ADAMA SCIENCE AND TECHNOLOGY UNIVERSITY

SCHOOL OF CIVIL ENGINEERING AND ARCHITECTURE

DEPARTEMENT OF CIVIL ENGINEERING

Effect of Sugarcane Bagasse Ash and Cement on Swelling Characteristics of Expansive Soil around Mojo

By

Dinkie Adimas

June, 2015

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A Thesis summited to Adama Science and Technology University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Geotechnical Engineering)

By

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APPROVAL PAGE

This thesis entitled with "**Effect of Sugarcane Bagasse Ash and Cement on Swelling Characteristics** of **Expansive Soil around Mojo**" has been approved by the following examiners in partial fulfillment of the requirement for the degree master of science in **Civil Engineering (Geotechnical Engineering).**

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DECLARATION

I, Dinkie Adimas Tesema, declare this thesis is my own original work and that has not been presented and will not be presented by me to any other University for similar or any other degree award.

Signature: ____________

Date: _______________

DEDICATION

Dedicate to my Family, Great Ethiopian Renaissance Dam, M.K and Ethiopian killed by ISIS.

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First of all I am thank full to the almighty God with his mother St. Virgin Mary whose grace and blessings have sustained me for all these years and the rest of my day to day life.

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SYMBOLS AND ABBREVIATIONS

Al_2O_3 Alumina;

ANSS Anyway Natural Soil Stabilizer;

ASTM American Standards of Testing Materials;

BA Bagasse Ash;

CaO Calcium Oxide;

CBR California Bearing Ratio;

CD Direct Shear Test;

CE Cement (Ordinary Portland);

CH High Clay Content;

CH Inorganic clays of high plasticity;

CU Unconfined Compressive Test;

ERA Ethiopian Road Authority;

ES Expansive soil;

Fe_20_3 Ferrite;

FSI Free Swell Index;

kPa Kilo Pascal;

L.O.I Loss on Ignition;

MDD Maximum Dry Density;

MgO Magnesium Oxide;

OMC Optimum Moisture Content;

PI Plasticity Index;

SiO_2 Silicon Dioxide and

UCS Unconfined Compressive Strength.

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ABSTRACT

Expansive soils are available all over the globe and having swelling and shrinkage characteristics depending upon the moisture variations. During rainy season, these soils undergo enormous swelling and cause lot of swelling pressure and during summer season, they undergo shrinkage i.e. volume reduction. The increase in volume during rainy season and decrease in volume during dry season is a cyclic process and one can notice in these soils commonly.

This study aimed to investigate the effect of applying bagasse ash in combination with cement or Ordinary Portland Cement on the engineering properties of treated soils. The laboratory work involved grain size analysis and index property determination to classify soil samples. Atterberg limit test, Proctor test, unconfined compressive test, California Bearing Ratio Tests, swelling potential and swelling pressure were used to evaluate properties of treated soils. Expansive soil was treated using 5%, 10%, 15% and 20% bagasse ash and 2%, 4%, 6% and 8% cement with fixed addition of 10% BA the respective combinations of the additives by dry weight of the soil. The optimum percentage of bagasse ash was found out be 10% BA based on the Unconfined Compressive Strength (UCS) test result. Soil sample taken for this study was clay with high plasticity (CH) which truly requires to be strengthened. The soil was stabilized with different percentages of bagasse ash and a small amount of cement. Observations were made for the changes in the properties of the soil such as Maximum dry density (MDD), Optimum moisture content (OMC), California Bearing Ratio (CBR) and Unconfined Compressive Stress (UCS). The result shows that the increase in bagasse ash content increases the OMC but decreases the MDD. Also, the value of CBR and UCS of soil increase considerably improved with the BA and cement content. The optimum amount was found to be in the proportion of 88% ES plus10%BA and 2%CE.This study shows that the stabilized material was suitable for pavement.

Keywords*:-Expansive soil, Bagasse Ash, Maximum Dry Density (MDD), Optimum Moisture Content (OMC), California Bearing Ratio (CBR),Unconfined Compressive Strength (UCS)and Swelling pressures.*

1 INTRODUCTION

1.1 Back ground

Large areas of Ethiopia are covered with expansive soils such as dark and light grey clay soils. Expansive soil is a term generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. This volume change causes most damage to structures/ particularly light buildings/ and pavement. These clays have caused persistent difficulties in road construction and are common occurrences in Ethiopia [27].

Its problem in Civil Engineering structures was first identified in the later part of 1930's. Since so many countries reported the failure of structures founded on swelling soils, such as Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Israel, Iran, Kenya, Mexico, Morocco, Zimbabwe, South Africa, Spain, Turkey, USA, Venezuela, and a number of other countries [27].

An expansive soil possesses their expansive characteristics mainly due to the presence of clay mineral which is montmorillonite. The most important clay mineral cause for expansive nature is montmorillonite, has an octahedral sheet sandwiched between two silica sheets. When this mineral was exposed to moisture, water absorbed between interlayering lattice structures and exert an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with expansive soils [31].

Expansive soils form a major soil group in Ethiopia and occur in the highlands mostly in the western, central and southwestern part of the country and in most cases the major clay mineral component is montmorillonite. The road sectors in Ethiopia are suffering from the high shrink-swell behavior of this expansive soil. Many damages occur each year and road construction over such expansive soil creates serious problems including increasing cost of construction and maintenance [31].

The probability of damages increases for structures on swelling foundation soils if the climate and other field environment, effect of construction, and effect of occupancy tend to promote moisture changes in the soil [32].

Recently sugarcane bagasse ash, which was a byproduct of sugar factories found after burning sugarcane bagasse which was found after the extraction, has been tested in some parts of the world for its pozzolanic property and has been found to improve some of the properties of the soil [14]. The oxide composition of BA such as SiO_2, Al_2O_3 and Fe_2O_3 is more than 70. This shows that, it has a good pozzolanic that could help mobilize the CaOH in the soil for the formation of cementations compounds [13].

1.2 Statement of the problem

In Ethiopia, a lot of damages have been occurred due to expansive soil. The economic losses due to failures associated with civil engineering construction are attributed to expansive soils. These expansive soils have been the major problem in the construction sector of the country. Cracks are visible on roads and buildings with shallow foundations due to shrink swell and differential settlement property of the expansive soils [27].

Based on the problem of expansive soil the Ethiopian Roads Authority has been doing much to alleviate problems caused by expansive soils by conventional method of removing the unsuitable expansive soil and replacing it with better quality fill material and constructing thicker pavements to account for the lower load bearing capacity of the expansive soil.

In recent years the Authority has started stabilizing expansive soil with chemical additives on road sections since the conventional method of replacing expansive sub-grade soil with suitable fill material was found to be more costly, time taking and machinery intensive. Accordingly, the authority has contracted foreign chemical additive supplying companies to undertake stabilization of expansive sub-grade soil on trial road sections in the Addis Ababa city road, Ambo Gedo road section etc.[33]. This shows the problem due to expansive soils is serious in the road sector development activity in country. Based on the above conditions the road development sector have initiated to search and evaluate for the various alternatives which enables to improve the engineering characteristics of the expansive sub-grade soil [33].

1.3 Objectives

The objective of this thesis work was:-

- To study the influence of bagasse ash and cement on free swell index, swelling potential and swelling pressure of expansive soils,
- To suggest the optimum proportion of bagasse ash and cement in controlling the swelling characteristics such as free swell index, swelling potential and swell pressure,
- To modifying or increase the engineering properties of soil by using bagasse ash and cement and
- To increase the strength or stability of soil and to determine the optimum content of: expansive soil- bagasse ash- cement.

1.4 Scope of investigation

To meet the above-mentioned objectives soil samples of expansive soil from mojo areas were collected for the investigation. The samples of soils were taken from 2m depth below the ground surface. The samples were collected from areas where intermingling of the surface soil have been observed up to considerable depth. Thus, it becomes necessary to go up to 2m depth to begin sample collection. The collected samples were treated with the bagasse ash and cement. That sample was tested with UCS test. The various tests used to evaluate the change in engineering properties of the soils are described next. This study has been supported by secondary resources and a series of laboratory experiments. The results are also specific to the type of additives used and test procedures that have been adopted in the experimental work.

1.5 Methodology

The present study started by setting a clear frame work of the research by reviewing secondary resources form the literature, research bulletins and previous studies. Samples of soil that represent expansive soil were collected from Dibandiba kebele found in 3km away Mojo area. Primary data was collected for the study by conducting a series of laboratory tests. Sample preparation of the experimental work involved air and sun-drying for expansive clay and sieving of soil to the required particle size. Classification of soil was determined by running index property and grain size analysis tests. Improvement in the index properties of the soil were assessed using Atterberg limit test. In addition index tests conducted to evaluate the change in important engineering properties of the soils;

- Unconfined cooperation test,
- Soaked CBR and unsoaked CBR tests,
- Swelling pressure test and swelling potential.

Expansive clay was treated with bagasse ash, cement and combination of the two additives. Expansive soil was treated in the following mixing ratios; 0%, 5%, 10%, 15% and 20% of bagasse ash by dry weight of the soil and 0%, 2%, 4%, 6% and 8% plus 10% bagasse ash by dry weight of the soil based on the maximum UCS value of two mixing which are soils and bagasse ash.

1.6 Outline

The thesis was comprised of six chapters. Chapter one gives a brief description of the thesis background, statement of the problem, objectives and scope of the investigation. The second chapter discusses basic soil characteristics such as definition, formation, properties of clay soil, expansive soil investigation, cement stabilization and bagasse ash stabilization. The third chapter gives a brief description of material and metrology. In the fourth chapter present the tests conducted, results and discussion, Chapter five presents the conclusions and the recommendations made regarding the influence of the bagasse ash and cement under study.

2 LITERATURE REVIEW

2.1 Nature and occurrence of expansive soils

2.1.1 Definition

Tropical expansive soils are often called black cotton soils. The latter name was believed to have originated in India where locations of these soils were favorable for growing cotton. Other local terms are used to describe these soils in many areas such as "regur" soils in India, "margalitic" soils in Indonesia and "black turfs" in Africa. Some investigators in India have defined black cotton soils as "soils derived from the weathering of trap rock (basalt) in particular, which are black, heavy and climatologically suited to the growth of cotton" [32].

Although many local terms are used, all black clays have very common characteristics which are of interest to the engineer i.e. they have high clay content with appreciable plasticity, dark colour and the tendency to expand with moisture increase. For general engineering purposes therefore, tropical black clays may be defined as "dark to black soils either moderate to high clay contents in which montmorillonite is the principal clay mineral and are generally expansive" [31].

2.1.2 Origin

The parent materials of expansive soils may be classified into two groups. The first group comprises of basic igneous rocks. Here feldspar and pyroxene minerals of the parent rocks decompose to form montmorillonite, the predominant mineral of expansive soil and other secondary minerals. The second group comprises of sedimentary rocks that contain montmorillonite. The expansive soils of Ethiopia are derived from both groups [31].

Climate and topography play a very significant role in the formation of tropical soils. Expansive soils are common in areas of moderate rainfall and poor drainage. Under these conditions and in an alkaline environment where there is adequate supply of magnesium, silica and alumina, montmorillonite, which is the principal clay mineral, is formed. These soils are rich in alkaline earths. Horizons of calcium sometimes develop or lime concretions may be found scattered through the profile [32].

Table 1 Chemical mineralogical property expansive soils [32]

location	sample No.	parents material	clay mineralogy	cation exchang capacity in M.E/100Gram	exchangeable cations	PH	Organic content
Chade Basin	CB-1	Alluvium		34.9	mostly Ca;with Mg;Minor K and	8.4	<1.0%
Chade Basin	CB-2	Alluvium		32.83	mostly Ca;with Mg;Minor K and	6.3	<1.0%
Chade Basin	CB-3	Alluvium	Kaolinite,illite, montmorillonite				
Chade Basin	CB-4	Alluvium	Quartz,Kaolinite,montmorillonite			6.3	
Nigeria	NBC-1	Calcereos Rocks	montmorillonite, kaolinite	44.08	mostly Ca;with Mg;Minor K and Na		<1.0%
Ethiopia	ETH-1	Basalt	montmorillonite, kaolinite	64.9	mostly Ca;with Mg;Minor K and Na	7.2	<1.0%
Kenya	KEN-6	Basalt	montmorillonite, kaolinite			8.8	
Kenya	KEN-9	Basalt		45.02	mostly Ca;with Mg;Minor K and Na	7.2	
Zambia	ZAM-1	Alluvium		32.16	mostly Ca;with Mg;Minor K and Na		1.0_1.5%
Zambia	ZAM-2	Basalt		33.34	mostly Ca;with Mg;Minor K and Na	6.4	<1.0%
Uganda	U-12	Basalt		56.74	mostly Ca;with Mg;Minor K and Na	7.4	1.00%
Tanzania	TAN-X-1	Alluvium		30.07	mostly Ca;with Mg;Minor K and Na	9.2	<1.0
Tanzania	TAN-3A	Alluvium					
Morocco	MOR-2	Alluvium from basalt	montmorillonite, kaolinite, chiorite				
Morocco	MOR-3	Alluvial from basalt	montmorillonite, kaolinite, illite, quartz, calcite				
Sudan	SU-1	Alluvium from basalt	montmorillonite, kaolinite, illite, quartz, calcite			8.9	
Sudan	SU-2	Alluvium from basalt	montmorillonite, kaolinite, illite, quartz, calcite				
Sudan	SU-3	Alluvium from basalt	montmorillonite, kaolinite, illite, quartz, calcite				

Expansive soils rarely develop thick profiles. Where these soils occur as residual deposits the profile is usually about one to two meters in thickness, but in transported regions it may be as much as 8m or more. Natural deposits of expansive soils are characterized by a general pattern of cracks especially during the long dry season of the years [32].

2.1.3 Chemical and mineralogical characteristics

Generally clay-minerals can be divided into three general groups on the basis of their crystalline arrangements such as: Kaolinite, Montmorillonite and Illite group.

Kaolinite mineral: - The kaolinite structural unit consists of alternating layers of silica tetrahedral with the tips embedded in an aluminum octahedral unit. The alternating of silica and gibbsite layers produces sometimes called a 1: 1 basic unit. The kaolinite cluster is a stacking of 70 to 100 or more sheets as a book with hydrogen bonds and van der Waals forces at the interface [30]. The bonding combination of hydrogen and van der Waals forces results in considerable strength and stability with little tendency for inter-layers to take on water and swell. Kaolinite is the least active of the clay minerals [30].

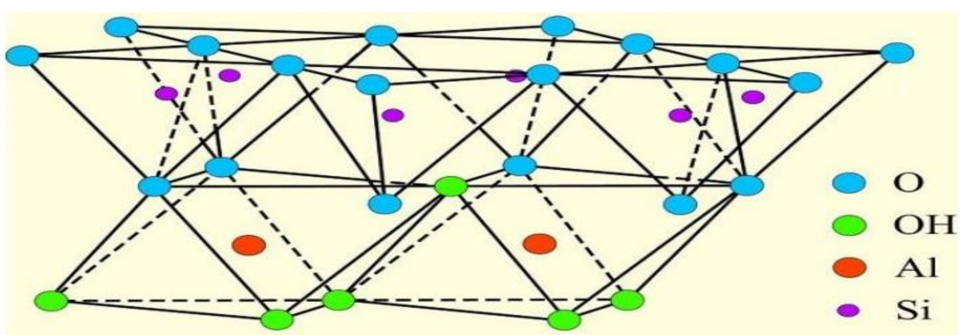


Figure 1 Structure of kaolinite layer [30]

Illite group:-The illite clay mineral consists of an octahedral layer of gibbsite sandwiched between two layers of silica tetrahedral. This produces a 1: 2 mineral with additional difference that some of the silica positions are filled with aluminum atoms and potassium ions are attached between layers to make up the charge deficiency. The bonding results in a less stable condition than for kaolinite, and thus activity of illite is greater than kaolinite [30].

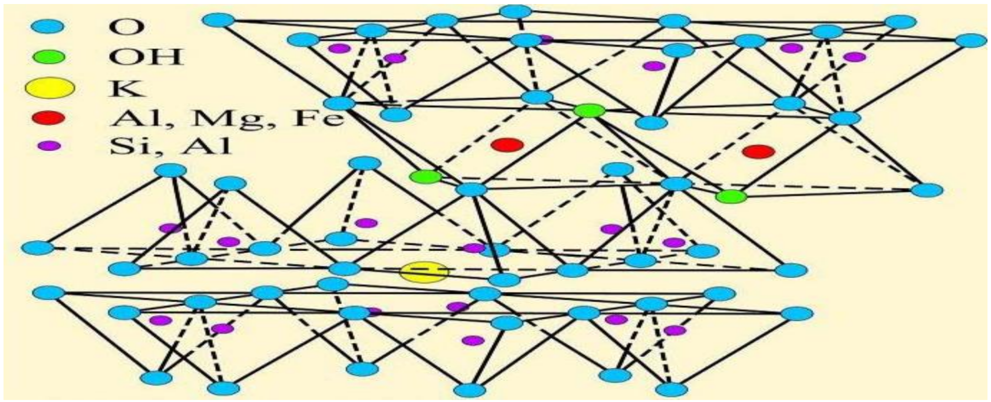


Figure 2 Structure of illite/mica layer [30]

Montomorillonite minerals: - Montmorillonite is a three layered mineral having a single octahedral alumna sheet Sandwiched between two silica sheets to give a 2:1. The inter-sheet bonding is due to mainly to van der Waals force and thus, very week relative to hydrogen or other ion bonding and dipolar molecules of water can enter between the sheets causing them to expand readily. Various substitutions take place, including Al for Si in the tetrahedral and Mg, Fe, Li, or Zn, for Al in the octahedral layer. These exchanges result in a relatively large net unbalanced negative charge on the mineral, with resulting large cation exchange capacity and affinity for water with H^+ ion in the absence of metallic ions. Soils containing substantial amounts of montmorillonite will exhibit high shrinkage and swelling characteristics. Weathering of montmorillonite clay minerals often produces kaolinite clay [30].

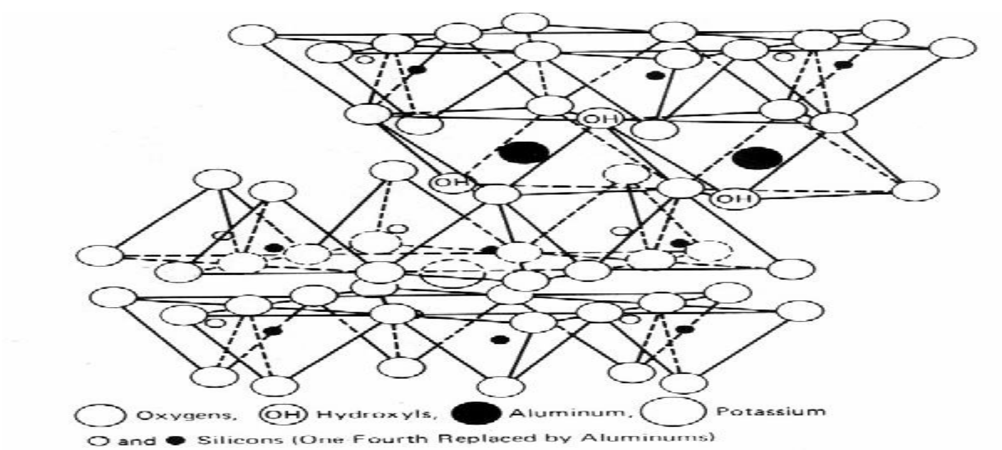


Figure 3 Structure of montomorillonite layer [30]

2.2 Identification and classification of expansive soils

2.2.1 Expansive soil identification

2.2.1.1 Field identification

It was evident that expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Some of the important field identification method that indicates the potential for expansiveness of soil the following [28]:-A shiny surface was easily obtained when a partially dry piece of the soil have been polished with a smooth object such as the top of a finger nail.

- The wet samples of the soil are sticky and it will be relatively difficult to clean the soil from the hands.
- The appearance of cracking in nearby structures.
- They usually have a color of black and gray.
- In the regions where there were seasonal moisture variation:-open or closed fissures (a joint or similar discontinuity), slickenside (highly polished or glossy fissure surface) and Shattering or micro-shattering (presence of fissures forming granular fragments of clayey soils) [28].

2.2.1.2 Laboratory identification

Generally, there are three different method of identifying expansive soil in the laboratory

2.2.1.2.1 Mineralogical identification

This method used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles and change deficiency and surface activity of clay particle. These properties are a fundamental factor controlling expansive soil behavior [28].The various techniques under these methods are:-

- X-ray diffraction,
- Differential thermal analysis,
- Dye absorption,

- Electron microscope and Base exchange capacity, etc

But these methods are not suitable for routine tests because of the following reason; these are time consuming, require expensive test equipment and the test results have been interpreted by specially trained technicians.

2.2.1.2.2 Indirect methods

These methods include simple soil property test a practicing engineer resort to use for identifying expansive soil. Such tests are easy and can be performed in most soil mechanics laboratory, and yield an excellent indices of expansive properties [28]. The various tests under these methods are:

i) Atterberg limits:

In this method, measurement of the plasticity and the shrinkage characteristics of the soil were conducted for identification of all engineering soil and provide a wide acceptable means of rating. Especially when combined with other tests they can be used to classify expansive soils [31]. The different types of limits under this method are Liquid limit, Plastic limit and Shrinkage limit.

Table 2 Indicative properties of Ethiopian expansive soils [28]

Clay content smaller than (2 microns) or 0.002mm	50-80%
Liquid limit	80-120%
Plasticity limit	55-90%
Shrinkage limit	10-16%
Free swell	90-23%

Table 3 Swelling potential vs plasticity index [31].

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

ii) Linear shrinkage tests

Linear shrinkage is the change in length of a soil sample as it dries to the shrinkage limit, SL, expressed as a percentage of the original length. This test also in combination with other tests is used for classification of expansive soil [28].

iii) Free swell tests

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The test is performed by pouring, 10cc of dry soil passing a sieve size of 0.425mm (No. 40), into a 100cc graduated jar filled with water. Then the swelled volume of the soil after the material settles is measured [28].

2.2.1.2.3 Direct method

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement. The method quantitatively evaluates the volume change characteristics of expansive soil. The test can be done using consolidometer but care should be taken on the test procedures. A standardized procedure that consider the factors that affect the shrink swell potential as well as simulate the expected loading condition should be adopted. The test can be done by the use of a conventional one-dimensional consolidation which is available in most soil mechanics laboratories [30].

Table 4 Swelling potential prediction in soils [10]

Parameter	Reference	Degree of Expansion			
		Low	Medium	High	very high
LL (%)	Chen, 1975	<30	30-40	40-60	>60
PI (%)	Chen, 1975	0-15	10-35	20-55	>55
	Holtz and Gibbs,1956	<20	12-34	23-45	>45
Swell Percent (%)	Thomas et al., 2000	<3	3-6	6-9	>9
Swell Pressure (kPa)	Thomas et al., 2000	<81	81-153	153-225	>225

Note Potential expansion is given for a confined sample with vertical pressure equal to overburden pressure expressed as a percentage of simple weight. Studied soil properties and their proposed relation to degree of expansion are summarized in above table.

2.2.2 Classification of expansive soils

Expansive soils classification:-The key to all expansive soil classification systems is the method of measuring swell potential. Swell potential may be measured directly in swell test or indirectly determined by correlation with other test results of swell test data. In almost every case swell potential is evaluated in the laboratory in a consolidation test device. This may yield swell potential different from those for in-situ soils. Thus an accurate correlation between swell potential and other test results for a purpose of prediction of in-situ heave is difficult. These procedures, however, do provide good indicators of swell potential when the soil is subjected to the conditions used in the laboratory test [30].

Bureau of reclamation method:-This method is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The measured volume change is taken from odometer swell tests using 1 psi surcharge pressure from air dry to saturation [30]. The degree of expansion and limits of correlated properties are shown in below Table.

Table 5 Classification of expansive soils based on Bureau of reclamation method [30]

liquid limit(in %age)	plasticity limit(in % age)	shrinkage limit(in %age)	free swell index (in % age)	Degree of expansion	Degree of severity
20-35	<12	<15	<50	Low	non critical
35-50	12-23	15-30	50-100	Medium	marginal
50-70	23-32	30-60	100-200	High	Critical
70-90	>32	>60	>200	very high	Severe

2.3 Distributions

The expansive soil is found in over the world such as Africa, India, and Australia. These apparently form in poorly drained areas over basaltic rocks where climatic conditions would ordinarily cause reddish soils to form if the drainage were good. South-East Asia has many deposits of black clays [32].The most important area of expansive soils in Africa is the Lake Chad basin, which covers North–East Nigeria, Northern Cameroon and Southern Chad. Most of the soils in this area were lacustrine or alluvial with some exceptions in Northern Cameroon and North Eastern Nigeria where residual deposits have been founded over basic extrusive rocks [30].

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has been studied thoroughly; the Southern, South-East and South-West part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Modjo, Addis-Ambo, Addis-Woliso, Addis-Debre Birhan, Addis-Gohatsion area are covered by expansive soils and also areas like Mekele and Gambelaare covered by expansive soils [32]. The distributions of expansive soils in Ethiopia are shown in below Figure.

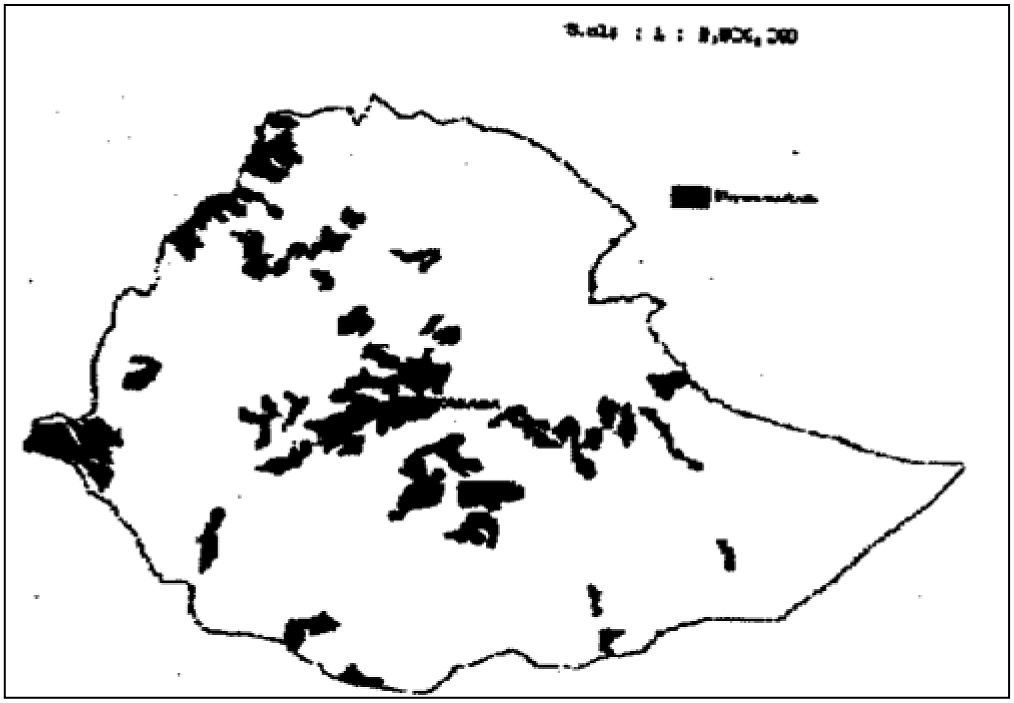


Figure 4 Distribution of expansive soil in Ethiopia [32]

2.4 Expansive soil investigation

Teferra A. and Yohannes S. (1986) studied investigations on the expansive soils of Addis Ababa. From the study concluded that, the black and grey soils found in the eastern and southern part of Addis Ababa are highly expansive, the percentage of montmorillonite present in the black and grey soils ranges from 45 to 87 percent. There was not any distinction between the heaving characteristics of the grey and black soils. Structures founded on conventional masonry wall foundations resting on an expansive soil are liable to crack. Structures which did not manifest any sign of cracks in their recorded history have cracked severely when an adjoining structure is constructed.

Teklu D. (2003) examined the swelling pressure of Addis Ababa area expansive soil. So many damages were reported as a result of expansive soil. Such damages occur when the pressure exerted by the soils greater than the foundation pressure. Consequently assessing the swelling pressure is an important step in designing foundation on expansive soil. Several researchers have developed an empirical relation for the estimation of swelling pressure. The empirical expressions relate the swelling parameters to the geotechnical parameters that are determined by the identification tests. The prediction of swelling pressure by empirical relationships cannot be expected to yield accurate results.

Sisay A. (2004) studied the assessment of damage of buildings constructed on expansive soils located around Addis Ababa. Out of the surveyed buildings constructed on expansive soils of Addis Ababa 64% are adversely affected that means shows cracks associated with expansive soils. Drainage is the most critical initiating factors for building damages. 84 % of the damaged buildings do not have proper drainage system or the drainage system provided is in adequate. The damages occurred on walls are more than the damaged that occurred on floors and ceilings. Buildings with deep isolated footings and Stiffened mat foundations are not damaged or showed very minor damage. The precautions to be taken in treating expansive soil using the available material are not practiced in most buildings of Addis Ababa either at design or construction stages and the adverse economic consequence of damages related to expansive soils in Addis Ababa is substantial.

Tilahun D. (2004) investigated the influence of drainage condition on shear strength parameters of expansive soils. By using the ordinary plasticity chart, it was reported that most of the expansive soils tested came under the CH group i.e. inorganic clays of high plasticity. The role of matrix suction on shear strength is very apparent in expansive soils and hence needs to be accounted for when evaluating tests procedures and results interpretations, if samples are to be saturated prior to shear testing. The strength parameters for the CU tests would range from 18 to 83 kPa, while for the internal friction would range from 12 to 39 degrees.

Zewdie A. (2004) investigated into shear strength characteristics of expansive soil of Ethiopia. Moisture has a significant effect on the shear strength property of expansive soil. The shear strength of expansive soil reduces when the moisture content increased. When the shear strength of expansive soil is mentioned, it should always be referenced with the moisture content. Factors such as swelling and shrinkage, for example, influence the soil strength significantly and their effects are difficult to account for in most laboratory shear tests.

Kassa M. (2005) investigated the relationship between consolidation and swelling characteristics of expansive soils of Addis Ababa area. It is known that the magnitude of the swelling of expansive soils varies with environmental conditions and it is observed from this research that the consolidation characteristic of expansive soils is influenced by the swelling. Therefore, the consolidation characteristic of expansive soil is not inherent property of the soil mass. It is pointed out from this research that the consolidation parameters are not constant for a given expansive soil. But the Atterberg limits are inherent property of the given soil. Therefore, empirical relations developed to determine compression index from the plasticity in non-expansive soil will no more be applied for expansive soil.

Kassahun H. (2006) examined the Atterberg limits for expansive soils. Based on the test results the following conclusions were made: from field tests i.e. artificial flooding and moisture content below water table revealed that the water holding capacity of expansive soil is found between liquid limit and plastic limit. The desiccation process of expansive soils, and red clay soils, gives the same pattern. However; it was observed that rewetting of completely dry pat of expansive soils, at the moisture content below shrinkage limit shows from 4.16 to 9.76 % of volume change for the additional moisture content ranges from 6.62 to 11.64 % of dry mass.

2.5 Cementations Stabilization

Tesfaye A. (2001) studied improvement of expansive soil by addition of lime and cement on black cotton soil from different parts of Addis Ababa. Index properties, compaction characteristics and swelling pressure of soil-cement and soil-lime were determined using Atterberg limit test, moisture-density relations, free swell and swelling pressure tests. The conclusions and findings drown from the study are; Expansive soil becomes moderately active to inactive based on the amount of lime and cement added, swelling pressure of expansive soil decreases with increasing lime, cement and molding water content, 4-6% of lime and 9-12% of cement yielded significant improvement on plasticity and swelling properties of expansive soils.

Nebro D. (2002) evaluated lime and liquid stabilizer called Con-Aid for stabilization of potentially expansive subgrade soil on samples collected form Addis-Jimma road which had indicated different pavement damages exacerbated by the presence of expansive soils. The experimental study involved Atterberg limit test, moisture-density relation, UCS, CBR and CBR swell. The findings and conclusions of the study can be summarized as follows: Addition of lime reduced maximum dry density and increased the optimum moisture content, 4% of lime by dry weight of the soil was optimum lime content to stabilize the soil even though increased quantity of lime led to increased strength and addition of lime reduces the swelling potential but no significant improvement in the engineering properties of the soil was attained by addition of Con-Aid.

Christopher M. (2005) investigated the suitability of traditional and non-traditional stabilizers on three highly plastic Virginia soils that have caused problems or resulted in poor performance service. The applied soil stabilizers were: quicklime, hydrated lime, palletized lime, cement, lingo sulfate, synthetic polymer, magnesium chloride. The research used unconfined compressive strength test to compare effects of stabilization. The conclusions and findings drawn from the study are; traditional lime and cement stabilizers were far more effective than liquid stabilizers in increasing strength, liquid stabilizers were ineffective on soils with high moisture content, proprietary cementations stabilizer was more effective in increasing strength than lime but was not as effective as a cement stabilizer, quick and hydrated lime increased workability of soils although they did not produce strengths comparable to cement, strength of soils stabilized with cement and proprietary cementations stabilizer can be estimated based on the water-amendment ratio of the mixture and the strength of soils stabilized with lime can be estimated based on combination of plasticity index and water amendment ratio of the mixture.

Argu Y. (2008) has made a study on light gray and red clay soils of Addis Ababa using SA44/LS-40, lime and combination of the two additives at different application rates. The experimental study was made by Atterberg limit test, soaked CBR and CBR swell, swelling pressure and free swell tests. The conclusions and findings of the study are;

Applications of lime alone results in modest improvement in the CBR value of the soil, while applications of the chemical with lime results in remarkable improvement in the

CBR value of the soil, The application of SA-44/Ls-40 alone results promising reduction in the swelling pressure in light gray clay; however, it is ineffective in improving the soaked CBR value, 0.3lit/m3 of SA-44/LS-40 and 2% lime is an optimum proportion in increasing soaked CBR and reducing swell potential of light gray soil and 0.08lit/m3 of SA-44/LS-40 and 4% lime is an optimum mix for red clay soil.

2.6 Sugarcane bagasse ash stabilization

Mohammed A. (2007) investigates the influence of compactive effort on bagasse ash with cement treated lateritic soil. The optimum moisture content (OMC) increased while maximum dry density (MDD) decreased with increasing bagasse and cement content. The increase in optimum moisture content may be due to water requirement for cement hydration and pozzolanic reaction of the lime released during hydration of cements. The effect of bagasse content of cement modified laterite soil on cohesion and angle of internal friction is that, the cohesion decrease while the angle of internal friction increases.

Amu et al (2011) studied the geotechnical properties of lateritic soil stabilized with sugarcane straw ash based on the results it was concluded that sugarcane straw ash was an effective stabilizer for improving the geotechnical properties of lateritic soil samples.

Chittaranjan M. et al (2011) studied the agricultural wastes as soil stabilizers. In this study agricultural wastes such as sugar cane bagasse ash, rice husk ash and groundnut shell ash are used to stabilize the weak sub grade soil. The weak sub grade soil is treated with the above three wastes separately at 0%, 3%, 6%, 9%,12%and 15% and CBR test is carried out for each per cent the results of these tests showed improvement in CBR value with the increase in percentage of waste.

Ken C. Onyelowe (2011) studied cement stabilized akwuete lateritic soil and the uses of bagasse ash were collected at a depth below 1.5m to avoid the top soil. This Soil was stabilized using 4% and 6% cement with variations of bagasse ash ranging from 0%(control), 2%, 4%, 6%, 8%, and 10% by weight of the dry soil. The OMC, MDD, and CBR tests were carried out on the mixture of soil with cement and with bagasse ash as admixture. The results of the optimum moisture content, maximum dry density and

California bearing ratio for the 4% and 6% cement content for the varying percentages of bagasse ash are as at 4% cement content, with bagasse ash as admixture, there is a general reduction in the maximum dry density while there is an increase in the maximum dry density with increase in bagasse ash content at 6% cement content. The optimum moisture content generally increased with increase in the bagasse ash content. There was also a tremendous improvement in the CBR with bagasse ash compared to the natural soil.

Moses G., K. J. Osinubi (2013) studied the influence of compactive efforts on cement-bagasse ash treatment on expansive black cotton soil. The index properties were determined on the natural and treated soils with stepped percentages of cement (i.e., 0, 2, 4, 6 and 8%) were admixed with 0, 2, 4, 6 and 8% of bagasse ash by dry weight of soil. All the compactions involving moisture-density relationships, CBR and UCS tests were carried out by using energies derived from the standard Proctor (SP), West African Standard (WAS) and modified Proctor (MP) energies. Finally an optimal blend of is 8% OPC/4% BA is recommended for treatment of expansive black cotton soil for use as a sub-base material.

Amit S. Kharade et al (2014) investigate waste product bagasse ash from sugar industry can be used as stabilizing material for expansive soils from the result the use of agricultural waste slightly improves the properties of expansive soils, bagasse ash can be used as replacement in black cotton soil up to certain limits. The properties which improve are discussed here; the initial laboratory test showed that collected black cotton soil is solid and stiff. It has low permeability, high compressibility and low bearing capacity. The effective percentage replacement of bagasse ash was found to be 6%. The results improved at 6% replacement are as follows: The maximum dry density increased by 5.8%, California bearing ratio (CBR) increased by 41.52% and compressive strength increase by 43.58%. The blend suggested from this research is black cotton soil + 6% replacement by bagasse ash.

Eltayeb et al (2014) studded the usage of the sugarcane bagasse ash in Sudanese Sugar Company as pozzolanic-cement binder based on the result bagasse ash obtained produced a reactive material relatively free from carbon and other organic matter. SCBA with high specific surface area, high contents of amorphous silica and calcium oxide; fulfilled the principal requirements of a pozzolanic material. A little amount of crystallization of silica present in ash, may have affected the degree of reactivity. Mechanical strength tests confirmed the actual behavior of the SCBA- blended concrete specimen as 10% substitution of OPC with SCBA, gave better strength results in comparison with OPC specimen.

2.7 Conclusion of the literature review

From the previous research works the following main findings can be summarized;

- The optimum moisture content (OMC) increased while maximum dry density (MDD) decreased with increasing bagasse and cement content.
- The weak sub grade soil is treated with agricultural wastes such as sugar cane bagasse ash, rice husk ash and groundnut shell ash the treated separately at 0%, 3%, 6%, 9%, 12%and 15% and CBR test is carried out for each percent the results of these tests showed improvement in CBR value with the increase in percentage of waste.
- The index properties were determined on the natural and treated soils with stepped percentages of cement (i.e., 0, 2, 4, 6 and 8%) were admixed with 0, 2, 4, 6 and 8% of bagasse ash by dry weight of soil. Bagasse ash is used as replacement in black cotton soil up to certain limits.
- The usage of the sugarcane bagasse ash in as pozzolanic-cement binder based on the result, bagasse ash obtained produced a reactive material relatively free from carbon and other organic matter.
- Bagasse ash with high specific surface area, high contents of amorphous silica and calcium oxide; fulfilled the principal requirements of a pozzolanic material.
- Adding 4% lime by dry weight of the soil is an optimum quantity in terms of improving plasticity and swelling properties.
- Adding 4-6% of lime and 9-12% of cement yielded significant improvement on plasticity and swelling properties of expansive soils.
- As the amount of cementations stabilizer increases, swelling pressure of expansive soil decreases.

- Lime reduces maximum dry density and increases the optimum moisture content of expansive soils.
- Mixing traditional stabilizers (especially lime) with non-traditional additives can be a viable option of soil stabilization. Traditional stabilizers such lime and cement are far more effective than liquid stabilizers in increasing strength.
- Quick and hydrated lime increased workability of soils though they did not produce strengths comparable to cement.

The expansive soil has been investigated and stabilization was relatively well researched in Ethiopia. Most of these studies focused on expansive soil investigation and evaluating the suitability of chemical additives introduced in the country for expansive subgrade stabilization. However, no study has been made on effect of bagasse ash and cement stabilization. In this research mechanism have been used bagasse ash and cement alone to stabilize the soil.

3 MATERIALS AND METHODS

3.1 Materials

The three materials such as expansive soil, bagasse ash and cement were used in this study and explained in detail in the next section.

3.1.1 Expansive soil

The soil sample was collected around mojo, Dibandiba kebele 3km away from Mojo to Addis Ababa road. The sample was collect blow a depth of 2 m from ground level. The soil samples that represent expansive subgrade and sub base soil that does not fulfill ERA's grading requirement 2002. Figure 5 shows an overview of the samples.

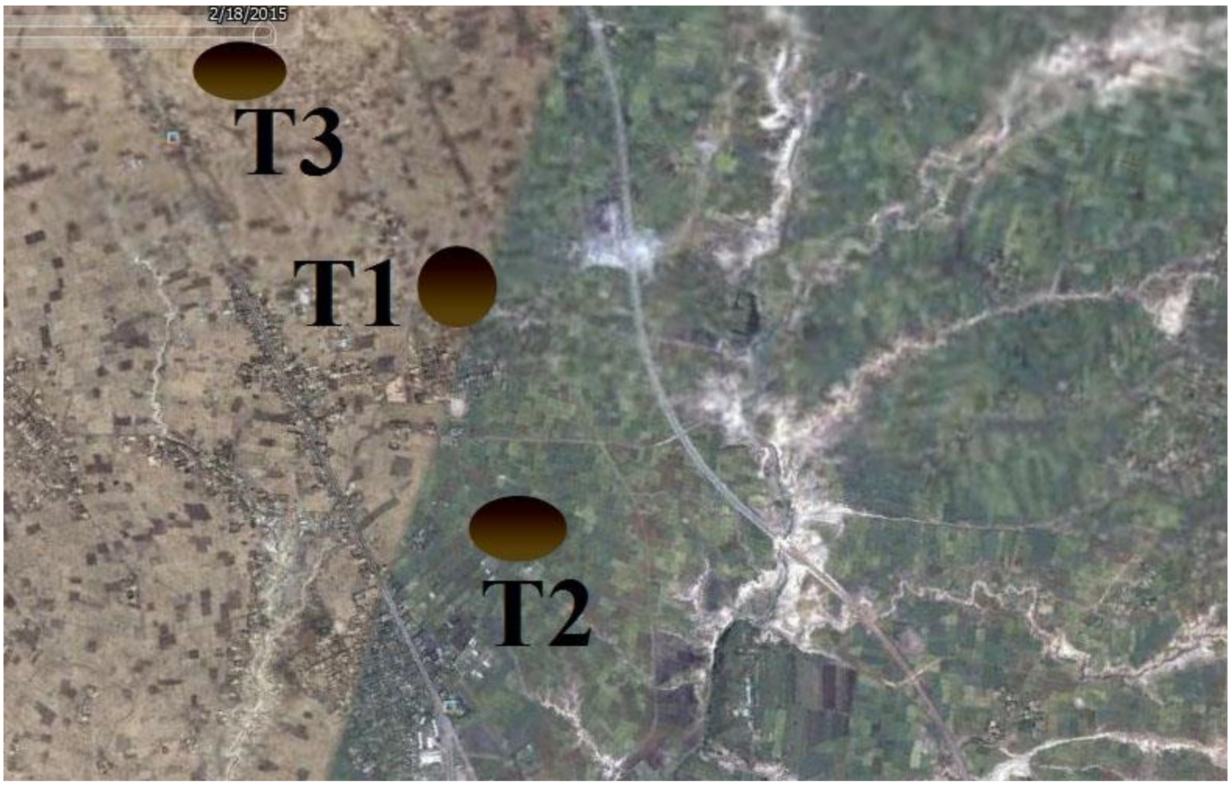
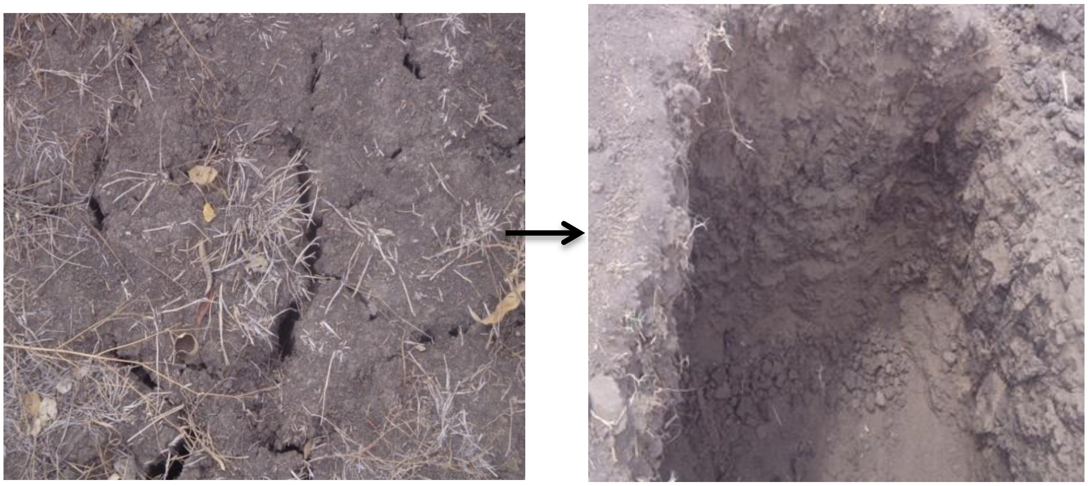


Figure 5 Location of test pit in the study area



a) Overview of existing crack b) sample of test pit

Depth(m)	Bore hole log	Visual description	Field test type	Sampled for
∇ - 2.00		Black expansive soil	Natural moisture contents	Index, engineering and swelling tests

Figure 6 Sample of soil profile

3.1.2 Sugarcane bagasse ash (BA)

The bagasse ash was collected from Wenji Shewa sugar factory. Bagasse ash is the residue obtained from the incineration of bagasse in sugar producing factories. This material usually poses a disposal problem in sugar factories particularly in tropical countries including Ethiopia. In many tropical countries there are substantial quantities of bagasse is rich in amorphous silica that indicate it has pozzolanic properties [14].

Utilization of industrial and agricultural waste products in the construction of roads has been the focus of research for economic and environmental reasons.

The oxide composition of BA is shown in Table 6.The combine percent composition of SiO_2, Al_2O_3 and Fe_2O_3 is more than 70%. This shows that, it is a good pozzolanic material that could help mobilize the CaOH in the soil for the formation of cementations compounds [13].

Table 6 Chemical composition of Sugarcane bagasse ash [14]

Chemical compositions	% weight
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	0.25
P_2O_5	1.35
H_2O	0.2
SO_3	0.18
LOI	10.48

Table 7 Physical property of Sugarcane bagasse ash [14]

Property	Value
Colour	Black
PH	7.86
Specific gravity	2.25

3.2.2.1 Pozzolanic property of Sugarcane bagasse ash

Pozzolanic materials are siliceous and aluminous materials which alone possess little or no cementations value but will, in finely divided form and in the presence of moisture, react chemically with calcium hydroxide at ordinary temperature to form compounds possessing cementations properties. Sugarcane bagasse ash has also been found to have such pozzolanic property [3].

Pozzolanic materials can be divided into two groups: natural pozzolana and artificial pozzolana. Clay and shales, opalinc chert, diatomaceous earth, and volcanic ash are an example of natural pozzolans while fly ash, blast furnace slag, silica fume, rice husk ash, and metakaoline are example of artificial pozzolans. Most of the pozzolans in use today are mainly byproduct materials that are widely available [13].

Bagasse ash was also tested to have such property. It acts as a pozzolanic material when added to cement because of its silica (SiO_2) content which reacts with free lime released during the hydration of the cement and forms additional calcium silicate hydrate (CSH) as a new hydration product [14]. Because of the diversity of pozzolans their chemical composition also varies. Therefore classifying pozzolans only depending on their chemical composition would be difficult. For this reason ASTM C 618 classifies pozzolans depending on performance basis such as N, F, C and S etc [3]. Then based on the reason specify combined weight of silica, alumina and ferrous oxides as 70% for classes of N, F and C pozzolanas, which the Bagasse Ash has, satisfy. The physical and chemical compositions of bagasse ash are satisfactory and confirm to the requirements of [3], class N pozzolanas in finances and specific gravity and oxide composition.

Table 8 Chemical requirement for pozzolan [14]

Chemicals	Pozzolan class		
	N	F	S
Sio_2+Al_2o_3+Fe_2o_3 (min %)	70.0	70.0	70.0
Mgo (max %)	5.0	----	5.0
SO_3 (max %)	4.0	5.0	4.0
Moisture content (max %)	3.0	3.0	3.0
loss on ignition(max)	10.0	12.0	10.0
Available alkalis as Na_2O (max %)	-----	1.5	1.0

Pozzolanic reaction:-The hydration of tri-calcium silicate and di-calcium silicate with water gives calcium silicate hydrate and calcium hydroxide.

The principal reaction taking place is as shown in below.

Pozzolan+ Calcium Hydroxide+ water→ Calcium Silicate Hydrate (gel) ---------- (Eq.3.1)

3.2.2.2 Availability of Sugarcane bagasse ash in Ethiopia

In order to assess the potential of bagasse ash production in Ethiopia, it is imperative to evaluate the sugarcane crop yield in the country. There were three regional state owned sugar factories functioning in the country in 2018. Their annual production capacity is about 300,000 tons [14], the sugarcane covering about 10,000 hectares of land. This annual production is not sufficient to satisfy the local sugar demand forcing the government to annually import 1.5 million quintals from abroad [14]. To avoid this shortage of sugar in the country the government plans to establish eight new sugar factories in the coming five years with a total estimated capacity of 2.250 million tons at the start of their production according to the strategic plan and covering about 225,000 hectares [14].

Beside this the government is undertaking expansion projects on the existing factories to increase their production capacity. The expected future sugar production of the country and the respective bagasse ash potential is summarized Table 9. It can be seen from Table 9 about 0.94 million tons of bagasse ash is going to be generated annually when the five-year strategic plan comes into reality. Currently with sugar production of about 300,000 tons annually, the bagasse ash potential is about 72,000 tons annually [14].

Table 9 Estimated Sugarcane bagasse ash potential of Ethiopia [14]

Factory	Expected future production of sugar (Tons/year)	Expected Bagasse (Tons/year)	Expected Bagasse Ash (Tons/year)
Wonji/Shewa	350,000	1,050,000	84,000
Metahara	190,000	570,000	45,600
Fincha	270,000	810,000	64,800
Tendaho	600,000	1,800,000	144,000
New 8	25,000,000	7,500,000	600,000
Total	3,910,000	11,730,000	938,400

Figure 7Sugarecane bagasse ash samples

3.1.3 Cement

The cement was collected Derba Ordinary Portland cement factory. There are different types of cement depending on their composition, method of manufacturing grinding, burning, etc. The chemical and physical property of Derba Ordinary Portland cement factory is shown in below table. Derba cements are available at markets.

Table 10 Chemical composition of Ordinary Portland Cement of Derba [factory]

Oxides	% weight
SiO_2	24.87
Al_2O_3	6.01
Fe_2O_3	3.88
CaO	57.24
MgO	1.78
SO_3	2.81
Na_2O	0.24
K_2O	0.47
Na_2O eqv	0.55
Free lime	0.91
LOI	1.67

4 LABORATORY RESULT AND DISCUSSION

In this section description, classification of materials, the important of laboratory test and test results have been presented.

4.1 Laboratory test results of natural soil

ASTM and AASHTO standards are used for testing expansive soils in the laboratory.
The summary of the test results on the expansive soils was indicated in Table11. From the summary of the test result shown in below Table characterized as highly expansive and high plasticity soil.

Table 11 Summary of test results of expansive soils

No	Property	Value
	Index property	
1	Water content (%)	18
2	Gravel (%)	1
3	Sand (%)	8
4	Fines silt (%)	36
5	Clay (%)	55
6	Liquid Limit (LL) (%)	84
7	Plastic Limit (PL) (%)	34
8	Plasticity Index (PI) %	50
9	USCS Classification	CH
10	Free Swell Index (%)	118
11	Specific Gravity (G)	2.69
12	Optimum Moisture Content OMC (%)	21.5
13	Maximum Dry Density MDD (/KN/m^3)	16.35
	Engineering property	
1	Unconfined Compressive Strength (kPa)	87.68
2	CBR (Unsoaked) (%)	8.25
3	CBR (4 day soaked) (%)	2
4	swelling pressure(kPa)	250

4.2 Grain size distribution analysis

Soil mostly consists of different sized particles as major constituent ingredient. The determination of the fractions of the particles will help to identify the soil type as well as to estimate many other engineering properties such as strength and permeability. Two methods are used to determine grain size distribution: - Namely; sieve analysis test was for coarse grained portion of the soil and hydrometer analysis test was done for fine grained portions.

4.2.1 Sieve analysis

This test was done in accordance with ASTM-D422. Sieve analysis was done for coarse grained soils that were retained in 0.075mm (No.200). The sieve analyses have been carried out by sieving a known wet weight of sample through the set of sieves. The total mass of the sample used (Tm) =500g.

Table 12 Wet sieve analysis expansive soils

Sieve No	Sieve opening (mm)	Mass of sieve (g)	Mass of sieve + Re. soils(g)	Mass of Retained soils(g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passig (%)
No 4	4.75	1261.2	1261.2	0.0	0.0	0.0	100.0
No 10	2	953.8	959.2	5.4	1.1	1.1	98.9
No 16	1.18	892.5	898.0	5.5	1.1	2.2	97.8
No 40	0.425	784.6	795.8	11.2	2.2	4.4	95.6
No 50	0.3	748.5	754.0	5.5	1.1	5.5	94.5
No 100	0.15	745.3	753.0	7.7	1.5	7.1	92.9
No 200	0.075	761.9	768.0	6.1	1.2	8.3	91.7
Pan	-----	733.1	1191.0	457.9	91.6	99.9	----

4.2.2 Hydrometer analysis

The tests were performed according to the ASTM D-422 procedure. The test has been conducted for soils that were passed sieve size 0.075mm (N0.200). Approximately 50 grams of dry soil was treated with a dispersing agent for 18 hours. A hydrometer analysis was performed to measure the amount of silt and clay size particles.

Table 13 Hydrometer analysis expansive soil

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (mm)	Coefficient K	Grain Size (mm)	Finer (%)	Finer Combined (%)
0.45	1.0315	0.0027	1.0288	7.97	0.0134	0.057	91.7	84.09
1	1.0305	0.0027	1.0278	8.23	0.0134	0.038	88.5	81.17
2	1.0300	0.0027	1.0273	8.36	0.0134	0.027	86.9	79.71
4	1.0290	0.0027	1.0263	8.63	0.0134	0.019	83.7	76.79
8	1.0280	0.0027	1.0253	8.89	0.0134	0.014	80.5	73.87
15	1.0270	0.0027	1.0243	9.16	0.0134	0.010	77.4	70.95
30	1.0265	0.0027	1.0238	9.29	0.0134	0.007	75.8	69.49
60	1.0250	0.0027	1.0223	9.69	0.0134	0.005	70.9	65.11
120	1.0240	0.0027	1.0213	9.95	0.0134	0.004	67.8	62.19
240	1.0220	0.0027	1.0193	10.48	0.0134	0.003	61.4	56.35
480	1.0210	0.0027	1.0183	10.75	0.0134	0.002	58.3	53.43
1440	1.0170	0.0027	1.0143	11.80	0.0134	0.001	45.5	41.75

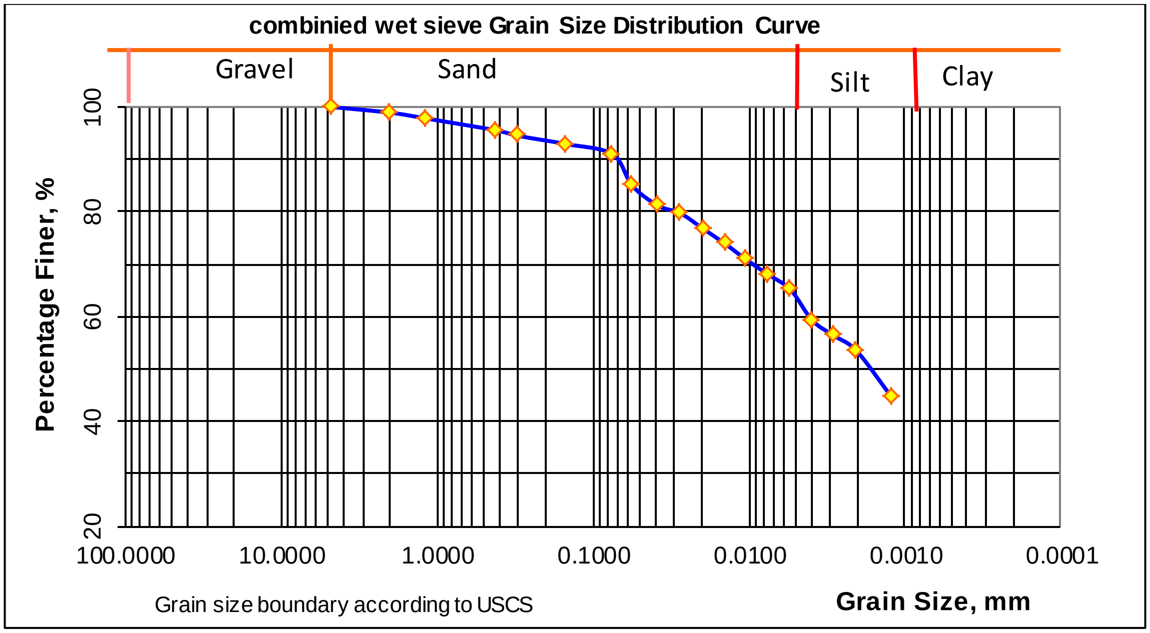


Figure 8 Combined wet sieve grain size distribution curve

Table 14 Combined wet sieve grain size distribution

Summary(combined)	%
1.Particles larger than 2mm	1
2.Sand 2mm - 0.075mm	8
3.Silt 0.075-0.002mm	36
4.Clay smaller than 0.002mm	55
Total	100

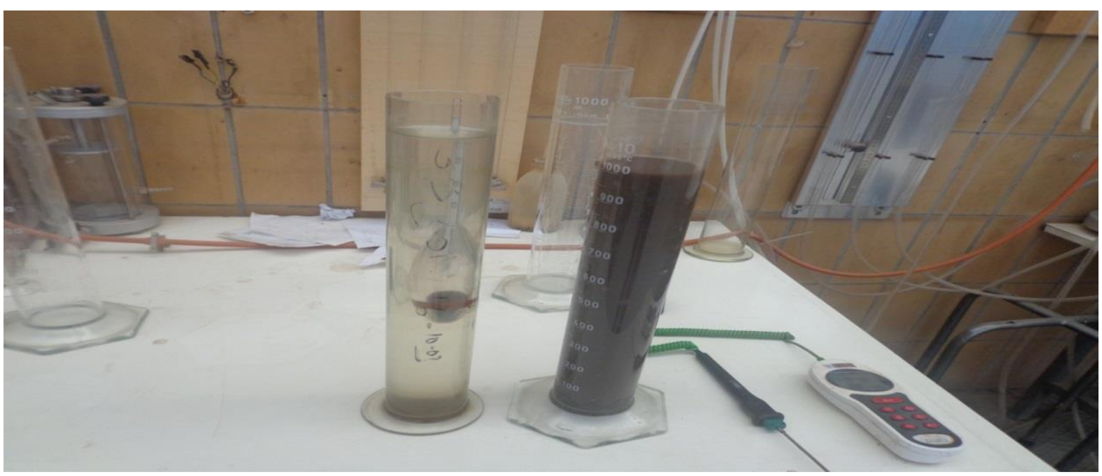


Figure 9 Hydrometer used in this study

Table 15 Dry sieve analysis of expansive soil

Total mass (Tm) = 4188g

Sieve No	Sieve opening (mm)	Mass of sieve(g)	Mass of sieve + Re. soils(g)	Mass of Retained soils(g)	Percentage Retained (%)	Cum. Retained (%)	Perc. Passing (%)
1/2"	12.5	1215.7	1215.7	0.0	0.0	0.0	100.0
3.8"	9.5	1217.3	1226.0	8.7	0.2	0.2	99.8
No 4	4.75	1261.2	1879.4	618.2	14.8	15.0	85.0
No 8	2.36	988.2	1389.2	401.0	9.6	24.5	75.5
No 10	2	953.8	992.3	38.5	0.9	25.5	74.5
No 16	1.18	892.5	998.8	106.3	2.5	28.0	72.0
No 30	0.6	829.2	926.3	97.1	2.3	30.3	69.7
No 40	0.425	784.6	857.8	73.2	1.7	32.1	67.9
No 50	0.3	748.5	805.6	57.1	1.4	33.4	66.6
No 100	0.15	745.8	781.3	35.5	0.8	34.3	65.7
No 200	0.075	761.5	768.4	6.9	0.2	34.4	65.6
Pan	-----	733.1	3473.7	2740.6	65.4	99.9	-------

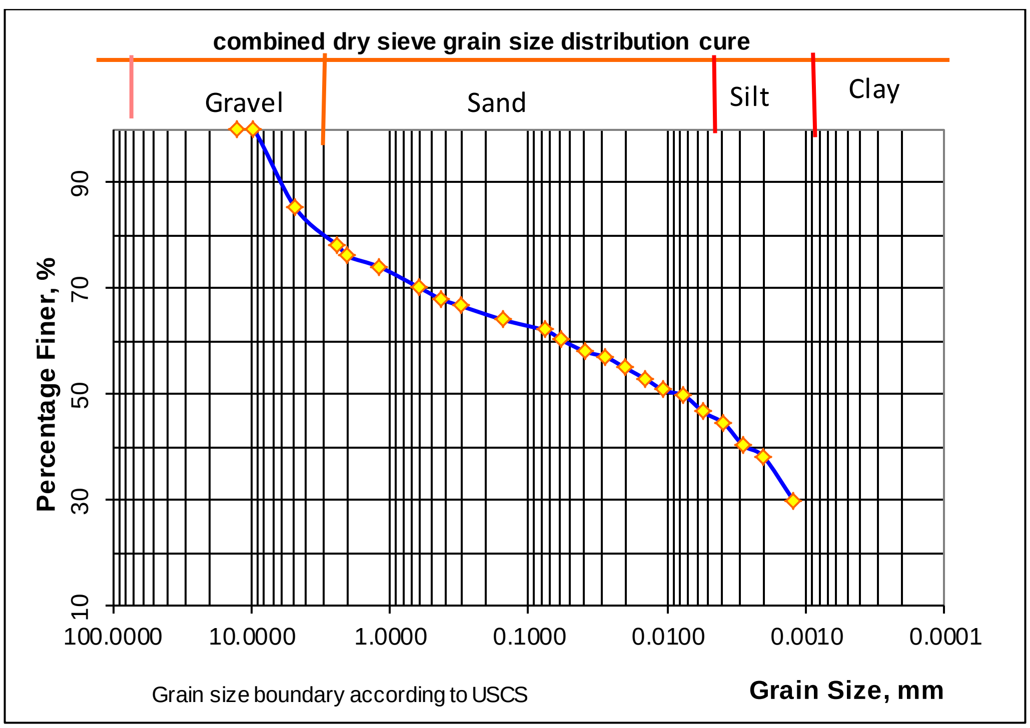


Figure 10 combined dry sieve grain size distribution cure

From the above combined dry grain size distribution cure the expansive soil was classified USCS. The gravel 25%, Sand 9%, silt 27% and the clay content was 38.2%.

4.3 Atterberg limit test

Atterberg limit test were conducted according to ASTM D-4318 standard. An Atterberg limits device was used to determine the liquid limit of soil using the soil passing through a 475 µm (No. 40) sieve. Representative sample of soil were subjected to Atterberg limits testing to determine the plasticity of the soils. The plastic limit of soil was determined by using soil passing through a 475 µm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for soil based on the liquid and plastic limit obtained. The liquid limit and plastic limit tests were conducted on the expansive soil and treated soil samples. The liquid limit and plasticity index were used to classify soils. The test results of atterberg limits for bagasse ash and cement treated soil are presented in below Table.

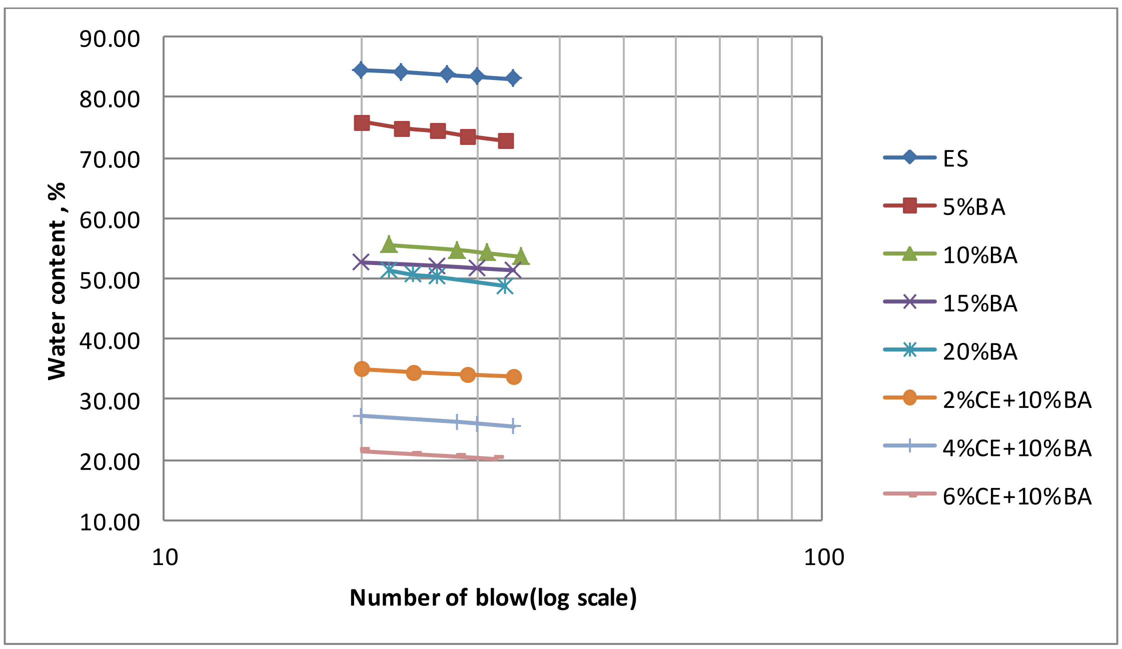


Figure 11 liquid limit of expansive and treated soil

Table 16 Summery of liquid limit and plastic limit for the expansive soil and mix soils

Mix No	Mix Type	LL (%)	PL (%)	PI (%)	%Decrease PI Value
1	100 % ES	84	34.1	49.9	---
2	95 % ES+ 5% BA	74.5	31.6	42.9	14.0
3	90 % ES + 10 % BA	55	24.3	30.7	38.5
4	85 % ES + 15% BA	52	23.6	29.2	41.5
5	80 % ES + 20% BA	50.5	22.8	27.7	44.5
6	88 % ES + 10% BA+2% CE	34	19	15	68.9
7	86 % ES + 10% BA+4% CE	27	14.2	12.8	74.3
8	84 % ES+ 10% BA+6% CE	21	11.4	9.6	80.8
9	82 % ES + 10% BA+8% CE	18	9.3	8.7	82.6

The Atterberg limits are a basic measure of the nature of a fine-grained soil. Liquid limit results for soil, bagasse ash and cement combinations at different percentages were shown in above Table. The general decrease in liquid limit at all soil - bagasse ash combination was attributed to the fact that the bagasse ash reaction forms compounds possessing cementations properties with soil particles. This trend conforms to finding that the liquid limit reduces with increasing bagasse ash and cement combinations [18] and [19].

The variations of plastic limit for the soil- bagasse ash and cement combination at different percentages were shown in Table16. For the soil- combinations at different percentage of bagasse ash and cement, there was a general decease in plastic limit with increasing bagasse ash and cement content. The plasticity index decreases with increasing bagasse ash and cement content. A reduction in plasticity index causes a significant decrease in swell potential.

4.4 Soil classification

The soil classification was made according to ASTM D-2487 and AASHTO soil classification standards. Based on the test result, the soil was classified as CH according to Unified soil classification and A-7-6 according to AASHTO, as shown in below Table.

Table 17 Expansive soil classification

Depth	LL	PL	PI	Free swell index	percentage passing sieve NO.200	Unified soil classification	AASHTO
2m below	84	34	50	118	91	CH	A-7-6

4.5 Free swell index

The free swell index test was conducted in accordance with IS: 2720 (Part 40) 1977. This test have been performed by placing 10gm of dry soil passing the No 40 sieve into a graduated cylinder one filled with water and another with kerosene to 100 ml. Swelled volume of the soil in water is measured after 24 hours and the volume of soil in kerosene gives initial volume of the samples.

$$\text{Free swell} = \frac{\text{Final volume- Initial volume}}{\text{Initial volume}} * 100 \text{-----------} (\text{Eq.4.1})$$

Table 18 Summery of average free swells index for expansive soil and mixed soils.

Mix No	Mix Type	Free swelling index (%)	Percent decrease in free swelling index
1	100 % ES	118.18	----
2	95 % ES+ 5% BA	76.47	35.3
3	10 % ES + 10% BA	54.00	54.3
4	85 % ES + 15% BA	45.00	61.9
5	80 % ES + 20% BA	41.00	65.3
6	88 % ES + 2% CE+ 10% BA	30.00	74.6
7	86% ES + 4% CE + 10% BA	10.00	91.5
8	84 % ES + 6% CE + 10% BA	4.00	96.6
9	82 % ES + 8% CE+ 10% BA	2.00	98.3

From the table 18, the expansive soil samples have been a free swell index value nearly 118%. The potential of expansive soil was high and the soil has been treated as highly expansive. The result show that with the addition of bagasse ash there was a considerable reduction in the free swell index values from 118% to 76.47%, 54% with 5% bagasse ash and 10% bagasse ash respectively.

With the addition of 2%cement, 4% cement, 6%cement and 8%cement with constant 10% bagasse ash, the free swell index value decreases from 118% to 30%, 10%, 4% and 2% respectively. Further increasing percent of cement the free swelling index become zero.

4.6 Moisture-Density relations

After determining natural moisture content of expansive soil, the samples were compacted according to ASTM D-698 with standard Proctor test to find moisture-density relationships of untreated soil samples.

Air dried soil passing no 4 sieve was used to determine moisture-density relation of the soil mixed with varying proportions of the additives. The soil was mixed with ratios of 5%, 10%, 15% and 20% of bagasse ash and 2%, 4% 6% and 8% of cement with a fixed amount of 10% bagasse ash. Moisture content versus dry density graph was produced and optimum moisture content (OMC) and maximum dry density (MDD) were determined from the graph. The summery test results are shown in below Table.

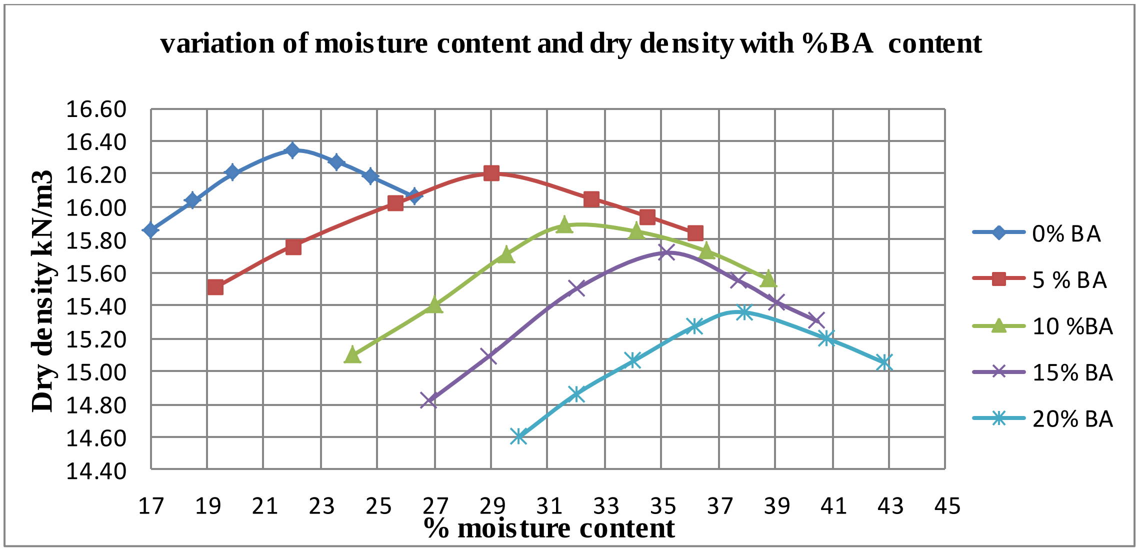


Figure 12 Moisture content versus dry density graph with variation of BA

Table 19 Summary of MDD and OMC for expansive soils and mix soils

Mix No	Mix Type	Maximum dry density (kg/m^3)	Optimum moisture content (%)
1	100 % ES	16.34	21.5
2	95 % ES + 5% BA	16.2	29
3	10 % ES + 10% BA	15.89	32
4	85 % ES + 15% BA	15.63	35
5	80 % ES + 20% BA	15.32	37
6	88 % ES + 2% CE+10% BA	15.73	33
7	86% ES + 4%CE+10% BA	15.61	35
8	84 % ES + 6% CE+10% BA	15.3	36
9	82 % ES + 8%CE+10% BA	14.89	38

The result shows that the MDD and OMC for expansive soil were 16.35 kN/m^3 and 22%, respectively. The OMC increases from 22% to 31.5% and the MDD decreases from

16.35kN/m^3 to 15.89kN/m^3 from virgin soil to 10% bagasse ash treated soil. The MDD and OMC for the optimum mix were 15.89kN/m^3 and 31.5%, respectively at 10% of bagasse ash. The optimum mix was determined from the UCS test results which are discussed in the next section. From the Table 19, the OMC of treated soil increases with increase in bagasse ash but the MDD decreases with increase in bagasse ash. This could be due to the absorption of excess water by bagasse ash [7], the relatively lower specific gravity of bagasse ash and change in particle size compared to clay soil [7], [18] and [21]. From Table 19 the maximum dry density of soil decreases gradually with an increase of cement content. The reduction in dry density was a result of flocculation and agglomeration of fine grained soil particles which occupies larger space leading to a corresponding drop in maximum dry density. It was also the result of initial coating of soils by cement to form larger aggregate, which consequently occupy larger spaces [19] and [22]. On the other hand, the optimum moisture content of soil increases with increase cement content, because cement is finer than soil. The more fines the more surface area, so more water was required to provide well lubrication. The cement content also decrease the quantity of free silt and clay fraction, forming coarser materials, which occupy larger spaces for retaining water. The increase of water content was also attributed by the pozzalanic reaction of cement with the soil [19] and [22]. Therefor both bagasse ash and cement reduce the dry density of natural soil. The decrease in dry density leads to low compactive energy and the cost of compaction becomes economical for soils compacted at low compactive energy [22].

4.7 Unconfined Compressive Strength (UCS) test and result

Unconfined compressive strength was conducted according to ASTM D-2166 standard. The Unconfined compressive strength test samples were prepared with optimum moisture and maximum dry density with standard proctor test. The prepared sample were cured for different day such as 7day, 14day and 28day .The test was conducted on sample having a dimension of 38mm diameter and 76mm height. Unconfined compressive strength testing was performed on all extracted specimens using a strain rate of approximately one percent per minute. A data acquisition system was used to record the applied load and deformation. Corrections to the cross-sectional area were applied prior to calculating the compressive stress on the specimen. Each specimen was loaded until peak stress was obtained. The detail test results of unconfined compressive strength have been shown in Table 20. The most critical condition for the soil usually occurs immediately after construction, which represents the un drained condition, when the untrained shear strength is basically equal to cohesion(c).This expressed as:

$$S_u = C = \frac{qu}{2} ------------------ (Eq.\ 4.2)$$

To calculate the axial strain to the nearest 0.1 %, for a given applied load, as follows:

$$\varepsilon_1 = \frac{\Delta L}{L} ---------------------------- (Eq. 4.3)$$

Where, ΔL= change in elevation and

L=initial length of the sample

The average cross sectional area, for the given applied load

$$A = \frac{A_o}{(1-\varepsilon_1)} ------------------------------ (Eq. 4.4)$$

Where, A_o=Initial cross sectional area of the specimen and

ε_1=axial strain for the given load, %

Compressive stress,

$$\sigma_c = \frac{P}{A} -------------------------------- (Eq4.5)$$

Where, A= average cross sectional area and

P= applied load, KN

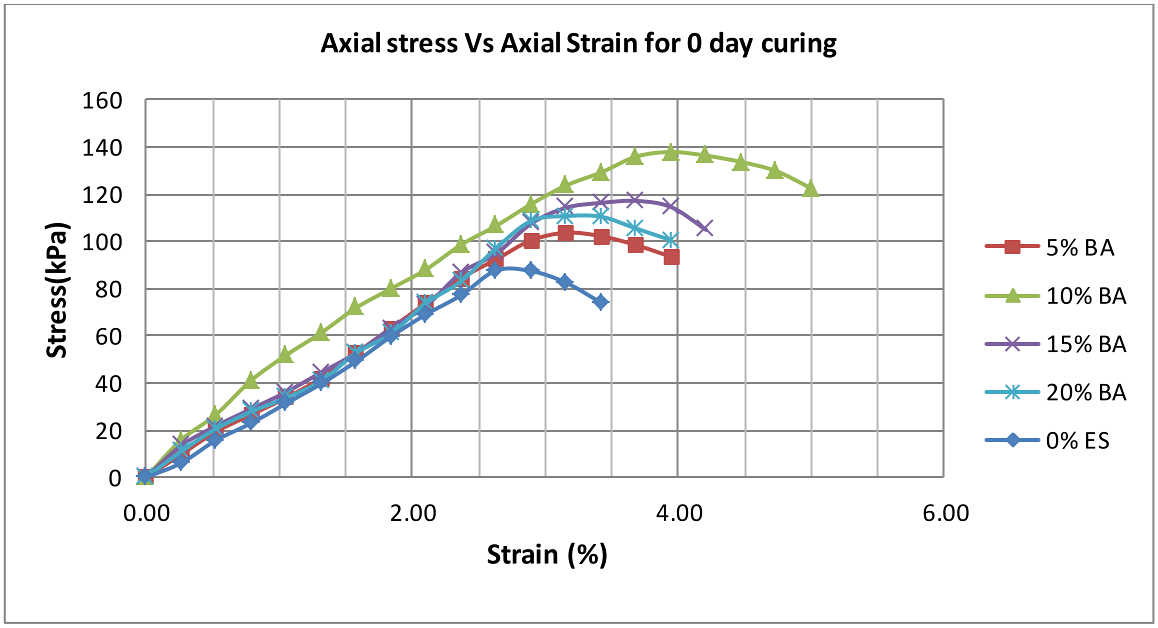


Figure 13 Axial stress vs Axial strain for 0 day curing with variation of BA

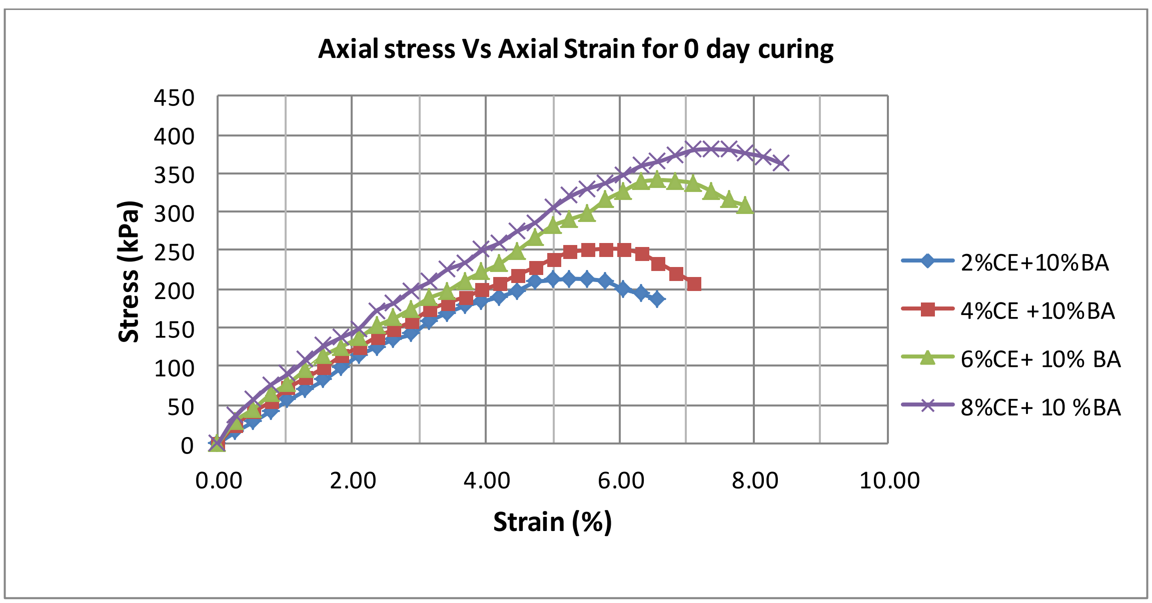


Figure 14 Axial stress vs Axial strain for 0 day curing with variation of CE

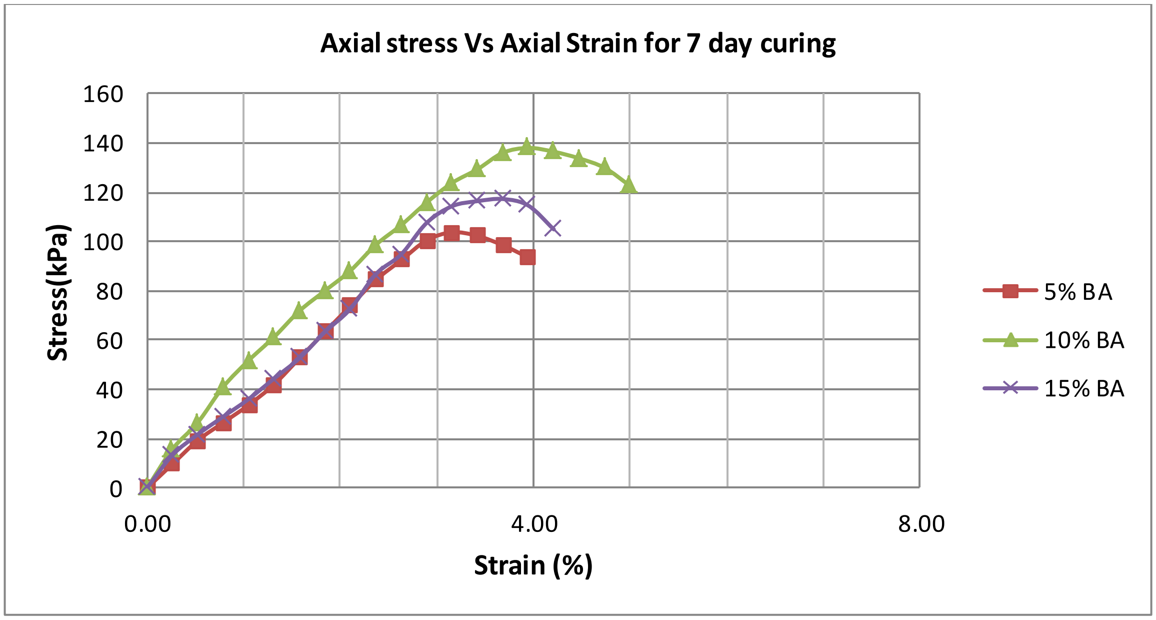


Figure 15 Axial stress vs Axial strain for 7 day curing with variation of BA

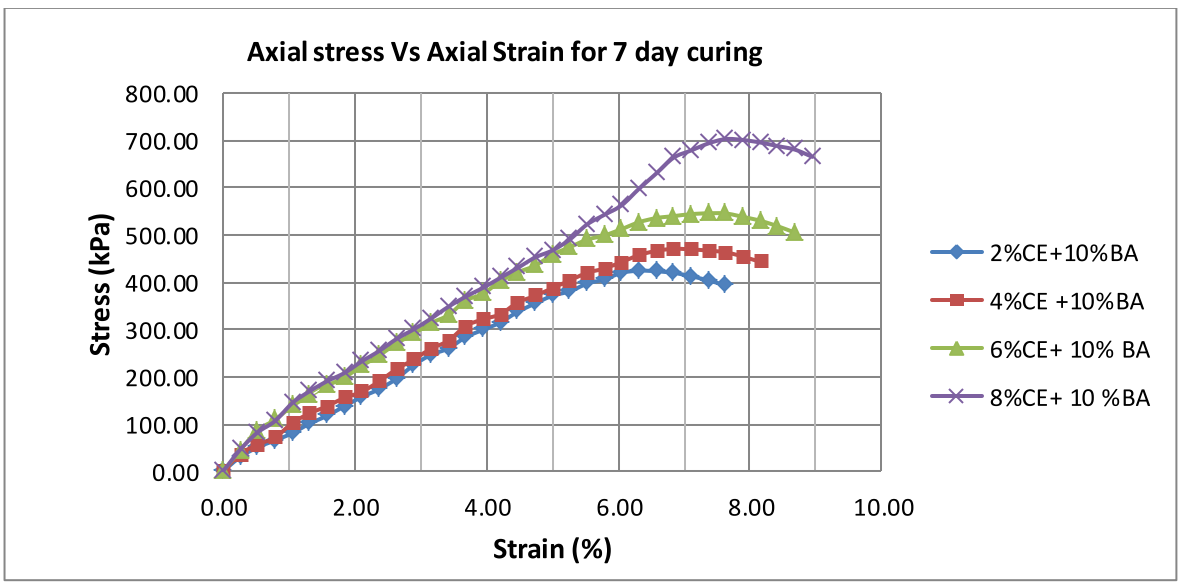


Figure 16 Axial stress vs Axial strain for 7 day curing with variation of CE

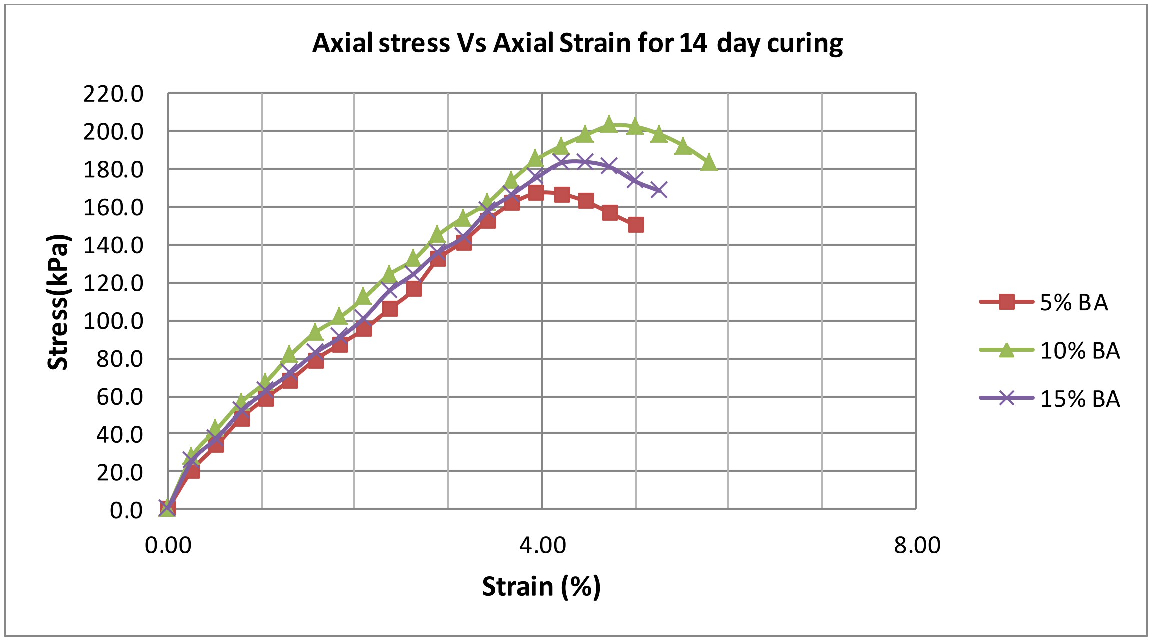


Figure 17 Axial stress vs Axial strain for 14 day curing with variation of BA

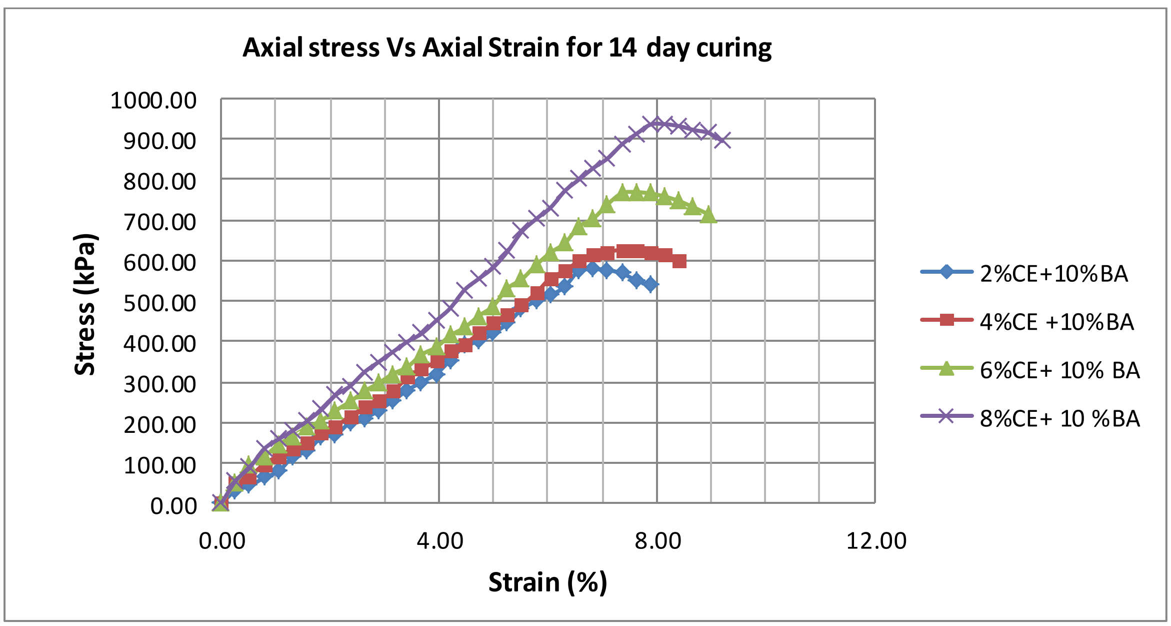


Figure 18 Axial stress vs Axial strain for 14 day curing with variation of CE

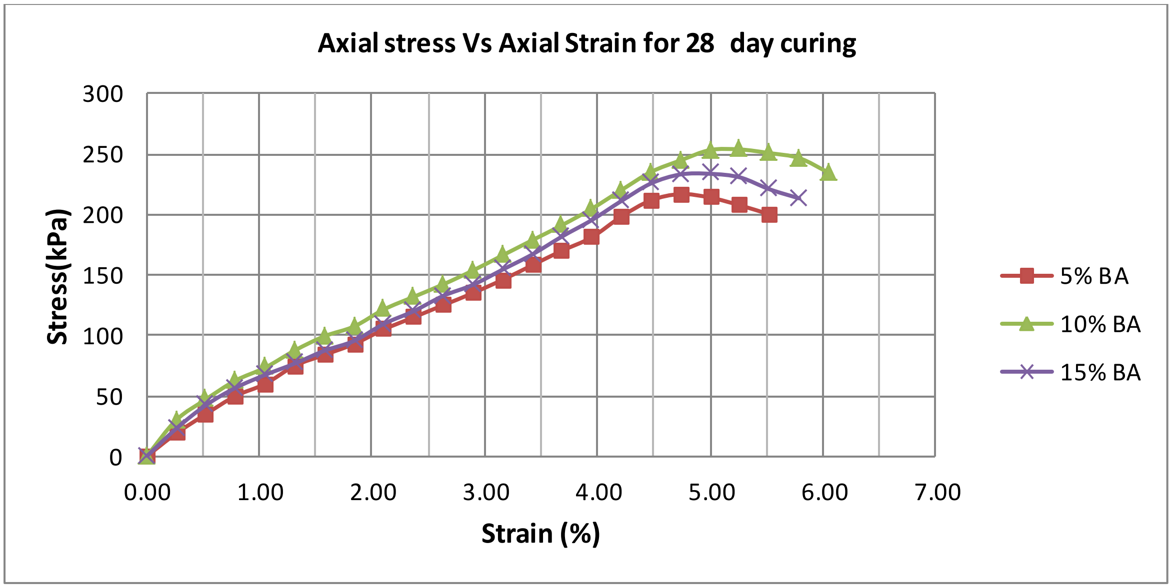


Figure 19 Axial stress vs Axial strain for 28 day curing with variation of BA

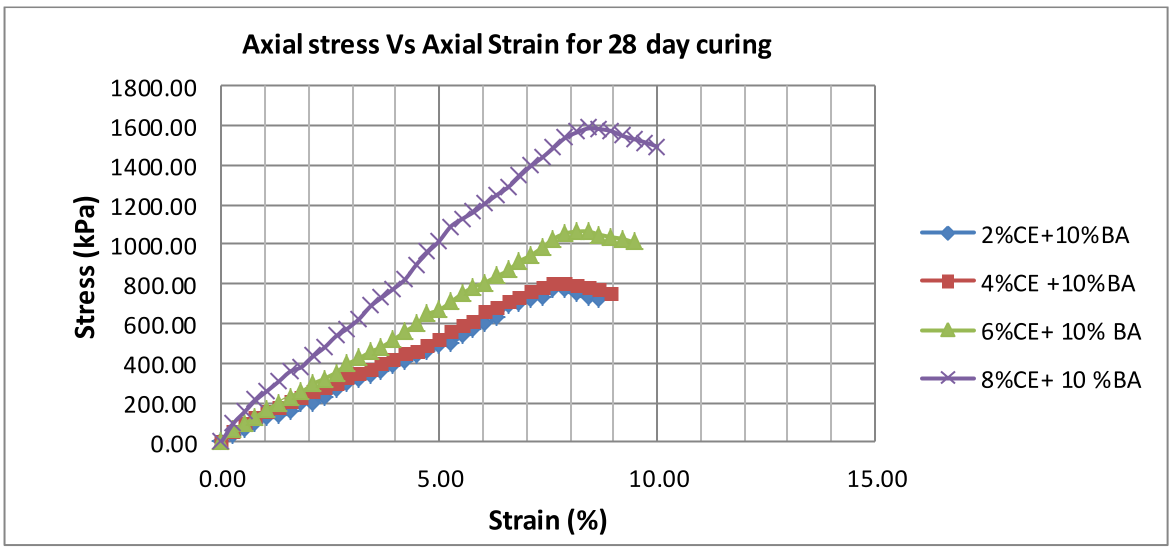


Figure 20 Axial stress vs Axial strain for 28 day curing with variation of CE

Table 20 Summary of average UCS test result for expansive soil and mix soil

Mix code	Mix type	UCS value (kPa)			
		0 day curing	7 day curing	14 day curing	28 day curing
1	100 % ES	87.7	---	---	---
2	95 % ES+ 5% BA	103.7	133.6	167.2	216.8
3	10 % ES+ 10% BA	137.9	168.6	202.9	253.6
4	85 % ES+ 15% BA	117.2	147.3	183.7	233.5
5	80 % ES + 20% BA	110.8	---	---	---
6	88 % ES +2% CE+10% BA	213.3	424.1	579.3	770
7	86 % ES +4% CE+10% BA	252.2	470.5	626.8	801.4
8	84 % ES+6% CE+10% BA	341.1	546.7	768.9	1065.4
9	82 % ES +8 % CE+10% BA	381	702.5	936.6	1587

Unconfined compressive strength (UCS) is the most common and adaptable method of evaluating the strength of stabilized soil. It is the main test recommended for the determination of the required amount of additive to be used in stabilization of soil [12]. The optimum mix was having the highest UCS values for the given proportion and which was found to be 90% Soil and 10% bagasse ash for two mixes.

The laboratory test result of unconfined compressive strength has been shown in Table 20. The UCS value of expansive soil increase from 88kN/m^2 to 104kN/m^2, 133.6kN/m^2, 167.2kN/m^2 and 216.8kN/m^2 with increase at 5% bagasse ash for curing period of 0, 7, 14 and 28 day respectively. At 10% increase in bagasse ash the UCS value increases from 88kN/m^2 to 137.9kN/m^2, 168.6kN/m^2, 202.9kN/m^2 and 253.6kN/m^2 at 0, 7, 14 and 28 curing day respectively. With addition of bagasse ash the strength of treated soil increases with increase in curing period. This could be the result of the increased pozzolanic reaction with increased curing day which results in the formation of calcium silicate hydrates and micro fabric changes which was responsible for increased strength [17].

The variation of unconfined compressive strength for soil at different percentages of cement and bagasse ash at 0 day, 7 day, 14day and 28 days have been shown in Table20. There was a rapid increase of unconfined compressive strength with the addition of cement and 10% bagasse ash. The reason for this improvement is due to the pozzolanic reactions of cement and bagasse ash with soil. This results in agglomeration in large size particles and causes the increase in compressive strength [17].

Generally, the result shows that the strength of the treated soil has been increased with increase cement plus 10% of bagasse ash. For different curing periods, the strength of the treated soil increases with increase in curing periods as shown in below Figure.

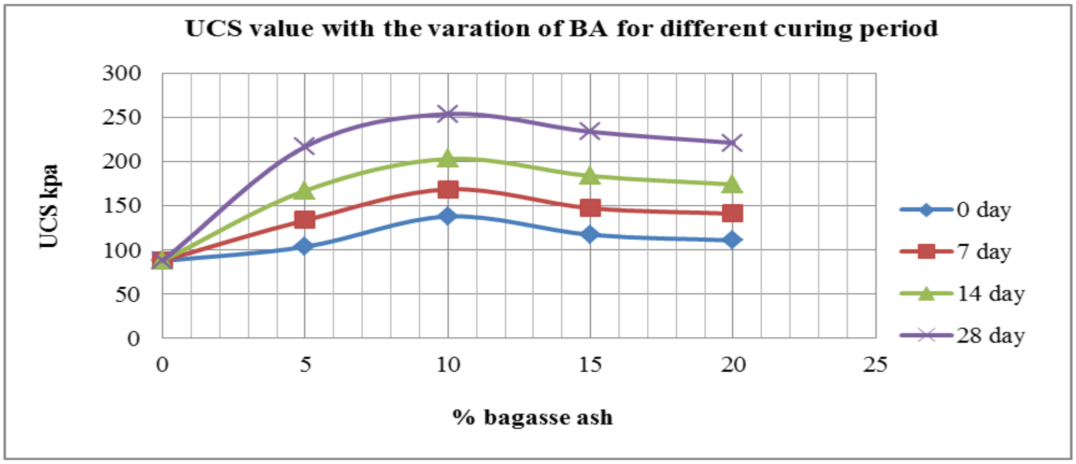


Figure 21 UCS value with the varation of BA

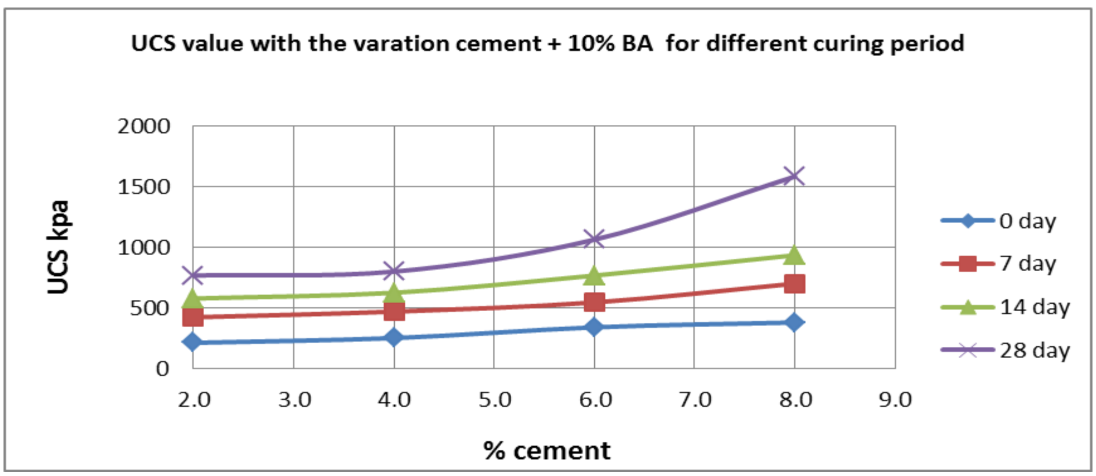


Figure 22 UCS value with the varation of cement +10%BA

4.8 California Bearing Ratio (CBR) test and results

The CBR tests were carried in accordance to AASHTO T193 and ASTM D1883. The CBR tests samples were prepared with optimum moisture and maximum dry density with CBR mold. The CBR sample were prepared by a standard mold with an internal diameter of 152.4 mm and a height of 177.8 mm then specimens were compacted in 5 layers with 65 blows. Two samples are usually prepared for CBR tests; one is tested directly after sample preparation and the other after soaking in water for 4day, 7day and 14day. The result of CBR value have been tabulated in the below table.

The amount of water to be added was calculated=

$$M(\frac{(W_o-W_1)}{(100+W_1)})---------------------------(Eq.4.6)$$

Where, M=mass of air dry sample,

M_o= optimum moisture content and

W_1=natural moisture content

$$CBR = \frac{\mathit{Measured}\ \text{pressure}}{\text{Standared pressure}}*100----------------(Eq.4.7)$$

Table 21 4 day Soaked CBR value of expansive soil (ES)

	Trial 1			Trial 2			
pen(mm	Dial RDG	load(KN)	CBR	Dial RDG	load(KN)	CBR	Average CBR %
0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.64	0.00	0.00	0.00	0.00	0.00	0.00	
1.27	5.00	0.10	0.75	10.00	0.20	1.50	
1.91	5.00	0.10	0.75	10.00	0.20	1.50	
2.54	12.00	0.24	1.80	14.00	0.28	2.10	1.95
3.81	15.00	0.30	2.25	18.00	0.36	2.70	
4.45	17.00	0.34	2.55	20.00	0.40	3.00	
5.08	18.00	0.36	1.79	20.00	0.40	1.99	1.89
7.62	32.00	0.64	2.42	42.00	0.83	3.17	
10.16	47.00	0.93	2.94	65.00	1.29	4.06	

Table 22 4 day soaked CBR value of 95% ES+ 5%BA

pen(mm)	Trial 1			Trial 2			
	Dial RDG	load(KN)	CBR	Dial RDG	load(KN)	CBR	Average CBR
0	0	0	0	0	0	0	
0.64	5	0.1	0.75	10	0.2	1.5	
1.27	10	0.2	1.5	15	0.3	2.25	
1.91	15	0.3	2.25	15	0.3	2.25	
2.54	25	0.5	3.75	23	0.46	3.45	3.6
3.18	30	0.6	4.5	25	0.5	3.75	
3.81	30	0.6	4.5	25	0.5	3.75	
4.45	32	0.64	4.8	30	0.6	4.5	
5.08	36	0.72	3.58	33	0.66	3.28	3.43
7.62	42	0.83	3.17	50	0.99	3.78	
10.16	57	1.13	3.56	60	1.19	3.75	

Table 23 4day Soaked CBR value of 90 %ES+ 10% BA

	Trial 1			Trial 2			
pen(mm)	Dial RDG	load(KN)	CBR	Dial RDG	load(KN)	CBR %	Average CBR%
0	0	0	0	0	0	0	
0.64	20	0.4	3	15	0.3	2.25	
1.27	30	0.6	4.5	25	0.5	3.75	
1.91	37	0.74	5.55	34	0.68	5.1	
2.54	52	1.03	7.8	48	0.95	7.2	7.5
3.18	55	1.09	8.25	50	0.99	7.5	
3.81	57	1.13	8.55	54	1.07	8.1	
4.45	65	1.29	9.75	60	1.19	9	
5.08	70	1.39	6.95	65	1.29	6.46	6.71
7.62	85	1.69	6.42	77	1.53	5.82	
10.16	98	1.95	6.12	90	1.79	5.62	

Table 24 4 day Soaked CBR value of 88% ES + 10% BA+ 2% cement

pen(mm)	Trial 1			Trial 2			
	Dial RDG	load(KN)	CBR %	Dial RDG	load(KN)	CBR%	Average CBR %
0	0	0	0	0	0	0	
0.64	50	0.99	7.5	45	0.89	6.75	
1.27	75	1.49	11.26	80	1.59	12.01	
1.91	120	2.38	18.01	130	2.58	19.51	
2.54	180	3.58	27.01	190	3.78	28.51	27.76
3.18	210	4.17	31.52	220	4.37	33.02	
3.81	234	4.65	35.12	240	4.77	36.02	
4.45	244	4.85	36.62	250	4.97	37.52	
5.08	265	5.27	26.33	270	5.36	26.82	26.58
7.62	324	6.44	24.48	330	6.56	24.93	
10.16	377	7.49	23.56	380	7.55	23.74	

Table 25 4 Soaked CBR value of 86%ES + 10%BA+ 4% cement

	Trial 1			Trial 2			
pen(mm)	Dial RDG	load(KN)	CBR %	Dial RDG	load(KN)	CBR %	Average CBR%
0	0	0	0	0	0	0	
0.64	70	1.39	10.51	80	1.59	12.01	
1.27	120	2.38	18.01	135	2.68	20.26	
1.91	150	2.98	22.51	170	3.38	25.51	
2.54	260	5.17	39.02	266	5.29	39.92	39.47
3.18	300	5.96	45.02	295	5.86	44.27	
3.81	340	6.76	51.03	320	6.36	48.02	
4.45	385	7.65	57.78	360	7.15	54.03	
5.08	405	8.05	40.24	375	7.45	37.26	38.75
7.62	510	10.13	38.53	490	9.74	37.02	
10.16	564	11.21	35.24	510	10.13	31.87	

Table 26 4 day soaked CBR value of 84%ES + 10%BA+ 6%cement

pen(mm)	Trial 1			Trial 2			
	Dial RDG	load(KN)	CBR %	Dial RDG	load(KN)	CBR %	Average CBR%
0	0	0	0	0	0	0	
0.64	160	3.18	24.01	180	3.58	27.01	
1.27	270	5.36	40.52	280	5.56	42.02	
1.91	350	6.95	52.53	365	7.25	54.78	
2.54	400	7.95	60.03	412	8.19	61.83	60.93
3.18	480	9.54	72.04	490	9.74	73.54	
3.81	520	10.33	78.04	510	10.13	76.54	
4.45	560	11.13	84.04	550	10.93	82.54	
5.08	590	11.72	58.62	585	11.62	58.12	58.37
7.62	670	13.31	50.62	615	12.22	46.46	
10.16	790	15.7	49.36	755	15	47.18	

Table 27 4 day soaked CBR value of 82%ES + 10% BA+ 8% cement

pen(mm)	Trial 1			Trial 2			
	Dial RDG	load(KN)	CBR %	Dial RDG	load(KN)	CBR%	Average CBR %
0	0	0	0	0	0	0	
0.64	205	4.07	30.77	250	4.97	37.52	
1.27	350	6.95	52.53	405	8.05	60.78	
1.91	485	9.64	72.79	498	9.9	74.74	
2.54	760	15.1	114.06	780	15.5	117.06	115.56
3.18	835	16.59	125.31	855	16.99	128.31	
3.81	880	17.49	132.07	905	17.98	135.82	
4.45	965	19.17	144.82	985	19.57	147.82	
5.08	1100	21.86	109.29	1145	22.75	113.76	111.52
7.62	1205	23.94	91.04	1250	24.84	94.44	
10.16	1335	26.53	83.42	1356	26.94	84.73	

Table 28 Summary of average CBR test result for expansive soil and mix soils

Mix No	Mix type	CBR value (%)			
		Unsoaked	4 day soaked	7 day soaked	14 day soaked
1	100 % ES	8.3	2	1.7	---
2	95 % ES+ 5% BA	9	3.6	3.2	---
3	10 % ES + 10%BA	14.3	7.5	6.9	---
4	85 % ES+ 15%BA	12	6.2	5.7	---
5	80 % ES + 20%BA	8.7	3.2	3	---
6	88% ES + 2% CE+10%BA	25.7	27.8	35.1	41.1
7	86% ES + 4%CE+10%BA	36	39.6	49.2	55.4
8	84 % ES + 6% CE+10%BA	51.9	60.9	74.7	80.7
9	82 % ES + 8%CE+10%BA	93.9	115.6	125.3	131.4

The CBR test is a simple strength test that compares the bearing capacity of a material with that of a well-graded crushed stone (thus, a high quality crushed stone material should have a CBR @ 100%) [1].

The California Bearing Ratio (CBR) test is a relatively simple test that is commonly used to obtain an indication of strength of a subgrade soil, sub-base and the base course materials for use in road and airfield pavement design [2]. The results of the California Bearing Ratio test are shown in the Table 28.

From the Table 28 shown that both unsoaked and soaked CBR value of the soil with bagasse ash mixture was increased linearly up to 10% bagasse ash content, further addition of bagasse ash content the CBR value was slightly decreased. The decrease CBR value after 15% bagasse ash content due to the excess amount of bagasse ash content which was not mobilized in the reaction as the presence of naturally occurring CaOH in soil is small. The excess bagasse ash occupies space within the specimen and reduces the clay and silt content in soil and hence reduces the bond/cohesion in the soil- bagasse ash mixture [17]. The reason increase in CBR value due to addition of bagasse ash attributed to the gradual formation of cementitious compounds between the bagasse ash and CaOH naturally present in soil [17]. The value of soaked CBR remain below the unsoaked CBR although both value increases linearly with up to 10% bagasse ash content. Addition of cement and 10% bagasse ash to the soil shows marked improvement in the CBR value compared to the low CBR value of 2% recorded for the soaked expansive soil. Both unsoaked and soaked CBR values increase with increased in cement contents for specific 10% bagasse ash contents. The increase in CBR is due to the cementing property of cement and the pozzolanic property of bagasse ash. These improvements resulted from the secondary cementitious materials resulted from the reaction between the cement liberated from the hydration reaction of cement and the pozzolanic property of bagasse ash. This reaction also contributed to inter particle bonding [16] and [17]. There for the increase in CBR value shows that the treated material quality is good enough for subgrade material.

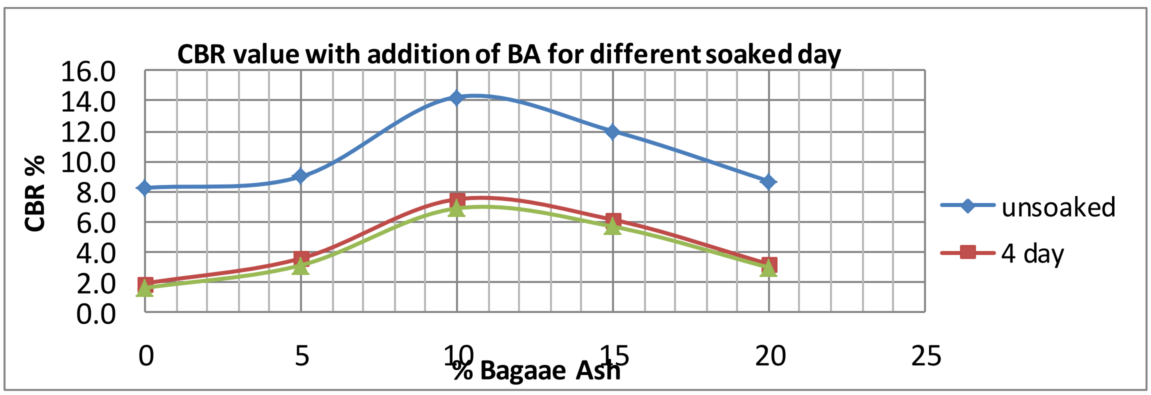


Figure 23 CBR value with addition of Bagasse Ash

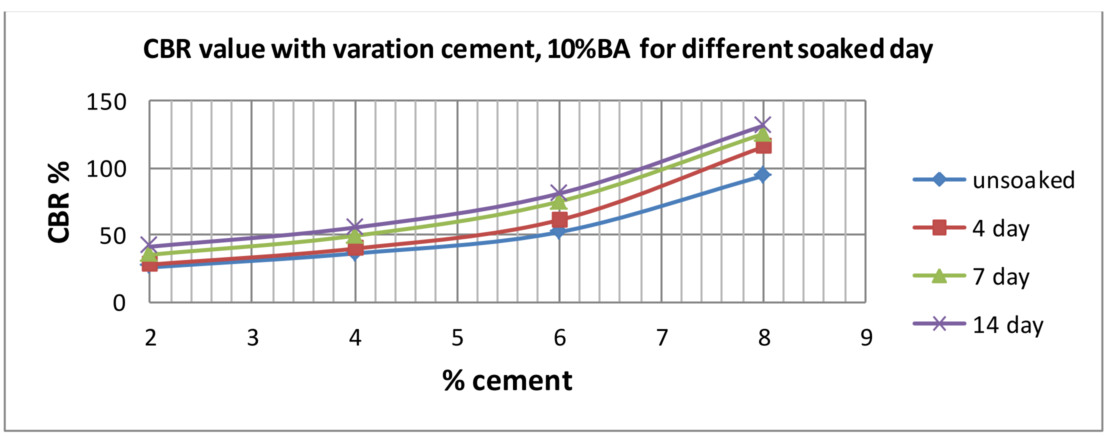


Figure 24 CBR value with addition of cement

4.9 Swelling pressure tests and results

This test was done in accordance with ASTM D- 2435 standard with one dimensional consolidation method. Swell pressure test samples was prepared at maximum dry density and optimum moisture content by using standard Proctor's test .The sample sizes were 50 mm and 20 mm in diameter and height respectively.

In this test the samples were placed in the consolidation ring height of 20mm then, subjected to under a 7 kPa applied load is wetted and allowed to fully swell. At this point a standard consolidation test is conducted by applying incremental loads starting with 50 kPa and ending with 1600kPa. The pressure required to revert the specimen to its initial void ratio (height) is used to define the swelling pressure. The swelling pressure tests were conducted on the treated and expansive soil samples. The summaries of swelling pressure and swelling potential test results have been given in Table 30.

Table 29 Swelling pressure test result for expansive soil

Applied pressure, (kPa)	Final Dial Reading (mm)	Change In Specimen Height (mm)	Final Specimen Height(mm)	Void Height, Hv (mm)	Void Ratio, E
Loading					
7	6.5	0	20	9.56	0.92
7	4.42	-2.08	22.08	11.64	1.11
50	5.08	-1.42	21.42	10.98	1.05
100	5.31	-1.19	21.19	10.75	1.03
200	6.17	-0.33	20.33	9.89	0.95
400	7.15	0.65	19.35	8.91	0.85
800	8.01	1.51	18.49	8.05	0.77
1600	8.43	1.93	18.07	7.62	0.73
1600	8.43	1.93	18.07	7.62	0.73
800	8.33	1.83	18.17	7.73	0.74
400	8.24	1.74	18.26	7.82	0.75
200	8.14	1.64	18.36	7.92	0.76
100	7.8	1.3	18.7	8.26	0.79
50	7.42	0.92	19.08	8.64	0.83
7	7.17	0.67	19.33	8.88	0.85

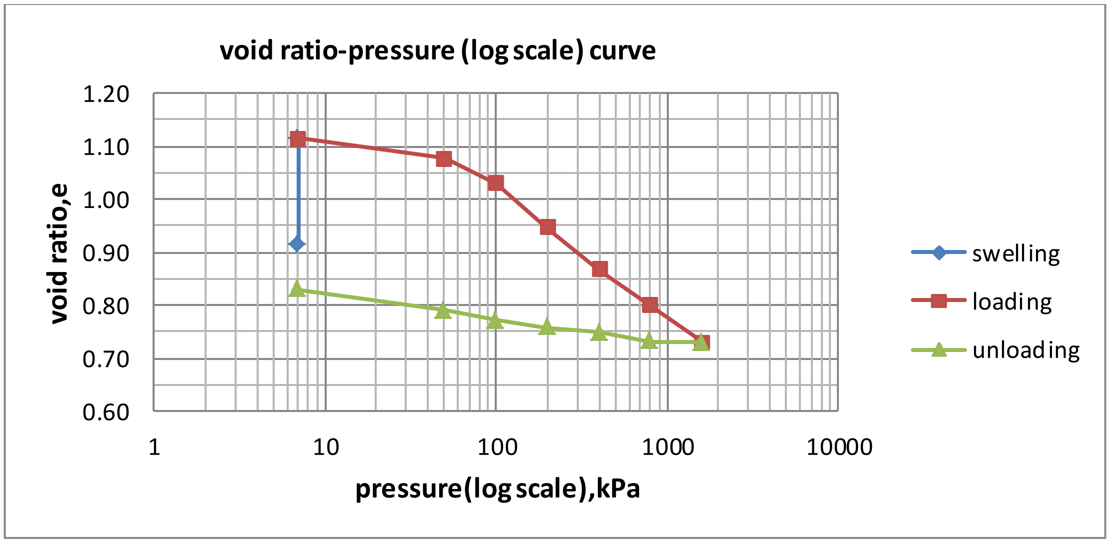


Figure 25 Plot of void ratio versus Log-Pressure for expansive soil

Table 30 Summary of swelling pressure and swelling potentials

Mix No	Mix type	Swelling potential (%)	Swelling pressure (kPa)	% Decrease swelling potential	% Decrease swelling pressure
1	100 % ES	10.39	250	---	---
2	95 % ES + 5% BA	6.5	165	37.4	34
3	90 % ES + 10%BA	4.9	100	53.3	60
4	85 % ES+ 15%BA	4	80	61.5	68
5	88 % ES + 2% CE+10%BA	1.75	60	83.2	76
6	86% ES +4%CE+10%BA	1.5	50	85.6	80
7	84 % ES + 6% CE+10%BA	0.75	18	92.8	92.8
8	82 % ES + 8%CE+10%BA	0.35	10	96.6	96

The variation of the swell potential with the addition of the bagasse ash and cement was shown in Table30. Expansive soil sample have been swelling potential of 10.4% and gradually reduced to 6.5% and 5% with the addition of 5% bagasse ash and 10%BA respectively. With the addition of 2% cement, 4% cement, 6% cement and 8% cement and 10% bagasse ash was considered as optimum; the swell potential reduced to 1.75%, 1.5%, and 0.75% and 0.35% respectively. Generally, the result shows that the swelling potentials of treated soil decrease with increase bagasse ash and cement content.

In the Table 30 the variation of swell pressure with the addition of bagasse ash and cement to the expansive soil samples was presented. Based on the laboratory test result the expansive soil sample was a swell pressure 250 kPa and gradually reduced with the addition of bagasse and cement. The swell pressure values were reduced by present of 34% and 60% with the addition of 5% bagasse ash and 10% bagasse ash respectively. Similarly with addition of 2% cement, 4% cement, 6% cement and 8% cement with 10% bagasse ash was considered as optimum and the swell pressure reduced by present of 76%, 80%, and 92.8% and 96% respectively. Further increase the amount of bagasse ash and cement both swelling potentials and swelling pressure become zero.

Figure 26 One dimensional consolidation used in this study

4.10 Sample of cost comparison analysis

Sample direct cost comparison analysis was done both the treated soils and replaced sub grade and sub base expansive soils. The analysis was done considering only excavation, cart away and transportation, compaction and materials costs for 1m ^{3} sub grade works. Accordingly, the direct unit cost for treated soil and replaced for 1m^3 sub grades and sub base works are tabulated as shown Table31 and the detail unit rate analysis are shown in Appendix.

Table 31 Summary of direct cost comparison analysis

Item	Unit	Price per unit (Birr/m^3)		
		Treated soil		Replaced
		2%CE, 88%ES and 10% BA	4%CE, 86%ES and 10% BA	100% selected material
Sub grade	m^3	326.24	344.04	400.6
Sub base	m^3	311.44	293.64	341.08

From Table 31 it can be observed that for 1m^3 of 88% expansive soil, 2% cement and 10% bagasse ash stabilized subgrade work was taken 326.24 Birr and for 1m^3 of 100% replaced sub grade take 400 Birr. From economy point of view and strength requirements using 2% cement, 88% expansive soil and 10% Bagasse Ash stabilized sub grade is more economical compared to 100% selected material replaced 1m^3 sub grades and sub base materials.

On the other hand, 4% cement 86%, expansive soil and 10% Bagasse Ash stabilized 1m^3 subgrade work taken 344.04 Birr. Even though, this resulted to high CBR value it can be used for high traffic sub grade and sub base roads but, it was obtained costy. Some high amount of direct cost for 4% cement, 86%expansive soil and 10% Bagasse Ash stabilized 1m^3 sub grade caused by the 2% to 4% increases of cement contents. Therefore, to be used as sub base materials detail cost benefit analysis should be done.

In addition to reduction of cost of sub grade materials, stabilization of expansive soils using waste materials to be suitable for sub grade materials reduces the cutting of natural slopes as selected materials which causes environmental impacts.

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusions

The study has been conducted to investigate the fundamental properties of treated and untreated soil. It can be concluded that there was an improvement of the soil properties by addition of bagasse ash and cement. The following conclusions have been drawn in the study:-

i. The maximum dry density of soil decrease with the addition of bagasse ash and cement but the optimum moisture content of soil increase with the addition of bagasse ash and cement. This is because of the pozzalonic action of bagasse, cement and soil, which needs more water.

ii. At 10% bagasse ash the UCS value increase to 65% compared to the natural soil.

iii. The CBR value of untreated soil increase from 2% to 7.5% at 10% bagasse ash content.

iv. The optimum amount of treated soil to be 88%ES+ 10%BA+ 2%CE.

v. The use of bagasse ash stabilized material has advantage because it resolve disposal problem in sugar industry and free of cost.

vi. The treated subgrade soil with bagasse ash and cement has less cost compared to conventional method.

vii. Finally, this study helps for utilization of treated expansive soils as construction and foundation materials for light weight buildings and roads in the study.

5.2 Recommendation

From the study, the following recommendations are drawn:-

i. The study results of soils show that the improvements in engineering properties of the soils after applying the additive are erratic. The optimum mix of a given soil should be established from test results of various trials.

ii. The effect of weather condition on the treated soils was not taken in to account while undertaking the investigation. It may be helpful to study the effect of different weather conditions on the response of the soils.

iii. The environmental impacts of the additive application were not taken into account in the study, Further studies of this aspect may be important.

iv. The results of the investigations were obtained based on immediate compaction after treating the soil. The effect of delay between mixing and compaction of the soils should be investigated.

v. Finally, the results and findings of the present study considered as indicative only as these findings were based on limited parameters and on small number of samples. More elaborate sampling and testing would be mandatory before implementation of results from the present study.

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APPENDIX

Sample detail unit rate analysis

I Conventional metohd [Cut and replacment using selected materials]

ANALYSIS SHEET FOR DIRECT UNIT COSTS											
WORK GROUP	Sub grade						LABOUR HOURLY OUTPU		28	m^3/hr.	
WORK ITEM:	Excavation to a debth 1.0m						excavator				
TOTAL QANTITY OF WORK IT		1	m^3				RESULT:	32.6	Birr/m^3		
Material Cost (1:01)					Labour Cost (1:02)			Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	Hourly Co	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	operator	1	50	excavator	1	800	800
				0	Forman	1	35				0
				0	ass.operater	1	30				0
Total (1:-01)				0	Total (1:02)		115	Total (1:03)			800
A= Materials Unit C	0	Birr/m^3		B= Manpower Unit Cost 4.1Birr/m3			C= Equipment Unit Cost			28.5	Birr/m^3
				Direct Cost of Work Item = A+B+C =				32.6	Birr/m^3		
Remark ____											
*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.											

ANALYSIS SHEET FOR DIRECT UNIT COSTS											
WORK GROUP	Sub grade						LABOUR HOURLY OUTPU	5.5	m^3/hr.		
WORK ITEM:	Back fill selected mats. 100% from out side [1-5km]						EQUIPEMENT:				
TOTAL QANTITY OF WORK IT		1	m^3				RESULT:	341.0881818	Birr/m^3		
Material Cost (1:01)					Labour Cost (1:02)		Equipment Cost (1:03)				
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	Hourly Co	Type of Equipment	No.	Hourly Rental	Hourly Cost
selected material	m3	1.25	40	55	DL	2	20	Grider	1	1000	1000
				0	operator	1	50	Compacter	1	400	400
				0	Forman	1	35	Dump truck	1	330	330
Total (1:-01)			40	55	Total (1:02)		105	Total (1:03)			1730
A= Materials Unit C	7.272727273	Birr/m^3		B= Manpower Unit Cost		19.2'	C= Equipment Unit Cost			314.5454545	Birr/m^3
				Direct Cost of Work Item = A+B+C =				341.0881818	Birr/m^3		
Remark ______											
*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.											

ANALYSIS SHEET FOR DIRECT UNIT COSTS											
WORK GROUP	Sub grade						LABOUR HOURLY OUTPU	35	m^3/hr.		
WORK ITEM:	Cartaway surplus excavated material to appropriate tip [1-5 km]						EQUIPEMENT: Tool				
TOTAL QANTITY OF WORK IT		1	m^3				RESULT:	27.38571429	Birr/m^3		
Material Cost (1:01)					Labour Cost (1:02)			Equipment Cost (1:03)			
Type of Material	Unit	Qty *	Rate	Cost per Unit	Labour by Grade	No.	Hourly Co	Type of Equipment	No.	Hourly Rental	Hourly Cost
				0	Operater	1	50	Loader	1	450	450
				0	Driver	1	40	Dump truck	1	330	330
				0	ass.oppr	1	30				0
				0	ass.Driv	1	25				0
				0	Forman	1	35				0
Total (1:-01)				0	Total (1:02)		180	Total (1:03)			780
A= Materials Unit C	0	Birr/m^3		B= Manpower Unit Cost		5.1Birr/m3	C= Equipment Unit Cost			22.28571429	Birr/m^3
				Direct Cost of Work Item = A+B+C =				27.38571429	Birr/m^3		
Remark ______											
*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.											

ANALYSIS SHEET FOR DIRECT UNIT COSTS

WORK GROUP Sub grade LABOUR HOURLY OUTPU 5.5 m^3/hr.

WORK ITEM: Mixing and compaction [2% C + 88% ES + 10% BA] EQUIPEMENT: Tool

TOTAL QANTITY OF WORK IT m^3 RESULT: 293.6454545 Birr/m^3

Material Cost (1:01)					Labour Cost (1:02)			Equipment Cost (1:03)			
Type of Material	Unit	Qty *	**Rate	Cost per Unit	Labour by Grade	No.	Hourly Co	Type of Equipment	No.	Hourly Rental	Hourly Cost
Bagasse Ash	m^3	0.1	2	2	Operater	1	50	Grider	1	1000	1000
Cement	qt	0.28	98	98	Forman	1	35	Compacter	1	400	400
Expansive soil	m^3	0.9	0	0	ass.oppr	1	30				0
Total (1:-01)				100	Total (1:02)		115	Total (1:03)			1400

A= Materials Unit C 18.2 Birr/m^3 B= Manpower Unit Cost 20.90 Birr/n C= Equipment Unit Cost 254.5454545 Birr/m^3

Direct Cost of Work Item = A+B+C = 293.6454545 Birr/m^3

Remark ____

*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.

**: INCLUSIVE OF TRANSPORTATION COST.

II. For stabilized expansive soils [2% CE + 10%BA+88%ES] and [4% CE + 10%BA+86%ES]

ANALYSIS SHEET FOR DIRECT UNIT COSTS											
WORK GROUP	Sub grade and sub base						LABOUR HOURLY OUTPU		5.5	m^3/hr.	
WORK ITEM:	Mixing and compaction		[4% C + 86% ES + 10% BA]				EQUIPEMENT: Tool				
TOTAL QANTITY OF WORK IT		1	m^3				RESULT:	311.4454545	Birr/m^3		
Material Cost (1:01)					Labour Cost (1:02)			Equipment Cost (1:03)			
Type of Material	Unit	Qty *	**Rate	Cost per Unit	Labour by Grade	No.	Hourly Co	Type of Equipment	No.	Hourly Rental	Hourly Cost
Bagasse Ash	m^3	0.1	2	2	Operater	1	50	Grider	1	1000	1000
Cement	qt	0.56	196	196	Forman	1	35	Compacter	1	400	400
Expansive soil	m^3	0.8	0	0	ass.oppr	1	30				0
Total (1:-01)				198	Total (1:02)		115	Total (1:03)			1400
A= Materials Unit C	36	Birr/m^3		B= Manpower Unit Cost	20.9Birr/m3		C= Equipment Unit Cost			254.5454545	Birr/m^3
				Direct Cost of Work Item = A+B+C =				311.4454545	Birr/m^3		
Remark ____											
*: INCLUSIVE OF WASTAGE, TRANSPORTING, HANDLING, ETC.											
**: INCLUSIVE OF TRANSPORTATION COST.											