vk} * 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

Where:

V_w = final volume in water

V_k= final volume in kerosene

The relation between the degree of expansion and differential free swell index is shown in Table 2.2. It is normal to quantify 10cc as the volume occupied by 10g of soil. This does not account for variations in density

Table 2. 2 Degree of expansion and differential free swell index(Wubshet M, 2013)

Free swell index (%)	Degree of expansion
Less than 20	Low
20 to 35	Moderate
35 to 50	High
Greater than 50	Very high

iv. Free Swell Ratio test

To determine the swell property, Sridharan and Prakash proposed the free swell ratio method of characterizing soil swelling. The definition of free swell ratio is the ratio of sediments volume of 10cc oven dried-soil passing through 425µm sieve in distilled water to that of Kerosene Equation (2.3).

$$\text{Free swell ratio} = \frac{Vw}{Vk} x100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

Where: V_w = final volume in water,

V_k = final volume in kerosene

Table 2. 3 Classification of Soils based on free swell ratio(Wubshet M ,2013)

Free Swell Ratio	Soil Expansivity	Clay Type
<1	Negligible	Non-swelling
1.0-1.5	Low	Mixture of non-swelling & Swelling
1.5-2.0	Moderate	Swelling
2.0-4.0	High	Swelling
>4	Very high	Swelling

v. Cation Exchange Capacity (CEC)

The CEC is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles. CEC is expressed in milliequivalents per 100 grams of dry clay. CEC is related to clay mineralogy. High CEC values indicate a high surface activity. In general, swell potential increases as the CEC increases. Typical values of CEC for the three basic clay minerals are given in Table 2.4.

Table 2. 4 Typical CEC values of basic clay mineral (Firdawek B, 2019)

Clay Mineral	CEC(meq/100gm)
Kaolinite	3 - 15
Illite	10 - 40
Montmorillonite	80 -150

2.4.5 Direct Measurement

These methods present the most useful data by direct measurement of determining the swelling potential and swelling pressure of expansive clay soil and tests are simple to perform and do not require complicated equipment. Direct measurement of expansive soils can be achieved by the use of a conventional one-dimensional odometer which is available in most soil mechanics laboratories. The method quantitatively evaluates the volume change characteristics of Expansive soil.

2.5 Classification of Expansive Soils

Parameters determined from expansive soil identification tests have been combined in a number of different classification schemes. The classification system used for expansive soils is based on an indirect and direct prediction of swell potential as well as combinations to arrive at a rating. There are several classification systems. The following are some of the common methods.

2.5.1 Classification Using General Methods

The most widely used general classification systems are:

i. AASHTO Classification

As shown on Table 2.5 soils rated A-6 or A-7 by AASHTO can be considered potentially expansive(Berhane, 2015).

Table 2. 5 AASHTO soil classification chart

General Classification	Granular Materials (35 percent or less of total sample No.200)							Silt-clay Materials (more than 35 percent of total sample passing No.200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No.10	50 max										
No.40	30 max	50 max	51 min								
No.200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No.40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P	10max	10 max	11min	11 max	10max	10 max	11min	11 max
Usual types of significant constituent materials	Stone fragments-gravel and sand		Fine sand	Silty or clayey gravel sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

ii. Unified Soil Classification Systems

In this classification system a correlation is made between swell potential and unified soil classification as follows.

Table 2. 6 Unified soil classification

Category	Soil Classification In Unified System
Little or no expansion	GW,GP,GM,SW,SP,SM
Moderate expansion	GW,SC,ML,MH
High volume change	CL,OL,CH,OH

The following is a summary of the classification system mentioned above:

a. All clay soil and organic soils exhibit high volume change.

b. All clayey gravels and sands and all silts exhibit moderate volume changes.

c. All sands and gravels exhibit little or no expansion.

2.5.2 Classification Specific to Expansive Soil

The above classification system may give an initial alert that the soil may have expansive character but it does not provide useful information. A parameter determined from the expansive soil identification tests has been combined in several different classification schemes to give a qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to overly conservative construction in some places and inadequate construction in some areas(Nelson & Miller, 1993). Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

2.5.3 Classification based on indirect predictions of swell potential

An indirect prediction of swell potential includes correlations based on index properties, swell, and a combination of them. Some of such classification systems are discussed below:

- **Alemayehu and Mesfin [1999]**

One may use to check as the accuracy of laboratory test results for Expansive soils found in Ethiopia.

Table 2. 7 Indicative properties of Ethiopian Expansive soils

Clay content smaller than 2μm	50-80%
Liquid limit	80-120%
Plasticity limit	55-90%
Shrinkage limit	10-16%
Free swell	90-123%

- **Skempton's method (Mckeen, 1976)**

This method classifies clays according to their activities which is developed by Skempton (1953) by correlating Atterberg limits and clay content (% percent by weight finer than 2μ m) into a single parameter called activity. Skempton classifies clays according to their activities. Following his classification, three degrees of colloidal activity

(Activity, Ac = $\frac{\text{plasticity index}}{\text{percent clay}(<0.002\text{mm})}$ have been established as indicated in table 2.8.

Table 2. 8 degree of colloidal activity (Skempton's method)

Degree of activity	Activity
Inactive	< 0.75
Normal	0.75-1.25
Active	>1.25

Following this classification: -

montmorillonite clay (expansive clay) is defined as active

Illitic clay is normal and

Kaolinitic clay is inactive.

U.S.B.R Classification Method

This method is developed by Holtz and Gibbs; it is based on the direct correlation of observed volume change with colloid content, plastic index, and shrinkage limit. The classification is given in Table 2.9.

Table 2. 9 U.S.B. R Classification method

Colloid contests (%)	Plasticity index (%)	Shrinkage limit (%)	Probable expansion (%)	Degree of expansion
<18	>15	>15	<10	Low
15-18	15-28	10-20	10-20	Medium
25-41	25-41	20-30	20-30	High
>35	>35	>30	>30	Very high

Chen Method: In this method, a correlation is made between swell data and percent less than No. 200 sieves, liquid limit, and standard penetration resistance. The classification is given in table 2.10(Tamrat A, 2013).

Table 2. 10 Chen method of classification of Expansive soil

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

The classification system developed based on single property alone such as: based on activity (Skempton, 1953), based on shrinkage limit and linear shrinkage (Altmeyer, 1956), based on index property (Kanntey and Brink, 1952), etc. are difficult to use alone as a classification system because they may lead to a wrong conclusion. But, Chen’s method of classification of expansive soil, classification based on the bureau of reclamation method, the U.S.B.R Classification method, etc. are better indirect classification systems developed by combining index property, swell and physical indicators.

2.6 Mechanics of Swelling

Swelling in expansive soils will take place if there is a change in the environment. Environmental Change can take the form of pressure release from exca-vation, desiccation from an increase in temperature, and volume growth from the introduction of moisture. By far the most important element for swelling is the effect of water on expansive soils. With the introduction of water volumetric expansion takes place. If pressure is applied to prevent expansion, the pressure required to maintain the initial volume is the swelling pressure(Tamrat A, 2013). The pattern of moisture migration depends on the geological formations, climatic conditions, topographic features, soil types, and groundwater level. The most common method of moisture transfer is by gravity. The seepage of surface water, precipitation, and snow melting into the soil are common examples. Fractures and fissures, shrinkage cracks, capillary force, vapor transfer, thermal gradients, etc. are some of the sources that cause moisture migration and swelling on expansive soils.

In general, the movement of expansive soil occurs in an uneven pattern and the resulting expansion is a magnitude that cannot be predicted by the classical elastic or plastic theory(Uba, 2011). However, the swelling behavior can be related to the combined effect of interacting factors that can be grouped into (a) local geology; (b) engineering properties, and; (c) the local environment of deposition. The main geological factors include the rock type, and age as related to the type and amount of clay minerals, the type and amount of cementing material, and the soil particles arrangement. The engineering factors include moisture content, Atterberg limits, and dry density. The environmental factors include the confining pressure, type and degree of weathering as related to the amount of clay fraction, initial water content, and water(Uba, 2011). The swelling potential is related to geological and engineering factors while the amount, and rate of swelling are controlled by environmental conditions(Uba, 2011).

2.6.1 Factors affecting volume change

The following factors influence the results obtained in loaded swell tests on soils of any mineralogical composition:

1. Initial moisture content. In testing undisturbed samples, care should be taken in selecting the sample with the most critical moisture content. Usually, tests should be performed on the driest sample. The frequency of testing is important to cover all possible conditions.

For remolded samples, it is obvious that the initial moisture content will control the volume change. Field condition and construction specifications dictate the moisture requirement. In addition, there should be a lot of focus directed to the time element-the time elapsed between sampling and testing and the time elapsed when the sample is placed in the consolidometer, and when the wetting takes place or when the load is applied.

2. Initial dry density. The single most significant variable influencing swelling characteristics of swelling soils is density. In remolded tests, the initial compaction condition is critical. Swell tests may decide the degree of compaction required in the placement of fill. Since moisture content and dry density are closely related, they should be examined concurrently.

3. Surcharge pressure. As the initial surcharge pressure increases, the volume change of expansive soil decreases but the swelling pressure remains constant.

4. Time allowed for swell. The time required for the soil to reach its maximum swell potential may vary considerably depending essentially on the initial density, permeability, and thickness of the sample. For remolded samples, generally, 24 hours is sufficient to obtain 95 percent of the total available swell. At the same time, for undisturbed high-density clay shale, it may require several days or even a week before complete saturation can be achieved. For remolded samples, the initially added water must be evenly distributed. This requires a minimum curing time of 6 hours for reproducible results.

5. Size and thickness. Sample thickness affects the time required for total saturation. To expedite testing time, a sample thickness of less than 1 inch should be used. Greater thickness may introduce excessive side friction. At the same time, in a cutting from an undisturbed sample, a small thickness may introduce surface disturbance and exclude the possible effect of granular particles, fissures, and seams in the soil.

6. As the sample is subjected to moisture, the degree of saturation increases as a result swelling potential increases but the swelling pressure is unchanged.

2.7 Review on soil stabilization

Soil stabilization is a process whereby increased strength and stability of the soil are attained mainly by mechanical or chemical means. The most common improvements achieved through stabilization include better soil gradation, reduction of plasticity index or swelling potential, increase in durability and strength. In wet weather, stabilization may also be used to provide a working platform for construction operations. These types of soil quality improvement are referred to as soil modification. Soil stabilization decreases damage caused by settlement, washing, and collapsing(Nigussie, 2011). The objective is to improve the characteristics at the site and make the soil capable of carrying a load and to increase the shear strength, decrease the compressibility of the soil so that the bearing capacity of the soil is increased and the settlement of the structures built on it are reduced. Sometimes, the aim is to decrease the permeability of the soil(Shawl et al., 2017).

Black cotton soil is an expansive soil with low bearing capacity when it is subjected to moisture, can absorb and dissipate water with subsequent changes in volume.

Construction of any structure on soil of this kind requires either replacement of the soil by importing a better foreign one or by addition of chemical(s) that will improve the soil towards the desired property(B Firdawek, 2019). If these structures are founded on soil with low bearing capacity, they are likely to fail either during or after construction, with or without the application of wheel load on them. Where the pavement or building is founded in inherently weak soil, this material will be typically then removed and replaced with a stronger granular material or improve the soil towards the desired property by the addition of chemical(s)(Wubshet, 2013).

This removal and replacement technique can be both costly and time-consuming. Where aggregates are scarce, the use of these non-renewable resources is viewed as non-sustainable, particularly if haulage distances are significant.

An alternative to the removal and replacement option is to chemically stabilize the host material. This eliminates the requirement to replace the material and ensures the engineering characteristics and performance of the host material are enhanced to allow for its use within the pavement structure.

2.7.1 Uses of Stabilization

Pavement design is based on the premise that minimum specified structural quality will be achieved for each layer of material in the pavement system. Each layer must be resistant to shearing, avoid severe deflections that could result in fatigue cracking either inside the layer or in layers above it, and limit excessive permanent deformation. As the quality of a soil layer increase, the ability of that layer to distribute the load over a greater area generally increases so that a reduction in the required thickness of the soil and surface layers may be permitted. Commonly, improvement attained from soil treatment can be summarized (Wubshet M, 2013)

Quality improvement: the most common increases in durability and strength as well as a decrease in the plasticity index or swelling potential are all benefits of stabilization with a better soil gradation. Stabilization may also be employed in rainy conditions to give construction projects a working platform. (B Firdawek, 2019).

Thickness reduction: the strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in the design thickness of the stabilized material compared with an unstabilized or unbound material. The design thickness can be reduced if the strength, stability, and durability requirement of a base or sub-base course is indicated to be suitable by further analysis(B, 2019).

2.7.2 Types of Soil Stabilization

Types of Soil Stabilization the two frequently used methods of stabilizing soils are stabilization by compaction or stabilization by chemical additives

2.7.2.1 Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized into compaction, mixing or blending of two or more gradations, applying geo-reinforcement, and mechanical remediation(Wubshet M, 2013). Sometimes, soils with coarse particles are added to achieve the necessary grade to a soil with fine particles or fine soils are removed, mechanical stabilization is also known as granular stabilization.

The soils were separated into two groups for mechanical stabilization.

1. Aggregates: - these are soils that have a granular bearing skeleton and have particles of size larger than 75μ.
2. Binders: - these are soils that have particles smaller than 75μ. They do not possess a bearing skeleton.

The aggregates consist of strong, well-graded, angular particles of sand and gravel which provide internal friction and incompressibility to the soil. The binders provide cohesion and are impervious to soil. These are composed of silt and clay. The quantity of binder should be sufficient to provide plasticity to the soil, but it should not cause swelling. Proper blending of aggregates and binders is done to the achieve required gradation of the mixed soil. The blended soil should possess both internal friction and cohesion.

The mixed soil's mechanical stability is influenced by the aggregate's mineral strength, mineral composition, gradation, plasticity properties, and compaction. Mechanical stabilization is the simplest method; it improves the sub-grades of low bearing capacity.

2.7.2.2 Chemical Stabilization

The oldest and most popular approach to ground improvement is soil stability using chemical admixtures. Chemical stabilization is the mixing of soil with one or a combination of admixtures of powder, slurry, or liquid to improve or control its stability, strength, swelling, permeability, and durability. It involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride, or combinations of these materials into the soil. The soil conditions, stabilizer characteristics, and kind of structure all affect how effective these chemicals are (i.e., houses, roads, etc.). The selection of a particular additive depends on its costs, benefits, availability, and practicality of its application(Umer, 2014).

Soil improvement using chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation-agglomeration and pozzolanic reaction.

a) Cation Exchange

The excess ions with the opposite charge to that of the surface of the clay, over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge, by altering the chemical composition of the equilibrium electrolyte solution. Negatively charged clay particles adsorb cations of a specific types and amounts. The ease of replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace cations of lower valence. For ions of the same valence, the size of the hydrated ion becomes important; The more the ion, the bigger the replacement power. If other conditions are equal, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations(Mada, 2016). A typical replace ability series is:

$$Na^{+} < Mg^{2+} < Ca^{2+} < Ba^{2+} < Cu^{2+} < Al^{3+} < Fe^{3+} < Th^{4+}$$

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. An example of the cation exchange;

$$Ca^{2+} + Na^{+}\text{-Clay} \longrightarrow Ca^{2+}\text{-Clay} + Na^{+}$$

The thickness of the diffused double layer decreases by replacing the divalent ions (Ca^{2+}) from stabilizers with monovalent ions (Na^{+}) of clay. Thus, swelling potential decreases.

b) Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with a consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Due to changes in texture, a considerable decreasing in the swelling of the soil occurs(Mada, 2016).

c) Pozzolanic Reactions

Time-dependent pozzolanic reactions play a major role in the stabilization of the soil since they are responsible for the improvement in the various soil properties.

Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate which bond the adjacent soil particles together.

(CAH). $Ca^{2+} + 2(OH)^{-} + SiO2 \text{ (Clay Silica)} \longrightarrow CSH$

$$Ca^{2+} + 2(OH)^{-} + Al2O3 \text{ (Clay Alumina)} \longrightarrow CAH$$

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength(Lgwe and Emeg, 2009).

2.7.3 Lime Stabilization

Quick lime or unslaked lime are common names for the inorganic chemical substance known as lime obtained from limestone, a naturally occurring chemical. Quick lime, also known as calcium oxide, is a strong caustic substance that is frequently used in the building industry to prepare plasters and mortar. Lime decreases soil density and increases soil strength.

With medium, moderately fine, and fine-grained soils, lime interacts to create decreased plasticity, greater workability, and enhanced strength. The chemical reactions between lime and soil particles are the main cause of the strength increase (Umer, 2014). A successful approach to identifying the solubility of the components in the clay has been discovered. To provide durability, soils must be stabilized with sufficient lime to develop pozzolanic reactions. Limes should not be exposed to air through proper handling methods before mixing, this Practice avoids premature carbonation of the lime(Umer, 2014).

Lime acts with soils in three ways. The first of the reactions is a change of water film around the clay minerals. The second process by which lime changes soil is that of coagulations or flocculations of the soil particles. The third process through which lime affects soil is a reaction of lime with soil ingredients to form new chemicals. The two main ingredients of soil which act with lime are alumina and silica. This is a long-term reaction that strengthens the soil lime mixture also known as Pozzolanic action. The degree to which lime will react with soil depends on the quantity of lime, soil type, and period of curing for the soil lime mixture.

In general, all lime-treated fine-grained soils show a reduction in plasticity, an improvement in workability, a reduction in volume change characteristics, and an increase in soil strength. Nevertheless, not all soils, display the characteristics of increased strength. It should be underlined that numerous factors affect the properties of soil-lime mixes.

The most critical factors are soil type, lime type, lime proportion, and curing conditions (time, temperature, and moisture) (Umer, 2014).

2.7.3.1 Types of lime

For many years, several types of lime have been employed successfully as soil stabilizing agents. However, the most commonly used products are:

- High calcium, quick lime (CaO)
- Hydrated, high calcium lime [Ca (OH)2]
- Dolomitic lime (CaO + MgO)
- Normal, hydrated dolomitic lime [Ca (OH)2 + MgO]
- Pressure, hydrated dolomitic lime [Ca (OH) 2 + MgO2]

Quick lime works as a stabilizer better than hydrated lime, but the latter is safer and more convenient to handle. Hydrated lime is used most often because it is much less caustic than quicklime; however, the use of quicklime for soil stabilization has increased in recent years mainly with slurry-type applications. The design lime contents determined from the criteria presented herein are for hydrated lime. The higher the magnesium content of the lime, the less affinity for water and the less heat generated during mixing.

2.7.4 Chemical properties of Quick lime and hydrated lime

Quicklime and hydrated limes are reasonably stable compounds. However, quicklime is vulnerable to water; even the moisture in the air produces a destabilizing effect by air slaking. Hydrated lime is more stable since water does not cause a change in its composition. The primary factor influencing the stability of hydrated lime is carbon dioxide which reacts with either quicklime or hydrated lime to form calcium carbonate(Umer, 2014).

The reactivity of quicklime with water is great practical importance as this reactivity is the basis for the production of hydrated lime from quicklime. The production of hydrated lime from quicklime through a slaking process at a construction site produces a very reactive product for soil stabilization(Umer, 2014).

The characteristics of soil-lime combinations depend on: a) the properties of the soil, b) The properties of the lime (chemical composition and gradation), c) lime content, d) method of mixing, e) the duration of mellowing period (time between mixing and Placement,) curing environment (moisture conditions, chemistry, and temperature), and) age (Umer, 2014).

Monohydrate lime was significantly higher than the strengths of the soil treated with calcitic slaked lime. Unslaked lime was more effective than hydrated lime in improving strength. At lime contents above two percent, the coarser unslaked limes were more effective because the fine limes caused significant flocculation, resulting in a reduction in the density of the soil. Chemical stabilization is an alternative to the improvement method for poor soils. Chemicals such as lime improve the low load-bearing capacity of poor soils and lower the plasticity index and percent swell of highly expansive soil(Lemi A, 2015). The impact and effectiveness of the current study of lime, one of the chemical stabilizers, on highly expansive and low load bearing soil is evaluated which is mainly based on laboratory test results of free swell, Atterberg limits, standard compaction, consolidation test, and unconfined Compressive strength.

2.7.5 Soils suitable for lime stabilization

Medium, moderately fine, and fine-grained soils have been found to successfully react with lime, resulting in a decrease in the flexibility and swell potential of expansive soils and an increase in their workability and strength qualities. According to a study, lime may be a useful stabilizer in soils with clay contents as low as 7% and plasticity indices lower than 10. According to the National Lime Association, lime should have a plasticity index of at least 10 to be considered as a possible stabilizer, however, the U.S. Army Corps of Engineers advises lime to have a plasticity index of at least 12 to successfully stabilize it.

According to AASHTO categorization, soil types A-4, A-5, A-6, and A-7, as well as certain A-2-6 and A-2-7, are acceptable for lime stabilization. Although benefits have been found for lower PI silty soil with less clay, a minimum clay content of ($\leq 2\mu$) of around 10% and a plasticity index of more than 10 are preferred (Umer, 2014).

2.7.5.1 Engineering properties of lime stabilized soils

In general, all lime-treated fine-grained soils show lower flexibility, reduced shrink-swell, and increased workability. But not every soil exhibits a significant increase in strength. Pozzolanic reactivity is the cause of this increase in strength. The type of soil, the type of lime used, the amount of lime used, and the curing conditions, such as time, temperature, and moisture, all affect how much the physical attributes of the soil are improved.

Lime-treatment results in both immediate and long-term effects on soil properties. These discussions of the engineering properties of lime treated soils are divided into two categories. These are immediate (uncured) and long-term (cured) strength properties. The curing period refers to a time when temperature and moisture are sufficient to provide an adequate environment for pozzolanic strength gain (Umer, 2014).

Uncured mixtures; The immediate effects of lime treatment on soils that are suitable for lime stabilization are due to the mechanism of cation exchange, Ca (OH)2 adsorption to the clay surfaces, and some extent rapid development of pozzolanic products. The level of physical property changes in the soil which result from lime treatment is quite soil dependent. However virtually all fine-grained soils, regardless of soil lime reactivity, derive some level of physical property consistency improvements through lime treatment as reflected by changes in atterberg limits and changes in volumetric measurements due to moisture fluctuations(Umer, 2014).

Cured mixtures; The most important effects of long-term curing are the development of pozzolanic products. The development of more pozzolanic products results in more glue to hold the particles of the soil together a mineralogical change favorable to greater strength. One of the most important engineering properties of soils to be considered in cured mixtures is the unconfined compressive strength of soils. The unconfined compressive strength of soils is one of the most widely used measures of the shear strength of lab-fabricated lime-stabilized soils.

This is because the UCS is a measure of compressive shear strength and is presented in engineering units that can be used in engineering calculations, analysis, and design.

The unconfined compressive strength of lime-treated soils increases with the increase of the curing period(Umer, 2014). Lime is one of the most utilized chemical stabilizers used for fine-grained soils, especially for clay minerals. In most literature and reference books, the stabilization of expansive soils with lime shows:

- ✓ Reducing the liquid limit, plasticity index, shrinkage limit, and linear shrinkage of the problematic soil
- ✓ Increasing the bearing capacity of the soil
- ✓ Increasing the unconfined compressive strength of the soil

- ✓ Improving the workability of the problematic soil
- ✓ Reducing the degree of expansion and swelling

In general, all lime-treated fine-grained soils exhibit a reduction in plasticity, decreased shrink-swell and improved workability. Lime-treated soils have also shown a reduction in maximum dry density with an increase in the optimum moisture content. Lime-treated soils have also improved the strength of the soil through the pozzolanic reactions of the lime soil mixtures.

2.7.6 Saw Dust Ash stabilization

Sawdust is a by-product waste in the form of fine granules of wood that is generated during woodworking operations like sawing, milling, planning, drilling, and sanding of timber in timber industries that process timber to be supplied for various allied manufacturing industries. The majority of this sawdust is utilized in particle boards, but it can also be used to make wood pulp, mulch, charcoal briquettes, and as fuel.

In sawmills, where they are generated in huge quantities, they are also used in the sawdust burners to produce heat for milling operations. The resultant end product is SDA or wood ash (WA). SDA has been used in the production of concrete and, more recently, in the stabilization of soil and stabilized soil blocks(James, 2019). Due to the significant amount of silica in SDA, it is a pozzolanic substance. A pozzolan is a finely divided siliceous or aluminous substance that, when combined with calcium hydroxide and water, produces cemented products. Therefore, it would make sense to combine SDA with a major binder rather than using it as a standalone stabilizer(James, 2019).

One such byproduct from the timber and wood-cutting industries is sawdust. It is the residue powder left after the combustion of sawdust. The property of sawdust ash is highly dependent on several factors including type and source of wood, operating parameters (especially combustion temperature), and ash collection methods. Saw dust by itself has little cementitious value but in the presence of moisture it reacts chemically and forms cementitious compounds and attributes to the improvement of strength and compressibility characteristics of soils. So, to achieve both the need of improving the geotechnical properties of clays and also to make use of industrial wastes, the present experimental study will be taken up.

The chemical makeup demonstrates that sawdust ash has an adequate quantity of silica and alumina. Saw dust ash has pozzolanic qualities due to the presence of silica, which makes it a valuable cementitious material(Khan & Khan, 2016). This paper aims to find a useful way to modify the geotechnical properties of soil for better field construction by using sawdust ash. Therefore, they can be used as cheap stabilizers to reduce the construction costs of civil engineering projects. The chemical composition of sawdust ash is given in the table below.

Table 2. 11 Chemical Properties of SDA(Tyagher et al., 2011)

Conistetuents	%by weight
SiO2	67.95
Al2O3	4.29
Fe2O3	2.15
CaO	9.47
MgO	5.84
MnO	0.01
Na2O	0.06
K2O	0.11
P2O5	0.46
SO3	0.56

Generally, there are a lot of tons of wood ash produced annually in Ethiopia. This amount of wood ash has been exposed to the environment as a waste product. Therefore, waste management should be the big deal to control pollution of the water bodies and to avoid bad smells in the environment. Uncontrolled exposed wastes of Saw dust ash will cause the health effect on humans and animals. It is either dumped at certain locations and allowed to decompose in the open atmosphere or is burnt in the open air by the various timber processing plants. These disposal techniques are used despite the environmental risks they pose to air and water bodies because, most of the excess sawdust produced is not reused either in agriculture or in the production of briquettes, despite being made freely available to anyone who would make use of it.

Waste management programs are required to dispose of wood ash in a cost-effective and environmentally acceptable. The management of saw dust ash is an important factor in the environmental and economic analysis of wood burning and to collect the dried area. Effects of seepage from permeable landfill sites which are dangerous to groundwater, the threat of polluting surface water resources particularly small tributaries with low water flow rates, and other environmental issues are problems to be solved by effective utilization ash of the sawdust in the construction industry.

When saw dust ash seeps into the water body, it causes water born disease, especially in rural areas. In rural areas population fetch water from the river for drinking purposes but in urban populations the water lines will be polluted due to seepage and leakage also their health affects by bad smells like TB and other breathing organ diseases. As we discussed above, if we use sawdust ash a soil stabilizer, we can mitigate all the above problems.

2.7 Summary of literature review

Poor soils have been encountered by geotechnical engineers all over the world during developmental activities and execution of infrastructure projects. They have always posed a problem for the engineers in one way or the other, either during or after construction. In order to make the soil a suitable engineering material, its properties need to be modified or engineered to suit the requirements of a particular infrastructure project. In recent times, addition of solid wastes has also been practiced to improve soils of various types to suit varying requirements. Solid wastes can originate from industrial wastes an increased acceptance in their utilization and soil modification. So, this study was conducted by using easily available, low cost and environmentally friendly material from industrial wastes that were utilized for the treatment of expansive soil. The following idea was summarized from the previous researches done.

(Nalawade & Jadhao, 2019) study on Stabilization of expansive soils with lime. In this study, hydrated lime, of percent varying from 2%,4%,6%, 8%,10% and 12% percent (by dry weight of the soil) was used for stabilization of expansive soils. The MDD, UCS, and CBR increased while the optimum moisture content (OMC) and free swell index (FSI) decreased with addition of lime. Generally, the results show the stabilization of expansive soil using lime was reduced plasticity, swell potential, and increased the strength of weak soil. However, this study was used only lime to stabilize the expansive soil.

(Butt et al., 2016) evaluated the potential of wood ash to stabilize clayey soil with SDA, in 0%, 4%, 8%, and 12%, The results showed the addition of saw dust ash to clay soils helps to reduce liquid limit and plasticity index and MDD while UCS, OMC and CBR values are increased with increasing percentage saw dust ash. Thus, the studies showed that SDA was economical and enough to improve the strength of the soil. The research was limited to considering the effect of SDA only without the combination of lime.

However, there was limited research done on the stabilization of expansive soil by combination lime and SDA especially in Ethiopia. Thus, this work aims at the evaluating the potential of SDA used in combination with lime and analyzing the strength benefit achieved, when used for foundation. This will encourage the use of lime and SDA as stabilizer in road construction and foundation of building structure of expansive soil.

CHAPTER THREE

3.MATERIALS AND METHOD

3.1 Introduction

In this chapter, the materials used and the method employed for the study were described and laboratory experiments carried out to achieve the aim of the research are described.

3.2 Study Area

The study area is located in the southeast of Addis Ababa, Akaki kality sub-city of, Ethiopia. One of the sub cities of Addis Ababa is Akaki kality sub city which encounters the largest plot area of 118.08 square kilometer, with a population of 195,273 and is located at 8^{0} 55'38'' latitude and 38^{0} 46'5'' longitude with an elevation of 2161m.

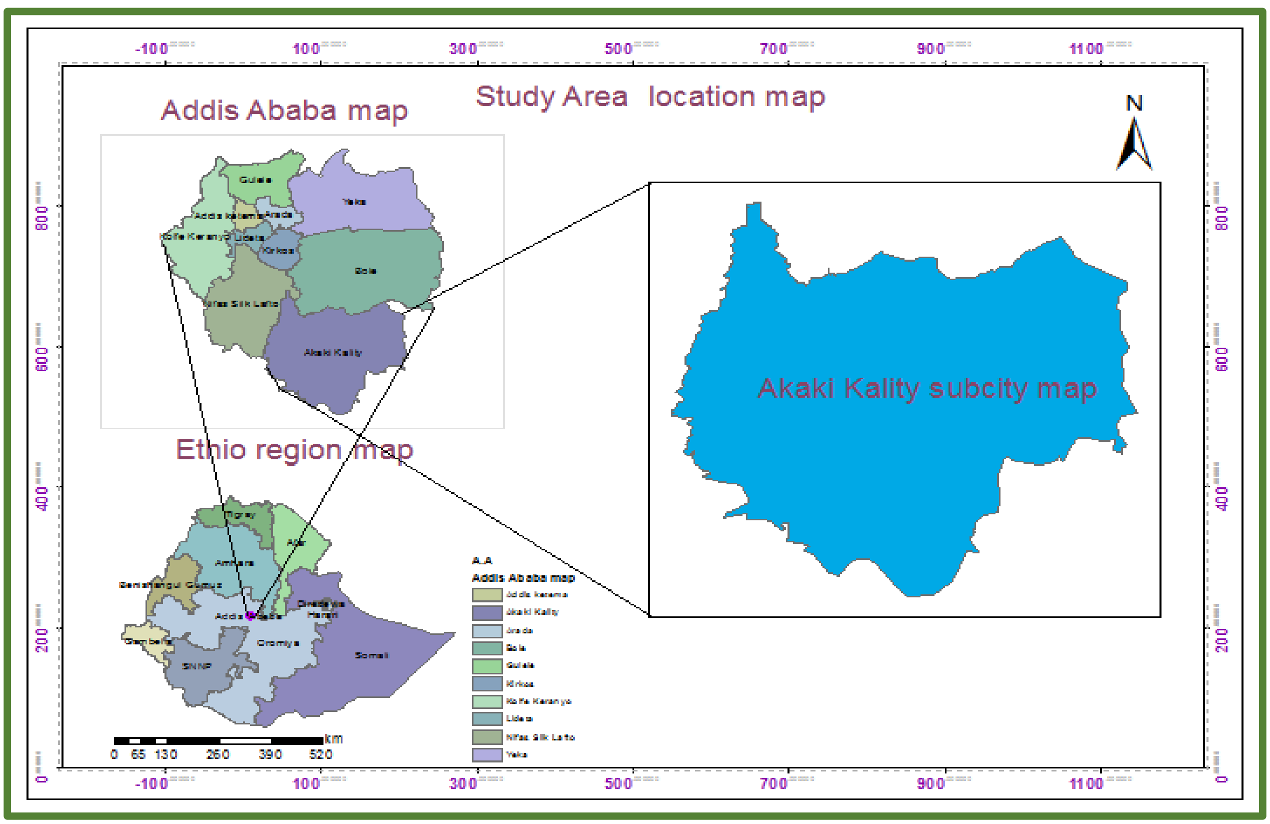


Figure 3. 1 Study area map

3.3 Materials

3.3.1 Expansive soils

The Expansive soil sample used for this research work is collected from Addis Ababa, Akaki Kality Sub City around Tullu Dimtu condominium area. The soil is grayish-black highly plastic clay. Disturbed or remolded soil samples were collected from two test pits at a depth of about 2.0m below the natural ground level to avoid the inclusion of organic matter and transported by plastic bags to the soil laboratory where the actual experimental work on the samples was carried out. The Climate of Addis Ababa is WoinaDega type(Gadise Tesema, 2015). The Rainfall has a unimodal pattern, one distinct rainy and dry season. The dry season is from October through May and the wet one is from June to September. The rainfall peak is in August. The long-term mean annual rainfall observed at Addis Ababa Observatory is 1254mm(Gadise Tesema, 2015). The maximum temperature of Addis Ababa ranges between 20^0c (in the wet season) to 25^0c (in the dry season), while the minimum falls between 7 - 12^0c in the year. This indicates that the daily variation of temperature is highly pronounced. Full properties of the soil is addressed in the method section. Soil sampling from the test pit is shown in the figure below.

(a) (b)

Figure 3. 2 (a) and (b)Photo of soil samples from Test pit one and Test pit two

3.3.2 Hydrated Lime

Hydrated lime is one of the additives which are widely used in the stabilization of expansive soil. The hydrated lime used in this study was purchased from Merry Sanitation Chemicals Manufacturing PLC. which is located in Sebeta, Ethiopia.

3.3.3 Saw Dust Ash

Saw dust was obtained from locally available saw mills or timber factories. In this study saw dusts are taken from Mulugeta Chala timber factory that is located in Sebeta, Ethiopia. The collected sawdust was air dried to facilitate burning and burned /converted into ash by open-air burning on a metal sheet to prevent the ash from mixing with dust materials prepare sawdust ash. Then, saw dust ash was collected into the sack and transported to the laboratory. After collecting saw dust ash the burned saw dust ash was grounded by a milling machine and sieving was done through a sieve of 600μm to remove unnecessary wastes and get very fi

Figure *3.3* Process of saw dust ash (SDA) sample preparation

3.4 Method

The soil sample in this study was collected from Addis Ababa, Akaki kality sub-city area at 2m from two test pits to conduct laboratory tests. All experimental tests was conducted following with the recommendations of ASTM. The tests were performed on the natural black cotton soil, black cotton soil mixed with lime and black cotton soil mixed with combined sawdust ash, and the amount of lime. The engineering property of the soil sample including particle size distributions of the soil sample and index property of soil before stabilization will be determined. Disturbed or remolded soil sample was used for the unconfined compressive strength (UCS) test. Disturbed samples will be used for classification, grain size distribution, atterberg limits, and frees well test, Compaction test, and also for consolidation test value determination. Sampling were done by excavating the soil manually.

3.5 Laboratory Test

Laboratory tests for this study work was conducted for expansive soil classification and evaluation of the effect of lime and saw dust ash. After the representative soil samples were collected, the following laboratory tests will be conducted.

Grain size analysis (sieve size and hydrometer)
Atterberg limit (liquid limit, plastic limit, plasticity index
Free swell test
Specific gravity
Compaction test
UCS test
Consolidation test

Soil Classification will be performed on the sample by conducting index property and grain size analysis tests following with ASTM (American Society for Testing and Materials). Index property tests conducted includes the Atterberg limit test, grain size analysis, free swell test, and Specific Gravity testing. The strength development of the soil will be studied using the Compaction test, Unconfined Compressive Strength test, and consolidation test. The Mixing will be carried out manually and the tests will be conducted as per standard procedures.

Table 3. 1 laboratory test conducted for expansive soils

No.	Types of test	Standard
1	Atterberg limit	ASTMD 4318
2	Grain size analysis	ASTMD 422
3	Free swell test	Holtz and Gibbs, (1956)
4	Specific gravity	ASTMD 854
5	Compaction test	ASTMD 698
6	Consolidation test	ASTMD 2435
7	UCS test	ASTMD 2166

3.6 Sample Preparation

This study is aimed to accomplish the proposed objectives related to the concept of improving the engineering properties of expansive soil. Laboratory tests were conducted on disturbed and remolded soil samples.

Disturbed samples were air dried and sieved with different sieve sizes after pulverizing depending on the requirement of specific test procedures. The tests were first carried out on black cotton soil without adding stabilizers.

In this study, the content of the stabilizer is defined in the ratio of the weight of lime and saw dust ash to the dry weight of the soil sample. Based on proportions used in previous studies to obtain the optimum dosage of lime and saw dust ash, first expansive soil and lime were mixed alone in the percentage of 2%,3%,4%, and 5%(ASTM, 2000)of dry weight soil, and data analysis was done on the result obtained from unconfined compressive strength test. Then the optimum dosage of lime was obtained from the laboratory test result and was chosen based on the value of maximum values of UCS test result.

Similarly, the soil sample was mixed with optimum values of lime and saw dust ash at 4%,8%,12%, and 16%(Ikeagwuani et al., 2019) and the Atterberg limit test, free swell test, specific gravity, compaction tests, Consolidation test, and UCS tests were carried out.

Then the effect of lime and saw dust ash on some geotechnical properties of natural soils were assessed according to the following table showing the mixing proportions.

Table 3. 2 Mix proportion of Expansive soils, saw dust ash and lime

Sample	ES%	SDA%	Lime%	Total %age
expansive soil	100%	-	-	100%
expansive soil-lime	98%	-	2%	100%
	93%	-	3%	100%
	96%	-	4%	100%
	95%	-	5%	100%
ES-lime -SDA	ES	4%	Optimum %age of lime	100%
	ES	8%		100%
	ES	12%		100%
	ES	16%		100%

3.6.1 Moisture Content

The test is conducted following ASTM D 2216 Small representative samples of the natural soil are obtained and oven-dried at 105 ± 5°C for at least 12 hours. The samples were then reweighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content of the soil a dry weight basis.

3.6.2 Grain size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method is used to determine the distribution of the finer particles as per standard. For grain size analysis the procedure given in ASTMD 422 is used.

3.6.3 Atterberg Limits Testing

The test is a consistency limit identification test based on of moisture content. It includes, the determination of; the liquid limits, plastic limits, and the plasticity index for the natural soil and the soil-lime and saw dust ash mixtures. An Atterberg limit test was conducted according to the ASTM D-4318 standard testing procedures. The soil sample for liquid limit is air dried and 100g of the material passing through the No. 40 sieve was obtained and thoroughly mixed with water to form a homogeneous paste on a flat glass plate. A portion of the soil water mixture is then placed in the cup of the Casagrande apparatus, leveled off parallel to the base, and divided by drawing the grooving tool along the diameter through the center of the hinge. The cup is then lifted and dropped by turning the crank until the two parts of the soil come into contact at the bottom of the groove. The number of blows at which that occurred was recorded and a little quantity of the soil was taken and its moisture content determined. The values of the moisture content determined and the corresponding number of blows are then plotted on a semi-logarithmic graph and the liquid limit is determined as the moisture content corresponding to 25 blows. The same procedure is also carried out for the soil treated with varied contents of lime and saw dust ash.

Plastic Limit: Plastic limit may be defined as the minimum moisture content at which the soil remains in plastic condition. A portion of soil used for the liquid limit test is retained for the determination of the plastic limit. The ball of soil is molded between the fingers and rolled between the palms of the hand until it dried sufficiently, even though the soil is already relatively drier than the ones used for liquid limit. The sample is rolled into a thread between the first finger and the thumb. The thread is then rolled between the tip of the fingers of one hand and the glass. This continued until the diameter of the thread is reduced to about 3mm.

The crumbled soil is then put in the container and the moisture content is determined. The same procedure is also carried out for the soil treated with varied contents of lime and saw dust ash.

Plasticity Index: The plasticity index of the natural soil and the soil treated with lime and saw dust ash is the difference between the liquid limits and their corresponding plastic limits. The plasticity indexes of the samples are calculated as:

PI =LL –PL where, PL = plastic limit, LL = liquid limit, PI =plasticity index

3.6.4 Free Swell Tests

The test includes the determination of the free swell for the natural soil and soil treated with lime and saw dust ash mixture. This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs, (1956) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of the expansiveness of the soil sample. The procedure consists of pouring very slowly 10 cubic centimeters of that part of the dry soil passing the No. 40 sieve into a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately 24 hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted. Finally, the free swell value is calculated using Equation below.

$$\text{Free swell} = \frac{Vf-Vi}{Vi} * 100 \quad (3.1)$$

Where: V_i = initial volume, V_f = final volume

3.6.5 Free Swell Index

The test includes the determination of the free swell index of the natural soil and the soil with lime and saw dust ash mixture. Two samples of oven-dried soil 10cc each, passing through a 425-micron sieve are taken. One is put in a 100 cubic centimeters graduated glass cylinder containing kerosene. The other sample is put in a similar cylinder containing distilled water. Both the samples are left undisturbed for 24 hours and then their volumes are noted. Then the free swell index is determined using the equation below. The same procedure is also carried out for the treated soil with increment lime and saw dust ash content.

$$\text{Free swell index (\%)} = \frac{Vk-Vw}{Vk} * 100 \quad (3.2)$$

Where: V_w = final volume in water

V_k = final volume in kerosene

3.6.6 Free Swell Ratio Test

In this study,10gm oven-dried soil passing through 425 microns is added to 100ml of distilled water in a jar and another 10gm of the same sample is added to 100ml of Kerosene. After 24 hours, sediment volumes of samples are measured to determine the free swell ratio. The free swell ratio is the ratio of change in volume in water to change in volume in kerosene after 24 hours. Then the free swell ratio is determined using the equation below. The same procedure is also carried out for the treated soil with an increment of lime and saw dust ash content.

Free swell ratio = $\frac{Vw}{Vk} * 100$……………………………………………….………...(3.3)

Where V_w =final volume in water,

V_k = final volume in kerosene

3.6.7 Specific gravity

The soil's specific gravity (Gs) is the measure of the heaviness of the soil particles. It is defined as the mass ratio in the air of a given volume of soil particles to the mass in the air having an equal volume of gas-free distilled water at a stated temperature. This test was conducted following ASTM D 854 testing procedure.

3.6.8 Compaction

This test includes the determination of the maximum dry density and the optimum moisture content according to ASTMD 698 standard testing procedures. The test is conducted for both the natural soil and soil treated with lime and saw dust ash mixture. By varying the moisture content for each trial, air-dried fresh soil samples of about 2.5 kg is used. Every sample is then compacted into the 944 cubic centimeters of mass; in three layers of approximately equal mass with each layer receiving 25 blows. The blows are uniformly dispersed over the surface of each layer. One small representative sample is then taken from the top, middle, and bottom of compacted soil for calculating the moisture content. The bulk and dry densities are then calculated for each compacted specimen. The values of the dry densities are plotted against their respective moisture contents; MDD is deduced as the maximum point on the resulting curves. The corresponding value of moisture contents at maximum dry densities, which is deduced from the graph of dry density against moisture content, gives the optimum moisture content OMC.

3.6.9 Unconfined compressive strength

According to ASTM D 2166-00, the unconfined compressive strength test was conducted. The sample was remolded with a 38mm diameter and height of 76mm which was prepared with optimum moisture content determined from standard compaction test for both untreated and treated soil samples. This test is a fast and economical means of approximating the shear strength at shallow depths without any confining pressure, and is subjected to an axial compressive load until failure occurs. This technique only works with cohesive materials. In this study, the sample is remolded using optimum moisture content and dry density obtained from standard compaction test. After the samples were extracted from the mold they were covered with a plastic membrane and cured for seven days.

Figure 3. 4 photographic view of UCS test

3.6.10 One Dimensional Consolidation test

When a laterally constrained soil specimen is subjected to various vertical pressures, a consolidation test is conducted to measure the amount and rate of volume reduction that occurs. This test is conducted according to the ASTM D2435 standard testing procedure for soils' one-dimensional consolidation characteristics. The samples were prepared with moisture content determined from the standard compaction test. The initial reading was taken immediately and the final reading was taken after the specimen stopped swelling. Then loading and Unloading of the loads are applied and reading is taken for 24 hours. From the recorded data, the consolidation curve (relationship between pressure-void ratio) can be sketched. This information can be used to calculate the compression index, the recompression index, and the coefficient of consolidation.

Moreover, for one-dimensional consolidation the odometer test was conducted to determine, parameters such as compression index (Cc), recompression index (Cr), coefficient of consolidation (Cv), and coefficient of volume change (mv) for test pit one of expansive in the study area.

Figure 3. 5 photographic view of Consolidation test

CHAPTER FOUR

4.RESULTS AND DISCUSSIONS

In this chapter laboratory test results are presented and their analysis is briefly discussed. The relevant engineering property of the soil is evaluated both for natural and treated/stabilized soil samples with lime and saw dust ash separately for test pit one (TP-1) which is weaker than test pit two (TP-2) depending on MDD and UCS test result. The tests include the Atterberg limits test (liquid limit, plastic limit, and plasticity index test), moisture density relationship /compaction test, the strength tests: -unconfined compressive strength UCS test (uncured, 7-days cured) and one-dimensional Odometer, consolidation test is conducted on the natural soil and soil treated with different percentage mixture of lime and saw dust ash according to ASTM. Tabular and graphical analyses were adopted to determine whether stabilization affects the properties of the soil samples.

4.1 Properties of Materials Used in the study

4.1.1 Natural Expansive Soil

The results of the tests conducted for identification and/or determination of properties of the natural soil before adding a mixture of lime and saw dust ash are presented in Table 4.1. For this study, TP-1 soil was taken for treatment because it is weaker than TP-2. The soil is grayish-black in color. As shown in Figure 4.1 on the particle size distribution curve almost 93.1% of the soil is passing through the No. 200 sieve; it exhibits a liquid limit of 104% a plastic limit of 43.49% and a plasticity index of 60.51% for TP-1. Liquid limit of less than 35% indicates low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity, and between 70% and 90% very high plasticity. Hence, these values indicate that the soil is highly plastic clay. Accordingly, the soil falls under the A-7-5 soil class based on AASHTO and CH based on the USCS soil classification system. This type of soil is typically categorized as a material with weak engineering properties to be used as a sub-grade material and any foundation construction work.

Results that are related to the swelling characteristics of the soil also indicate that the soil is highly expansive clay with a free swell of almost 150% and a free swell index of 138.1%. The maximum dry density of soil is 1.31 g/cm3, optimum moisture content of 34.95%.

The undrained shear strengh value for the remolded sample is 65.477 (kPa). A one-dimensional consolidation test is also conducted on remolded soil samples, according to this test result, the compression index Cc, of the soil is, 0.651, the coefficient of consolidation, Cv, ranges from 0.00284-0.00528cm^2/min., and the coefficient of volume change, Mv, of the area is $2.42*10^{-4}$ m^2/KN, recompression index 0.355 depending on these data the soil has very compressibility properties. Hence, the soil was indicated to be highly plastic expansive clay with low bearing capacity. Therefore, the soil requires initial modification and/or stabilization to improve its workability and engineering property.

Table 4. 1 Geotechnical Properties of natural expansive soil for TP-1 and TP-2

Properties	TP-1	TP-2
Natural moisture content %	40.85	37.47
Percentage passing No. 200 sieve %	93.1	91.2
Liquid limit %	104	92
plastic limit %	43.49	41.27
Plasticity index %	60.51	50.73
AASHTO soil classification	A-7-5	A-7-5
USCS soil classification	CH	CH
Specific gravity	2.714	2.729
Free swell %	150	100
Maximum dry density g/cm^3	1.31	1.33
Optimum moisture content %	34.95	36.5
Unconfined compressive strength (kPa)	65.477	81.06
Free swell(%)	150	120
Free swell index(%)	138.1	100
Free swell ratio	2.4	2
Color	Grayish black	Grayish black

Depending on the above test result determined for both test pits, from the two test pits, test pit one (TP-1) sample was taken to investigate the effect of lime and saw dust ash because test pit one is weaker and slightly higher expansive than test pit two by depending on the result obtained. The detailed geotechnical properties of both soil samples were discussed below and given in the Appendix part. Moreover, discussion on stabilization of test pit one(TP-1) with lime and saw dust ash is discussed in the next section.

4.1.2 Particle size distribution for natural soil

The grain size analysis test was performed in accordance with ASTM D 422 standard test method. The basic use of a soil classification system is to identify the soil type as well as determination of the amount and distribution of the particle sizes in the soil. The distribution of particle sizes larger than 0.075 mm (No. 200 sieves) is determined by wet sieving and particle sizes smaller than 0.075 mm are determined by hydrometer test. Thus, wet sieving was done by taking 1000g of natural soil, by washing on a sieve size of 75µm, then a hydrometer test is conducted on 50gm of the soil samples passing sieve No.200 and soaked in sodium hex-meta phosphate chemical solution for 24 hours. The tabular laboratory results are presented in the appendix and the particle size distribution curve is shown in Figure 4.1.

Figure 4. 1 Particle size distribution curve of the Expansive soil for TP1 and TP2

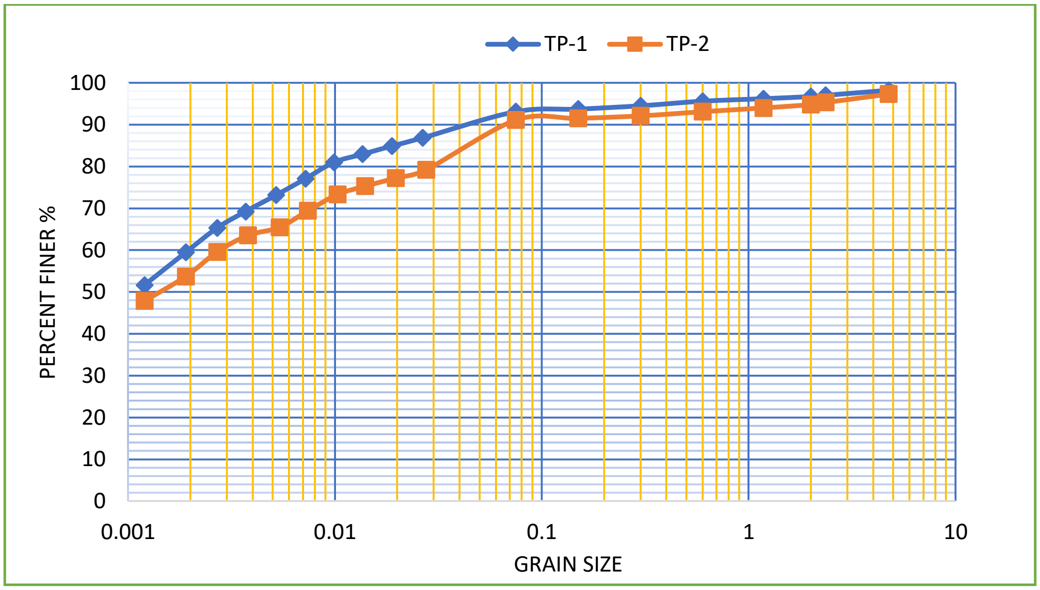


Table 4. 2 Particle size proportion of TP-1 and TP-2 soils

Particle size	TP-1	TP-2
Clay and silt(%)	93.1	91.2
Sand(%)	5.1	6
Gravel(%)	1.8	2.7

4.1.3 Atterberg limit test of natural soil

The test was performed to determine the liquid and plastic limits of fine-grained soil. The test was performed using ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. Atterberg limit test results of natural soil are high and are not recommended for subgrades and foundation construction use based on the standards, thus the soil needs treatment.

Table 4. 3 Summary of Atterberg limit test results of the natural expansive soils

No.	designation	Liquid limit (%)	Plastic limit(%)	Plasticity index (%)
1	TP-1	104	43.49	60.51
2	TP-2	92	41.27	50.73

The liquid limit, the plastic limit, and plasticity index of natural soil samples are indicating that 104 %, 43.49 %, and 60.51 % respectively for Test pit 1 (TP-1) 92 %, 41.27 %, and 50.73 % respectively for Test pit 2 (TP-2). Both soil samples fail under the A-7-5 soil class based on the AASHTO system of classifying soil which has poor engineering properties. The result obtained from the plastic index indicates that the soil possesses high swelling (degree of expansiveness) properties since its plasticity index is greater than 35%, thus the soil needs treatment for the construction of any foundation on it.

4.1.4 Specific gravity of natural soil

The laboratory test was performed to determine the specific gravity of soil by using a water pycnometer a soil sample passing No. 10 sieves and oven-dried at 105 degrees centigrade. Specific gravity is the proportion of the mass of a specified volume of soil to the mass of a specified volume of gas-free distilled water at a specified temperature. For this study, an average of the soil sample's specific gravity is given in the Appendix and Table 4.4 below.

Table 4. 4 Summary of Specific Gravity test results of the natural expansive soils

Sample No.	TP-1	TP-2
1	2.717	2.725
2	2.710	2.732
Average(Gs)	2.714	2.729

4.1.5 Free swelling characteristics for natural soil

Free swelling tests (free swell, free swell index, and free swell ratio) are simple laboratory tests as discussed in chapter two, that are used to determine the expansion and swelling nature of expansive soil. Free swell tests consist of placing 10cc of dry soil passing a sieve size 0.425mm (No. 40) into a 100cc graduated jar filled with water and noting the swelled volume after 24-hour material settles, without any surcharge, to the bottom of a graduated cylinder. The distinction between the final and initial volume expressed as a percentage of the initial volume was measured.

Table 4. 5 Summary of swelling properties for natural soil

Free swelling	TP-1	TP-2
Free swell (%)	150	120
Free swell index (%)	138.1	100
Free swell ratio (%)	2.4	2

For TP-1, from the laboratory test result, as shown in the table above, the soil has a free swell of 150%, which indicates that it is highly expansive and a free swell ratio of 2.4, which also indicate that the soil possesses high swelling characteristics.

Furthermore, a free swell index of 138.1% shows, the soil can cause considerable damage to lightly loaded structures. Generally, the soil is poor engineering properties as a result it needs the possible treatment for the construction of subgrade and foundation construction.

4.1.6 Compaction test result for natural soil

Standard proctor compaction tests were performed on the soil to establish the relationship between the moisture content and dry density according to ASTM D-698. The natural expansive soil sample for (TP-1 and TP-2) has a maximum dry density of 1.31gm/cm3, 1.33 gm/cm3, and an optimum moisture content of 34.95%, 36.5% respectively as shown in figure 4.2. Furthermore, the detailed tabular form is shown in Appendix Part.

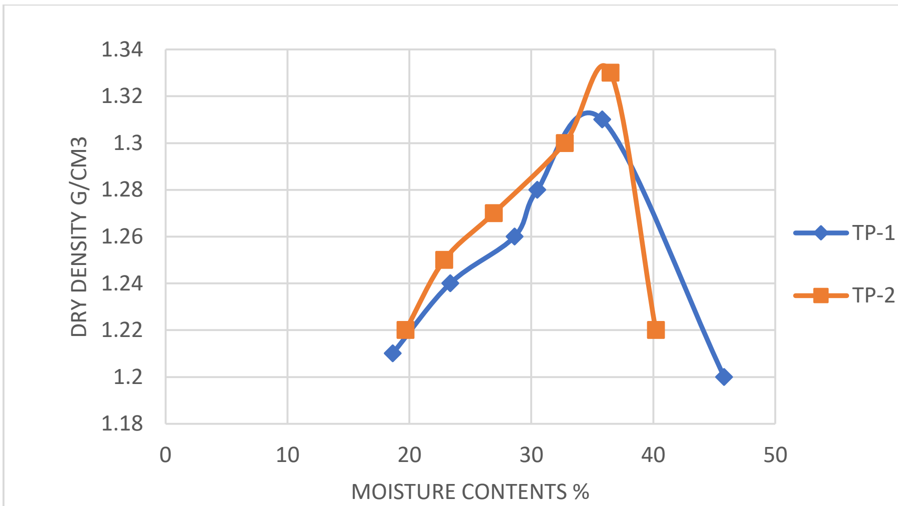


Figure 4. 2 Moisture-density relationship for the natural Soils

4.1.7 Unconfined compressive strength test on the natural soils

The unconfined compressive strength (UCS) test is an unconsolidated and undrained load test conducted for determining the unconfined compressive strength of cohesive clay soil samples where the lateral confining pressure is equal to zero during the test. UCS is the compressive stress that an unconfined cylindrical soil sample fails in a load test and is taken as the maximum load attained per unit area during loading. For UCS test specimens' height-to-diameter ratio is between 2, the undrained shear strength was calculated to be half of the compressive stress at failure. The soil sample for test pits one and two (TP-1 and TP-2) has a maximum unconfined compressive strength of 130.953kPa,162.110kPa and undrained shear strength of 65.477kPa, 81.055kPa indicates that the soil possesses properties of stiff clay respectively as shown in the graph below.

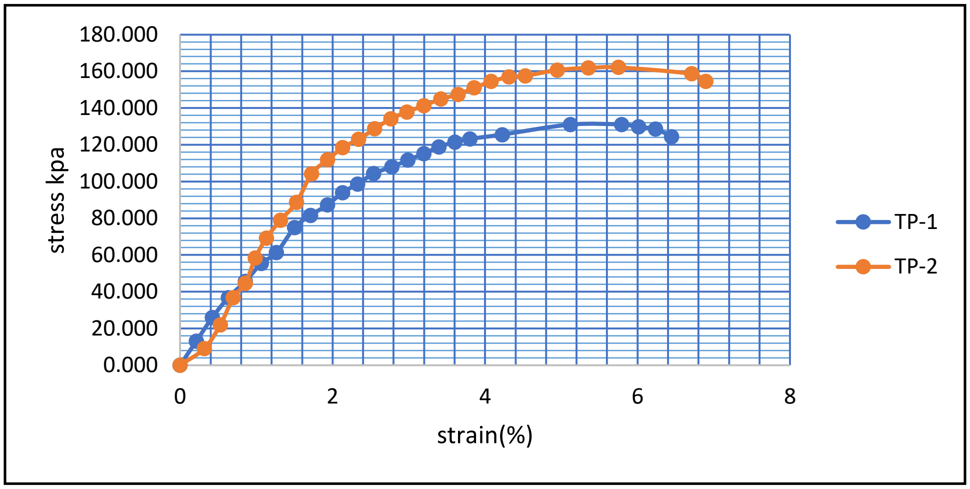


Figure 4. 3 Axial strain Vs Axial stress for natural soils

Table 4. 6 Unconfined compression strength test result for natural soils

Sample No.	Cu(kPa)
TP-1	65.477
TP-2	81.055

4.1.8 One dimensional consolidation test result for natural soil

One-dimensional consolidation test was conducted according to ASTM D2435. This test is run to find out how much and how quickly the volume of a laterally constrained soil specimen decreases under various vertical pressures. From the one dimensional consolidation test results to determine, Rate of consolidation under normal load, Degree of consolidation at any time, Pressure-void ratio relationship, Coefficient of consolidation at various pressures, Compression index, and recompression index enable estimates to be made of the behavior of foundation under load.

The consolidation test result of the study area shows that the consolidation coefficients range from (0.00284-0.00528 (cm^2/min)) and the coefficients of volume compressibility 0.0002422(m^2/KN) furthermore, the compression index and recompression index of 0.651 and 0.355 respectively, it has to indicate that the soil possesses very compressibility and consolidation behavior.

4.2 Effect of lime on compaction test.

The effect of lime on the optimal moisture content and maximum dry density of the expansive soil is shown in figure 4.4. As shown in the figure, maximum dry density decreases from 1.31g/cm^3 to 1.24g/cm^3, and optimum moisture content increases from 34.95% to 39.17% in soil samples with increased lime content from 0% to 5%.

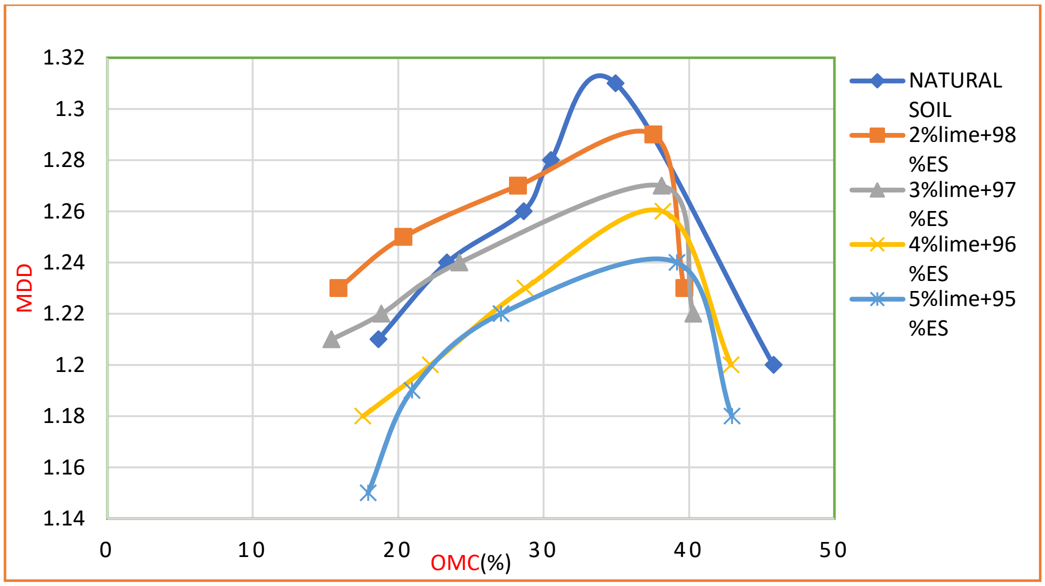


Figure 4. 4 Effect of lime on Moisture density relationship of soil sample

The sample's dry density has decreased as the amount of lime increased but, the moisture content has increased. The addition of lime changes the optimum moisture content (OMC) and maximum dry density (MDD) of soils because the effects of cation exchange and short-term pozzolanic reactions between lime and the soil results in flocculation and agglomeration of clay particles leading to textural changes.

- The decrease in the maximum dry density is mainly due to the lightweight and low specific gravity value of lime than that of replaced soil.
- The increase in the optimum moisture content was mainly due to the lime being finer than the soil. The more fines the more surface area, so more water is required to provide well lubrication.

4.3 Effect of lime on unconfined compressive strength test.

The test was performed using ASTM D 2166. The summary of laboratory results of the UCS with the addition of lime is tabulated in figure 4-5 below.

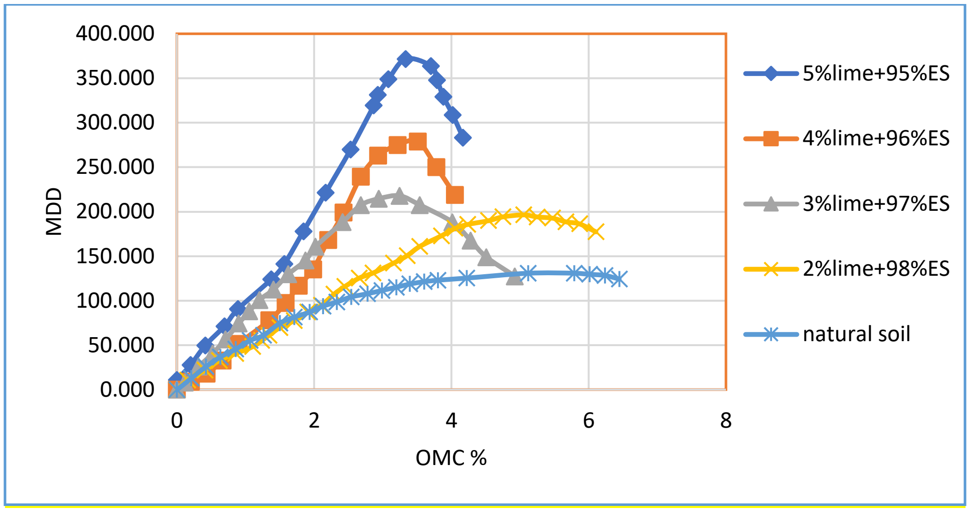


Figure 4. 5 effect of lime on unconfined compressive strength

As can be seen from figure 4.5 the UCS value has increased with the increased percentage of lime from 2%-5%. Undrained shear strengh value for the natural soil is 65.50 kPa. The UCS of the natural soil increased with lime contents up to 5%(185.864 kPa) and the optimum UCS value of soil treated with lime is when the lime content is 5%. This shows that The capacity to support the soil sample increased significantly with lime treatment on the UCS values. The reason for this improvement is due to the pozzolanic reactions of lime with soil.

4.3.1 Optimum lime Contents

The results of the tests conducted on the expansive soil samples mixed with different proportions of limes are given in Table 4.7. Then the optimum dosage of lime was obtained from the laboratory test result taken based on the value of maximum value UCS test result. The addition of lime in percentages ranging from 2% to 5 % by an increment of two decreases the maximum dry density from 1.31g/cm^3 to 1.24g/cm^3 for TP-1 which is the weaker test pit.

The addition of 5% lime resulted in an increase in the UCS and undrained shear strength value from130.953kpa to 371.728kpa and 65.5kPa to 185.864kPa respectively. Therefore 5 % of lime content by dry weight of the soil was taken as an optimum amount as shown in table 4.7 below.

Table 4. 7 Summary of soil properties stabilized with lime addition

Sample	UCS(kPa)	Remark
100%soil	65.5	
2%lime+98%soil	98.271	
3%lime+97%soil	109.258	
4%lime+96%%soil	139.426	
5%lime+95%soil	185.864	Optimum

4.5 Laboratory test results of stabilized Expansive soil

This section deals with evaluating the impact of the optimum amount of lime (5%) and saw dust ash on some expansive soil characteristics by using 4%,8%,12% 16% of proportions by dry weight of the soil. The impact of lime and sawdust ash on the characteristics of expansive soil was evaluated by performing the Atterberg limit tests, free swelling tests, specific gravity, standard compaction tests, unconfined compressive strength, consolidation test, and the results for each test were discussed in the following section.

4.5.1 Effect of lime and saw dust ash on Atterberg Limits.

The effect of lime and saw dust ash on the plasticity index of the soil is shown in Figure 4.6. As shown in the figure plasticity index decreased with an increment in lime and saw dust ash content. The liquid limit and plastic index of expansive soil decreased significantly from a value of 104% to 49.5% and 60.51% to 19.76% respectively with the addition of the optimum amount of lime and varying percentage of saw dust ash and changed from high swelling potential to medium swelling potential characteristic of black cotton soil.

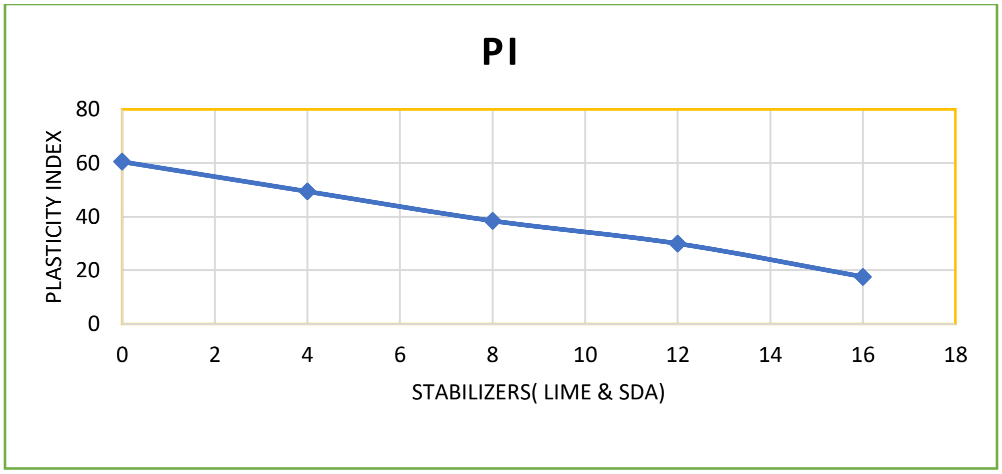


Figure 4. 6 Variation of plasticity index with the addition 5%lime and of different % SDA

Table 4. 8 Summary of liquid limit, plastic limit and plasticity index for the 5%lime and varying percentage of SDA with soils mixture

No.	Mix. Type	LL(%)	PL(%)	PI(%)
1	100% ES	104	43.39	60.51
2	5%LIME+4%SDA	92	42.59	49.41
3	5%LIME+8%SDA	77.8	39.3	38.45
4	5%LIME+12%SDA	64	36.07	29.93
5	5%LIME+16%SDA	49.5	31.96	17.54

In general, the soil's plasticity is decreased by the addition of lime and saw dust ash content. These effects are due to the partial replacement of expansive soil particles with lime and saw dust ash which is non-plastic material and flocculation and agglomeration of clay particles caused by cation exchange may be the other cause. Details of the Atterberg limit test results are shown in the Appendix part.

4.5.2 Effect of lime and saw dust ash on Swelling characteristics

A. Free Swell

The effect of lime and saw dust ash on the free swell of soil is shown in Figure 4.7. As it is shown in the figure below, the reduction in free swell is directly proportional to the quantity of lime and saw dust ash. The highest reduction in free swell is attained when the expansive soil is treated with 5%lime and 16%SDA which is an 80% reduction compared to the untreated soil sample.

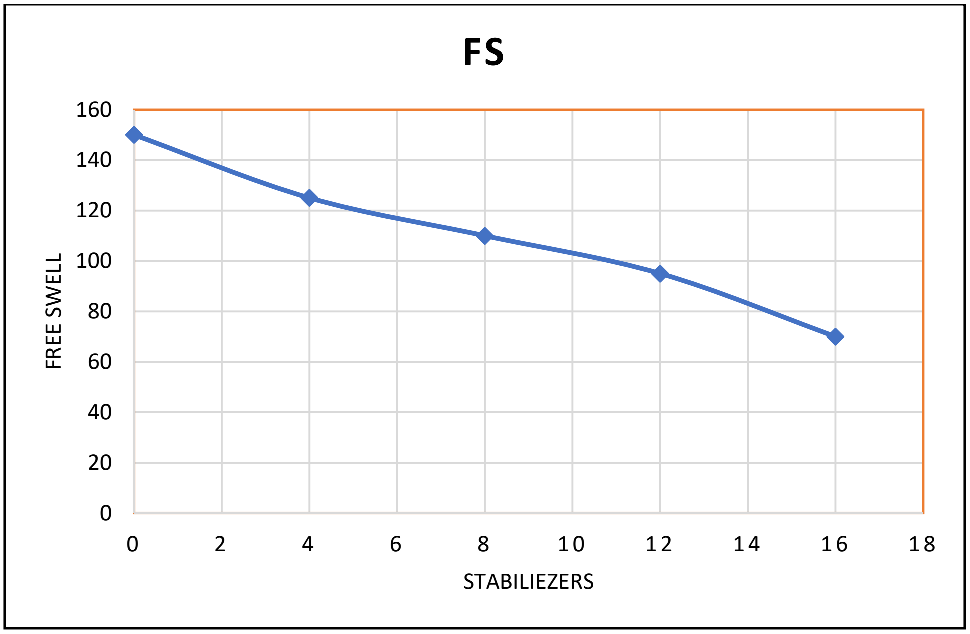


Figure 4. 7 Effect of addition of lime and saw dust ash on free swell of expansive soil

B. Free Swell Index

The effects of lime and sawdust ash on the free swell index of the expansive soil are shown in figure 4.8. The free swell index value decreased from 138.1% to 25.926% with an increased 5%lime and SDA content from 0% to 16% with 5% lime. From the analysis of test results the free swell index of expansive soil is 138.1%. According to swell classification based on the free swell index in Chapter two, the soil has very high degree of expansion. As the percentage of stabilizer increases, the free swell index value decreases and decreased up to 25.926% at 5%lime with 16% SDA and makes the soils to be moderate/medium swelling properties.

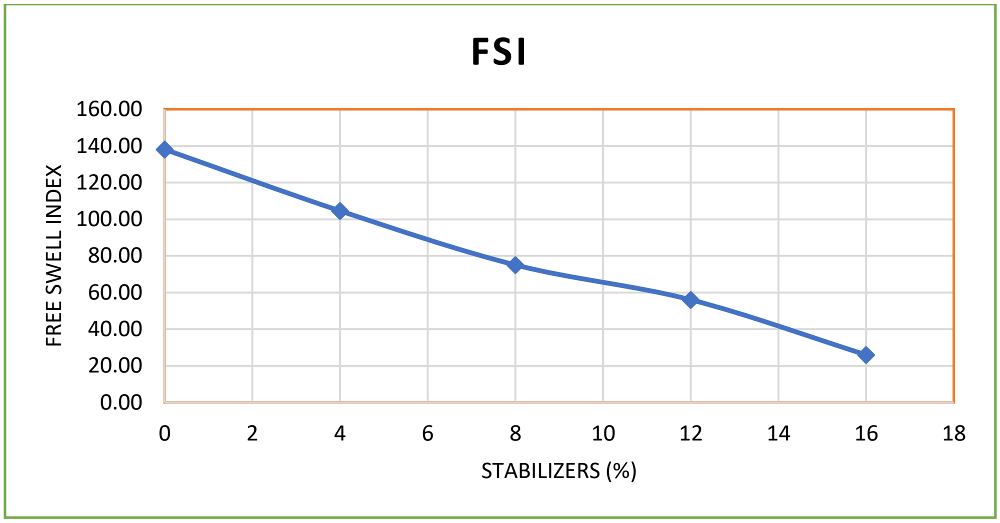


Figure 4. 8 Effect of addition of lime and saw dust ash on free swell index of expansive soil

B. Free Swell Ratio

As it is shown in Figure 4.9 when lime and saw dust ash is added to the soil the free swell ratio decreases. The free swell ratio decreases from 2.4 to 1.259 with an increased 5%lime and saw dust ash content from 0% to 16%. From the analysis of test results the free swell ratio of expansive swell classification depends on the free swell ratio in Chapter two, the soil is considered as high swelling soil. As the percentage of stabilizers increases, the free swell ratio value decreased up to 1.259 at 5%lime and 16% SDA and makes the soil low swelling potential characteristic.

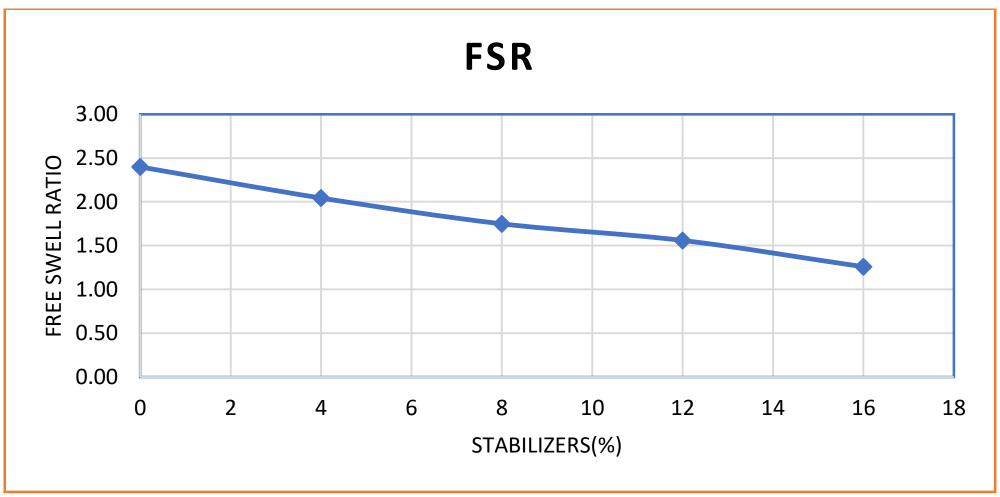


Figure 4. 9 Effect of addition of lime and saw dust ash on free swell ratio of expansive soil

4.5.3 Effect of lime and saw dust ash on Specific Gravity

The impact of lime and saw dust ash on the specific gravity of the expansive soil is shown in Figure 4.10 Specific gravity decreased from 2.714 to 2.590 with increased SDA content from 0% to 16% and 5%lime. As is seen in Figure 4.10, the reduction in specific gravity is direct relationship to the amount of lime and saw dust ash, this is due to the lightness of the lime and SDA which have specific gravity of 2.507 and 2.424 respectively.

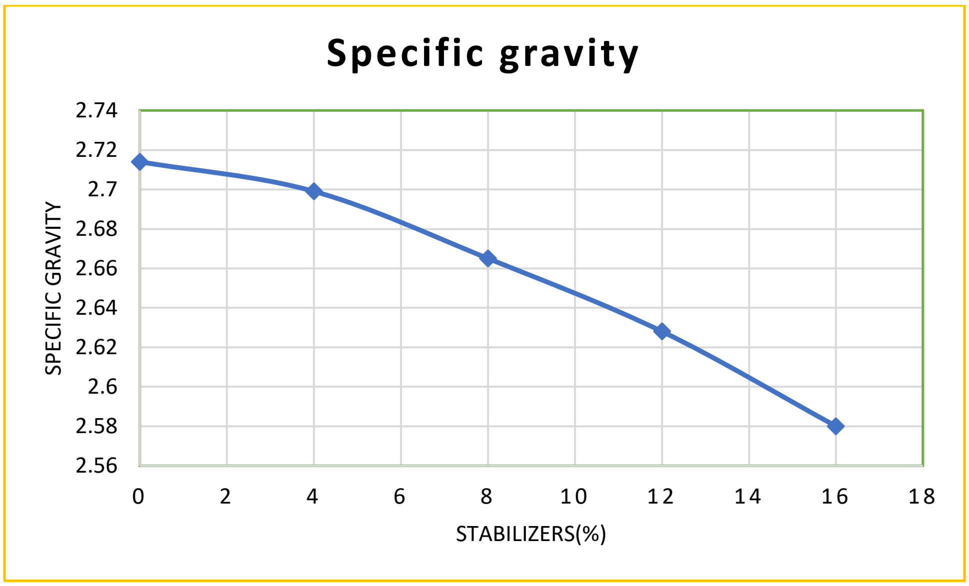


Figure 4. 10 Variation of specific gravity of soil stabilized with lime and saw dust ash content

4.5.4 Effect of lime and saw dust ash on compaction

The effect of lime and saw dust ash on the MDD and OMC of the expansive soil is shown in Figure 4.11. Air-dried and pulverized soil passing No 4 sieve was used for the compaction test carried out according to ASTM D 698. Optimum moisture content (OMC) and maximum dry density (MDD) were determined from the graph and the moisture content versus dry density graph was plotted. The corresponding results were plotted as shown on the graph in table 4.9 below.

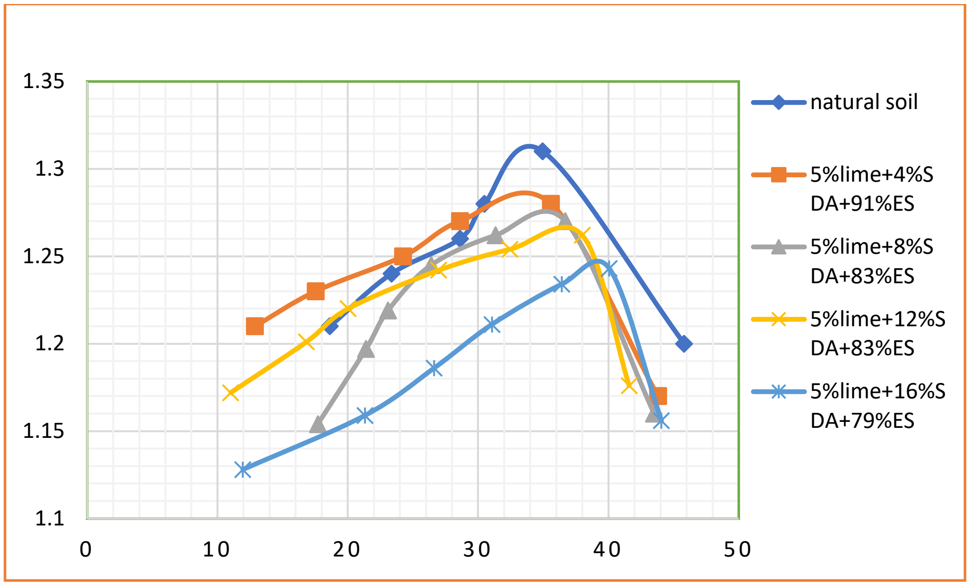


Figure 4. 11 Summary of compaction with different SDA contents and 5%lime samples

Table 4. 9 Summary on the effect of lime and SDA on MDD and OMC of expansive soil sample

No.	Mix. Type	MDD(g/cm^3)	OMC(%)
1	100% ES	1.31	34.95
2	5%LIME+4%SDA	1.28	35.59
3	5%LIME+8%SDA	1.27	36.14
4	5%LIME+12%SDA	1.26	38.71
5	5%LIME+16%SDA	1.24	40.05

As it is seen in Figure 4.11 and Table 4.9, maximum dry density decreased from 1.31/cm^3 to 1.24 g/m^3 and optimum moisture content increased from 34.95% to 40.05% for soil samples increased SDA content from 0% to 16% with 5 %lime.

4.5.5 Effect of lime and saw dust ash on UCS of the expansive soil

The unconfined compressive strength of the remolded samples prepared at MDD and optimum moisture content. When the natural soil was mixed at 5% lime and 12% SDA content the maximum values of undrained shear strength is 205.393kPa and 212.534kPa for uncured and 7 days cured stabilized soil samples respectively changed from stiff clay to hard clay soil properties.

From the test result the optimum UCS and undrained shear strength value of the soil stabilized sample is when the 5%lime and 12%SDA content added is maximum value is 425.068kpa, 212.534kPa respectively which is cured for 7 days.

The effect of lime and SDA on the unconfined compressive strength values of the soil mixtures are shown in Figures 4.12 and 4.13 for uncured and 7 days cured respectively.

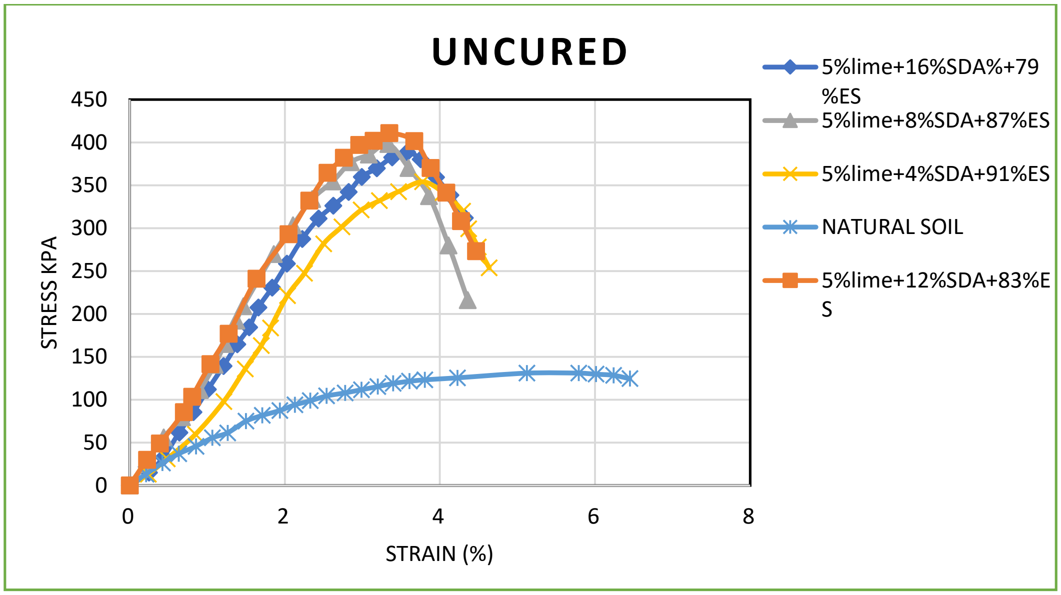


Figure 4. 12 UCS curves for un-cured stabilized soil samples

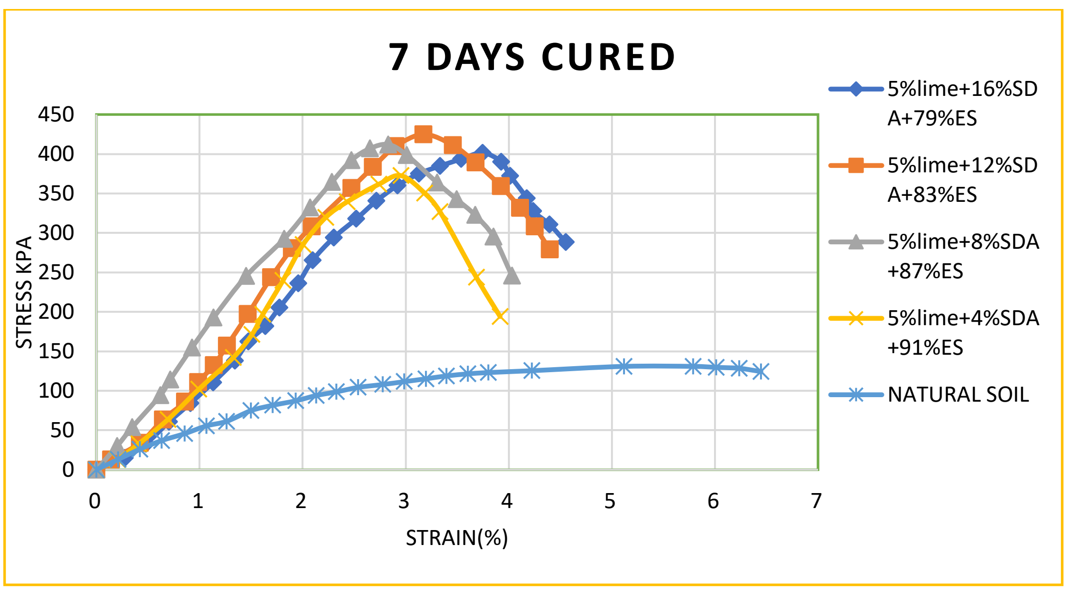


Figure 4. 2 UCS curves for 7 days cured stabilized soil samples

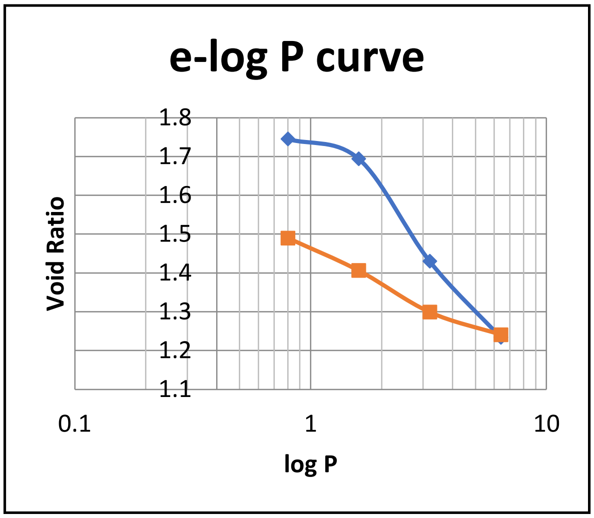
4.5.6 Effect of lime and saw dust ash on consolidation test of the expansive soil

A consolidation property of soils was the process in which reduction in volume takes place by the expulsion of water under long-term static loads. It occurs when a load is applied to soil that causes the soil Particles to pack together more tightly. Therefore, reduce its bulk volume by squeezing out the water from the soil. When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting thereby in an excess pore water pressure. If drainage is permitted, the resulting hydraulic gradients initiate a flow of water out of the clay mass and the mass begins to compress. Determination of compressibility characteristic of soils is one of the most important requirements in any soil investigation. The oedometer test is used to determine the compressibility characteristics of soils represented by (i) the coefficient of compressibility, a_v which is defined as the slope of the void ratio vs. effective stress curve; (ii) the coefficient of volume compressibility, m_v which is defined as the ratio of a_v to $(1 + e_i)$, e_i being the initial void ratio and (iii) the compression index, Cc defined as the slope of the straight line portion of e vs. log p. The consolidation characteristics of soil like compression index cc and coefficient of consolidation C_v play a vital role in predicting the compressibility of soil mass. Settlement is estimated from compression index cc, which is obtained from void ratio e versus vertical effective stress σ' in the semi-logarithmic plane. These consolidation characteristics are obtained from data of several one dimensional consolidation tests according to ASTM Standard using oedometer apparatus for varying loading increments. The collected data are plotted as graphical construction, from which the consolidation characteristics are determined. The consolidation behavior was investigated by studying the parameters such as compression index (C_c), swelling index (C_s), consolidation coefficient (C_v), and coefficient of volume change (m_v).

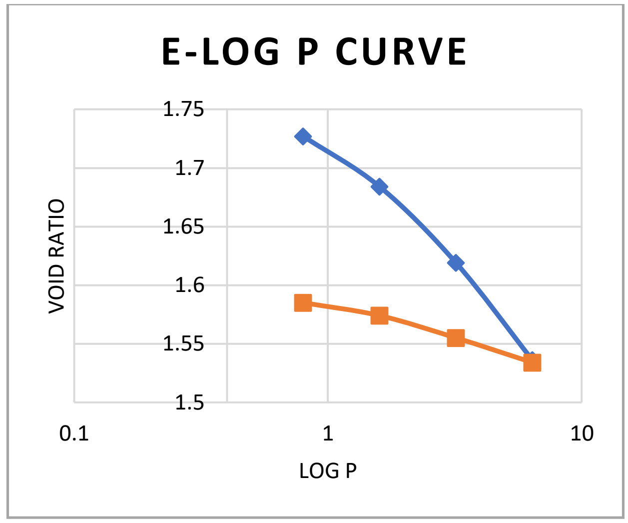
Table 4. 10 *Consolidation test result*

	Pressure (kPa)	Void ratio	Time for 50% consolidation (min.)	Coefficient of consolidation cm^2/min	Coefficient of volume change(m^2/KN)	Compression index (Cc)	Recompression index (Cr)
Natural soil	80	1.745	36.67	$5.28*10^{-3}$	$2.422*10^{-4}$	0.651	0.355
	160	1.694	44.45	$4.23*10^{-3}$			
	320	1.430	46.26	$3.50*10^{-3}$			
	640	1.234	53.96	$2.84*10^{-3}$			
Soil+5%lime +4%SDA	80	1.727	42.62	$5.16*10^{-3}$	$1.96*10^{-4}$	0.418	0.276
	160	1.684	51.54	$4.27*10^{-3}$			
	320	1.619	55	$3.40*10^{-3}$			
	640	1.536	68.62	$2.38*10^{-3}$			
Soil+5%lime +8%SDA	160	1.550	45.5	$4.14*10^{-3}$	$1.85*10^{-4}$	0.339	0.106
	320	1.545	56	$3.5*10^{-3}$			
	640	1.41	72.33	$2.88*10^{-3}$			
Soil+5%lime +12%SDA	160	1.596	56.2	$4.12*10^{-3}$	$1.52*10^{-4}$	0.219	0.0465
	320	1.557	64.5	$3.36*10^{-3}$			
	640	1.491	83.5	$2.54*10^{-3}$			

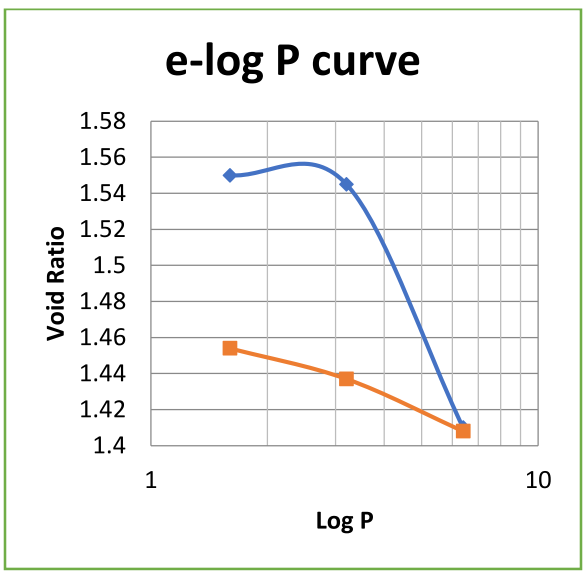
The consolidation coefficient versus the 5%lime+%SDA was a decrease in the values of the consolidation coefficient as the percentage of the lime and sawdust increases with increasing loading pressure. The reduction in the consolidation coefficient could be due to the rearrangement of the soil particles and the pozzolanic reaction taking place between the soil particles and the mixture. The consolidation coefficient C_v is show the amount of settlement in each loading period. For this research, the value of the consolidation coefficient Cv was obtained by applying the Casagrande method. Generally, the coefficients of consolidation and volume compressibility are the main parameters in virgin compression.



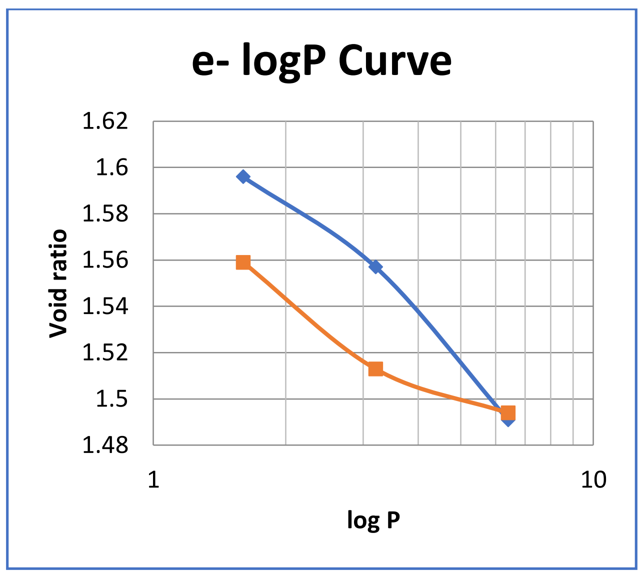
a)



b)



c)



d)

Figure 4. 14 Consolidation test results void ratio Vs pressure (log scale) loading and unloading

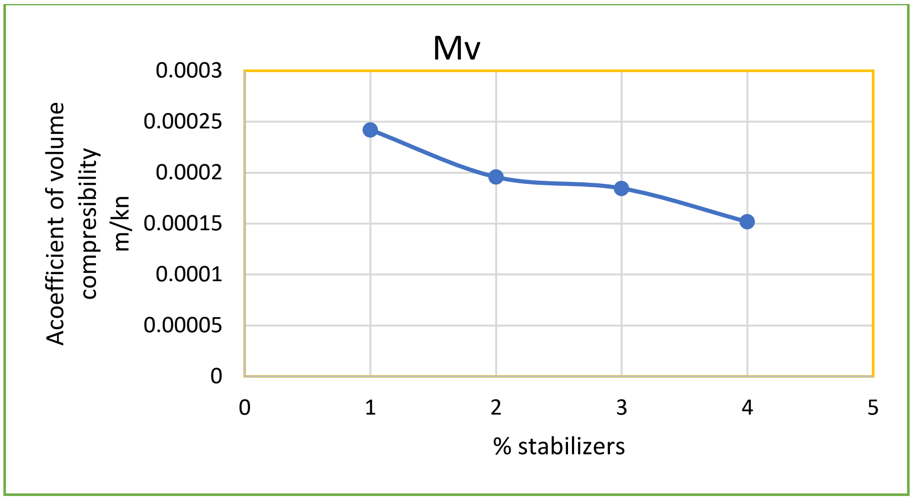


Figure 4. 15 Coefficient of volume change Vs 5%lime+%SDA +ES

Coefficient of volume change, mv is the volume reduction of a unit volume of soil for every unit increase in effective pressure during compression. The coefficient of volume changes versus the 5%lime+%SDA as shown in figure 4.14 that there was a decrease in the values of the coefficient of volume change from $2.42*10^{-4}$ to $1.518*10^{-4}$ m2/KN as the percentage of the lime and sawdust increases. The result of reduction of air in the voids and rapid rearrangement of the soil particles at the point of the 12% sawdust ash and 5% lime mixture thus, making the soil particles more compact and less compressible.

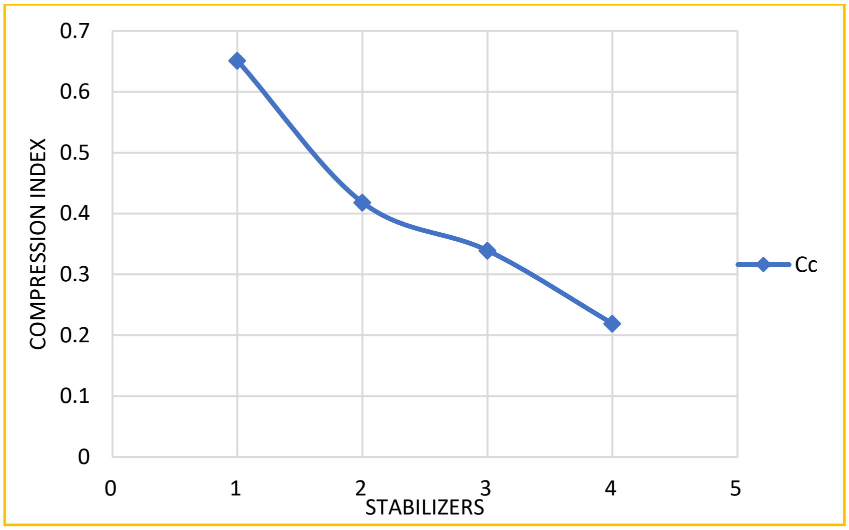


Figure 4.16 Compression index Vs 5%lime+%SDA+ES

The results of compression the natural soil and stabilized soil specimens by lime and saw dust ash are summarized in the figure above and recompression in the figure blow respectively. Figure 4.15 shows the compression index and figure 4.16 show recompression index versus the % of stabilizers respectively. It can be seen from the figure above that as increasing lime and saw dust ash there was a decrease in the compression index and from 0.651-0.269 and from the figure below as increasing lime and saw dust ash there was a decrease in the recompression index from 0.355-0.0465 respectively as a result of the pozzolanic action occurring between the additives and the expansive soil particle. It can be seen that the combination of 12%SDA + 5% lime proved to be the most effective.

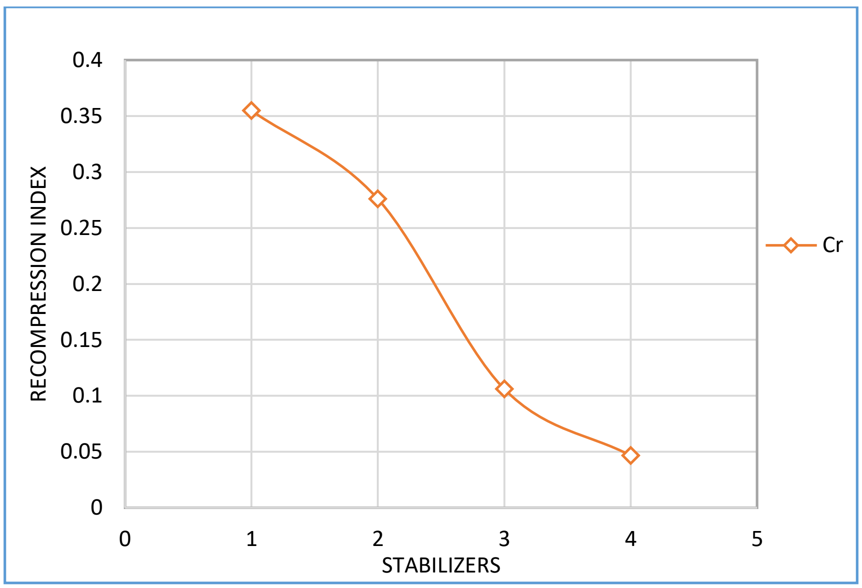


Figure 4.17 recompression index Vs 5%lime+%SDA+ES

CHAPTER FIVE

5.CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The study has been conducted on the effect of lime and saw dust ash on some black cotton soil's geotechnical characteristics specifically found around the Akaki Kality area. The following conclusions can be drawn from the results of the laboratory tests**.**

The liquid limit and plasticity index showed a reduction with increasing percentages of lime and sawdust ash

The optimum moisture content increased while the maximum dry density values decreased with an increment of lime and saw dust ash content.

The addition of lime and saw dust ash for the studied soil has resulted in decreasing the specific gravity of the soil sample.

Free swell, free swell index, and the free swell ratio of the stabilized samples decreased with increasing lime and saw dust ash content.

The addition of lime in combination with saw dust ash improved the unconfined compressive strength. The improvement is more significant when the sample is cured. A combination of 5%lime and 12% saw dust ash by dry weight of a soil results in the maximum UCS values, thus it was taken to be optimum stabilizer content. Hence, the combination of lime and saw dust ash can improve the strength of the soil.

Generally, lime and saw dust ash combination can be an alternative expansive soil stabilizing chemical where the sufficiently abundant performance of the soil has improved by treating the expansive soil with lime and saw dust ash.

5.2 Recommendations

It is obvious that saw dust ash is disposed of at different place in the environment. So, it is recommended to make intensive research to use this waste material with a combination of lime for soil improvement as it is cost-effective and environmentally friendly.

There is not enough investigation done on lime and saw dust ash as expansive soil treated in Ethiopia. So it is recommended that extensive research on soil samples taken from different places in Ethiopia with different samples of lime and saw dust ash should be done.

The mineralogy of lime and saw dust ash was not investigated in terms of suitability in this study. It may be helpful to study mineralogical suitability of lime and saw dust ash.

All the test results obtained in this study were for non-cured samples for a certain periods of time except the unconfined compressive strength test. It may be useful to study the effect of curing lime and saw dust ash stabilized soil samples. Hence, anyone who has an interest to study the effect of lime with saw dust ash stabilized expansive soil by including the cured case.

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Appendix

A: Laboratory test result of the soil sample

Table A 1: Natural moisture content for test pit-1

	Test pit-1			Test pit-2		
Can No.	1	2	3	1	2	3
Mass of can (g)	30.15	29.9	30.14	30.6	29.85	30.27
Mass of can+ mass of wet soil(g)	121.17	125.71	132.34	139.05	138.35	143.43
Mass of can+ mass of dry soil (g)	94.81	98.46	102.09	108.86	109.64	112.36
weight of dry soil (g)	64.66	68.56	71.95	78.26	79.79	82.09
weight of moisture (g)	26.36	27.25	30.25	30.19	28.71	31.07
Moisture content (w)%	40.77	39.75	42.04	38.58	35.98	37.85
Average water content(w)%	40.85			37.47		

Appendix B - Grain Size and Hydrometer Analysis Test Result

Table B- 1 Grain size analysis test result (TP-1)

Sieve No.	Sieve size (mm)	Mass of sieve(g)	Mass of sieve + Retained soils(g)	Mass of Retained soils(g)	Percentage Retained (%	Cum. Percentage Retained (%)	Perc. Passing (%
No.4	4.75	440	458	18	1.80	1.80	98.20
No.8	2.36	406	418	12	1.20	3.00	97.00
No.10	2	383	386	3	0.30	3.30	96.70
No.16	1.18	359	364	5	0.50	3.80	96.20
No.30	0.6	329	335	6	0.60	4.40	95.60
No.50	0.3	296	307	11	1.10	5.50	94.50
No.100	0.15	279	287	8	0.80	6.30	93.70
No.200	0.075	275	281	6	0.60	6.90	93.10
pan		253	253	0	0.00	100.00	…

Table B-2 Hydrometer analysis test result (TP-1)

Elapsed Time, t, min	Actual Hydrometer Reading R	Temp (^{0}C)	Effective Depth, L (mm)	K From Table	Temperature Correction Factor Ct	Modified Hydrometer Reading R= Ra-Cz+Ct	Particle Diameter, D (mm	Percentage Pass (%)	Corrected Percentage Pass (%)
2	49	22	8.3	0.01307	0.4	44.4	0.0266	87.912	86.839
4	48	22	8.4	0.01307	0.4	43.4	0.0189	85.932	84.884
8	47	22	8.6	0.01307	0.4	42.4	0.0136	83.952	82.928
15	46	22	8.7	0.01307	0.4	41.4	0.0099	81.972	80.972
30	44	22	9.1	0.01307	0.4	39.4	0.0072	78.012	77.060
60	42	22	9.4	0.01307	0.4	37.4	0.0052	74.052	73.149
120	40	22	9.7	0.01307	0.4	35.4	0.0037	70.092	69.237
240	38	22	10.1	0.01307	0.4	33.4	0.0027	66.132	65.325
480	35	22	10.6	0.01307	0.4	30.4	0.0019	60.192	59.458
1440	31	22	11.2	0.01307	0.4	26.4	0.0012	52.272	51.634

Table B- 3 Grain size analysis test result (TP-2)

Sieve No.	Sieve size (mm)	Mass of sieve(g)	Mass of sieve + Retained soils(g)	Mass of Retained soils(g)	Percentage Retained (%	Cum. Percentage Retained (%)	Perc. Passing (%
No.4	4.75	440	467	27	2.70	2.70	97.30
No.8	2.36	406	425	19	1.90	4.60	95.40
No.10	2	383	388	5	0.50	5.10	94.90
No.16	1.18	359	367	8	0.80	5.90	94.10
No.30	0.6	329	338	9	0.90	6.80	93.20
No.50	0.3	296	306	10	1.00	7.80	92.20
No.100	0.15	279	285	6	0.60	8.40	91.60
No.200	0.075	276	280	4	0.40	8.80	91.20
pan		253	253	0	0.00	100.00	….

Figure B- 1 Particle size distribution curve for TP-1 and TP-2

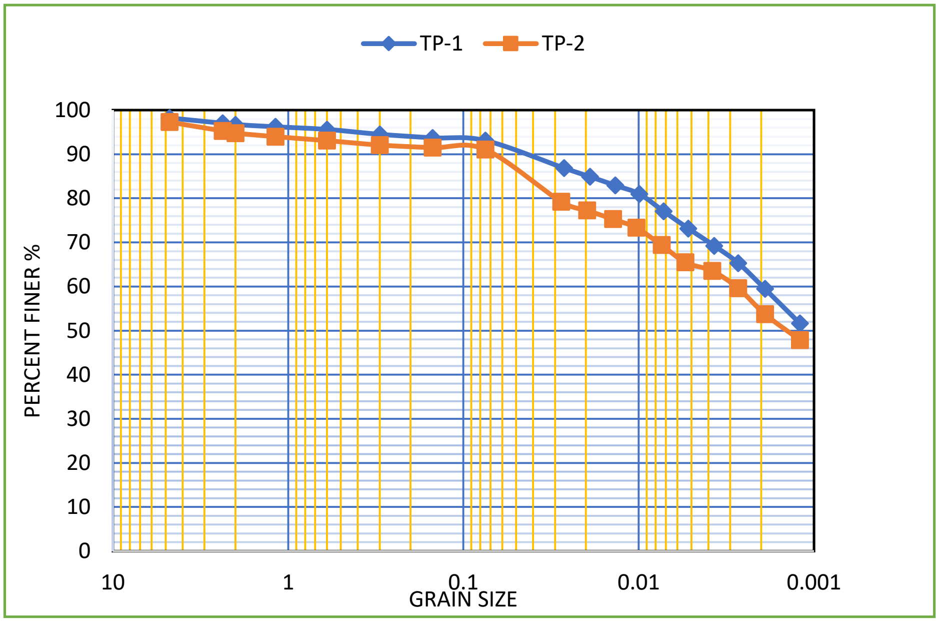


Table B- 4 Hydrometer analysis test result (TP-2)

Elapsed Time, t, min	Actual Hydrometer Reading R	Temp (^{0}C)	Effective Depth, L (mm)	K From Table	Temperature Correction Factor Ct	Modified Hydrometer Reading R=Ra-Cz+Ct	Particle Diameter, D (mm)	Percentage Pass (%)	Corrected Percentage Pass (%)
2	45	22	8.9	0.01307	0.4	40.4	0.0276	79.992	79.208
4	44	22	9.1	0.01307	0.4	39.4	0.0197	78.012	77.247
8	43	22	9.2	0.01307	0.4	38.4	0.014	76.032	75.287
15	42	22	9.4	0.01307	0.4	37.4	0.0103	74.052	73.326
30	40	22	9.7	0.01307	0.4	35.4	0.0074	70.092	69.405
60	38	22	10.1	0.01307	0.4	33.4	0.0054	66.132	65.484
120	37	22	10.2	0.01307	0.4	32.4	0.0038	64.152	63.523
240	35	22	10.6	0.01307	0.4	30.4	0.0027	60.192	59.602
480	32	22	11.1	0.01307	0.4	27.4	0.0019	54.252	53.720
1440	29	22	11.5	0.01307	0.4	24.4	0.0012	48.312	47.839

Appendix C - Specific Gravity Test Results

Table C- 1 Specific gravity test result for natural expansive soil

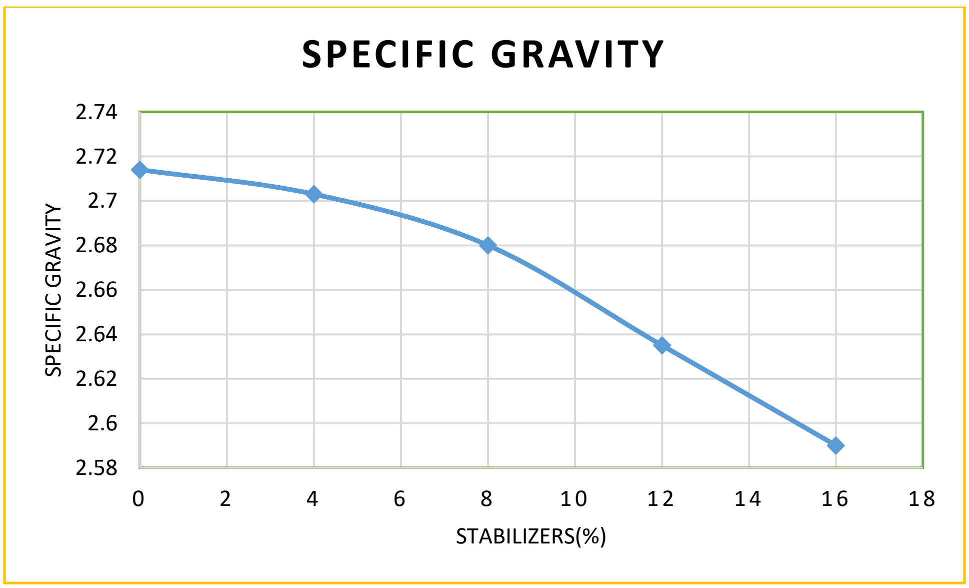
	Test pit-1		Test pit-2	
specimen number	1	2	1	2
Pycnometer number	57	38	63	37
WP = Mass of empty, clean pycnometer (g)	35.83	35.52	34.85	35.57
WPS = Mass of empty pycnometer + dry soil (g)	45.83	45.52	44.85	45.57
WB = Mass of pycnometer + dry soil + water (g)	141.82	139.83	141.77	142.39
WA = Mass of pycnometer + water (grams)	135.5	133.52	135.44	136.05
Specific Gravity (G_S)	2.717	2.710	2.725	2.732
average specific gravity	2.714		2.729	

Table C - 2 Specific gravity test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

	5%lime+4%SDA+ES		5%lime+8%SDA+ES	
specimen number	1	2	1	2
Pycnometer number	37	38	63	57
WP = Mass of empty, clean pycnometer (g)	35.56	36.45	36.41	34.68
WPS = Mass of empty pycnometer + sample (g)	45.56	46.45	46.41	44.68
WB = Mass of pycnometer + sample+ water (g)	142.72	140.46	141.66	141.78
WA = Mass of pycnometer + water (grams)	136.40	134.18	135.12	135.22
Specific Gravity (G_S)	2.717	2.688	2.695	2.664
average specific gravity	2.703		2.680	

Table C - 3 Specific gravity test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

	5%lime+12%SDA+ES		5%lime+16%SDA+ES	
specimen number	1	2	1	2
Pycnometer number	37	38	57	63
WP = Mass of empty, clean pycnometer (g)	35.56	36.45	36.41	34.68
WPS = Mass of empty pycnometer +sample (g)	45.56	46.45	46.41	44.68
WB = Mass of pycnometer + sample+ water (g)	141.58	139.54	141.19	141.31
WA = Mass of pycnometer + water (grams)	135.37	133.34	134.56	134.68
Specific Gravity (G_S)	2.639	2.632	2.585	2.594
average specific gravity	2.635		2.590	



Appendix D – free swelling Test Results

Table D-1: Free Swelling Test Result for TP-1 and TP-2

	TP-1		TP-2	
Description	water	kerosene	water	Kerosene
Initial reading	10	10	10	10
Final reading	25	11	22	11
Free swell	150.00		120.00	
Free swell index	138.10		100.00	
Free swell ratio	2.40		2.00	

Table D - 2 free swelling test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

Description	5%lime+4%SDA+ES		5%lime+8%SDA+ES	
	water	kerosene	water	Kerosene
Initial reading	10	10	10	10
Final reading	22.5	11	21	12
Free swell	125.000		110.00	
Free swell index	104.545		75.00	
Free swell ratio	2.045		1.750	
Description	5%lime+12%SDA+ES		5%lime+16%SDA+ES	
	water	kerosene	water	Kerosene
Initial reading	10	10	10	10
Final reading	19.5	12.5	17	13.5
Free swell	95		70	
Free swell index	56.00		25.926	
Free swell ratio	1.56		1.259	

Appendix E – Atterberg limit Test Results

Table E-1 Atterberg limit for TP-1 natural expansive soil

Types of tests	Liquid limit			plastic limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	34	28	18	-	-	-
Mass of wet soil+can	28.62	29.13	28.42	22.93	22.94	22.02
Mass of dry soil+can	24.47	24.15	23.71	21.89	21.88	20.94
Mass of can	19.56	19.22	19.53	19.36	19.46	18.57
Mass of dry soil	4.91	4.93	4.18	2.53	2.42	2.37
Mass of moisture	4.15	4.98	4.71	1.04	1.06	1.08
Water content	84.52	101.01	112.68	41.11	43.80	45.57
Average	………			43.49		
Plastic index	60.51					

LL =104 %
Pl =43.49 %
Ip=60.51%

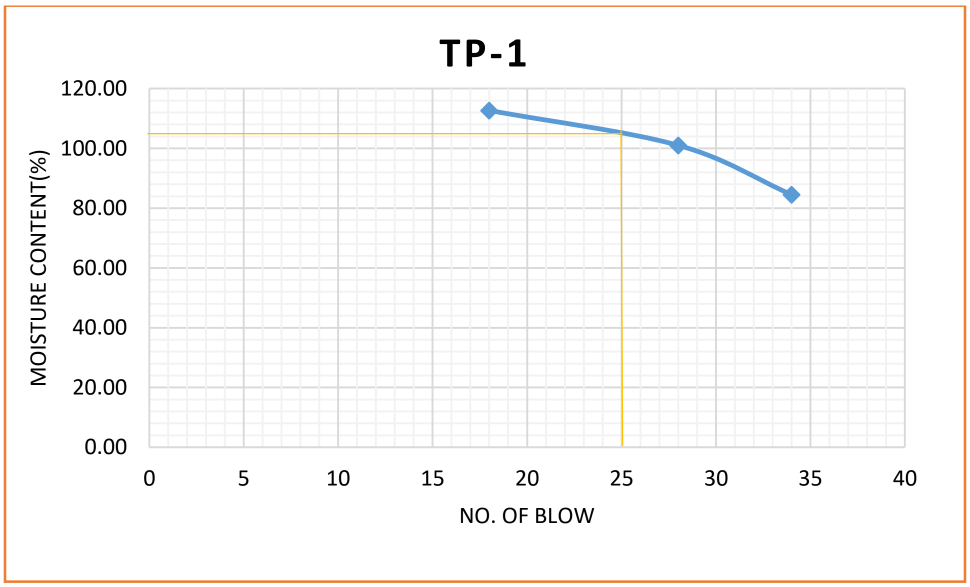


Table E-2 Atterberg limit for TP-2 natural expansive soil

Types of tests	Liquid limit			plastic limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	33	24	18	-	-	-
Mass of wet soil+can	25.88	26.42	26.93	24.41	23.76	23.57
Mass of dry soil+can	23.41	22.7	23.23	22.94	22.43	22.27
Mass of can	19.71	18.87	19.61	19.31	19.15	19.23
Mass of dry soil	3.7	3.83	3.62	3.63	3.28	3.04
Mass of moisture	2.47	3.72	3.7	1.47	1.33	1.3
Water content	66.76	97.13	102.21	40.50	40.55	42.76
Average				41.27		
Plastic index	50.73					

LL =92 %
Pl =41.27 %
Ip = 50.73%

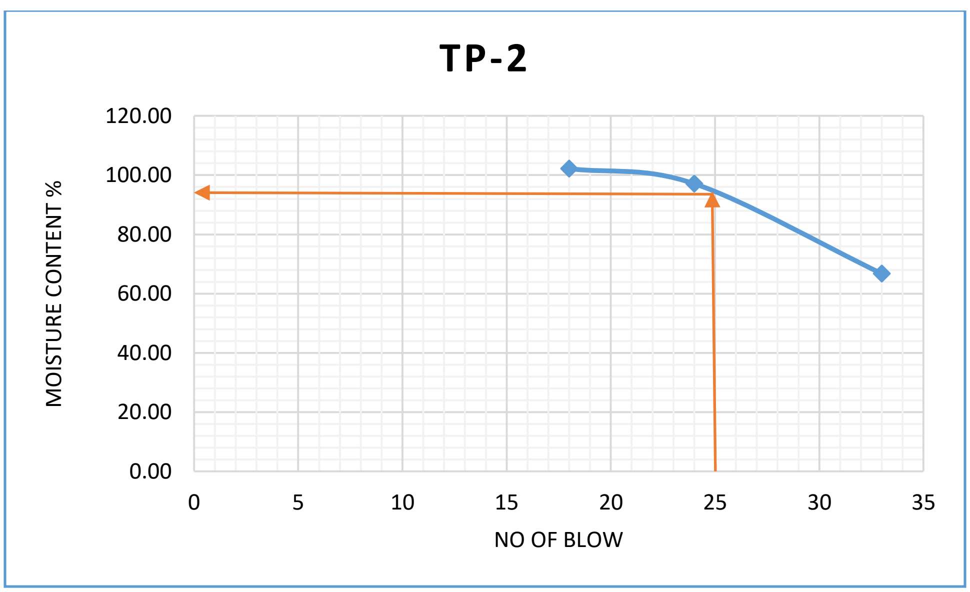


Table E - 3 atterberg limit test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

Types of tests	5%lime+4%SDA					
	Liquid limit			plastic limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	16	27	33	-	-	-
Mass of wet soil+can	33.52	33.22	34.24	29.35	29.18	27.23
Mass of dry soil+can	29.03	28.69	30.12	27.87	27.28	26.21
Mass of can	24.52	23.67	24.28	24.31	22.89	23.87
Mass of dry soil	4.51	5.02	5.84	3.56	4.39	2.34
Mass of moisture	4.49	4.53	4.12	1.48	1.9	1.02
Water content	99.56	90.24	70.55	41.57	43.28	43.59
Average	-----------			42.81		
Plastic index						

ll=92%
pl=42.59
Ip=49.41%

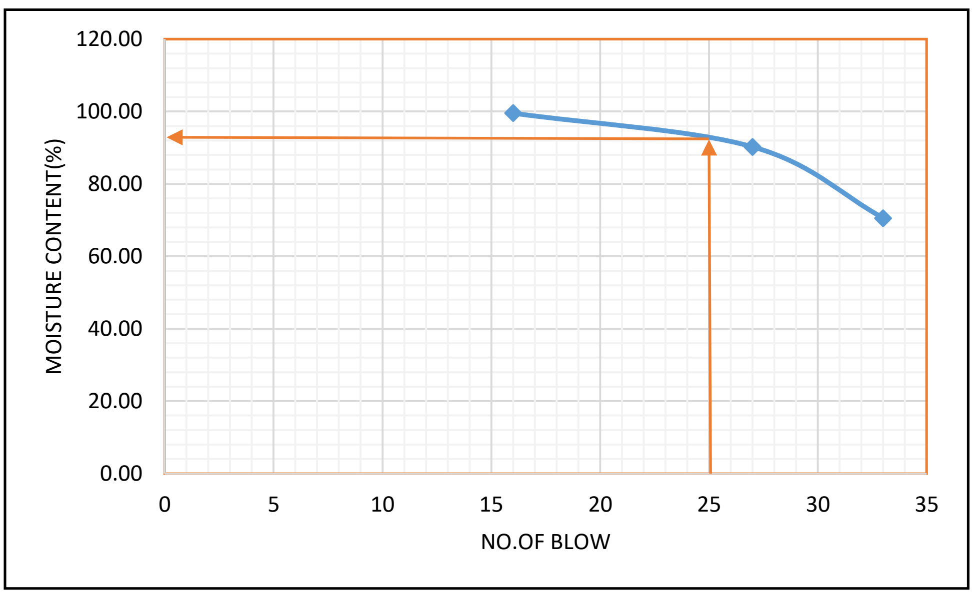


Table E – 4 atterberg limit test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

Types of tests	5%lime+8%SDA					
	Liquid limit			Plastic limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	16	24	33	-	-	-
Mass of wet soil+can	33.84	32.78	31.41	29.12	18.54	30.78
Mass of dry soil+can	29.4	29.08	27.7	27.61	16.93	28.94
Mass of can	24.2	24.34	22.75	23.59	13.22	23.98
Mass of dry soil	5.2	4.74	4.95	4.02	3.71	4.96
Mass of moisture	4.44	3.7	3.71	1.51	1.61	1.84
Water content	85.38	78.06	74.95	37.56	43.40	37.10
Average				39.35		
Plastic index						

ll=77.8%
pl=39.30%
Ip=38.45%

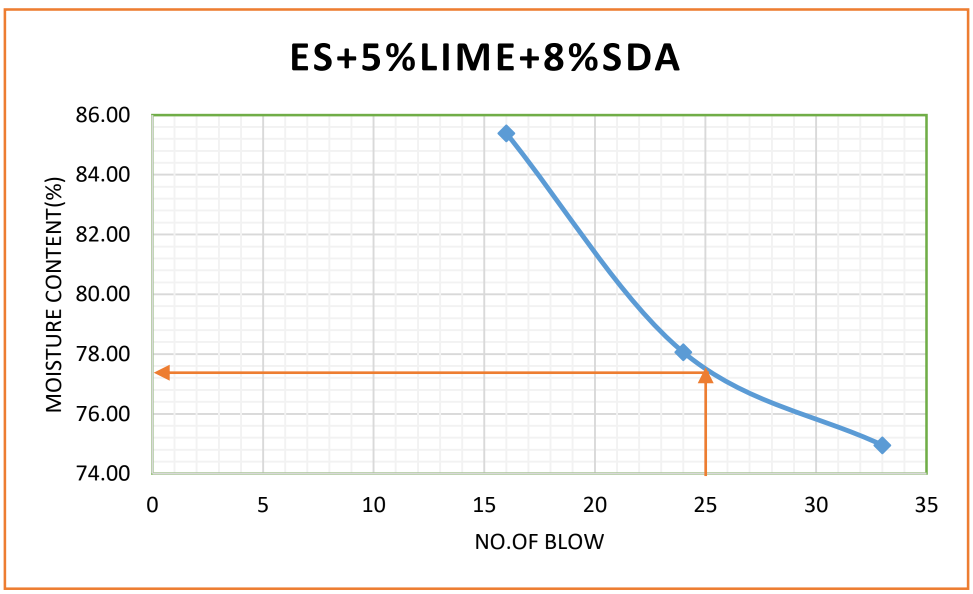


Table E - 5 atterberg limit test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

Types of tests	5%lime+12%SDA					
	Liquid Limit			Plastic Limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	16	27	34	-	-	-
Mass of wet soil+can	21.28	33.52	30.84	18.14	29.22	18.38
Mass of dry soil+can	17.84	29.92	28.64	16.87	27.84	17.02
Mass of can	13.19	24.04	23.75	13.33	24.12	13.16
Mass of dry soil	4.65	5.88	4.89	3.54	3.72	3.86
Mass of moisture	3.44	3.6	2.2	1.27	1.38	1.36
Water content	73.98	61.22	44.99	35.88	37.10	35.23
Average				36.07		
Plastic index						

Ll=64%
pl=36.07%
Ip=29.93%

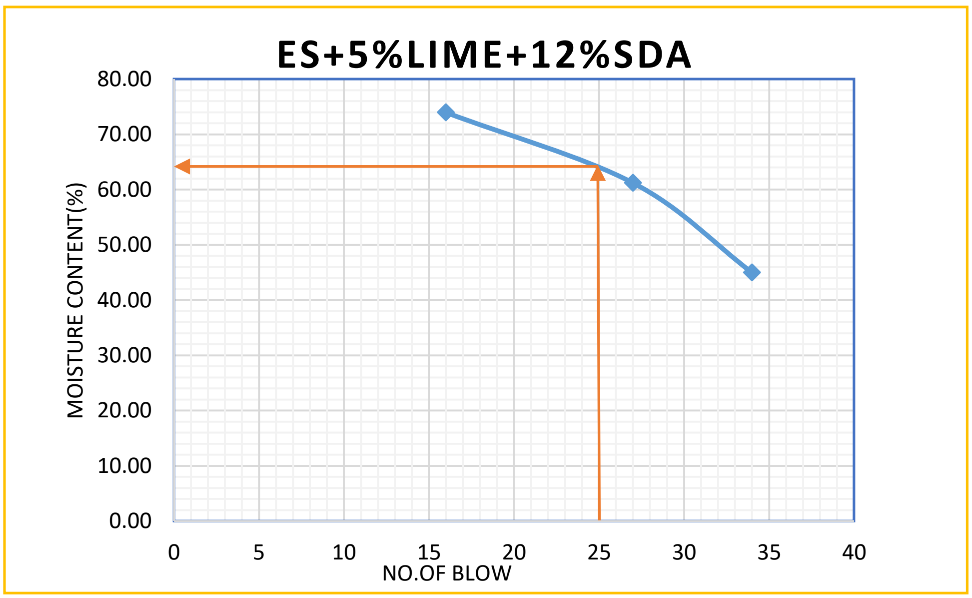
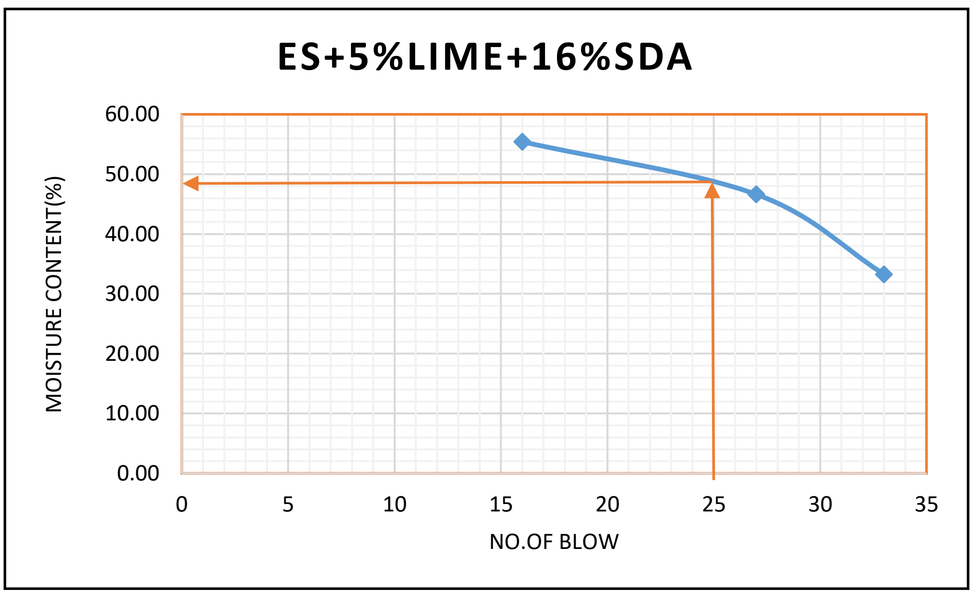


Table E – 6 atterberg limit test result for TP-1 expansive soil with lime and Sawdust Ash (SDA)

Types of tests	5%lime+16%SDA+79% ES					
	Liquid Limit			Plastic Limit		
	trial 1	trial 2	trial 3	trial 1	trial 2	trial 3
Number of blows	16	27	33	-	-	-
Mass of wet soil+can	30.17	32.44	34.66	27.64	26.88	28.39
Mass of dry soil+can	27.92	29.76	32.06	26.72	26.02	27.44
Mass of can	23.86	24.01	24.24	23.92	23.22	24.5
Mass of dry soil	4.06	5.75	7.82	2.8	2.8	2.94
Mass of moisture	2.25	2.68	2.6	0.92	0.86	0.95
Water content	55.42	46.61	33.25	32.86	30.71	32.31
Average				31.96		
Plastic index						

ll=49.5%
pl=31.96%
lp=17.54%



Appendix F – Compaction Test Results

Table F-1 Compaction test result for TP-1 natural expansive soil

Trial No	1	2	3	4	5	6
Wt. of Mold +wet soil (g)	5114	5208	5286	5335	5432	5410
Wt. of mold(g)	3760	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1354	1448	1526	1575	1672	1650
Wet density, (g/cm3)	1.43	1.53	1.62	1.67	1.77	1.75
moisture content determination						
Mass of can (g)	19.50	19.58	19.55	19.41	22.89	30.35
mass of wet soil + can (g)	42.34	49.55	43.35	50.87	53.10	80.38
mass of dry soil + can(g)	38.75	43.85	38.03	43.68	45.34	64.66
mass of moisture(g),	3.59	5.70	5.32	7.19	7.76	15.72
Mass of dry soil (g)	19.25	24.27	18.47	24.27	22.45	34.31
Moisture content (%	18.63	23.35	28.62	30.49	34.95	45.81
dry density	1.21	1.24	1.26	1.28	1.31	1.20

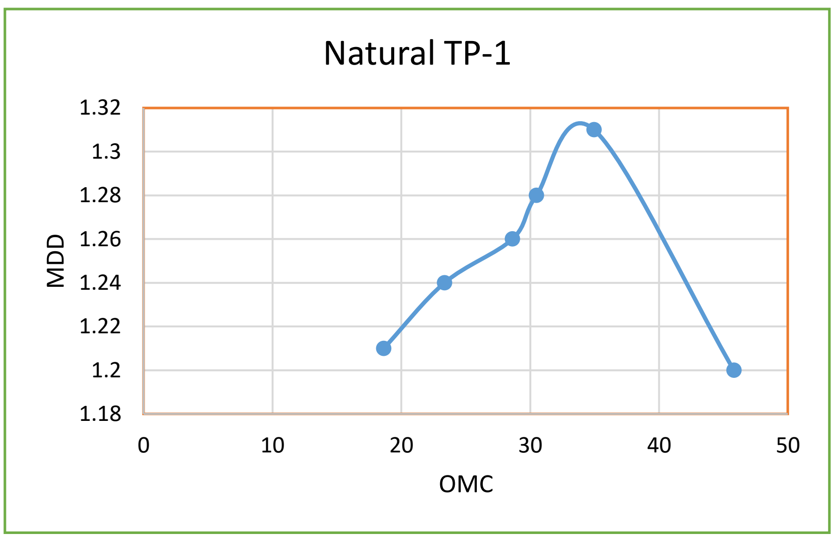


Table F-2 Compaction test result for TP-2 natural expansive soil

Trial No	1	2	3	4	5	6
Wt. of Mold +wet soil (g)	5140	5210	5283	5392	5471	5383
Wt. of mold(g)	3760	3760	3760	3760	3760	3755
Wt. of wet soil (g)	1380	1450	1523	1632	1711	1628
Volume of mold cm3	944	944	944	944	944	944
Wet density, (g/cm3)	1.46	1.54	1.61	1.73	1.81	1.72
moisture content determination						
Mass of can (g)	19.16	19.36	19.47	19.44	19.52	19.23
mass of wet soil + can (g)	49.77	45.05	44.72	54.25	45.62	62.54
mass of dry soil + can(g)	44.69	40.28	39.35	45.67	38.64	50.26
mass of moisture(g),	5.08	4.77	5.38	8.58	6.98	12.28
Mass of dry soil (g)	25.53	20.92	19.87	17.17	19.12	31.02
Moisture content (%	19.69	22.86	26.91	32.74	36.50	40.21
dry density	1.22	1.25	1.27	1.30	1.33	1.23

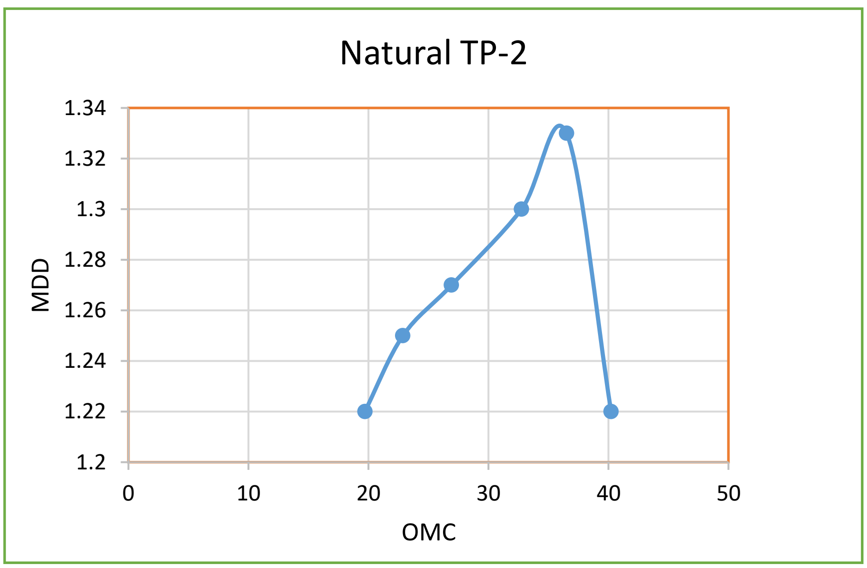


Table F - 3 Compaction test result for TP-1 expansive soil with 2%lime

	2%lime+ES				
Trial No	1	2	3	4	5
Wt. of Mold +wet soil (g)	5104	5185	5302	5431	5379
Wt. of mold(g)	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1344	1425	1542	1671	1619
Volume of mold cm3	944	944	944	944	944
Wet density, (g/cm3)	1.42	1.51	1.63	1.77	1.72
moisture content determination					
Mass of can (g)	30.11	30.25	29.94	30.36	30.12
mass of wet soil + can (g)	59.93	63.73	62.31	70.21	50.47
mass of dry soil + can(g)	55.99	58.08	55.19	59.33	41.88
mass of moisture(g),	3.94	5.65	7.12	10.88	12.96
Mass of dry soil (g)	25.88	27.83	25.25	28.97	32.67
Moisture content (%	15.89	20.36	28.22	37.54	39.67
dry density	1.23	1.25	1.27	1.29	1.23

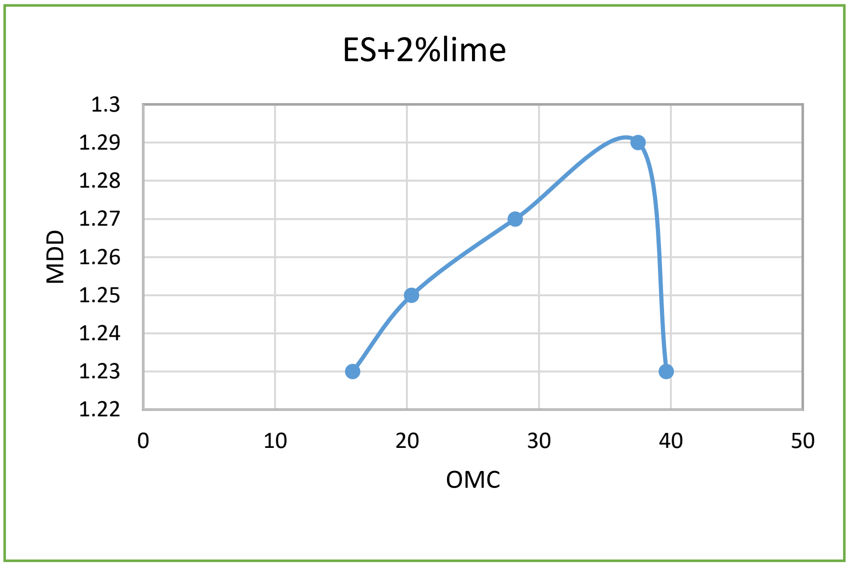


Table F - 4 Compaction test result for TP-1 expansive soil with 3%lime

Trial No	1	2	3	4	5
Wt. of Mold +wet soil (g)	5082	5137	5226	5415	5372
Wt. of mold(g)	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1322	1377	1466	1655	1612
Volume of mold cm3	944	944	944	944	944
Wet density, (g/cm3)	1.400	1.459	1.553	1.753	1.708
moisture content determination					
Mass of can (g)	29.85	30.04	30.07	29.88	30.13
mass of wet soil + can (g)	68.78	67.15	63.52	72.04	76.00
mass of dry soil + can(g)	63.54	61.26	57.02	60.39	62.83
mass of moisture(g),	5.24	5.90	6.51	11.65	13.17
Mass of dry soil (g)	33.69	31.22	26.95	30.51	32.70
Moisture content (%	15.55	18.89	24.14	38.17	40.28
dry density	1.212	1.227	1.251	1.269	1.217

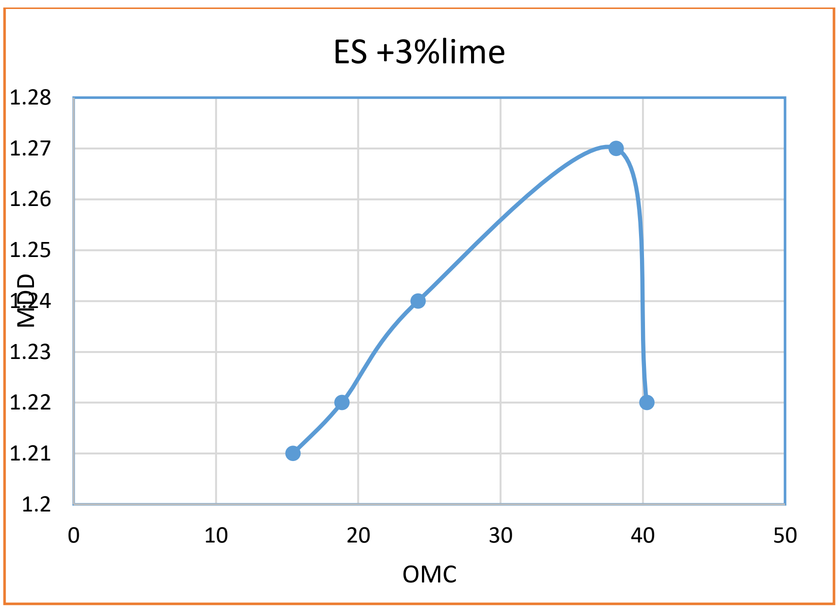


Table F - 5 Compaction test result for TP-1 expansive soil with 4%lime

Trial No	1	2	3	4	5
Wt. of Mold +wet soil (g)	5068	5142	5258	5408	5376
Wt. of mold(g)	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1308	1382	1498	1648	1616
Volume of mold cm3	944	944	944	944	944
Wet density, (g/cm3)	1.386	1.464	1.587	1.746	1.712
moisture content determination					
Mass of can (g)	30.57	30.05	29.80	30.19	30.23
mass of wet soil + can (g)	71.75	71.32	74.05	77.89	80.34
mass of dry soil + can(g)	65.66	63.83	64.17	64.70	65.33
mass of moisture(g),	6.09	7.49	9.88	13.19	15.01
Mass of dry soil (g)	35.09	33.78	34.36	34.50	35.10
Moisture content (%	17.35	22.16	28.76	38.24	42.77
dry density	1.181	1.198	1.232	1.263	1.199

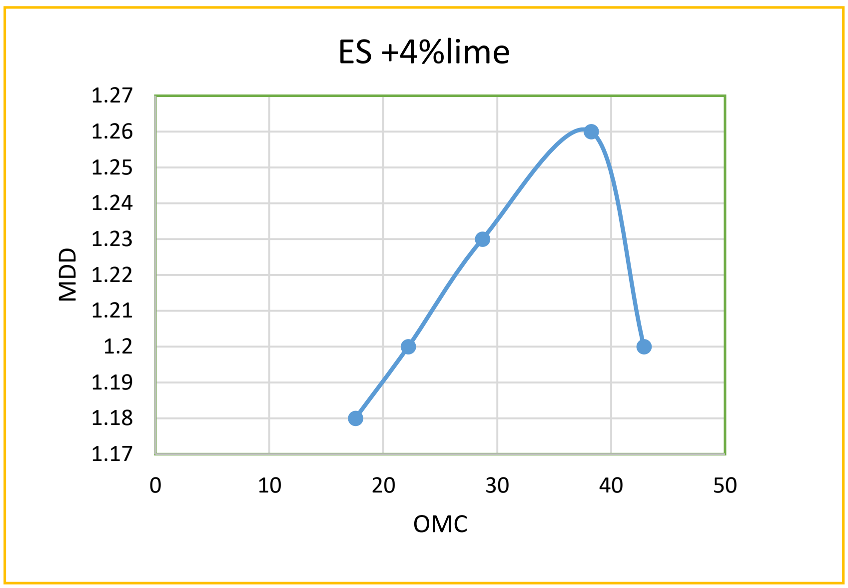


Table F - 6 Compaction test result for TP-1 expansive soil with 5%lime

Trial No	1	2	3	4	5
Wt. of Mold +wet soil (g)	5044	5124	5218	5388	5352
Wt. of mold(g)	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1284	1364	1458	1628	1592
Volume of mold cm3	944	944	944	944	944
Wet density, (g/cm3)	1.360	1.445	1.544	1.725	1.686
moisture content determination					
Mass of can (g)	29.85	30.21	26.43	22.51	19.56
mass of wet soil + can (g)	80.72	68.84	70.52	52.28	59.55
mass of dry soil + can(g)	73.04	62.24	61.16	44.80	47.55
mass of moisture(g),	7.68	6.60	9.36	8.20	12.00
Mass of dry soil (g)	43.19	32.03	34.73	21.56	27.99
Moisture content (%	17.79	20.61	26.95	39.17	42.86
dry density	1.155	1.198	1.217	1.239	1.181

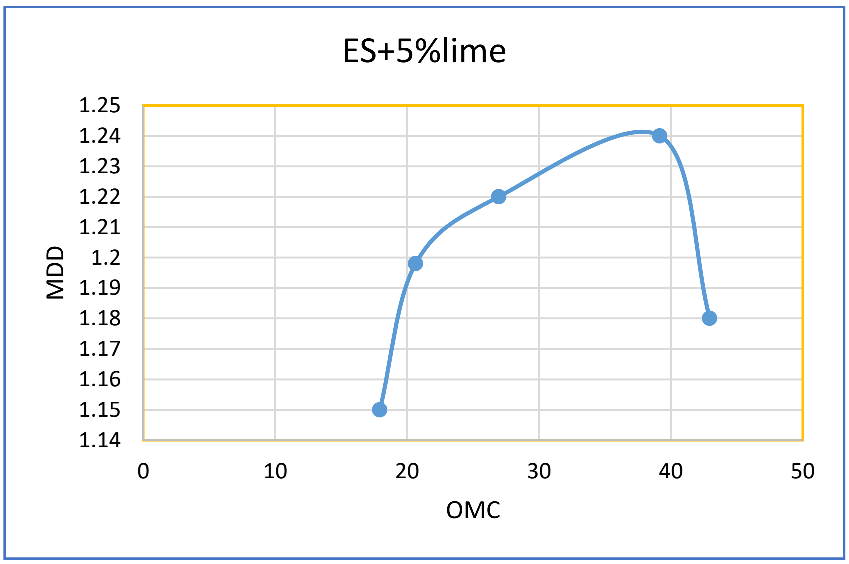


Table F -7 Compaction test result for TP-1 expansive soil with 5%lime +4% SDA

Trial No	1	2	3	4	5	6
Wt. of Mold +wet soil (g)	5080	5120	5224	5298	5395	5348
Wt. of mold(g)	3760	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1320	1360	1464	1538	1635	1588
Volume of mold cm3	944	944	944	944	944	944
Wet density, (g/cm3)	1.398	1.441	1.551	1.629	1.732	1.682
moisture content determination						
Mass of can (g)	19.46	19.46	19.14	19.45	19.56	19.51
mass of wet soil + can (g)	45.49	43.35	42.70	53.13	58.01	62.63
mass of dry soil + can(g)	42.02	39.98	38.11	45.58	47.98	49.51
mass of moisture(g),	3.47	3.36	4.58	7.55	10.03	13.12
Mass of dry soil (g)	22.56	20.53	18.97	26.13	28.42	30.00
Moisture content (%	15.40	16.39	24.16	28.91	35.30	43.72
dry density	1.212	1.238	1.249	1.264	1.280	1.170

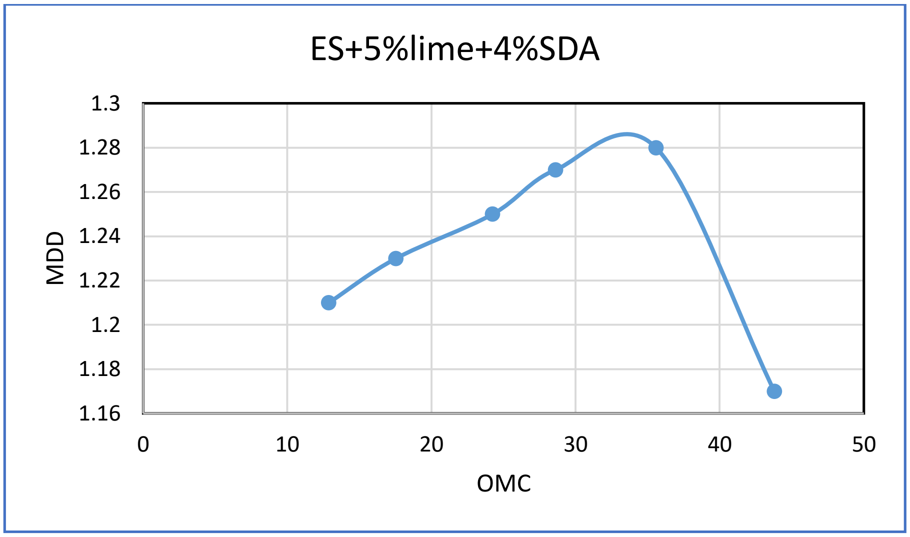


Table F -8 Compaction test result for TP-1 expansive soil with 5%lime +8% SDA

Trial No	1	2	3	4	5	6	7
Wt. of Mold +wet soil (g)	5042	5132	5176	5246	5344	5392	5334
Wt. of mold(g)	3760	3760	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1282	1372	1416	1486	1584	1632	1574
Volume of mold cm3	944	944	944	944	944	944	944
Wet density, (g/cm3)	1.358	1.453	1.500	1.574	1.678	1.729	1.667
moisture content determination							
Mass of can (g)	19.12	19.42	19.41	29.87	30.23	30.07	29.81
mass of wet soil + can (g)	40.77	38.27	40.75	67.98	66.96	75.60	76.60
mass of dry soil + can(g)	37.51	34.95	36.75	60.02	57.85	63.38	62.42
mass of moisture(g),	3.26	3.32	4.00	7.96	9.11	12.22	14.18
Mass of dry soil (g)	18.39	15.53	17.34	30.15	27.62	33.31	32.61
Moisture content (%	17.70	21.40	23.09	26.40	32.97	36.70	43.50
dry density	1.154	1.197	1.219	1.245	1.262	1.265	1.162

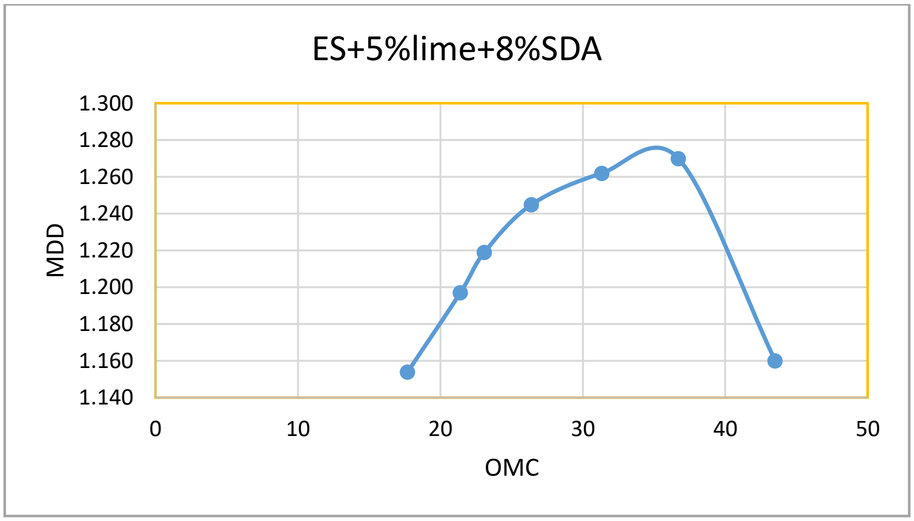


Table F -9 Compaction test result for TP-1 expansive soil with 5%lime +12% SDA

Trial No	1	2	3	4	5	6	7
Wt. of Mold +wet soil (g)	4988	5082	5142	5248	5328	5404	5332
Wt. of mold(g)	3760	3760	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1228	1322	1382	1488	1568	1644	1572
Volume of mold cm3	944	944	944	944	944	944	944
Wet density, (g/cm3)	1.301	1.400	1.464	1.576	1.661	1.742	1.665
moisture content determination							
Mass of can (g)	19.45	19.41	19.16	19.36	26.95	19.18	23.12
mass of wet soil + can (g)	39.36	41.36	44.40	39.77	58.39	45.33	72.46
mass of dry soil + can(g)	37.38	38.24	40.19	35.43	50.69	38.13	57.97
mass of moisture(g),	1.98	3.13	4.21	4.33	7.70	7.20	14.49
Mass of dry soil (g)	17.93	18.82	21.03	16.07	23.74	18.95	34.85
Moisture content (%	11.02	16.61	20.02	26.96	32.44	37.99	41.59
dry density	1.172	1.201	1.220	1.242	1.254	1.262	1.176

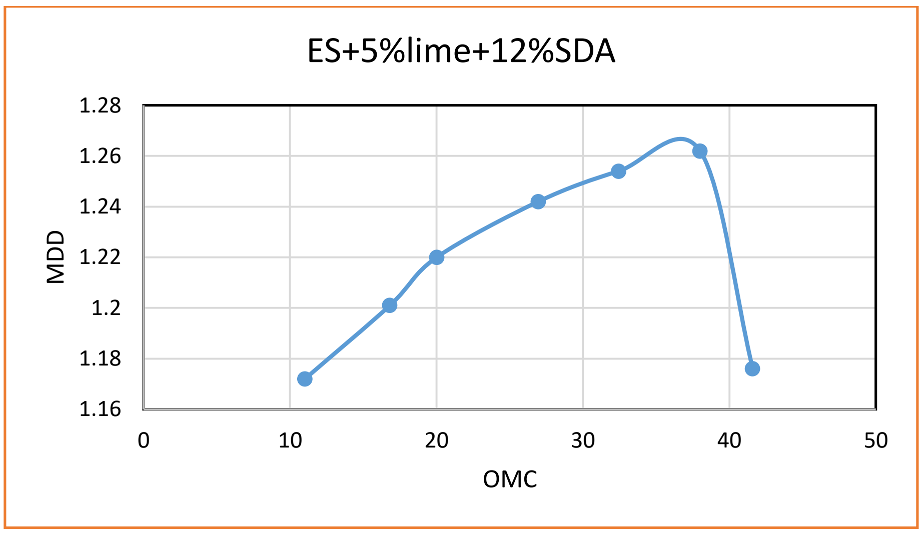
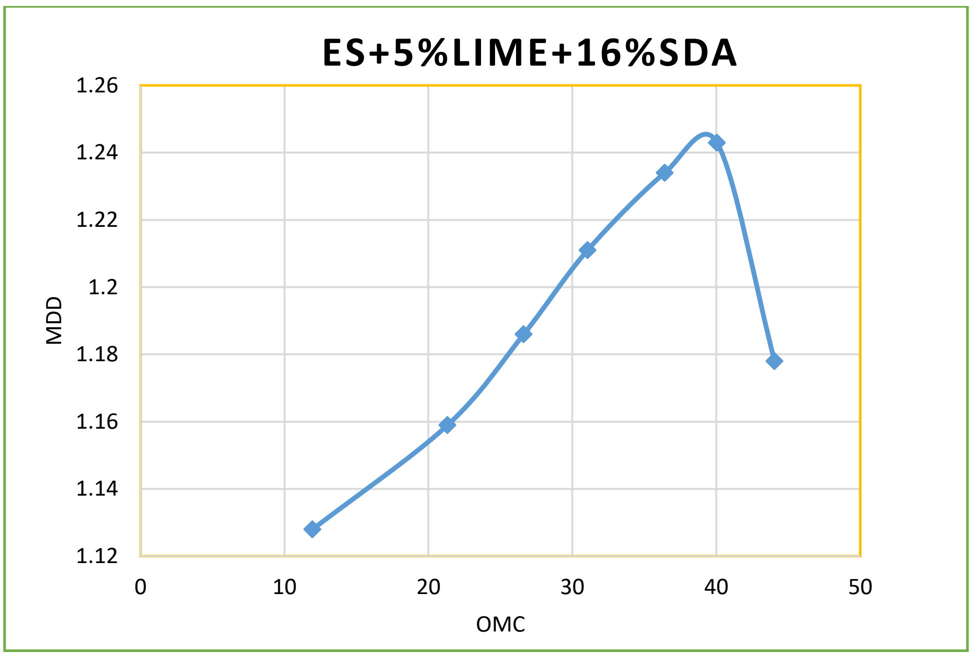


Table F -10 Compaction test result for TP-1 expansive soil with 5%lime +16% SDA

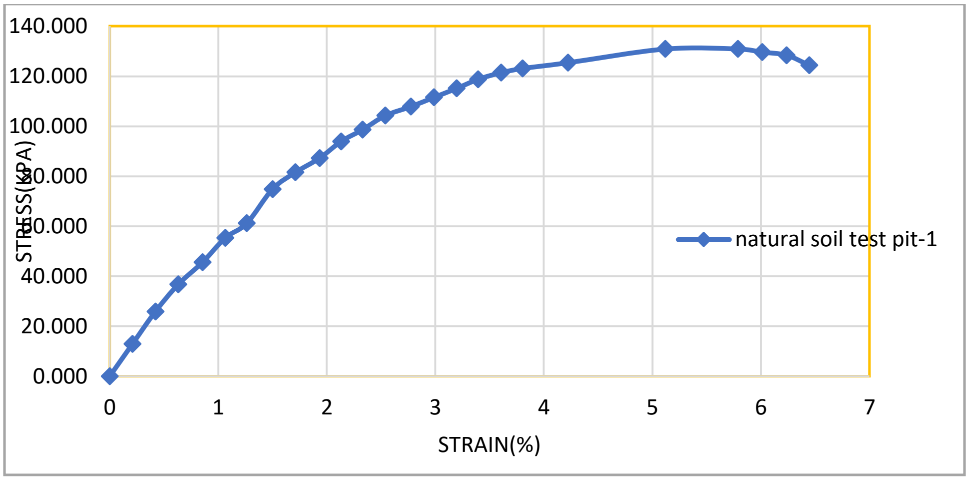
Trial No	1	2	3	4	5	6	7
Wt. of Mold +wet soil (g)	4952	5088	5174	5258	5344	5396	5362
Wt. of mold(g)	3760	3760	3760	3760	3760	3760	3760
Wt. of wet soil (g)	1192	1328	1414	1498	1584	1636	1602
Volume of mold cm3	944	944	944	944	944	944	944
Wet density, (g/cm3)	1.263	1.407	1.498	1.587	1.678	1.733	1.697
moisture content determination							
Mass of can (g)	30.17	30.21	30.20	30.19	30.24	29.77	29.82
mass of wet soil + can (g)	56.65	59.22	66.24	66.44	67.51	73.71	82.83
mass of dry soil + can(g)	53.82	54.12	58.66	57.85	57.65	61.28	66.62
mass of moisture(g),	2.83	5.10	7.58	8.59	9.85	12.44	16.21
Mass of dry soil (g)	23.65	23.91	28.46	27.66	27.42	31.51	36.80
Moisture content (%	11.95	21.33	26.62	31.06	35.94	39.47	44.06
dry density	1.128	1.159	1.183	1.211	1.234	1.243	1.178



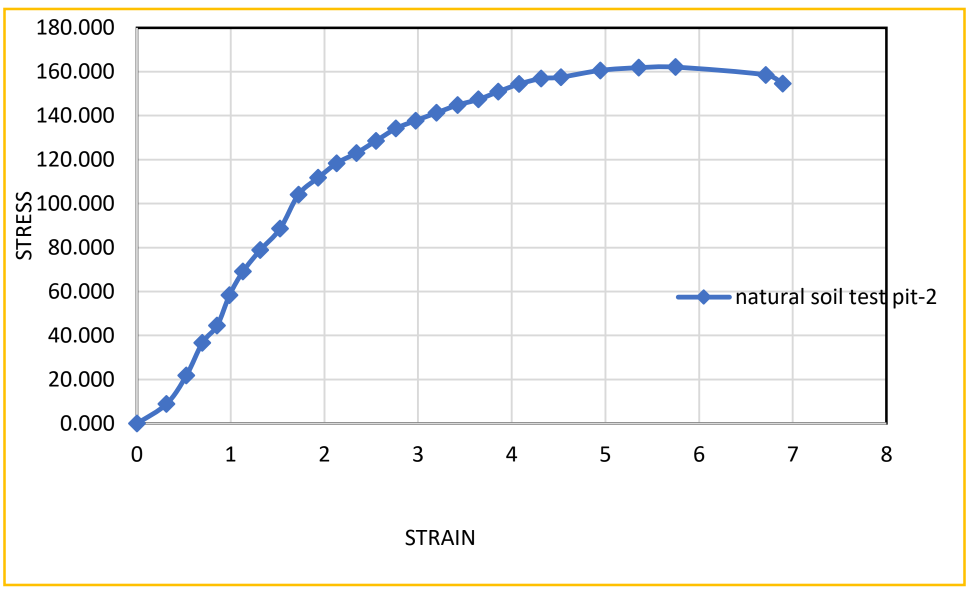
Appendix G - Unconfined Compressive Strength Test Results

Table G- 1 UCS test result for TP-1 natural expansive soil

diameter of sample =38mm			cross - sectional area of sample = 0.001m2				
length of sample 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area (m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0.000
0.16	0.002105	0.210526	0.013	0.001	0.00100211	3.9	12.973
0.32	0.004211	0.421053	0.026	0.001	0.00100423	5.7	25.891
0.48	0.006316	0.631579	0.037	0.001	0.00100636	9.3	36.766
0.65	0.008553	0.855263	0.046	0.001	0.00100863	11.1	45.607
0.81	0.010658	1.065789	0.056	0.001	0.00101077	12.9	55.403
0.96	0.012632	1.263158	0.062	0.001	0.00101279	14.7	61.217
1.14	0.015	1.5	0.076	0.001	0.00101523	16.5	74.860
1.3	0.017105	1.710526	0.083	0.001	0.0010174	18.3	81.580
1.47	0.019342	1.934211	0.089	0.001	0.00101972	20.1	87.279
1.62	0.021316	2.131579	0.096	0.001	0.00102178	21.8	93.954
1.77	0.023289	2.328947	0.101	0.001	0.00102384	23.6	98.648
1.93	0.025395	2.539474	0.107	0.001	0.00102606	25.4	104.283
2.11	0.027763	2.776316	0.111	0.001	0.00102856	27.2	107.918
2.27	0.029868	2.986842	0.115	0.001	0.00103079	29	111.565
2.43	0.031974	3.197368	0.119	0.001	0.00103303	30.8	115.195
2.58	0.033947	3.394737	0.123	0.001	0.00103514	32.6	118.824
2.74	0.036053	3.605263	0.126	0.001	0.0010374	34.4	121.457
2.89	0.038026	3.802632	0.128	0.001	0.00103953	36.1	123.133
3.21	0.042237	4.223684	0.131	0.001	0.0010441	37.9	125.467
3.89	0.051184	5.118421	0.138	0.001	0.00105395	46.9	130.937
4.4	0.057895	5.789474	0.139	0.001	0.00106145	48.6	130.953
4.57	0.060132	6.013158	0.138	0.001	0.00106398	50.4	129.702
4.74	0.062368	6.236842	0.137	0.001	0.00106652	52.2	128.456
4.9	0.064474	6.447368	0.133	0.001	0.00106892	54	124.425



qu=130.953kpa Cu=qu/2= 65.477kpa



qu=162.11kpa Cu= qu/2=81.06kpa

Table G- 2 UCS test result for TP-2 natural expansive soil

diameter of sample =38mm			cross - sectional area of sample = 0.002m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0.000
0.24	0.003158	0.315789	0.009	0.001	0.00100317	3.9	8.972
0.4	0.005263	0.526316	0.022	0.001	0.00100529	5.7	21.884
0.53	0.006974	0.697368	0.037	0.001	0.00100702	7.5	36.742
0.65	0.008553	0.855263	0.045	0.001	0.00100863	9.3	44.615
0.75	0.009868	0.986842	0.059	0.001	0.00100997	11.1	58.418
0.86	0.011316	1.131579	0.07	0.001	0.00101145	12.9	69.208
1	0.013158	1.315789	0.08	0.001	0.00101333	14.7	78.947
1.16	0.015263	1.526316	0.09	0.001	0.0010155	16.5	88.626
1.31	0.017237	1.723684	0.106	0.001	0.00101754	18.3	104.173
1.47	0.019342	1.934211	0.114	0.001	0.00101972	21.1	111.795
1.62	0.021316	2.131579	0.121	0.001	0.00102178	21.9	118.421
1.78	0.023421	2.342105	0.126	0.001	0.00102398	23.6	123.049
1.94	0.025526	2.552632	0.132	0.001	0.00102619	25.4	128.631
2.1	0.027632	2.763158	0.138	0.001	0.00102842	27.2	134.187
2.26	0.029737	2.973684	0.142	0.001	0.00103065	29	137.777
2.43	0.031974	3.197368	0.146	0.001	0.00103303	30.8	141.332
2.6	0.034211	3.421053	0.15	0.001	0.00103542	32.6	144.868
2.77	0.036447	3.644737	0.153	0.001	0.00103783	34.4	147.424
2.93	0.038553	3.855263	0.157	0.001	0.0010401	36.2	150.947
3.1	0.040789	4.078947	0.161	0.001	0.00104252	37.9	154.433
3.28	0.043158	4.315789	0.164	0.001	0.0010451	39.7	156.922
3.44	0.045263	4.526316	0.165	0.001	0.00104741	41.5	157.532
3.76	0.049474	4.947368	0.169	0.001	0.00105205	45.1	160.639
4.07	0.053553	5.355263	0.171	0.001	0.00105658	48.7	161.843
4.37	0.0575	5.75	0.172	0.001	0.00106101	52.2	162.110
5.1	0.067105	6.710526	0.17	0.001	0.00107193	61.2	158.592
5.24	0.068947	6.894737	0.166	0.001	0.00107405	62.9	154.555

Table G- 3 UCS test result for TP-1 expansive soil + 2%lime+98%ES

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.13	0.001711	0.171053	0.012	0.001	0.00100171	7.8	11.979
0.35	0.004605	0.460526	0.022	0.001	0.00100463	9.6	21.899
0.53	0.006974	0.697368	0.036	0.001	0.00100702	11.5	35.749
0.73	0.009605	0.960526	0.046	0.001	0.0010097	13.4	45.558
0.94	0.012368	1.236842	0.052	0.001	0.00101252	15.2	51.357
1.05	0.013816	1.381579	0.06	0.001	0.00101401	17.1	59.171
1.14	0.015	1.5	0.068	0.001	0.00101523	18.9	66.980
1.28	0.016842	1.684211	0.08	0.001	0.00101713	20.8	78.653
1.46	0.019211	1.921053	0.088	0.001	0.00101959	22.7	86.309
1.62	0.021316	2.131579	0.096	0.001	0.00102178	24.5	93.954
1.79	0.023553	2.355263	0.108	0.001	0.00102412	26.4	105.456
1.95	0.025658	2.565789	0.122	0.001	0.00102633	28.2	118.870
2.08	0.027368	2.736842	0.132	0.001	0.00102814	30.1	128.387
2.27	0.029868	2.986842	0.144	0.001	0.00103079	32	139.699
2.43	0.031974	3.197368	0.15	0.001	0.00103303	33.8	145.204
2.69	0.035395	3.539474	0.164	0.001	0.00103669	35.7	158.195
2.86	0.037632	3.763158	0.174	0.001	0.0010391	37.5	167.452
3.02	0.039737	3.973684	0.188	0.001	0.00104138	39.4	180.529
3.28	0.043158	4.315789	0.2	0.001	0.0010451	41.2	191.368
3.45	0.045395	4.539474	0.21	0.001	0.00104755	43.1	200.467
3.61	0.0475	4.75	0.216	0.001	0.00104987	45	205.740
3.87	0.050921	5.092105	0.222	0.001	0.00105365	46.8	210.696
4.05	0.053289	5.328947	0.228	0.001	0.00105629	48.7	215.850
4.31	0.056711	5.671053	0.232	0.001	0.00106012	50.5	218.843
4.48	0.058947	5.894737	0.23	0.001	0.00106264	52.4	216.442
4.66	0.061316	6.131579	0.228	0.001	0.00106532	54.2	214.020
4.83	0.063553	6.355263	0.224	0.001	0.00106787	56.1	209.764
5	0.065789	6.578947	0.222	0.001	0.00107042	58	207.395
5.18	0.068158	6.815789	0.212	0.001	0.00107314	59.8	197.551

Table G-4 UCS test result for TP-1 expansive soil +3%lime+97%ES

diameter of sample =38mm			cross-sectional area of sample .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.088	0.001158	0.11578947	0.008	0.001	0.001001	11.3	7.991
0.242	0.003184	0.31842105	0.026	0.001	0.001003	13.2	25.917
0.396	0.005211	0.52105263	0.041	0.001	0.001005	15.1	40.786
0.539	0.007092	0.70921053	0.057	0.001	0.001007	16.9	56.596
0.682	0.008974	0.89736842	0.075	0.001	0.001009	18.8	74.327
0.801	0.010539	1.05394737	0.089	0.001	0.001011	20.7	88.062
0.912	0.012	1.2	0.102	0.001	0.001012	22.5	100.776
1.068	0.014053	1.40526316	0.114	0.001	0.001014	24.4	112.398
1.234	0.016237	1.62368421	0.132	0.001	0.001017	26.3	129.857
1.424	0.018737	1.87368421	0.148	0.001	0.001019	28.2	145.227
1.535	0.020197	2.01973684	0.164	0.001	0.001021	30	160.688
1.836	0.024158	2.41578947	0.193	0.001	0.001025	31.9	188.338
2.036	0.026789	2.67894737	0.213	0.001	0.001028	33.8	207.294
2.236	0.029421	2.94210526	0.221	0.001	0.001030	35.6	214.498
2.465	0.032434	3.24342105	0.225	0.001	0.001034	37.5	217.702
2.69	0.035395	3.53947368	0.215	0.001	0.001037	39.4	207.390
3.05	0.040132	4.01315789	0.196	0.001	0.001042	41.5	188.134
3.253	0.042803	4.28026316	0.175	0.001	0.001045	43.4	167.510
3.425	0.045066	4.50657895	0.156	0.001	0.001047	45	148.970
3.742	0.049237	4.92368421	0.134	0.001	0.001052	46.8	127.402

Table G-5 UCS test result for TP-1 expansive soil 4%lime+96%ES

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.154	0.002026	0.20263158	0.009	0.001	0.001002	2.2	8.982
0.33	0.004342	0.43421053	0.018	0.001	0.0010044	4.1	17.922
0.506	0.006658	0.66578947	0.033	0.001	0.0010067	5.9	32.780
0.682	0.008974	0.89736842	0.052	0.001	0.0010091	7.8	51.533
1.024	0.013474	1.34736842	0.079	0.001	0.0010137	9.7	77.936
1.204	0.015842	1.58421053	0.099	0.001	0.0010161	11.6	97.432
1.35	0.017763	1.77631579	0.119	0.001	0.0010181	13.4	116.886
1.508	0.019842	1.98421053	0.138	0.001	0.0010202	15.3	135.262
1.676	0.022053	2.20526316	0.172	0.001	0.0010225	17.2	168.207
1.846	0.024289	2.42894737	0.204	0.001	0.0010249	19.1	199.045
2.038	0.026816	2.68157895	0.246	0.001	0.0010276	20.9	239.403
2.229	0.029329	2.93289474	0.271	0.001	0.0010302	22.8	263.052
2.443	0.032145	3.21447368	0.284	0.001	0.0010332	24.7	274.871
2.669	0.035118	3.51184211	0.289	0.001	0.0010364	26.6	278.851
2.874	0.037816	3.78157895	0.260	0.001	0.0010393	28.4	250.168
3.078	0.0405	4.05	0.228	0.001	0.0010422	30.3	218.766

Table G-6 UCS test result for TP-1 expansive soil +5%lime+95%ES

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0.000	0	0	0	0.001	0.001	0	0
0.152	0.002	0.2	0.011	0.001	0.001002	11.5	10.978
0.313	0.004118	0.41184211	0.028	0.001	0.0010041	13.4	27.885
0.526	0.006921	0.69210526	0.05	0.001	0.001007	15.3	49.654
0.677	0.008908	0.89078947	0.072	0.001	0.001009	17.2	71.359
1.046	0.013763	1.37631579	0.092	0.001	0.001014	19	90.734
1.190	0.015658	1.56578947	0.126	0.001	0.0010159	20.9	124.027
1.401	0.018434	1.84342105	0.144	0.001	0.0010188	22.8	141.345
1.646	0.021658	2.16578947	0.182	0.001	0.0010221	24.7	178.058
1.924	0.025316	2.53157895	0.227	0.001	0.001026	26.5	221.253
2.180	0.028684	2.86842105	0.278	0.001	0.0010295	28.4	270.026
2.225	0.029276	2.92763158	0.329	0.001	0.0010302	30.3	319.368
2.343	0.030829	3.08289474	0.342	0.001	0.0010318	32.2	331.457
2.533	0.033329	3.33289474	0.361	0.001	0.0010345	34.1	348.968
2.813	0.037013	3.70131579	0.386	0.001	0.0010384	35.9	371.713
2.880	0.037895	3.78947368	0.378	0.001	0.0010394	37.8	363.676
2.950	0.038816	3.88157895	0.362	0.001	0.0010404	39.7	347.949
3.054	0.040184	4.01842105	0.343	0.001	0.0010419	41.5	329.217
3.168	0.041684	4.16842105	0.322	0.001	0.0010435	43.2	308.578
3.318	0.043658	4.36578947	0.296	0.001	0.0010457	44.6	283.077
3.421	0.045013	4.50131579	0.277	0.001	0.0010471	46.4	264.531

Table G-7 UCS test result for TP-1 5%lime+4%SDA+91%ES uncured

diameter of sample =38mm			cross-sectional area of sample = .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.183	0.002408	0.240789	0.013	0.001	0.001002	6	12.969
0.367	0.004829	0.482895	0.031	0.001	0.001005	7.8	30.850
0.645	0.008487	0.848684	0.06	0.001	0.001009	9.7	59.491
0.922	0.012132	1.213158	0.099	0.001	0.001012	11.6	97.799
1.133	0.014908	1.490789	0.138	0.001	0.001015	13.5	135.943
1.29	0.016974	1.697368	0.166	0.001	0.001017	15.3	163.182
1.379	0.018145	1.814474	0.187	0.001	0.001018	17.2	183.607
1.546	0.020342	2.034211	0.226	0.001	0.001021	19.1	221.403
1.712	0.022526	2.252632	0.253	0.001	0.001023	20.9	247.301
1.9	0.025	2.5	0.289	0.001	0.001026	22.8	281.775
2.078	0.027342	2.734211	0.31	0.001	0.001028	24.7	301.524
2.267	0.029829	2.982895	0.331	0.001	0.001031	26.6	321.127
2.445	0.032171	3.217105	0.343	0.001	0.001033	28.4	331.965
2.634	0.034658	3.465789	0.355	0.001	0.001036	30.3	342.696
2.868	0.037737	3.773684	0.368	0.001	0.001039	32.2	354.113
3.071	0.040408	4.040789	0.356	0.001	0.001042	34	341.615
3.262	0.042921	4.292105	0.334	0.001	0.001045	35.9	319.664
3.322	0.043711	4.371053	0.313	0.001	0.001046	37.7	299.319
3.414	0.044921	4.492105	0.291	0.001	0.001047	39.6	277.928
3.522	0.046342	4.634211	0.266	0.001	0.001049	40.4	253.673

Table G-8 UCS test result for TP-1 5%lime+8%SDA+87%ES uncured

diameter of sample =38mm			cross - sectional area of sample = .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.171	0.00225	0.225	0.03	0.001	0.001002	11.6	29.933
0.297	0.003908	0.390789	0.049	0.001	0.001004	13.4	48.809
0.531	0.006987	0.698684	0.086	0.001	0.001007	15.3	85.399
0.612	0.008053	0.805263	0.104	0.001	0.001008	17.2	103.163
0.791	0.010408	1.040789	0.143	0.001	0.001011	19	141.512
0.971	0.012776	1.277632	0.179	0.001	0.001013	20.9	176.713
1.241	0.016329	1.632895	0.245	0.001	0.001017	22.8	240.999
1.556	0.020474	2.047368	0.299	0.001	0.001021	24.7	292.878
1.759	0.023145	2.314474	0.34	0.001	0.001024	26.6	332.131
1.938	0.0255	2.55	0.374	0.001	0.001026	28.5	364.463
2.098	0.027605	2.760526	0.393	0.001	0.001028	30.3	382.151
2.25	0.029605	2.960526	0.409	0.001	0.001031	32.2	396.891
2.391	0.031461	3.146053	0.415	0.001	0.001032	34.1	401.944
2.542	0.033447	3.344737	0.425	0.001	0.001035	35.9	410.785
2.789	0.036697	3.669737	0.417	0.001	0.001038	37.8	401.697
2.949	0.038803	3.880263	0.385	0.001	0.00104	39.6	370.061
3.1	0.040789	4.078947	0.356	0.001	0.001043	41.5	341.479
3.245	0.042697	4.269737	0.322	0.001	0.001045	43.4	308.251
3.394	0.044658	4.465789	0.286	0.001	0.001047	45.2	273.228

Table G-9 UCS test result for TP-1 5%lime+12%SDA+83%ES uncured

diameter of sample =38mm			cross - sectional area of sample = .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.19	0.0025	0.25	0.03	0.001	0.00100251	11.6	29.925
0.33	0.004342	0.434211	0.055	0.001	0.00100436	13.4	54.761
0.59	0.007763	0.776316	0.096	0.001	0.00100782	15.3	95.255
0.68	0.008947	0.894737	0.118	0.001	0.00100903	17.2	116.944
0.88	0.011579	1.157895	0.168	0.001	0.00101171	19	166.055
1.08	0.014211	1.421053	0.197	0.001	0.00101442	20.9	194.201
1.38	0.018158	1.815789	0.252	0.001	0.00101849	22.8	247.424
1.73	0.022763	2.276316	0.302	0.001	0.00102329	24.7	295.126
1.97	0.025921	2.592105	0.343	0.001	0.00102661	26.6	334.109
2.17	0.028553	2.855263	0.378	0.001	0.00102939	28.5	367.207
2.35	0.030921	3.092105	0.407	0.001	0.00103191	30.3	394.415
2.52	0.033158	3.315789	0.423	0.001	0.0010343	32.2	408.974
2.69	0.035395	3.539474	0.429	0.001	0.00103669	34.1	413.816
2.86	0.037632	3.763158	0.416	0.001	0.0010391	35.9	400.345
3.14	0.041316	4.131579	0.381	0.001	0.0010431	37.8	365.259
3.32	0.043684	4.368421	0.359	0.001	0.00104568	39.6	343.317
3.49	0.045921	4.592105	0.339	0.001	0.00104813	41.5	323.433
3.66	0.048158	4.815789	0.311	0.001	0.00105059	43.4	296.023
3.83	0.050395	5.039474	0.259	0.001	0.00105307	45.2	245.948

Table G-10 UCS test result for TP-1 5%lime+16%SDA+79%ES uncured

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.189	0.002487	0.248684	0.015	0.001	0.001002	15.2	14.963
0.332	0.004368	0.436842	0.033	0.001	0.001004	17.1	32.856
0.486	0.006395	0.639474	0.062	0.001	0.001006	19	61.604
0.625	0.008224	0.822368	0.086	0.001	0.001008	20.9	85.293
0.767	0.010092	1.009211	0.113	0.001	0.00101	22.7	111.860
0.92	0.012105	1.210526	0.141	0.001	0.001012	24.6	139.293
1.054	0.013868	1.386842	0.167	0.001	0.001014	26.5	164.684
1.17	0.015395	1.539474	0.187	0.001	0.001016	28.4	184.121
1.262	0.016605	1.660526	0.211	0.001	0.001017	30.2	207.496
1.396	0.018368	1.836842	0.235	0.001	0.001019	32.1	230.683
1.54	0.020263	2.026316	0.264	0.001	0.001021	34	258.651
1.691	0.02225	2.225	0.294	0.001	0.001023	35.8	287.459
1.851	0.024355	2.435526	0.319	0.001	0.001025	37.7	311.231
1.994	0.026237	2.623684	0.335	0.001	0.001027	39.6	326.211
2.145	0.028224	2.822368	0.352	0.001	0.001029	41.5	342.065
2.273	0.029908	2.990789	0.371	0.001	0.001031	43.3	359.904
2.421	0.031855	3.185526	0.382	0.001	0.001033	45.2	369.831
2.571	0.033829	3.382895	0.396	0.001	0.001035	47.1	382.604
2.72	0.035789	3.578947	0.403	0.001	0.001037	48.9	388.577
2.852	0.037526	3.752632	0.395	0.001	0.001039	50.8	380.177
3.002	0.0395	3.95	0.374	0.001	0.001041	52.7	359.227
3.143	0.041355	4.135526	0.353	0.001	0.001043	54.5	338.402
3.284	0.043211	4.321053	0.326	0.001	0.001045	56.4	311.913

Table G-11 UCS test result for TP-1 5%lime+4%SDA+91%ES 7 days cured

diameter of sample =38mm			cross-sectional area of sample = .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.155	0.002039	0.203947	0.013	0.001	0.001002	6	12.973
0.312	0.004105	0.410526	0.033	0.001	0.001004	7.8	32.865
0.527	0.006934	0.693421	0.064	0.001	0.001007	9.7	63.556
0.755	0.009934	0.993421	0.103	0.001	0.00101	11.6	101.977
1.009	0.013276	1.327632	0.144	0.001	0.001013	13.5	142.088
1.147	0.015092	1.509211	0.174	0.001	0.001015	15.3	171.374
1.227	0.016145	1.614474	0.2	0.001	0.001016	17.2	196.771
1.375	0.018092	1.809211	0.244	0.001	0.001018	19.1	239.586
1.523	0.020039	2.003947	0.291	0.001	0.00102	20.9	285.169
1.691	0.02225	2.225	0.327	0.001	0.001023	22.8	319.724
1.85	0.024342	2.434211	0.348	0.001	0.001025	24.7	339.529
2.083	0.027408	2.740789	0.372	0.001	0.001028	26.6	361.804
2.246	0.029553	2.955263	0.384	0.001	0.00103	28.4	372.652
2.42	0.031842	3.184211	0.362	0.001	0.001033	30.3	350.473
2.534	0.033342	3.334211	0.338	0.001	0.001034	32.2	326.730
2.802	0.036868	3.686842	0.253	0.001	0.001038	34	243.672
2.977	0.039171	3.917105	0.202	0.001	0.001041	35.9	194.087

Table G-12 UCS test result for TP-1 5%lime+8%SDA+87%ES 7 days cured

diameter of sample =38mm			cross - sectional area of sample = .001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.152	0.002	0.2	0.03	0.001	0.001002	11.6	29.940
0.264	0.003474	0.347368	0.054	0.001	0.001003	13.4	53.812
0.472	0.006211	0.621053	0.095	0.001	0.001006	15.3	94.410
0.544	0.007158	0.715789	0.115	0.001	0.001007	17.2	114.177
0.704	0.009263	0.926316	0.156	0.001	0.001009	19	154.555
0.864	0.011368	1.136842	0.195	0.001	0.001011	20.9	192.783
1.104	0.014526	1.452632	0.249	0.001	0.001015	22.8	245.383
1.384	0.018211	1.821053	0.298	0.001	0.001019	24.7	292.573
1.576	0.020737	2.073684	0.339	0.001	0.001021	26.6	331.970
1.736	0.022842	2.284211	0.373	0.001	0.001023	28.5	364.480
1.880	0.024737	2.473684	0.402	0.001	0.001025	30.3	392.056
2.016	0.026526	2.652632	0.418	0.001	0.001027	32.2	406.912
2.152	0.028316	2.831579	0.424	0.001	0.001029	34.1	411.994
2.288	0.030105	3.010526	0.411	0.001	0.001031	35.9	398.627
2.512	0.033053	3.305263	0.376	0.001	0.001034	37.8	363.572
2.656	0.034947	3.494737	0.355	0.001	0.001036	39.6	342.594
2.792	0.036737	3.673684	0.335	0.001	0.001038	41.5	322.693
2.928	0.038526	3.852632	0.307	0.001	0.00104	43.4	295.172
3.064	0.040316	4.031579	0.256	0.001	0.001042	45.2	245.679

Table G-13 UCS test result for TP-1 5%lime+12%SDA+83%ES 7 days cured

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.107	0.001408	0.140789	0.013	0.001	0.001001	2.2	12.982
0.321	0.004224	0.422368	0.034	0.001	0.001004	7.8	33.856
0.491	0.006461	0.646053	0.064	0.001	0.001007	9.7	63.587
0.652	0.008579	0.857895	0.087	0.001	0.001009	11.6	86.254
0.75	0.009868	0.986842	0.112	0.001	0.00101	13.4	110.895
0.866	0.011395	1.139474	0.134	0.001	0.001012	15.3	132.473
0.961	0.012645	1.264474	0.159	0.001	0.001013	17.2	156.989
1.116	0.014684	1.468421	0.2	0.001	0.001015	19.1	197.063
1.288	0.016947	1.694737	0.248	0.001	0.001017	21	243.797
1.442	0.018974	1.897368	0.286	0.001	0.001019	22.8	280.574
1.587	0.020882	2.088158	0.315	0.001	0.001021	24.7	308.422
1.88	0.024737	2.473684	0.366	0.001	0.001025	26.6	356.946
2.038	0.026816	2.681579	0.394	0.001	0.001028	28.5	383.435
2.205	0.029013	2.901316	0.422	0.001	0.00103	30.4	409.756
2.412	0.031737	3.173684	0.439	0.001	0.001033	32.2	425.068
2.628	0.034579	3.457895	0.426	0.001	0.001036	34.1	411.269
2.796	0.036789	3.678947	0.404	0.001	0.001038	36	389.137
2.983	0.03925	3.925	0.374	0.001	0.001041	37.8	359.321
3.124	0.041105	4.110526	0.346	0.001	0.001043	38.5	331.778
3.231	0.042513	4.251316	0.322	0.001	0.001044	40.3	308.311
3.342	0.043974	4.397368	0.292	0.001	0.001046	42.4	279.160

Table G-14 UCS test result for TP-1 5%lime+16%SDA+79%ES 7 days cured

diameter of sample =38mm			cross-sectional area of sample =.001m2				
length of sample = 76mm							
Axial Deformation [mm]	strain	Axial strain [%]	Axial Load in [KN]	Area(m^2)	Corrected Area [m^2]	Time(s)	Axial stress in [Kpa]
0	0	0	0	0.001	0.001	0	0
0.208	0.002737	0.273684	0.015	0.001	0.001003	15.2	14.958947
0.366	0.004816	0.481579	0.033	0.001	0.001005	17.1	32.841079
0.535	0.007039	0.703947	0.061	0.001	0.001007	19	60.570592
0.693	0.009118	0.911842	0.085	0.001	0.001009	20.9	84.224934
0.861	0.011329	1.132895	0.112	0.001	0.001011	22.7	110.73116
1.020	0.013421	1.342105	0.14	0.001	0.001014	24.6	138.12105
1.121	0.01475	1.475	0.165	0.001	0.001015	26.5	162.56625
1.245	0.016382	1.638158	0.185	0.001	0.001017	28.4	181.96941
1.349	0.01775	1.775	0.209	0.001	0.001018	30.2	205.29025
1.488	0.019579	1.957895	0.241	0.001	0.00102	32.1	236.28147
1.594	0.020974	2.097368	0.271	0.001	0.001021	34	265.31613
1.750	0.023026	2.302632	0.301	0.001	0.001024	35.8	294.06908
1.916	0.025211	2.521053	0.326	0.001	0.001026	37.7	317.78137
2.064	0.027158	2.715789	0.35	0.001	0.001028	39.6	340.49474
2.220	0.029211	2.921053	0.371	0.001	0.00103	41.5	360.16289
2.377	0.031276	3.127632	0.387	0.001	0.001032	43.3	374.89607
2.533	0.033329	3.332895	0.398	0.001	0.001034	45.2	384.73508
2.690	0.035395	3.539474	0.408	0.001	0.001037	47.1	393.55895
2.847	0.037461	3.746053	0.417	0.001	0.001039	48.9	401.37896
2.985	0.039276	3.927632	0.406	0.001	0.001041	50.8	390.05382
3.051	0.040145	4.014474	0.388	0.001	0.001042	52.7	372.42384
3.173	0.04175	4.175	0.359	0.001	0.001044	54.5	344.01175
3.220	0.042368	4.236842	0.342	0.001	0.001044	56.4	327.51
3.341	0.043961	4.396053	0.325	0.001	0.001046	58.2	310.71283
3.460	0.045526	4.552632	0.302	0.001	0.001048	60.1	288.25105

Appendix H - Consolidation Test Results

Table H-1 Consolidation test result for TP-1 of natural soil

Consolidation Test

Sample No___BE-N-01________________________Project No___________________
Location ______________________________Boring No___________________
Depth of sample_______________________Date of Test__________________
Tested by_______________________Description of soil__Natural soil ___________

[A]Specimen data

Wt. of specimen Ring +soil	162.24gm
Wt of specimen Ring	76.74gm
Wt of specimen	85.5gm
Diameter of specimen	6cm
Initial Height of Specimen	2cm
Initial Wet density	1.512 gm/ cm^3
Initial Moisture Content	49.3%
Initial Dry density of specimen	1.013 gm/ cm^3
Specific gravity	2.714
Volume of Voids in soil specimen	35.45 cm^3
Volume of Water in soil specimen	28.22 cm^3
Initial degree of saturation	79.6%
Final Moisture content %	52.21%
Final degree of saturation	81.86%
Final volume of solid	20.35 cm^3 =HS= 0.719cm
Initial void ratio	1.779

[B]Time versus Deformation data Pressure increment from .8kg/cm^2__to___6.4kg/cm^2

Date	Time	Elapsed time(min)	Deformation Dial Reading				
			0.8kg/cm^2	**1.6kg/cm^2**	**3.2kg/cm^2**	**6.4kg/cm^2**	**Rebound**
20/07/22	9:05AM	0.10	0.06	0.29	1.28	2.77	
		0.25	0.079	0.30	1.33	2.79	1.6=2.680
		0.50	0.089	0.31	1.36	2.816	3.2=3.454
		1.00	0.103	0.329	1.45	2.844	6.4=3.867
		2.00	0.115	0.35	1.497	2.888	
		4.00	0.129	0.371	1.559	2.948	
		8.00	0.136	0.40	1.721	3.030	
		15.00	0.143	0.424	1.842	3.122	
		30.00	0.152	0.458	2.018	3.275	
		60.00	0.159	0.491	2.190	3.510	
		120.00	0.178	0.53	2.350	3.694	
		240.00	0.205	0.571	2.465	3.898	
		480.00	0.244	0.610	2.540	4.03	
21/07/22	9:05AM	1440.00	0.26	0.640	2.60	4.12	

[C] Void ratio

Pressure	Initial Deformation dial reading @ beginning of 1st Loading	Deformation dial reading Representing 100% primary consolidation	Change in thickness of specimen ΔH (cm)	Change in Void Ratio ΔH/Hs	Void Ratio
					1.779
80kpa	0	0.244	0.0244	0.034	1.745
160kpa	0	0.610	0.0610	0.085	1.694
320kpa	0	2.510	0.2510	0.349	1.430
640kpa	0	3.920	0.3920	0.545	1.234
Rebound Value					
80kpa	0	2.078	0.2078	0.289	1.490
160kpa	0	2.680	0.2680	0.373	1.406
320kpa	0	3.454	0.3454	0.480	1.299
640kpa	0	3.867	0.3867	0.538	1.241

[C] Coefficient of Consolidation

pressure	Initial Height of specimen @ beginning of test H_o	Deformation dial reading @ 50% Consolidation	Thickness of specimen @50% consolidation	Half-Thickness of specimen @50% consolidation	Time for 50% consolidation (Min.)	Coefficient of consolidation cm^2/min
	2.00					
80kpa	2.00	0.01795	1.982	0.991	36.67	$5.28*10^{-3}$
160kpa	2.00	0.0464	1.954	0.977	44.45	$4.23*10^{-3}$
320kpa	2.00	0.1892	1.811	0.906	46.26	$3.50*10^{-3}$
640kpa	2.00	0.3353	1.765	0.882	53.96	$2.84*10^{-3}$

Table H-2 Consolidation test result for TP-1 of 5%lime+ 4%SDA +91%ES

Sample No BE-N-S-L-02______Project No____________________
Location ______________________________Boring No____________________
Depth of sample________________________Date of Test_________________
Tested by__Bekele________Description of soil_ Natural soil+SDA, 4%+Lime 5%

[A]Specimen data

Wt of specimen Ring +soil	162.96gm
Wt of specimen Ring	76.42gm
Wt of specimen	86.54gm
Diameter of specimen	6cm
Initial Height of Specimen	2cm
Initial Wet density	1.531 gm/ cm^3
Initial Moisture Content	49.34%
Initial Dry density of specimen	1.025 gm/ cm^3
Specific gravity	2.703
Volume of Voids in soil specimen	35.09cm^3
Volume of Water in soil specimen	28.60 cm^3
Initial degree of saturation	80.5%
Final Moisture content %	56.44%
Final degree of saturation	86.52%
Final volume of solid	20.46 cm^3 ==H_S = 0.724cm
Initial void ratio	1.753

[B]Time versus Deformation data Pressure increment from 1.6kg/cm^2__to___6.4kg/cm^2

Date	Time	Elapsed time(min)	Deformation Dial Reading			
			1.6kg/cm^2	**3.2kg/cm^2**	**6.4kg/cm^2**	**Rebound**
02/08/2022	**9:14AM**	0.10	0.241	0.566	0.942	
		0.25	0.247	0.585	0.972	1.6=0.586
		0.50	0.253	0.592	0.985	3.2=0.784
		1.00	0.256	0.601	0.997	6.4=0.862
		2.00	0.258	0.601	1.008	
		4.00	0.263	0.605	1.02	
		8.00	0.267	0.613	1.034	
		15.00	0.268	0.617	1.04	
		30.00	0.272	0.626	1.052	
		60.00	0.277	0.63	1.064	
		120.00	0.288	0.639	1.07	
		240.00	0.306	0.646	1.082	
		480.00	0.318	0.657	1.096	
03/08/2022	**9:14AM**	1440.00	0.327	0663	1.102	

[C] Void ratio

Pressure	Initial Deformation dial reading @ beginning of 1st Loading	Deformation dial reading Representing 100% primary consolidation	Change in thickness of specimen ΔH (cm)	Change in Void Ratio ΔH/Hs	Void Ratio
					1.753
80kpa	0	0.190	0.0190	0.0262	1.727
160kpa	0	0.310	0.031	0.0428	1.684
320kpa	0	0.654	0.0654	0.0903	1.594
640kpa	0	1.09	0.109	0.1506	1.448
Rebound Value					
80kpa	0	0.544	0.0544	0.075	1.687
160kpa	0	0.642	0.0642	0.0887	1.589
320kpa	0	0.784	0.0784	0.1083	1.481
640kpa	0	0.868	0.0873	0.1208	1.360

[C] Coefficient of Consolidation

pressure	Initial Height of specimen@ beginning of test H_o(cm)	Deformation dial reading @ 50% Consolidation (cm)	Thickness of specimen @50% consolidation	Half-Thickness of specimen @50% consolidation	Time for 50% consolidation (min.)	Coefficient of consolidation cm^2/min
	2					
80kpa	2	0.0144	1.986	0.993	37.63	$5.16*10^{-3}$
160kpa	2	0.0283	1.972	0.986	44.82	$4.27*10^{-3}$
320kpa	2	0.0622	1.938	0.969	51.33	$3.60*10^{-3}$
640kpa	2	0.0865	1.914	0.957	72.67	$2.46*10^{-3}$

Appendix I: Photos taken during the study