Effects of Maturation Period on the Geotechnical Properties of Bagasse Ash and Lime Stabilized Expansive Soil: In Case of Gelan Town

Bilisuma Heyi Huriso

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

October, 2023
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DECLARATION

I hereby declare that this Master Thesis entitled "Effects of Maturation Period on the Geotechnical Properties of Bagasse Ash and Lime Stabilized Expansive Soil: In Case of Gelan Town "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.

Bilisuma Heyi	____________________	____________________
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I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "Effects of Maturation Period on the Geotechnical Properties of Bagasse Ash and Lime Stabilized Expansive Soil: In Case of Gelan Town" prepared under my guidance by Bilisuma Heyi submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Civil Engineering (specialization in Geotechnical Engineering). Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

Dr.Argaw Asha	____________________	________________
Advisor	Signature	Date

APPROVAL OF BOARD OF EXAMINER

I, the advisor of this thesis entitled "Effects of Maturation Period on the Geotechnical Properties of Bagasse Ash and Lime Stabilized Expansive Soil: In Case of Gelan Town" and developed by Bilisuma Heyi, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Dr.Argaw Asha ____________ ____________________ ____________________

Advisor Signature Date

We, the undersigned, members of the Board of Examiners of the thesis by Bilisuma Heyi have read and evaluated the thesis entitled "Effects of Maturation Period on the Geotechnical Properties of Bagasse Ash and Lime Stabilized Expansive Soil: In Case of Gelan Town" 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 (specialization in Geotechnical Engineering).

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Finally, approval and acceptance of the thesis is contingent upon the 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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ABSTRACT

Until now for expansive soils different stabilization techniques were applied, however in this study the combined effect of bagasse ash and lime was investigated. The aim of this study is to evaluate the effects of maturation period on the geotechnical properties of bagasse ash and lime stabilized expansive soil. For this study expansive soil samples from two test pits were collected to be identified as the weakest test pit based on their plasticity property, swelling property and compressive strength and further the weakest soil, selected additives and soil-additive mixtures were characterized for their mineralogical compositions using X-ray diffraction (XRD) analysis. Atterberg limit, free swell, specific gravity, compaction and UCS tests were used to evaluate properties of stabilized soil with bagasse ash at 0%, 5%, 10%, 15% and 20% by dry weight of soil. For evaluate the effects of maturation period on geotechnical properties of bagasse ash and lime stabilized expansive soil, the UCS test is observed for uncured and 7,14,28,60 and 90 days cured condition. The results of these tests have shown that the bagasse ash can reduce PI from 45.9% to 32.2%, MDD from 1.232g/cc to 1.187g/cc and FSI from 110% to 71%. The UCS value is shows increase up to 15% from 110.18kPa to 185.32kPa for uncured and from 116.12kPa to 195.8kPa for 7 day cured and it's decreased at 20%.So from result 15% is shows the best result and it was used as optimum value of BA. From this UCS result is increased up to 90 day cured for 15% only, but for 5% and 10% UCS value is increase up to 28 days then it shows decrease after 60 days. And also15% is combined by 3:1 (11.25% of BA and 3.75% of lime) and 2:2 (7.5% of BA and 7.5% of lime). The plastic limit, swelling characteristics, compaction test and UCS test were conducted to see the effects of lime on bagasse ash stabilized expansive soil. Again, the result shows better improvement of soil properties by reducing plasticity property when increasing the lime content the maximum dry density decreases and the optimum moisture content increases. The UCS was improved and shows increment trough maturation period rather than using bagasse ash alone. So, these two stabilizing materials can be used for ground improvement. The combination of 15% BA and 2:2 of 15% (7.7% of BA and 7.5% of lime) gave a better result at 90 day cured. From result the combination of BA and lime is more effective than the addition of bagasse ash alone for the strength improvement of expansive soil due to high content of Ca ion from lime and also the maturation period have positive effects on geotechnical properties to improvement of expansive soil related to strength because BA and lime is pozzolanic reaction that is time dependent to gain best strength.

Keywords: Bagasse ash, lime, Expansive soil, strength improvement, and X-ray diffraction (XRD)

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Standard of Testing Materials
BA	Bagasse ash
CBR	California Bearing Ratio
CH	Inorganic Highly Plastic Clays
CL	Inorganic Clays of Low Plasticity
FS	Free Swell
FSI	Free swell index
IS	Indian standard
LL	Liquid Limit
MDD	Maximum Dry Density
MH	Inorganic Silt of High Plasticity
ML	Inorganic Silt of Low Plasticity
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
OH	Organic Silts and Organic Clays of High Plasticity
OL	Organic Silts and Organic Clays of Low Plasticity
PI	Plasticity Index

PL	Plastic Limit
SCBA	Sugarcane bagasse ash
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 Background

Now a day's cities are expanding and land become scarce, as a result agricultural lands became construction sites(Afewerk, 2004). Most of the agricultural lands are black cotton soils which is not suitable for construction unless it is treated or replaced by a suitable construction material(Dinku., 1988). Expansive soils occur in many parts of the world. However, the problem of expansion and shrinkage is associated with high moisture changes(Mokhtari, 2012). The large volume change with the seasons of wetting and drying can cause extensive damages in civil engineering infrastructures; mainly on small buildings, shallow foundation and other lightly loaded structures(Melese, 2018) .

Civil engineering projects located in areas with soft or weak soils have traditionally incorporated improvement of soil properties by using various methods(Boardman, D.L., Glendinning, S., & Rogers, 2001). Soil Stabilization is being used for a variety of engineering project, the most common method being in the construction of road and pavements, where the main idea is to increase the strength or stability of soil and to reduce the construction cost by making best use of the locally available materials(Srilatha & Praveen, 2021).

Expansive soils expand significantly with the increase in their moisture content and shrink when the moisture content is decreased(Osinubi et a., 2009). An expansive soil expands due to the presence of largely reactive clay minerals like montmorillonite and smectite. Montmorillonite is composed of units made of two silica tetrahedral sheets with the center alumina octahedral sheet(Holtz & Gibbs, 1956). The outstanding feature of montmorillonite structure is that water and other polar molecules, such as certain organic molecules, can enter between the unit layers causing to expand(Blayi et al., 2020).

This continuous change in soil volume can cause light structure construct on this soil settle and crack. The problem of expansive soils has appeared as cracking and break-up of pavements, roadways, building foundations, slab-on-grade members, channel linings, irrigation systems, water pipelines, sewer lines, and gas pipelines(Endalew, 2016).

When civil work constructed on expansive soil, especially road, railways, and embankments, face many difficulties(Kebede et al., 2022). In Ethiopia most of the roads constructed, lightly loaded residential and commercial buildings, airfields, and a substantial amount of the newly planned railway routes in the country pass through the heart of expansive soils(Demesew.E, 2021). Expansive soils are located in area of central Ethiopia, following the major road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa - Debere Berehan, Addis Ababa - Gohatsion, Addis Ababa - Mojo(Kerima.H, 2022). Also, they cover areas like Mekelle, Bahirdar, Gambela, Arba Minch, and the most Southern, South-west, and South-east part of the capital Addis Ababa area in which the most major recent construction is being carried out(Blayi et al., 2020).

In Ethiopia, applying eco-friendly stabilizers for pavement stabilization is not a widespread practice(Wubshet, 2013). However, both traditional and non-traditional stabilizers have been introduced and applied to road construction. If the soil is expansive, we don't construct any structure on it before improving the strength properties(Kerima.H, 2022). So it should be replaced by another soil having strength according to the requirement, to achieve economy in construction. If the appropriate soil is not available in the nearby areas to replace the expansive soil, it has to be transported from a long distance, and then there is an increase in cost and time of construction(Demesew.E, 2021). Hence, there is a need for soil improvement by using locally available materials.

The damage caused by expansive soils is controlled, and proper operation of soil stabilization methods can significantly reduce the damage that results from these problem soils(Rogers et al., 1993). In an attempt to make the poor soils more appropriate for use in engineering work, different stabilization methods are used(Endalew, 2016). However, well-established stabilizers like cement, lime, and bitumen are associated with environmental challenges. This has attracted the attention of the looking for environmentally friendly, low cost, and sustainable stabilizers(Niyomukiza et al., 2021)**.**

In this study bagasse ash and lime is going to be used for the improvement of the expansive soil. Bagasse is a fibrous substance left after the birth of juice from sugarcane(Wubshet, 2013). It is used as a fuel in the sugarcane industry that produces steam for electricity generation and the residue obtained after incineration is known as bagasse ash(Hailu, 2011). The application of the waste attained from sugarcane has proved to be profitable with respect to the economy and considering the environmental aspect as well(Osinubi et al., 2009b). Bagasse ash is predominantly composed of silica, alumina, and iron oxide which is why it participates in the pozzolanic reactions(Payá et al., 2018). Bagasse ash is profitable in multiple applications whether it be in combination with concrete or with soil. It also used to protect the environment due to the disposal problem of bagasse ash it is available widely in Ethiopia from the sugar factories(Demesew, 2021).

1.2 Statement of the Problem

Construction on expansive soils has made problems for civil engineering projects in the world, Such as highways, railways, embankments, and foundations. In some situation, construction on such type of soil cannot be avoided, and the sources of suitable materials may not be near to the site and time consuming. A large part of Gelan town is also covered by expansive soil and it affects the stability of many light weight structures, residential buildings, and roads. Most of them are failed before their design life completion(Kebede et al., 2022). Therefore, the strength improvement of such type of soil is important. For reducing transportation of selected material cost and to reduce damages. The application of locally available materials and waste material is important for cost-effective and environmental benefits. From which bagasse ash and lime is more important. Even though bagasse ash and lime is a chemical additive when it is mixt with soil the chemical reaction between materials is continuous react through curing time. Limited research has been done on the maturation period on combination of expansive soil with bagasse ash and lime. Therefore, in this study the effect of maturation period of lime and bagasse ash to improve the geotechnical properties of the expansive soil was investigated.

1.3 Research Questions

1) What are the geotechnical properties of expansive soils in Gelan town?
2) What is the optimum content of the stabilizing materials?
3) What properties of the expansive soil will be improved?
4) Is there any effects of maturation period on the geotechnical properties of bagasse ash and lime stabilized expansive soil?

1.4 Objective

1.4.1 General Objective

The main objective of this research work is to study effects of maturation period on the geotechnical properties of bagasse ash and lime stabilized expansive soil.

1.4.2 Specific Objective

- To determine the basic geotechnical properties of expansive soils in Gelan town.
- To determine the optimum content of stabilizing materials needed to improve the expansive soil.
- To make a comparison on the properties of expansive soil before and after stabilization.
- To assess the effects of maturation period on the geotechnical properties of bagasse ash and lime stabilized expansive soil

1.5 Significance of the Study

To characterize and study the use of locally available industrial waste and chemical (i.e., Bagasse ash and lime) in the improvement of the engineering properties of problematic soil. To make it a suitable foundation material that meets the strength requirement by avoiding the cut, disposal, and fill material price and protecting the environment from disposal of bagasse ash on the environment and the effect on the society.

1.6 Scope and Limitation of the Study

In this study, samples of expansive soil were collected from Gelan town. To take sample soils, the area where expansive soils are found was identified by reviewing previous research on this area. By field identification, two test pits were excavated for soil sampling. A different laboratory test was conducted as per the ASTM manual and IS manual. The most problematic soil of all two test pits was selected to be stabilized with bagasse ash and lime.

- The study covers improving the strength characteristics of expansive soil using lime and bagasse ash uses different mix proportions. First, soil samples with different contents of bagasse ash (5%, 10%, 15% and 20%) were prepared. Then, the laboratory tests conducted were Atterberg limits, free swell index test, linear shrinkage test ,specific gravity test, standard proctor compaction test, unconfined compressive strength test and mineralogical characterization(XRD) conducted on soil- bagasse ash and lime mixtures. The UCS test result has been used to select the optimum bagasse ash. To study the effects of maturation period of those materials was limited only by unconfined compressive strength test result.

1.7 Organization of the Study

The thesis includes five chapters. In the first chapter the background, statement of the problem, objectives, research questions, significance of study and scope and limitation of study of the work is described. The second chapter deals with a brief literature review comprising a study on previous similar works that is important to this study about expansive soil, soil stabilization techniques with bagasse ash and lime, and industrial wastes. In the third chapter the method, description of the study area, the material used, procedures to be followed, and tools and techniques performed in the laboratory to achieve the objectives are presented. The fourth chapter presents the analysis and results obtained from the experimental work. Finally, in the fifth chapter conclusions and recommendations are made from the findings of the study.

CHAPTER TWO

LITERATURE REVIEW

2.1 Expansive Soils

Expansive soils are fine-grained soil or decomposed rocks that show huge volume changes when exposed to the fluctuations of moisture content(A. Tefera, 1999). volume changes behavior is likely to take place near the ground surface where it is directly regarding to seasonal and environmental variations(Dang et al., 2017). Most soil classification systems arbitrarily define clay particles as having an effective diameter of two microns (0.002mm) or less. Particle size alone doesn't determine clay minerals(Murthy, 2002). Probably the most important grain property of fine-grained soils is the mineralogical composition(Sridharan, 1990). For small size particles, the electrical forces acting on the surface of the particles are much greater than the gravitational force(Barasa, 2016). These particles are in a colloidal state. The colloidal particles consist primarily of clay minerals that were derived from parent rock by weathering(Holtz & Gibbs, 1956). The origin and distribution of expansive soils are generally a function of the geologic history, sedimentation, and present local climatic conditions(Nigussie, 2011). These three factors, acting individually or in combination, have contributed to the formation of earth materials which may present serious design and construction problems by the absorption of water and a resulting increase in the volume of the material(Bethlehem Worku, 2015). The term "expansive material" as used here signifies any earth materials which exhibit significant volume changes in the presence of water(Asres.E, 2017).

The parent materials of expansive soils may be classified into two groups(Chen, 1975). The first group comprises basic igneous rocks. Here feldspar and pyroxene minerals of the parent rocks decompose to form montmorillonite - the predominant mineral of expansive soil - and other secondary minerals(Sridharan, 1990). The second group comprises sedimentary rocks that contain montmorillonite(Chen, 1975). The expansive soils of Ethiopia are derived from both groups(T. Tefera & Yohannes, 1986).Potentially swelling clays can be recognized in the laboratory by their plastic and swelling properties. Generally, clays of high plasticity usually have high swelling potential(Mokhtari, 2012). Expansive soils are characterized by a plasticity index over 30%, liquid limit exceeding 50%, and have high swelling potential(Zumrawi et al., 2017). In the field, expansive clays can be recognized in the dry season by the deep cracks of roughly polygonal patterns(Das & Roy, 2014).

2.2 Mineralogy of clay Soil

Expansive soils have very high clay content that particle sizes are less than 2 microns (0.002 mm) in diameter. (Nigussie, 2011).Clay minerals are complex aluminum silicates composed of two basic units these are silica tetrahedron and alumina octahedron(Murthy, 2002). Each tetrahedron unit consists of four oxygen atoms surrounding a silicon atom Clay minerals are formed by sandwiching tetrahedral and octahedral layer sand sheets together(Teferra, 2023). A tetrahedron sheet can be considered as a layer of silicon ions between a layer of oxygen and a layer of hydroxyl ions(Das & Roy, 2014). This also is called a gibbsite sheet sometimes magnesium replaces the aluminum element in the octahedral sheets; in this case, the octahedral sheet is called a Brucite sheet(Murthy, 2002). Absorption of water by clays leads to expansion From the mineralogical standpoint, the magnitude of expansion depends on the kind and amount of clay minerals present, their exchangeable ions, electrolyte content of the aqueous phase, and the internal structure(Chen, 1975). There are three types of clay minerals. In order of increasing their expansiveness, they are listed as Kaolinite, Montmorillonite and Illite.

Kaolinite: consists of repeating layers of elemental silica-gibbsite sheets in a 1:1 lattice (Das & Roy, 2014).Kaolinite is stable and water is unable to penetrate between the layers. It shows little swelling during wetting.

Montmorillonite: - Montmorillonite is the clay mineral that presents most of the expansive soil problems. Its basic structure consists of an alumina sheet squeezed between two silica sheets. The basic Montmorillonite units are stacked with one on top of the other but the bond between individual units is relatively weak so water can easily penetrate between the sheets and separation of them produces swelling. It is extremely active and its activity decreases as the adsorbed cation exchanges in the order of Sodium, Lithium, Potassium, Calcium, Magnesium, and Hydroxyl(A. Tefera, 1999).

Illite: consists of a gibbsite sheet bonded to two silica sheets one at the top and another at the bottom. It is occasionally called clay mica. The illite layers are bonded by potassium ions. The negative charge to balance the potassium ions comes from the replacing of aluminum for some silicon in the tetrahedral sheets(Das & Roy, 2014).The illite units are stable and so the mineral swells less than montmorillonite(A. Tefera, 1999).

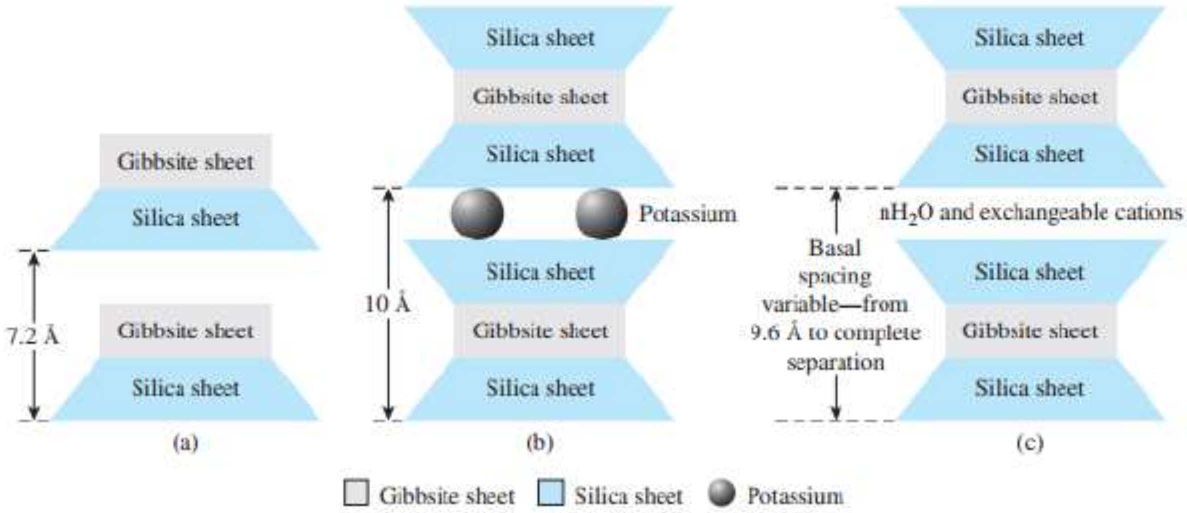

Figure 2.1: Structure of a) kaolinite, b) illite and c) montmorillonite (Das & Roy, 2014)

2.3 Problems Associated with Expansive Soil

Expansive soils have their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water particle and expand; and also as they dry they shrink, leaving large voids in the soil. Swelling of expansive soil can control the property of virtually any type of soil if the percentage of clay is more than about 5 percent by weight. Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties(Rogers et al., 1993).Due to their peculiar nature, expansive soils are a challenge for engineers everywhere in the world. In tropical countries due to wide variation in temperature moisture content of soils shows variation. The following problems generally occur in expansive soils.

High Compressibility: - expansive soils are highly plastic and compressible when they are saturated. Footing, resting on this type soils undergoes consolidation settlements of high magnitude. A stress increase caused by the construction of foundations or other loads compresses soil layers. The compression is caused by:-

(a) deformation of soil particles.

(b) Relocations of soil particles, and

(c) Expulsion of water or air from the void space

Swelling: - A structure built in a dry season, when the natural water content is low shows differential movement as a result of soils during the subsequent wet season. This causes structures on swelling soils to lift up and crack(Miliko N, 2019). A slight change in moisture content in the magnitude of 1 to 2% is sufficient to cause detrimental swelling. Soils with moisture content below 15% indicate danger because it easily absorbs moisture but moisture content above 30% indicates that most expansion has taken place and further expansion will be small. Due to their swelling property expansive soil damages the foundation by uplift when moisture increases. Swelling soils lift up the structure and causes crack for low-loaded, continuous strip footings, and frequently cause distress in floor slabs.

Shrinkage: - A structure built at the end of the wet season when the natural water content is high, shows settlement and shrinkage cracks during the dry seasons. In the dry season, the soil shrinks excessively and shrinkage crack as deep as two or three meters is a common occurrence. Swelling pressures can be get heaving, or lifting, of structures whilst shrinkage can cause differential settlement.

2.4 Damages Caused by Expansive Soil

In the dry state, the expansive soils exhibit a high bearing capacity which is gradually lost with an increase in moisture content. If prevented from swelling following exposure to moisture, the soils exert high swelling pressure. The pressure is usually build-up is usually responsible for cracking of buildings, distortion of pavement surfaces, and damage to other structures (A. Tefera, 1999). Damage due to expansive soils could occur on any type of building, road, and other civil engineering constructions that are not properly designed and/or constructed. However light structures like one or two-story buildings, warehouses retaining walls, and buried facilities are more vulnerable to damage because these structures couldn't exert a heavy pressure to counteract the uplift pressure from the expansive soils(Chen, 1975). Different researches have shown the damages caused by expansive soil in some parts of Ethiopia.

(Teklu.D, 2003) examined the swelling pressure of Addis Ababa area expansive soil. So numerous damages were reported as a result of expansive soil. Those damages do when the pressure applied by the soils is greater than the foundation pressure. Accordingly, assessing the swelling pressure is an important step in designing a foundation on expansive soil. (Afewerk, 2004) assessed the damage that occurred on buildings that are constructed on expansive soil areas by taking ninety-six randomly selected houses in the city of Addis Ababa. The houses are located in Bole, Olympia, Nifasilk, Lafto, Old Airport, Mekanisa, Gergi, and Bole bulbula localities of the city. The study showed that 64 % of the houses suffered heavy damage, 8 % of the houses were slightly damaged and 28% of the houses showed no damage He found that drainage is the most critical initiating factor for building damages. 84 % of the damaged buildings do not have a proper drainage system or the drainage system provided is adequate.

Figure 2.2: Typical damages due to soil swelling and shrinking characteristics show distress on lightweight structures (Tibebu, 2015)

Out of randomly visited 78 buildings 68 of them are damaged due to swelling and shrinkage cycle of the soil, which is 87%. In most cases, more than one building component is affected. The building component which is highly susceptible to damage due to expansive soil is the walkway around the building. The extent of damage decreases as we proceed to walls, floors, and foundations in their respective orders.

2.5 Identification of Expansive Soil

Soil deposits can be recognized in the field through visual examination. In the field, expansive clays can be recognized in the dry season by the deep cracks of roughly polygonal patterns(Das & Roy, 2014). The expansive soils show a polygonal pattern of surface cracks having 2.5 cm wide and over 1 m in depth is not uncommon in the dry season. The cracks close down after the rainy season. A shiny surface is also 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 samples of the soil are sticky and it will be relatively difficult to clean the soil with your hands. They are very hard when dry and they usually have a color of black or gray. One can also easily recognize expansive soil areas by simply looking at the failures of low loaded structure as shown in Figure 2.3. Swelling soils lift lightly loaded foundations and frequently cause distress in floor slabs, and vertical and horizontal cracks develop in the walls. Diagonal cracks that develop below the windows are indications of swelling movement(Kassahun, 2006).

The black and gray soils found in the eastern and southern parts of Addis Ababa are highly expansive and the percentage of montmorillonite present in the black and gray soils ranges from 45 to 87 percent(T. Tefera & Yohannes, 1986).

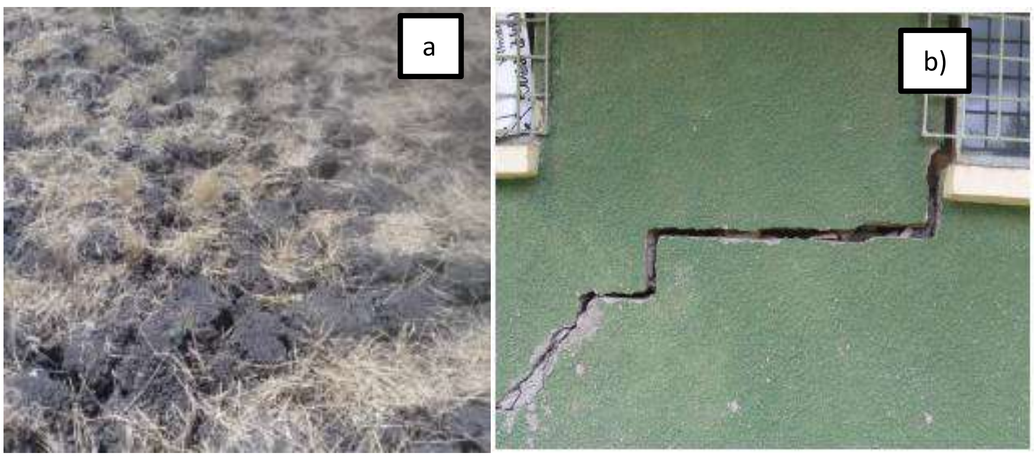

Figure 2.3: (a) Typical crack on expansive soil during the dry season(b) Diagonal cracks that develop below windows (Kssahuan, 2006)

Expansive soil can be identified by conducting several laboratory tests. These can be categorized as mineralogical identification and direct and indirect methods. Mineralogical identification can be useful in material evaluation, but is not sufficient by itself when dealing with natural soils. The various method of mineralogical identification is important in a research laboratory in exploring the basic properties of clays, but are impractical and uneconomical for practicing engineers(Chen, 1975). The five techniques which may be used are for mineralogical identification are differential thermal analysis-ray diffraction, dye adsorption, Chemical analysis and Electron microscope resolution. Due to their property of time consumption, expensiveness, and need for skilled technicians for result interpretation, these tests cannot be considered routine tests(Chen, 1975).

2.5.1 Indirect Methods for the identification of expansive soil

It includes simple soil property tests that a practicing engineer resorts to using for identifying expansive soil. Such tests are easy and can be performed in most soil mechanics laboratories, and yield excellent indices of expansive properties. The various tests under these methods are:

i) Atterberg limit tests

The Atterberg limit defines the moisture content boundaries between states of consistency of fine-grained soils. Here measurements of the plasticity index and liquid limit are commonly used to characterize soils and are useful indices for the identification of the swelling of expansive soils. The plasticity index alone can be used as a preliminary indicator of the swelling properties of most clay. Expansive soils are characterized by a plasticity index above 30%, a liquid limit above 50% and a high swelling (Chen, 1975). The relation between the degree of expansion and the plasticity index is given in Table 2.1.

Table 2.1: Relation between degree of expansion and Atterberg limits (A. Tefera, 1999)

NO.	Degree of expansion	Liquid limit (%)	Shrinkage limit (%)	Plastic index (%)
1	Very high	60-70	>30	>35
2	High	40-60	20-30	30-35
3	Medium	30-40	10-20	10-20
4	Low	20-30	<10	<10

Daksanamurthy and Raman (1973) also presented a single index method for identifying expansive soils using only liquid limits. They suggested four classes of clays according to their liquid limits as shown in Table 2.1 USBR Method is developed by Holtz and Gibbs; it is based on a direct correlation of observed volume change (degree of expansion) with colloid content, plastic index, and shrinkage limit.

ii. Linear shrinkage

Linear shrinkage is the change in length of a soil sample as it dries to the shrinkage limit, SL, expressed as a percentage of the original length. This test is also used in combination with other tests to classify expansive soil (Alemayehu Teferra, 1999). Swelling potential is assumed to be related to the opposite property of linear shrinkage, which is measured by a very simple test. In theory, it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential. (Chen, 1975).

Table 2.2: Potential expansiveness for various values of shrinkage limits and linear shrinkage

Shrinkage limit as a percentage	Linear shrinkage as a percentage	Degree of expansion
Less than 10	Greater than 8	Critical
10-1 2	5- 8	Marginal
Greater than 12	0-5	Non-critical

iii. Free swell

This test is a simple test conducted in the laboratory for the determination of the degree of expansiveness. Free swell tests consist of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of the initial volume, is the free swell value. The swell test is very crude and was used in the early days when refined testing methods were not available. Experiments conducted by (Holtz & Gibbs, 1956) indicated that a good grade of high swelling commercial bentonite will have a free swell value of from 1200 to 2000 percent. Holtz stated that soils with free swelling values as low as 100 percent can significantly damage lightly loaded structures, and soils with less than 50 percent free swelling values rarely experience significant volume change even under very light loading.(Chen, 1975).

$$\text{Free Swell (\%)} = \left(\frac{\text{final volume} - \text{initial volume}}{\text{initial volume}}\right) * 100 \qquad (2.1)$$

Table 2.3: Degree of expansiveness and free swell (Chen, 1975)

Degree of expansiveness	DFS (%)
Low	<20

Moderate	20-35
High	35-50
Very high	>50

iv. Free swell index

A free swell index is 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 and 10ml of each was placed separately in two 100ml graduated cylinders. One cylinder is filled with kerosene oil and the other is filled with distilled water. Both samples are mixed and left undisturbed for at least 24 hours, after which the final volumes of soil of the cylinders are recorded. The free swell index is calculated as:

$$\text{Free Swell (\%)} = \left(\frac{\text{final volume in water} - \text{final volume in kerosene}}{\text{final volume in kerosene}}\right) * 100 \qquad (2.2)$$

v. Activity index

Atterberg limits and clay contents have been combined in a single parameter called activity ratio sometimes called activity index (A) developed by Skempton (1953). It is Skepton's method of clay soil classification which is based on their activity. Montmorillonite clay (expansive clay) is defined as active. Based on Skepton: Colloidal Activity (A) = PI/C where C is the percent of clay fraction finer than 2µ and PI is the plasticity index.

Table 2.4: Skepton's Method of clay Soil Classification based on activity

Degree of activity	activity
In active clay	<0.75
Normal clay	0.75-1.25
Active clay	>1.25

2.5.2 Direct Methods

These methods offer the most useful data by direct measurement, and tests are simple to perform and do not require complicated equipment. Testing should be performed on several samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of a conventional one-dimensional consolidometer (Chen, 1975).

Table 2.5: Relationship between swelling potential and % of expansion in oedometer

Swell potential	% Expansion in oedometer (Holtz and Gibbs, 1956)	% Expansion in oedometer (seed et al.,1962)
Low	<10	0-1.5
Medium	10-20	1.5-5
High	20-30	5-25
Very high	>30	>25

2.6 Distribution of Expansive Soils in Ethiopia

The distribution of expansive soils is generally due to geological history, sedimentation, and local climatic conditions. The arid climatic conditions and severe weather conditions in northeastern Africa favor the spread of expansive soils. In Ethiopia, expansive soils covered nearly 40% surface area of the country.

Figure 2.4 Distributions of Expansive Soils in Ethiopia(Nigussie, 2011)

Expansive soils are observed in the area such as central Ethiopia, following the major trunk road like Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa – Debere Berehan, Addis

Ababa- Gohatsion, and Addis Ababa - Mojo. Also covers the area like Mekelle, Bahirdar, Gambela, Arba Minch, and the most Southern, South-west and southeast part of the capital Addis Ababa area in which the most major recent construction is being carried out. The black and grey soils found in the eastern and southern part of Addis Ababa are highly expansive(T. Tefera & Yohannes, 1986)

2.7 Reviews on Soil Stabilization

Soil stabilization is the process of alteration of one or more soil properties to create an improved soil material possessing the desired engineering properties(Mekonnen et al., 2020). Soil stabilization can be classified into two broad categories as mechanical and chemical 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(Demesew.E, 2021). 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(Sunder et al., 2017).Chemical stabilization depends on the chemical reaction between the stabilizer and soil minerals to achieve the desired effect. 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. Chemical stabilization can achieve improvement of soil through three chemical reactions(Asres.E, 2017).

2.8 Stabilization Process

Soil stabilization is the treatment of clay soils to improve their index properties and strength characteristics such that they permanently become suitable for construction and meet engineering design standards(Salahudeen & Ochepo, 2015). Cementitious materials stabilize the soil and change its properties through cation exchange, flocculation and agglomeration and pozzolanic reactions(Mekonnen et al., 2020).The strength, bearing capacity and durability of soils can be increased by addition of some chemical materials. The two commonly used methods of soil stabilization are stabilization by compaction or stabilization by chemical additives. 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(Patel, 2019). 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(Makusa, 2012).

2.8.1 Chemical Stabilization

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, sodium chloride, or visco-elastic material such as bitumen. The main chemical stabilizing agent for expansive clay is lime which may be calcium oxide (CaO) or calcium hydroxide Ca (OH)2.Laboratory tests show that lime reacts with medium, moderately fine, and fine-grained soils to reduce plasticity, improve workability, and increase strength (Little, 1995). The increase in strength is mainly due to chemical reactions between lime and soil particles. These chemical reactions occur in two stages, with immediate and long-term benefits (Gidday and Mittal, 2020).

The first phase of the chemical reaction involves immediate changes in soil structure and soil properties caused by cation exchange. When calcium comes in contact with the pore water, hydration occurs resulting in the formation of calcium hydroxide. Some of this calcium hydroxide is adsorbed onto the soil particles. Ion exchange occurs and the soil becomes drier and coarser due to the slaking process and flocculation of clay particles (Boardman, D.L., Glendinning, S., & Rogers, 2001). Free calcium from lime exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles(Alrubaye et al., 2016). This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, causing flocculation/agglomeration of the clay particles, which transforms the clay into a more silt-like or sand-like material(Baykal, 2021). In general, the flocculation and agglomeration phase of lime stabilization results in soil that is easier to mix, process and ultimately compact. According to (Christopher, 2005), almost all fine-grained soils experience this rapid cation exchange and flocculation/agglomeration reaction when treated with lime in the presence of water. The second stage of the chemical reaction involves pozzolanic reactions occurring in the lime-soil mixture, which lead to an increase in strength over time. When lime combines with clay soil, the PH of the mixture increases, and at 12.4, the silica and alumina from the clay become soluble and are released from the clay mineral(Okechukwu, 2022). The calcium hydroxide not consumed in the first process is free to react with the silica (S) and alumina (A) contained in mineral present in the soil. The reactions result into the formation of Calcium aluminate silicate hydroxide (CASH), Calcium Silicate hydroxide (CSH) and calcium aluminate hydroxide (CAH) which has cementitious properties, that strengthens gradually over several years(Quintus & Smith, 2004). As long as there is enough calcium from the lime to combine with the soluble silica and alumina, the pozzolanic reaction will continue as long as the pH is maintained high enough to maintain the solubility of the silica and alumina (Little, 1995). The increase in strength also depends largely on the amount of silica and alumina obtained from the clay; thus, lime stabilization was found to be more effective for montmorillonitic soils than for kaolinitic soils Lees, Abdelkader, and Hamdani, (1982). Due to limited silica in clay soil, there is always free calcium form lime that is not utilized. Therefore the use of bagasse ash wastes which is rich in silica enhances the pozzolanic reactions. (Barasa, 2016).

Cation Exchange

Cation exchange is the interchange between a cation on the surface of any negatively charged particle (clay particle) and the stabilizer. 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. The size of the hydrated ion becomes important for ions of the same valence.

A typical replaceability series is

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

An example of the cation exchange:

$Ca^{2+} + Na^+ - Clay \rightarrow Ca^{2+} - Clay + Na^+$

The thickness of the diffused double layer decreases as replacing the divalent ions (Ca^{2+}) from stabilizers with monovalent ions (Na^+) of clay. Negatively charged clay particles adsorb cations of specific type and amount. The replacement or exchange of cations depends on several factors, primarily the valence of the cation(Getu, 2018). Higher valance cations such as the calcium ion (Ca^{++}) easily replace cations of lower valance such as sodium ions (Na^+). For ions of the same valence, the size of the hydrated ion becomes important; the larger the ion, the greater the substitutional power. If other conditions are equal, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations(Masteika &

Čepinskis, 2015).The thickness of the diffused double layer decreases as replacing the divalent ions(Ca^{2+}) from stabilizers with monovalent ions (Na^{+}) of clay. Thus, swelling potential decreases(Baser, 2009).

Flocculation and Agglomeration

Flocculation and Agglomeration: Cation exchange results in a change in the electrical charge around the clay particles, therefore, an increase in the inter-particle attraction causes flocculation and agglomeration. This leads to a reduction of clay-sized particles and thus the surface area of the soil, which explains the plasticity and expansion of the soil (Asres.E, 2017)(Asres.E, 2017).

Cation exchange reaction result in the flocculation and agglomeration of the soil particles with consequent reduction in the amount of clay-size materials and hence the soil surface area, which inevitably accounts for the reduction in plasticity. Flocculation and agglomeration change the clay texture from that of a plastic, fine grained material to that of a granular soil(Syah, 2004).

Pozzolanic Reactions

Pozzolanic Reactions: Time-dependent pozzolanic reactions play a major role in the stabilization of the soil since they are responsible for the improvement of the various soil properties. Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).

The initially formed calcium silicate gel covers and binds the clay pieces together. The gel then crystallizes to form an interlocking structure which increases the soil strength (Aschalew, 2021).

Calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) are the two outputs in pozzolanic reactions.

$$Ca^{2+} + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} \rightarrow CSH \qquad (2.2)$$

$$Ca^{2+} + 2(OH)^{-} + Al_2O_3 \text{ (Clay Alumina)} \rightarrow CAH \qquad (2.3)$$

Pozzolans are a broad class of silica or silica-alumina-containing materials that have little or no cementitious value in themselves but are finely divided and presence of water; react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties. The ability of pozzolan to react with calcium hydroxide and water is quantitatively obtained by measuring its pozzolanic activity(Barasa, 2016).

2.9 Bagasse Ash

These days sustainability plays the major role in every aspect of human activities. Many technologies came to end because they were not in harmony with the idea of sustainable development(Mahendran et al., 2019). Sustainability is concerned about the world we will be leaving behind for future generations. It focuses on the social, environmental and economic subjects of human activity. Therefore it requires every activity to be environmental friendly, economical and safe for the social(Wubshet, 2013).

Bagasse ash contains large amounts of silica, which is the most important component of cement replacement materials. It is also found in large quantities as a by-product of sugar factories. Despite this abundance and silica content, relatively little has been done to examine the potential of this material for soil stabilization. Even though little, the conducted researches conform the suitability of this material for soil stabilization as an admixture with lime and cement. However, its suitability as a standalone material is still questionable(Wubshet, 2013).

Bagasse is the fibrous material that remains after sugarcane juice has been extracted. It is used as a fuel in the sugarcane industry that produces steam for electricity generation and the residue obtained after incineration is known as bagasse ash. The disposal of these sugarcane wastes has proven to be economical as well as environmentally friendly. Bagasse ash is predominantly composed of silica, alumina, and iron oxide which are why it participates in the pozzolanic reactions. In Ethiopia, where sugar production is currently about 300,000 tons, the potential for bagasse ash is about 72,000 tons per year. Hence, use of bagasse ash for upgrading of soil should be encouraged. Meanwhile, bagasse ash is classified as pozzolan, which contains about 1.78% calcium oxide (CaO), 5.78% iron oxide (Fe2O3), 1.23% magnesium oxide (MgO), 65.58% silicon oxide (SiO2) and 5.78% aluminum oxide (Al_2O_3)(Hailu, 2011).Bagasse ash is advantageous in multiple activities whether it be in combination with concrete or with soil(Melese, 2018).

2.10 Review of Previous Work

a) Bagasse ash as a Standalone Stabilizer

Melese, (2018) the soil was stabilized with bagasse ash in proportion of 5%, 10%, 15%, 20%, 25% and 30% of BA by dry weight of the soil. The results shows CBR values slightly increased with the addition of bagasse ash. The change in CBR value is insignificant for both cured & uncured samples. Addition of bagasse ash alone does not improve the strength of soils due to presence of only reactive silica with low amount of calcium content in bagasse ash. The addition of bagasse ash in combination with lime improved the CBR value. The improvement is more significant when the sample is cured. Hence, combination of bagasse ash and lime can strongly improve the strength of the expansive soil.

Kharade,(2014) studied the partial replacement of bagasse ash in black cotton soil at different proportions such as 3%, 6%, 9%, 12% of BA. They show that the optimum proportion of bagasse ash was 6%.

- ✓ The results at this proportion were like MDD increased by 5.8.Improvement in the maximum dry density was observed slowly for 3% and 6% replacement of bagasse ash, but then after further increase in blend MDD decreases. This decrease may be due to low specific gravity bagasse ash replaces higher specific gravity soil and it is fibrous in nature. It was observed that as % of bagasse ash increases the optimum moisture content decreases. The reduction of MDD is shows at figure 2.5

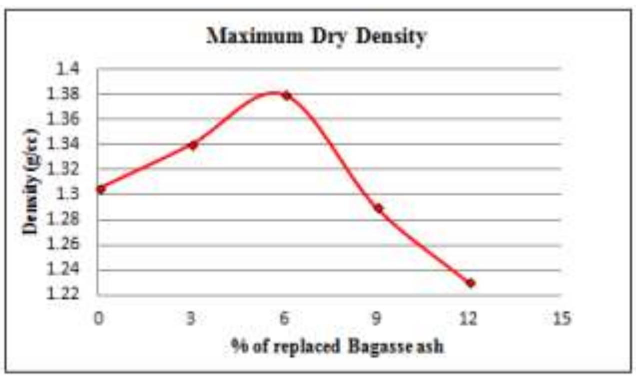

Figure 2.5: Maximum dry density variations with % replacement bagasse ash(Kharade,(2014)

Bethlehem Worku, (2015) the natural soil used in the work is an A-7-6 (10) soil according to AASHTO soil classification system or ML in the USCS. Studied on engineering properties of soil using bagasse ash (0%, 2%, 4%, 6%, 8% and 10% of BA) by weight of dry soil. The addition of bagasse ash resulted in the general decrease of free swell up to 6% followed by an increase at 8%. This again may be due to the high affinity of water of the organic material, which resulted in an increase of free swell when the ash content increased. This can be seen from the result of free swell of the ash alone. The bagasse ash had a free swell of about 50%, more than the natural soil. Therefore, increasing the swell of the natural soil as the percentage of the ash increased.

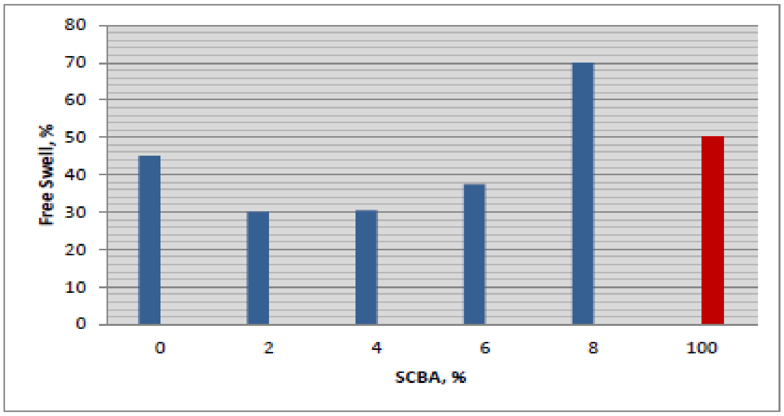

Figure 2.6:Variation of free swell with SCBA content(Bethlehem Worku, (2015)

As a result, the addition of the ash did not bring a significant change on the strength of the soil. Based on the results discussed above, sugarcane bagasse ash was not an effective stabilizer for improving the geotechnical properties of the lateritic soil sample.

(Gandhi.K, 2012)investigated on stabilization of expansive soil using bagasse ash. The experimental study involved Atterberg limit test, free swell index and swelling pressure tests.

- ✓ The results show that as the proportion of bagasse ash in the soil sample increases, the plasticity of the soil decreases as shown in the table 2.6

Table 2.6: Variations of Atterberg limits with addition of bagasse ash

Bagasse ash	LL (%)	PL (%)	PI (%)
0%	72	30	42
3%	67	29	38
5%	63	28	35
7%	58	26	32
10%	52	25	27

The research concludes that soil stabilization by applying waste product bagasse ash successfully improves the existing poor and expansive subgrade soil.

(Osinubi et al., 2009a)evaluated bagasse ash stabilization of lateritic soil. Figure 2.7 shows the effect of bagasse ash content on the UCS of lateritic soil specimens compacted at the BSL energy level and cured for 7, 14 and 28 days, respectively.

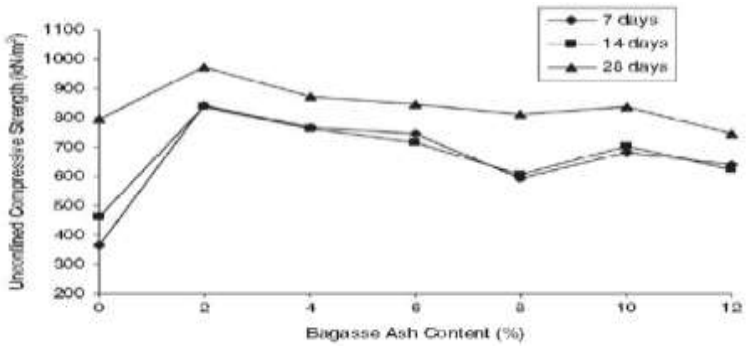

Figure 2.7: Variation of unconfined compressive strength with bagasse ash content(Osinubi et al., 2009)

The improvement in strength of bagasse ash-treated soil has been attributed to soil-bagasse ash reactions, which result in the formation of cementitious compounds that bind soil aggregates. Peak UCS values of 836, 842 and 973 kN/m2 were recorded at 2% bagasse ash content for 7, 14 and 28 days curing periods, respectively. As expected strength increased with age but decreased with higher bagasse content. Generally the research concluded that bagasse cannot be used as a standalone stabilizer but should be employed in admixture stabilization.

b) Bagasse Ash as an Admixture with Lime

(Wubshet & Tadesse, 2014) the soil was stabilized with bagasse ash in proportion of 5%, 10%, 15%, 20%, 25% and 30% of BA by dry weight of the soil. The preliminary investigations of the soil showed that it belongs to A-7-5 class in the soil classification system of AASHTO. When treating the expansive soil at 30% bagasse ash content the uncured soil bagasse sample gives a peak CBR value of 2.91% while that of 7 days cured CBR value is 2.94% from a value of 0.91% for the natural soil. It was found out that bagasse ash stabilized soil do not meet the minimum requirement of ERA pavement manual specification so that it could be used as a sub-grade material for pavement construction. Additional study is also incorporated as a supplementary work to investigate the effect of applying 3% lime as an activator in combination with 15% bagasse on the geotechnical properties of the soil for uncured and cured soil samples.

(Sabat, 2012)studied the use of bagasse ash and lime mud in the construction of flexible pavements in expansive soil areas.

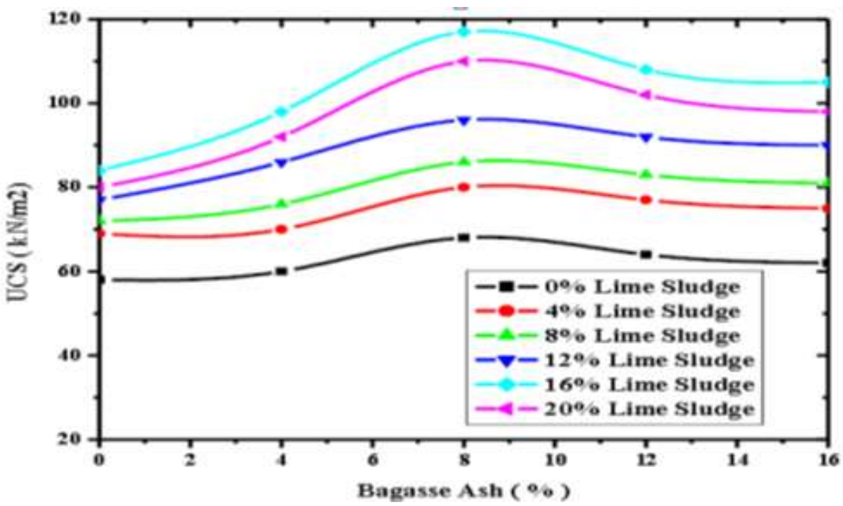

Figure 2.8: Variation of UCS with percentage of bagasse ash and lime sludge(Sabat, 2012)

- ✓ As the percentage bagasse ash increase, UCS increase to peak value at about 8% uniformly decrease with addition of lime sludge, in a similar manner bagasse ash increases the aforementioned soil strength parameters where the peak is at 16%. Hence, the optimum proportion of the soil: bagasse ash: lime sludge is found to be 76:8:16.

Overall, the research concluded that industrial waste such as bagasse ash and lime sludge can be used to reinforce the subgrade soil, with significant savings in construction costs.

Ochepo, J. and Osinubi, K. J., (2013) studied the effect of compaction strength and wear time on the strength of lime-bagasse-ash stabilized expansive clay from Gombe, Nigeria. The experimental study involved unconfined compressive strength. The following summarize are drawn from the study:

- ✓ The obtained results show that UCS values increase with lime and bagasse ash on proving and curing time.

c) Bagasse Ash as an Admixture with cement

(Hailu, 2011) studied on engineering properties of soil using bagasse ash (0%, 5%, 10%,15%,20% and 25% of BA) by weight of dry soil. The workability of mortar and concrete containing BA decreases slightly as the BA content increases, due to the higher water requirement of BA bagasse ash resulted in better normal consistency and longer setting.

- ✓ The results of the concrete work revealed that, the unit weight of the concretes containing bagasse ash have shown a slight reduction.
- ✓ It was found that a reduction of unit weight up to 1.65% was observed when 25% of the cement was replaced by bagasse ash. The flexural strength of concrete reduced as the BA replacement percentage increased. At 28 days the control concrete and the concrete containing 5% bagasse ash have shown the same flexural strength whereas the 15% and 25% replacements have shown a lower flexural strength value.
- ✓ Finally, the results of this study showed that cement can be cost-effectively replaced with up to 10% of bagasse ash. As a result of this exchange, concrete properties similar to the control concrete are obtained. Higher replacement percentages can also be used with a slight reduction in concrete efficiency.

2.11 Availability of Bagasse ash in Ethiopia

In order to assess the potential of bagasse ash production in Ethiopia, it is imperative to evaluate the sugarcane crop yield in the country. There are three state owned sugar factories functioning in the country in 2013. Their annual production capacity is about 300,000 tons, the sugarcane covering about 10,000 hectares of land. This annual production is not sufficient to the local sugar demand forcing the government to annually import 1.5 million quintals from abroad. To avoid this shortage of sugar in the country the government plans to establish eight new sugar factories in the coming five years with a total estimated capacity of 2.250,000,000 tons at the start of production according to the strategic plan and covers about 225,000 hectares(Wubshet, 2013).

In addition, the government is implementing expansion projects in existing factories to increase their production capacity. At the end of this expansion projects on Fincha, Methara and Wonji-Shoa sugar factories the additional total production capacity is expected to be around 365,000 tons of sugar annually. Specifically, Fincha, located in the western part of the country, planned to increase its production up to 270,000 tons; Wonji-Shoa discovered 125 km east of Addis Ababa, plans to increase production to 350,000 tons; The Methara sugar factory was found 200 kilometers east of Addis Ababa and is also expected to increase its annual production to 190,000 tons, according to Sugar Development study paper. Tendaho sugar factory is expected to have an annual production capacity of 600,000 tons is expected to be completed at the end of 2014(Wubshet, 2013).

As can be seen from the above discussion the sugar production in the country is boosting at a high rate, even planning to hold 2.5% of the world sugar in the coming years according to the strategic plan. Such intensification of sugar production also leads to abundant sugar bagasse and sugar ash. Table 2.8 summarizes the expected future sugar production of the country and the respective bagasse ash potential.

Table 2.7 : Estimated bagasse ash potential of Ethiopia(Hailu, 2011)

Factory	Expected future production of sugar(tons/year)	Estimated Bagasse Ash(tons/year	Estimated Bagasse Ash(tons/year)
Wonji-shoa	350,000	1,050,000	84,000
Metehara	190,000	570,000	45,600
Fincha`a	270,000	810,000	648,000
Tendaho	600,000	1,800,000	144,000
New	2,500,000	7,500,000	600,000
Total	3,910,000	11,730,000	938,000

The above estimation is based on the targeted annual future sugar production in the country. Sugarcane consist about 30% bagasse, whereas sugar recovered is about 10% of the sugarcane, the bagasse leaves about 8-10% bagasse ash as a waste. As can be seen from Table 3.5, during the implementation of the five-year strategic plan, approximately 0.94 million tons of BA will be generated annually. Currently, where sugar production is approximately 300,000 tons per year, the potential of bagasse ash is approximately 72,000 tons per year(Wubshet, 2013).

Table 2.8: Summary of related works on bagasse ash and lime stabilization

Authors	Type of Additive	% of Additive	Type Soil	Major Finding
Bekele (2022)	Bagasse	0%,6%,9%,12%	Expansive	Plasticity property is reduced and UCS value is increased up to 9%

	Ash	of BA	Soil	addition of BA. But the UCS is observed only for uncured day
Worku(2015)	Bagasse Ash	2%,4%,6%,8%,16% of BA	Lateritic Soil	Slight increase in plasticity index, shrinkage limit, OMC, and unsoaked CBR values
Arpitha I.V.Gowda(2019)	Bagasse Ash	3%,6%,9% and 12% of BA	Expansive Soil	UCS is shows increased and Plastic liquid is decreased as % of BA increase. But the UCS is observed only for uncured day.
Daleep Kumar Gautam(2019)	Bagasse Ash	5%,10%,15%,20%,25% and 30% of BA	Expansive Soil	OMC shows increase and Dry density decreased with increased BA and also CBR is increased up to 20%
Surjarndri et at.(2017)	Bagasse Ash	5%,10%,15%, and 30% of BA	Expansive Soil	The plasticity index decreased MDD increased. The percentage of swelling decreased from 5.48 to 3.29% at the optimum of 15% bagasse ash.
Kharade et a1.(2014)	Bagasse Ash	3%,6%,9%, and 12% of BA	Black Cotton	MDD increased by 5.8%, CBR increased by 41.52% and UCS increased by 43.58% by the addition of bagasse ash
Wubshet and Tadese (2014)	Bagasse Ash with Lime	15%BA,3%Lime and 3% lime + 15%BA	Expansive Soil	Increase in (OMC) and CBR value; and a decrease in (MDD) and plasticity of the soil for all additives increase.
Barasa,(2016).	Bagasse Ash with Lime	4%,5%&6% of lime and 1:4,3:2,2:3&4:1 BA	Expansive Soil	The result shows PI of clay soil decreased with increase % of lime, ash and ratio lime to ash in all the samples .CBR was increased when % of lime is increase
Liet Chi Dang (2015)	Bagasse Ash with Lime	6%,10%,18%,25%,4.5%,7.5%,13.5%&18.25% BA and 1.5%,2.5%,4.5% & 6.25% of lime	Expansive Soil	UCS test is cured for 3, 7 and 28 day of curing. The finding is shows the increase of strength is considerable curing time but, after 28 day is not studied. Using lime and BA ash is not stabilizing expansive soil but also use of waste materials is to treat environments
Dinke(2015)	Bagasse Ash and	5%,10%,15% and 20% BA	Expansive Soil	CBR and UCS of soil increase at the optimum of 88% soil plus 10%BA

	Cement	with 2%,4%,6% and cement		and 2% Cement. But the UCS is observed only for uncured day

2.12 Research Gaps

From the review of literature of related works, research gaps were identified for further study to improve the results and create a better understanding of the application of bagasse ash and lime. Here are the research gaps identified. It has been observed that the bagasse ash and lime mixture with expansive soil have been tested for their engineering characteristics where they curing day is conducted up to 28 day only. Even though bagasse ash and lime is a chemical admixture these where no studies conducted on the maturation period after day 28 aspect of the behaviour of these mixtures i.e. the change in strength behaviour with time. Therefore, it is necessary to investigate the change in strength behaviour of materials after 28 day of curing. Investigation of soil stabilization by two materials was not considered the effect of decreasing content of expansive soil at second stage when the second admixture is added. Therefore it is necessary to study without changing the content of expansive soil in first stage as it's for understanding effective of admixture materials on engineering property of expansive soil.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Description of the study area

Gelan is located in Finfinnee special zone or by new name known as sheger city, Oromia region, Ethiopia with a total planning area of 7,516.8 hectares. It is located at 25 km in southeast direction on the Addis Ababa -Adama express way .The border between the cities of Gelan and Dukem in the southeast is made by an artificial border formed at the intersection of the Ethio-Djibouti railway and the Addis Ababa-Adama road. In the absolute term, its geographical coordinates are 8° 52' 0'' N and 37° 47' 0'' E longitude and its located in 2051-2373km above sea level.it is average annual rainfall is about 920mm and annual average temperature is 20°C.Type of area is populated place a city, town, village, or other agglomeration of buildings where people live and work.

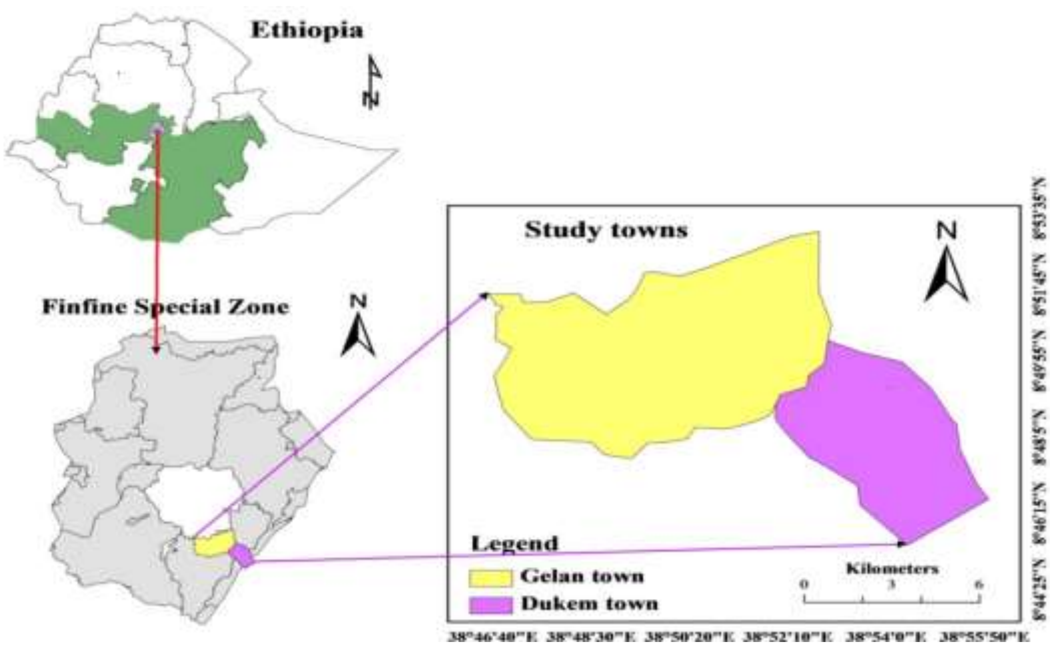

Figure 91: Map of the study area

The population is also rapidly growing because of Gelan town is nearness to Addis Ababa and economic importance. As a result of rock weathering and subsequent erosion and deposition, thick residual clay and silty clay cover most of the flat topographic landforms. The profile of urban soils can be divided into expansive and non-expansive soils. The Expansive soil covers a wider area of the town. Expansive soil is clay soil which may be black or gray in color approximately 80% of the town is covered by black clay soil. 20% of the area is covered by mountains which mainly consist of cinder and gravel. The remaining part of the town is used for residential and industrial areas. In this town, most lightweight residential houses show uplifting and cracking. There was also a crack shown on the ground surface around the study area.

3.2 Materials

For investigating this research three materials were used for this study was expansive soil from two test pits and the materials involved in the stabilization process which are bagasse ash and lime.

3.2.1 Soil

The soil samples used in this investigation were collected from Gelan town by excavating from two pits. This study is considered applicable for light structure which is used isolated footing and stand at 1.5m.So sample is collected at 1.5m from the natural ground surface for each location to avoid the inclusion of organic matter.

Figure 3.10 Geographical location of the test pits

After pit excavations, the soil samples were transported in plastic bags to the Adama science and technology university geotechnical laboratory. Laboratory tests such as natural moisture content, specific gravity, Atterberg limits, wet sieve, hydrometer analysis, compaction and UCS tests were performed on disturbed soil samples to characterize the properties of soil in the study area and to select the weaker soil for stabilization work.

3.2.2 Lime

The lime used in this study was hydrated calcium hydroxide, commonly known as slaked lime or road lime. It was purchased from sankale lime factory Addis Ababa district. Hydrated lime was preferred for its suitability as it is relatively less reactive compared to quicklime. This makes it easier to use compared to highly reactive quicklime.

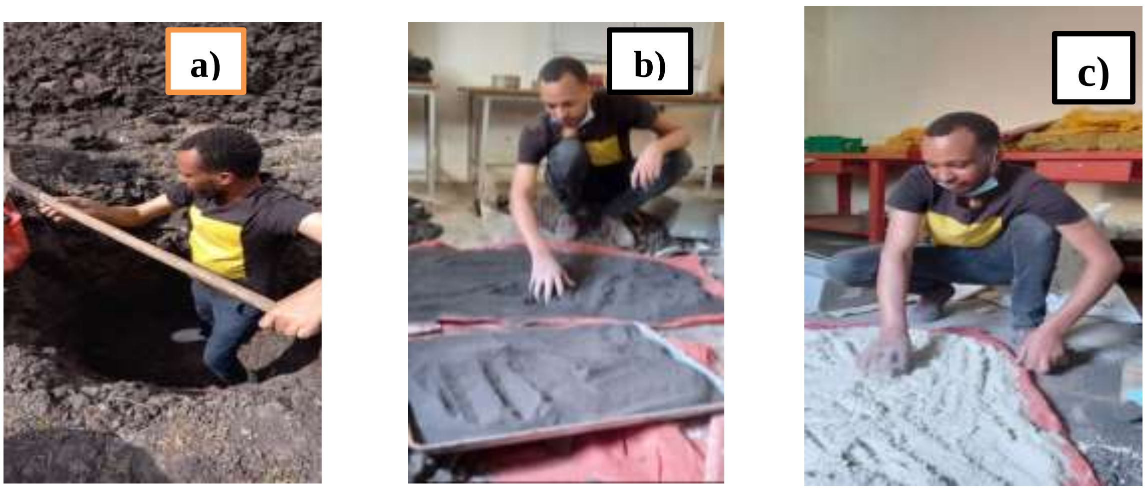

Figure 3.11 photograph of a) test pits, b) Bagasse ash and c) lime

3.2.3 Bagasse Ash

Bagasse is the fibrous residue obtained from sugarcane after the extraction of juice at sugar mill factories and previously was burnt as a means of solid waste disposal. However, as the cost of fuel oil, natural gas and electricity has increased, bagasse has become to be regarded as a fuel rather than refuse in the sugar mills. The fibrous residue used for this purpose leaves behind about 8-10% of bagasse ash, (Hailu, 2011). Bagasse ash was collected from wonji sugar factory.

Table 3.1: Chemical composition of bagasse ash (Hailu, 2011)

chemical composition	sio2	Al2o3	Fe2o3	CaO	MgO	Na2O	K2O	MnO	Ti2O	P2O5	H2O	SiO3	LOI
%	65.58	5.87	4.32	1.78	1.23	1.02	6.41	0.05	0.25	1.35	0.2	0.18	10.5

3.3 Methods

To achieve the objective of the thesis reviewing the literature, academic journals, research articles, and any previous works that are related to the study topic have done before conducting the tests. The soil samples were collected from two test pits.

Figure 3.12The process of method to cure and testing UCS test

Then a series of laboratory tests such as natural moisture content, grain size analysis, specific gravity, Atterberg limits, standard compaction, and unconfined compressive strength (UCS) test was conducted. Based on the test results the weakest soil was selected for stabilization work. Then the soil was mixed with bagasse ash in percentages ranging from 5% ,10%,15% & 20% of the dry weight of soil and the samples were subjected to Atterberg limit, specific gravity, standard compaction, and unconfined compressive strength (UCS) tests and were mineralogical characterization (XRD) is conducted. The optimum content of bagasse ash was chosen based on the UCS values. The bagasse ash stabilized soil was combined with different ratio 3:1 and 2:2 by dry mass of soil). For expansive soil treated with bagasse ash and lime compaction and UCS, tests were conducted. Then the optimum proportion of bagasse ash and lime was chosen based on UCS values.

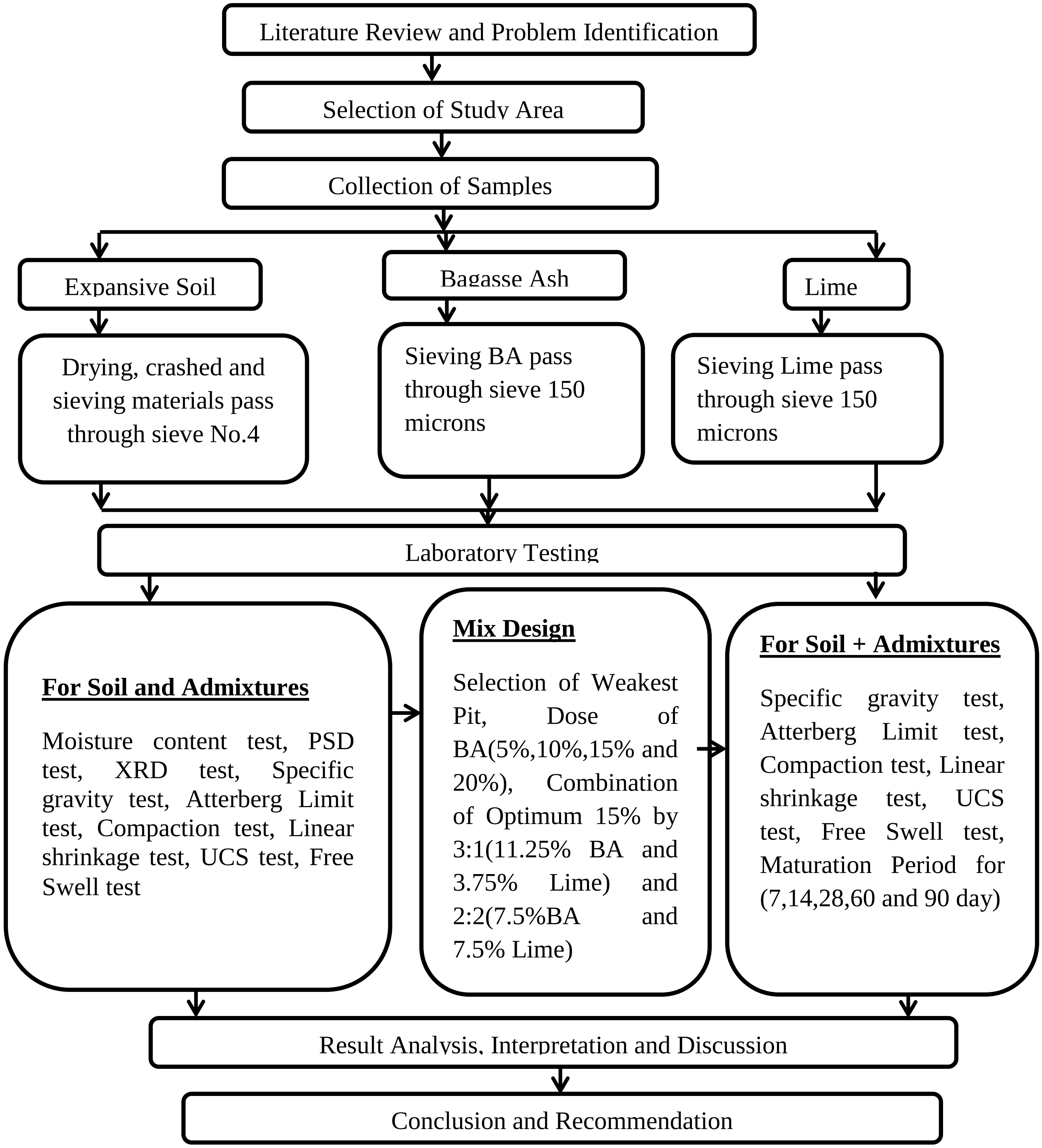

Figure 13: Summarize methodology chart of the study

3.3.1 Sample preparation

The sample preparation was made following the method described in AASHTO T87-86. The soil samples taken from two test pits are dried and the larger particles of the soils are crushed and then properly prepared for the laboratory tests. The laboratory tests were done either by air drying of samples or oven drying. Bagasse ash was sieved 150 μm and mixed with the soil by varying the percentages of 5 %, 10%, 15%, and 20% then conducting laboratory tests. The lime was taken at 3:1and 2:2 ratio by dry weight of the soil.

Table 3.2: Mix design and material proportion

NO.	Phase 1	Phase 2
1	100% expansive soil + water	3:1 of BA & Lime (85% soil+11.25% BA+3.75 Lime)
2	95% expansive soil + 5% BA + water	
3	90% expansive soil + 10% BA + water	2:2 of BA & Lime (85% soil+ 7.5% BA + 7.5% Lime).
4	85% expansive soil + 15% BA + water	
5	80% expansive soil + 20% BA + water	

3.3.2 Natural moisture content

This test was performed to determine the moisture content of soil samples. Water content is the percentage of "weight of pore" or "free" water by weight of dry soil solids in a given soil mass. Equipment used for this test was Drying oven, Balance, Moisture can, Gloves and Spatula. This test was conducted following ASTM D2216 standard test methods for laboratory determination of water (moisture) content of the soil.

3.3.3 Specific gravity

The specific gravity is defined as the ratio of the weight of a given volume of the material to the weight of an equal volume of distilled water. The specific gravity of the soil sample was determined using ASTM D 854 procedures. It is used to calculate parameters such as void ratio, porosity, particle size distribution using a hydrometer, and degree of saturation. The test was conducted for pure soil, bagasse ash, bagasse ash –soil mix, and lime.

3.3.4 Particle size distribution

This test is done to determine the percentage of different grain sizes present in the soil. Wet sieve and hydrometer analysis was conducted in this study according to ASTM D422-63. The wet sieve analysis was used to determine the distribution of the course, larger-sized particles, and the hydrometer method is used to determine the distribution of finer soil particles which were passed sieve size 0.075mm (N0.200) to measure the amount of silt and clay size particles. A hydrometer reading was conducted by using hydrometer 151H.

3.3.5 Atterberg limit

Atterberg limit tests are conducted on fine-grained soils to determine the amount of water required to attain different behavioral states. For AASHTO and USCS, soil classification systems value of Atterberg tests is required. The Atterberg limit test that was done in this study includes the determination of the liquid limits, plastic limits, and the plasticity index for the natural soil and soil-bagasse ash and BA-lime and expansive soil mixtures. The tests were conducted following ASTM-D4318. Air-dried samples passing the No.40 (0.425mm) sieve was used to conduct the test.

Liquid limit (LL): The liquid limit is described as the moisture content at the boundary between the liquid and plastic states. To accomplish the test first, the soil or soil-bagasse ash sample was thoroughly mixed with water by repeatedly stirring, kneading, and chopping with the spatula and placed in the Casagrande cup. The cup containing the sample was lifted and dropped at a rate of 2 drops/sec until the two sides of the sample come in contact at the bottom. The number of blows required to close the grove to this distance was recorded and its moisture content was determined. The relationship between the water content and the number blows is drawn. The water content corresponding to the intersection of the line with the 25-blow abscissa is the liquid limit of the soil.

Plastic limit (PL): Plastic limit of soil is defined as the water content at which the soil changes from a plastic state to a semi-solid state. A portion of the natural soil or the soil-bagasse ash mixture used for the liquid limit test was taken for the determination of the plastic limit. Then the soil or soil-bagasse ash sample was rolled into a diameter of 3mm thread until it begins to crumble.

The plasticity index (PI) is the difference between the liquid limit and the plastic limit of soil, the plasticity index is important in classifying fine-grained soils. This is fundamental to the Casagrande plasticity chart, which is currently the basis of the Unified Soil Classification System.

3.3.6 Compaction

Compaction is the densification of soil by the removal of air, which requires mechanical energy. The degree of soil compaction is measured by its dry weight. If water is added to the soil during compaction, it acts as a softening agent in the soil particles. Soil particles slide over each other and move into a tightly packed position. The dry unit weight after compaction first increases as the moisture content increases beyond a certain moisture content any increase in the moisture content tends to reduce the dry unit weight. This phenomenon occurs because the water takes up the spaces that would have been occupied by the solid particles. The moisture content at which the maximum dry unit weight is reached is generally called the optimum moisture content. In this study, compaction was done by the standard proctor test according to ASTM D698. Air-dried soil passing no 4 sieve was used to determine the moisture-density relation of the soil mix. Compaction was done for three pits untreated soil samples and soil treated with bagasse with different percentages (i.e. 5%, 10%, 15%, and 20% by dry weight of soil).

It was also done for lime sample with the optimum of the soil-bagasse ash content by different ratio of 3:1 and 2:2 by dry weight of soil. Moisture content versus dry density curve was plotted. The optimum moisture content (OMC) and maximum dry density (MDD) were obtained from the graph.

Figure 3.14 (a) compacting each layer of soil, BA, and L; (b) Compacted spaceman recording weight

3.3.7 Unconfined compressive strength (UCS) test

The UCS test is a special unconsolidated-undrained test commonly applicable for clay samples. In this test, an axial load is rapidly applied to the specimen, causing failure. The unconfined compressive strength UCS value of compacted soil is the most common and adaptable method of evaluating the strength of stabilized soil. In this study, the UCS test was conducted according to the ASTM D-2166 standard. UCS test includes soil, soil with varying percentages of bagasse ash (5%, 10%, 15%, and 20%) and soil - bagasse ash and lime mix by 3:1(11.25% of BA and 3.75% of lime) and 2:2 (7.5% of BA and 7.5% of lime). To conduct the test, the sample was mixed with its optimum moisture content and dry density and then compacted in a sample tube having 38mm diameter and 76 mm height. Then the sample was removed from the sample tube using a soil extruder and trimmed to make the ends smooth.

In this study for investigate the effects of maturation period on geotechnical properties of BA and lime stabilized expansive soil the UCS test was conducted for uncured and 7,14,28,60 and 90 days curing. Curing was done by putting the samples in the desiccator. The sample was placed in a loading frame and an axial load was applied to the sample. The force applied to the sample and deformation was recorded. Finally, the stress versus percent strain graph was plotted. Unconfined compression strength (UCS) was taken as the maximum load attained per unit area during loading. The undrained shear strength was calculated as half of the compressive stress at failure.

Figure 3.15 Reading the result of UCS test

3.3.8. X-ray diffraction

XRD is a technique used to know about the various component of minerals found in materials used for this study. For XRD testing, the fine powder of weakest soil, BA and lime was sieved through a sieve size of 75 μm and the portion of the powder passing this sieve was used for X-ray diffraction testing. X-ray diffraction pattern was recorded with X-ray diffractometer with CuKα radiation (λ= 1.5406Å) at diffraction angle 2θ ranged between 10° to 80° in steps of 2θ=0.02°. Finally, different phases of each material were obtained and identified by analyzing the peaks of XRD results patterns with the help of "X-Pert High Score" software tool and inter planar spacing (d) was calculated by using Bragg's equation shown below.

$$d=\frac{n\lambda}{2\sin\theta} \qquad (3.1)$$

Where: n is Bragg's constant equal to 1, λ is wave length =1.5406Å, θ is Bragg's angle in radian and d is interplanar spacing in Å.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1. Overview

The laboratory test results that are used to identify the effect of bagasse ash and lime on the properties of the given expansive soil are discussed in this chapter. Laboratory experiments such as specific gravity, Atterberg limit, particle size analysis, compaction, free swell index, were done on natural soil collected from all two test pits to classify and select the weakest soil for further studies. Therefore, based on their UCS value, the soil sample collected from Pit-2 was selected as the weakest soil needed to be studied for the effect of mixing with BA and lime. Then after pit 2 is selected, nature soil is mixed with BA with the proportion of 5%,10%,15% and 20% to determine the optimum value of BA mixed with lime by ratio of 3:1(11.25% of BA and 3.75% of lime) and 2:2(7.5% of BA and 7.5% of lime) to study their combined effect on natural soil's . Then finally, effect of their maturation period on geotechnical properties of soil was studied by using UCS test, which is cured for day 7,14,28,60 and 90.

4.1.1 Moisture content of natural soil

Moisture content is the ratio expressed as a percent of the mass of pore or free water in a given mass of material to the mass of the solid soil. A standard temperature of 110°+ 5 °C is used to determine mass of dry solid soil (ASTM D 2216, 1998). Natural moisture content determination of soil sample was used to know existing condition of material and depth of groundwater table at the sampling sites. Based on laboratory results, natural moisture contents are 25.26% and 26.76% for Pit and Pit-2 respectively. The result shows sample from Pit-2 is the wettest soil due to there may be some moisture infiltration or shallow groundwater table depth.

4.1.2. Specific gravity of natural soil and additives

Table 4.1: Specific gravity of soil samples and additives

Materials name	Specific gravity, Gs
Pit1	2.67
Pit2	2.65
BA	2.20
lime	2.35

The specific gravity of material used for the study was summarized as in table 4.1 which show natural soil sample from Pit1 and Pit2 has Gs of 2.67 and 2.65 respectively. BA and lime powder also have specific gravity of 2.20 and 2.35 respectively. The result shows lime is a heavyweight material than BA. Specific gravity determination of materials for this study follows (ASTM D 854, 2002)standard method.

4.1.3. Particle size distribution analysis of natural soil

Table 4.2: Summary of particle size analysis for soil

Sample	Particle size(mm)			
	Gravel (%) >=4.75	Sand (%) 0.075-4.75	Silt (%) 0.002-0.075	Clay (%) <=0.002
Pit1	0.4	9.2	50.4	40
Pit2	0	6.6	57.4	36

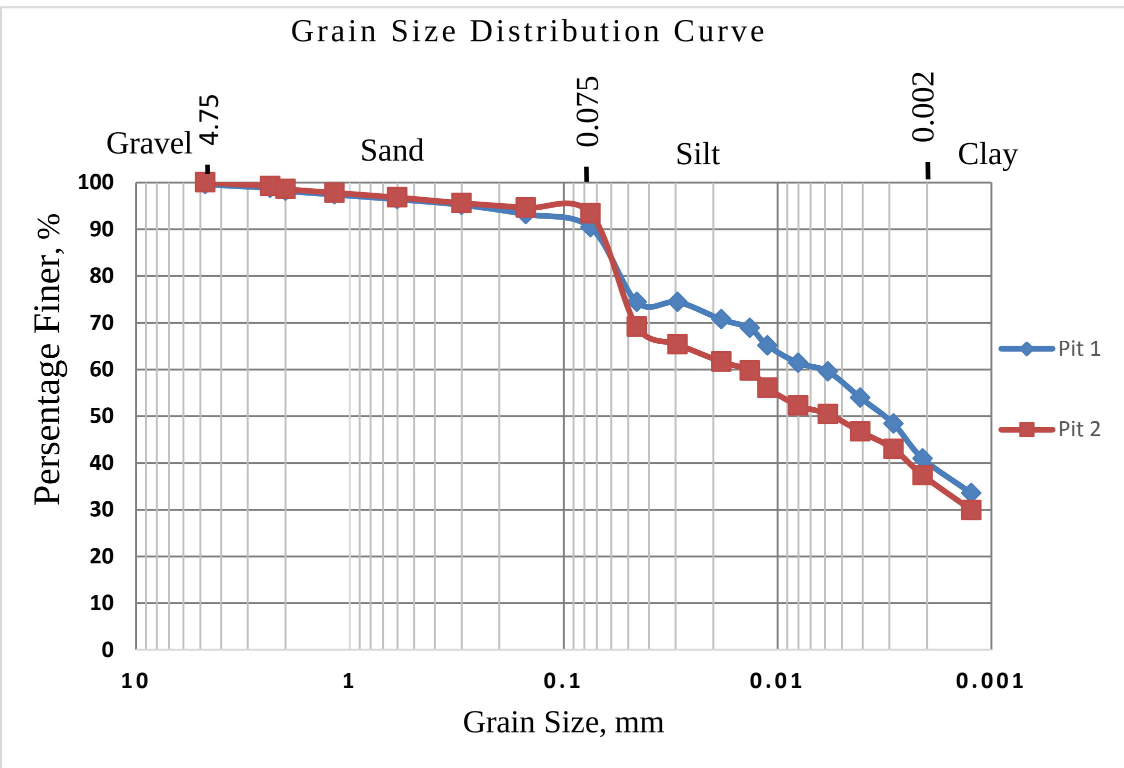

Figure 16: Grain size distribution curve for Pit1 and Pit2

Particle size analysis is the quantitative determination of the distribution of particle size in soils which is performed under the control of (ASTM D 6913, 2004) for coarse-grained soils and ASTM D 7928, (2021) fine-grained soils. From table 4.2, for soil collected from Pit1 and Pit2 there were 90.4% and 93.4% of particles that are passes through sieve size 0.075 mm respectively, which are classified as the fine-grained materials. Referring plasticity chart under USCS, both pit have liquid limit greater than 50% and Pit2 is falls above A-line which categorized as CH and Pit1 is falls below A-line which categorized as MH, inorganic fat clay having high plasticity group.

4.1.4. Atterberg limit of natural soil

Table 4.3: Atterberg limit test results of natural soils

Atterberg limit (%)				
Sample	LL	PL	PI	SL
Pit1	71.5	36.9	34.6	19.66
Pit2	82	36.1	45.9	22.81

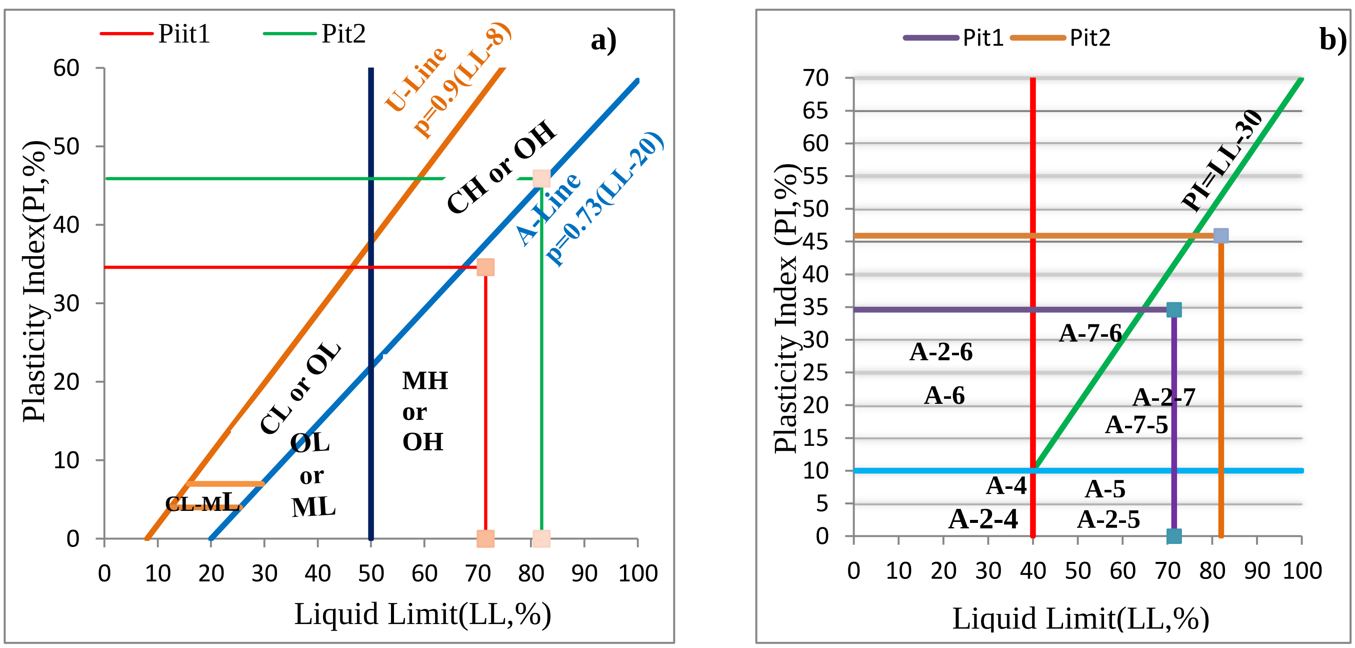

Figure 17: Plasticity chart for natural soil classification system a) USCS and b) AASHTO system

Atterberg limit test results include liquid limit, plastic limit, plastic index result under which all are followed by (ASTM D - 4318, 2020) and ASTM D4943 was used to determine linear shrinkage property of soil. From test results liquid limit, plastic limit, plasticity index and linear shrinkage of soil sample from Pit-1 and Pit-2 are (71.5%, 36.9%, 34.6%, 19.66%) and (82%, 36.1%, 45.9%, 22.81%) respectively. Plastic limit and plasticity index result obtained are mainly used in soil classification system and shows soil grouped as CH & MH for pit2 and pit1 respectively and A-7-5 as per USCS and AASHTO system respectively as shown on figure 4.2. The size of linear shrinkage is used to characterize soil and interpret cracking size & characteristics occurred due to seasonal moisture variation.

4.1.5. Standard compaction test result of natural soil

Table 4.4: Standard compaction test result of natural soil

Sample	MDD(g/cm^3)	OMC (%)
Pit1	1.306	26.3
Pit2	1.2321	29.6

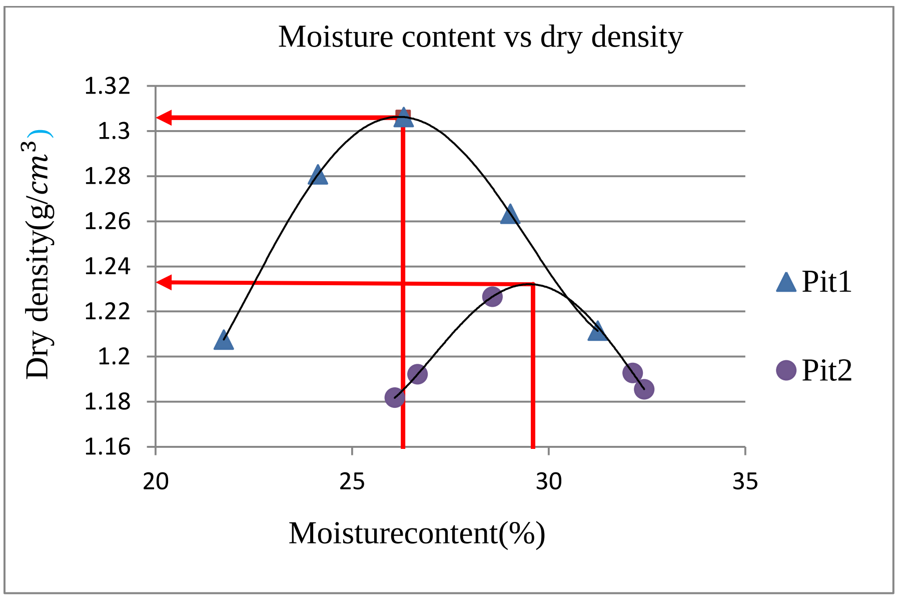

Figure 18: Moisture-density relationships for Pit1 and Pit2

Compaction of soils is a procedure in which soil sustains under mechanical stress is densified by reducing the volume of void in the mass. Soil mass consists of solid particles and voids filled with water or/and air. When soil is subjected to stress, soil particles are redistributed within the soil mass and the void volume decreases, resulting in densification. The test follows ASTM D 698, (2000) standard procedures and used to increase strength, decrease compressibility and permeability of soils. The result of compaction test of natural soil was summarized in the table 4.4 as shown above. Results obtained reveals that natural soil collected from Pit1 and Pit2 have optimum moisture content of 26.3% & 29.6% and maximum dry density of 1.306/cm3 & 1.2321g/cm3 respectively

4.1.5. Free swell index of natural soil

Table 4.5: FSI and FSR result of natural soils

Sample	FSI (%)	FSR (%)
pit1	91	1.91
pit2	110	2.1

Free swell is the increase in volume of a soil, without any external constraints, due to submergence in water and used to identify the potential of a soil to swell which might need further detailed investigation regarding swelling and swelling pressures under different field conditions. The test was performed under control of IS 2720 part 40, (2002) standard method. Table 4-5: FSI and FSR result of natural soils .The soil samples from Pit1 and Pit2 have FSI of 91%, and 110% and the FSR of 1.91and 2.1 respectively which shows material from pit2 have more swelling potential when compared with Pit1.

4.1.7. Unconfined compressive strength result of natural soil

Table 4.6: Unconfined compressive strength result of natural soil

Sample	UCS(kPa)
Pit1	130.56
Pit2	110.18

Samples of soil collected from two test pits were also tested for their strength through unconfined compressive strength test following (ASTM D2166−00, 2000) standard method. Based on this, material collected from Pit1 and Pit2 have compressive strength of 130.56Kpa & 110.18Kpa respectively. In addition to swelling property, compressive strength result shows Pit2 material has less UCS value and selected as the weakest sample needed for further study by mixing with respective additives.

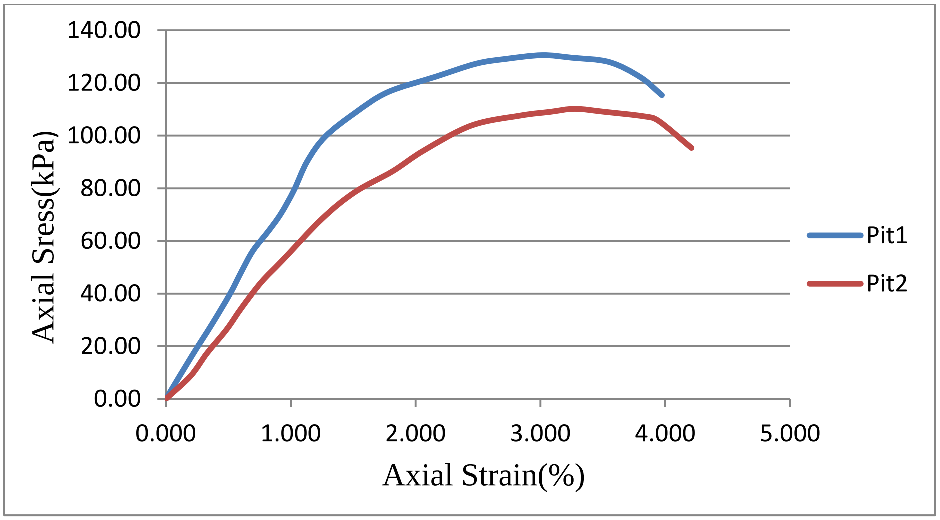

Figure 19: Unconfined compressive strength curve of natural soils

Table 4.7: Summary of geotechnical properties of natural soils

Type of tests		Test Methods	Pit 1	Pit 2
Depth of Sampling, (m)			1.5	1.5
Natural moisture content, (%)		ASTM D2216	25.26	26.76
Grain Size Analysis	Gravel (%)	ASTM D422	0.4	0
	Sand (%)		9.2	6.6
	Silt (%)		50	57.4
	Clay (%)		40	36
Specific Gravity		ASTM D854	2.67	2.65
Atterberg limits, (%)	Liquid Limit, %	ASTM D4318	71.5	82
	Plastic Limit, %		36.9	36.1
	Plastic Index, %		34.6	45.9
Free swell, (%)		IS 2720	110	120
Free swell index, (%)			91	110
Free swell ratio			1.91	2.1
AASHTO Classification		AASHTO	A-7-5	A-7-5
USCS Classification		USCS	MH	CH
Standard compaction test	MDD, (g/cc)	ASTM D698	1.306	1.232
	OMC, (%)		26.3	29.6
UCS test		ASTM D2166	130.56	110.18
Color			Black gray	Black

Table 4.8: Physical of properties of bagasse ash used in the study

Property of BA	Gs	LL (%)	PL (%)	MDD	OMC	Color
Value	2.20	43	NP	1.103	43	Black

4.2 Mineralogical Composition of Materials

Based on the test results, soil sample collected from the second pit have less strength and more swelling property when compared to pit2 natural soil samples collected from the same study area. Therefore, the effect of mixing this weakest soil with both additives in case of its geotechnical properties was investigated. And also characteristics of natural soil and both admixtures were assessed through qualitative tests like XRD analysis.

4.2.1. XRD result analysis of materials

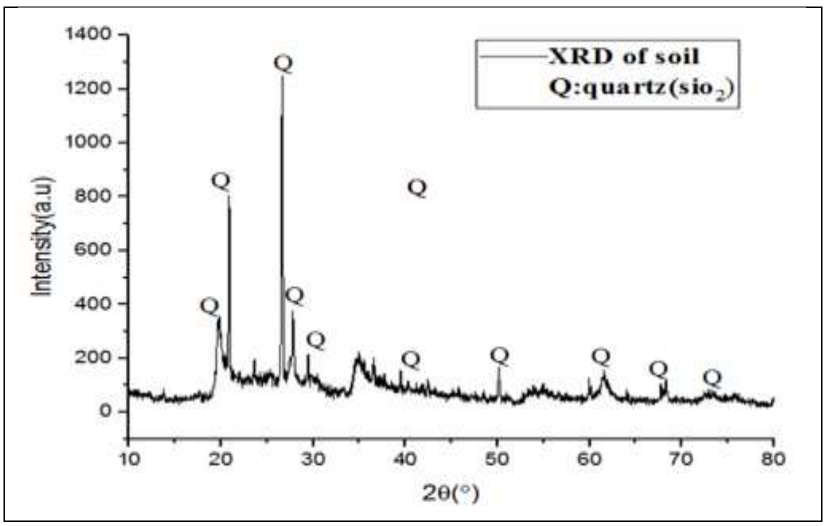

Figure 20: Image showing (a) XRD pattern result of natural soil

The XRD pattern of expansive soil on Figure 4.5 shows that soil sample mainly composed of quartz minerals. The characteristic peak of quartz occurred at 2θ=26.62929° or d=3.34479Å is present as a sharp peak that indicates the pronounced crystallinity of the SiO2 phase close to the values obtained from the JCPDS (046-1045) standard data.

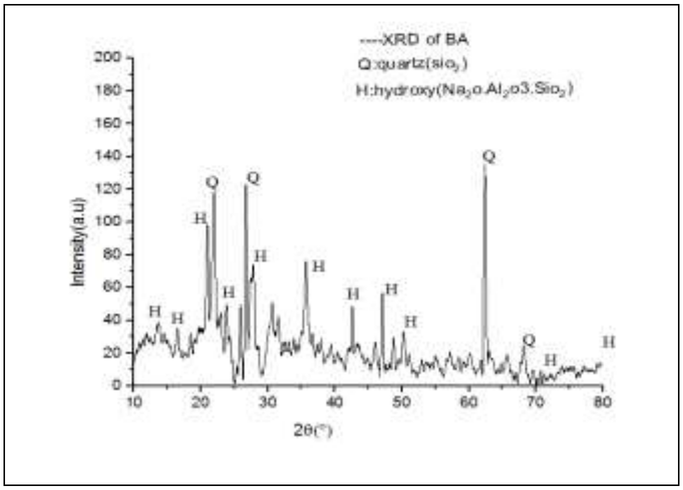

Figure 21: Image showing (a) XRD pattern result of BA

The XRD pattern of BA on Figure 4.6 shows that soil sample mainly composed of quartz and hydroxyl (sodium aluminum silicate hydrate) minerals. The characteristic peak of quartz occurred at 2θ=27.256142° or d=3.269271Å is present as a sharp peak that indicates the pronounced crystallinity of the SiO2 phase close to the values obtained from the JCPDS (046-1045) standard data and the XRD pattern of lime on Figure 4.7, shows that soil sample mainly composed of portlandite (calcium hydroxide) and calcite (calcium carbonate) minerals. The characteristic peak of portlandite occurred at 2θ=34.16154° or d=2.62257Å is present as a sharp peak that indicates the pronounced crystallinity of the ca(OH)2 phase close to the values obtained from the JCPDS (046-1045) standard data.

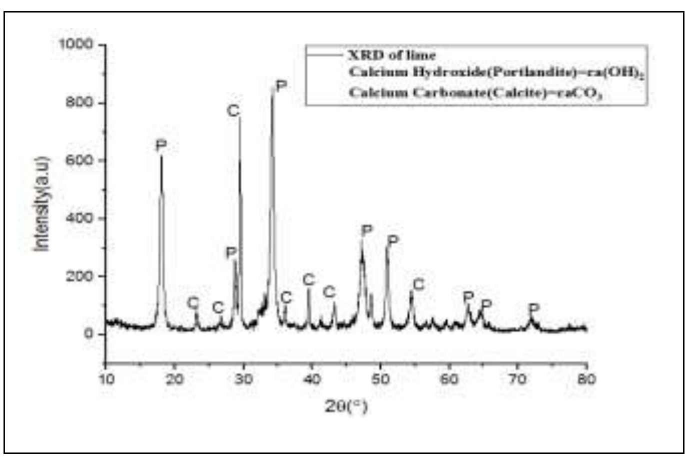

Figure 22: Image showing (a) XRD pattern result of BA

4.3 Effect of Bagasse Ash on Properties of Expansive Soil

For studying the effect of stabilizing materials on expansive soil, the Pit-2 soil was selected. Since it has less strength value compared to other pits Pit1 the other properties are almost similar.

4.3.1 Effect of bagasse ash on Atterberg limit

When the addition of bagasse ash to soil the liquid limit and plastic limit was decreased with the increasing bagasse ash content. It also decreased the plasticity index of soil when content of BA is increase but, its increase again at 20% BA this is shows 15% BA is optimum. It is due to the partial replacement of plastic soil particles with non-plastic particles of bagasse ash. This led to flocculation and agglomeration of clay particles. When there is a reduction in plasticity index significant decrease in swell potential will come. The summary of the effect of bagasse ash on the Atterberg limit is shown in Table 4.9.

Table 4.9: Effect of bagasse ash on Atterberg limit (24 hour cured)

% Bagasse ash	Liquid limit (LL)	Plastic limit (PL)	Plastic index (PI)
0	82	36.1	45.9
5	77	34.8	42.2
10	74	35.2	38.8
15	65.4	34.4	31
20	61.6	29.4	32.2

The decrease in the plasticity index is due to the partial replacement of plastic soil particles with non-plastic bagasse ash materials and may be due to a reduction in clay fraction (which is responsible for the plasticity of the soil)(Wubshet & Tadesse, 2014). It is also due to the cation exchange and flocculation of clay particles.

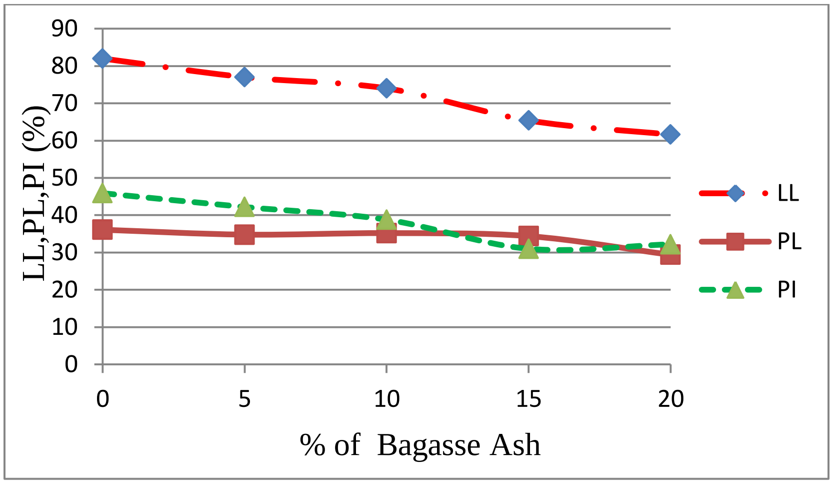

Figure 23: Atterberg limit test result for different percentages of bagasse ash (24 hour cured)

A reduction in plasticity index causes a significant decrease in swell potential and removal of some water that can be absorbed by clay minerals. The higher the plasticity index, there will be the greater engineering problems associated with using the soil as an engineering material, such as road subgrade and foundation support for a light building.

4.3.2 Effect of bagasse ash on linear shrinkage of expansive soil

The effect of different percentages of bagasse ash on linear shrinkage of expansive soil is summarized in Table 4.10. The addition of bagasse ash decreases the linear shrinkage value From 22.81 to 13.82 the soil is changed from critical stage to moderate shrinkage behavior of expansive soil. The increase in bagasse ash resulted in a decrease in linear shrinkage of soil. These could be due to the flocculation and aggregation phenomena of clay particles induced by the presence of bagasse ash contributing to the formation of coarser particles and enhancing the strength of the treated soil(Melese, 2018).

Table 4.10: Effect of bagasse ash on linear shrinkage of expansive soil

% BA	Initial length	Final length	Shrinkage limit
0	14	11.4	22.81
5	14	11.7	19.66
10	14	11.9	17.65
15	14	12.1	15.70
20	14	12.3	13.82

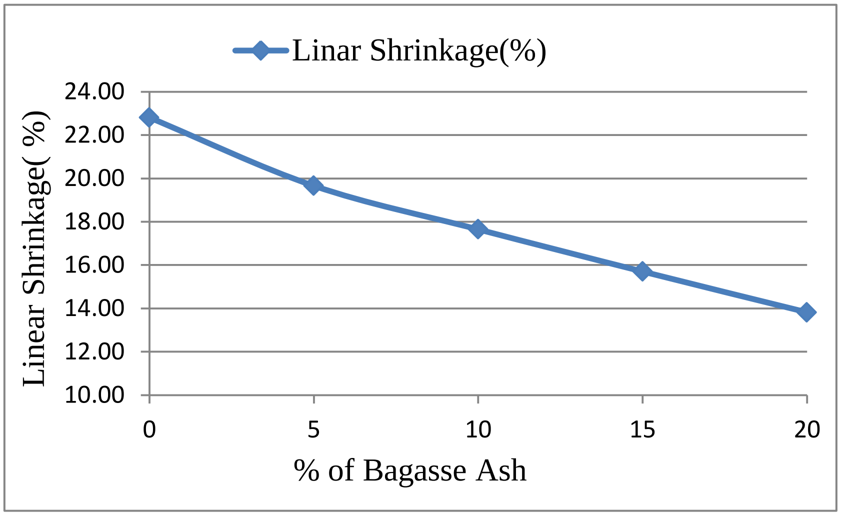

Figure 24: Linear shrinkage of expansive soil mixed with various bagasse ash contents

4.3.3 Effect of bagasse ash on the free swell, free swell index, and free swell ratio

The addition of bagasse ash to expansive soil slightly decreases the free swell, free swell index, and free swells ratios. The highest decrease is obtained when the soil is treated with 20% bagasse ash. The free swell index value decreased from 110% to 71 % with an increased amount of bagasse ash percentages from 0% to 20%. According to swell classification based on a free swell, the treated soil still has a high degree of expansion as discussed by (Chen, 1975). The decrease in free swell values is mainly due to the replacement of expansive clay minerals with bagasse ash which have less affinity for absorbing water as compared to expansive soil. The summary of the free swell, free swell index, and the free swell ratio is discussed in Table 4.11 and Figures 4.10

Table 4.11: The summary of the free swell, free swell index, and free swell ratio

% BA	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	Degree of expansion
0	10	22	10.5	110	120	2.10	Very high
5	10	21.5	11	95	115	1.95	Very high
10	10	20	10.5	90	100	1.90	Very high
15	10	19	11	73	90	1.73	Very high

20	10	18	10.5	71	80	1.71	Very high

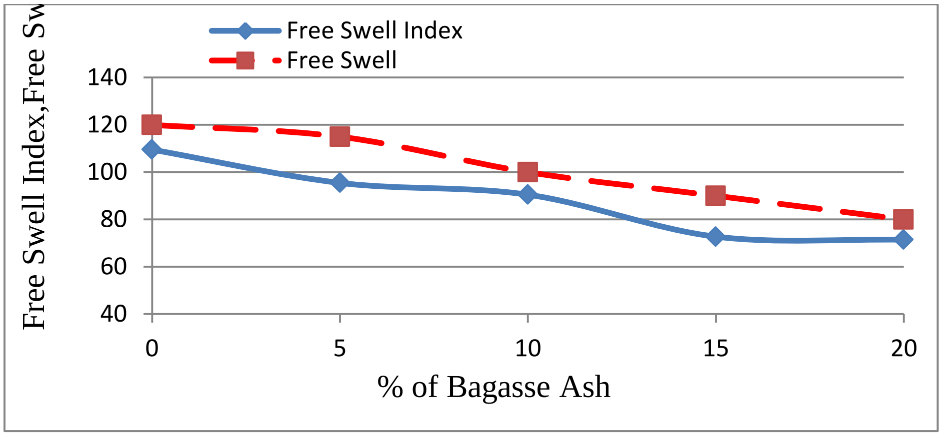

Figure 250: Results of the free swell index and free swell

Free swell index of natural soil collected from Pit 2 was observed as 110%. This makes the soil to fall under very high degree of expansion category as per the classification stated in(IS 2911, 1980). Whereas, with mixing of 5%, 10%, 15% and 20% of BA to soil the swelling property of natural soil reduces to 95%, 90%, 73% and 71% respectively and all value is above 50% and it falls under high degree of expansion.. Therefore, bagasse ash alone is not effective and efficient admixture to reduce the swelling characteristics of expansive soils.

4.3.4 Effect of bagasse ash on the specific gravity of expansive soil

When different percentages of bagasse ash (0%, 5%, 10%, 15% and 20%) are added to the natural soil sample the specific gravity is decreased from 2.65 to 2.56 as shown in Table 4.12. This decrease in specific gravity is due to the partial replacement of higher specific gravity soil with bagasse ash which has lower specific gravity. It can be seen that the more the amount of stabilizer lesser would be the specific gravity. It is due to the replacement of soil with the tiny particles of stabilizers, which are lighter in weight than soil particles. The effect of bagasse ash on the specific gravity of the soil.

Table 4.12: Specific gravity of natural soil with various percentage of bagasse ash

Specific gravity of expansive soil at different % of bagasse ash

% of BA	O%	5%	10%	15%	20%
Specific gravity (Gs)	2.65	2.62	2.60	2.58	2.56
% Decrease in Gs	0	1.13	1.88	2.64	3.39

4.3.6 Effect of bagasse ash on compaction characteristics

Table 4.13: Compaction test results of soil with different bagasse ash percentage

Bagasse ash (%)	MDD(g/cc)	OMC (%)	% Decrease (MDD)	%Increase (OMC)
0	1.232	29.6	-	-
5	1.222	31.5	0.812	6.418
10	1.201	33.8	2.516	14.189
15	1.188	35.5	3.571	19.932
20	1.187	37	3.656	25

Maximum dry density decreases from 1.232g/cc to 1.187g/cc of soil samples with increased bagasse ash content from 0% to 20%. It shows the addition of bagasse ash on the expansive soil sample reduced the maximum dry density compared to that of natural soil. Thus, the decrease in the maximum dry density is mainly due to the partial replacement of comparatively heavy soils with lightweight bagasse ash. And also, the decrease is due to the flocculation and agglomeration of clay particles in the presence of sufficient water, resulting in an increase in voids and a corresponding decrease in dry density(Bethlehem Worku, 2015).

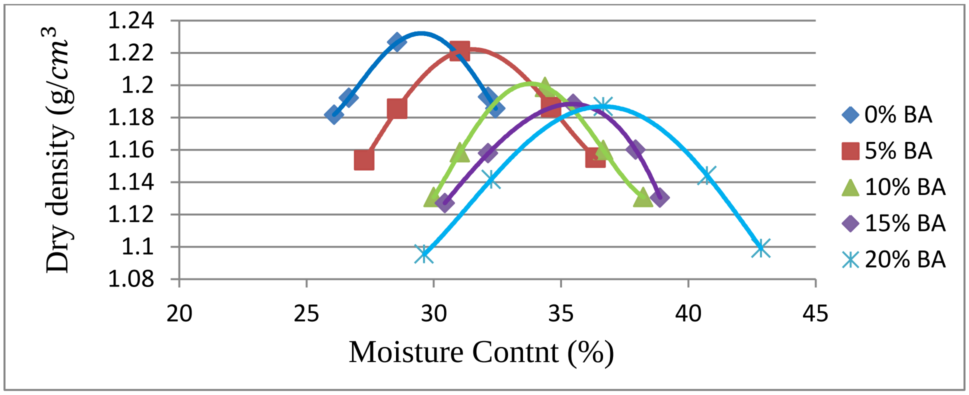

Figure 26: The effect of bagasse ash on compaction

The optimum moisture content of the soil samples increased from 29.6% to 37% with increases in bagasse ash content of 0% to 20% .The reason for the increasing OMC may be due to the affinity of the soil-bagasse ash mixed samples for more water to complete the cation exchange reaction. Thus, the increase in OMC may be due to more water absorption by bagasse ash. This implies that more water is needed to compact the soil with bagasse ash the optimum moisture content of the soil increases with increases in bagasse ash because it is finer than the soil. The more fines the more surface area, so more water is required to provide well lubrication. The increase in water content was also attributed to the pozzolanic reaction of the mixes with the soil.

4.3.7 Effect of bagasse ash on unconfined compressive strength (UCS) expansive soil

The summary of the UCS test result for different percentages of bagasse ash is tabulated in table 4.14 and figure 4.12 for cured 7,14,28,60 and 90 days cured samples. For 0% BA after day 7 curing is not important because there is reaction and for 20% BA after optimum value is determine cured for other day is not needed. As maturation period or curing time is increased within increase percent of bagasse ash the value of UCS is increased up to 28 day cured for 5% and 10% but it is decreased for 60 and 90 day cured for both 5% and 10% of BA content. But, for 15% of BA as maturation period or curing time is increase the value of UCS is increased.

Table 4.14: UCS test results of soil with different bagasse ash percentage

UCS test result for different percentages of bagasse ash (kPa)						
% of BA	Uncured	Cured day 7	Cured day14	Cured day28	Cured day60	Cured day90

0%	110.18	116.12	-	-	-	-
5% BA	153.82	163.5	178.35	204.8	195.48	191.46
10% BA	161.4	173.2	186.2	207.61	200.84	195.18
15% BA	185.32	195.8	211.35	234.26	241.52	245.84
20%BA	167.00	187.64	-	-	-	-

From this, the value of unconfined compressive strength result of uncured and cured day 7, up to 15% addition of BA was increased, and then sudden decrease was observed at 20% addition of BA to the soil. The maximum value of UCS is 185.32kPa and 195.8kPa observed at 15% bagasse ash content for uncured and cured for 7 day conditions. After identified optimum value, the curing time was observed for 5%, 10% and 15% of bagasse ash content for 7,14, 28, 60 and 90 days curing to understand more the influence of maturation period on engineering property of bagasse ash-soil mixture. Generally, it can be expressed that as curing time increase within additions of bagasse ash expansive soils result in chemical reactions in the forms of cation exchange, pozzolanic reaction. During cation exchange, flocculation of clay particles occurs giving rise to agglomeration into coarser particles helping resist the compressive stresses much more effectively in comparison with those of natural soil. In addition, the cementation process plays a key role in binding the clay particles together and consequently increasing the compressive stress.

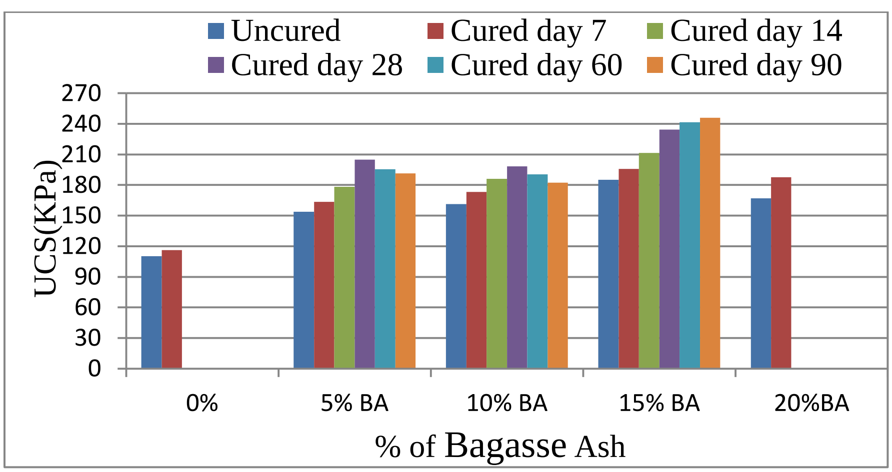

Figure 27: Effect of maturation period on stress-strain behavior of soil with different % of bagasse ash

The substantial improvement in compressive strength could be attributed to cation exchange and pozzolanic reactions due to a high amount of alumina and silica available in bagasse ash and this reaction is not stopped by once it`s continued within time increase. It is also seen that the unconfined compressive strength of treated expansive soil increased with increasing curing time as illustrated in Table above. The compressive strength of treated soil increases significantly with curing time, which reveals the higher increase in additive content stabilized expansive soil, produced a larger impact on the compressive strength of treated soils.

4.4 Effect of Lime on Bagasse Ash Stabilized Expansive Soil

The optimum bagasse ash content (15%) and percentages of lime with ratio (3:1or 11.25% BA& 3.75% Lime and 2:2 or 7.5% BA & 7.5% Lime) were used for further investigation of the effect of lime on bagasse ash stabilized expansive soil. A compaction test and unconfined compressive strength (UCS) test were done.

4.4.1Effect of lime with bagasse ash on compaction characteristics

Table 4.15: Compaction test results of soil with different bagasse ash percentage

Material %	MDD(g/cm3	OMC (%)	% Decrease (MDD)	%Increase (OMC)
15% BA	1.188	35.5	-	-
3:1, 15% BA & Lime	1.174	35.5	1.18	0
2:2, 15% BA & Lime	1.123	37.6	5.47	5.92

The maximum dry density decreases from 1.188g/cc to 1.174g/cc and 1.123g/cc for 3:1 and 2:2 respectively of soil samples with increased lime content to that of natural soil. Thus, the decrease in the maximum dry density is mainly due to the partial replacement of dense soils with the light weight bagasse ash and high content of Cao in lime or it is due to the partial replacement of higher specific gravity soil with low specific gravity bagasse ash and lime(Barasa, 2016). And also, the decrease is due to the flocculation and agglomeration of clay particles in the presence of sufficient water, resulting in an increase in voids and a corresponding decrease in dry density.

The optimum moisture content of the soil samples increased from 35.5% to 35.5 and 37.6 % for 3:1 and 2:2 respectively with increases in bagasse ash and lime content. The reason for the increasing OMC may be attributable to the affinity of the soil-bagasse ash and lime mixed samples for more water to complete the cation exchange reaction. Thus, the increase in OMC may be due to more water absorption by bagasse ash and lime. This implies that more water is needed to compact the soil with bagasse ash and lime mixture.

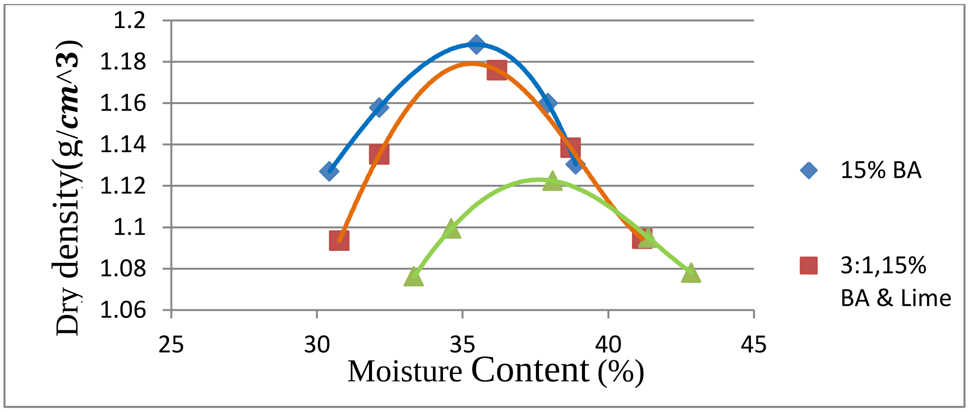

Figure 28: The effect of bagasse ash on compaction

4.4.2 Effect of lime with bagasse ash on Atterberg Limit

Table 4.16: Effect of lime mix with bagasse ash on Atterberg limit

Material %	LL (%)	PL (%)	PI (%)
15% BA	65.4	34.4	31.0
3:1,15% BA& Lime	63.0	38.6	24.4
2:2,15% BA & Lime	60	39.2	20.8

The liquid limit was decreased from 65.4% to 60% with the increasing lime from 15% BA alone to 2:2, 15% of both material content and plastic limit was increased from 34.4% to 39.2% with the increasing lime from 15% BA alone to 2:2, 15% of both material content. It also decreased the plasticity index of soil 31% to 20.8% with the increasing lime from 15% BA alone to 2:2, 15% of both material content. Decrease in liquid limit was caused by the free Ca^{2+} obtained from lime and exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles. This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, to form a cluster or floc of the clay particles. This led to flocculation and agglomeration of clay particles. When there is a reduction in plasticity index significant decrease in swell potential will come. The decrease in the plasticity index is due to the partial replacement of plastic soil particles with non-plastic lime materials and may be due to a reduction in clay fraction (which is responsible for the plasticity of the soil)(Teddy et al., 2021). It is also due to the cation exchange and flocculation of clay particles. A reduction in plasticity index causes a significant decrease in swell potential and removal of some water that can be absorbed by clay minerals.

4.4.3 Effect of lime with bagasse ash on swelling property of soil

Table 4.17: swelling index of soil mixed with BA and Lime by different ratio

% of material	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	Degree of expansion
15%BA	10	19	11	72.7	90	1.73	Very high
3:1,15%BA & Lime	10	18	12	50	80	1.5	Very high
2:2,15%BA & Lime	10	15.5	13	19.23	55	1.19	Low

When 15% of BA is mixed with natural soil free swell index is 72.7% and it fall under very high degree of expansion but after lime added by 3:1 and 2:2 ratio it decreased to 50% and 19.23% respectively. Hence, for mixing of 2:2/15% BA and lime, soil sample was improves its swelling potential and falls into low category degree of expansion. Therefore, using a combination of both 2:2/15% of BA and lime is a more effective. The combination of bagasse ash and lime is give good result for method in reducing the swelling characteristics of expansive soils than using BA alone(Wubshet, 2013).

4.4.4. Effect of lime on unconfined compressive strength of BA stabilized soil

Table 4.18: UCS test results of soil 15% of bagasse ash and lime with 3:1 and 2:2

Unconfined compressive strength(KPa)						
% of materials	Uncured	Cured day 7	Cured day 14	Cured day 28	Cured day 60	Cured day 90
0% BA	110.18	116.12	-	-	-	-

15% BA	185.32	195.8	211.35	234.26	241.52	245.84
15%,3:1 BA & Lime	230.08	308.26	384.21	494.56	715.4	730.84
15%,2:2 BA & Lime	265.74	412.36	542.86	960.46	1120.32	1340.62

Based on this when BA mixed with soil alone, 15% of BA provides better result under cured condition & proposed as optimum amount for the case and 90 day curing is best result. Again, when BA is combined with different ratio of lime, 2:2(7.5% of BA and 7.5% of lime) is shows best result rather than BA alone under cured state. The increase in unconfined compressive of bagasse ash stabilized expansive soil when mixed lime gives more strength value than using bagasse ash alone. The reason why the 2:2(7.5% of BA and 7.5% of lime) mixture is better than other combination is as increase the percent lime the amount of calcium ion also increased and when the calcium comes in contact with the pore water hydration is occurs resulting in the formation calcium hydroxide. Some of this calcium hydroxide is adsorbed onto the soil particle is becomes courser due to the flocculation of clay particles. And also 90 day cured is shows best result than other maturation period because BA and lime are a pozzolanic reaction which is time dependent. On both access to water, pozzolanic material and the content of calcium ions, and this is simply there is no enough calcium in bagasse ash. This shows that certain amount of calcium ion from lime is necessary to make successful reaction.

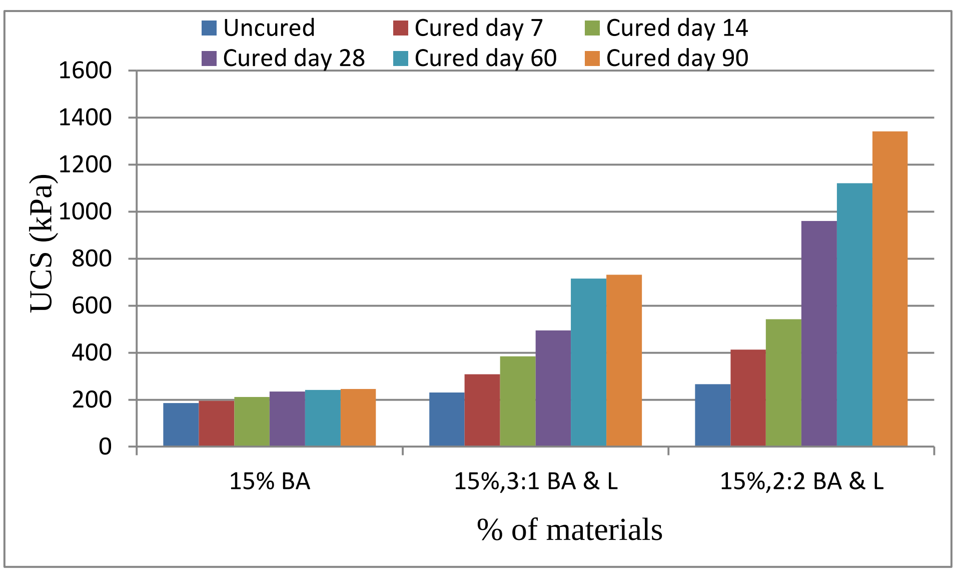

Figure 29: Effect of maturation time period on UCS test results of soil 15% of bagasse ash and lime with 3:1 and 2:2

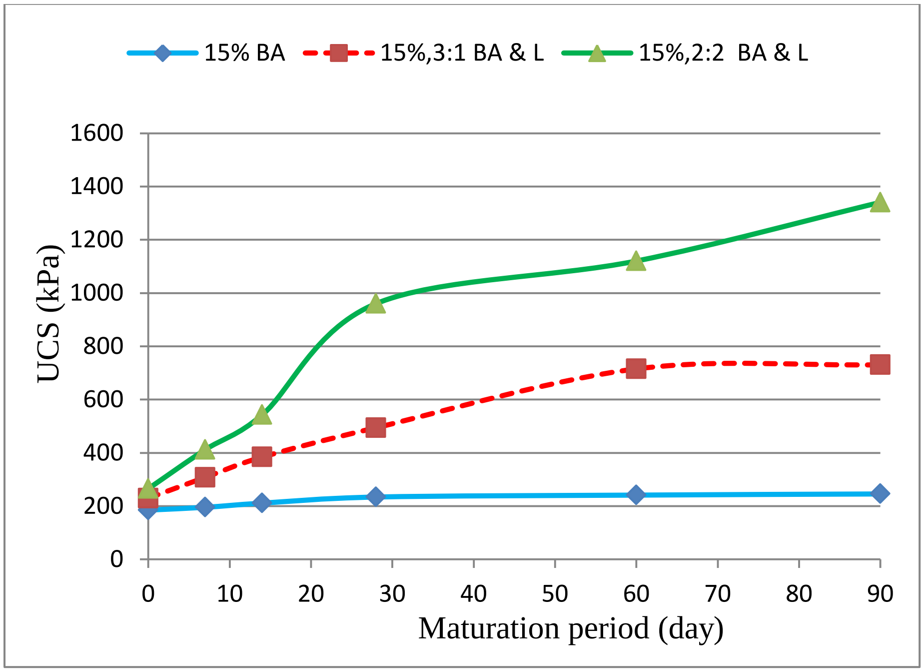

Figure 30: Effect of maturation time period on UCS test results of soil 15% of bagasse and lime by 3:1 and 2:2

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

This study was objected to investigate the influence of maturation time period of bagasse ash and lime on geotechnical behavior of expansive soils. Clayey soil was mixed with 5%, 10%, 15% and 20% of BA and 15% of BA and lime by 3:1 and 2:2 to assess exclusive effect of BA and combined effect of optimum BA with varied lime on properties of soil. Finally, the following results were obtained and concluded as follows.

- The plasticity index of expansive soil showed decrease as of addition of bagasse ash increase. OMC increased and MDD decreased and the specific gravity shows a decrease ,free swell, free swell index and free swell ratio also showed little decrement with an increasing amount of bagasse ash .
- For addition of BA up to 20% by dry weight, FSI decreased from 110% to 71%, liquid limit decreased from 82% to 61.6%, plastic limit decreased from 36.1% to 29.4%, plasticity index decreased from 45.9% to 32.2% and linear shrinkage decreased from 22.81% to 13.82%.
- Maximum dry density of sample was also decreased from 1.232g/cm3 to 1.187g/cm3 and optimum moisture content increased from 29.6% to 37% when BA was added up to 20% by dry weight.
- Unconfined compressive strength of samples was increased from 110.18Pa to 185.32kPa and 195.8kPa for both uncured and 7day cured conditions respectively with addition of 15% BA, which was the optimum value taken for this study. For more maturation time period or long curing time the UCS value was increased up to 90 day curing for only 15% of BA but it shows decrease for 5% and 10% of BA after 60 day curing time.
- After combined effect of lime and bagasse ash gives good strength than the strength given by stabilizing bagasse ash alone. Again after mixing of 15% of both BA with lime varied by ratio of 3:1 and 2:2 to soil sample, properties such as FSI, liquid limit and plasticity index decreases from 72.7% to 19.23%, 65.4% to 60% and 31% to 20.8respectively.

- Maximum dry density of sample was decreased from 1.188g/cm3 to 1.1.123g/cm3 and optimum moisture content increased from 35.5% to 37.6% when 15% of BA and lime is mixed by ratio of 2:2.
- The value of UCS is improved from 185.32kPa and 245.84kPa to 265.74kPa and 1340.62kPa for uncured and 90 day cured conditions respectively which show curing have more impact on strength of material and also maturation time period of bagasse ah-lime is effective change depend on time rather than bagasse ash mixed alone with nature soil. The proportion of 15% of both BA and lime by ratio of 2:2 with bare soil had provided better material property result and by considering the improvement of some geotechnical properties of expansive soil.

5.2. Recommendation

- ✓ To evaluate the influence of maturation time period of those materials was limited only by UCS test result, but it should also conducted additional test.
- ✓ Further study is recommended on the done cost estimation and check with cost of suitable materials.
- ✓ The study was tried to investigate only separate mineralogical characterization of soil and both admixtures, but it should also be mentioned to characterize combination of soil with additives.
- ✓ Stabilizing soils with material wasted from aluminum factory and bagasse ash on different types of soils.

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APPENDICES

Appendix: A Natural moisture content test result

Table A-1 Natural moisture content for Pit1 and Pit2

Sample name	Pit 1			Pit 2		
Container No	A1	A2	A3	B1	B2	B3
Mass of container, g	19	20	30	19	19	20
Mass of container + Wet soil, g	92	94	140	114	116	113
Mass of container + Dry soil, g	78	77	120	91	96	96
Mass of water, g	14	17	20	23	20	17
Mass of dry soil, g	59	57	90	72	77	76
Moisture content	23.73	29.82	22.22	31.94	25.97	22.37
Average moisture content (%)	25.26			26.76		

Appendix-B: Specific Gravity of Materials Used for the Study

Table B-1: Specific gravity of soil sample collected from Pit1 and Pit2

Sample name	pit1		pit2	
Pycnometer No.	pit1 A	Pit1 B	Pit2 A	Pit2 B
Weight of dry soil, ws (gm)	10	10	10	10
Weight of pycnometer + soil + water, Wpws (g)	146.84	145.28	144.52	141.76
Temperature, Tx(oc)	22	22	21	21
Weight of pycnometer + water at Tx, Wpw (at Tx) (g)	140.53	139.08	138.15	135.68
Conversion factor, K	0.9998	0.9998	0.9996	0.9996
Specific Gravity of soil at 20=(Kws/(ws+Wpw-Wps)	2.71	2.63	2.75	2.55
Average specific gravity of	2.67		2.65	

soil.		

Table B-2 Specific gravity test result for bagasse ash and lime

Sample name	BA only		Lime only	
Pycnometer No.	M1	M02	M03	M04
Weight of dry material, ws (gm)	10	10	10	10
Weight of pycnometer + material + water, Wpws (g)	142.13	143.42	141.86	142.37
Temperature, Tx(oc)	21	21	22	22
Weight of pycnometer + water at Tx, Wpw (at Tx) (g)	136.54	138.12	136.34	136.42
Conversion factor, K	0.9996	0.9996	0.9998	0.9998
Specific Gravity of soil at 20=(Kws/(ws+Wpw-Wps)	2.27	2.13	2.23	2.47
Average specific gravity of soil.	2.20		2.35	

Table B-3 Specific gravity test result for different percentages of BA +soil

Sample name	0%		5%		10%		15%		20%	
Pycnometer No.	PI1	PI2	BA0	BA1	BA2	BA3	BA4	BA5	BA6	BA7
Weight of dry material, ws (gm)	10	10	10	10	10	10	10	10	10	10
Weight of pycnometer + material + water, Wpws (g)	144.52	141.76	143.25	146.54	147.36	142.64	140.4	145.61	141.64	144.78
Temperature, Tx(oc)	22	22	21	21	21	21	21	21	22	22
Weight of pycnometer + water at Tx, Wpw (at Tx) (g)	138.15	135.68	137.05	140.36	141.2	136.5	134.31	139.45	135.56	138.68
Conversion factor, K	0.9998	0.9998	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9998	0.9998
Specific Gravity of soil at 20=(Kws/(ws+Wpw-Wps)	2.75	2.55	2.63	2.62	2.60	2.59	2.56	2.60	2.55	2.56
Average specific gravity of soil.	2.65		2.62		2.60		2.58		2.56	

Appendix-C: Swelling Properties of Materials

Table C-1: Free swell index and free swell ratio of soil sample Pit1 and Pit2

sample name	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	swell potential
Pit 1	10	21	11	91	110	1.91	High
Pit 2	10	22	10.5	110	120	2.10	High

Table C-2: Free swell index and free swell ratio of different % of BA

% BA	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	swell potential
0	10	22	10.5	110	120	2.10	High
5	10	21.5	11	95	115	1.95	High
10	10	20	10.5	90	100	1.90	High
15	10	19	11	73	90	1.73	High
20	10	18	10.5	71	80	1.71	High

Table C-2: Free swell index and free swell ratio of BA and lime combination

% of material	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	swell potential
% of BA alone	10	19	11	72.73	90	1.73	High
15% of BA & L by 3:1	10	18	12	50	80	1.50	High
15% of BA & L by 2:2	10	15.5	13	19	55	1.19	Low

Appendix-D: Particle Size Distribution Result

Table D-1: Grain size analysis result of soil sample collected from Pit-1

sieve size(mm)	mass sieve	mass sieve +retain soil	mass retain	% on each sieve	cumulative % retain	% finer 100
4.75	441	443	2	0.4	0.4	99.6

2.36	405	409	4	0.8	1.2	98.8
2	387	390	3	0.6	1.8	98.2
1.18	360	364	4	0.8	2.6	97.4
0.6	330	335	5	1	3.6	96.4
0.3	303	309	6	1.2	4.8	95.2
0.15	279	289	10	2	6.8	93.2
0.075	272	286	14	2.8	9.6	90.4
pan	252	704	452	90.4	100	0

Table D-2: Hydrometer analysis result of soil sample collected from Pit-1

t,min	Rh	Cm = +1	(Ra)	Temp (oC)	L (mm)	K (Table)	,D (mm)	a	. Rc	P	PA
1	44	1	45	24	11.7	0.0133	0.0455	0.996	39.996	79.67	74.41
2	44	1	45	22	9.7	0.0133	0.0293	0.996	39.996	79.67	74.41
5	42	1	43	22	9.7	0.01315	0.0183	0.996	37.996	75.69	70.69
10	41	1	42	22	10.25	0.0133	0.0135	0.996	36.996	73.70	68.83
15	39	1	40	22	10.55	0.0133	0.0112	0.996	34.996	69.71	65.11
30	37	1	38	22	10.9	0.0133	0.0080	0.996	32.996	65.73	61.39
60	36	1	37	24	11.7	0.01315	0.0058	0.996	31.996	63.74	59.53
120	33	1	34	22	11.4	0.0133	0.0041	0.996	28.996	57.76	53.95
240	30	1	31	24	11.7	0.013	0.0029	0.996	25.996	51.78	48.37
480	26	1	27	24	12.2	0.01315	0.0021	0.996	21.996	43.82	40.92
1440	22	1	23	22	12.5	0.0133	0.0012	0.996	17.996	35.85	33.48

Table D-3: Grain size analysis result of soil sample collected from Pit-2

sieve size(mm)	mass sieve	mass sieve + retain soil	mass retain	% on each sieve	cumulative % retain	% finer 100
4.75	441	441	0	0	0	100
2.36	405	409	4	0.8	0.8	99.2
2	387	390	3	0.6	1.4	98.6
1.18	360	364	4	0.8	2.2	97.8
0.6	330	335	5	1	3.2	96.8
0.3	303	309	6	1.2	4.4	95.6
0.15	279	284	5	1	5.4	94.6
0.075	272	278	6	1.2	6.6	93.4
	252	719	467	93.4	100	0

Table D-4: Hydrometer analysis result of soil sample collected from Pit-2

t,min	Rh	Cm = +1	(Ra)	Temp (oC)	L (mm)	K (Table)	D (mm)	a	Rc	P	PA
1	41	1	42	22	9.4	0.0133	0.0408	1	37	74.00	69.12
2	39	1	40	22	9.7	0.0133	0.0293	1	35	70.00	65.38
5	37	1	38	23	10.1	0.01315	0.0187	1	33	66.00	61.64
10	36	1	37	22	10.25	0.0133	0.0135	1	32	64.00	59.78
15	34	1	35	22	10.55	0.0133	0.0112	1	30	60.00	56.04
30	32	1	33	22	10.9	0.0133	0.0080	1	28	56.00	52.30
60	31	1	32	23	11.1	0.01315	0.0057	1	27	54.00	50.44
120	29	1	30	22	11.4	0.0133	0.0041	1	25	50.00	46.70
240	27	1	28	24	11.7	0.013	0.0029	1	23	46.00	42.96
480	24	1	25	23	12.2	0.01315	0.0021	1	20	40.00	37.36
1440	20	1	21	22	12.5	0.0133	0.0012	1	16	32.00	29.89

Appendix-E: Compaction Test Results

Table E-1: Compaction test results of natural soil collected from Pit-1

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5200	5320	5380	5360	5320
Mass of compacted soil (g)	1470	1590	1650	1630	1590
Wet unit weight (KN/m3)	1.47	1.59	1.65	1.63	1.59
Mass of wet soil +can(g)	52	60	48	63	65
Mass of dry soil +can(g)	47	53	43	54	55
Mass water (g)	5	7	5	9	10
Mass of can(g)	24	24	24	23	23
Mass of dry soil(g)	23	30	28	28	37
Moisture content (%)	21.739	24.139	26.316	29.032	31.25
Dry unit weight(KN/m3)	1.2075	1.281	1.306	1.263	1.211

Table E-2: Compaction test results of natural soil collected from Pit-2

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5220	5240	5307	5306	5300

Mass of compacted soil (g)	1490	1510	1577	1576	1570
Wet unit weight (KN/m3)	1.49	1.51	1.577	1.576	1.57
Mass of wet soil +can(g)	55	61	58	60	70
Mass of dry soil +can(g)	49	53	50	51	58
Mass water (g)	6	8	8	9	12
Mass of can(g)	26	23	22	23	21
Mass of dry soil(g)	23	30	28	28	37
Moisture content (%)	26.087	26.667	28.57	32.143	32.43
Dry unit weight(KN/m3)	1.182	1.192	1.2265	1.193	1.1855

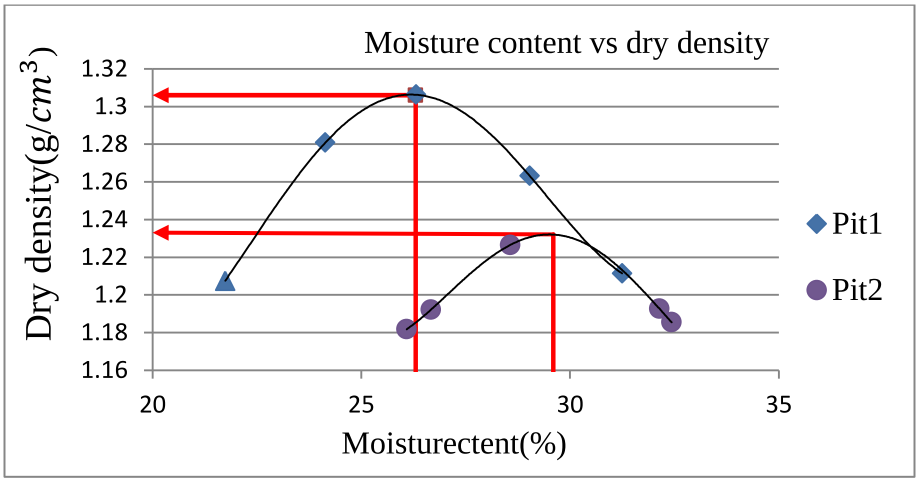

Figure E-1: Compaction curve of natural soil collected from Pit-1 and Pit-2

Table E-3: Compaction test results of 5% BA + soil

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5160	5238	5330	5320	5290
Mass of compacted soil (g)	1468	1524	1600	1597	1575
Wet unit weight (KN/m3)	1.468	1.524	1.6	1.597	1..575
Mass of wet soil +can(g)	51	60	62	59	65
Mass of dry soil +can(g)	45	52	53	50	53
Mass water (g)	6	8	9	9	12
Mass of can(g)	23	24	24	24	20
Mass of dry soil(g)	22	28	29	26	33
Moisture content (%)	27.273	28.571	31.0345	34.6154	36.3636
Dry unit weight(KN/m3)	1.153	1.185	1.221	1.1863	1.155

Table E-4 Compaction test results of 10% BA + soil

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5200	5248	5340	5315	5293
Mass of compacted soil (g)	1470	1518	1611	1585	1563
Wet unit weight (KN/m3)	1.47	1.518	1.611	1.585	1.563
Mass of wet soil +can(g)	53	62	65	66	70
Mass of dry soil +can(g)	47	53	54	55	57
Mass water (g)	6	9	11	11	13
Mass of can(g)	27	24	22	25	23
Mass of dry soil(g)	20	29	32	30	34
moisture content(%)	30	31.03	34.38	36.67	38.24
dry unit weight(KN/m3)	1.13	1.16	1.2	1.16	1.13

Table E-5 Compaction test results of 15% BA + soil

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5200	5260	5340	5330	5300
Mass of compacted soil (g)	1470	1530	1610	1600	1570
Wet unit weight (KN/m3)	1.47	1.53	1.61	1.6	1.57
Mass of wet soil +can(g)	51	60	63	65	71
Mass of dry soil +can(g)	44	51	52	54	57
Mass water (g)	7	9	11	11	14
Mass of can(g)	21	23	21	25	21
Mass of dry soil(g)	23	28	31	29	36
Moisture content (%)	30.434	32.142	35.484	37.931	38.889
Dry unit weight(KN/m3)	1.127	1.1578	1.1883	1.16	1.1304

Table E-6 Compaction test results of 20% BA + soil

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5150	5240	5352	5340	5300
Mass of compacted soil (g)	1420	1510	1622	1610	1570
Wet unit weight (KN/m3)	1.42	1.51	1.622	1.61	1.57
Mass of wet soil +can(g)	55	63	65	60	64
Mass of dry soil +can(g)	47	53	54	49	52
Mass water (g)	8	10	11	11	12
Mass of can(g)	20	22	24	22	25
Mass of dry soil(g)	27	31	30	27	28
Moisture content (%)	29.63	32.258	36.667	40.741	42.857

Dry unit weight(KN/m3)	1.095	1.142	1.187	1.144	1.099

Table E-7 Compaction test results of 15% of BA and lime by 3:1 + soil

Trial	1	2	3	4	5
Mass of compacted soil+ mold(g)	5160	5230	5330	5309	5275
Mass of compacted soil (g)	1430	1500	1600	1579	1545
Wet unit weight (KN/m3)	1.43	1.5	1.6	1.579	1.545
Mass of wet soil +can(g)	55	58	61	63	70
Mass of dry soil +can(g)	47	49	51.7	51	56
Mass water (g)	8	8	9.3	12	14
Mass of can(g)	21	21	26	20	22
Mass of dry soil(g)	26	28	25.7	31	34
Moisture content (%)	30.769	32.143	36.143	38.71	41.1764
dry unit weight(KN/m3)	1.094	1.135	1.175	1.138	1.0943

Table E-8 Compaction test results of 15% BA of and lime by 2:2 + soil

Trial	1	2	3	4	5
mass of compacted soil+ mold(g)	5120	5200	5260	5271	5260
mass of compacted soil (g)	1390	1470	1530	1541	1530
wet unit weight (KN/m3)	1.39	1.47	1.53	1.541	1530
mass of wet soil +can(g)	50	55	55	62	64
mass of dry soil +can(g)	43	46	46	50	51
mass water (g)	7	9	9	12	13
mass of can(g)	22	21	23	23	24
mass of dry soil(g)	21	25	23	27	28
moisture content(%)	33.333	36	39.13	44.44	46.428
dry unit weight(KN/m3)	1.043	1.081	1.1	1.067	1.0448

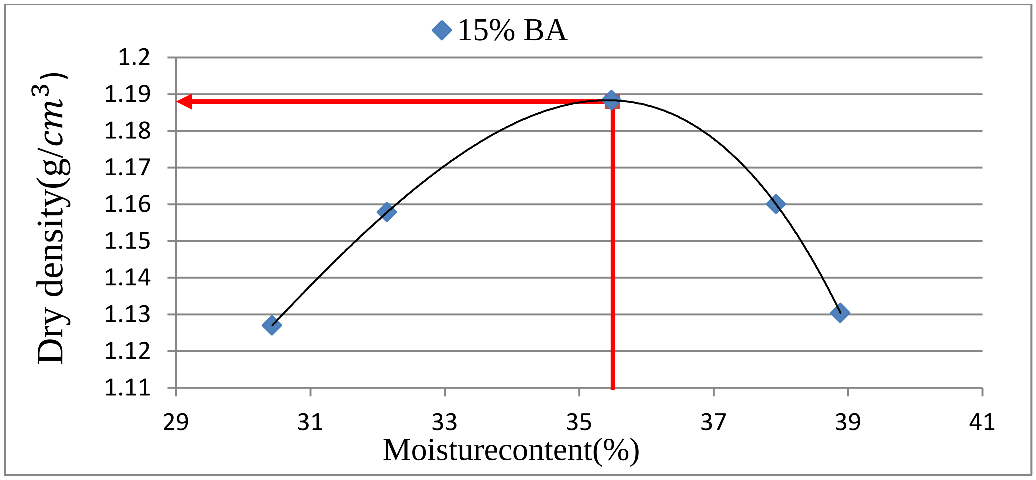

Figure E-1 Compaction curve for (Soil + 15% Bagasse ash)

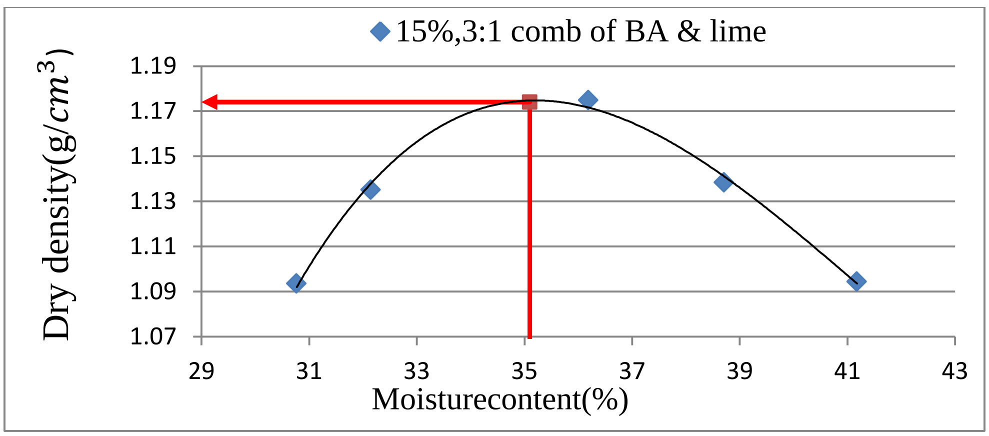

Figure E-2 Compaction curve for (Soil + 15% Bagasse ash with lime by 3:1)

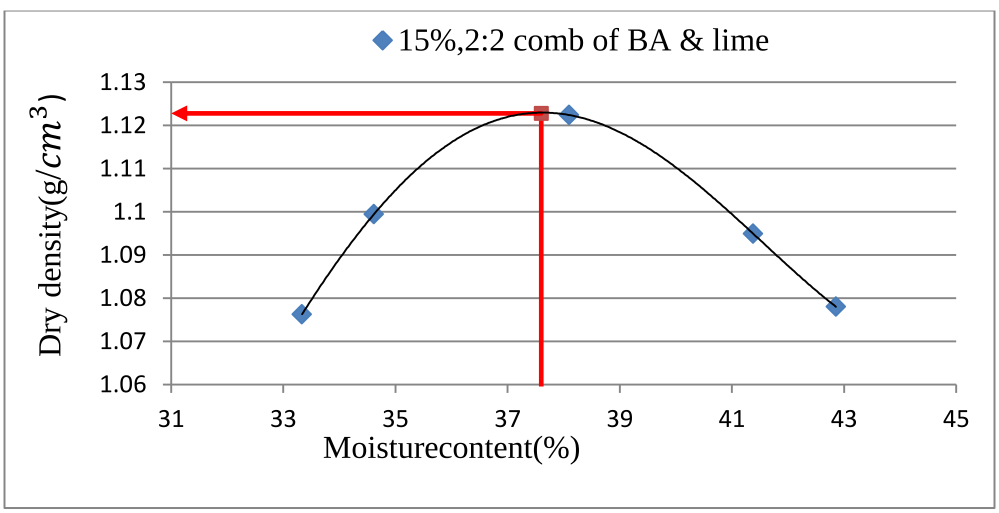

Figure E-3 Compaction curve for (Soil + 15% Bagasse ash with lime by 2:2)

Appendix-F: Atterberg Limit Test Results

Table F-1 Liquid limit and plastic limit results for Pit1

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	36	33	40	30	27	26
Mass of container + Dry soil, g	30	28	31	27	25	24
Mass of container, g	20	21	20	19	19	19
Mass of water, g	6	5	9	3	2	2
Mass of dry soil, g	10	7	11	8	6	5
Water content, %	60.0	71.4	81.8	37.5	33.3	40.0
No of blows	34	25	17			
LL (%)	71.5			36.9		

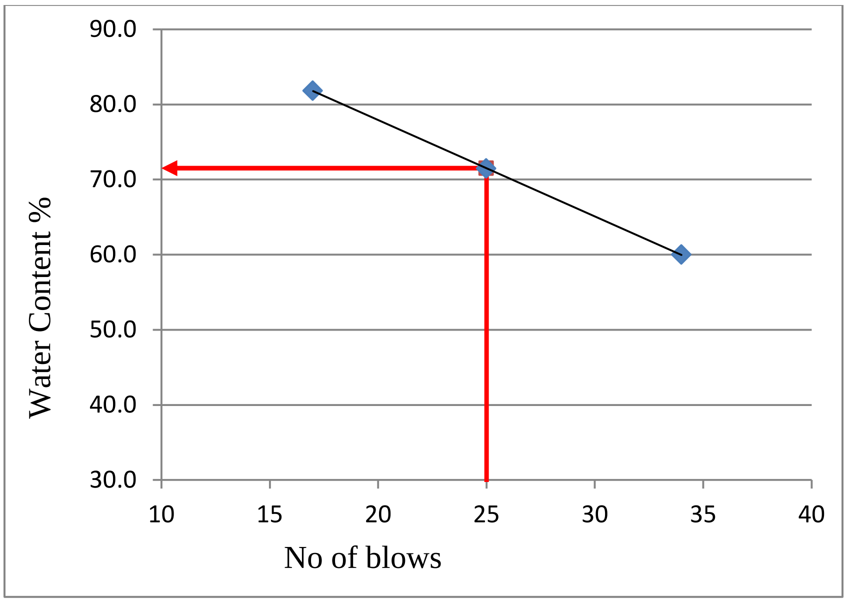

Figure F-1 Liquid limit determination for Pit1

Table F-2 Liquid limit and plastic limit results for Pit2

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	30	35	40	26	23	27
Mass of container + Dry soil, g	25	27	30	24	22	24
Mass of container, g	18	17	19	18	18	18
Mass of water, g	5	8	10	2	1	3

Mass of dry soil, g	7	10	11	6	4	6
Water content, %	71.4	80.0	90.9	33.3	25.0	50.0
No of blows	34	26	19	36.1		
LL (%)	82					

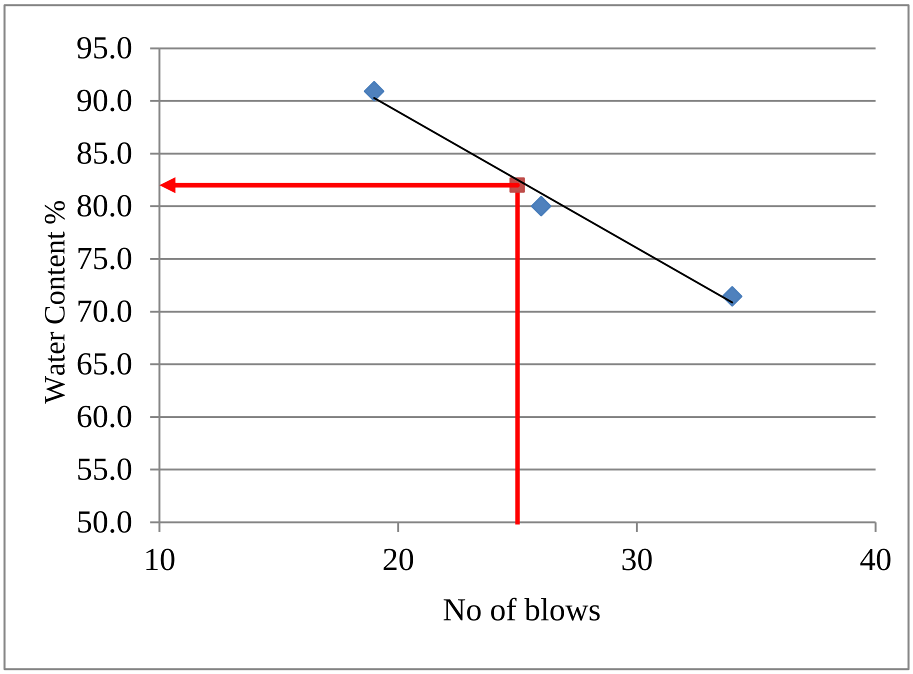

Figure F-2 Liquid limit determination for Pit2

Table F-3 Liquid limit and plastic limit results for 5%BA

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	33	32	31	25	26	26
Mass of container + Dry soil, g	27	26	25	23	24	24.1
Mass of container, g	18	18	18	18	18	18
Mass of water, g	6	6	6	2	2	1.9
Mass of dry soil, g	9	8	7	5	6	6.1
Water content, %	66.7	75.0	85.7	40.0	33.3	31.1
No of blows	32	27	19	34.8		
LL (%)	77					

Table F-4 Liquid limit and plastic limit results for 10%BA

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	30	34	33	27	24	28
Mass of container + Dry soil, g	25	27	26	25	22.5	25.2
Mass of container, g	17	17	17	19	18	18
Mass of water, g	5	7	7	2	1.5	2.8
Mass of dry soil, g	8	10	9	6	4.5	7.2
Water content, %	62.5	70	77.78	33.3	33.3	38.9
No of blows	34	27	23			
LL (%)	74			35.2		

Table F-5 Liquid limit and plastic limit results for 15%BA

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	31	31	50	30	29	27
Mass of container + Dry soil, g	26	26	37	27	26.5	24.8
Mass of container, g	17	18	19	19	19	18
Mass of water, g	5	5	13	3	2.5	2.2
Mass of dry soil, g	9	8	18	8	7.5	6.8
Water content, %	55.556	62.5	72.22	37.5	33.3	32.4
No of blows	33	28	19	34.4		

LL (%)	65.4	

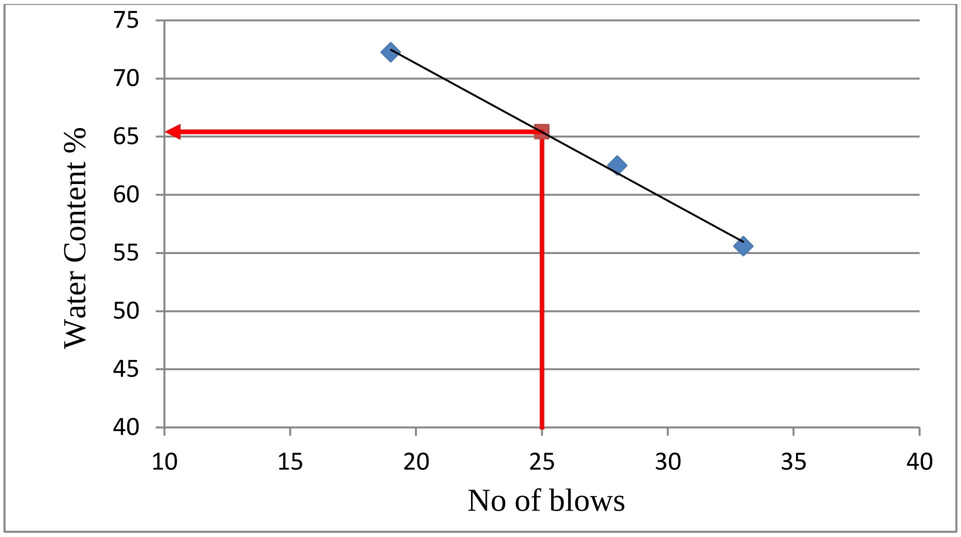

Figure F-3 Liquid limit determination for 15% BA

Table F-6 Liquid limit and plastic limit results for 20%BA

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	41	48	52	31	28	26
Mass of container + Dry soil, g	34	38	39	28	26.4	24
Mass of container, g	21	21	20	19	19	18
Mass of water, g	7	10	13	3	1.6	2
Mass of dry soil, g	13	17	19	9	7.4	6
Water content, %	53.85	58.83	68.43	33.3	21.6	33.3
No of blows	32	26	20	29.4		

LL (%)	61.6	

Table F-7 Liquid limit and plastic limit results for 15% of BA and lime by 3:1 ratio

Liquid limit				plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	44	47	44	27	29	31
Mass of container + Dry soil, g	36	37	34	25	26	27
Mass of container, g	20	21	20	18	19	18
Mass of water, g	8	10	10	2	3	4
Mass of dry soil, g	16	16	14	7	7	9
Water content, %	50	62.5	71.43	28.6	42.9	44.4
No of blows	32	25	21	38.6		
LL (%)	63					

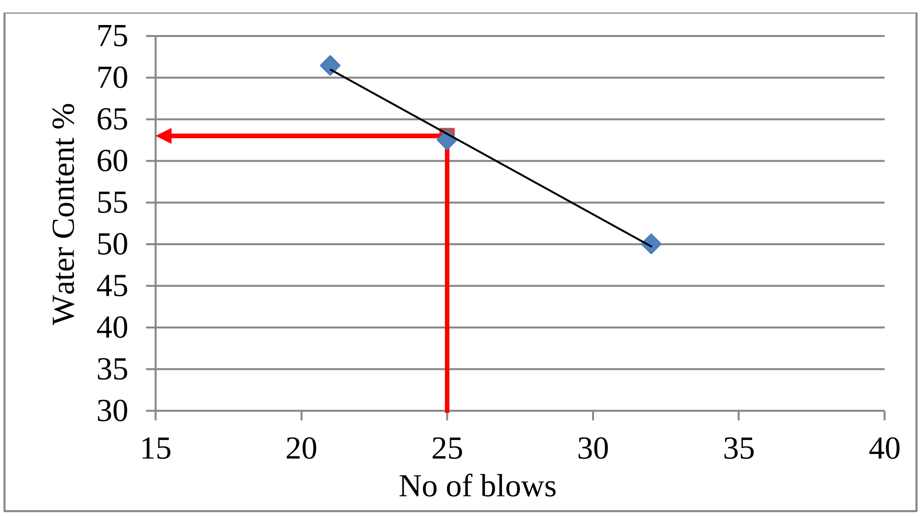

Figure F-7 Liquid limit determination for 15% of BA and lime by 3:1 ratio

Table F-8 Liquid limit and plastic limit results for 15% of BA and lime by 2:2 ratio

	Liquid limit			plastic limit		
No of trial	1	2	3	1	2	3
Mass of container + Wet soil, g	41	44	46	29	32	31
Mass of container + Dry soil, g	34.5	35.4	35.5	26	28	27.9
Mass of container, g	21	20	20	19	18	19
Mass of water, g	6.5	8.6	10.5	3	4	3.1
Mass of dry soil, g	13.5	15.4	15.5	7	10	8.9
Water content, %	48.15	55.84	67.742	42.9	40.0	34.8
No of blows	33	27	20	39.2		
LL (%)	60					

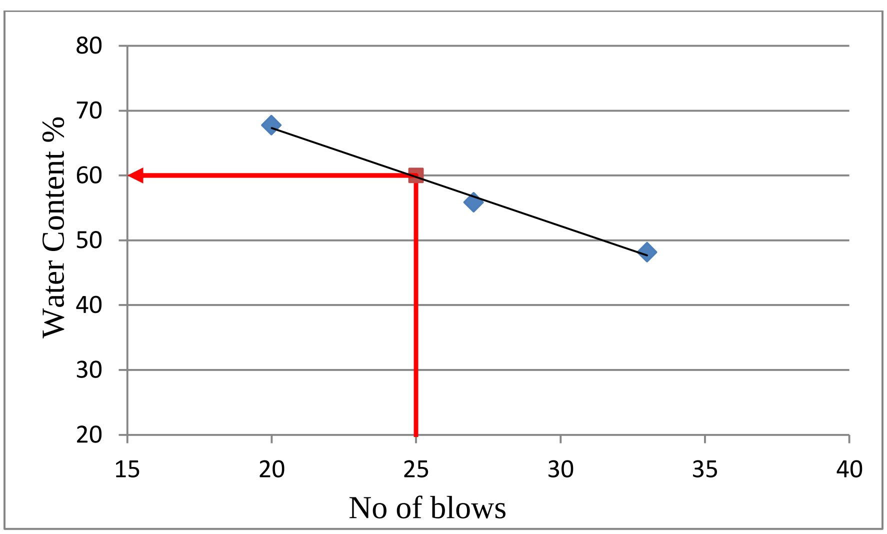

Figure F-7 Liquid limit determination for 15% of BA and lime by 2:2 ratios

Appendix: G Free swell, free swell index, and free swell ratio test result

Table G-1 Free swell, free swell index, and free swell ratio test results for Pit1, Pit2

sample name	Intial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	swell potential
Pit 1	10	21	11	91	110	1.91	High
Pit 2	10	22	10.5	110	120	2.10	High

Table G -2 Free swell, free swell index, and free swell ratio test results for different percentages of BA

% BA	Initial volume	sample volume in water(Vd)	sample volume in kerosene(Vk)	FSI	Free swell	FSR	swell potential
0	10	22	10.5	110	120	2.10	High
5	10	21.5	11	95	115	1.95	High
10	10	20	10.5	90	100	1.90	High
15	10	19	11	73	90	1.73	High
20	10	18	10.5	71	80	1.71	High

Table G -3 Free swell, free swell index, and free swell ratio test results for of BA with lime by different ratio

% of material	Initial volume	sample volume in water(Vd)	sample volume in kerosene (Vk)	FSI	Free swell	FSR	swell potential
15% BA	10	19	11	72.73	90	1.73	High
15% of BA & lime by 3:1	10	18	12	50	80	1.50	High
15% of BA & lime by 2:2	10	15.5	13	19.23	55	1.19	Low

Appendix-H: Unconfined Compressive Strength Test Results

Initial height of sample (mm) 76mm

Initial diameter of sample (mm)	36mm
Initial cross-sectional area of sample (mm2)	1134.11mm2
Initial volume of sample (mm3)	86192.74mm3

Table H-1: UCS test result sheet for Pit-1

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.16	0.002	0.211	1136.503	0.019	16.51
0.28	0.004	0.368	1138.304	0.032	28.34
0.39	0.005	0.513	1139.96	0.045	39.85
0.46	0.006	0.605	1141.016	0.055	48.34
0.53	0.007	0.697	1142.074	0.064	56.28
0.62	0.008	0.816	1143.438	0.072	63.40
0.70	0.009	0.921	1144.653	0.080	70.31
0.78	0.010	1.026	1145.87	0.091	79.28
0.86	0.011	1.132	1147.09	0.103	90.17
0.97	0.013	1.276	1148.772	0.114	99.61
1.15	0.015	1.513	1151.535	0.125	108.57
1.35	0.018	1.776	1154.62	0.135	116.49
1.64	0.022	2.158	1159.123	0.142	122.34
1.89	0.025	2.487	1163.033	0.148	127.34
2.07	0.027	2.724	1165.864	0.151	129.15
2.29	0.030	3.013	1169.344	0.153	130.56
2.47	0.033	3.250	1172.207	0.152	129.61
2.70	0.036	3.553	1175.885	0.150	127.94
2.89	0.038	3.803	1178.941	0.144031	122.17
3.02	0.040	3.974	1181.041	0.136268	115.38

Table H -2: UCS test result sheet for Pit-2(Uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.15	0.002	0.197	1134.11	0.010	8.69
0.25	0.003	0.329	1134.11	0.020	17.34
0.37	0.005	0.487	1134.11	0.030	26.41
0.46	0.006	0.605	1134.1	0.039	34.51
0.58	0.008	0.763	1134.1	0.050	44.28
0.71	0.009	0.934	1134.1	0.060	52.66
0.95	0.013	1.250	1134.1	0.078	68.38
1.15	0.015	1.513	1134.09	0.089	78.52
1.38	0.018	1.816	1134.09	0.098	86.45
1.57	0.021	2.066	1134.09	0.107	94.34
1.86	0.024	2.447	1134.09	0.118	103.85
2.16	0.028	2.842	1134.08	0.122	107.64
2.34	0.031	3.079	1134.08	0.124	109.04
2.49	0.033	3.276	1134.08	0.125	110.18
2.67	0.035	3.513	1134.07	0.124	109.05
2.91	0.038	3.829	1134.07	0.122	107.34
3.00	0.039	3.947	1134.07	0.120	105.61
3.20	0.042	4.211	1134.07	0.108	95.34

Table H -3: UCS test result sheet for Pit-2 (day 7 cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.000	0.000	0.000	1134.11	0.000	0.00
0.150	0.002	0.197	1134.108	0.023	20.21
0.230	0.003	0.303	1134.107	0.033	29.08
0.370	0.005	0.487	1134.105	0.045	39.57
0.490	0.006	0.645	1134.104	0.054	47.63
0.610	0.008	0.803	1134.102	0.063	55.36
0.750	0.010	0.987	1134.1	0.077	68.32

1.020	0.013	1.342	1134.097	0.087	76.58
1.320	0.017	1.737	1134.093	0.099	87.52
1.480	0.019	1.947	1134.091	0.107	94.18
1.670	0.022	2.197	1134.088	0.119	105.12
1.920	0.025	2.526	1134.085	0.132	116.22
2.230	0.029	2.934	1134.081	0.142	125.38
2.510	0.033	3.303	1134.077	0.153	134.64
2.870	0.038	3.776	1134.072	0.160	140.84
3.150	0.041	4.145	1134.069	0.164	144.54
3.320	0.044	4.368	1134.066	0.166	146.02
3.460	0.046	4.553	1134.064	0.132	116.12
3.530	0.046	4.645	1134.064	0.165	145.32
3.620	0.047632	4.763158	1134.062	0.162806	143.56

Table H -4 UCS test result for soil + 5% BA (Uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.18	0.002	0.237	1134.108	0.020	17.21
0.29	0.004	0.382	1134.106	0.032	28.65
0.34	0.004	0.447	1134.106	0.041	36.12
0.49	0.006	0.645	1134.104	0.051	44.95
0.61	0.008	0.803	1134.102	0.067	58.65
0.75	0.010	0.987	1134.1	0.076	67.14
0.88	0.012	1.158	1134.098	0.090	79.63
1.07	0.014	1.408	1134.096	0.104	91.65
1.34	0.018	1.763	1134.092	0.123	108.34
1.52	0.020	2.000	1134.09	0.140	123.55
1.83	0.024	2.408	1134.086	0.152	134.25
2.21	0.029	2.908	1134.081	0.161	142.26

2.46	0.032	3.237	1134.078	0.166	146.51
2.62	0.034	3.447	1134.076	0.169	149.15
2.81	0.037	3.697	1134.073	0.173	152.65
3.06	0.040	4.026	1134.07	0.174	153.82
3.28	0.043	4.316	1134.067	0.172	151.98
3.37	0.044	4.434	1134.066	0.171	150.48
3.51	0.046	4.618	1134.064	0.168	148.58

Table H-5 UCS test result for soil + 5% BA (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.16	0.002	0.211	1134.11	0.019	16.85
0.27	0.004	0.355	1134.11	0.029	25.47
0.35	0.005	0.461	1134.11	0.042	37.15
0.49	0.006	0.645	1134.1	0.052	46.26
0.62	0.008	0.816	1134.1	0.067	58.65
0.89	0.012	1.171	1134.1	0.081	71.28
1.08	0.014	1.421	1134.1	0.094	83.18
1.31	0.017	1.724	1134.09	0.107	94.15
1.48	0.019	1.947	1134.09	0.123	108.20
1.61	0.021	2.118	1134.09	0.131	115.34
1.85	0.024	2.434	1134.09	0.144	126.83
2.04	0.027	2.684	1134.08	0.152	134.25
2.29	0.030	3.013	1134.08	0.159	140.00
2.45	0.032	3.224	1134.08	0.165	145.84
2.61	0.034	3.434	1134.08	0.171	150.48
2.86	0.038	3.763	1134.07	0.175	154.36
2.97	0.039	3.908	1134.07	0.180	158.65
3.11	0.041	4.092	1134.07	0.182	160.19
3.30	0.043	4.342	1134.07	0.184	162.05
3.48	0.046	4.579	1134.06	0.185	163.14
3.65	0.048	4.803	1134.06	0.183	161.41
3.86	0.051	5.079	1134.06	0.181	159.71

Table H-6 UCS test result for soil + 5% BA (14 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [kPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.21	0.003	0.276	1134.11	0.025	22.35
0.36	0.005	0.474	1134.11	0.034	30.14
0.47	0.006	0.618	1134.1	0.044	38.50
0.58	0.008	0.763	1134.1	0.050	44.26
0.71	0.009	0.934	1134.1	0.056	49.52
0.92	0.012	1.211	1134.1	0.074	65.15
1.06	0.014	1.395	1134.1	0.080	70.84
1.28	0.017	1.684	1134.09	0.085	75.16
1.48	0.019	1.947	1134.09	0.093	82.25
1.69	0.022	2.224	1134.09	0.102	90.18
1.91	0.025	2.513	1134.08	0.113	99.54
2.08	0.027	2.737	1134.08	0.124	109.57
2.31	0.030	3.039	1134.08	0.135	118.65
2.49	0.033	3.276	1134.08	0.147	129.65
2.68	0.035	3.526	1134.07	0.159	140.60
2.95	0.039	3.882	1134.07	0.171	150.84
3.12	0.041	4.105	1134.07	0.180	158.50
3.32	0.044	4.368	1134.07	0.186	164.00
3.49	0.046	4.592	1134.06	0.194	170.64
3.67	0.048	4.829	1134.06	0.199	175.43
3.87	0.051	5.092	1134.06	0.202	178.31
4.05	0.053	5.329	1134.06	0.200	176.23
4.28	0.056	5.632	1134.05	0.197	174.15

Table H-7 UCS test result for soil + 5% BA (28 day cured)

Displacement [mm]	Unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [kPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.19	0.003	0.250	1134.11	0.020	17.54
0.27	0.004	0.355	1134.11	0.034	29.68

0.36	0.005	0.474	1134.11	0.044	38.54
0.49	0.006	0.645	1134.1	0.052	45.61
0.61	0.008	0.803	1134.1	0.064	56.36
0.85	0.011	1.118	1134.1	0.073	64.15
1.03	0.014	1.355	1134.1	0.086	75.65
1.21	0.016	1.592	1134.09	0.094	82.65
1.42	0.019	1.868	1134.09	0.105	92.25
1.60	0.021	2.105	1134.09	0.123	108.16
1.86	0.024	2.447	1134.09	0.135	119.45
2.04	0.027	2.684	1134.08	0.148	130.41
2.23	0.029	2.934	1134.08	0.161	141.64
2.47	0.033	3.250	1134.08	0.171	150.55
2.65	0.035	3.487	1134.08	0.183	161.25
2.88	0.038	3.789	1134.07	0.193	170.51
3.06	0.040	4.026	1134.07	0.205	180.58
3.28	0.043	4.316	1134.07	0.215	189.65
3.45	0.045	4.542	1134.06	0.221	195.18
3.69	0.049	4.855	1134.06	0.228	200.98
3.88	0.051	5.105	1134.06	0.231	203.41
4.02	0.053	5.289	1134.06	0.232	204.75
4.26	0.056	5.605	1134.05	0.231	203.52
4.47	0.059	5.882	1134.05	0.229	201.64
4.68	0.062	6.158	1134.05	0.225	198.25

Table H-8 UCS test result for soil + 5% BA (60 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.11	0.000	0.00
0.21	0.003	0.276	1134.11	0.028	24.46
0.42	0.006	0.553	1134.1	0.041	35.95
0.65	0.009	0.855	1134.1	0.055	48.71
0.81	0.011	1.066	1134.1	0.068	59.64
0.97	0.013	1.276	1134.1	0.076	67.43
1.04	0.014	1.368	1134.1	0.086	75.54
1.27	0.017	1.671	1134.09	0.101	89.36
1.39	0.018	1.829	1134.09	0.111	98.14

1.48	0.019	1.947	1134.09	0.121	107.00
1.63	0.021	2.145	1134.09	0.132	116.65
1.75	0.023	2.303	1134.09	0.138	121.25
1.93	0.025	2.539	1134.08	0.146	128.62
2.05	0.027	2.697	1134.08	0.152	134.29
2.24	0.029	2.947	1134.08	0.162	142.74
2.37	0.031	3.118	1134.08	0.171	150.38
2.48	0.033	3.263	1134.08	0.181	159.64
2.56	0.034	3.368	1134.08	0.188	165.85
2.74	0.036	3.605	1134.07	0.203	178.65
2.94	0.039	3.868	1134.07	0.209	184.35
3.08	0.041	4.053	1134.07	0.216	190.25
3.26	0.043	4.289	1134.07	0.220	193.65
3.41	0.045	4.487	1134.07	0.222	195.48
3.62	0.048	4.763	1134.06	0.220	194.32
3.84	0.051	5.053	1134.06	0.218	192.61

Table H-9 UCS test result for soil + 10% BA (uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.20	0.003	0.263	1134.107	0.010	8.51
0.35	0.005	0.461	1134.105	0.019	16.46
0.48	0.006	0.632	1134.104	0.028	24.76
0.56	0.007	0.737	1134.103	0.036	31.54
0.70	0.009	0.921	1134.101	0.044	38.45
0.86	0.011	1.132	1134.099	0.053	46.38
0.98	0.013	1.289	1134.097	0.063	55.64
1.04	0.014	1.368	1134.096	0.072	63.17
1.21	0.016	1.592	1134.094	0.080	70.15
1.37	0.018	1.803	1134.092	0.089	78.25
1.45	0.019	1.908	1134.091	0.098	86.25
1.58	0.021	2.079	1134.089	0.106	93.46

1.65	0.022	2.171	1134.088	0.118	103.84
1.79	0.024	2.355	1134.086	0.130	114.34
2.00	0.026	2.632	1134.084	0.142	125.21
2.18	0.029	2.868	1134.081	0.153	134.75
2.37	0.031	3.118	1134.079	0.165	145.35
2.49	0.033	3.276	1134.077	0.173	152.61
2.62	0.034	3.447	1134.076	0.179	157.67
2.86	0.038	3.763	1134.072	0.182	160.47
2.97	0.039	3.908	1134.071	0.183	161.35
3.07	0.040	4.039	1134.070	0.182	160.08
3.26	0.043	4.289	1134.067	0.179	158.15
3.48	0.046	4.579	1134.064	0.176	155.34

Table H-10- UCS test result for soil + 10% BA (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.17	0.002	0.224	1134.108	0.017	15.31
0.26	0.003	0.342	1134.107	0.031	27.14
0.39	0.005	0.513	1134.105	0.041	36.46
0.45	0.006	0.592	1134.104	0.049	42.78
0.61	0.008	0.803	1134.102	0.063	55.13
0.78	0.010	1.026	1134.100	0.078	68.62
0.90	0.012	1.184	1134.098	0.085	75.37
1.05	0.014	1.382	1134.096	0.095	83.46
1.25	0.016	1.645	1134.094	0.105	92.42
1.36	0.018	1.789	1134.092	0.115	101.65
1.47	0.019	1.934	1134.091	0.125	110.34
1.62	0.021	2.132	1134.089	0.138	121.40
1.86	0.024	2.447	1134.086	0.148	130.82

2.06	0.027	2.711	1134.083	0.157	138.48
2.20	0.029	2.895	1134.081	0.165	145.64
2.41	0.032	3.171	1134.078	0.172	151.74
2.56	0.034	3.368	1134.076	0.179	157.65
2.68	0.035	3.526	1134.075	0.184	162.25
2.85	0.038	3.750	1134.073	0.191	168.37
3.02	0.040	3.974	1134.070	0.193	170.54
3.24	0.043	4.263	1134.067	0.196	173.15
3.41	0.045	4.487	1134.065	0.195	172.32
3.67	0.048	4.829	1134.062	0.192	169.74

Table H-11- UCS test result for soil + 10% BA (14 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.16	0.002	0.211	1134.108	0.023	20.12
0.27	0.004	0.355	1134.106	0.033	29.46
0.39	0.005	0.513	1134.105	0.041	36.46
0.48	0.006	0.632	1134.104	0.054	47.52
0.60	0.008	0.789	1134.102	0.065	57.52
0.79	0.010	1.039	1134.100	0.072	63.48
0.92	0.012	1.211	1134.098	0.080	70.85
1.04	0.014	1.368	1134.096	0.088	77.87
1.24	0.016	1.632	1134.094	0.095	83.45
1.39	0.018	1.829	1134.092	0.105	92.46
1.50	0.020	1.974	1134.090	0.116	102.48

1.67	0.022	2.197	1134.088	0.131	115.64
1.91	0.025	2.513	1134.085	0.144	127.39
2.08	0.027	2.737	1134.083	0.157	138.00
2.22	0.029	2.921	1134.081	0.169	148.64
2.38	0.031	3.132	1134.079	0.179	157.49
2.52	0.033	3.316	1134.077	0.188	165.46
2.69	0.035	3.539	1134.075	0.196	172.81
2.88	0.038	3.789	1134.072	0.201	177.52
3.04	0.040	4.000	1134.070	0.207	182.94
3.19	0.042	4.197	1134.068	0.211	185.94
3.31	0.044	4.355	1134.066	0.211	186.16
3.48	0.046	4.579	1134.064	0.210	185.47
3.67	0.048	4.829	1134.062	0.207	182.25
3.82	0.050	5.026	1134.060	0.203	178.62

Table H-12- UCS test result for soil + 10% BA (28 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.17	0.002	0.224	1134.108	0.019	16.84
0.27	0.004	0.355	1134.106	0.032	28.64
0.39	0.005	0.513	1134.105	0.041	35.76
0.48	0.006	0.632	1134.104	0.052	46.21
0.61	0.008	0.803	1134.102	0.063	55.26
0.82	0.011	1.079	1134.099	0.073	64.57
1.01	0.013	1.329	1134.097	0.086	75.61
1.16	0.015	1.526	1134.095	0.095	83.47

1.29	0.017	1.697	1134.093	0.107	94.45
1.38	0.018	1.816	1134.092	0.115	101.80
1.52	0.020	2.000	1134.090	0.128	112.47
1.71	0.023	2.250	1134.088	0.138	122.00
1.85	0.024	2.434	1134.086	0.152	134.00
2.05	0.027	2.697	1134.083	0.162	143.00
2.18	0.029	2.868	1134.081	0.172	152.00
2.29	0.030	3.013	1134.080	0.184	162.00
2.43	0.032	3.197	1134.078	0.194	170.84
2.60	0.034	3.421	1134.076	0.203	178.62
2.77	0.036	3.645	1134.074	0.210	185.25
2.92	0.038	3.842	1134.072	0.220	194.14
3.08	0.041	4.053	1134.069	0.232	204.68
3.17	0.042	4.171	1134.068	0.235	207.61
3.31	0.044	4.355	1134.066	0.233	205.34
3.54	0.047	4.658	1134.063	0.227	200.17

Table H-13- UCS test result for soil + 10% BA (60 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.19	0.003	0.250	1134.108	0.025	22.47
0.27	0.004	0.355	1134.106	0.034	30.21
0.36	0.005	0.474	1134.105	0.044	38.46
0.49	0.006	0.645	1134.104	0.054	47.23
0.61	0.008	0.803	1134.102	0.064	56.00
0.82	0.011	1.079	1134.099	0.080	70.65
1.03	0.014	1.355	1134.096	0.096	84.65

1.19	0.016	1.566	1134.094	0.103	91.25
1.35	0.018	1.776	1134.092	0.116	102.61
1.52	0.020	2.000	1134.090	0.127	111.76
1.68	0.022	2.211	1134.088	0.137	120.86
1.88	0.025	2.474	1134.085	0.146	128.58
2.01	0.026	2.645	1134.084	0.159	140.13
2.21	0.029	2.908	1134.081	0.167	147.65
2.36	0.031	3.105	1134.079	0.182	160.27
2.53	0.033	3.329	1134.077	0.191	168.02
2.72	0.036	3.579	1134.074	0.199	175.66
2.95	0.039	3.882	1134.071	0.207	182.57
3.11	0.041	4.092	1134.069	0.218	192.37
3.26	0.043	4.289	1134.067	0.225	198.27
3.38	0.044	4.447	1134.066	0.228	200.84
3.51	0.046	4.618	1134.064	0.224	197.34
3.64	0.048	4.789	1134.062	0.218	192.18

Table H-14- UCS test result for soil + 15% BA (uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.16	0.002	0.211	1134.108	0.040	35.40
0.27	0.004	0.355	1134.106	0.060	53.03
0.39	0.005	0.513	1134.105	0.076	67.08
0.48	0.006	0.632	1134.104	0.087	76.85
0.61	0.008	0.803	1134.102	0.096	84.36
0.78	0.010	1.026	1134.100	0.105	92.59
0.92	0.012	1.211	1134.098	0.115	101.28

1.08	0.014	1.421	1134.096	0.126	110.83
1.19	0.016	1.566	1134.094	0.135	119.06
1.30	0.017	1.711	1134.093	0.144	127.38
1.46	0.019	1.921	1134.091	0.150	132.67
1.59	0.021	2.092	1134.089	0.159	140.12
1.73	0.023	2.276	1134.087	0.166	146.37
1.97	0.026	2.592	1134.084	0.173	152.61
2.10	0.028	2.763	1134.082	0.182	160.34
2.24	0.029	2.947	1134.081	0.188	166.00
2.45	0.032	3.224	1134.078	0.193	170.21
2.71	0.036	3.566	1134.074	0.199	175.84
2.84	0.037	3.737	1134.073	0.205	180.34
2.96	0.039	3.895	1134.071	0.210	185.32
3.12	0.041	4.105	1134.069	0.209	184.01
3.37	0.044	4.434	1134.066	0.208	182.98
3.52	0.046	4.632	1134.064	0.204	179.65
3.74	0.049	4.921	1134.061	0.200	176.31
3.92	0.052	5.158	1134.058	0.195	171.61
4.04	0.053	5.316	1134.057	0.187	165.25
4.26	0.056	5.605	1134.054	0.182	160.64
4.45	0.059	5.855	1134.051	0.177	156.36

Table H-15- UCS test result for soil + 15% BA (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.18	0.002	0.237	1134.108	0.038	33.12
0.27	0.004	0.355	1134.106	0.057	50.31
0.35	0.005	0.461	1134.105	0.067	58.65
0.49	0.006	0.645	1134.104	0.078	69.14
0.57	0.008	0.750	1134.103	0.091	80.36
0.71	0.009	0.934	1134.101	0.105	92.51
0.95	0.013	1.250	1134.098	0.119	104.61
1.16	0.015	1.526	1134.095	0.132	116.25
1.30	0.017	1.711	1134.093	0.137	120.48
1.58	0.021	2.079	1134.089	0.148	130.84
1.76	0.023	2.316	1134.087	0.157	138.31
1.88	0.025	2.474	1134.085	0.168	147.94
2.04	0.027	2.684	1134.083	0.175	154.64
2.28	0.030	3.000	1134.080	0.182	160.74
2.45	0.032	3.224	1134.078	0.188	165.48

2.66	0.035	3.500	1134.075	0.194	170.85
2.84	0.037	3.737	1134.073	0.201	177.15
3.02	0.040	3.974	1134.070	0.208	183.15
3.27	0.043	4.303	1134.067	0.214	188.47
3.39	0.045	4.461	1134.065	0.218	192.23
3.56	0.047	4.684	1134.063	0.222	195.82
3.68	0.048	4.842	1134.062	0.217	191.35
3.82	0.050	5.026	1134.060	0.212	186.71

Table H-16- UCS test result for soil + 15% BA (14 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.16	0.002	0.211	1134.108	0.047	41.30
0.28	0.004	0.368	1134.106	0.076	67.25
0.41	0.005	0.539	1134.105	0.092	81.36
0.65	0.009	0.855	1134.101	0.105	92.18
0.84	0.011	1.105	1134.099	0.119	104.98
1.03	0.014	1.355	1134.096	0.131	115.64
1.26	0.017	1.658	1134.093	0.143	126.42
1.39	0.018	1.829	1134.092	0.156	137.14
1.51	0.020	1.987	1134.090	0.168	148.27
1.72	0.023	2.263	1134.087	0.181	159.17
1.91	0.025	2.513	1134.085	0.193	170.14
2.12	0.028	2.789	1134.082	0.205	180.40
2.34	0.031	3.079	1134.079	0.214	188.34
2.56	0.034	3.368	1134.076	0.223	196.45
2.84	0.037	3.737	1134.073	0.229	202.10
3.04	0.040	4.000	1134.070	0.236	208.00
3.40	0.045	4.474	1134.065	0.240	211.32
3.60	0.047	4.737	1134.063	0.237	209.00
3.80	0.050	5.000	1134.060	0.235	207.36

4.00	0.053	5.263	1134.057	0.233	205.58

Table H-17- UCS test result for soil + 15% BA (28 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.23	0.003	0.303	1134.107	0.047	41.30
0.37	0.005	0.487	1134.105	0.085	75.31
0.49	0.006	0.645	1134.104	0.113	100.03
0.64	0.008	0.842	1134.102	0.137	120.50
0.86	0.011	1.132	1134.099	0.156	137.53
1.07	0.014	1.408	1134.096	0.168	148.36
1.26	0.017	1.658	1134.093	0.177	156.24
1.41	0.019	1.855	1134.091	0.190	167.13
1.65	0.022	2.171	1134.088	0.201	177.61
1.84	0.024	2.421	1134.086	0.214	188.73
2.10	0.028	2.763	1134.082	0.225	198.36
2.35	0.031	3.092	1134.079	0.237	208.65
2.56	0.034	3.368	1134.076	0.245	216.28
2.83	0.037	3.724	1134.073	0.254	224.12
3.13	0.041	4.118	1134.069	0.262	230.61
3.34	0.044	4.395	1134.066	0.265	233.63
3.58	0.047	4.711	1134.063	0.266	234.25
3.84	0.051	5.053	1134.059	0.264	232.41
4.12	0.054	5.421	1134.056	0.261	230.47
4.36	0.057	5.737	1134.053	0.258	227.61

Table H-18- UCS test result for soil + 15% BA (60 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.18	0.002	0.237	1134.108	0.047	41.30
0.27	0.004	0.355	1134.106	0.061	54.14
0.36	0.005	0.474	1134.105	0.073	64.08
0.51	0.007	0.671	1134.103	0.086	75.61
0.72	0.009	0.947	1134.101	0.105	92.45

0.81	0.011	1.066	1134.099	0.114	100.51
1.02	0.013	1.342	1134.097	0.139	122.32
1.23	0.016	1.618	1134.094	0.153	135.16
1.38	0.018	1.816	1134.092	0.168	148.57
1.52	0.020	2.000	1134.090	0.182	160.84
1.72	0.023	2.263	1134.087	0.194	170.65
1.88	0.025	2.474	1134.085	0.206	181.65
2.08	0.027	2.737	1134.083	0.216	190.57
2.29	0.030	3.013	1134.080	0.232	204.72
2.41	0.032	3.171	1134.078	0.239	210.48
2.65	0.035	3.487	1134.075	0.248	218.61
2.83	0.037	3.724	1134.073	0.250	220.18
3.04	0.040	4.000	1134.070	0.257	226.84
3.28	0.043	4.316	1134.067	0.264	232.74
3.41	0.045	4.487	1134.065	0.268	236.61
3.65	0.048	4.803	1134.062	0.271	239.08
3.82	0.050	5.026	1134.060	0.274	241.52
4.11	0.054	5.408	1134.056	0.273	240.58
4.26	0.056	5.605	1134.054	0.271	238.65
4.41	0.058	5.803	1134.052	0.266	234.17

Table H-19- UCS test result for soil + 15% BA (90 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.22	0.003	0.289	1134.107	0.047	41.30
0.38	0.005	0.500	1134.105	0.085	75.31
0.51	0.007	0.671	1134.103	0.113	100.03
0.73	0.010	0.961	1134.100	0.146	128.65
0.79	0.010	1.039	1134.100	0.156	137.53
0.97	0.013	1.276	1134.097	0.177	155.68
1.12	0.015	1.474	1134.095	0.188	165.73
1.22	0.016	1.605	1134.094	0.197	173.64
1.34	0.018	1.763	1134.092	0.201	177.61
1.61	0.021	2.118	1134.089	0.214	188.73
1.85	0.024	2.434	1134.086	0.225	198.36
1.98	0.026	2.605	1134.084	0.232	204.64
2.15	0.028	2.829	1134.082	0.239	210.91
2.26	0.030	2.974	1134.080	0.244	215.37

2.40	0.032	3.158	1134.078	0.251	221.75
2.51	0.033	3.303	1134.077	0.254	224.31
2.83	0.037	3.724	1134.073	0.264	232.46
3.07	0.040	4.039	1134.070	0.269	237.45
3.24	0.043	4.263	1134.067	0.272	240.15
3.37	0.044	4.434	1134.066	0.276	243.61
3.49	0.046	4.592	1134.064	0.279	245.84
3.56	0.047	4.684	1134.063	0.278	244.74
3.68	0.048	4.842	1134.062	0.275	242.16
3.85	0.051	5.066	1134.059	0.271	238.94

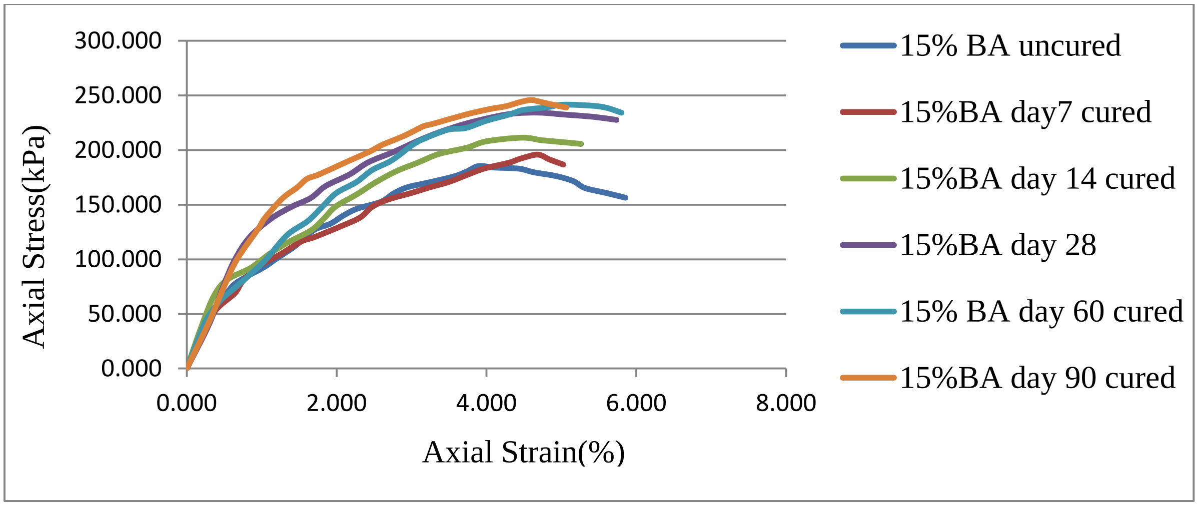

Figure H -1 UCS stress strain cure for 15% for uncured& 7, 14,28,60,90 day cured

Table H-20- UCS test result for soil + 20% BA (uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.0	0.000	0.000	1134.110	0.000	0.00
0.2	0.003	0.263	1134.107	0.034	30.01
0.4	0.005	0.526	1134.105	0.051	44.55
0.6	0.008	0.789	1134.102	0.066	57.78
0.8	0.011	1.053	1134.099	0.073	64.36
1.0	0.013	1.316	1134.097	0.085	75.06
1.2	0.016	1.579	1134.094	0.098	86.81
1.4	0.018	1.842	1134.092	0.108	95.34
1.6	0.021	2.105	1134.089	0.121	106.28

1.8	0.024	2.368	1134.086	0.129	113.75
2.0	0.026	2.632	1134.084	0.138	121.72
2.2	0.029	2.895	1134.081	0.147	129.37
2.4	0.032	3.158	1134.078	0.156	137.84
2.6	0.034	3.421	1134.076	0.167	146.86
2.8	0.037	3.684	1134.073	0.176	155.46
3.0	0.039	3.947	1134.071	0.186	164.09
3.2	0.042	4.211	1134.068	0.190	167.84
3.4	0.045	4.474	1134.065	0.187	165.23
3.6	0.047	4.737	1134.063	0.182	160.08

Table H-21- UCS test result for soil + 20% BA (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.0	0.000	0.000	1134.110	0.000	0.00
0.2	0.003	0.263	1134.107	0.041	36.00
0.4	0.005	0.526	1134.105	0.068	60.01
0.6	0.008	0.789	1134.102	0.087	76.32
0.8	0.011	1.053	1134.099	0.105	92.30
1.0	0.013	1.316	1134.097	0.120	105.63
1.2	0.016	1.579	1134.094	0.131	115.68
1.4	0.018	1.842	1134.092	0.143	125.86
1.6	0.021	2.105	1134.089	0.150	132.62
1.8	0.024	2.368	1134.086	0.162	142.69
2.0	0.026	2.632	1134.084	0.170	150.20
2.2	0.029	2.895	1134.081	0.184	162.41
2.4	0.032	3.158	1134.078	0.191	168.45
2.6	0.034	3.421	1134.076	0.198	174.87
2.8	0.037	3.684	1134.073	0.202	178.12
3.0	0.039	3.947	1134.071	0.206	181.32
3.2	0.042	4.211	1134.068	0.209	184.56
3.4	0.045	4.474	1134.065	0.213	187.64
3.6	0.047	4.737	1134.063	0.208	183.25

Table H-22- UCS test result for soil + 15% of BA with lime by 3:1 (uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00

0.23	0.003	0.303	1134.107	0.047	41.43
0.35	0.005	0.461	1134.105	0.069	61.25
0.49	0.006	0.645	1134.104	0.099	87.59
0.60	0.008	0.789	1134.102	0.117	102.94
0.71	0.009	0.934	1134.101	0.133	117.54
0.92	0.012	1.211	1134.098	0.156	137.85
1.06	0.014	1.395	1134.096	0.168	148.53
1.14	0.015	1.500	1134.095	0.175	154.36
1.28	0.017	1.684	1134.093	0.187	165.24
1.50	0.020	1.974	1134.090	0.206	181.57
1.72	0.023	2.263	1134.087	0.221	194.71
1.95	0.026	2.566	1134.084	0.232	204.37
2.02	0.027	2.658	1134.083	0.234	206.34
2.32	0.031	3.053	1134.079	0.242	213.25
2.41	0.032	3.171	1134.078	0.247	218.16
2.59	0.034	3.408	1134.076	0.252	221.84
2.68	0.035	3.526	1134.075	0.255	225.24
2.81	0.037	3.697	1134.073	0.259	228.23
3.05	0.040	4.013	1134.070	0.261	230.07
3.26	0.043	4.289	1134.067	0.260	229.31
3.48	0.046	4.579	1134.064	0.258	227.14

Table H-23- UCS test result for soil + 15% of BA with lime by 3:1 (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.16	0.002	0.211	1134.108	0.040	35.64
0.34	0.004	0.447	1134.106	0.080	70.61
0.48	0.006	0.632	1134.104	0.101	89.45
0.60	0.008	0.789	1134.102	0.121	107.13
0.71	0.009	0.934	1134.101	0.132	116.24
0.82	0.011	1.079	1134.099	0.144	127.19
0.90	0.012	1.184	1134.098	0.155	136.81
1.06	0.014	1.395	1134.096	0.171	150.94
1.28	0.017	1.684	1134.093	0.200	176.34
1.52	0.020	2.000	1134.090	0.222	195.62
1.66	0.022	2.184	1134.088	0.245	216.34
1.78	0.023	2.342	1134.087	0.259	228.76
1.85	0.024	2.434	1134.086	0.270	238.41
2.04	0.027	2.684	1134.083	0.284	250.32
2.24	0.029	2.947	1134.081	0.296	261.27

2.39	0.031	3.145	1134.079	0.308	271.34
2.51	0.033	3.303	1134.077	0.318	280.46
2.67	0.035	3.513	1134.075	0.327	288.47
2.86	0.038	3.763	1134.072	0.336	296.14
3.05	0.040	4.013	1134.070	0.343	302.79
3.18	0.042	4.184	1134.068	0.348	306.46
3.32	0.044	4.368	1134.066	0.350	308.26
3.46	0.046	4.553	1134.064	0.348	307.28
3.59	0.047	4.724	1134.063	0.346	305.16
3.66	0.048	4.816	1134.062	0.342	301.41

Table H-24- UCS test result for soil + 15% of BA with lime by 3:1 (14 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Aremm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.21	0.003	0.276	1134.107	0.064	56.34
0.40	0.005	0.526	1134.105	0.104	91.53
0.59	0.008	0.776	1134.102	0.137	120.54
0.78	0.010	1.026	1134.100	0.161	142.36
1.02	0.013	1.342	1134.097	0.188	165.34
1.22	0.016	1.605	1134.094	0.217	191.45
1.41	0.019	1.855	1134.091	0.239	210.87
1.62	0.021	2.132	1134.089	0.261	230.43
1.80	0.024	2.368	1134.086	0.285	251.67
1.96	0.026	2.579	1134.084	0.313	275.69
2.23	0.029	2.934	1134.081	0.347	305.64
2.35	0.031	3.092	1134.079	0.364	320.86
2.59	0.034	3.408	1134.076	0.392	345.57
2.81	0.037	3.697	1134.073	0.408	360.00
3.02	0.040	3.974	1134.070	0.424	373.84
3.22	0.042	4.237	1134.068	0.433	381.75
3.43	0.045	4.513	1134.065	0.436	384.21
3.61	0.048	4.750	1134.063	0.431	380.36
3.83	0.050	5.039	1134.060	0.429	378.45
4.04	0.053	5.316	1134.057	0.423	373.15

Table H-25- UCS test result for soil + 15% of BA with lime by 3:1 (28 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.26	0.003	0.342	1134.107	0.071	62.37
0.38	0.005	0.500	1134.105	0.114	100.52
0.52	0.007	0.684	1134.103	0.157	138.46
0.75	0.010	0.987	1134.100	0.215	189.64
0.97	0.013	1.276	1134.097	0.269	237.46
1.05	0.014	1.382	1134.096	0.291	256.54
1.22	0.016	1.605	1134.094	0.324	285.46
1.39	0.018	1.829	1134.092	0.354	312.38
1.48	0.019	1.947	1134.091	0.373	328.74
1.60	0.021	2.105	1134.089	0.393	346.23
1.82	0.024	2.395	1134.086	0.426	375.27
1.87	0.025	2.461	1134.085	0.435	383.46
2.06	0.027	2.711	1134.083	0.452	398.47
2.24	0.029	2.947	1134.081	0.478	421.56
2.41	0.032	3.171	1134.078	0.502	442.84
2.65	0.035	3.487	1134.075	0.521	459.37
2.89	0.038	3.803	1134.072	0.528	465.18
3.10	0.041	4.079	1134.069	0.534	471.00
3.28	0.043	4.316	1134.067	0.540	476.13
3.41	0.045	4.487	1134.065	0.547	482.25
3.58	0.047	4.711	1134.063	0.553	487.24
3.67	0.048	4.829	1134.062	0.558	491.75
3.79	0.050	4.987	1134.060	0.559	492.84
3.95	0.052	5.197	1134.058	0.561	494.56
4.12	0.054	5.421	1134.056	0.559	493.25
4.28	0.056	5.632	1134.054	0.557	491.5
4.45	0.059	5.855	1134.051	0.554	488.54

Table H-26- UCS test result for soil + 15% of BA with lime by 3:1 (60 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.25	0.003	0.329	1134.107	0.097	85.43
0.37	0.005	0.487	1134.105	0.137	120.41
0.52	0.007	0.684	1134.103	0.191	168.45
0.63	0.008	0.829	1134.102	0.231	203.67
0.72	0.009	0.947	1134.101	0.267	235.46
0.85	0.011	1.118	1134.099	0.322	284.35
0.98	0.013	1.289	1134.097	0.377	332.46
1.09	0.014	1.434	1134.096	0.397	350.28
1.22	0.016	1.605	1134.094	0.421	371.46
1.32	0.017	1.737	1134.093	0.449	396.17
1.42	0.019	1.868	1134.091	0.475	418.64
1.65	0.022	2.171	1134.088	0.509	449.21
1.73	0.023	2.276	1134.087	0.528	465.28
1.97	0.026	2.592	1134.084	0.577	508.61
2.15	0.028	2.829	1134.082	0.604	532.74
2.32	0.031	3.053	1134.079	0.625	551.42
2.49	0.033	3.276	1134.077	0.653	575.63
2.62	0.034	3.447	1134.076	0.678	597.41
2.85	0.038	3.750	1134.073	0.705	621.43
2.98	0.039	3.921	1134.071	0.720	635.15
3.12	0.041	4.105	1134.069	0.739	651.48
3.30	0.043	4.342	1134.067	0.755	665.47
3.49	0.046	4.592	1134.064	0.769	678.17
3.58	0.047	4.711	1134.063	0.784	691.14
3.73	0.049	4.908	1134.061	0.796	702.34
3.89	0.051	5.118	1134.059	0.803	708.42
4.13	0.054	5.434	1134.056	0.808	712.58
4.29	0.056	5.645	1134.054	0.811	715.4
4.42	0.058	5.816	1134.052	0.809	713.42
4.58	0.060	6.026	1134.050	0.806	710.95
4.65	0.061	6.118	1134.049	0.800	705.64
4.86	0.064	6.395	1134.046	0.793	699.14

Table H-27- UCS test result for soil + 15% of BA with lime by 3:1 (90 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.26	0.003	0.342	1134.107	0.099	87.61
0.39	0.005	0.513	1134.105	0.137	120.41

0.50	0.007	0.658	1134.103	0.191	168.45
0.63	0.008	0.829	1134.102	0.231	203.67
0.72	0.009	0.947	1134.101	0.267	235.46
0.79	0.010	1.039	1134.100	0.296	260.89
0.98	0.013	1.289	1134.097	0.352	310.63
1.09	0.014	1.434	1134.096	0.379	334.41
1.22	0.016	1.605	1134.094	0.407	358.91
1.28	0.017	1.684	1134.093	0.425	374.90
1.42	0.019	1.868	1134.091	0.460	405.97
1.52	0.020	2.000	1134.090	0.491	432.76
1.63	0.021	2.145	1134.089	0.519	457.42
1.89	0.025	2.487	1134.085	0.569	501.76
2.15	0.028	2.829	1134.082	0.604	532.41
2.32	0.031	3.053	1134.079	0.629	554.27
2.49	0.033	3.276	1134.077	0.654	576.84
2.62	0.034	3.447	1134.076	0.675	595.14
2.85	0.038	3.750	1134.073	0.710	626.43
2.98	0.039	3.921	1134.071	0.735	648.27
3.12	0.041	4.105	1134.069	0.751	661.84
3.30	0.043	4.342	1134.067	0.766	675.42
3.49	0.046	4.592	1134.064	0.792	698.14
3.58	0.047	4.711	1134.063	0.806	710.28
3.73	0.049	4.908	1134.061	0.814	717.61
3.89	0.051	5.118	1134.059	0.820	722.84
4.13	0.054	5.434	1134.056	0.825	727.23
4.29	0.056	5.645	1134.054	0.829	730.84
4.42	0.058	5.816	1134.052	0.826	728.51
4.58	0.060	6.026	1134.050	0.821	723.69
4.65	0.061	6.118	1134.049	0.815	718.41
4.86	0.064	6.395	1134.046	0.806	710.84

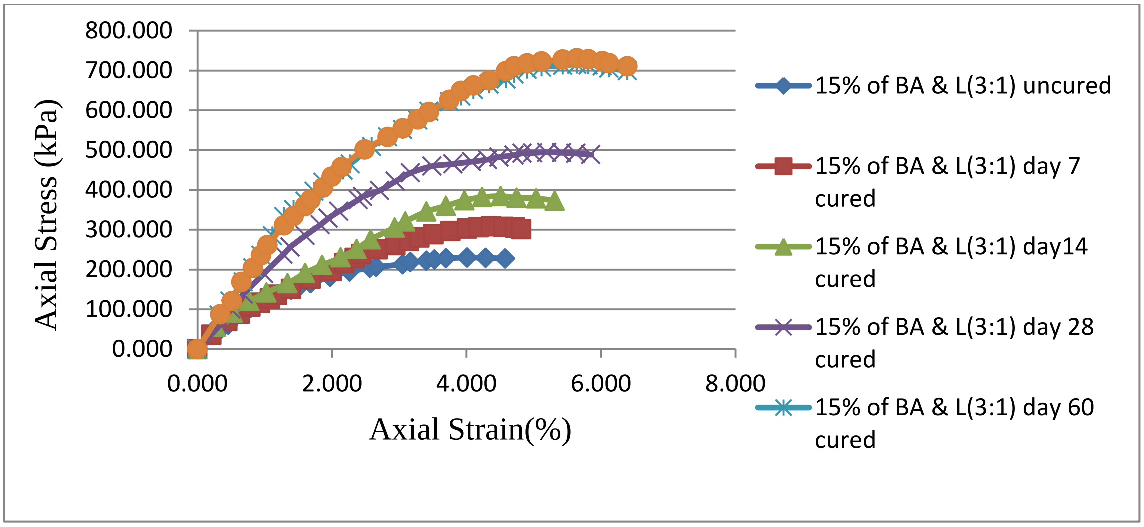

Figure H -2 UCS stress strain cure for 15% of BA & L(3:1) for uncured& 7, 14,28,60,90 day cured

Table H-28- UCS test result for soil + 15% of BA with lime by 2:2 (uncured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.21	0.003	0.276	1134.107	0.052	45.63
0.42	0.006	0.553	1134.104	0.096	84.63
0.60	0.008	0.789	1134.102	0.125	110.24
0.82	0.011	1.079	1134.099	0.148	130.12
1.00	0.013	1.316	1134.097	0.170	150.25
1.20	0.016	1.579	1134.094	0.191	168.74
1.40	0.018	1.842	1134.092	0.208	183.41
1.60	0.021	2.105	1134.089	0.227	200.58
1.80	0.024	2.368	1134.086	0.244	215.36
2.01	0.026	2.645	1134.084	0.255	224.50
2.12	0.028	2.789	1134.082	0.262	231.25
2.40	0.032	3.158	1134.078	0.280	246.64
2.52	0.033	3.316	1134.077	0.286	252.41
2.84	0.037	3.737	1134.073	0.294	259.65
3.00	0.039	3.947	1134.071	0.299	263.45
3.26	0.043	4.289	1134.067	0.300	264.85
3.43	0.045	4.513	1134.065	0.301	265.74
3.60	0.047	4.737	1134.063	0.296	260.87

Table H-29- UCS test result for soil + 15% of BA with lime by 2:2 (7 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.0	0.000	0.000	1134.110	0.000	0.00
0.25	0.003	0.329	1134.107	0.062	54.24
0.39	0.005	0.513	1134.105	0.104	91.63
0.48	0.006	0.632	1134.104	0.142	125.41
0.59	0.008	0.776	1134.102	0.170	150.26
0.78	0.010	1.026	1134.100	0.205	180.46
0.95	0.013	1.250	1134.098	0.233	205.32
1.12	0.015	1.474	1134.095	0.262	230.62
1.35	0.018	1.776	1134.092	0.298	262.34
1.56	0.021	2.053	1134.089	0.334	294.32
1.79	0.024	2.355	1134.086	0.368	324.84
1.93	0.025	2.539	1134.085	0.379	334.17
2.10	0.028	2.763	1134.082	0.397	350.08
2.33	0.031	3.066	1134.079	0.411	362.45
2.50	0.033	3.289	1134.077	0.428	377.62
2.74	0.036	3.605	1134.074	0.440	388.10
2.91	0.038	3.829	1134.072	0.447	394.28
3.15	0.041	4.145	1134.069	0.457	403.41
3.31	0.044	4.355	1134.066	0.463	408.69
3.46	0.046	4.553	1134.064	0.467	411.37
3.59	0.047	4.724	1134.063	0.468	412.36
3.78	0.050	4.974	1134.060	0.466	410.84
3.88	0.051	5.105	1134.059	0.462	407.16
4.06	0.053	5.342	1134.057	0.458	403.82
4.24	0.056	5.579	1134.054	0.449	395.97

Table H-30- UCS test result for soil + 15% of BA with lime by 2:2 (14 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.18	0.002	0.237	1134.108	0.050	43.89
0.30	0.004	0.395	1134.106	0.098	86.17
0.38	0.005	0.500	1134.105	0.131	115.84
0.53	0.007	0.697	1134.103	0.180	158.65
0.75	0.010	0.987	1134.100	0.242	213.64

0.92	0.012	1.211	1134.098	0.286	251.84
1.04	0.014	1.368	1134.096	0.328	289.47
1.12	0.015	1.474	1134.095	0.345	303.81
1.23	0.016	1.618	1134.094	0.362	319.50
1.35	0.018	1.776	1134.092	0.384	338.42
1.43	0.019	1.882	1134.091	0.402	354.76
1.58	0.021	2.079	1134.089	0.425	375.12
1.78	0.023	2.342	1134.087	0.457	402.53
2.04	0.027	2.684	1134.083	0.486	428.59
2.18	0.029	2.868	1134.081	0.503	443.94
2.34	0.031	3.079	1134.079	0.531	468.43
2.56	0.034	3.368	1134.076	0.550	485.27
2.92	0.038	3.842	1134.072	0.574	505.89
3.12	0.041	4.105	1134.069	0.593	522.46
3.32	0.044	4.368	1134.066	0.603	531.51
3.48	0.046	4.579	1134.064	0.610	538.12
3.60	0.047	4.737	1134.063	0.615	541.95
3.81	0.050	5.013	1134.060	0.616	542.81
4.06	0.053	5.342	1134.057	0.613	540.18
4.23	0.056	5.566	1134.054	0.610	537.49
4.35	0.057	5.724	1134.053	0.604	532.18
4.47	0.059	5.882	1134.051	0.594	523.61

Table H-31- UCS test result for soil + 15% of BA with lime by 2:2 (28 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.12	0.002	0.158	1134.108	0.098	86.17
0.18	0.002	0.237	1134.108	0.141	123.94
0.32	0.004	0.421	1134.106	0.254	224.17
0.41	0.005	0.539	1134.105	0.310	273.16
0.48	0.006	0.632	1134.104	0.354	312.18
0.55	0.007	0.724	1134.103	0.400	352.91
0.62	0.008	0.816	1134.102	0.451	397.46
0.75	0.010	0.987	1134.100	0.503	443.13
0.81	0.011	1.066	1134.099	0.543	478.37

0.95	0.013	1.250	1134.098	0.578	509.81
1.03	0.014	1.355	1134.096	0.624	550.63
1.16	0.015	1.526	1134.095	0.679	598.72
1.28	0.017	1.684	1134.093	0.725	638.91
1.34	0.018	1.763	1134.092	0.766	675.35
1.47	0.019	1.934	1134.091	0.820	723.14
1.59	0.021	2.092	1134.089	0.866	763.48
1.75	0.023	2.303	1134.087	0.902	795.36
1.89	0.025	2.487	1134.085	0.947	835.29
2.18	0.029	2.868	1134.081	0.981	864.74
2.36	0.031	3.105	1134.079	1.004	885.61
2.52	0.033	3.316	1134.077	1.026	904.37
2.67	0.035	3.513	1134.075	1.045	921.84
2.93	0.039	3.855	1134.071	1.061	935.13
3.24	0.043	4.263	1134.067	1.072	945.21
3.65	0.048	4.803	1134.062	1.080	952.48
3.96	0.052	5.211	1134.058	1.085	957.14
4.01	0.053	5.276	1134.057	1.089	960.46
4.24	0.056	5.579	1134.054	1.087	958.25
4.35	0.057	5.724	1134.053	1.083	955.34
4.51	0.059	5.934	1134.051	1.078	950.17
4.67	0.061	6.145	1134.049	1.070	943.61

Table H-32- UCS test result for soil + 15% of BA with lime by 2:2 (60 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.15	0.002	0.197	1134.108	0.098	86.17
0.25	0.003	0.329	1134.107	0.190	167.54
0.30	0.004	0.395	1134.106	0.254	223.64
0.38	0.005	0.500	1134.105	0.323	284.93
0.43	0.006	0.566	1134.104	0.359	316.48
0.51	0.007	0.671	1134.103	0.442	389.42
0.63	0.008	0.829	1134.102	0.511	450.18

0.69	0.009	0.908	1134.101	0.543	478.52
0.86	0.011	1.132	1134.099	0.619	545.86
0.91	0.012	1.197	1134.098	0.646	569.43
1.02	0.013	1.342	1134.097	0.694	612.38
1.12	0.015	1.474	1134.095	0.725	639.12
1.23	0.016	1.618	1134.094	0.766	675.03
1.31	0.017	1.724	1134.093	0.792	698.37
1.45	0.019	1.908	1134.091	0.831	732.82
1.56	0.021	2.053	1134.089	0.872	769.31
1.64	0.022	2.158	1134.088	0.911	802.98
1.78	0.023	2.342	1134.087	0.941	830.18
1.86	0.024	2.447	1134.086	0.966	851.37
1.97	0.026	2.592	1134.084	0.989	872.46
2.04	0.027	2.684	1134.083	1.018	897.61
2.23	0.029	2.934	1134.081	1.048	923.74
2.48	0.033	3.263	1134.077	1.073	945.81
2.57	0.034	3.382	1134.076	1.098	968.14
2.69	0.035	3.539	1134.075	1.114	982.37
2.86	0.038	3.763	1134.072	1.126	992.49
3.60	0.047	4.737	1134.063	1.250	1102.19
3.65	0.048	4.803	1134.062	1.260	1110.84
3.89	0.051	5.118	1134.059	1.266	1116.59
4.12	0.054	5.421	1134.056	1.271	1120.32
4.28	0.056	5.632	1134.054	1.265	1115.24
4.40	0.058	5.789	1134.052	1.258	1109.37
4.81	0.063	6.329	1134.047	1.133	999.15
4.95	0.065	6.513	1134.045	1.108	976.84
5.04	0.066	6.632	1134.044	1.080	952.14

Table H-33- UCS test result for soil + 15% of BA with lime by 2:2 (90 day cured)

Displacement [mm]	unit strain	Axial strain (%)	Area(mm2)	Load(KN)	Strength [KPa]
0.00	0.000	0.000	1134.110	0.000	0.00
0.14	0.002	0.184	1134.108	0.086	76.18
0.22	0.003	0.289	1134.107	0.128	113.25
0.33	0.004	0.434	1134.106	0.183	161.74

0.41	0.005	0.539	1134.105	0.228	200.83
0.52	0.007	0.684	1134.103	0.288	253.91
0.60	0.008	0.789	1134.102	0.342	301.38
0.71	0.009	0.934	1134.101	0.402	354.81
0.83	0.011	1.092	1134.099	0.455	401.20
0.94	0.012	1.237	1134.098	0.521	459.18
1.05	0.014	1.382	1134.096	0.584	515.37
1.16	0.015	1.526	1134.095	0.637	561.28
1.31	0.017	1.724	1134.093	0.708	624.30
1.45	0.019	1.908	1134.091	0.759	669.56
1.57	0.021	2.066	1134.089	0.807	711.84
1.69	0.022	2.224	1134.088	0.875	771.15
1.76	0.023	2.316	1134.087	0.909	801.34
1.84	0.024	2.421	1134.086	0.959	845.61
1.97	0.026	2.592	1134.084	1.018	897.34
2.13	0.028	2.803	1134.082	1.079	951.49
2.27	0.030	2.987	1134.080	1.132	998.12
2.35	0.031	3.092	1134.079	1.144	1008.97
2.41	0.032	3.171	1134.078	1.165	1027.14
2.52	0.033	3.316	1134.077	1.194	1052.48
2.65	0.035	3.487	1134.075	1.230	1084.21
2.74	0.036	3.605	1134.074	1.260	1110.84
2.86	0.038	3.763	1134.072	1.281	1129.34
3.02	0.040	3.974	1134.070	1.293	1140.28
3.18	0.042	4.184	1134.068	1.314	1158.47

3.31	0.044	4.355	1134.066	1.338	1179.61
3.52	0.046	4.632	1134.064	1.366	1204.37
3.68	0.048	4.842	1134.062	1.405	1238.49
3.85	0.051	5.066	1134.059	1.435	1265.28
4.12	0.054	5.421	1134.056	1.462	1289.13
4.30	0.057	5.658	1134.053	1.491	1314.75
4.48	0.059	5.895	1134.051	1.508	1329.89
4.67	0.061	6.145	1134.049	1.520	1340.62
4.83	0.064	6.355	1134.046	1.508	1330.00
5.07	0.067	6.671	1134.043	1.487	1310.84
5.24	0.069	6.895	1134.041	1.463	1289.71
5.48	0.072	7.211	1134.038	1.426	1257.42

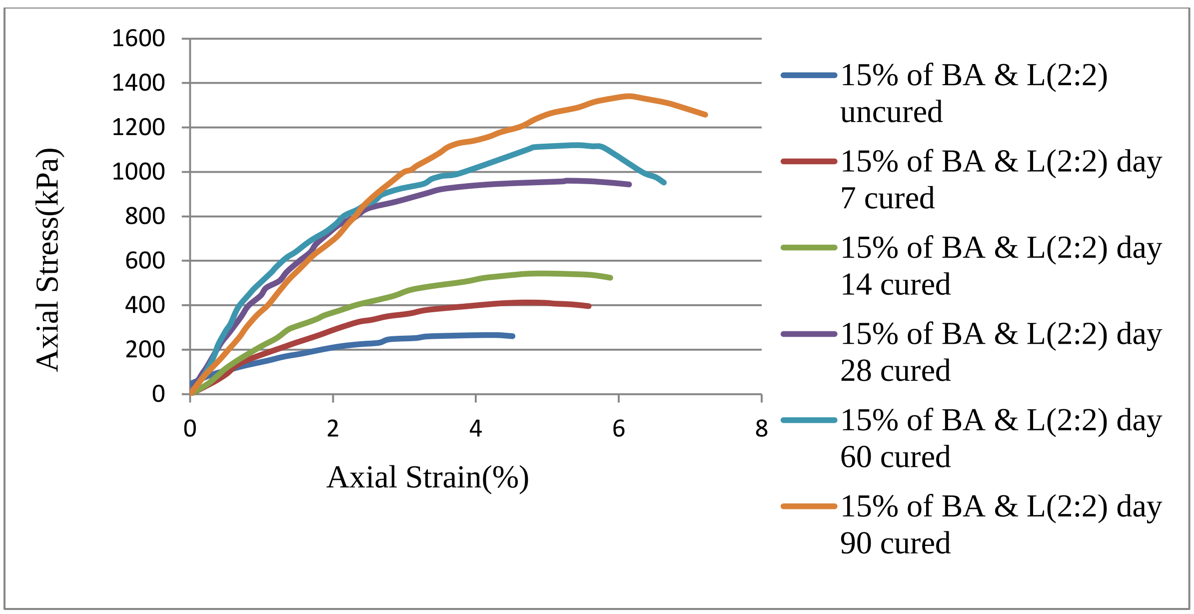

Figure H -3 UCS stress strain cure for 15% of BA & L by (2:2) for uncured& 7, 14,28,60,90 day