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Stabilization of Lateritic Soil with Sugarcane Bagasse Ash

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(Geotechnical Engineering)

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Declaration

I, the undersigned, declare that this project is my original work performed under the supervision of my research advisor Dr. –Ing. Samuel Tadesse and has not been presented as a thesis/project for a degree in any other university. All sources used for this paper have also been duly acknowledged.

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List of Symbols and Abbreviations

Designation		Unit
LL	Liquid Limit	%
PL	Plastic Limit	%
PI	Plasticity Index	%
A_c	Activity Number	---
LS	Linear Shrinkage	%
FS	Free Swell	%
OMC	Optimum Moisture Content	%
MDD	Maximum Dry Density	g/cm^3
CBR	California Bearing Ration	%
AASHTO	American Association of State Highway and Transportation Officials	---
ASTM	American Society for Testing and Material	---
USCS	Unified Soil Classification System	---
SCBA	Sugarcane bagasse ash	---
BS	British Standard	---
CAH	Calcium Aluminate Hydrate	---
CSH	Calcium Silicate Hydrate	---
FAO	Food and Agricultural Organization	---

Abstract

For soils to be suitable in civil engineering projects, they must meet existing local requirements for index properties in addition to certain strength criteria. Typically, specification limits these properties to some threshold values, which in most cases are project specific. Some lateritic soils, in their natural state, need some treatment or modification to meet these specification requirements.

Soil-stabilization is any treatment applied to a soil to improve its strength and reduce vulnerability to water. Disposal of large quantities of industrial by-products as fills on disposal sites adjacent to industries not only require large space but also create many geo-environmental problems. The disposal of the agro-industrial waste product of sugar mills-Bagasse ash also faces these problems. However, this ash were found to have pozzolanic properties, therefore have been used as stabilization agent in expansive and lateritic soils as well as in partial replacement of cement by many researchers.

In this research work, a lateritic soil was treated with sugarcane bagasse ash in stepped concentration of 2%, 4%, 6%, 8% and 16% by weight of dry soil. The effects of the ash on geotechnical properties of the soil were investigated. Test specimens were subjected to atterberg limit, free swell, linear shrinkage, compaction and CBR tests.

The lateritic soil under study belongs to A-7-6 class of soil according to AASHTO soil classification system. The analysis of results showed a slight decrease in the maximum dry density and soaked CBR values. The free swell also showed a slight decrement up to 4% of ash then showed an increase. A slight increase in plasticity index, shrinkage limit, optimum moisture content and unsoaked CBR values was also observed.

Generally, from the results it was concluded that sugarcane bagasse ash was not an effective stabilizer for the improvement of some of the geotechnical properties of the soil by its own.

Chapter One

Introduction

1.1. General

Laterites are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of rocks under tropical conditions (Blight, 1977). Laterite formation requires particular conditions, which concentrate the iron and aluminum rich weathering products sufficiently to allow concretionary development, resulting in cementing horizon within the weathering profile (CIRIA, 1995)

The formation of laterites requires hot and humid conditions where a mean annual temperature of around 25degrees is required for their formation, and in seasonal situation, there should be a coincidence of the warm and wet periods. The minimum annual rainfall required for laterite formation is generally at least 750mm. The higher the rainfall above this value, the greater is the leaching effect thus reducing the silica/sesquioxide ratio, as a result increasing the degree of laterization (CIRIA, 1995). Topography and drainage also affect the formation of laterites.

Laterites cover vast areas in the tropical countries with intermittently moist climate. The six main regions of the world in which laterites occur are Africa, India, South-east Asia, Australia, Central and South America. However, it should be noted that because of changes in climatic zone in the geological past, laterites could also be found in areas outside the tropics.

There have been several attempts to classify laterites and lateritic soils for many years. A pedological classification system given by D' Hoore classifies the soil in three main units; Ferruginous, Ferrallitic, and Ferrisols. According to Morine W. J. and Todor P. C. Ethiopian laterites fall under the last group (Lyon, 1971). Ferrisols tend to develop at deeper levels, because of surface erosion, and occur in regions of between 1250 and 2750mm rainfall annually.

Lateritic soils have found wide applications in such areas as pavements, embankments, low-cost houses, etc. However, weathered under conditions of high temperature and humidity with well-defined alternating wet and dry season results in poor engineering properties such as high plasticity, poor workability, low strength, high permeability, tendency to retain moisture and high natural moisture content (Agapitus, 2010). As a result, the effective use of these soils is hindered by difficulty is handling especially under moist and wet conditions typical to tropical regions, therefore can be used after improvement of their performance characteristics by appropriate stabilization methods.

Soil-stabilization is any treatment applied to a soil to improve its strength and reduce vulnerability to water. It refers particularly to the mixing of the parent soil with other soil, cement, lime, bituminous products, silicates and various other chemicals; natural or synthetic, organic and inorganic materials (Osinubi, 2008). Stabilization reduces the sensitivity of materials to moisture changes and loss of strength, it can also make the material easier to handle.

Soil-stabilization can be done using tradition stabilizers (lime, cement, etc.); by-product stabilizers (fly ash, quarry dust, bagasse ash, phospho-gypsum, slag, etc.) and non-traditional stabilizers (sulfonate oils, potassium compounds, polymers, enzymes, ammonium chlorides, etc.)

Disposal of large quantities of industrial by-products as fills on disposal sites adjacent to industries not only require large spaces but also create many geo-environment problems (Parhi, 2014). One such example is the by-product of sugar cane industry, bagasse ash. Bagasse ash has a problem to the environment due to its disposal. When bagasse is left in the open, it ferments and decays; this brings about the need for safe disposal of the pollutant. The treatment of soil with bagasse ash could be a safe way of reducing the menace (Osinubi et al, 2009). Bagasse ash has been found to be a pozzolanic material, which is very rich in silica and aluminum and sometimes calcium oxides (TRRL, 1977; Ogbonyomi, 1998, Guilherme et al, 2004). Many researchers have attempted to use this ash as soil stabilizers.

In Ethiopia sugar production is about 300,000 tons, the bagasse ash potential is about 72,000 tons annually. This figure will rise to about 0.94 million tons when the expansion of the sugar industry comes to reality. From the result of the geochemical test performed by (Hailu B., 2011) for his research work "Bagasse ash as cement replacing material", he has categorized bagasse ash under class N pozzolana, as per ASTM C618 classification. Therefore, the bulk utilization of this ash as a soil stabilizer will reduce the problem of disposal, which creates many geo-environmental problems.

This study focuses on using Sugarcane bagasse ash to stabilize lateritic soil found from a borrow pit in Mekenajo, 456km west of Addis Ababa. It will study the effects of bagasse ash on some properties of the lateritic soil.

1.2. Objective of the Study

The objectives of this research work are as follows;

1. Check whether the soil sample from Mekenajo is lateritic soil by comparing results from prior studies and literature.
2. Investigate the effect of sugarcane bagasse ash on some geotechnical properties of the soil.

1.3. Research Methodology

The following methodologies were implemented;

1. Different literatures on laterites, soil-stabilization, bagasse ash and other relevant materials were collected and reviewed.
2. Disturbed sampling was done at Mekenajo and sample was brought to the laboratory.
3. Sample preparation and testing was done on the natural and stabilized sample.
4. Analysis and discussion of test results were undertaken based on the findings.
5. Conclusions were drawn from the analysis.

1.4. Structure of the study

The paper has been divided into six chapters. In the first chapter the background, objective and brief summary of the work is presented. The second chapter gives a literature review on lateritic soil and soil stabilization. The third chapter explains the materials and methods used in the study. The test results and analysis are presented in the fourth chapter. The fifth chapter compares the result of the natural soil with other research findings. Finally, the last chapter presents the conclusions drawn and recommendations made from the findings.

Chapter Two

Literature Review

Lateritic Soil

2.1. Characteristics of Laterites

2.1.1. General

Lateritic soils abound in most parts of the tropics. They are results of the decomposition and weathering process of rocks under tropical conditions and are found in profiles varying from fresh rock at depth, through various stages of decomposition, to residual soil at the surface (CIRIA, 1995). Because of their various forms, the term laterite has been loosely applied and has created different interpretation among different professionals.

The first reference to Laterite was by Buchanan in 1807 (CIRIA, 1995);

> It is diffused in great masses, without any appearance of stratification, and is placed over the granite that forms the basis of Malayala (India). It is full of cavities and pores, and contains a very large quantity of iron in the form of red and yellow ochres. In the mass, while excluded from the air, it is so soft that any iron immediately cut into the shape wanted with a trowel or large knife. It very soon becomes as hard as a brick and resists the air and water much better than any brick I have seen in India.

Buchanan also coined the soil name laterite from the Latin word, later, meaning brick, because it was quarried to make bricks. Since Buchanan's time, the word laterite has been used to describe a wide variety of tropical soils without reaching an agreement on the exact origin, composition and properties of laterites. The term has been applied to clays, sands and gravels in various combinations, and to rock of different degrees of cementation. The properties of laterites thus depend on their weathering history, therefore on several factors including parent rock, climate, drainage, topography and site erosion, transportation and re-deposition as a secondary deposit (CIRIA, 1995)

Blight (1997) defines laterites as highly weathered and altered residual soils formed by the insitu weathering and decomposition of rocks under tropical conditions. The three major weathering processes are physical, chemical and biological processes. Physical processes results in increment of surface area in-turn making way for chemical attack to chemically change the parent rock by enrichment with iron and aluminum oxide. Therefore, the clay mineral component becomes largely kaolinite and silica content is reduced. This process usually produces yellow, red or purple materials, red being the predominant color. Biological weathering includes both physical and chemical processes.

2.1.2. Formation and Ocurrence

Laterite formation requires particular conditions, which concentrate the iron and aluminum rich weathering products sufficiently to allow concretionary development, resulting in a cemented horizon within the weathering profile (CIRIA, 1995). The three phases that are required to produce a concretionary laterite are as follows:

a) Tropical weathering to produce the mineral of laterite
b) Formation of discrete horizon.
c) Concretionary development within the horizon.

a) Tropical Weathering Process and Weathering Profile

Disintegration and decomposition are the two major processes of weathering. The former is the physical breakdown and the later is the chemical alteration of the primary minerals into secondary and residual products. These processes are highly influenced by the regional climatic zones. Laterite occurs mostly in the tropical and sub-tropical regions with hot and humid climatic conditions. In the tropical and sub-tropical regions, soil temperature is high through out the year. For every 10degree rise in temperature the rate of chemical reaction is doubled, meaning a rate of chemical weathering four times faster in the tropics than in the temperate regions of the world and twice as fast as the sub-tropical regions. (CIRIA, 1995)

Figure 2/1 below shows the relationship between climate and the formation of deep weathering profiles. It can be seen from the figure that there is significant chemical weathering and formation of clay minerals at the tropics where there is high rate of temperature and precipitation.

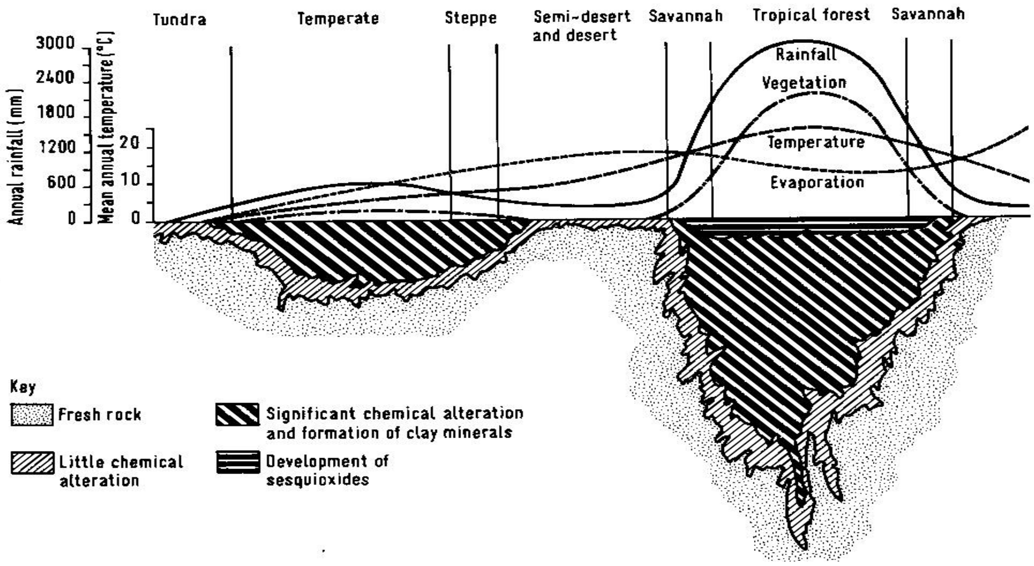

Figure 2/1 Schematic relationship between climate and weathering (CIRIA, 1995)

b) Formation of Discrete Horizon

For the concretionary development of true laterites to take place, the process of formation of a weathering profile needs to undergo by the concetration of the weathering products within the residual soil. Open-jointed rocks near the surface collects surface run-off which eventually penetrates through joints and fractures. This penetration leads to the isolation of blocks of fresh rocks surrounded by the decomposed material, ultimately producing a residual soil.

Alteration starts on the surface of fractures, with the decomposition of the least stable parent material. Crystalline rocks with low silica content such as gabbro, dolerite and basalt contain less stable, therefore easily weatherable minerals such as olivine, pyroxene and calcic plagioclase. Olivine alters first to serpentine and then to montomorillonite. Pyroxene to chlorite, calcite or montomorillonite; and feldspar to kaolinite.

The silica-rich rocks such as granite, gneiss and rhyolite contain more stable minerals such as quartz, potash and feldspar with micas and amphiboles. Feldspars weather to kaolinite, micas to montomorillonite then to kaolinite.

Iron and aluminum oxides are the main components of laterites. The iron oxide is usually present as haematitie and the aluminum is present as its hydrated oxides, gibbsite and diaspore. It's due to the presence of iron oxide that lateritic soils get there red color which ranges from light through bright to brown shades.

There are different factors that affect the development of tropical soils, thus it's weathering process. These are parent rock, climatic conditions and topography and drainage. There are two aspects of parent rock that affect the development of laterites. One is the availability of iron and aluminum minerals, which are readily available in the basic rock; and the other is the quartz content of the parent rock. Quartz remains as quartz grain in the weathering product because of its resistant and no mineralogical change occur unless subjected to slow solution of leaching.

The formation of laterites requires hot and humid conditions where a mean annual temperature of around 25degrees is required for their formation, and in seasonal situation there should be a coincidence of the warm and wet periods. The minimum annual rainfall required for laterite formation is generally at least 750mm. The higher the rainfall above this value, the greater is the leaching effect thus reducing the silica/sesquioxide ratio as a result increases the degree of laterizaation (CIRIA, 1995).

The other factors that affect the formation of laterites are topography and drainage. The amount of water moving downward through the weathering zone is basically controlled by the slope angle. If comparison is made between steep and flat slopes, run-off and erosion is higher in the former and not so marked in the later. Therefore, long and uninterrupted periods of weathering can occur on flatter slopes producing deep soil profiles. On the contrary, on level grounds where drainage is impeded, black montmorillonite soils dominate as a result of the absence of leaching. On slopes of between 5 and 15degree, there is less infiltration and the top 50-100mm of residual soil changes to kaolinite; slopes between 15 and 30 degree the kaolinite thickness increases; but on slopes greater than 30degrees nun-off erodes the weathered products (CIRIA, 1995).

c) Process of Concretionary Development

The development of concretion requires sufficient concentration of hydrated oxides of iron and aluminium for cementation or precipitation growth to start (CIRIA, 1995). Soil scientists have called the uncemented horizon of material, plinthite. Because this horizon is present, concretionary development is possible, therefore the material can be called laterite.

The concentration of the minerals in laterites depends on the relative mobility of silica and iron/aluminium oxides under certain physical and chemical conditions. Two processes are vital to cause a hardening or concretionary development; chemical precipitaion and loss of water of dehydration.The physical condition in which these processes develop may be provided by a fluctuating ground water level, which causes reducing and oxidizing conditions alternately. Precipitation and dehydration can only take place by the lowering of the ground water level.

2.1.3. Regional Distribution

Regional distribution of laterites is mainly governed by the world's climatic zones. Laterites cover vast areas in the tropical countries with intermittently moist climate. The six main regions of the world in which laterites occur are Africa, India, South-east Asia, Australia, Central and South America. However, it should be noted that because of changes in climatic zone in the geological past, laterites can also be found in areas outside the tropics.

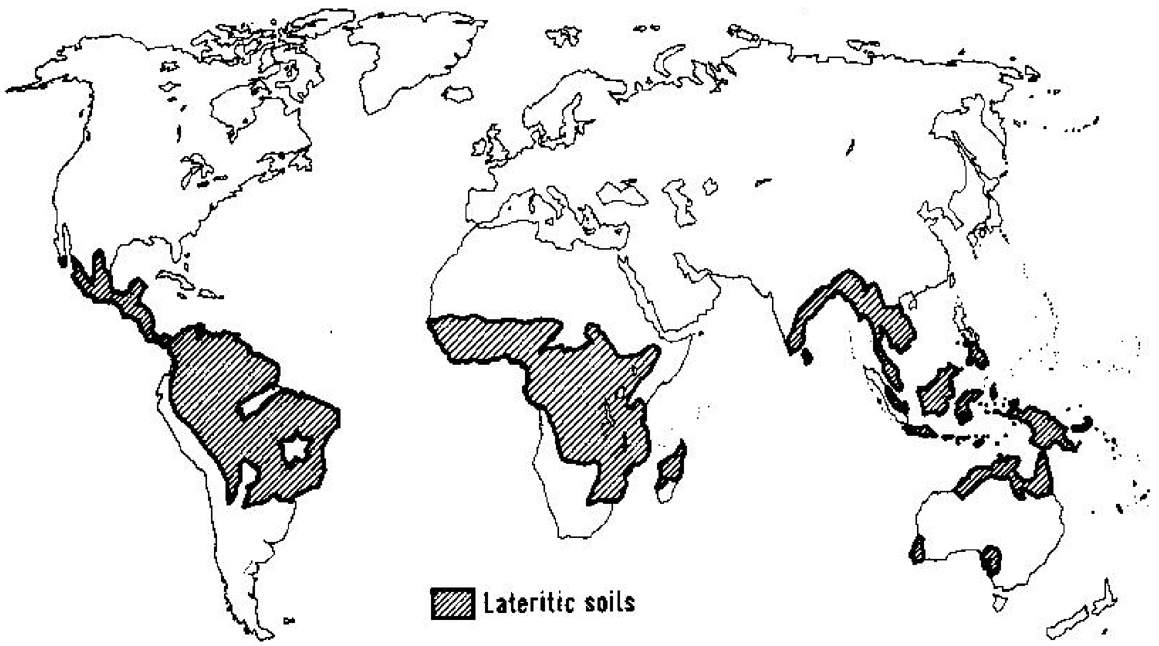

Figure 2/2 Generalized world map showing the distribution of lateritic soil (CIRIA, 1995)

2.2. Classification of Laterites

There have been several attempts to classify laterites and lateritic soils for many years, but none of the proposed classifcation sytem has been accepted universally. According to Maignien, 1966, these classification systems can be grouped as analytical classifications which are based mainly on morphological characterstics with a bias toward soil genetic considerations, synthetic classifications which are based on genetic factors or soil-genetic processes. Most classification systems does not aim to classify the soils according to their engineering behavior. Although there are some popular engineering classification systems, such as Unified Soil Clasification System (USCS) and the American Association of State Highway and Trasportation Officials System (AASHTO) which have been used satisfactorily for other soil types. However, these classification systems are based on plasticity and gradation data of the soils; but such characteristics of tropical soils are not reproducable by standard labratory tests. This is because laterites are highly influenced by sample preparation and handling which disrupts the natural structure of the soil.

According to CIRIA, 1995, a description of laterite should include all of the following characterstics:

1. Strength or consistency
2. Colour
3. Degree of induration or cementation
4. Structure
5. Soil name (clay, silt, sand gravel), grading and plasticity
6. Geolgical name

These characterstics are of particular importance for the description of laterites, because it governs their engineering performance. Therefore, the classification recommended by CIRIA is shown in Table 2/1 below.

Table 2/1 Classification system for laterite depending on the degree of concretionary development (CIRIA, 1995)

Age	Recommended Names	Characteristics	Equivalent terms in literature
Immature (Young)	PLINTHITE	Soil fabric containing a significant amount of laterite material. Hydrated oxides at the expense of some soil material. Unhardened nodules present, but may be slight evidence of concretionary development.	Plinthite, laterite, lateritic clay
↓	NODULAR LATERITE	Distinct hard concretionary nodules present as separate particles.	Laterite gravel, ironstone gravel, pisolitis gravel
↓	HONEYCOMB LATERITE	Concretions have coalesced to form a porous structure which may be filled with soil materials.	Vesicular laterite pisolitic ironstone, cellular ironstone, spaced pisolitic laterite
↓	HARDPAN LATERITE	Indurated laterite layer, massive and tough	Ferricrete, ironstone, laterite crust, vermiform laterite, packed pisolitic laterite
Mature (old)	SECONDARY LATERITE	Maybe nodular honeycomb or hardpan, but is result of erosion of pre-existing layer and may display brecciated appearance.	

A classification system of residual soils based on mineralogical composition is suggested by Wesley L.D. and Irfan T. Y. (Blight, 1997). These system classifies residual soil into three groups;

Group A: Soils without a strong mineralogical influence.

Group B: Soils with a strong meneralogical influence deriving from clay minerals also commonly found in transported soils.

Group C: Soils with a strong mineralogical influence deriving from clay minerals only found in residual soils.

Group C is further devided in to three sub-groups according to clay minerals of soils. These are

Sub-group (a) Halloysitic soils

Sub-groub (b) Allophanic soils

Sub-group (c) soils influenced by the presence of sesquioxides

Lateritic soils lay under group C, sub-group (c). The engineering properties of soils under this group are highly influenced by the presence of sesquioxides. Sesquioxide is the combined name of iron and aluminium oxide. Sesquioxides appear to act as a cementing agents which bind the othe mineral constituents into clusters or aggregations. The hard concretionary materials are fromed as a result of sufficient concretion of sesquioxides.

A classification system developed for pedological mapping is suggested by US Soil Conservation Service (USSCS). This is based on the presence or absence of certain diagnostic horizons within the soil profile near the surface. According to this classification system all soils fit into one of ten orders. Within these soil orders there are many sub-divisions; first into sub-goups, based on differences in soil moisture and soil temperature; and then into great groups by differentiating soil horizons and soil feature.

The soil orders which develop only in the tropics and subtropics are **oxisols, ultisols,** and **alfisols**. The latter two orders represent saprolitic materials in which considerable weathering has taken place, without the same degree of sesquioxides concentration. Oxisols are the most intensely weathered soils and are rich in iron and aluminium oxides. Laterite is generally found in this soil order,which includes concretionary laterite and weathered material which hardenes on exposure or after repeated wetting and drying.

Another pedolgical classification system is also given by D' Hoore. This system which is associated with French practice, broadly differentiates soils on a genetic basis, determined by soil forming factors. Three main units are used for the description and classification of red tropical soils (CIRIA, 1995).

Ferruginous soils show a marked separation of free iron oxides, either leached out of the profile or precipitated within the profile as concretions. There may be a high proportion of weatherable primary minerals remaining. Kaolinite is the dominant clay minerals. These soils are generally found in areas with under 1850mm rainfall a year and pronounced dry seasons.

Ferrallitic soils are generally deep, with only slightly differentiated horizons. Kaolinite is the dominant clay mineral and they contain free iron oxides and hydrated oxides of aluminium. They generally occur in more humid areas with more than 1500mm raifall per year.

Ferrisols have profiles similar to ferrallitic soils, but with very few weatherable minerals remaining. The entire clay-size fraction comprises kaolinite and amorphous oxides of iron and aluminium. Ferrisols tend to develop at deeper levels, because of surface erosion, and occur in regions of between 1250 and 2750mm raifall per year. According to Morine W.J. and Todor P.C., Ethiopian laterites fall under this group (Lyon, 1971)

A lithological classification systems of laterites based on particle size is also given (Lyon, 1971)

Lateritic clays	< 0.002mm
Lateritic silts	= 0.002 – 0.06mm
Lateritic sands	= 0.06 – 2mm
Lateritic gravels	= 2-60mm
And cuirasse	> 60mm

2.3. Engineering Properties of Laterites

Concretionary laterities are valuable road pavement materials, widely used in the tropics as a sub-base, base materials and gravel roads. However, the term laterite has tended to be indiscriminately applied in the tropical highway engineering to any red soil, and as a result the usefulness of laterite for road construction has been under-estimated (CIRIA, 1995).

These soils, however, weathered under conditions of high temperature and humidity with well-defined alternating wet and dry seasons resulting in poor engineering properties such as high plasticity, poor workability, low strength, high permeability, tendency to retain moisture and high natural moisture content (Agapitus, 2010). As a result, the effective use of these soils is hindered by difficulty in handling especially under moist and wet conditions typical to tropical regions, therefore can only be utilized after modification or stabilization.

Labratory testing to check the suitability of concretionary laterites to be used as a road pavement material should take into consideration how these materials are affected by testing procedures (CIRIA,1995). Some laterites show changes in physical properties when tested under different conditions. Laterites formed under continuously wet regions are likely to be characterized by high natural water content, high liquid limit and irreversible changes upon drying (Zelalem A., 2005). As a result, upon drying the plasticity decreases and grain size increases because the clay sized material agglomerates to the size of silt. The particle agglomeration is due to significant increase in capillary stress, therefore reducing the available surface for interaction with water. This results in the reduction of plasticity.

In the investigation of laterites fron Nejo-Mendi area by Zelalem, 1995, the laterites did not show sensitivity to pre-treatment, i.e. upon drying before testing. In addition the soil did not contain significant amount of loose molecular water, therefore he concluded that oven temperature of 105^{0}c could be used to determine water content. Since the area lies in the medium to high rainfall with the average annual value of around 1600mm mostly occuring during june to september;these findings are in agreement with the findings of Morin W.J. and Todor P.C., on lateritic soils from areas with distinct wet and dry season.

However, his findings also showed that the soil showed sensitivity to test procedures which are affected during testing manipulation. Therefore, he concluded that the atterberg limit and compaction tests should be conducted using fresh samples for each testing points.

When soils are manipulated their charactersitcs vary a lot. Pre-testing drying causes variations in some properties of lateritic soils and this behaviour is commonly atrributed to the dehydration of the colloidal hydrated oxides occuring in these soils. Mostly, the variation resulting from drying is irreversible and results in a soil with more granular characterstics (Tuncer, 1976). It is beause of these difficulties that it is derive an acceptable generalization for lateritic soils with regard to plasticity and gradation.

The behaviour of laterities to be gap-graded with a depleted sand-size fraction, to contian a variable percentage of fines, and to have coarse particles of variable strength which may break down in performance, limits their usefulness as pavement materials on highly trafficked roads (CIRIA,1995). Therefore, in order to use laterites for higher traffic intensities their performance characterstics has to be improved by appropriate stabilization methods.

Stabilization reduces the sensitivity of materials to moisture changes and loss of strength, it can also make the material easier to handle.

Soil Stabilization

2.1. Introduction

Lateritic soils have found wide applications in such areas as pavements, embankements, low-cost houses, etc. In some cases the properties of the soils in the immediate vicinity of the construction works may not meet the required specifications. The need thus arises to improve the properties of the available materials. The mechanical instability, which may manifest in form of remoulding and manipulation, as discussed above, results in the breakdown of cementation and structure. The engineering properties affected by this mechanical instability includes particle size, atterberg's limits and moisture-density ditribution (Amu, O.O., *et al,* 2011). The effects of this problems thus affect the strength of the material.

Soil-stabilization is any treatment applied to a soil to improve its strength and reduce vulnerability to water. It refers particularly to the mixing of the parent soil with other soil, cement, lime, bituminous products, silicates and various other chemicals and natural or synthetic, organic and inorganic materials (Osinubi, 2008).

2.2. Types of Soil Stabilization

Soil stabilization methods can be divided into two catagories, mechanical and chemical.

2.2.1. Mechanical stabilization

Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specifications. It can also be achieved through a physical process by altering the physical nature of native soil particles by either induced vibration or compaction.

2.2.2. Chemical stabilization

Chemical stabilization depends on chemical reaction between stabilizer and soil minerals to achieve the desired effect. It is the oldest and most widespread method of ground improvement (Meron, 2015). Chemical stabilization can achieve improvement of soil by three chemical reactions

i. 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 (Meron, 2015). Higher valence cations easily replace cations of lower valence. The size of the hydrated ion becomes important for ions of same valence. A typical replaceability series is (Meron, 2015):

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

ii. Flocculation and Agglomeration

Cation exchange results in change in the electrical charge around the clay particles, therefore, result in an increase in the interparticle attraction causing flocculation and agglomeration. This leads to the reduction of clay-sized particles, thus the soil surface area.

iii. Pozzolanic Reaction

Pozzolanic reaction is time dependent long-term reaction that plays a major role in the soil stabilization. Pozzolanic constituent produce stable calcium silicate hydrate (CSH), calcium aluminate hydrate (CAH), and calcium aluminosilicate hydrates (CASH) which generate long-term strength gain and improve the geotechnical properties of the soil.

Pozzolanic reaction of soil stabilization is as follows:

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

$$Ca(OH)_2 \longrightarrow Ca^{++} + 2\ (OH)$$

$$Ca^{++} + 2\ (OH) + SiO_2 \text{ (soluble clay silica)} \longrightarrow \text{CSH “Calcium silicate hydrate“}$$

$$Ca^{++} + 2\ (OH) + Al_2\ O_3 \text{ (Soluble clay alumina)} \longrightarrow \text{CAH “Calcium aluminate hydrate”}$$

The CSH formed initially coats and binds lumps of clay together, then it crystallizes to form an interlocking structure which increases the soil strength (Meron. 2015).

Stabilizers can slo be divided into three groups (Petry, 2002);

1. Tradition stabilizers (lime, cement, etc.)
2. By-product stabilizers (fly ash, quarry dust, baggase ash, phospho-gypsu, slag, etc.)
3. Non-traditional stabilizer (sulfonate oils, potassium compounds, polymers, enzymes, ammonium chlorides, etc.)

Disposal of large quantities of industrial by products as fiils on diposal sites adjacent to industries not only require large space but also create a lot of geo-environment problems (Parhi, 2014). Many researchers and organization have made attempts to use them in bulk as soil stabilizers.

Amu, O. O., Ogunniyi, S.A. and Oladeji, O.O. studied the geotechnical properties of lateritic soil (Osun state, Nigeria) stabilized with sugarcane straw ash. Geotechnical strength tests (compaction, UCS, triaxial and CBR) were performed. OMC and the CBR values increased as the amount of SCSA in the mixture increased form 0 to 8%. They found that sugarcane straw ash was an effective stabilizer for improving the geotechnical properties of lateritic soil samples.

Ogunribido T.H.T. studied the potentials of SCSA for lateritic soil stabilization in road construction. After investigating its effect on atterberg's limit, compaction, CBR and UCS on the sample from southwestern Nigeria, he has concluded that SCSA was not a very good stabilizer.

Agapitus Amadi sstudied the changes in index properties of lateritic soil using fly ash. The soil sample from Shika, Zaria, Nigeria showed that the introduction of fly ash (0-20% by dry weight of the soil) enhanced the gradation characterstics by reducing the amount of clay particles through flocculation and agglomeration of the clay particles. The natural soil also showed a decrease in plasticity index (PI), form 22.22% at 0% fly ash to 3.54% at 20% fly ash. The maximum dry unit weight of the soil mixtures decreased with higher fly ash content while OMC increased as the amount of fly ash in the mixture increased from 0-20%.

F.H.M. Prtelinha, D.C. Lima, M.P.F. Fontes and C.A.B. Carvalho studied the modification of a lateritic soil with lime and cement. The workability, chemical properties, mechanical behaviour and mineralogical composition were evaluated on samples form Brazil. The result showed that addition of 2% and 3% of lime or cement was enough to change the soil workability and mechanical strength.

Olugbenga Oludolapo Amu, Oluwoole Fakunle Bamisaya and Iyiola Akanmu Komolafe studied the suitability and lime stabilization requirement of some lateritic soil samples as pavement construction materials. The samples were stabilized with 0 to 10% lime and were subjected to consistency tests, compaction, CBR, UCS and undrained triaxial tests. The addition of lime cause a reduction in plasticity index. The CBR, compressive and shear strengths of the sample soil also improved.

K.J. Osinubi, V. Bafyau, A.O. Eberemu and O. Adrian studied stabilization of lateritic soil usin baggase ash. They found that the MDD and OMC of the treated soil showed trends of decreasing and increasing respectively with increasing SCBA content. The UCS increased from 366KN/M2 for the natural soil to 836,842 and 973KN/m2 for samples treated with 2% bagasse ash and cured for 7, 14 and 28 days, respectively. CBR value also increased from 10% to 16% for soil treated with 2% bagasse ash.

2.3. Application of Soil Stabilization

Soil stabilization is the treatment of soil in order to rectify its deficiencies in engineering properties and especially as a road construction material (Chmeisse, 1992). Some of the aims of soil stabilization are as follows;

- Increase in strength ans stiffness of soils
- Increase in durability
- Enhancement of workability
- Reduction of compressibility

- Reduction of permeability
- Reduction of volume instability
- Control of dust and protection from erosion

2.4. Factors affecting the strength of stabilized soil

Presence of organic matters, sulphates, sulphides and carbondioxide in the stabilized soils may contribute to undesirable strength of stabilized materials (Makusa, 2012).

i. Organic Matter

The top most layers of most soil constitute large amount of organic matters. These matters react with hydration product like calcium hydroxide ($Ca(OH)_2$) resulting into low pH vlaue. As a result, the hydration process maybe retarded and affect the hardening of stabilized soils making it difficult to compact (Makusa, 2012).

ii. Sulphates

When using calcium-based tabilizer in sulphate-rich soils, the stabilized sulphate rich soil in the presence of excess moisture reacts and forms calcium sulphoaluminate. This product occupies a greater volume than the combined volume of reactants, However, excess water to one initial present during the time of mixing may be required to dissolve sulphate in order to allow the reaction to proceed.

iii. Sulphides

In many waste materials and industrial by-product, sulphides in the form of iron pyrites (FeS_2) may be present. Oxidation of FeS_2 will produce sulphuric acid, which in the presence of calcium carbonate, may react to form gypsum (hydrated calcium sulphate) according to the reactions below.

a. $2FeS_2 + 2H_2O + 7O_2 = 2FeSO_4 + 2H_2SO_4$

b. $CaCO_3 + H_2SO_4 + H_2O = CaSO_4.2H_2O + CO_2$

The hydrated sulphate so formed, and in the presence of excess water may attack the stabilized material in a similar way as sulphate (Makusa, 2012).

iv. Compaction

Stabilized mixure has lower maximum dry density than that of unstabilize soil for a given degree of compaction. In cement stabilized soils, hydration process takes place immediately after cement comes into contact with water. This process involves hardening of soil mix which means that it is necessary to compact the soil mix as soon as possible. Any delay in compaction may result in hardening of stabilized soil mass and therefore extra compaction effort may be required to bring the same effect. In contrary to cement, delay in compaction for lime-stabilized soils may have some advantages. Lime stabilized soil require mellowing period to allow lime to diffuse through the soil thus producing maximum effects on plasticity. After this lime stabilized soil may be remixed and given its final compaction resulting into remarkable strength than otherwise (Makusa, 2012).

v. Moisture Content

In stabilized soils, enough moisture content is essential not only for hydration process to proceed but also for efficient compaction. Fully hydrated cement take up about 20% of its own weight of water from surrounding (Makusa, 2012); on the other hand, quicklime (CaO) takes up about 32% of its own weight of water from the surrounding. Insufficient moisture content will cause stabilizers to compete with soils in order to gain these amounts of moisture. For soils with great soil-water affinity (clay, peat, and organic soils), the hydration process maybe retarded due to insufficient moisture content, which will ultimately affect the final strength.

vi. Temperature

Pozzolanic reaction is sensitive to changes in temperature. In the field, temperature varies continuosly throughout the day. Pozzolanic reactions between stabilizers and soil particles will slow down at low temperature and result into lower strength of the stabilized soil.

vii. Freeze-Thaw and Dry-Wet Effect

Stabilized soils cannot withstand freeze-thaw cycles. Therefore, it is necessary to protect the soil from frost damage.

2.5. Pozzolanas

Pozzolanas are siliceous and aluminous materials, which in itself possess little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (ASTM 595). Artificial pozzolanas such as ashes are products obtained by heat treatment of natural materials containing pozzolanas such as clays, shales and certain silicious rocks (Makusa, 2012). Plants when burnt, silica taken from soils as nutrients remains behind in the ashes contributing to pozzolanic element. Rice husk ash and rice straw and bagasse ash are rich in silica and make as excellent pozzolana (Makusa, 2012).

2.6. Pozzolanic By-Products

Pozzolanic by-products or artificially burnt inorganic materials obtained as industrial or agricultural by-products are similar to such volcanic soils from the view point of good cementation with hydrated additives (Chmeisse, 1992). These by-products are increasingly playing an important role in road construction, therefore minimizing the problem of resource depletion, environmental degradation and energy consumption.

Fly ash is probaby the most commonly used artificial pozzolan globally. Many plant ashes have a high silica content which, by suitable treatment, can be made to be pozzolanic. Recently attention has been drawn to the uses of rice husk ash as a pozzolan although other agricultural residues such as bagasse, bamboo leaves and some timber species are also of interest.

2.7. Bagasse Ash

2.7.1. Description and Production

Sugarcane processing is focussed on the production of cane sugar from sugarcane. Other products of the processing include bagasse, molasses, and filtercake. Bagasse is the fibrous residue generated after the juice has been extracted from the sugar cane plant. About 40-45% of this fibrous residue is left which is reused in the same industry as fuel boiler for heat generation leaving behind 8-10% ash as waste. This ash is known as sugarcane bagasse ash (SCBA) and is deposited in stockpiles which are normally dumped in waste land fills and constitute environmental problems to the society.

Bagasse ash is a pozzolanic material which is very rich in the oxides of, silica and aluminum and sometimes calcium (TRRL, 1977; Ogbonyomi, 1998, Guilherme et al, 2004). Pozzolans usually require the presence of water in order for silica to combine with calcium hydroxide to form stable calcium silicate, which has cementitious properties, hence the need for curing which is the retention of moisture within a material so as to enhance its gradual increase in strength and reduce the shrinkage of the material. The longer the initial period of curing, the smaller the shrinkage when the layer subsequently dries. Curing also helps in preventing carbonation, which is the loss in strength of a modified material, (Netterberg, 1984).

Bagasse ash has a problem to the environment due to its disposal. When bagasse is left in the open, it ferments and decays; this brings about the need for safe disposal of the pollutant. It can be blown away by wind and inhaled and can create health problems. If the ash is inhaled in large doses it can cause a respiratory disease known as bagassiosis (Laurianne, 2004). The treatment of soil with bagasse ash could be a safe way of reducing the menace (Osinubi et al, 2009).

Since bagasse ash is a by-product of the cane sugar industry, the quantity of production is in line with the quantity of sugarcane produced. Sugarcane is the world's largest crop by production quantity. In 2012, FAO estimated it was cultivated on about 26 million hectares, in more than 90 countries, with a worldwide harvest of 1.83 billion tons. The world's top 10 sugar producing nations are shown below.

Table 2/2 Top ten sugarcane producers (FAO)

Top ten sugarcane producers – 2013 (FAO)	
Country	Production (thousan metric tons, TMT)
Brazil	739,267
India	341,200
China	125,536
Thailand	100,096
Pakistan	63,750
Mexico	61,182
Colombia	34,876
Indonesia	33,700
Philippines	31,874
United States	27,906

2.7.2. Availability of Bagasse Ash in Ethiopia

Sugar production in Ethiopia started in 1954/55. Currently, there are three large-scale sugar establishments in the country; two of them in the Awash Basin (Wonji/shewa and Metehara) and one in the Blue Nile Basin (Fincha). The present level of national production from all is estimated about 261,041 tons of sugar each year, but have a production capacity of 280,000 tons of sugar annually (EIA, 2012). The per capita consumption in Ethiopia is one of the lowest in the world. A forecast for Ethioipan consumption of sugar that was done taking population into account showed that consumption demand increase 15% anually. As a result Ethiopia is seeing to expand the industry.

Table 2/3 Estimated bagasse ash potential of Ethiopia (Hailu, B., 2011)

Factory	Expected future production of sugar (tons/year)	Estimated Bagasse (tons/year)	Estimates Bagasse Ash (tons/year)
Wonji-Shoa	350,000	1,050,000	84,000
Metehara	190,000	570,00	45,600
Fincha	270,000	810,000	648,00
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

Currently with the sugar production of about 300,000 tons annually, the bagasse ash potential is about 72,000 tons annually (Meron, 2015). It can be seen from the above table that about 0.94 million tons of bagasse ash is going to be generated annually when the expansion of the industry is finalized.

However, these ashes are normally produced under uncontrolled and non-uniform burning conditions, therefore, to get a homogeneous characterstics and the same effect on different soils is very difficult. The result obtained from different researchers are material dependent for that specific type, burning condition and temperature and place where it is obtained. High temperature helps eliminates impurities in bagasse ash. For obtaining amorphous and reactive sugarcane bagasse ash (SCBA), several trials were conducted to define optimum burning time and temperatures. A research conducted by (Ajay G., et al) on the properties and reactivity of sugarcane bagasse ash studied the effect of burning temperature on the ash and concluded that the suitable burning and residence time to be 600^{0}c for 5 hours. The higher the temperature, the higher the amount of silica content, but the resulting silica is in crystalline (less reactive) form rather than in amorphous (more reactive).

Chapter 3

Materials and Methods

3.1. Materials

3.1.1. Soil Sample

The soil sample used is a reddish brown lateritic soil obtained at a depth of one meter by a method of disturbed sampling from a borrow pit in Mekenajo, 456km west of Addis Ababa.

Figure 3/1 Mekenajo town (Mekenajo-Dembidolo road upgrading project design review, 2007

The observed minimum and maximum monthly temperatures obtained from Meteorological Department for that area is in the range of 25°c to 30°c. The mean annual rainfall is around 1700mm and rainfall season is form April to October. The topography of the sampling area is mostly mountainous with intervening escarpments and rolling areas (Mekenajo-Dembidolo Road Upgrading Project Design Review, 2007). Geologically the area under consideration comprises of the rock types genesis, schists and basalt which are basis for lateritic soil formation. Generally formation of laterite soils favour rolling slope with good water runoff, distinct rainy season having warm summer (Zelalem, 2005). The soil forming factors such as topography, amount of rain, climate and parent rock seem to favor for laterite formation. Therefore, one can consider the soil sample as laterite.

During soil preparation and testing procedures, the soil's sensitivity to drying was observed. Upon drying aggregation of fine particle was also observed. Through testing of Atterberg limits, the sensitivity of the soil to time of mixing was observed by varying the duration of mixing, which resulted in different results.

Geochemical oxide test was not carried out for this sample, hence the results of (Zelalem A., 2005, Basic Engineering Properties of Lateritic Soils found in Nejo-Mendi Road Construction Area, Welega)) were taken as reference and is shown in Appendix 1. Degree of laterization can be evaluated based on silica-sesquioxide (S-S) ratio given by;

$$S - S = \frac{SiO2}{(Fe2O3 + Al2O3)} \quad \text{.......Eq. 3/1}$$

An S-S ratio of 1.33 or smaller = laterite

An S-S ration of 1.33 to 2.0 = lateritic soil

A S-S ratio of 2.0 or higher = Non-lateritic, tropical soil

From the test result, he found out that the degree of laterization of all samples were below 1.33, therefore soils are all true laterites.

Figure 3/2 Sampling Area

3.1.2. Sugar Cane Bagasse Ash (SCBA)

The SCBA was obtained from Wonji Sugar Factory located in the Eastern Ethiopia in Oromiya Regional State. This organic industrial waste was collected directly from the hopper before it was dumped to a dump site. The ash collected is black in color.

Figure 3/3 View of Bagasse ash disposal site at Wonji Sugar Factory (Meron. 2015)

3.2. Methods

3.2.1. Sample Preparation

After the collection of disturbed soil,, moist sample was transported to the laboratory in plastic bags. Sample preparation was done in accordance with the method stated in AASHTO T87-86. Soil samples were air dried by spreading the material out in trays in the laboratory and leaving it open in the air for at least 10 days.

3.2.2. Moisture Content

In-situ moisture content was determined according to AASHTO T265-93 (2000); Blight, 1997, CIRIA, 1995. Tropical soils contain loosely bound water of hydration or molecular water, which can be lost at a high temperature resulting a change in the soil characteristics (Bowels, 1978). A. B. Fourie recommends that by using drying oven temperature of 105^{0}c and 50^{0}c, , moisture variation 4-6% or more indicates the presence of structural water (Blight, 1997). Therefore, two sets of samples were taken for moisture determination. One set were dried to constant weight using drying oven at temperature of 105^{0}c, and the other at a temperature of 50^{0}c. till a successive measurement showed constant value.

3.2.3. Particle size Analysis

The size of particles that constitute soils may vary from that of boulders to clay (Zelalem, 2005). The aim of the test is to determine the relative properties of different grain sizes which make up a soil mass. For coarser soil particles, test is conducted mechanically using nest of sieves. However, hydrometer method is used for finer soils. For the soil sample collected both mechanical and hydrometer testing methods were done.

Wet soil preparation was carried out on the moist soil sample for grain size analysis according to the procedures mentioned in AASHTO T146-96; Blight, 1997.

3.2.4. Atterberg Limits Tests

Atterberg limits are carried out to determine the consistency of fine-grained soils. The atterberg limits depend on the type of predominant mineral in the soil. If montmorillonite is the predominant mineral, the liquid limit can exceed 100% because the bond between the layers in montmorillonite is weak and large amounts of water can easily infiltrate the spaces between the layers. However, in the presence of kaolinite, the layers are held relatively tighter and water cannot easily infiltrate between the layers compared to montmorillonite. Therefore, liquid limit value will be much lower.

Morin W.J and Todor P.C. indicated that when liquid limit tests are carried out on lateritic soil samples, the aggregation of clay particles will be broken down by the manipulation process, this leads to difficulties in obtaining consistent values for the liquid limit (Lyon, 1971).

For the determination of Atterberg limits, procedures given in AASHTO T89-02 and AASHTO T90-00 were used for liquid limit and plastic limit respectively. The sample was air-dried, also time of mixing was also limited to 5 minutes. Fresh materials were used for each moisture content point in accordance with Blight, 1997.

3.2.5. Soil Classification

Even though there are other suggested classification systems for residual soils, the AASHTO classification system is convenient as a basis for classifying tropically weathered soils (Lyon, 1971). Therefore, AASHTO classification system was used to classify the soil sample under consideration; hence, AASHTO M145-91 was used. The soil was also classified according to the USCS method.

3.2.6. Specific Gravity

Specific gravity of the soil sample was determined using AASHTO T100-95 procedures. It is used to calculate parameters such as void ratio, porosity, particle size distribution using hydrometer, and degree of saturation (Zelalem, 2005). It is determined by using a soil sample passing No. 10 sieve and a pycnometer.

3.2.7. Shrinkage Limit

A soil mass shrinks when it gradually looses moisture. Any loss of water is accompanied by change in bulk volume. Shrinkage limit is the water content at which a soil changes from a solid to a semisolid state without further change in volume. This test was carried out in accordance to BS 1377 procedure. Linear shrinkage (LS) can be calculated as;

$$LS = \frac{Lo - Lf}{Lo} * 100 \qquad \text{Eq. 3/2}$$

Where; LS is linear shrinkage

Lo is length of wet soil bar

Lf is length of dry soil bar

Linear shrinkage was determined for the natural sample as well as for the stabilized samples.

3.2.8. Free Swell Test

To study the swelling property of the soil, the simplest test conducted is free swell test. This test was suggested by Holtz and Gibbs in 1995 to measure the expansive potential of cohesive soils (Meron, 2015). The test is performed by slowly pouring $10cm^3$ of dry soil which passed through a sieve openoing of 0.425mm (No. 4 sieve) into a 100 cm^3 graduated cylinder filled with distilled water. In this case tap water was used because of the unavailability of distilled water. Final volume of suspension is read after 24hrs.

Free swell is calculated as,

$$\text{Free swell} = \frac{\text{Final volume} - \text{Initial volume of the soil}}{\text{Initial volume}} * 100\% \quad \text{...Eq. 3/3}$$

3.2.9. Compaction

Knowledge of the optimum moisture content and the maximum dry unit weight of soils is very important for construction specifications of soil improvement by compaction (Budhu M., 2007). Compaction increases the strength, lowers the compressibility, and reduces the flow rate of water of a soil by rearranging its fabrics. The moisture content at which the maximum dry unit weight is obtained is referred to as the optimum moisture content. Adding water beyond the optimum value will reduce density.

The effect of bagasse ash content on the maximum dry density (MDD) and optimum moisture content (OMC) of lateritic soil was investigated on stabilized soils.. The samples were compacted according to AASHTO T099-95 (Standard Proctor test). Fresh soil samples were used for each point on the compaction curve. The samples were mixed with SCBA and wrapped with plastic bags and left to cure for 24 hrs prior to testing. The amount of soil sample for each point on the compation curve was taken as 3 kg on dry basis.

3.2.10. California Bearing Ration (CBR)

California Bearing Ratio Test was developed by the California Division of Highways in 1929 as a means of classifying the suitability of a soil for use as subgrade or base course material in highway construction (Zelalem, 2005). The test measures the shearing resisitance of a soil under controlled moisture and density conditions; usually at OMC and the corresponding MDD relevant to field compaction value.

The CBR test was carried out in accordance to AASHTO T193-93. The one-point CBR test was used. Both soaked and unsocked procedures were used for the stabilized and unstabilized soil specimen. Soil samples were mixed with respective percentage of SCBA and left to cure for 3 and 7 days. Some of the samples were soaked in water for 24hrs after curing.

Chapter 4

Test Results and Discussions

4.1. Properties of the natural soil

The properties of the natural soil are summarized in table 4/1 below. The natural moisture content was determined by the two methods mentioned in section 2.3.2., and a moisture variation was found to be less than 4-6%. This indicates that the soil does not contain loosely bound water of hydration. Consequently, a drying temperature of 105^{0}c was used for other tests.

The soil has a liquid limit of 42.5%, plastic limit of 29.44% and plasticity index of 13.06%. According to Muni Budhu, 2007, these are typical values for clays. Besides, the values are in the range of clays containing Kaolinite. (Braja M. Das, 2004).

Table 4/1 Properties of the natural soil

Properties	Quantity
Natural Moisture Content, %	18.36
Liquid Limit, %	42.50
Plastic Limit, %	29.44
Plasticity Index, %	13.06
Specific Gravity	2.67
Percent Passing No. 200 sieve	73.88
Group Index	10
AASHTO classification	A-7-6
Unified classification	ML (Silt with low plasticity)
Free swell, %	45
Maximum Dry Density, g/m^2	1.58
Optimum Moisture Content, %	24.75
Soaked CBR, %	2.43
Unsoaked CBR, %	3.55
Color	Reddish brown

The particle size distribution curve below shows that about 73.88% of the soil passes through No. 200 sieve. AASHTO classification system puts the soil under A-7-6 group, which are fair to poor when used as subgrade materials. Classification according to USCS shows that the soil lies under the group ML (inorganic silt with low plasticity).

The specific gravity of the soil was found to be 2.67 and lies within the range of 2.6-2.9 that is for clayey and silty soils according to Braja M. Das, 2002.

The soil has a maximum dry density of 1.58 Mg/m^2 and optimum moisture content of 24.75%. The soaked and unsoaked CBR value of the soil was found to be 2.43 and 3.55 respectively. According to Bowles, 1992, soil having 0-7% CBR values are very poor subgrade materials. They are considered as unstable subgrades and needs to be stabilized. Therefore, to use this soil as a subgrade material in road construction, the need arises to stabilize the soil in order to get an increase strength.

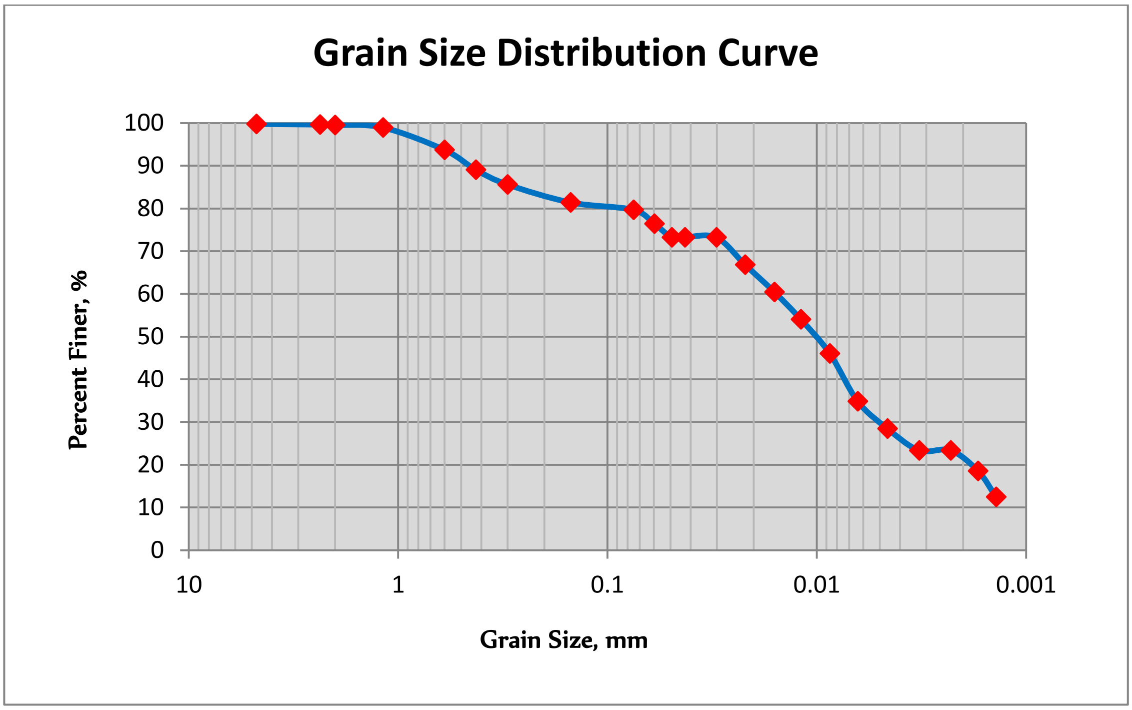

Figure 4/1 Plot of grain size distribution curve of the natural soil

4.2. Properties of Bagasse Ash

The geochemical (oxide) tests are carried out to know quantitatively main oxides of the bagasse ash. The result of the oxide composition was adapted from Hailu B., 2011, who brought the bagasse ash from the same place, Wonji Sugar Factory. The results are shown in the following table 4/2.

The bagasse ash can be assigned as a class N pozzolan, as per ASTM C618 classification since the sum of SiO_2 Al_2O_3, and Fe_2O_3 content is greater than 70% (Hailu B., 2011). The loss in ignition (LOI) value was found to be slightly greater than that specified by the same standard, i.e. 10%.

Table 4/2 Detectable oxide composition of bagasse ash (Hailu B., 2011)

Constituents	Oxide Composition,
SiO_2	65.58
Al_2O_3	5.87
Fe_2O_3	4.32
CaO	1.78
MgO	1.23
Na_2O	1.02
K_2O	6.41
MnO	0.05
TiO_2	025
P_2O_5	1.35
H_2O	0.2
SO_3	0.18
Cl_2	<0.1
Loss on Ignition	10.48

The fineness of pozzolanic materials plays a major role in chemical reactions. The finer it is the higher the surface area, thus, the higher the reactivity. For this study, SCBA used was pulverized to pass through 63μm.

4.3. Effect of SCBA on Atterberg Limits

For the determination of atterberg limits values for the natural and stabilized sample, SCBA was pulverized and added in 2% increment. The mixed soil was cured overnight in a plastic bag before the actual test was carried out.

The introduction of bagasse ash into the soil caused a general increase in liquid limit and plasticity index. This can be due to the addition of bagasse ash treatment which introduced more pozzolanic substance into the soil that required more water for hydration to be completed (J. A. Sadeeq et al, 2015)

Table 4/3 Atterberg limits for different percentage of soil-SCBA mixture

	SCBA, %	LL	PL	PI
Natural	0	42.50	29.44	13.06
Stabilized	2	47.20	30.11	17.09
	4	46.60	27.80	18.80
	6	47.25	30.83	16.42
	8	47.20	29.03	18.17
	12	44.87	27.80	17.07
	16	45.70	32.35	13.35
	20	48.15	28.18	19.97

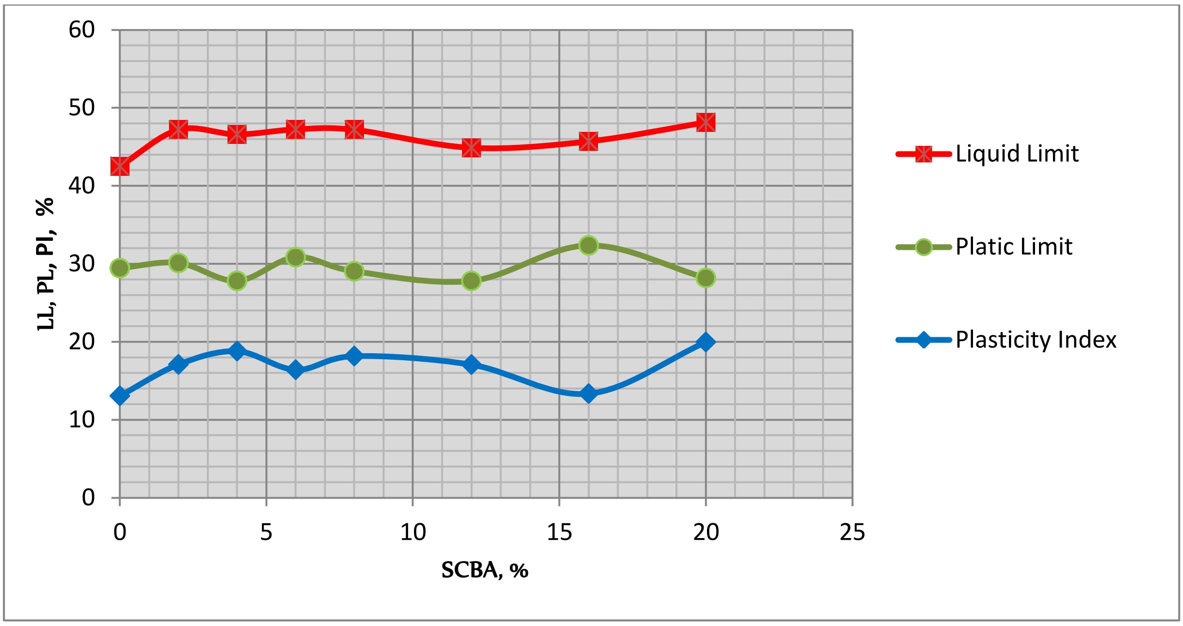

Figure 4/2 Plot of liquid limit, plastic limit and plasticity index for soil-SCBA mixture

Plasticity Chart

Plasticity Index which is the numerical difference between liquid limit and plastic limit, represents the range of water content over which the soil deforms plastically.

Experiment results from soils tested from different parts of the world were plotted on a graph of plasticity index versus liquid limit. It was found that clays, silts, and organic soils lie in distinct regions of the graph (Budhu, 2007) A line called the “A-line”, defined by Eq. 6/1 delineates the boundaries between clays (above the line) and silts and organic soils (below the line).

PI = 0.73 (LL – 20) %.. Eq. 4/1

A second line called the U-Line, expressed by Eq. 6/2, defines the upper limit of the correlation between plasticity index and liquid limit (Budhu, 2007). Since results above this line indicate erronneous values, repeating the test is recommended.

PI = 0.90 (LL – 8) % .... Eq. 4/2

From figure 4/4 the test results are all bellow the U-Line, and are considered acceptable. A value just below the Á-Line’shows that there is a mineral content of Kaolinite (Lyon, 1971). Furthermore, lateritic soils which plot below the A-line are likely to be troublesome (Gidigasu, 1976). The soil sample seems to be in conformity with these.

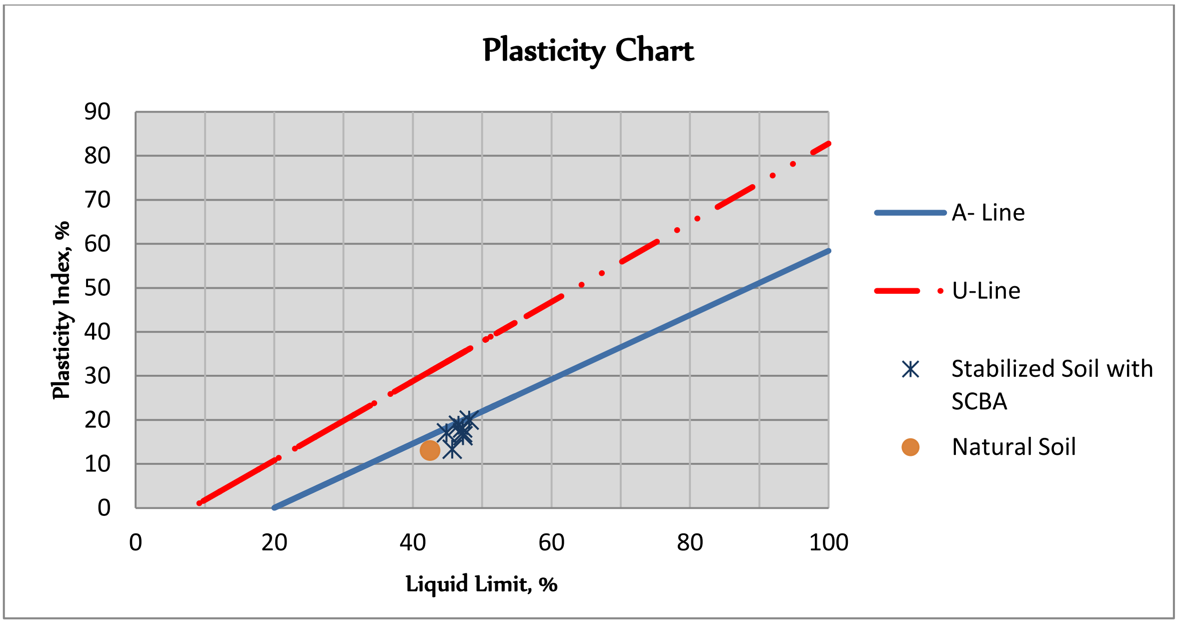

Figure 4/3 Plasticity Chart

Activity Number (A_c)

The change in volume of a clay soil during shrinkage or swelling is a function of plasticity index and the quantity of colloidal clay particles present in the soil. Skempton observed that the plasticity index of a given soil is directly proportional to the percent clay-size fraction (percent finer than 0.002mm).

$$Ac = \frac{PI}{C} \qquad \text{Eq. 4/3}$$

Where C is percent of clay-size fraction by weight.

Activity number have been used as an index property to determine the swelling potential of clays (Das, 1997). Soil classification according to their activity numbers are shown in table 4/4 below.

Table 4/4 Degree of Colloidal Activity

Activity Number, A_c	Soil Type
< 0.75	Inactive
0.75 – 1.25	Normal
> 1.25	Active

The activity number for the soil sample for PI value of 13.06% and clay-sized fraction of 23.34% is **0.56.** Hence, Skempton's colloidal activity number for the soil sample is less than 0.75. This result is in agreement with Morin W.J. and Todor P.C. that lateritic clay soils are classified as normal or inactive (Lyon, 1971).

4.4. Effect of SCBA on Shrinkage Limit

Linear shrinkage was determined for the natural sample as well as for the stabilized samples. The results are shown below.

Table4/5 Linear shrinkage test results for soil SCBA-mixture

Linear Shrinkage, LS (%)	SCBA
Natural	5.4
2% SCBA	7.8
4% SCBA	7.8
6% SCBA	7.1
8% SCBA	7.8

From the test results, one can see that the addition of baggase ash increased the linear shrinkage by 1-3%, which maybe due to the fact that bagasse ash, being an organic material, have high affinity to water therefore, requiring more mixing water, hence resulting in higher shrinkage.

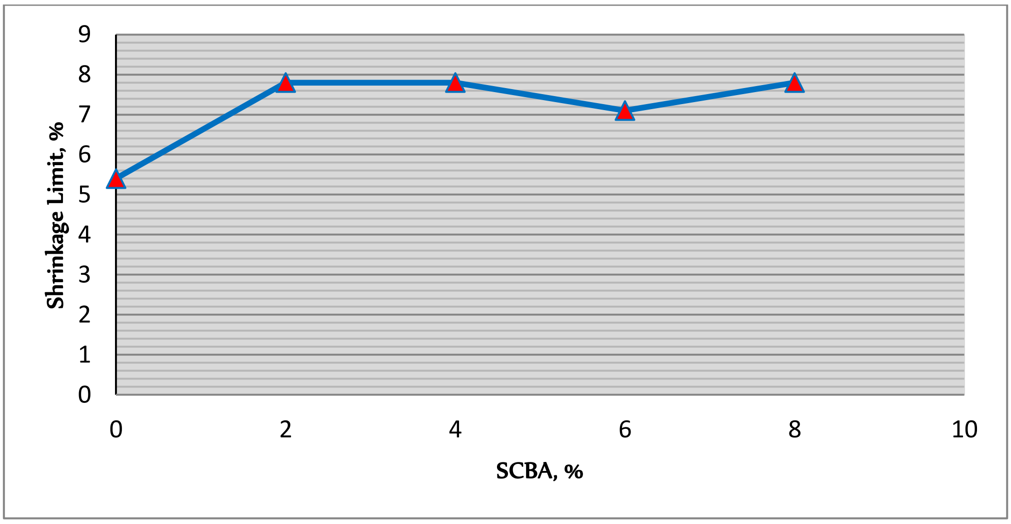

Figure4/ 4 Variation of shrinkage limit with SCBA content

4.5. Effect of SCBA on Free Swell

The SCBA used for free swell test was ash passing No. 4 (0.425mm) seive. It was difficult to perform the test for the finner ash because the ash was not settling, therefore it was difficult to read the final volume. Free swell test was done for soil-SCBA mixture and also for the ash alone.

Table4/6 free swell result for soil-SCBA mixture

% SCBA	Free Swell
0	45.0
2	30.0
4	30.5
6	37.5
8	70.0
SCBA	50.0

It can be seen that 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.

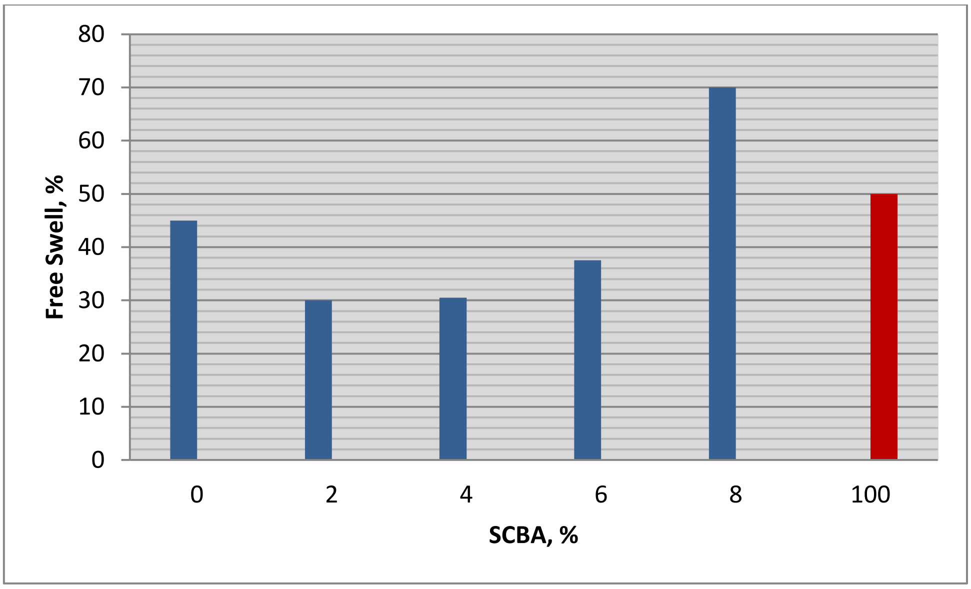

Figure4/ 5 Variation of free swell with SCBA content

4.6. Effect of SCBA on Compaction Characteristics

The effect of bagasse ash content on the maximum dry density (MDD) and optimum moisture content (OMC) of lateritic soil was investigated on stabilized soils (0, 2, 4, 6, and 8% SCBA by dry weight of natural lateritic soil). Fresh soil samples were used for each point on the compaction curve. The samples were mixed with SCBA and wrapped with plastic bags and left to cure for 24 hrs prior to testing. The amount of soil sample for each point on the compation curve was taken as 3 kg on dry basis.

Table4/7 Compaction test results for different percentage of soil-SCBA mixture

	SCBA, %	OMC, %	MDD, Mg/m^3
Natural	0	24.5	1.58
Stabilized	2	22.3	1.59
	4	25.3	1.54
	6	24.3	1.57
	8	25.5	1.53
	16	27.0	1.50

The effect of bagasse ash content on the maximum dry density (MDD) and optimum moisture content (OMC) of the lateritic soil is shown in the Fig. The MDD shows a general decrease and OMC shows an increase in value with higher bagasse ash content.

The small drop in MDD could be a result of the flocculation and agglomeration of fine particles, caused by cation exchange, occupying larger spaces leading to coresponding decrease in dry density. Also, the drop in density with higher stabilizer content maybe because the bagasse ash has less specific gravity than the soil. The bagasse ash may also act as a filler in the voids.

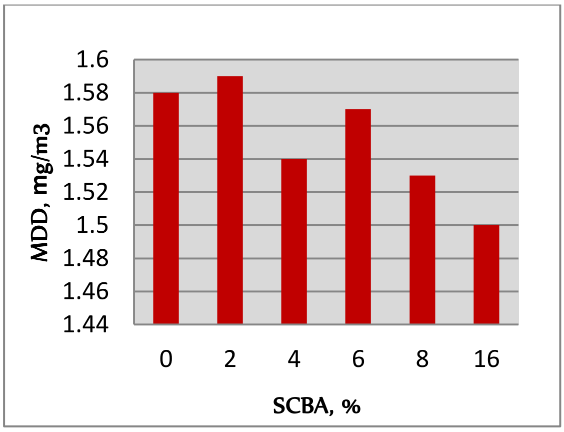

Figure4/ 6 Variation of MDD with SCBA content

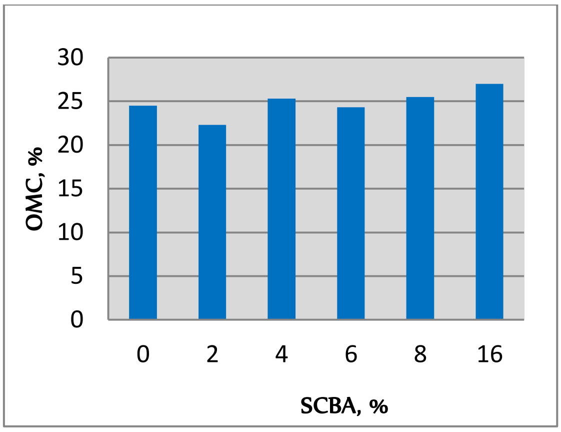

Figure4/ 7 Variation of OMC with SCBA content

The variation of OMC with bagasse ash content shows a small increase from the unstabilized soil with higher bagasse ash content. This may be due to the high affinity of the bagasse ash to water.

Decrease in MDD and increase in OMC was also reported by the following;

Researcher (Author)	**Research Title**
Amu, O.O., et al	Geotechnical properties if Lateritic soil stabilized with sugarcane straw ash
K/J. Osinubi et al	Bagasse Ash Stabilization of Lateritic Soil

Ogunribido T.H.T.	Potential of Sugar Cane Straw Ash for Lateritic Soil Stabilization in Road Construction
Adrian, Oshioname Eberemu	Effect of Bagasse Ash on the Hdraulic Properties of Laterite

4.7. Effects of SCBA on California Bearing Ratio

Soil samples were mixed with respective percentage of SCBA and left to cure for 3 and 7 days. Some of the samples were soaked in water for 24hrs after curing. The CBR values of the soaked and unsoaked stabilized samples, cured for 3 and 7 days are showed in the table 4/7 below.

Curing Day	SCBA, %	Soaked CBR	Unsoaked CBR
3	0	2.43	3.55
	4	2.19	
	8	1.71	6.84
	16	0.9	
7	0	2.66	3.42
	4	1.81	
	8	1.68	5.03
	16	1.16	

Table 4/8 CBR test result for different percentage of soil-SCBA mixture

It can be seen from the result that the CBR values of the soaked samples have decreased with increasing amount of SCBA; however an increase is seen in the CBR value of the unsoaked samples. The latter may be due to the time dependent pozzolanic reaction of the lime in the bagasse ash, with the soluble alumina and silica from the clay, in the presence of water, to produce stable calcium silicate hydrate (CSH), and calcium aluminate hydrates (CAH) which generate long-term strength gain, but the amount of CaO in the ash is very low. Therefore the slight increase is probably due to cation exchange causing flocculation and agglomeration.

The decrease in CBR value for the soaked samples may be due to the effect of the soaking water on the soil-SCBA bond. The pozzolanic reaction created during curing might have reversed when the sample is soaked in water.

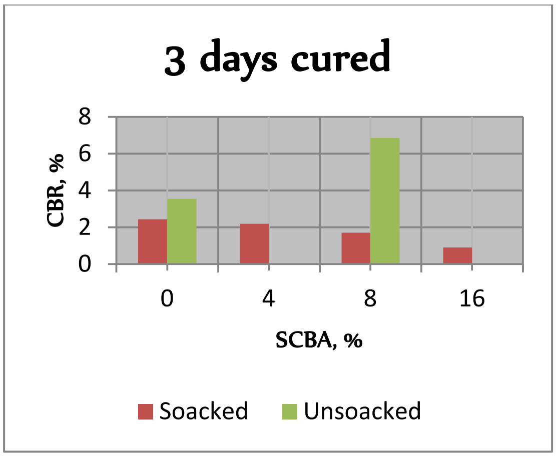

Figure 4/8 Variation of CBR value cured for 3 days with SCBA content

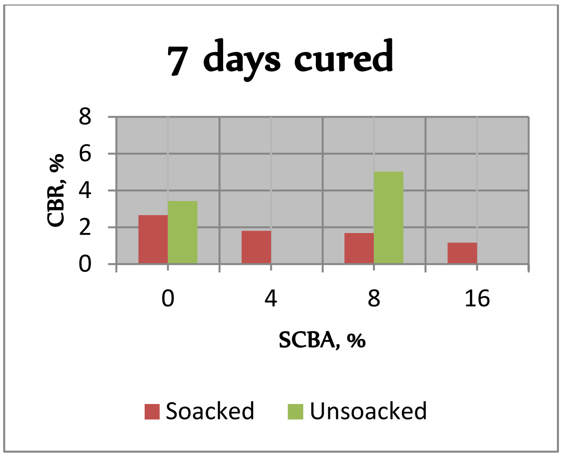

Figure 4/ 9 Variation of CBR value cured for 7 days with SCBA content

Chapter 5

Comparisons of Test Results

5.1. Comparison of test results with Laterites and Lateritic Soils of Africa

A study was done by Lyon Association Inc. on soils of tropical Africa with an emphasis on Ghana. The samples were collected from different parts of Africa such as Ghana, Ethiopia, Kenya, Uganda, Niger, etc... The soil samples from Ethiopia were classified, according to D' Hoore's classification system, as ferrisol.

Table 4/9 to 4/11 shows the average values of various tests done for different countries.

Table 5/1 Typical soil test results for Feruginous Soils. (Lyon, 1971)

Country	AASHTO	GI	LL %	PL %	PI %	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm
Ghana	A-2-6	0	26	15	11						45	20
Senegal	A-2-7	0	39	20	19	95	80	68	46	33	27	20
Upper Volta	A-2-6	1	38	14	24	100	88	80	66	51	38	25
Niger	A-2-4	0	21	11	10	97	79	73	64	55	40	25
Tanzania	A-2-6	6	34	19	15	100	100	100	100	100	93	61
Kenya	A-2-7	0	45	31	14	100	97	94	88	52	40	28
Uganda	A-2-6	2	38	17	22	100	99	96	83	61	51	34
Sudan	A-2-4	0	21	12	9	100	100	100	100	98	5	27
Gambia	A-2-6	0	36	16	20	98	60	53	42	34	28	22

Table 5/2 Typical soil test result for Ferralitic Soils (Lyon, 1971)

Country	AASHTO	GI	LL %	PL %	PI %	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm
Ghana`	A-6	6	38	18	20	100	100	100	100	95	81	67
Liberia	A-2-7	2	56	29	27	100	95	84	72	57	41	36
Gabon	A-2-4	0	35	18	17							
Sierra Leone	A-2-6	1	55	31	24	100	98	95	90	68	37	29
Burundi	A-6		31	16	16	100	100	96	92	84	76	74
Dahomey	A-2-7	4	4	21	24	100	100	100	99	85	72	55
Ivory Coast	A-7-6	22	62	31	31	100	100	100	100	100	99	88
Mali	A-6	3	35	21	14	100	100	99	89	67	55	51
Uganda	A-6	2	39	19	20	100	100	96	91	82	73	53

Table 5/3 Typical soil test results for Ferrisol Soils

Country	AASHTO	GI	LL %	PL %	PI %	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm
Ghana	A-7-6	3	53	34	19							50
Niger	A-2-6	0	28	16	12	100	100	100	98	82	68	39
Ivory Coast	A-7-6	18	48	24	24	100	99	92	83	65	60	50
Mali	A-7-5	3	55	31	24	100	100	100	89	61	51	43
Uganda	A-2-7	0	46	21	25	100	100	97	91	56	31	24
Kenya	A-7-5	27				100	100	100	100	100	99	98
Cameroon	A-7-5	19	65	37	27	100	100	100	100	98	97	88
Ethiopia	A-7-5	19	68	33	35	100	100	100	98	84	63	62
Ghana	A-7-6	12	57	25	32							65
Soil Sample (Mekenajo)	A-7-6	10	43	29	13	100	100	100	99	98	94	73

The soil sample from Mekenajo, when compared with the previously tested laterite and lateritic soils of Africa show similarities with soils from the group Ferrisol. The soil sample is also compared to some lateritic soils from Western Ethiopia.

Table 5/4 Some index test results of lateritic soils of Western Ethiopia

Area	AASHTO	GI	LL, %	PL, %	PI, %	25 mm	19 mm	9.5 mm	4.75 mm	2 mm	0.425 mm	0.075 mm
Soil Sample	A-7-6	10	43	29	13	100	100	100	99	98	94	73
Nejo-Mendi	A-7-5	18	59	39	20	100	100	100	100	100	85	76
Nejo-Mendi	A-7-5	17	54	34	20	100	100	100	100	100	90	80
Assosa	-	-	40.6	23.5	17.1	-	-	-	-	-	-	-
Assosa	-	-	46.3	28.9	17.4	-	-	-	-	-	-	-
Assela	A-7-6	20	61	32	29	100	97	96	95	85	60	88

Chapter 6

Conclusion and Recommendation

6.1. Conclusions

An experiment was undertaken to investigate the effects of sugar cane bagasse ash on some geotechnical characteristics of a lateritic soil. The following conclusions can be drawn from the results of the study;

1. 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.
2. The liquid limit and plasticity index showed a general increase with increasing amount of bagasse ash.
3. The shrinkage limit and the free swell also showed an increase with increment of the bagasse ash.
4. The maximum dry density and optimum moisture content decreased and increased respectively with the increment of SCBA. However, the changes observed were insignificant.
5. The soaked and unsoaked CBR values decreased and increased respectively with the increment of bagasse ash. Increase of curing period showed an insignificant effect on the CBR values of the stabilized soils.
6. The bagasse ash had very little content of lime (CaO) and same chemical composition as the lateritic soil sample. As a result, the addition of the ash did not bring a significant change on the strength of the soil.
7. Based on the results discussed above, sugarcane bagasse ash was not an effective stabilizer for improving the geotechnical properties of the lateritic soil sample.

7.2. Recommendations

1. Effects of pre-treatment on the stabilized soil was not the scope of this study, therefore further study can be made on the effects of pre-treatment on the soil-SCBA mixture.
2. The effect of bagasse ash on lateritic soil with an addition of a lime (CaO) activator should be studied.
3. The effect of different incineration temperature on the reactivity of bagasse ash should be studied and the suitable range of temperature identified for effective stabilization.
4. The effect of bagasse ash on soils treated with other chemical stabilizers such as lime and cement should be studied.
5. The effect of different degree of fineness of the bagasse ash on soil stabilization should be studied.

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Appendix 1

The geochemical test results done by Zelalem A. for his thesis "Basic Engineering Properties of Lateritic Soils found in Nejo-Mendi Road Construction Area, Wolega" are shown in the table below.

Table A1 Oxide Composition in Percent. (Zelalem A., 2005)

Designation	Sampling depth [m]	SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	k_2O	MnO	H_2O	LOI	Ti_2O	P_2O_5	$\frac{SiO_2}{R_2O_3}$
Sp-1	0 ~ 0.60	25.46	23.76	32.40	< 0.01	< 0.01	< 0.01	0.02	0.05	0.92	12.69	3.27	0.28	0.45
Sp-2-1	0.50 ~ 1.50	30.10	27.81	22.75	< 0.01	< 0.01	< 0.01	0.02	0.05	0.37	14.36	2.83	0.34	0.60
Sp-2-3	2.50 ~ 3.00	19.90	31.57	25.44	< 0.01	< 0.01	< 0.01	< 0.01	0.07	1.49	15.98	4.46	0.37	0.35
Sp-3-1	0.40 ~ 1.40	20.20	23.43	36.60	< 0.01	0.03	< 0.01	0.06	0.22	0.20	14.36	2.90	0.58	0.34
Sp-3-2	1.40 ~ 2.50	21.62	26.12	31.90	< 0.01	0.09	< 0.01	0.14	0.17	0.80	14.17	3.23	0.47	0.37
Sg-1	0.50 ~ 1.50	32.59	28.79	16.97	< 0.01	0.02	< 0.01	< 0.01	0.14	1.22	13.35	3.07	0.22	0.71

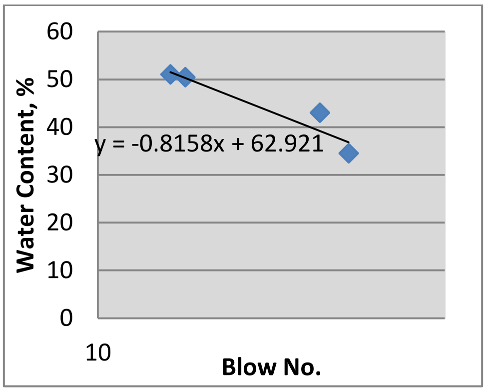
Appendix 2

Atterberg Limits and Free Swell

Natural Soil

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
C1	15.2	36.8	29.5	7.3	14.3	51.0489	14
C2	14.2	30.3	24.9	5.4	10.7	50.467	15
C27	14.1	26.4	22.7	3.7	8.6	43.023	28
R3	15.5	28.55	25.2	3.35	9.7	34.536	32
						LL = 42.50%	



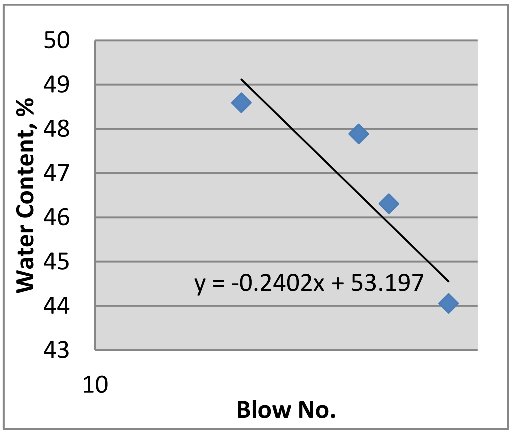
Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
C9	14	15.2	14.9	0.3	0.9	33.333
H4	15.6	16.9	16.6	0.3	1	30
MA10	15.6	16.6	16.4	0.2	0.8	25
						PL = 29.44%

Soil+ 2% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
M1	10.9	32	25.1	6.9	14.2	48.5915	17
G1	15.5	36.5	29.7	6.8	14.2	47.8873	26
A3	15.6	37.4	30.5	6.9	14.9	46.3087	29
R3	15.5	36.1	29.8	6.3	14.3	44.0559	36
						LL = 47.20%	



Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
T2	11.5	17	15.7	1.3	4.2	30.9524
C3	15.7	21	19.8	1.2	4.1	29.2683
						PL = 30.11%

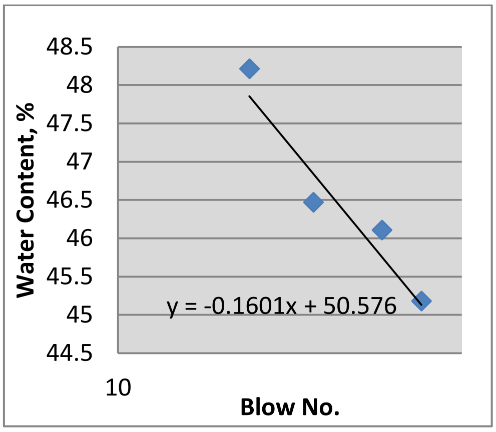
Soil+ 4% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
R	15.5	40.4	32.3	8.1	16.8	48.21429	17
B1	15.5	40.4	32.5	7.9	17	46.47059	22
21	15.2	39.6	31.9	7.7	16.7	46.10778	29
LL3	15.3	39.4	31.9	7.5	16.6	45.18072	34
						LL = 46.60%	

Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
C27	14.1	18.8	17.8	1	3.7	27.02703
A1	15.6	21	19.8	1.2	4.2	28.57143
						PL = 27.80%



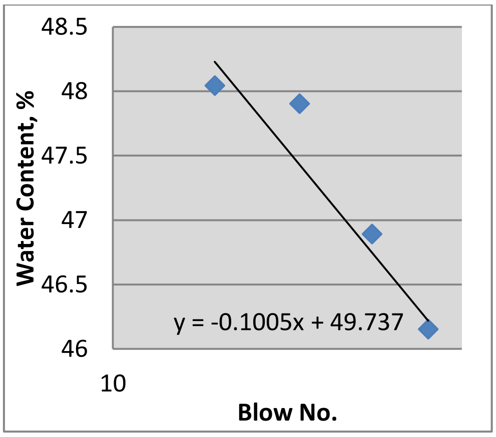
Soil+ 6% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
T12	15.7	42.2	33.6	8.6	17.9	48.04469	15
B4	15.7	40.4	32.4	8	16.7	47.90419	21
W2	15.4	41.4	33.1	8.3	17.7	46.89266	28
A3	15.7	44.2	35.2	9	19.5	46.15385	35
						LL = 47.25%	

Plastic Limit

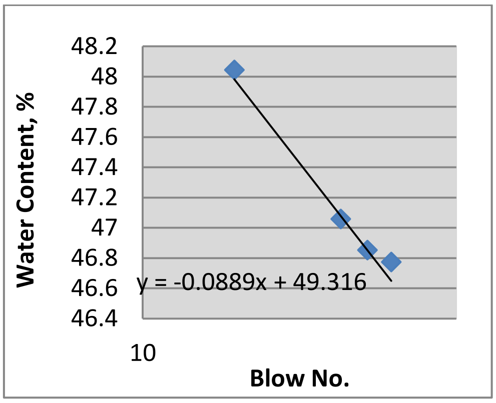
Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
C2	15.6	20	19	1	3.4	29.41176
J3	15.6	19.7	18.7	1	3.1	32.25806
						PL = 30.83%



Soil+ 8% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
T12	15.7	42.2	33.6	8.6	17.9	48.04469	15
P5	15.2	40.2	32.2	8	17	47.05882	24
M2	15.5	36.5	29.8	6.7	14.3	46.85315	27
R2	15.7	43	34.3	8.7	18.6	46.77419	30
							LL = 47.20%



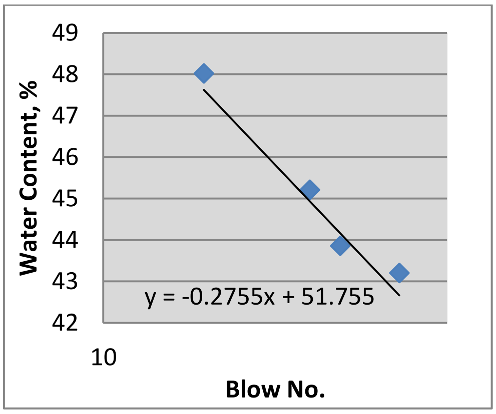
Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
D4	15.7	20.2	19.1	1.1	3.4	32.35294
G2	15.7	20.1	19.2	0.9	3.5	25.71429
						PL = 29.03%

Soil+ 12% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
R	15.4	41.6	33.1	8.5	17.7	48.0226	15
J74	15.8	43.1	34.6	8.5	18.8	45.21277	23
A2	15.7	40.3	32.8	7.5	17.1	43.85965	26
R4	15.7	45.2	36.3	8.9	20.6	43.20388	33
							LL = 44.87%



Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
H11	15.6	19.7	18.7	1	3.1	32.25806
C1	10.9	14.6	13.9	0.7	3	23.33333
						PL = 27.80%

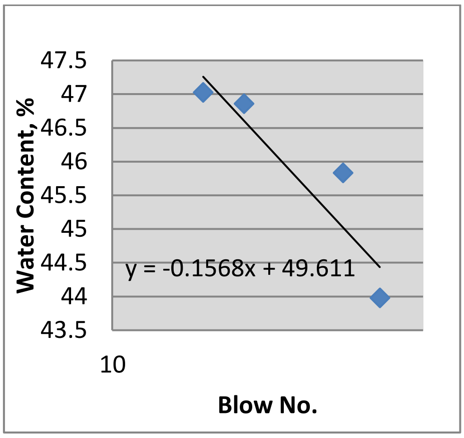
Soil+ 16% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
C3	15.8	43	34.3	8.7	18.5	47.02703	15
G1	15.8	46.2	36.5	9.7	20.7	46.8599	18
I2	14.8	42.8	34	8.8	19.2	45.83333	28
T3	15.5	43	34.6	8.4	19.1	43.97906	33
						LL = 45.69%	

Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
21	15.2	19.6	18.6	1	3.4	29.41176
24	15.7	20.3	19.1	1.2	3.4	35.29412
						PL = 32.32%



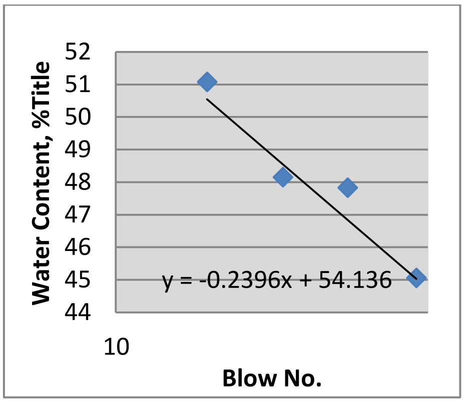
Soil+ 20% SCBA

Liquid Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content	blow no
H10	15.4	43.5	34	9.5	18.6	51.07527	15
D1	15.7	35.7	29.2	6.5	13.5	48.14815	21
B2	11	34.8	27.1	7.7	16.1	47.82609	28
C5	15.5	41.9	33.7	8.2	18.2	45.05495	38
						LL = 48.45%	

Plastic Limit

Con no.	Can wt.	can wt. + wet soil	can wt. + dry soil	mass of water	mass of dry soil	water content
C1	15.1	19.8	18.7	1.1	3.6	30.55556
C2	15.5	19.4	18.6	0.8	3.1	25.80645
						PL = 28.18%



Free Swel Test Results

Natural Soil

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	14	15
Free Swell, %	40	50
Avg. Free Swell, %		45

Soil + 2% SCBA

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	13	13
Free Swell, %	30	30
Avg. Free Swell, %		30

Soil + 4% SCBA

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	12.1	14
Free Swell, %	21	40
Avg. Free Swell, %		30.

Soil + 6% SCBA

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	13.5	14
Free Swell, %	35	40
Avg. Free Swell, %		37.5

Soil + 8% SCBA

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	17	17
Free Swell, %	70	70
Avg. Free Swell, %		70

100% SCBA

Trial	1	2
Initial Volume, ml	10	10
Final Volume, ml	15	15
Free Swell, %	50	50
Avg. Free Swell, %		50

Appendix 3
Compaction Test Results

Natural Soil

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	23	25
actual avg water content	13.33333333	15.50633	24.26778	26.44231	29.71014
mass of compacter soil and mold	6076.2	6164.1	6361.4	6352.8	6305.3
mass of mold	4511.1	4509.3	4508.9	4510.4	4510.4
wet mass of soil in mold	1565.1	1654.8	1852.5	1842.4	1794.9
wet density,g/cm^3	1.657944915	1.752966	1.962394	1.951695	1.901377
dry density	1.462892572	1.517636	1.579166	1.543546	1.465866
compacted soil sample no	1	2	3	4	5
moisture can no	P4	P5	A1	MA10	H
Mc=mass of empty clean can	15.6	15.2	15.6	15.6	15.7
Mcms=mass of can+moist soil	34.3	51.7	45.3	41.9	33.6
Mcds=mass of can +dry soil	32.1	46.8	39.5	36.4	29.5
Ms=mass of soil solids	16.5	31.6	23.9	20.8	13.8
Mw=mass of pore water	2.2	4.9	5.8	5.5	4.1
water content	13.33333333	15.50633	24.26778	26.44231	29.71014
					OMC = 24.5%, MDD = 1.58

Soil + 2% SCBA

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	23	25
actual avg. water content	14.28571	17.00879765	21.92691	26.11684	31.06061
mass of compacter soil and mold	6100.6	6208.8	6335.7	6360.9	6292.8
mass of mold	4511.3	4510	4510.5	4511.5	4510.3
wet mass of soil in mold	1589.3	1698.8	1825.2	1849.4	1782.5
wet density,g/cm^3	1.683581	1.799576271	1.933475	1.95911	1.888242
dry density	1.473133	1.537983731	1.585765	1.553409	1.440739
compacted soil sample no	1	2	3	4	5
moisture can no	T1	T5	3G1	G3	MA1
Mc=mass of empty clean can	15.5	15.2	15.7	15.7	15.7
Mcms=mass of can+moist soil	49.1	55.1	52.4	52.4	50.3
Mcds=mass of can +dry soil	44.9	49.3	45.8	44.8	42.1
Ms=mass of soil solids	29.4	34.1	30.1	29.1	26.4
Mw=mass of pore water	4.2	5.8	6.6	7.6	8.2
water content	14.28571	17.00879765	21.92691	26.11684	31.06061
					OMC = 22.3%, MDD = 1.59

Soil + 4% SCBA

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	25	30
actual avg. water content	15.7676348	18.93939	24.82517	28.4507	34.01163
mass of compacter soil and mold	6112.2	6173.9	6319.6	6333.6	6256.3
mass of mold	4510.9	4510.1	4510	4509.2	4509.3
wet mass of soil in mold	1601.3	1663.8	1809.6	1824.4	1747
wet density,g/cm^3	1.696292373	1.7625	1.916949	1.932627	1.850636
dry density	1.465256136	1.481847	1.535707	1.504567	1.380952
compacted soil sample no	1	2	3	4	5
moisture can no	T6	H4	T1	Z3	B2
Mc=mass of empty clean can	15.6	15.4	15.6	15.6	13.9
Mcms=mass of can+moist soil	43.5	46.8	51.3	61.2	60
Mcds=mass of can +dry soil	39.7	41.8	44.2	51.1	48.3
Ms=mass of soil solids	24.1	26.4	28.6	35.5	34.4
Mw=mass of pore water	3.8	5	7.1	10.1	11.7
water content	15.76763485	18.93939	24.82517	28.4507	34.01163
					OMC = 25.3%, MDD = 1.54

Soil + 6% SCBA

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	23	25
actual avg. water content	15.8004158	20.2765	23.88451	25.79853	27.21713
mass of compacter soil and mold	6136.1	6247.5	6350.9	6358.1	6332.9
mass of mold	4511	4510.7	4510.3	4509.7	4510.5
wet mass of soil in mold	1625.1	1736.8	1840.6	1848.4	1822.4
wet density,g/cm^3	1.721504237	1.839831	1.949788	1.958051	1.930508
dry density	1.486613174	1.529668	1.573876	1.556497	1.517491
compacted soil sample no	1	2	3	4	5
moisture can no	T5	M2	LL3	E2	R
Mc=mass of empty clean can	15.2	15.5	15.5	15.6	15.5
Mcms=mass of can+moist soil	70.9	67.7	62.7	66.8	57.1
Mcds=mass of can +dry soil	63.3	58.9	53.6	56.3	48.2
Ms=mass of soil solids	48.1	43.4	38.1	40.7	32.7
Mw=mass of pore water	7.6	8.8	9.1	10.5	8.9
water content	15.8004158	20.2765	23.88451	25.79853	27.21713
					MDD = 24.3%, MDD = 1.57

Soil + 8% SCBA

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	25	30
actual avg water content	13.0526	17.53086	22.54098	25.4041570	28.4574
mass of campacter soil and mold	4692.9	4766.5	6117.8	6572.7	6562
mass of mold	3122.9	3123.2	4409.8	4756.3	4754.8
wet mass of soil in mold	1570	1643.3	1708	1816.4	1807.2
wet density,g/cm^3	1.66313	1.740784	1.809322	1.92415254	1.91440
dry density	1.47111	1.481129	1.476504	1.53436105	1.49030
compacted soil sample no	1	2	3	4	5
moisture can no	A20	F1	A20	A27	C3
Mc=mass of empty clean can	15.7	15.5	15.7	15.6	15.6
Mcms=mass of can+moist soil	69.4	63.1	75.5	69.9	63.9
Mcds=mass of can +dry soil	63.2	56	64.5	58.9	53.2
Ms=mass of soil solids	47.5	40.5	48.8	43.3	37.6
Mw=mass of pore water	6.2	7.1	11	11	10.7
water content	13.05263	17.53086	22.54098	25.4041570	28.4574
					OMC = 25.5%, MDD = 1.53

Soil + 16% SCBA

compacted soil sample no	1	2	3	4	5
assumed water content	12	16	21	25	30
actual avg water content	13.81733021	18.08279	22.90249	27.22646	31.29161
mass of campacter soil and mold	6035.7	6088.8	6176.2	6271.2	6244.4
mass of mold	4473.2	4473.5	4473.7	4473.3	4473.5
wet mass of soil in mold	1562.5	1615.3	1702.5	1797.9	1770.9
wet density,g/cm^3	1.655190678	1.711123	1.803496	1.904555	1.875953
dry density	1.454251892	1.449087	1.46742	1.49698	1.428845
compacted soil sample no	1	2	3	4	5
moisture can no	A20	F1	A20	A27	C3
Mc=mass of empty clean can	15.5	15.8	15.7	15.7	15.7
Mcms=mass of can+moist soil	64.1	70	69.9	65.7	65
Mcds=mass of can +dry soil	58.2	61.7	59.8	55	53.25
Ms=mass of soil solids	42.7	45.9	44.1	39.3	37.55
Mw=mass of pore water	5.9	8.3	10.1	10.7	11.75
water content	13.81733021	18.08279	22.90249	27.22646	31.29161
					OMC = 27.0%, MDD = 1.50

Compaction curves for the different percentage of soil-SCBA mixtures are shown in Figure below.

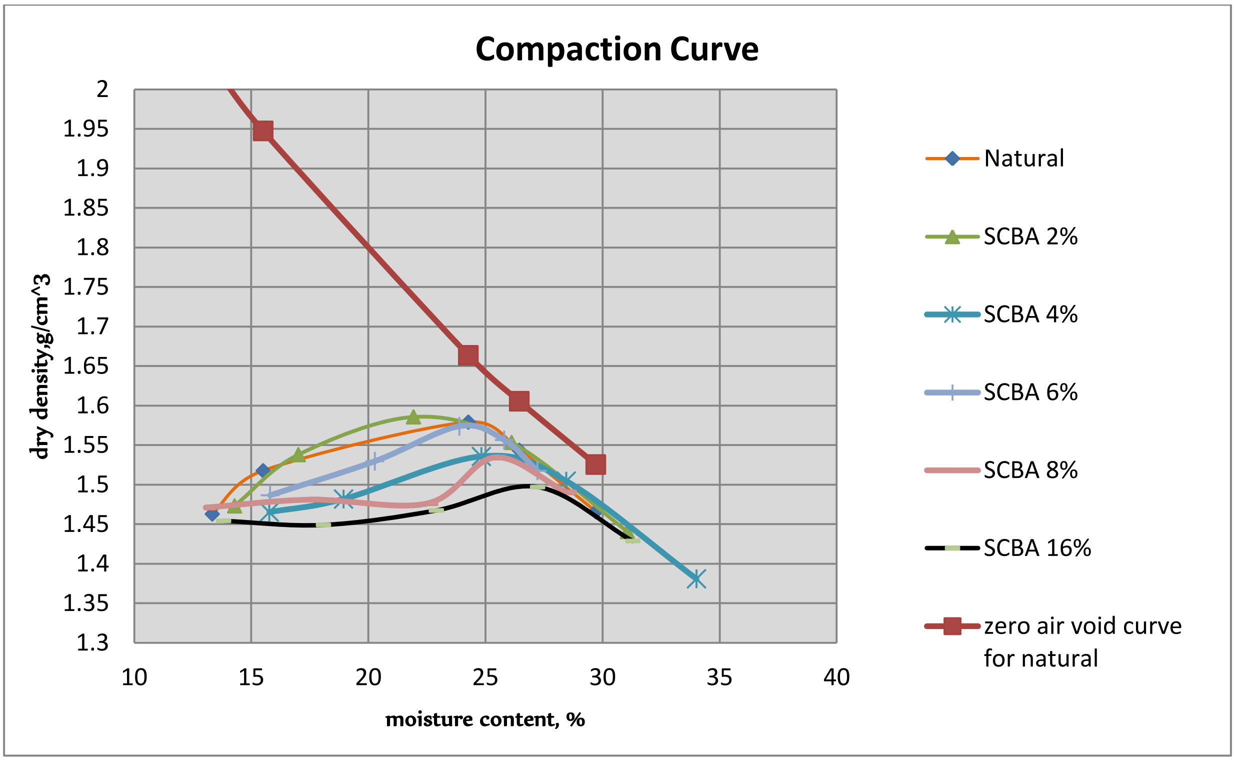


Figure A1 Compaction cure for different soil-SCBA mixture

Appendix 4

California Bearing Ratio Test Results

1. Soacked CBR

Natural Soil (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm^2)	(N/mm^2)	
0.00	0.0	0.00	0.00		
0.64	1.0	25.71	0.01		
1.27	3.0	77.12	0.04		
1.91	5.0	128.54	0.07		
2.54	7.5	192.80	0.12	6.9	1.74
3.18	10.0	257.07	0.13		
3.81	12.0	308.48	0.16		
4.45	15.0	385.61	0.20		
5.08	17.0	437.02	0.25	10.3	2.43
7.62	28.0	719.80	0.37		
10.16	38.0	976.87	0.50		
12.70	45.0	1156.82	0.60		

Natural Soil (7days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm^2)	(N/mm^2)	
0.00	0.0	0.00	0.00		
0.64	1.0	51.41	0.03		
1.27	3.0	115.68	0.06		
1.91	5.0	179.95	0.09		
2.54	7.5	257.07	0.14	6.9	2.03
3.18	10.0	308.48	0.16		
3.81	12.0	359.90	0.19		
4.45	15.0	437.02	0.23		
5.08	17.0	514.14	0.27	10.3	2.62
7.62	28.0	745.50	0.39		
10.16	38.0	976.87	0.50		
12.70	45.0	1131.11	0.58		

Soil + 4% SCBA (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	51.41	0.03		
1.27	3.0	115.68	0.06		
1.91	5.0	154.24	0.08		
2.54	7.5	205.66	0.11	6.9	1.54
3.18	10.0	257.07	0.13		
3.81	12.0	321.34	0.17		
4.45	15.0	385.61	0.20		
5.08	17.0	437.02	0.23	10.3	2.19
7.62	28.0	642.68	0.33		
10.16	38.0	848.33	0.44		
12.70	45.0	989.72	0.51		

Soil +4% SCBA (7days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	59.13	0.03		
1.27	3.0	102.83	0.05		
1.91	5.0	154.24	0.08		
2.54	7.5	179.95	0.09	6.9	1.35
3.18	10.0	231.36	0.12		
3.81	12.0	282.78	0.15		
4.45	15.0	321.34	0.17		
5.08	17.0	359.90	0.19	10.3	1.81
7.62	28.0	539.85	0.28		
10.16	38.0	681.24	0.35		
12.70	45.0	796.92	0.41		

Soil +8% SCBA (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	25.71	0.01		
1.27	3.0	38.56	0.02		
1.91	5.0	77.12	0.04		
2.54	7.5	128.54	0.08	6.9	1.16
3.18	10.0	154.24	0.08		
3.81	12.0	192.80	0.10		
4.45	15.0	231.36	0.12		
5.08	17.0	282.78	0.18	10.3	1.75
7.62	28.0	462.73	0.24		
10.16	38.0	642.68	0.33		
12.70	45.0	796.92	0.41		

Soil + 8% SCBA (7 days cured

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	30.85	0.02		
1.27	3.0	64.27	0.03		
1.91	5.0	102.83	0.05		
2.54	7.5	154.24	0.08	6.9	1.16
3.18	10.0	179.95	0.09		
3.81	12.0	231.3	0.12		
4.45	15.0	282.78	0.15		
5.08	17.0	334.19	0.17	10.3	1.68
7.62	28.0	514.14	0.27		
10.16	38.0	694.09	0.36		
12.70	45.0	822.62	0.43		

Soil + 16% SCBA (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	20.57	0.01		
1.27	3.0	30.85	0.02		
1.91	5.0	51.41	0.03		
2.54	7.5	77.12	0.04	6.9	0.58
3.18	10.0	102.83	0.05		
3.81	12.0	128.54	0.07		
4.45	15.0	154.24	0.08		
5.08	17.0	179.95	0.09	10.3	0.90
7.62	28.0	282.78	0.15		
10.16	38.0	385.61	0.20		
12.70	45.0	462.73	0.24		

Soil + 16% SCBA (7 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	25.71	0.01		
1.27	3.0	51.41	0.03		
1.91	5.0	77.12	0.04		
2.54	7.5	107.97	0.06	6.9	0.81
3.18	10.0	141.39	0.07		
3.81	12.0	167.10	0.09		
4.45	15.0	200.51	0.10		
5.08	17.0	231.36	0.12	10.3	1.16
7.62	28.0	308.48	0.16		
10.16	38.0	411.31	0.51		
12.70	45.0	488.43	0.25		

2. Unsoaked CBR

Natural soil (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	128.54	0.07		
1.27	3.0	205.66	0.11		
1.91	5.0	38.48	0.16		
2.54	7.5	385.61	0.20	6.9	2.89
3.18	10.0	462.73	0.24		
3.81	12.0	547.56	0.28		
4.45	15.0	629.82	0.33		
5.08	17.0	706.94	0.37	10.3	3.55
7.62	28.0	976.87	0.50		
10.16	38.0	1208.23	0.62		
12.70	45.0	1439.59	0.74		

Natural soil (7 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	77.12	0.04		
1.27	3.0	154.24	0.08		
1.91	5.0	231.36	0.12		
2.54	7.5	334.19	0.17	6.9	2.50
3.18	10.0	437.02	0.23		
3.81	12.0	514.14	0.27		
4.45	15.0	591.26	0.31		
5.08	17.0	681.24	0.35	10.3	3.42
7.62	28.0	951.16	0.49		
10.16	38.0	1208.23	0.62		
12.70	45.0	1465.30	0.76		

Soil + 8% SCBA (3 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm²)	(N/mm²)	
0.00	0.0	0.00	0.00		
0.64	1.0	179.95	0.09		
1.27	3.0	321.34	0.17		
1.91	5.0	488.43	0.25		
2.54	7.5	642.68	0.33	6.9	4.81
3.18	10.0	822.62	0.43		
3.81	12.0	1002.57	0.52		
4.45	15.0	1182.52	0.61		
5.08	17.0	1362.47	0.70	10.3	6.84
7.62	28.0	2005.15	1.04		
10.16	38.0	2519.29	1.30		
12.70	45.0	2904.89	1.50		

Soil +8% SCBA (7 days cured)

Penet.	Ring Reading	Load	Stress	Standard stress	CBR (%)
(mm)	(Div.)	(N)	(N/mm^2)	(N/mm^2)	
0.00	0.0	0.00	0.00		
0.64	1.0	174.81	0.09		
1.27	3.0	282.78	0.15		
1.91	5.0	385.61	0.20		
2.54	7.5	514.14	0.27	6.9	3.85
3.18	10.0	642.68	0.33		
3.81	12.0	771.21	0.40		
4.45	15.0	874.04	0.45		
5.08	17.0	1002.57	0.52	10.3	5.03
7.62	28.0	1491.01	0.77		
10.16	38.0	1902.32	0.98		
12.70	45.0	2082.27	1.08		

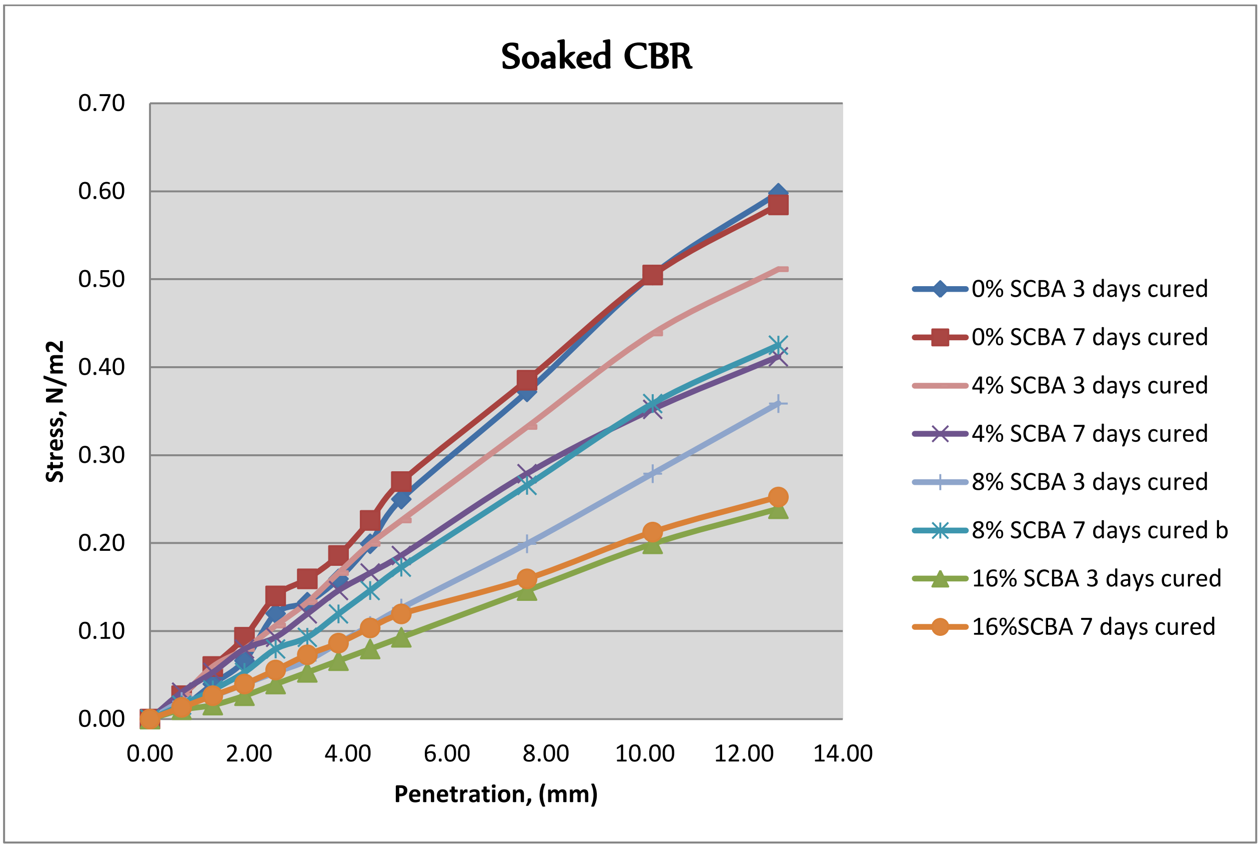


Figure A2 Plot of CBR results of soaked soil-SCBA mixture

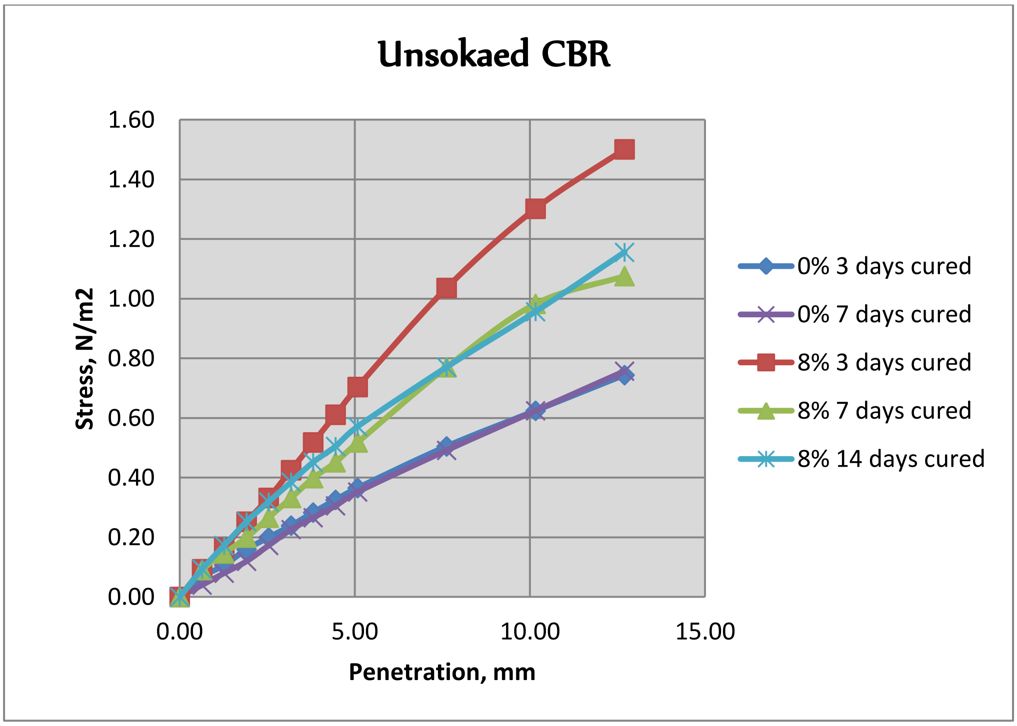


Figure A3 Plot of CBR results of unsoaked soil-SCBA mixture