EFFECT OF BOTTOM ASH ON STABILIZING EXPANSIVE SOIL

(THE CASE OF GELLAN TOWN)

Fikreyesus Yami

A Thesis Submitted to

The Department of Civil Engineering

School of Civil Engineering and Architecture

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

Office of Graduate studies

Adama Science and Technology University

February, 2023

Adama, Ethiopia

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Advisor Prof. Sudhir Kumar Tewatia

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DECLARATION

I hereby declare that this Master Thesis entitled "**Stabilization of Expansive Soil Using Bottom Ash (The Case of Gellan Town)**" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "**Stabilization of Expansive Soil Using Bottom Ash (The Case of Gellan Town)**" prepared under my guidance by **Fikreyesus Yami** submitted in partial fulfilment of the requirements for the degree of Masters of Science in Civil Engineering (Geotechnical Engineering). Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

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APPROVAL PAGE

I, the advisors of the thesis entitled "**Stabilization of Expansive Soil Using Bottom Ash (The Case of Gellan Town)**" and developed by **Fikreyesus Yami**; hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of Examiners of the thesis by **Fikreyesus Yami** have read and evaluated the thesis entitled "**Stabilization of Expansive Soil Using Bottom Ash (The Case Of Gellan Town)**" and examined the candidate during open defence. This is, therefore, to certify that the thesis is accepted for partial fulfilment of the requirement of the degree of Master of Science in Civil Engineering (Geotechnical Engineering).

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TABLE OF CONTENT

LIST OF TABLES

LIST OF FIGURES

ACRONYM

AASHTO	American Association of Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
CH	Clay of High Plasticity
CL	Clay of Low Plasticity
EEPCo	Ethiopian Electric Power Corporation
ERA	Ethiopian Roads Authority
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
UCS	Unconfined compressive strength
USCS	Unified soil classification systems (USCS)

UNITS

mm	millimeter
cm	centimeter
m	meter
km	Kilometer
g	gram
kg	Kilogram
N	Newton
kN	Kilo Newton
kPa	Kilo Pascal
^{0}	Degree

ABSTRACT

Expansive (black cotton) soil is a worldwide problem for civil engineers it causes damages to foundations and pavements due to the uplift pressure and cracking because of the presence of excess montmorillonite in the soil. Many researchers are tried to improve the geotechnical properties of expansive soil to reduce problems associated with expansive soil using stabilization of soil with different types of additives. The main objective of the present Research work aimed to investigate whether or not bottom ash is used as a stabilizing material for expansive soil. The bottom ash was previously collected from Reppi Koshe waste to electric energy production plant and the expansive soil was collected from Gellan town near Ethiopian Electric Power (EEP) substation. The proposed samples are prepared with different proportions from observed published papers as 5%, 10%, 15%, 20%, 25% and 30% by weight which were used for laboratory tests such as grain size distribution, specific gravity which both samples passed through sieve No 10 (2mm), Atterberg Limit test, samples passed through sieve No 40 (0.425mm) where distilled water were used for these testes. Soaked California bearing ratio (CBR %), California bearing ratio swell (CBR swell %), compaction and Unconfined Compressive Strength (UCS) all samples passed through sieve No 4 (4.75mm) and the water utilized was from the optimum moisture content test result for CBR (%), CBR swell (%) and UCS laboratory tests. The black cotton soil was initially characterized on index property tests and found to be A-7-5, OH according to AASHTO and USCS soil classification system respectively which implies exceedingly expansive and have a poor load bearing capacity. Plasticity index, Optimum moisture content (OMC), California bearing ratio swell (CBR swell) decreased with increment of bottom ash. California bearing ratio (CBR), Maximum Dry Density (MDD) and unconfined compressive strength (UCS) increased when percent of bottom ash increased. There was a significant improvement in the properties of black cotton soil for CBR and CBR swell but it doesn't meet the minimum value of ERA specification for sub-grade and there was a good achievement in UCS which can be used as backfill material for retaining wall and other small structures by comparative study of the design load. However further investigation was needed to use potentially available by-product.

KEY WORDS: Stabilization, Bottom Ash, Expansive soil (black cotton), Soaked CBR, CBR swell

Chapter 1
Introduction

1.1 Background of the study

Plastic clay, known as expansive soil or active soil, exhibits a change in volume when exposed to humidity fluctuations (Dasgupta, 2006). Black cotton soils, semitropical soils found in areas where the annual rainfall is 500 to 750 mm. They range from black to dark grey. They tend to be tough with very large cracks (large-volume-change soils) when dry and very soft and spongy when wet. These soils are located in big areas of Australia, India and south east Asia (Bowels, 1996).The soil is over-consolidated with the significant amount of expansive clay minerals (montmorillonite), mainly darkish grey to reddish grey in colour. Located on the tropical coast of the semi-arid region of East Africa, the case study experiences two major seasons, the rainy season and the dry season. During the rainy season, expansive clay minerals attract water molecules, resulting in a significant change in volume (DaSGupta, 2006).

In the dry and rainy seasons, we can easily find these soils in the fields, Shrinkage cracks can be seen on the floor during the dry season. The maximum width of these cracks is 20 mm or even more and penetrates deep into the ground. During the rainy season, these soils become very sticky and very difficult to cross (Girma, 2017b).

The soil must be able to withstand the loads of the engineering structures placed on it without shear failure and the resulting subsidence must be tolerated for this structure. Failure of ground thrust can lead to excessive deformation or collapse of the building. Excessive subsidence can cause structural damage to the frame of the building (Bowels, 1996).

The most obvious way expansive soils can damage the foundation is due to uplift as they expand as the water content increases. Swollen soil often lifts and tears lightly loaded continuous strip foundations, loading floor slabs. Due to the different building loads on different parts of the foundation of the structure, the resulting lift will vary in different areas. The outer corners of the evenly loaded rectangular slab foundation apply only about a quarter of the normal pressure to the swollen soil applied to the central part of the slab. As a result, the corners tend to be higher relative to the central part. This phenomenon can be caused by different water levels in the soil at the edge of the slab. These different movements of the foundation can also affect the framework of the structure. (Rogers et al., 1993).The physical properties of soils can often economically be improved by the use of admixtures. Some of the more widely used admixtures include lime, Portland cement and asphalt. The process of soil stabilization first involves mixing with the soil a suitable additive which changes its property and then compacting the admixture suitably. This method is applicable only for soils in shallow foundations or the base courses of roads, airfield pavements, etc. (Girma, 2002).

With the exception of industrial process wastes and agricultural solid wastes, all wastes produced in a community are commonly referred to as municipal solid wastes (MSW) and the sources of these wastes are mainly institutional, commercial, residential, municipal services, construction and demolition excluding treatment works (Seyoum, 2007). In this study the stabilization of expansive/Black cotton soil using municipal waste bottom ash was analysed using field exploration and laboratory tests for the utilization of subgrade and backfill material.

1.2 Statement of the problem

A relatively large area of Ethiopia is covered with vast clay soils. These soils create on going difficulties in the construction of buildings, civil structures and retaining walls. Removing and transporting the unwanted excavated poor engineering behaviour soils and replacing with selected Backfill material required additional cost and time so that to minimise these conditions, it is suitable to re-use the expansive soil using available stabilizer. Increasing population, urbanization and rising standards of living, there has been increased generation of solid wastes from industrial, domestic and agricultural sources (Sanewu et al., 2013). Recycling of these wastes in soil stabilization might potentially reduce construction costs, enhance environmental sustainability and prepare a site for disposal and dumping in limited land resource. In this study, bottom ash from Reppi (Koshe) waste to electric energy power plant bi-product was utilized.

1.3 Research question

- Does the bottom ash improve geotechnical property of expansive soil?
- What is geotechnical property of expansive soil found at Gellan town near EEP substation?
- What percentage of addition bottom ash in expansive soil gives the maximum strength to mixture as per proposed?
- Can black cotton soil be used for subgrade and back fill material after stabilization or treatment with incinerated waste bottom ash?

1.4 Objective of the study

1.4.1 General Objective study

- The general objective of this research is to study effect of bottom ash on stabilizing expansive soil.

1.4.2 Specific Objective

- To determine property of expansive soil and bottom ash separately before stabilizing.
- To evaluate the changes in engineering properties of expansive soil after stabilizing using bottom ash.
- To determine gain the maximum strength of expansive soil by adding bottom ash up to 30.
- Analyse for utilization of subgrade and backfill material after stabilization.

1.5 Significance of the study

If the bottom ash is used as stabilizer for expansive soil, it will reduce cost and limited availability of disposal site which impact the growing population in urban areas which is in need lands for different types of structures and constructions.

1.6 Expected outcome of the study

In this research using bottom ash as stabilizer of expansive soil where needed in different parts of the country solves problems associated with foundation, retaining wall and highway roads faller.

1.7 Limitation of the study

The proposed study was up to thirty per cent increment of bottom ash by weight but, if more will be done the result was expected to change for

Chapter 2
Literature Review

2.1 Expansive soil

2.1.1 Origin of expansive soil

The origin of expansive soil is associated with a combination of conditions and processes that lead to the formation of clay minerals with a particular chemical composition that expand when in contact with water. Changes in conditions and processes can also be forming other clay minerals, most of which are not expansive. The conditions or processes that determine clay minerals include the composition of the raw material and the degree of physical and chemical weathering to which the material is exposed (Chen, 1979).

The first group includes Basic igneous rocks such as: the Basalt's of the Deccan Plateau in India, the dolerite sills and dykes in the central region of South Africa and the gabbro's and nudities west of Pretoria North, Transvaal. In these soils, the feldspar and pyroxene minerals of the parent rocks have decomposed to form montmorillonite and other secondary minerals. The second group includes the sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils. In North America, bedrock shale found in the Pierre Formation and the more recent Laramie and Denver Formations are an example of this type of rock. In Israel, there are the marls and limestone and in South Africa, the shale of the Ecca Series. The clay minerals are formed from many raw materials in a complex process from feldspars, micas, and limestone (Chen, 1979).

2.1.2 Mineralogy of expansive soil

Clay minerals are separated from gravel, sand and silt because they carry electrical charges on their surface. Clay soil also creates significant engineering problems such as high volume increase when comes into the contact of water and volume decrease when dried. These volume changes create cracks in the soil which are harmful to heavy construction and roads. Almost all the clay minerals are made up of two fundamental crystal sheets joined through different bonds.

One is tetrahedral Sheet which is also called silica sheet and another one is octahedral also called Alumina Sheet or Gibbsite Sheet (Arora, 2004)

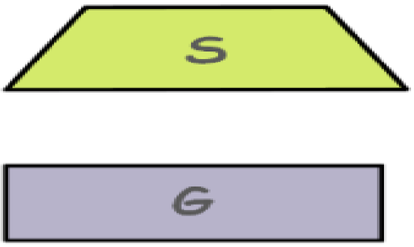

Figure 2.1 Tetrahedral (Arora, 2004)

i. Tetrahedral Sheet is made up of several Silica tetrahedral units combined together. One Silica Tetrahedral Unit consists of a silicon ion (Si_{4+}) surrounded by four oxygen ions (O_{2-}) forming a shape of tetrahedron. Silicon sits at the centre and oxygen ions sit at the tips of tetrahedron. Many such tetrahedral units combine to form the Silica Sheet.

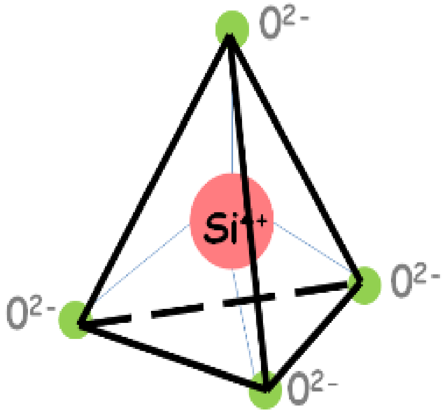

Figure.2.2 One silica tetrahedral unit $(SiO_4)^{4-}$ (Arora, 2004)

Each oxygen ion at the base of the unit is common to two adjacent units. Hence two negative charge of each oxygen ion is shared by two units leaving one negative charge for one unit.

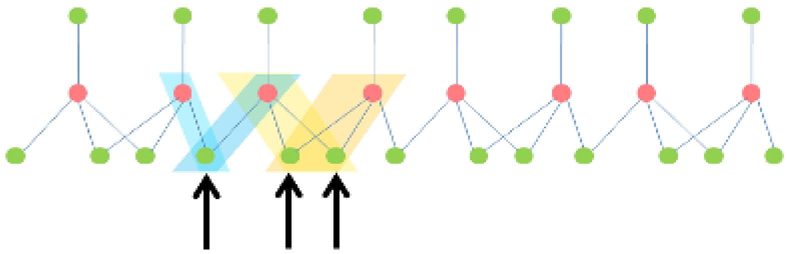

Figure 2.3 Flow of oxygen ions (Arora, 2004)

Oxygen ion sitting at the top does not share its charge with anyone. Central ion has plus four charge and base oxygen have -1 charge.

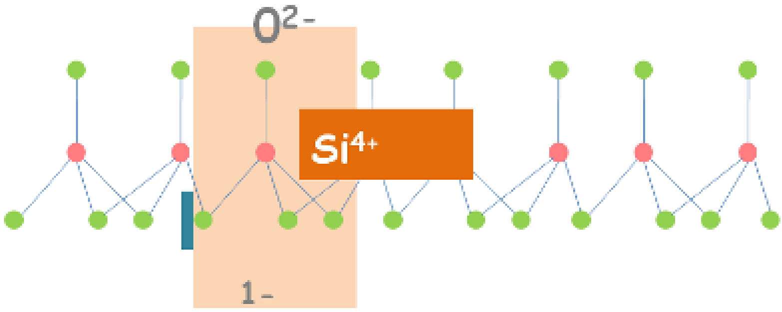

Figure 2.4 Positions of oxygen and silica ions (Arora, 2004)

Hence in the sheet net charge per tetrahedral unit is -1.

(-2) + (+4) + {3x (-1)} = -1

This complex structure of silica sheet is symbolically represented as this

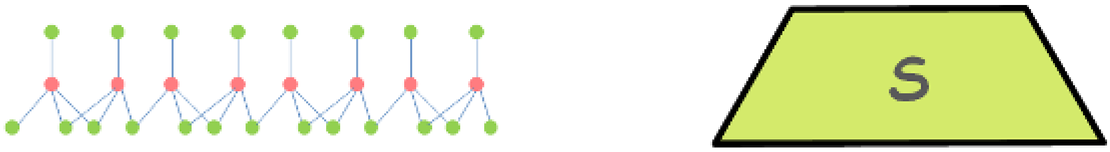

Figure 2.5 Silica sheet $(Si_2O_5)^{2-}$ (Arora, 2004)

Alumina Sheet or Gibbsite Sheet: A gibbsite sheet is made up of several Alumina octahedral units combined together. One octahedral unit consists of six hydroxyls forming a configuration of an octahedron and having one aluminium ion at the centre.

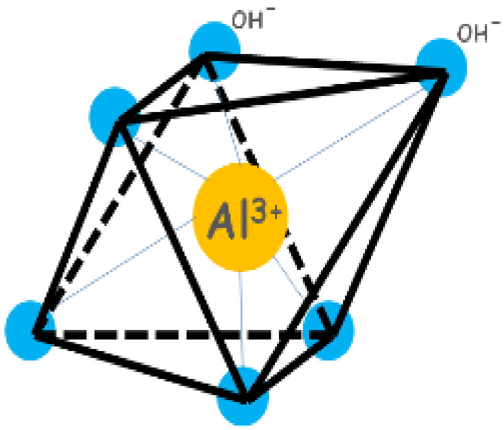

Figure 2.6 Octahedral units (Arora, 2004)

Several such octahedral units combine to form a Gibbsite Sheet. It is represented by this symbol.

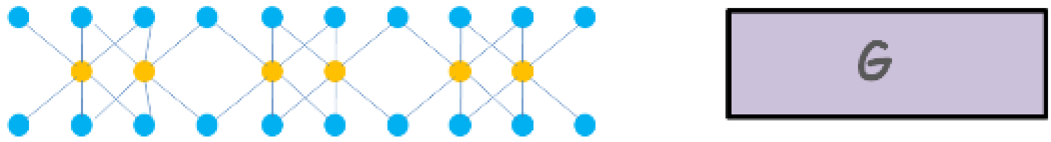

Figure 2.7 Alumina Sheet or Gibbsite Sheet (Arora, 2004)

If the central Aluminium ion in the octahedral is replaced by Magnesium ion, it results in formation of different crystal sheet called Brucite Sheet and his substitution of a metallic ion by other metallic ion of lower valence but of same physical size is called isomorphs substitution. This kind of substitution leads to formation of different soil mineral which has different physical properties.

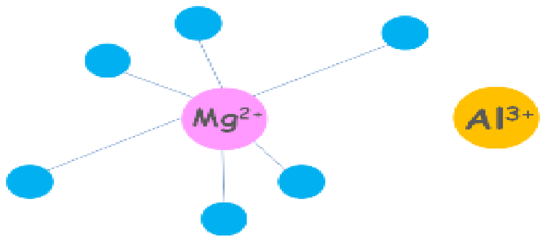

Figure 2.8 Minerals stack together (Arora, 2004)

Different clay minerals simply consist of these two Basic sheets stacked together in certain unique fashion.

Clay can refer to both the size and class of minerals. As a size term, it refers to all components smaller than a particular size (usually 0.002 mm in the technical classification). As a mineral term, it describes certain clay minerals that differ by

(1) Small particle size,

(2) A net negative electrical charge,

(3) Plasticity when mixed with water, and

(4) High weathering resistance.

ii. Clay minerals are mainly hydrous aluminium silicate. Not all clay particles are smaller than 0.002 mm, and not all non-clay particles are coarser than 0.002 mm. However, the amount of clay minerals in soil is often estimated by the amount of substances finer than

0.002 m. Therefore, to avoid confusion, it makes sense to use the terms clay size and clay mineral content. Another important difference between clay minerals and non-clay minerals is that non-clay is composed primarily of bulky particles. Most clay mineral particles, on the other hand, are bevel-shaped and, in some cases, needle-shaped or tubular (Soltani et al, 2019) .

Clays can be divided into three general groups on the bases of their crystal arrangement. They are:

1. Kaolinite

The kaolinite of minerals is the most stable in clay minerals. The kaolinite mineral is formed by the stacking of the crystal layers one above the other with the base of the silica sheet bonding to hydroxyls of the gibbsite sheet by hydrogen bonds. Since hydrogen bonds are comparatively strong, the kaolinite crystal consists of many sheet stacking that are difficult to dislodge. The mineral is, therefore, stable and water cannot enter between the sheets to expand the unit cells (MURTHY 2002). In its basic structural unit these are stacked one over the other and tips of silica sheets are embedded in the gibbsite sheet. The thickness of such structural unit is about 7 Angstrom where, 1 Angstrom is equal to 10^{-10} m or 1 m is equal to 10^{10} Angstrom.

As this mineral is formed by stacking of one layer of each sheet it is sometimes called a 1:1 clay mineral.

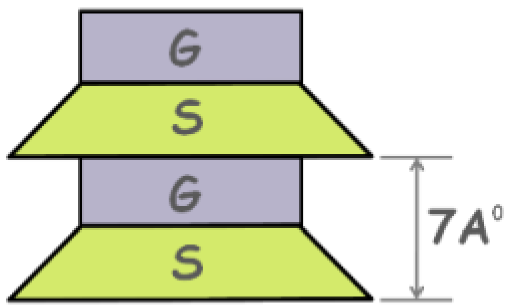

Figure 2.9 1:1 Mineral Kaolinite / $Al_2O_32SiO_22H_2O$ (Arora, 2004)

Kaolinite mineral is formed by staking one over the other such several basic units. This unit extends indefinitely in other dimensions. These structural units join together by hydrogen bond between hydroxyls of alumina sheet and oxygen of silica sheet.

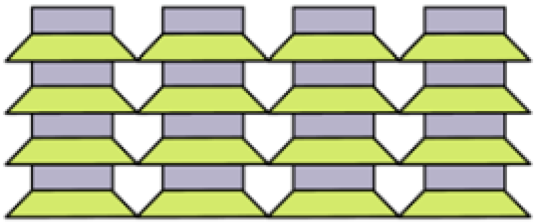

Fig.ure2.10 Kaolinite / $Al_2O_32SiO_22H_2O$ (Arora, 2004)

As the hydrogen bond is sufficiently strong, the kaolinite mineral is stable and water cannot easily enter between the structural units and cause expansion. Kaolinite is the least active of all clay minerals.

2. Montmorillonite

The structural arrangement of the montmorillonite mineral consists of two tetrahedral silicon dioxide layer units with a central aluminium oxide octahedral layer. The silica and gibbsite layers are combined so that the tip of the tetrahedron of each silica layer and one of the hydroxyl layers of the octahedral layer form a common layer. The atom common to both the silica and gibbsite layers is oxygen, not hydroxyl and when these combined units are stacked on top of each other, the oxygen layer of each unit is adjacent to the oxygen of the adjacent units, resulting in weak bonds and excellent cleavage between them. Water can penetrate between the panels, causing the panels to expand significantly and the structure to be divided into units. Soils containing a significant proportion of montmorillonite minerals exhibit high swelling and contraction behaviour (.MURTHY, 2002). The basic structural unit of montmorillonite consists of a gibbsite sheet sandwiched between two silica sheets to form a single layer. The thickness of this layer is about 10 A and clearly it is a 2:1 mineral

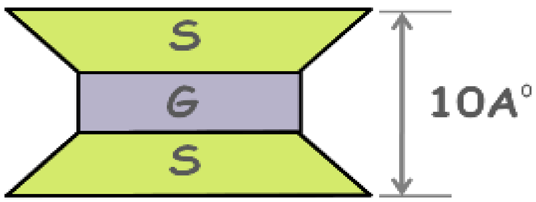

Figure2.11 Gibbsite sheet sandwiched between two silica sheets to form a single layer (Arora, 2004)

Montmorillonite mineral is formed by many such structural units joined together by Van der Waals force which is a very weak force when compared to hydrogen bond.

Clay soils that attract more water have more plasticity, more swelling or shrinkage. Water molecules are dipolar and negatively charged surface of silica sheet attracts these molecules in the space between two structural units causing the layers of the mineral to be further separated which results in the expansion of the mineral. That is why soils containing clay mineral montmorillonite exhibit high volume change. They swell as the water enters into the structure and shrink as the water is removed. Its excessive swelling capacity may seriously endanger the stability of overlying structures

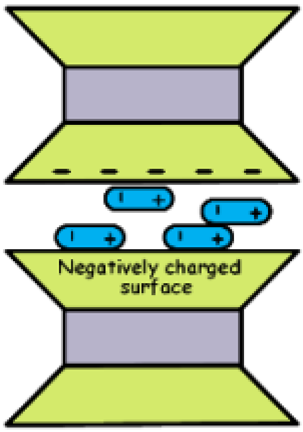

Figure 2.12 Montmorillonite / $Al_2H_2O12Si_4$ (Arora, 2004)

3. Illite

The illite mineral groups have the same structural arrangement as that of montmorillonite group. The presence of potassium as the bonding materials between units makes the illite minerals swell less.

It is also a 2:1 mineral and layer thickness is also about 10 A. In this mineral there is isomorphs substitution of silicon ions in silica sheet by aluminium ion. Also the potassium ions occupy the space between different structural units and do not allow water to take its place. Potassium ion bonds the two layers together more firmly which was not the case in the montmorillonite. Therefore, illite does not swell as much in the presence of water as montmorillonite, but still it does much more than kaolinite.

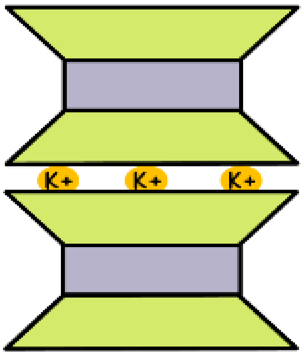

Figure 2.13 Illite / $Al_9FFeHK_3MgO_{41}Si_{14}^{+8}$ (Arora, 2004)

2.1.3 Distribution of expansive soil

The expansive soil distribution is widespread throughout the world (Fig. 2.13). The countries where these soils have been reported are: Argentina, Iran, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Israel, Mexico and Morocco. Rhodesia, South Africa, Spain, Turkey, U.S.A and Venezuela (Chen, 1979).

Potentially expanding soils are confined to semi-arid regions of tropical and temperate climates. There are many vast soils where the annual evapotranspiration exceeds the rainfall. This follows the theory that in semi-arid areas the lack of exudation favours the formation of montmorillonite. Potentially expanding soils can be found almost anywhere in the world. In developing countries, many of the widespread soil problems may not be recognized. As the amount of construction increases, it is expected that larger soil areas will be discovered each year (Chen, 1979).

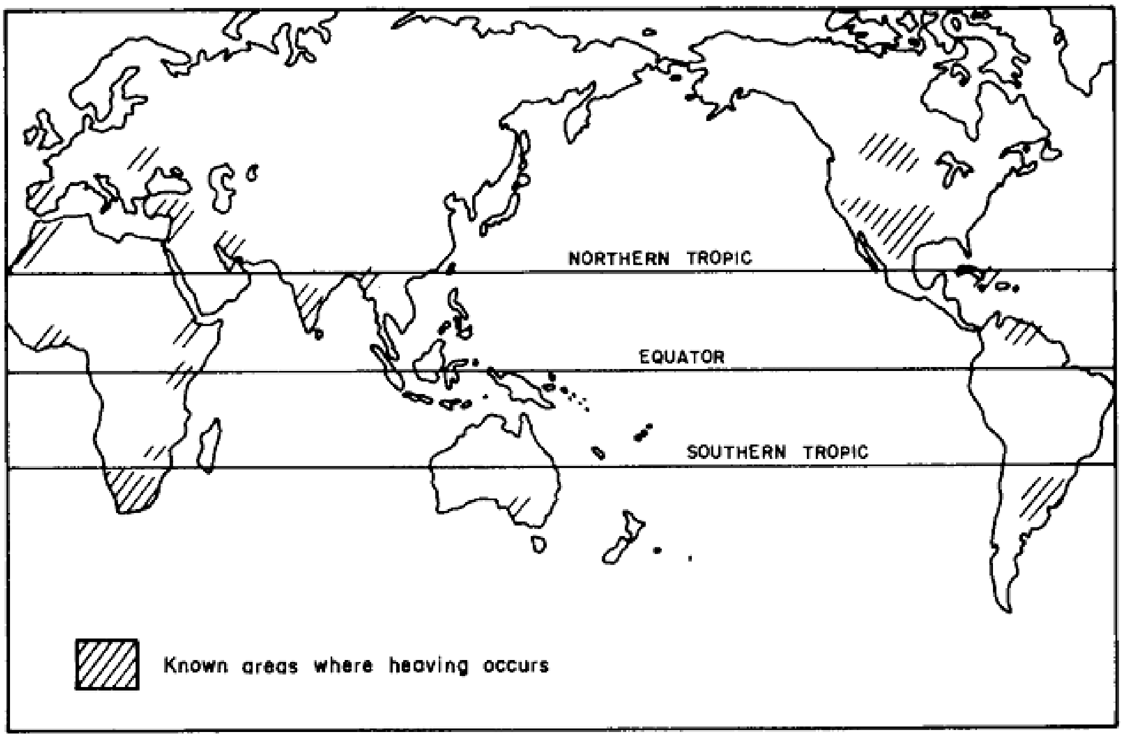

Figure 2.14 Distribution of expansive soil in the world (Chen, 1979)

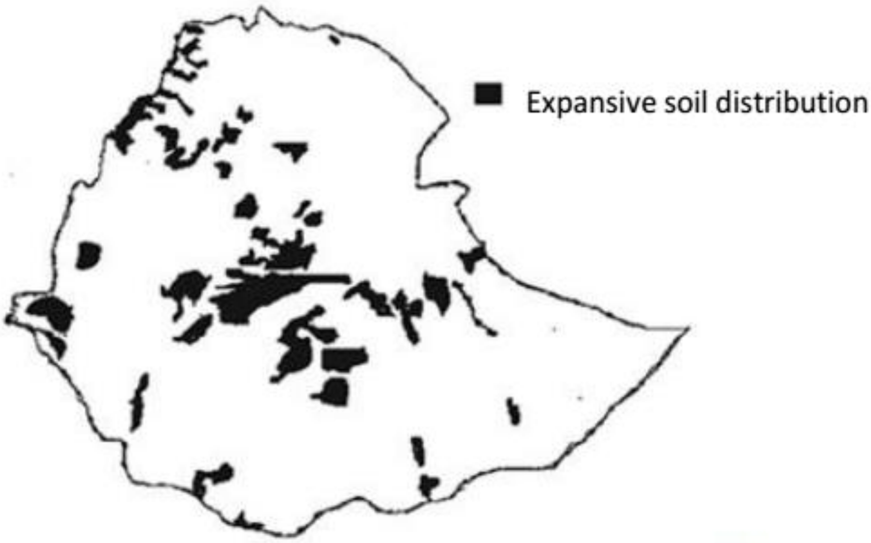

Figure 2.15 the distribution of expansive soil in Ethiopia (Girma, 2017)

Ethiopia's expansive soil coverage is estimated at 24.7 million acres from 274.8 million acres which is 8.99%. They are widespread in central Ethiopia and follow major truck routes such as Addis Ambo, Addis Wolliso, Addis Debrebirhan, Addis Gohastion and Addis Modjo. Areas such as Mekelle and Gambela are also covered with expansive soils (Girma, 2017).

2.1.4 Identification of expansive soil

2.1.4.1 Field Identification

Field observations and surveys can provide valuable data on the extent and nature of expansive soils and the problems associated with them (Lee D Jons, 1998)

Some identifications are:

1) Spacing and width of wide or deep shrinkage cracks
2) High dry strength and low wet strength – high plasticity soil
3) Stickiness and low traffic ability when wet
4) Surfaces have glazed or shiny appearance
5) Evidence of low permeability evident by surface drainage and infiltration features

2.1.4.2 Laboratory Identification

2.1.4.2.1 Mineralogical identification

The mineralogical composition of expansive soils has an important bearing on the swelling potential; the negative electric charges on the surface of the clay minerals, the strength of the interlayer bonding and the cation exchange which is a process by which cations are reversibly adsorbed on charged surfaces of sediments from solution and all contributes to the swelling potential of the clay.

Clay minerals can be identified using a variety mineralogical techniques, the more common of which are listed below (Jones et al., 2012).

1) X-Ray Diffraction Method: In colloidal clay it consists essentially of comparing the ratios of the intensities of diffractions from the different minerals with the intensities from the standard substance.
2) Differential Thermal Analysis: Differential thermal analysis when used in conjunction with X-ray diffraction and chemical analysis enables the identification of otherwise difficult materials. It is well established as a technique for the control of materials which undergo characteristic changes on heating. The use of differential thermal analysis technique in identifying expansive soil is not always accurate.
3) Dye Adsorption: Dyestuffs and other reagents which exhibit characteristic colours when adsorbed by clay have been used to identify clay. When a clay sample has been pre-treated with acid, the colour assumed by the adsorbed dye depends on the Base

Exchange capacity of the various clay minerals present. The presence of montmorillonite can be detected if its amount is greater than about 5 to 10%. The relatively simple testing procedure and speed of dye staining tests compared with X-ray diffraction and differential thermal analysis justify wider application of the colour method.

4) Chemical Analysis: Chemical analysis can be a valuable supplement to other methods such as X-ray analysis in identifying clays. In the montmorillonite group of clay minerals, chemical analysis can be used to determine the nature of isomorphism and to show the origin and location of the charge on the lattice. The isomorphs character of the montmorillonite group can probably be shown in no other way. The isomorphism involves three basic variations in the substitution: the substitution for Al for Si in tetrahedral positions in the lattice; the substitution of Fe for Al in the octahedral coordination; and the substitution of Mg for Al in the octahedral positions.
5) Electron Microscope Resolution: Microscopic examination of clay minerals offers a direct observation of the material. Two clays may give the same X-ray pattern and the same differential thermal curve but will show up distinct morphological characteristics under electron microscope resolution. The main purpose of the microscopic examination is to determine mineralogy composition, texture, and internal structure.

The various methods listed above should generally be used in combination. Using combinations of the methods, the different types of clay minerals present in a given soil can be evaluated quantitatively. However, the test results require expert interpretation and the specialized apparatus required are costly and not economically available in most soil testing laboratories (Miller, 1983).

2.1.4.2.2 Indirect Methods

In this method simple soil property tests can be used for the evaluation of identifying expansive clay soils. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include Atterberg limits test, and grain size distribution, free swell test, free swell index test, free swell ratio test etc. (Nelson & Miller, 1992).

2.1.4.2.3 Direct methods

Direct measurement, offers the most useful data for a practicing engineer. The tests are simple to perform and do not require any costly and exotic laboratory equipment. A word of caution should be introduced here. Testing should be performed on a number of samples through actual measurement of swelling pressure and volume change of soil. (Chen, 1979).

2.1.5 Classification of Expansive soil

2.1.5 .1 General Method

Soils are classified in the general schemes; Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials Method (AASHTO) according to index properties. Soils rated CL or CH by USCS and A6 or A7 by AASHTO may be considered potentially expansive (Nelson and Miller 1992).

i. AASHTO Soil Classification

This system was originally proposed in 1928 by the U.S. Bureau of Public Roads for use by highway engineers. A Committee of highway engineers for the Highway Research Board met in 1945 and made an extensive revision of the public road System. This system is known as the AASHTO System of Soil Phase Relationships, Index Properties and Soil Classification (Murthy, 2002).

ii) Unified Soil Classification System (USCS)

Unified soil classification system is adopted by ASTM D-2487-98 for classification and identification of soils for general engineering purpose and is broadly classified into three divisions.

a) Coarse grained Soils: In these soils, 50% or more of the total material by weight is larger than 75 micron (No. 200 sieve) IS sieve size and are divided into two sub-divisions:

- Gravels: In these soils more than 50% of the coarse fraction larger than 4.75 mm IS sieves size. This sub-division includes gravels, gravelly soil and is designated by symbol G.
- Sands: In these soils, more than 50% of the coarse fraction is smaller than 4.75mm IS sieve size. This sub-division includes sands and sandy soils and is designated by symbol S.

Each of the above sub-divisions are further divided into four groups depending upon grading and inclusion of other materials.

W: Well Graded C: Clay binder

P: Poorly graded M: Containing fine materials not covered in other groups

b) Fine grained soils: In these soils, 50% or more of the total material by weight is smaller than 75 micron (No. 200 sieve) IS sieve size and are divided into three sub-divisions:

- Inorganic silts and very fine sands which is designated by symbol M.
- Fine grained soils are further Inorganic clays which is designated by symbol C.
- Organic silts and clays and organic matter which is designated by symbol O.

Each of the above sub-divisions are further divided in to the following groups on the basis of the following arbitrarily selected values of liquid limit which is a good index of compressibility:

- Silts and clays of low compressibility: Having a liquid limit less than 35 and represented by symbol L.
- Silts and clays of medium compressibility: Having a liquid limit greater than 35 and less than 50 and represented by symbol I.
- Silts and clays of high compressibility: Having a liquid limit greater than 50 and represented by a symbol H.

Combination of these symbols indicates the type of fine grained soil.

c) Highly organic soils and other miscellaneous soil materials: These soils contain large percentage of fibrous organic matter, such as peat, and the particles of decomposed vegetation. In addition, certain soils containing shells, cinders and other non-soil materials in sufficient quantities are also grouped in this division.

Sediment Classification According to United Soil Classification System

GW Well-graded gravels, gravel/sand mixture, little or no fines

GP Poorly graded gravels, gravel/sand mixture, little or no fines

GM Gray calcareous gravel, sand/silt mixture

GC Gray calcareous gravel/sand/clay mixtures

SW	Well-graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, sand/silt mixtures
SC	Clayey sands, sand/clay mixtures
ML	Inorganic silts and very fine sands, silty/clayey fine sand
CL	Inorganic clays of low to medium plasticity, sandy silty or lean clays
OL	Organic silts and organic silty clays of low plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Fat clays
OH	Fat organic clays
PT	Peat, humus, and other organic swamp soils
SP-SM	Sandy, gravelly, silty mixtures
L	Limestone
S	Sandstone

Source: Florida International University, Miami, FL

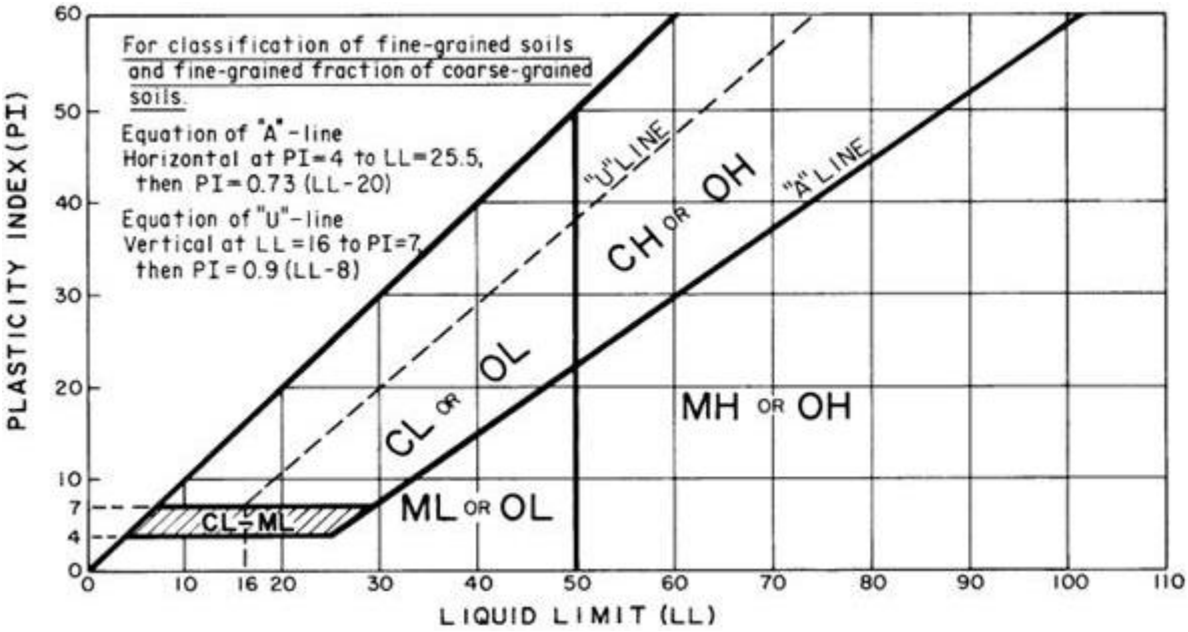

Figure 2.16 Plasticity chart (Trans, n.d.)

2.1.5.2 Specific Method

i. Method of Chen

Table 2.1 method of Chen (1988).

Swelling Potential	Liquid limit
Low	20< LL <35
Medium	35< LL <50
High	50< LL <70
Very High	LL >70

ii. USBR (United State Bureau of Reclamation) method

Holtz and Gibbs developed this method based on direct correlation of observed volume change with colloid content; plastic index and shrinkage limit (Rao, 2002). As shown below

Table 2.2 USBR method

Colloid Content, (%)	Colloid Content, (%)	Shrinkage Limit, (%)	Probable Expansion, (%)	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-28	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

iii. Method of Daksanamurthy and Raman (1973)

The identification of expansive soil is based on the single index method, where only the liquid limit is used. The classification by this method is shown below.

Table 2.3 Relation between the swelling potential of clay and the liquid limit

Swelling Potential	Liquid Limit
Low	$20<LL\leq35$
Medium	$35<LL\leq50$
High	$50<LL\leq70$
Very high	$LL>70$

iv. Method of Skepton

Skempton (1953) defined activity by the following expression

$$A_C = \frac{PI}{\%\,clay} \tag{2.1}$$

Table 2.4 Method of Skepton Relation between clay activity and potential of expansion

Activity	Potential of expansion
Ac < 0.75	Low (inactive)
0.75 < Ac < 1.25	Medium (normal)
Ac > 1.25	High (active)

v. Method of Seed et al.

Seed et.al as cited in Chen, 1988 has developed a chart based on activity and percent clay size. The activity here is defined by the formula (Murthy, 2002):

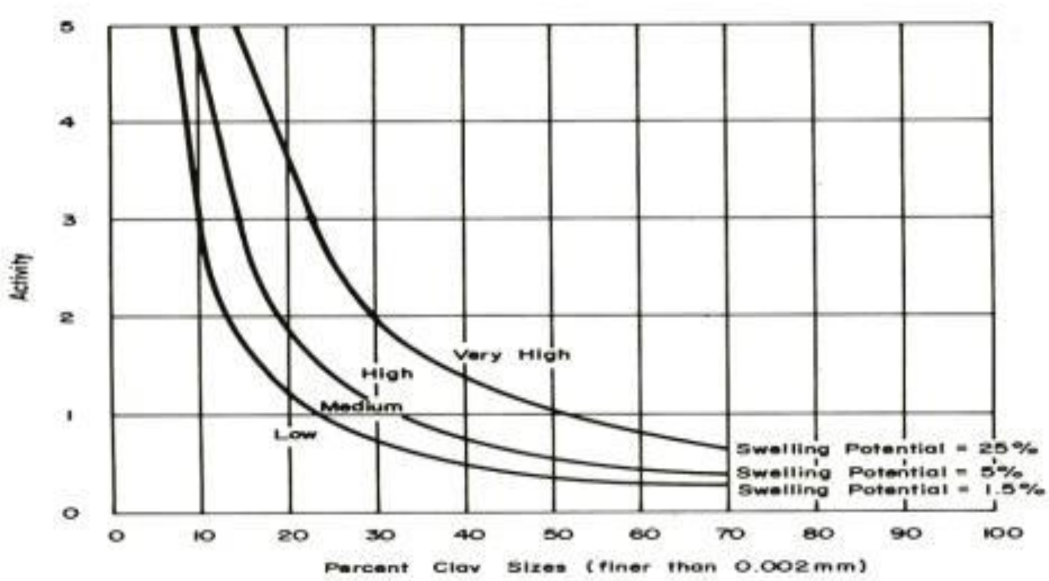

Figure 2.17 Classification chart for swelling potential (Murthy, 2002)

2.1.6 Problems and treatment of expansive soil

There are two main issues related to black cotton soil: water saturation and design issues which are explained below. Water is known to be the worst enemy of pavement, especially in large areas of the ground. Capillary action allows water to penetrate the pavement from three sides through the surface and substructure. During the process of various road survey projects to determine the cause of a road failure, it was discovered that water could easily enter the roadway. It saturates the subsoil and thus reduces its load-bearing capacity, which ultimately leads to deep depressions and subsidence. In the base course, water lubricates the binding material and destabilizes the mechanical interlock. In the top bituminous surfacing, ravelling, stripping and cracking develop due to water stagnation and its seepage into these layers. The same problems are happen for structural buildings. The second problem black cotton soil is associated with design which is lack of using the proper way regarding to solve the problems in black cotton soil (Dahiya, 2013) The expansive soil treatment alternatives may be employed either singly or in combination, to control heave. However, it must be recognized that in any case, site-specific conditions could make one or more methods ineffective. The choice of the technique to be used should be assessed with respect to Economic factors, Relative expected control of volume changes by implementing different treatment alternatives, Site-specific conditions such as potential for volume change, moisture variations, degree of fissuring, permeability, Nature of the project, Necessary strength of the foundation soils, Time frame available for treatment (jones, 1998).

Ethiopia is one of the countries which is exposed with black cotton problems in the construction industry and is covered by a large area with 24.7 million hectares which is 8.99% from the total area. Therefore, it is necessary to improve the Engineering properties of such soils to avoid failure or damage of structures (Girma, 2017b).

2.2 Introduction to soil stabilization

Soil stabilization uses weak soils to improve its engineering properties such as compressibility, shear strength, permeability and durability. (Makusa, 2012). It also helps in increasing the bearing capacity and the resistance to erosion, dust formation, or frost heaving Components of stabilization technology include soil or soil minerals and stabilizers or binders (cementations materials) (Makusa, 2012).

Stabilization is the mechanical mixing of this natural soil and stabilizing material to achieve uniform mixing, or the addition of stabilizing material to undisturbed soil sediments to penetrate the soil voids. These stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents. Different soil stabilization are available which are shown below (Afrin, 2017).

2.2.1 Mechanical stabilization

Mechanical methods are those in which the compaction or bulk density is increased like dynamic or vibro-compaction (Dahiya, 2013). Mechanical stabilization is the process of improving the properties of a soil by changing the gradation of the soil. This process involves compaction and compaction of the soil by applying mechanical energy using various types of rollers, rammers, vibration techniques. In this method, the stability of the soil depends on the inherent properties of the soil material mixing of two or more types of natural soil to create a composite material that is superior to all ingredients. Mechanical stabilization is also achieved by mixing or blending two or more grades of soil to obtain materials that meet the required specifications (Afrin, 2017).

2.2.2 Stabilization based on different types of additives

A. Lime Stabilization

Lime is a calcium product that is composed of calcium oxide or calcium hydroxide, It is the civil engineering material that is used for binding two or more mineral materials and it has many qualities like compressive strength, workability, setting time. Lime provides an economical way to stabilize the soil. A soil improvement method that adds lime to the soil to improve its properties is called lime stabilization. The types of lime used in soil are slaked lime with high calcium content, monohydrated dolomite lime, calcite lime, and dolomite lime. The amount of lime used in most soil stabilizers in the range of 5% to 10% (Afrin, 2017).

B. Cement Stabilization

Cement is the binding material that is composed of calcium, silicon, aluminium, iron and many other materials. It is mostly used as a binding material throughout the world because of its qualities like high compressive strength, setting time. Soil cement stabilization is a combination of soil particles caused by the hydration of cement particles, which grow into crystals that can be hooked on each other, resulting in high compressive strength. Cement particles need to coat most material particles for successful bonding. Cement and soil need to be mixed with a specific particle size distribution to ensure good contact between soil particles and cement and thus to ensure efficient soil cement stabilization. Soil cement is a highly compressed mixture of soil / aggregate, cement, and water. Soil cement is also sometimes referred to as cement stabilizing base or cement treated aggregate base. Soil cement becomes a hard and durable material as the cement hydrates and increases its strength. Cement stabilization takes place during the compression process. When cement fills the voids between soil particles, the porosity of the soil decreases. When water is added to the soil, the cement reacts with the water and hardens. Therefore, the unit weight of the floor increases. As the cement hardens, it also improves shear strength and load bearing capacity. Cement helps lower the limits of liquids and plasticity index and workability of clayey soils. The reaction of cement does not depend on soil minerals and its important role is the reaction with water. Water can be present in any soil. This is why cement is used to stabilize various floors. There are many types of cement in the market (Afrin, 2017).

C. Fly ash Stabilization

Fly ash stabilization has become more important these days due to its wide availability. This method is cheaper and faster than any other method. It has a long history as an engineering material and is successfully used in geotechnical applications. Fly ash is a by-product of coal-fired power plants (Afrin, 2017).

D. Bottom ash as Stabilization

Earth as a building material has been used over the years in the construction industry. However its strength characteristics have been inadequate. Therefore, stabilizers are used to enhance its strength. Where conventional stabilizing agents like cement and lime have been used, they have considerably increased the cost of construction. It is with this backdrop that this paper describes the pozzolanic characteristics of municipal solid waste ash (MSWA) and its use as a stabilizing (Sanewu et al., 2013).

2.3 Review on bottom ash from Municipal waste to energy by-product.

Millions of tons of house hold waste are collected globally each year, with the vast majority disposed of in landfill dumps. The biomass resources in municipal solid waste comprises putrescible, paper, and plastic and average 90% the total municipal solid waste collected. The remaining 10% inert material includes sand, ferrous metal, aluminium, cans, glass and soil which are sorting facility and channelled to the proper recycling facility.

The use of municipal solid waste as feedstock for Cambridge plant at Rappi (Koshe) is also going to be a major source of income for EEPCO, potentially generating 25 million birr every year. Municipal solid waste will be converted in to energy at the Cambridge plant by direct gasification. The municipal solid waste used to produce energy by the Cambridge facility will be collected scrubbed and cleaned before feeding in to internal combustion engine or gas turbines to generate power.

2.3.1 Source of Bottom Ash

Municipal waste is collected from Addis Ababa city by small scale association to supply Rappi (koshe) waste to electric energy generation using collecting trucks which supplies and feed the waste to the combustion chamber which produces bottom ash as a bi-product (Cambridge industries et al, 2017)

2.3.2 Annual output of Bottom Ash

Table 2.5 Annual output of bottom ash from Addis Ababa city.

Months	Waste received (tone)	Waste burned (tone)		Total	Bottom ash (tone)
		Boiler no.1	Boiler no.2		
September	41,859.580	18878.165	18834.676	37712.952	7403.72
October	48461.74	21152.428	20081.392	41233.82	7566.1
November	40042.561	15595.41	15002.405	50597.815	5717.14
December	43277.54	16694.508	16564.317	33258.825	5351.16
January	50892.27	21584.9	16395.666	37980.500	7443.7
February	51157.34	22003.016	22540.88	44543.896	8835.02
March	47768.55	20687.533	23477.496	44165.029	8177.02
April	40033.16	19864.954	23670.55	43535.504	7201.26
May	37293.44	21673.788	14826.241	36500.029	6861.58
June	44880.97	20302.285	20794.027	41096.312	8858.02
July	51870.86	23572.13	25083.88	46656.00	9353.08
August	37383.58	24802.28	8486.901	33219.181	7647.44
Total	534921.54	246811.392	225758.520	472569.912	90215.24

2.4 Review on previous related literature

The study analysed different properties of black cotton soil without stabilization and stabilizing using lime and fly ash. The unconfined compressive strength test results of different proportion with effect of strength when fly ash is varied keeping lime constant and effect of strength when lime is varied keeping fly ash constant (DAHIYA (2013).

The stabilization of black cotton soil for the laboratory is performed making the black cotton constant and varying the fly ash by 10, 20, 30, 40, 50 and 60% and that of lime by 4,6,8,10,12 and 14%; The final observation and reading have been seen and analysed using charts and graphs, the result is concluded below. The strength of black cotton soil increases with increasing lime content and then decreases with increasing lime content. Therefore, the strength verses lime percentage rises first and then falls. The strength of black cotton soil increases with increasing fly ash content and decreases with increasing fly ash content.

Properties of black cotton soil with 12% lime and 30% fly ash Liquid limit 42%, Plastic limit 27.14%, Plasticity index 14.86%, Shrinkage limit 19.01%, Clay 9 %, Differential swell 25%, Optimum moisture content 30%, Maximum dry density 1.3 gm /cc, Unconfined compressive strength 195 kN. The optimum proportion of black cotton soil of Sangli is known to be 12% lime and 30% fly ash to attain the maximum strength. The comparison between black cotton soil without stabilizer and black cotton soil added with 12% lime and 30% fly ash is shown below.

Table 2.6 Comparison of black cotton soil with and without additives

Index properties	Black cotton soil with no stabilizer	Black cotton soil with 12 %	remarks
Liquid limit	60.33 %	42.00	Liquid limit has decreased
Plastic limit	21.50	27.14	Plastic limit has increased
Plasticity index	38.83	14.86	Plasticity index has reduced
Shrinkage limit	13.66	19.01	Shrinkage limit has increased
Differential swell	70%	25%	Swelling has decresed considerably
Optimum moisture content	26.4%	30%	OMC has increased
Maximum dry density	1.33 gm/cc	1.3 gm/cc	MDD has decreased little.
Unconfined compressive strength	30 KN	195 KN	Strength has increased considerably

It was found that adding up to 30% of bottom ash to the black cotton soil reduces the Optimum moisture content (OMC) and increases the maximum dry density (MDD), whereas adding 40% of bottom ash reverses this trend. This is due to good selection of soil bottom ash mixtures with up to 30% bottom ash, but with large amounts of bottom ash, intergranular filling disturbances can lead to reduced maximum dry densities (MDD). In addition, bottom ash has a lower specific gravity than soil, so replacing soil with excess bottom ash can reduce maximum dry density. Unconfined compression strength (UCS) of the soil-BA mix was also found to increase up to 30% Bottom ash content, and there was a reduction in unconfined compression strength (UCS) with 40% bottom ash. The unconfined shear strength (UCS) test results for Soil- bottom ash- cement mixtures with different ratios of Areca fibres showed that the addition of fibres significantly improved the strength of the Soil- bottom ash- cement mixture. The increase in unconfined shear strength (UCS) of the mixture with 1.5% betel nut fibre was 191% at 90 days' cure time. This study confirms that hardening has a significant effect on the strength behaviour of soil and cement mixtures. This can be traced back to the cement hydration process with soil and bottom ash.

Based on the result of the investigation using Dandili fly ash as a stabilizer of black cotton soil significantly improves index properties, compaction and strength of black cotton soil, plastic limit and liquid limit decreases which gain shear strength, increase the maximum dry density (MDD), unconfined compressive strength (UCS) California bearing ratio (CBR). The geotechnical properties of HubBAlli Dharwad black cotton soil can be favourably changed using the Dandlli fly ash and an optimum quantity between 30 to 40% yield the best possible result (**D.Hakari** and Puranik (2012).

Wubishet (2013): In this study, Bagasse ash does not result significantly change to use in expansive soil as a sub-grade material. Therefore, Bagasse ash is not effective Independent stabilizer for highly plastic swelling soils. However, Bagasse Ash in addition with lime, these soils can be effectively stabilized. Wide expansive soil Stabilized with Bagasse Ash in addition with lime can be used as a good subgrade. Therefore, the combination of the two local ingredients which are Bagasse ash and lime can effectively improve and support the sub-grade (Sudhakara, et al 2021).

Chapter 3
Materials and Methods

3.1 Introduction

Prior to laboratory testing, field investigation was conducted. In the field investigation process the colour and texture of the soil were determined to identify the soil. Laboratory investigation was done on the disturbed and undisturbed samples that were taken and the soil was described with different laboratory tests.

3.2 Materials

3.2.1 Expansive Soil

The expansive soil (black cotton soil) was collected from Gellan town near EEP substation at about 18km from Addis Ababa in Southeast direction at a depth 1.5-2m. The site was located just on the right side of Gellan main road about 4km at Merino village at Longitude 38^0 $80^{``}$ 27'', Latitude $8^0$ $82^{``}$ 32'' and elevation 2085m.

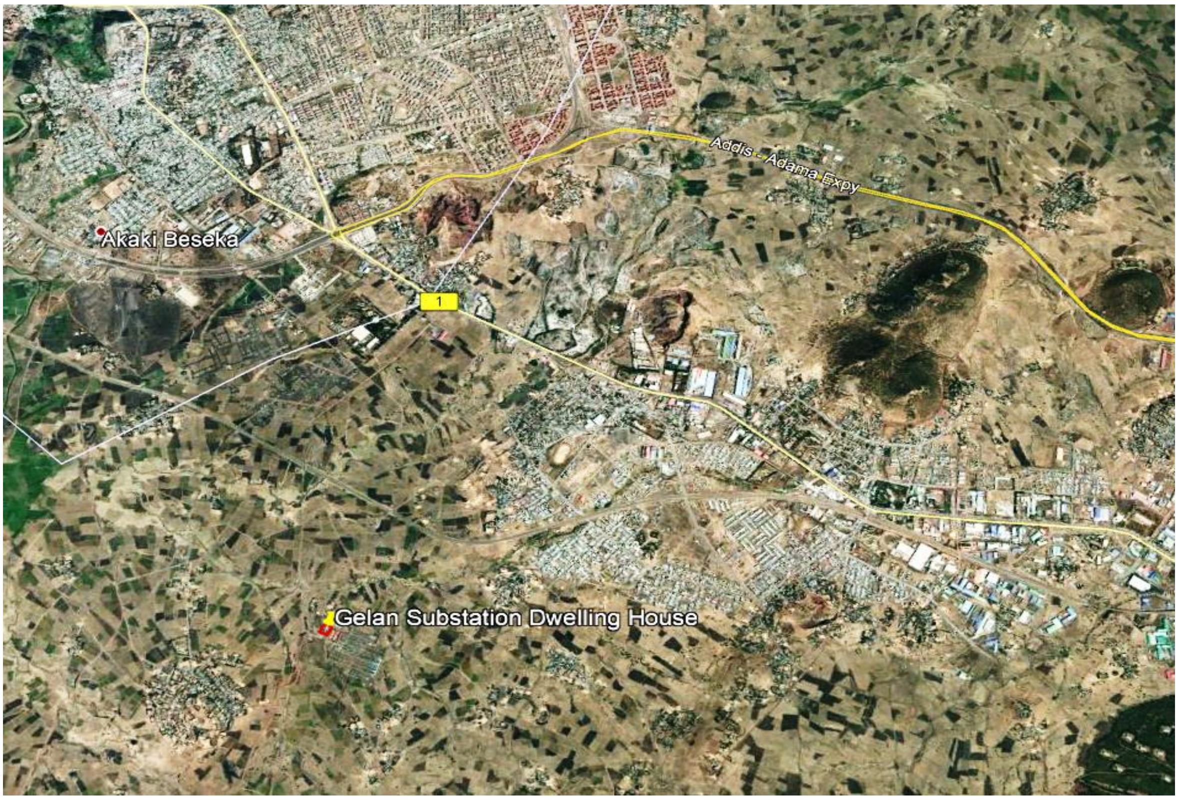

Figure 3.1 Location map of Gellan town.

Figure 3.2Test Pit1 (TP1)

3.2.2 Bottom Ash

The bottom ash that used as a stabilizer was collected from Reppi (koshe) waste to electric generation. The percentage composition for organic component of solid waste of A.A. city is about 64% by weight which are combustible materials (21%), Non-combustible materials (3%), Organic fines (34%), Fines less than 10mm size (29%) and Recyclable materials (13%) (Seyoum, k 2007). The bottom ash obtained after incineration of municipal solid waste has been found to have pozzolanic characteristics which would be beneficial in the stabilization of soils (Sanewu et al., 2013). According to the Ethiopian geological survey of laboratory, the breakdown by weight shows, the oxide elements determined by AAS (Automatic absorption spectrometry) were Al_2O_3, Fe_2O_3, CaO, MgO, Na_2O, K_2O, and MnO (Yigzaw, 2021).

Figure 3.3 Sources of bottom ash at Reppi (koshe) waste to energy electric Generation

3.2.3 Water

Clean tap water was used for laboratory test of stabilizing expansive soil (black cotton soil) using bottom ash for the proposed investigation except distilled water was used for specific gravity and atterberg limit tests.

3.3 Methods

3.3.1 Sample collection

The undisturbed and disturbed samples were collected for laboratory investigation with greater care for it should represent exactly the nature of the soil. The sample was collected from Gellan town near EEP substation at a depth of 1.5-2m site with great care using plastic bags to avoid the natural character of the soil and transported to Ethiopian Electric Power, AASTU, ASTU and ERA geotechnical laboratory for investigation.

3.3.2 Sample preparation

Prior to treatment and testing, the sample was prepared in accordance with the method Described in AASHTO T87-86.

This method involves:

1) Air dried of collected black cotton soil and bottom ash samples by spreading the materials to be exposed to air.
2) Breaking up the soil aggregates by rubber covered mallet and hand and magnet separation of ferrous and non-ferrous materials from the bottom ash was performed for test purpose.
3) Soil and bottom ash were treated manually by weight to get uniform mix ratio.

3.3.3 Laboratory Tests

3.3.3.1 Grain Size Analysis test

Grain size distribution and sieve analysis were carried out using ASTM D422 method before and after Hydrometer to identify the size of the particles in the soil and bottom ash. To find out the silt and clay fraction, 50g of dry samples passing No10 sieve were treated with 40 g dispersing agent (sodium hexa metha phosphate) $(NaPO_3)_6$ in 125 ml distilled water for 24 hours then filling with 1000ml distilled water then hydrometer analysis was performed to measure the amount of silt and clay size, as a result the sample was washed with sieves No.200 and the retained was oven dried for 24 hours then sieve was performed from sieve No

16 to 200 to determine the percentage of sand-sized particles in the specimens. In addition, based on ASTM D2487, sieve analysis of soil was classified the soils into groups by determining the particle sizes characteristics and distribution. The size distribution was often of critical importance to the way the material performs in use.

3.3.3.2 Specific gravity test

The ratio of the mass of unit volume of soil at a stated temperature to the mass of an equivalent volume of gas-free distilled water at the same temperature. The test gives an insight to how dense a soil sample and bottom ash was which in turn helps to distinguish the soil and bottom ash according to ASTM D 854 test method. By taking a 10 gram of oven-dried soil samples and bottom ash passed No 10 (2mm) sieve and carefully put in the 100 ml pycnometer which was then half-filled with distilled water. The air entrapped in the sample was removed by vacuum pump method, the bottle was then topped up with distilled water up to a calibration mark and brought up to a constant temperature after carefully dried and weighed. The result was expressed mathematically as shown

$$Gs = \frac{(W2-W1)}{[(W4-W1)-(W3-W2)]} \tag{3.1}$$

Where: W_1 = weight of density bottle, W_2 = weight of bottle + soil,

W_3 = weight of bottle + soil + water and W_4 = weight of bottle + water

3.3.3.3 Atterberg limit test

When a clayey soil combined with a large amount of water, it can become Depending on the moisture content of the soil, it was behaving like a liquid, semisolid, or solid material when it was gradually dried. These tests were performed using soil passed through No 40 (0.425mm) sieve and oven dried for the treated and untreated soil samples according to AASHTO T 88 and T 90 which the liquid Limit (LL) at the moisture content which soil begun to behave as a liquid material and flow, the plastic limit of a soil is the moisture content, expressed as a percentage of the weight of the oven-dry soil, at the boundary between the plastic and semisolid states of consistency and plastic index (PI) the difference of liquid limit and plastic limit. Approximately 150 g of soil and treated with bottom ash was soaked with distilled water for 24 hours. The soaked samples were mixed using a spatula to form a uniform paste then placed in a Casagrande cup and flattened with a standard grooving tool. A soil sample was cut in the centre, then the handle rotated at a rate of about 2 revolutions per second, the groove was closed with a count of 15 -35 blows. The procedure continued until three attempts. After plotting the water content for the number of strokes, the water content corresponding to 25 strokes was determined as the liquid limit. The plastic limit was then determined by mixing the soil with water and rolled in to a thread ⅛ in (3 mm) until it begun to crumble, mixing and rolling process was repeated until the average moisture content of the crushed part of the yarn is determined by percentage of the weight of the oven-dry soil.

3.3.3.4 Moisture content

The test was conducted in accordance with ASTM D 2216. The ratio of the weight of water to the weight of the solids in a given mass of soil. The laboratory procedures were performed by weighing the apparatus with the soil mass (100gm) and put in an oven up to 105-110^0 centigrade and weighing the dry as the formula shown below.

$$w\ (\%) = \frac{(W_2 - W_3)}{(W_3 - W_1)} * 100 \qquad (3.2)$$

Where, w (%) = water content, W_1 (g) = Weight of container,

W_2 (g) = Weight of container + wet soil and W_3 (g) =Weight of container +dry soil.

3.3.3.5 CBR Swell test

The CBR swell was the force necessary to prevent volume expansion in soil when it came into contact with water. The test was performed of treated and untreated soil samples with soaked CBR tests. The volume change of the compacted specimen was measured before and after soaking using dial gauge reading then calculated using the following formula and compared the result with ERA specification.

$$\text{CBR SWELL (\%)} = \frac{(Change\ in\ height\ during\ soaking)}{116.3\,mm} * 100\% \qquad (3.3)$$

3.3.3.6 California Bearing Ratio (CBR) test

California Bearing Ratio (CBR) used as indirect method of measuring soil shear strength for use as a sub-grade or sub-base material in flexible road construction or sidewalks. The test was performed using optimum moisture content (OMC) and maximum dry density (MDD) from compaction test results for treated and untreated samples passed No 4 (4.75mm) for soaked CBR tests. One point soaked CBR and CBR-swell tests were conducted for treated and untreated soil samples according to ASTM D1883 with 56 blows for each 5 layers of CBR mould tests. The moulds were soaked for four days (96 hours) which was believed that the soil was fully saturated and after 4 days of soaking, the samples were removed from the soaking tank, poured the water allowed to drain downward for 15 minutes. The CBR mould along with the soaked soil sample was brought to the CBR test machine for load and penetrations readings at 0.64 mm, 1.27 mm, 1.91mm, 2.54 mm, 3.81mm, 5.08 mm,7.62 mm, 10.16 and 12.70.

Stress-penetration curve: Calculate the penetration stress (dividing the load by the plunger area) and plot the stress penetration curve.

Bearing ratio: The bearing ratio can be obtained using stress values taken from the stress penetration curve for 2.54 mm and 5.08 mm penetration. The bearing ratio is calculated by dividing the corrected stress values at 2.54 mm and 5.08 mm by the standard stresses of 13.2 MPa and 20 MPa respectively and multiplying by 100. But the CBR is selected at 2.54 mm penetration. If the ratio at 5.08 mm penetration is greater, rerun the test. If the check test gives a similar result, the ratio at 5.08 mm is used, then the CBR value calculated by the formula shown below.

$$CBR\ (\%) = \left(\frac{Test\ load}{Standard\ load}\right) \times 100 \qquad (3.3)$$

Table 3.1 ERA manual-2002 rating of subgrade, sub-base & base-course materials based on CBR value.

CBR (%)	General Rating	Use
0-3	Very poor	Sub-grade
3-7	Poor to fair	Sub-grade
7-20	Fair	Sub-Base
20-50	Good	Base Coarse/Base
>50	Excellent	Base coarse

Figure 3.4 Initial swell reading

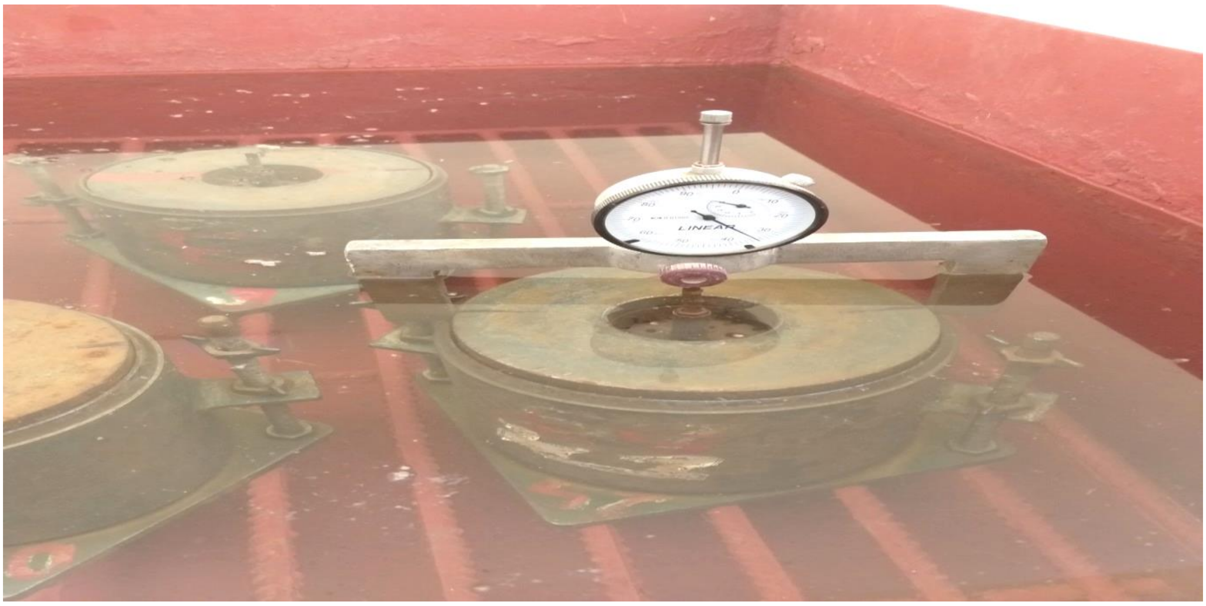

Figure 3.5 Final swell reading

Figure 3.6 CBR stress- strain test

3.3.3.7 Unconfined Compressive Strength

This test was performed according to ASTM D 2166 for treated and untreated soil sample passed No 4 (4.75mm) using Optimum moisture content from compaction test result. For soils, the undrained shear strength (su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (su) of clays is commonly determined from an unconfined compression test. The undrained shear strength (su) of a cohesive soil is equal to one-half the unconfined compressive strength (qu) when the soil is under the f = 0 condition (f = the angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion which is expressed as the formula below.

$$Su = \frac{qu}{2} \tag{3.4}$$

Then, as time passes, the pore water in the soil slowly dissipates, and the intergranular stress increases, so that the drained shear strength (s), given by s = c+ s‘tan f, must be used. Where s‘= intergranular pressure acting perpendicular to the shear plane; and s‘= (s - u), s = total pressure, and u = pore water pressure; c’ and φ’are drained shear strength parameters. The test procedures were the specimens were molded in the standard compaction mold, extracted using Shelby tube samplers and cut a soil specimen so that the ratio (L/d) is approximately between 2 and 2.5 then, carefully placed the specimen in the compression device and centered on the bottom plate. Adjust the device so that the upper plate just made contact with the specimen and set the load and deformation dials to zero, applied load up to the failed of the specimen and the max result was recorded for comparative of the strength gain for the percent of addition of the bottom ash.

3.3.3.8 Moisture-density relation (compaction) test

This laboratory test was performed to determine the relationship between the moisture content and the dry density of treated and untreated soil samples according to ASTM D1557. The compaction effort was the amount of mechanical energy that applied to the soil mass. Several different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, the soil used was passed No 4 (4.75mm) to determine the moisture content and the dry density of treated and untreated samples. There are two types of methods, the Standard Proctor test and the Modified Proctor Test. In this laboratory the modified proctor was used.

CHAPTER 4
Test results and discussion

4.1 Introduction

In this Chapter laboratory test results are briefly discussed and the effect of Bottom ash was evaluated for each laboratory tests. The physical and engineering properties are summarized to discuss the results of laboratory tests of grain size analysis, specific gravity, natural moisture content, Atterberg limits, swelling pressure, California bearing ratio (CBR), swell (CBR) unconfined compressive strength (UCS), moisture density relation (compaction).

4.2 Properties of Expansive soil (black cotton soil) and bottom ash

4.2.1 Grain size analysis test.

The grain size analysis test was determined by ASTM D 422 standard test method. The combined wet sieve and hydrometer test curve indicates that, the expansive soil (black cotton soil) contain fine gravel 1.57%, sand fractions 7.77%, silt 25.94% and clay 64.74% which shows that 90.68% of the soil passed through No 200 sieve. The soil falls under A-7-5 soil class based on AASHTO CH (Fat clays) as per the USCS classification systems and it's Potential of expansion was Low (inactive) as per Relation between clay activity and potential of expansion. Soils under this class are generally classified as material of poor engineering property for construction (Murthy, 2002) and the bottom ash showed that coarse gravel 20.88 %, fine gravel 32.29%, coarse sand, 20.82%, medium sand 18.96%, fine sand 1.86%, silt 0.05% and clay 5.15%. The gravel was 53.17% that is retained on No 4(4.75mm) which was well graded gravel symbolized by GW. Pozzuolana are defined as siliceous and aluminous materials that have little or no cementations properties in themselves but, will chemically react with calcium hydroxide in the presence of water to form compounds that have cementations properties. According to the geological survey of Ethiopian geochemical laboratory test result, the chemical composition of MSWI bottom ash was determined by the method of AAS (Automatic absorption spectrometry) was as follows SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, Na_2O, K_2O, MnO, P_2O_5, TiO, H_2O and LOI. The percentage combination of SiO_2, Al_2O_3 and Fe_2O_3 was 64.73% satisfies minimum of 50%, moisture content was 0.31% satisfies maximum 3% and Loss on ignition (LOI) was 3.82% satisfies maximum 6%, according to ASTM C 618 (2003) indicates it fulfilled the standard for chemical stabilizer and classified as pozzuolana Class C (Eden Asres, 2017).

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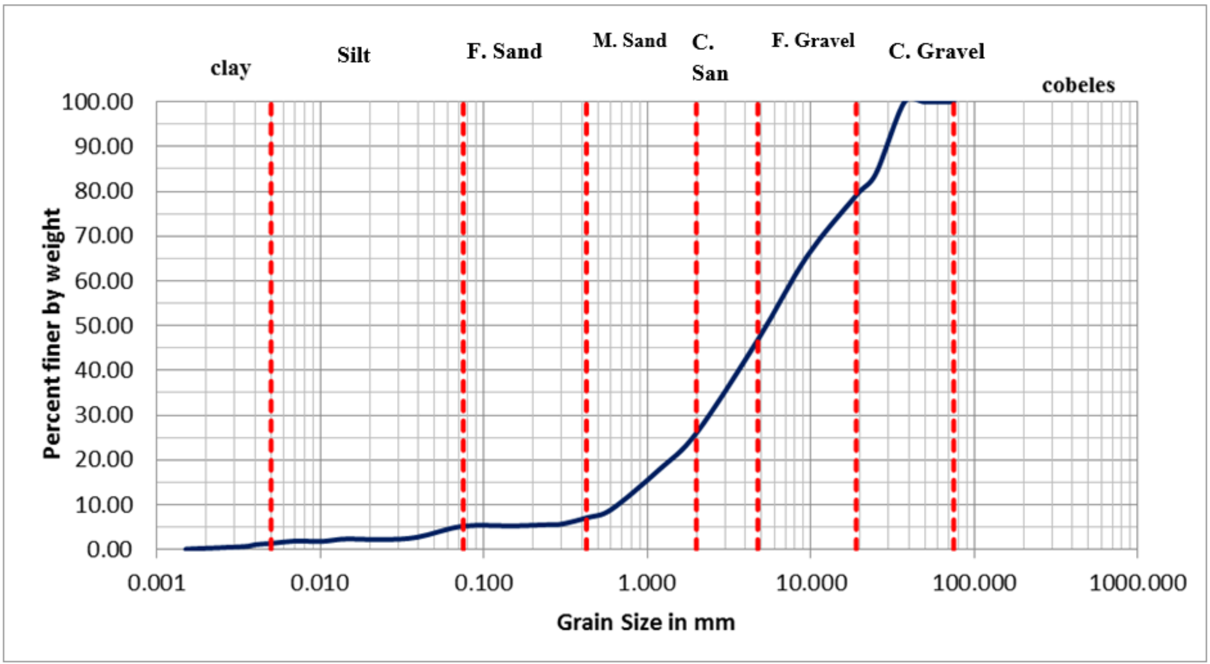

Figure 4.1 Sieve analysis and hydrometer combined of untreated black cotton soil

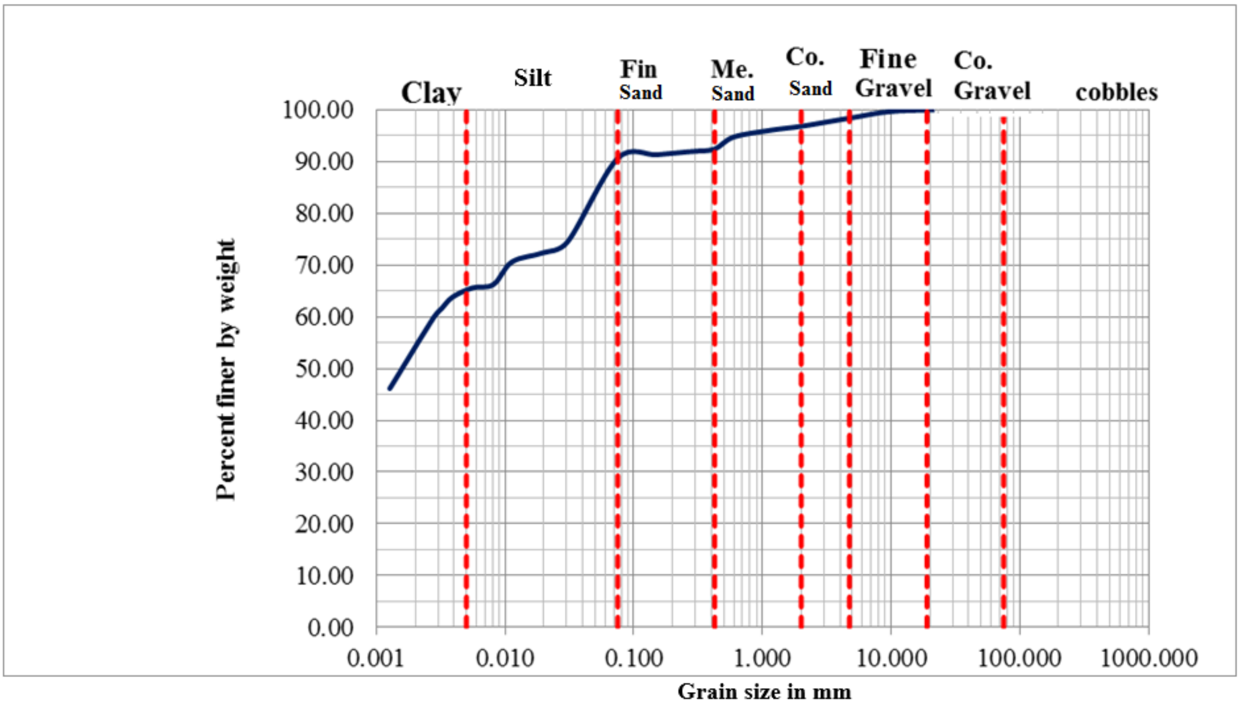

Figure 4.2 Sieve analysis and hydrometer combined of bottom ash

4.2.2 Natural moisture content

The determination of natural moisture content of an expansive soil (black cotton soil) was determined by bringing undisturbed soil samples from the proposed 1.5m, 1.75m and 2.0m depth of the pit by using a plastic bag not to lose its natural state and the test was conducted according to ASTM D 2216 and the result was taken the average of the three and was 40.5%.

4.3 Effects of bottom ash on Engineering Properties of expansive Soil

Bottom ash samples were collected from Reppi waste to electric energy in Addis Ababa and expansive soil (black cotton soil) is excavated and collected with great care for laboratory tests. The expansive soil (black cotton soil) in this research was treated with bottom ash which is both passed sieve No. 4 (4.75mm) and for attereberg limit only both samples which passed through Sieve No 40 (0.425mm) with ratio of 0%, 5%, 10%, 15%, 20%, 25% and 30% by weight of dry soil. The engineering properties of expansive soil (black cotton soil) and treated with bottom ash are presented for different types of tests as follows.

4.3.1 Specific gravity test results

According to The test method ASTM D 854 the result showed that the specific gravity value decreased while increasing per cent of bottom ash as shown in the table and figure below. The expansive soil (black cotton soil) was 2.76 and bottom ash 2.62 and the result of treated with bottom ash was shown in the table below. The reason specific gravity decreases was the bottom ash was well graded sand in sieve analysis laboratory test result.

Table 4.1 Specific Gravity verses % of bottom ash

S. No.	% of Bottom ash added	Specific Gravity
1	Pure or 0%	2.76
2	5%	2.74
3	10%	2.73
4	15%	2.73
5	20%	2.70
6	25%	2.67
7	30%	2.67

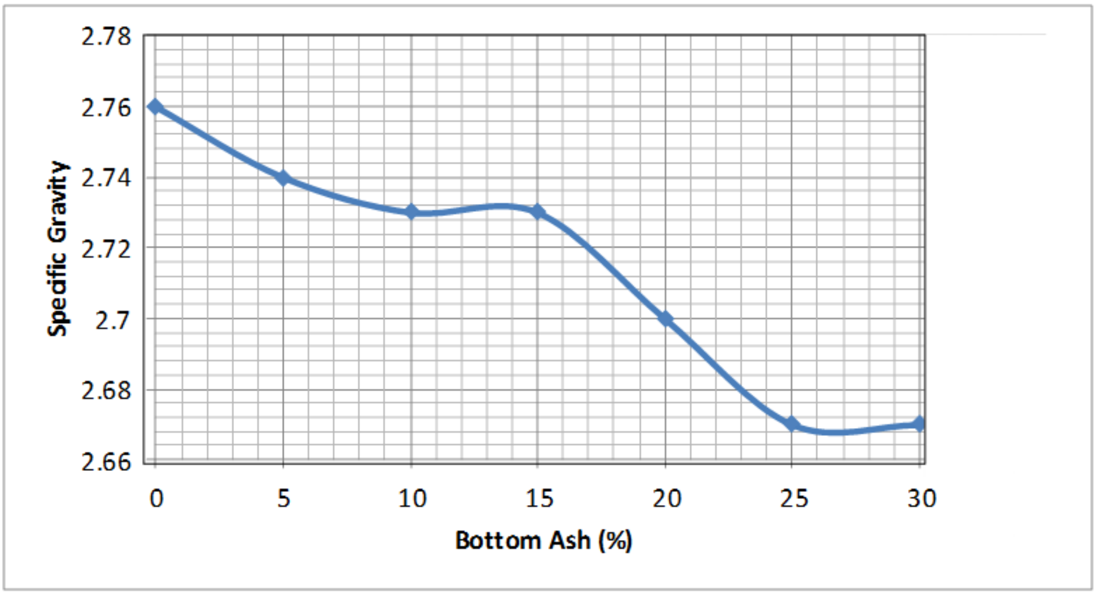

Figure 4.3 Specific Gravity verses % of bottom Ash

4.4 Test results of treated and untreated expansive soil using bottom ash.

4.4.1 Result observed on atterberg limits

The test was performed according to AASHTO T89 and T90 standard respectively. The result of liquid limit, plastic limit and plastic index decreased when treated with bottom ash and the maximum was attained at 30% of increment as shown in the table below. The reason was that bottom ash was well graded gravel (GW) from sieve analysis result and als- --ans that it is a Non-plastic material.

Table 4.2 Liquid limit, Plastic limit and Plastic index verses % of bottom ash.

S. No	% of bottom ash added	Liquid limit (PL) %	Plastic limit (PL) %	Plastic index (PI) %
1	0%	105	51	54
2	5%	100	49	51
3	10%	90	45	45
4	15%	87	44	43
5	20%	81	43	38
6	25%	77	42	35
7	30%	77	41	36

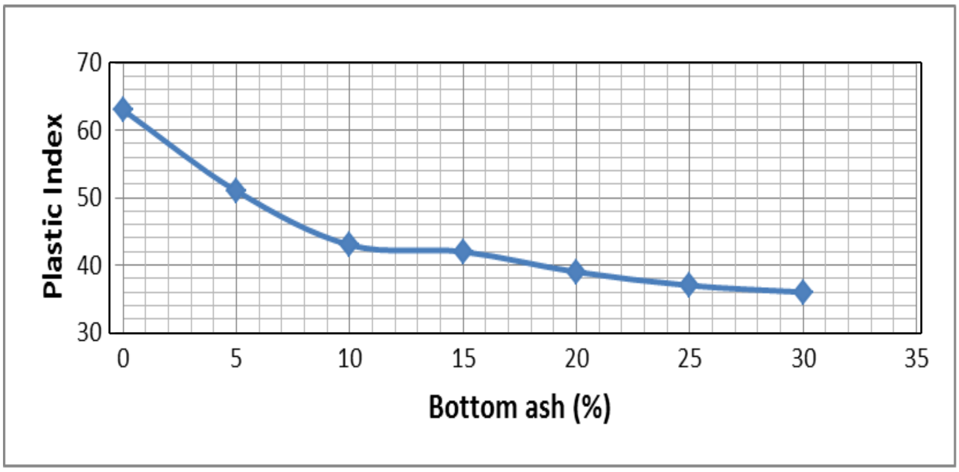

Figure 4.4 Liquid limit verses % of bottom ash

4.4.2 Result observed on CBR swell

The volume change which is swelling and shrinkage of soil due to changes in moisture are one of the important identification and classifying of sub-base and subgrade soils according to its swelling capacity or expansiveness. The result shows that the maximum value is 13.85% at 30% of addition of bottom ash and the reason for this improvement was the pozzuolana property and the chemical reactions taking place in the treated soil sample. It was good improvement but according to 2002 Ethiopian Road Authority specification, it should be less than 2% to use for subgrade.

Table 4.3 CBR swell (%) verses % of bottom ash

S. No	% of Bottom Ash Added	CBR swell (%)
1	Pure or 0%	24.23
2	5%	21.59
3	10%	16.89
4	15%	15.52
5	20%	15.49
6	25%	13.85
7	30%	13.54

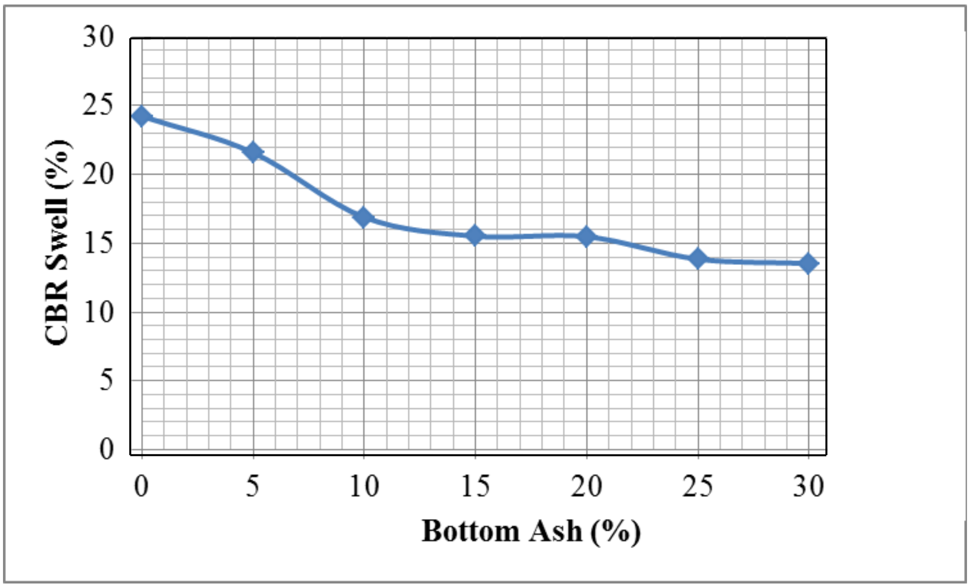

Figure 4.5 CBR swell (%) verses % of bottom ash

4.4.3 Result observed on soaked California Bearing Ratio (CBR) Test.

The soaked CBR test was conducted according to ASTM D 1883 by taking the optimum moisture content for each per cent treatment of expansive soil (black cotton soil) from compaction result. It is observed that when the per cent of bottom ash added increased, the value of CBR also increased, the reason was the chemical composition of the major constituents of MSWI ash are oxides of Calcium, Silicon, Iron and Aluminium and in addition it's pozzuolana property which impact to increase the strength of black cotton soil. It was good improvement but according to 2002 Ethiopian Road Authority specification, it should be greater than 3% to use for subgrade.

Table 4.4 CBR (%) verses % of bottom ash

S. No	% of Bottom Ash Added	CBR (%)
1	Pure or 0%	0.57
2	5%	0.96
3	10%	1.50
4	15%	1.74
5	20%	1.96
6	25%	2.47
7	30%	2.63

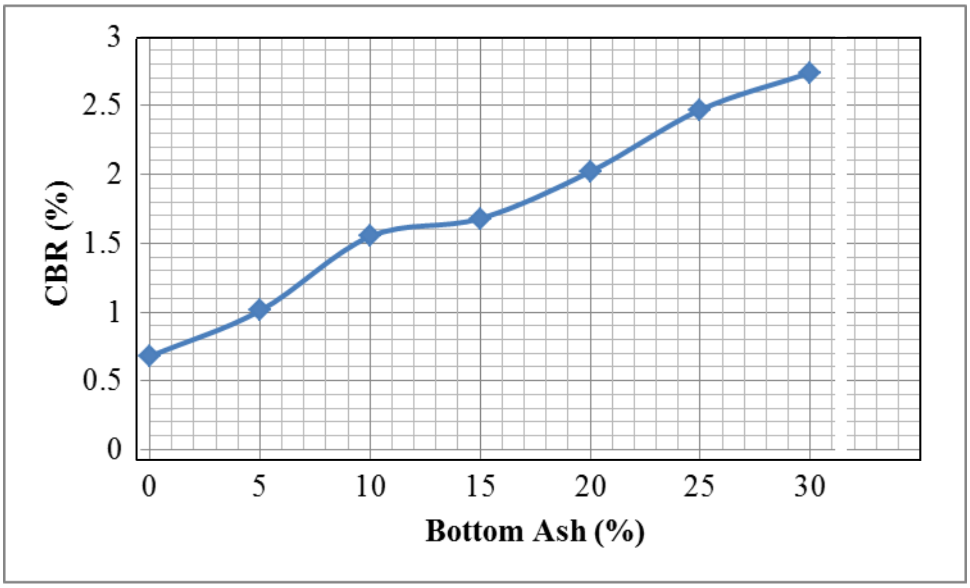

Figure 4.6 Soaked CBR verses % of bottom ash

4.4.4 Result observed on unconfined compressive strength (UCS)

The test was performed by using the optimum moisture content that has observed from compaction test results for each treated and untreated soil samples which was done by replacement from the natural soil. The test was conducted according to ASTM D 2166 and the result showed that the unconfined compressive strength test increased as the addition of per cent bottom ash increased. The cause of increasing the unconfined compressive strength while increasing per cent of bottom ash was its pozzolonic property which was sand with 50% or more coarse fraction smaller than No. 4 sieve size which was under well graded sand SW according to unified soil classification system (USCS).

Table 4.5 UCS verses % of bottom ash

S. No	% of Bottom Ash Added	UCS (kPa)
1	Pure or 0%	92.6
2	5%	132.33
3	10%	172.02
4	15%	181.73
5	20%	187.02
6	25%	191.43
7	30%	203.8

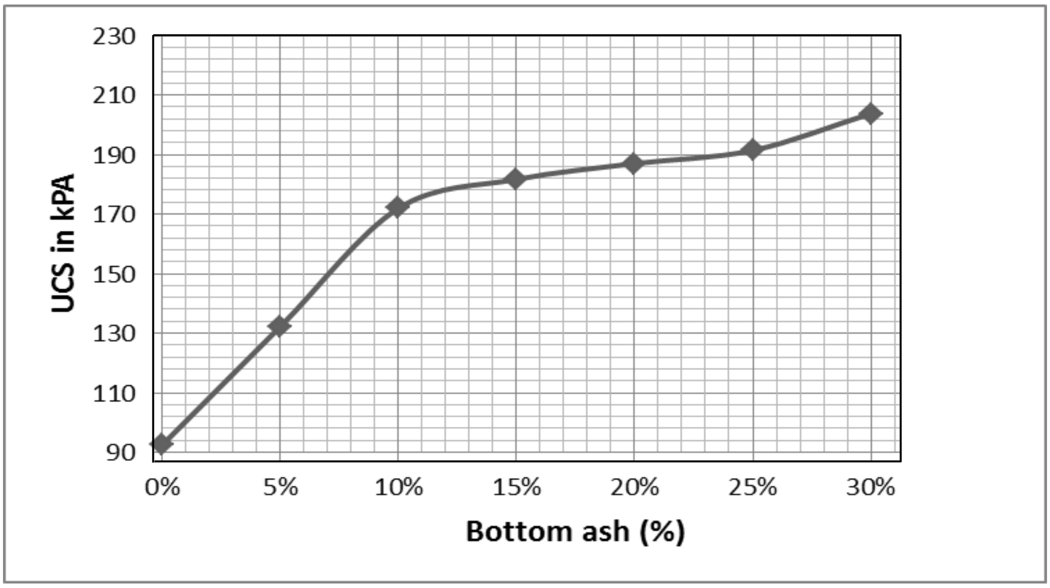

Figure 4.7 UCS verses % of bottom ash

4.4.5 Result observed on moisture density relation (compaction)

The result of expansive soil (black cotton soil) and bottom ash passing No 4 (4.75 mm) sieve were used to determine moisture-density relation of the expansive soil (black cotton soil) mixed with varying proportions of bottom ash additives in accordance to ASTM D 1557 and from the table and the graph of moisture content versus per cent of bottom ash the minimum optimum moisture content and from maximum dry density verses per cent of bottom ash the maximum result was observed at 30% and lesser OMC better was soil as if achieve MDD sooner at low water content because of its pozzuolana property.

Table 4.6 OMC (%) verses % of bottom ash

S. No	% of Bottom Ash Added	OMC (%)
1	Pure or 0%	29.51
2	5%	26.4
3	10%	28.6
4	15%	25.2
5	20%	25
6	25%	23
7	30%	20.6

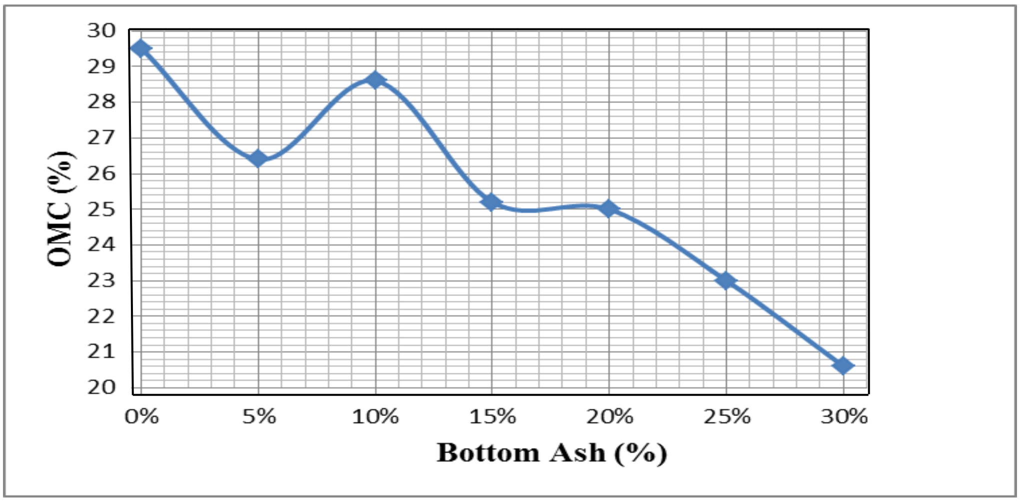

Figure 4.8 OMC verses % of bottom ash

Table 4.7 MDD (g /cm^3) verses % of bottom ash

S.No	% of bottom ash added	Maximum Dry Density (g/cm^3)
1	0%	1.38
2	5%	1.39
3	10%	1.41
4	15%	1.43
5	20%	1.44
6	25%	1.45
7	30%	1.46

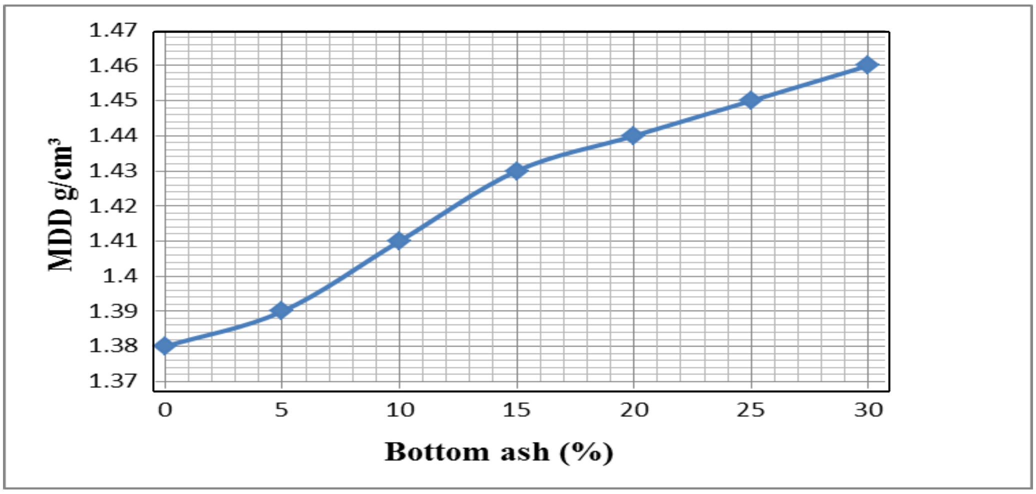

Figure 4.9 MDD verses % of bottom

The study was conducted to observe the change with the value at which per cent of bottom ash was observed as from the experiment according to the proposal approved so that, the maximum value was seen at 30% of increment of bottom ash.

Conclusion and Recommendations

Conclusion

This research work has been done to investigate the geotechnical properties of bottom ash stabilized by weight replacement with expansive soil and the following conclusion was drawn from the experiment test result.

1. The plasticity index decrease significantly with increment of bottom ash in per cent by weight.
2. The Optimum Moisture content (OMC) and Maximum Dry Density (MDD) decreasing and increasing respectively with increase of bottom ash.
3. The CBR slightly increased with increment of bottom ash on the proposed stabilization of expansive soil (black cotton) soil but didn’t meet ERA specification for sub grade.
4. The CBR swell per cent decreased from thirty per cent (30%) with pure or (0%) expansive (black cotton) soil with addition of thirty per cent (30%) bottom ash to thirteen point fifty-four per cent (13.54%) which was a good achievement but didn’t attained the Ethiopian Road Authority (ERA) specification for the sub grade.
5. The unconfined compressive strength test result showed increasing while increasing per cent of bottom ash and can be used as backfill material since the value observed at 30 % bottom ash is 203.8 kPa which was greater than the value observed of the soil.
6. The above test result showed that the increment per cent by weight of bottom ash improves the geotechnical properties of expansive (black cotton) soil.

Recommendations

According to the study of the geotechnical properties of stabilization of expansive (black cotton) soil using bottom ash by weight, the following are recommended.

1. Excess availability of bottom ash is present as waste product from waste to electric generation production at Reppi here in Addis Ababa Ethiopia which was the problem of Ethiopian Electric Power (EEP) and Addis Ababa city administration for dumping space where the average annual output was 90,252 tonnes.
2. In this research the effect of bottom ash as stabilizer improves the strength and geotechnical properties of expansive soil.
3. In regard to the result obtained further study should be considered by using additives or and increasing the percent of bottom ash to meet the Ethiopian Road Authority (ERA) standard to utilize for sub-grade.
4. Environmental issue should be considered at dumping site.
5. In regard to the bottom ash by-product, interested companies can utilize for input in the construction industry.

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Appendixes

Appendixes i: AASHTO soil classification

Table 1.1: AASHTO soil classification chart (Bello, 2015).

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	…	…	…	…	…	…	…	…	…	…
0.425 (No. 40)	30 max	50 max	51 min	…	…	…	…	…	…	…	…
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	…		…	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Note: Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL - 30

In the AASHTO system:

- **gravel is material** smaller than 75 mm (3 in.) but retained on a No. 10 sieve;
- **coarse sand is material** passing a No 10 sieve but retained on a No. 40 sieve; and fine sand is material passing a No. 40 sieve but retained on a No. 200 sieve.
- Material passing the No. 200 sieve is **silt-clay** and is classified based on Atterberg limits.

Appendix ii: Natural Moisture content

Table 1.2 Moisture content determinations

ETHIOPIAN ELECTRIC POWER		GEOTECHNICAL INVESTIGATION LABORATORY					
		Moisture Content ASTM D-2216					
Sample No:		S-1		S-2		S-3	
Sample Depth (m)		1.5	1.5	1.75	1.75	2	2
Container	no	P	BY	NDN	AW	KJ	LL
Weight of con + wet soil (M_1)	gm	138.81	138.52	137.50	134.03	139.29	138.14
Weight of con + dry soil (M_2)	g	111.19	110.53	108.19	104.79	109.96	108.71
Weight. of water (M_w) = M_1-M_2	g	27.62	27.99	29.31	29.24	29.33	29.43
Weight. of con (M)	g	38.81	38.52	37.50	34.03	39.29	38.14
Weight of dry soil (MM_{ds}) = M_2-M	g	72.38	72.01	70.69	70.76	70.67	70.57
Moisture content (M) = Mw/Mds*100	%	38.2	38.9	41.5	41.3	41.5	41.7
Average	%	38.5		41.4		41.6	

Average Moisture Content at 1.5-2.0 m=40.5

Appendix iii: Specific Gravity Determination

Table 2.1 Specific gravity for 0% (pure) bottom ash

Sample Number	A	B
Pycnometer #	75	63
A. Mass of Pycnometer (empty, clean)	35.9	34.6
B. Mass of empty pycno + Dry soil	45.9	44.6
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	137.1	136.8
E. Mass of pycno + Soil + full of water	143.5	143.1
F. $Gs = \frac{C}{(C+(D-E))}$	2.78	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.74	

Table 2.2 Specific gravity for 5% bottom ash

Sample Number	A	B
Pycnometer #	75	63
A. Mass of Pycnometer (empty, clean)	35.9	34.6
B. Mass of empty pycno + Dry soil	45.9	44.6
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	137.1	136.8
E. Mass of pycno + Soil +full of water (boiled)	143.5	143.1
F. $Gs = \frac{C}{(C+(D-E))}$	2.78	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.74	

Table 2.3 Specific gravity for 10% bottom ash

Sample Number	A	B
Pycnometer #	40	57
A. Mass of Pycnometer (empty, clean)	35.5	35.9
B. Mass of empty pycnometer + Dry soil	45.5	45.9
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	137.3	135.9
E. Mass of pycnometer + Soil +full of water	143.6	142.3
F. $Gs = \frac{C}{(C+(D-E))}$	2.70	2.77
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.73	

Table 2.4 Specific gravity for 15% bottom ash

Sample Number	A	B
Pycnometer #	38	37
A. Mass of Pycnometer (empty, clean)	35.5	35.2
B. Mass of empty pycnometer + Dry soil	45.5	45.2
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	134	136.9
E. Mass of pycnometer + Soil +full of water	140.4	143
F. $Gs = \frac{C}{(C+(D-E))}$	2.77	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.73	

Table 2.5 Specific gravity for 20 % bottom ash

Sample Number	A	B
Pycnometer #	7	21
A. Mass of Pycnometer (empty, clean)	35.9	34.6
B. Mass of empty pycnometer + Dry soil	45.9	44.6
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	137	135.6
E. Mass of pycnometer + Soil +full of water	143.9	141.9
F. $Gs = \frac{C}{(C+(D-E))}$	2.70	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.70	

Table 2.6 Specific gravity for 25 % bottom ash

Sample Number	A	B
Pycnometer #	5	4
A. Mass of Pycnometer (empty, clean)	35.5	35.9
B. Mass of empty pycnometer + Dry soil	45.5	44.9
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	137.1	137
E. Mass of pycnometer + Soil +full of water	143.3	143.3
F. $Gs = \frac{C}{(C+(D-E))}$	2.70	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.67	

Table 2.7 Specific gravity for 30 % bottom ash

Sample Number	A	B
Pycnometer #	20	33
A. Mass of Pycnometer (empty, clean)	35.5	34.6
B. Mass of empty pycnometer + Dry soil	45.5	44.6
C. Mass of Dry Soil (No. 10 sieve)	10	10
D. Mass of Pycnometer full of water	134	135.5
E. Mass of pycnometer + Soil +full of water	140.2	141.8
F. $\text{Gs} = \frac{C}{(C+(D-E))}$	2.63	2.70
Average Specific Gravity at 20^0 centigrade = $\frac{(1+2)}{2}$	2.67	

Figure 2.1 Vacuum pump

Figure 2.2 Removing air bubble with vacuum pump

Appendix IV: Atterberg Limit Determination

Table 3.1 Liquid Limit and Plastic Limit test for 0% bottom ash (pure soil)

Moisture can number	E-4	D-10	B-11	A-3	C-30
Number of Drops (N)	32	27	17	-	-
MC = Mass of empty, clean can grams	24.1	23.6	24	23	24
MCMS = Mass of can and moist soil grams	45.5	45.1	45.4	30	30.6
MCDS = Mass of can and dry soil grams	34.7	34.1	34.2	27.6	28.4
Mass of soil solids (MS) = (MCDS – MC) grams	10.6	10.5	10.2	4.6	4.4
Mass of pore water (MW) = (MCMS- MCDS) grams	10.8	11	11.2	2.4	2.2
w = Water content, w%	101.9	104.7	109.8	52.1	50

Liquid Limit (%) = 105, Plastic Limit (%) = 51 and Plastic Index = 54

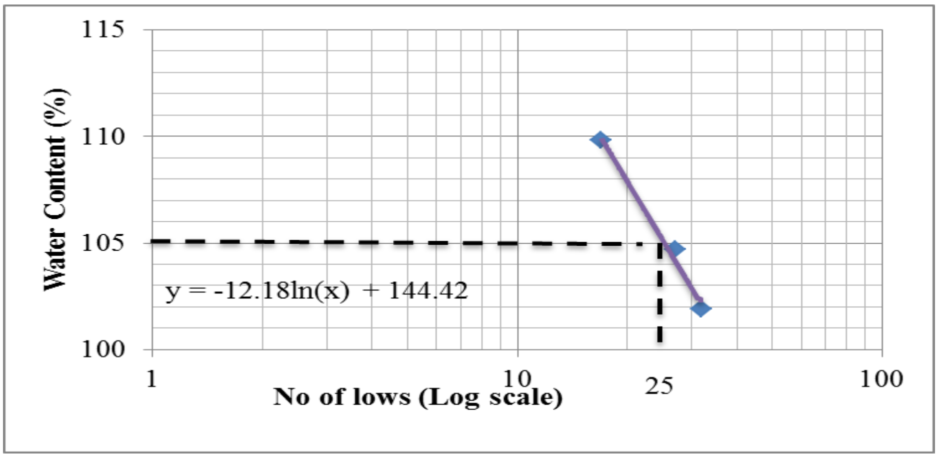


Figure 3.1 Liquid limit tests for 0% bottom ash

Table 3.2 Liquid Limit and Plastic Limit test for 5% bottom ash

Moisture can number	C-20	C-21	C-24	A-4	C-15
Number of Drops (N)	34	26	18	-	-
MC = Mass of empty, clean can grams	24.2	23.9	24.3	23	24
MCMS = Mass of can and moist soil grams	40.1	37.5	36.9	29.6	30.8
MCDS = Mass of can and dry soil grams	32.3	30.7	30.5	27.6	28.4
Mass of soil solids (MS) = (MCDS – MC) grams	8.1	6.8	6.2	4.6	4.4
Mass of pore water (MW) = (MCMS- MCDS) grams	7.8	6.8	6.4	2.2	2.2
w = Water content w%	96.3	100	103.3	47.8	50

Liquid Limit (%) = 100, Plastic Limit (%) = 49 and Plastic Index = 51

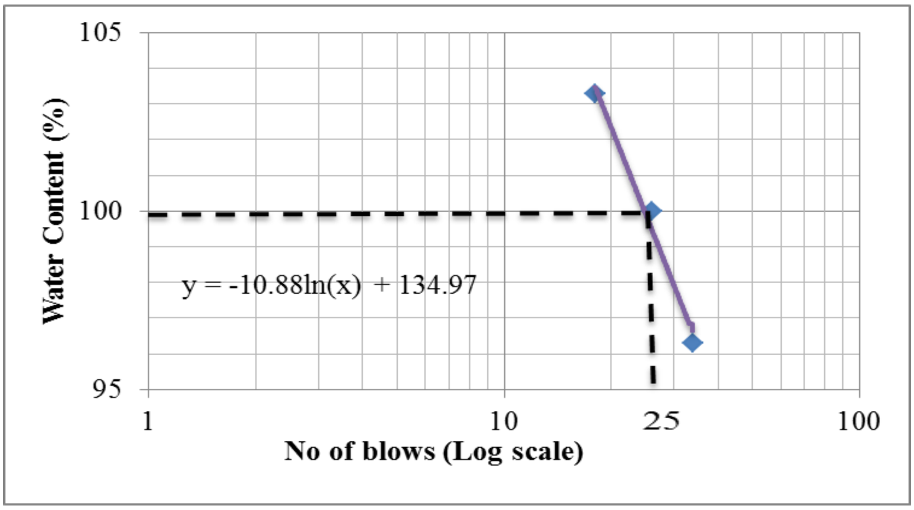


Figure 3.2 Liquid Limit test for 5% bottom ash

3.3 Liquid limit and Plastic Limit test for 10% bottom ash

Moisture can number	C-11	C-B	E-0	A-8	E-11
Number of Drops (N)	33	28	18	-	-
MC = Mass of empty, clean can grams	23.9	23.6	23.7	22.7	23.6
MCMS = Mass of can and moist soil grams	34.9	34.9	37	29.8	30.7
MCDS = Mass of can and dry soil grams	29.8	29.6	30.6	27.6	28.5
Mass of soil solids (MS) = (MCDS – MC) grams	5.9	6	6.9	4.9	4.9
Mass of pore water (MW) = (MCMS- MCDS) grams	5.1	5.3	6.4	2.2	2.2
w = Water content, w%	86.4	88.3	94.1	44.9	44.9

Liquid Limit (%) = 90, Plastic Limit (%) = 45 and Plastic Index = 45

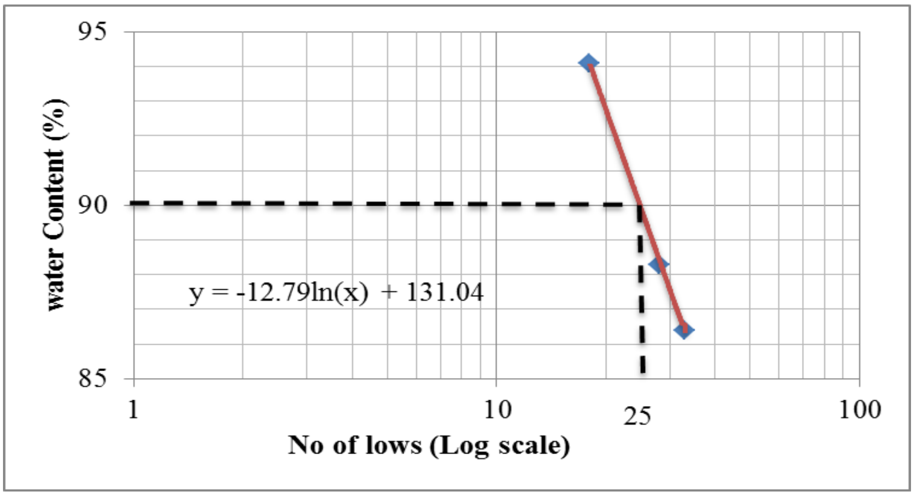


Figure 3.3 Liquid Limit test for 10% bottom ash

Table 3.4 Liquid limit and Plastic Limit test for 15% bottom ash

Moisture can number	E-22	B-6	D-5	D	O
Number of Drops (N)	32	27	17	-	-
MC = Mass of empty, clean can grams	23.8	23.3	23.9	23.5	23.9
MCMS = Mass of can and moist soil grams	43.7	38	38	29.4	29.9
MCDS = Mass of can and dry soil grams	34.7	31.2	31.2	27.6	28
Mass of soil solids (MS) = (MCDS – MC) grams	10.9	7.9	7.3	4.1	4.1
Mass of pore water (MW) = (MCMS- MCDS) grams	9	6.8	6.8	1.8	1.8
w = Water content, w%	82.5	86	93.1	43.9	43.9

Liquid Limit (%) = 87, Plastic Limit (%) = 44 and Plastic Index = 43

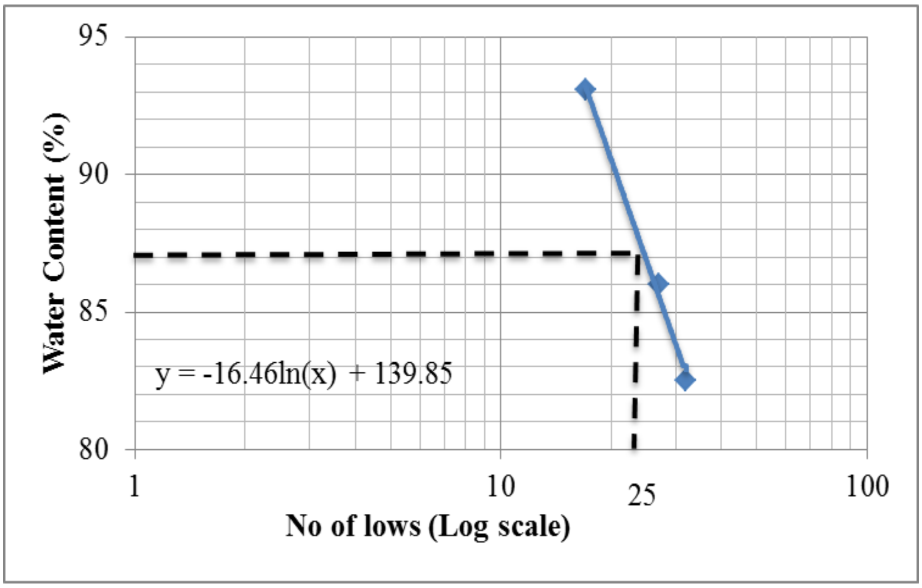


Figure 3.4 Liquid Limit test for 15% bottom ash

Table 3.5 Liquid limit and Plastic Limit test for 20% bottom ash

Moisture can number	E-16	E-27	P-11	12	13
Number of Drops (N)	33	27	18	-	-
MC = Mass of empty, clean can grams	40.5	24.6	23.7	24	24
MCMS = Mass of can and moist soil grams	56.3	40	38.7	30	29.3
MCDS = Mass of can and dry soil grams	48.8	32.8	32.3	28.2	27.7
Mass of soil solids (MS) = (MCDS – MC) grams	8.3	8.2	8.6	4.2	3.7
Mass of pore water (MW) = (MCMS- MCDS) grams	7.5	7.1	6.4	1.8	1.6
w = Water content, w%	90.3	86.5	74.4	42.8	43.2

Liquid Limit (%) = 81, Plastic Limit (%) = 43 and Plastic Index = 38

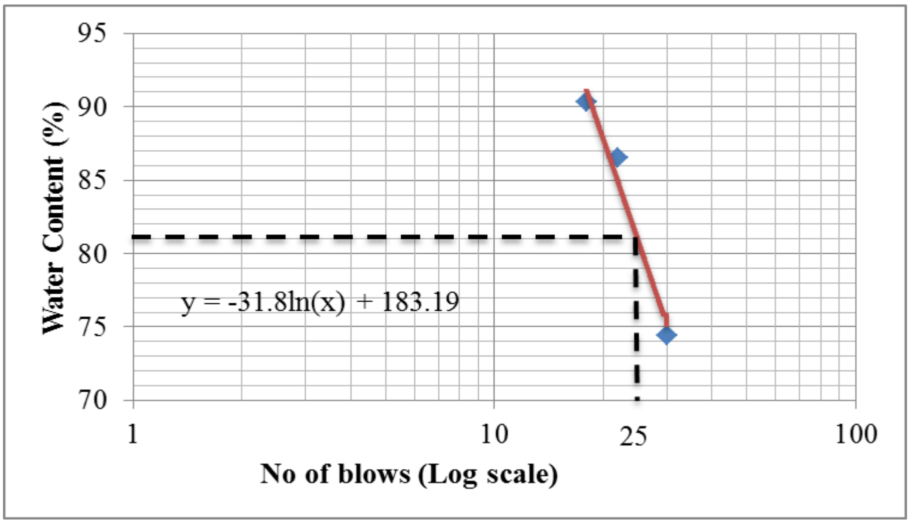


Figure 3.5 Liquid Limit test for 20% bottom ash

Table 3.11 Liquid Limit and Plastic Limit test for 25% bottom ash

Moisture can number	E-6	C-13	E-32	K	A
Number of Drops (N)	32	27	17	-	-
MC = Mass of empty, clean can grams	24.6	24.2	25.3	23.6	23.3
MCMS = Mass of can and moist soil grams	41.8	40.8	35.9	30	30.7
MCDS = Mass of can and dry soil grams	34.5	33.6	31.1	28.1	28.5
Mass of soil solids (MS) = (MCDS – MC) grams	9.9	9.4	5.8	4.5	5.2
Mass of pore water (MW) = (MCMS- MCDS) grams	7.3	7.2	4.8	1.9	2.2
w = Water content, w%	73.7	76.6	82.7	42.2	42.3

Liquid Limit (%) = 77, Plastic Limit (%) = 42 and Plastic Index = 35

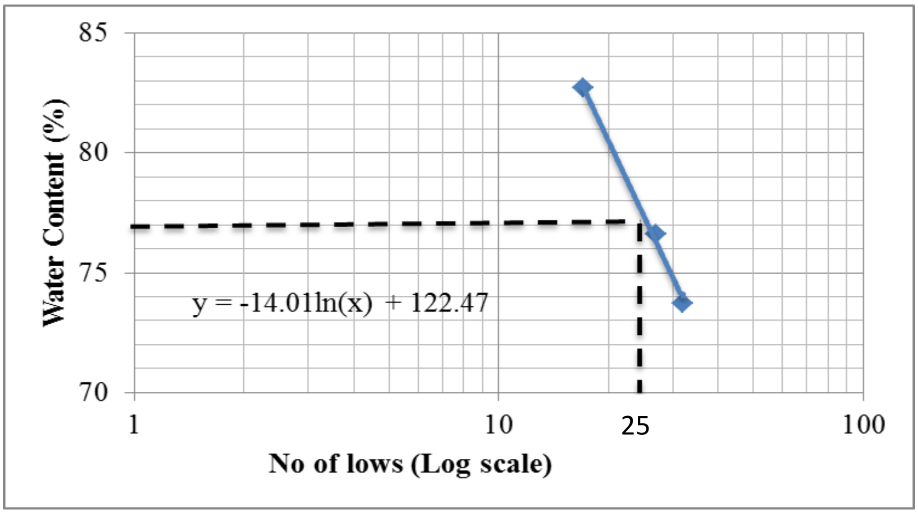


Figure 3.6 Liquid Limit test for 25% bottom ash

Table 3.13 Liquid Limit and Plastic Limit test for 30% bottom ash

Moisture can number	3	B-22	B-1	A-7	C-1
Number of Drops (N)	34	26	16	-	-
MC = Mass of empty, clean can grams	24.1	23.6	23.7	23.5	23.2
MCMS = Mass of can and moist soil grams	38.2	37.5	40.9	30	30.7
MCDS = Mass of can and dry soil grams	32.2	31.5	33.2	28.1	28.5
Mass of soil solids (MS) = (MCDS – MC) grams	8.2	7.9	9.5	4.6	5.3
Mass of pore water (MW) = (MCMS- MCDS) grams	6	6	7.7	1.9	2.2
w = Water content, w%	74	75.9	81	41.3	41.5

Liquid Limit (%) = 77, Plastic Limit (%) = 41 and Plastic Index = 36

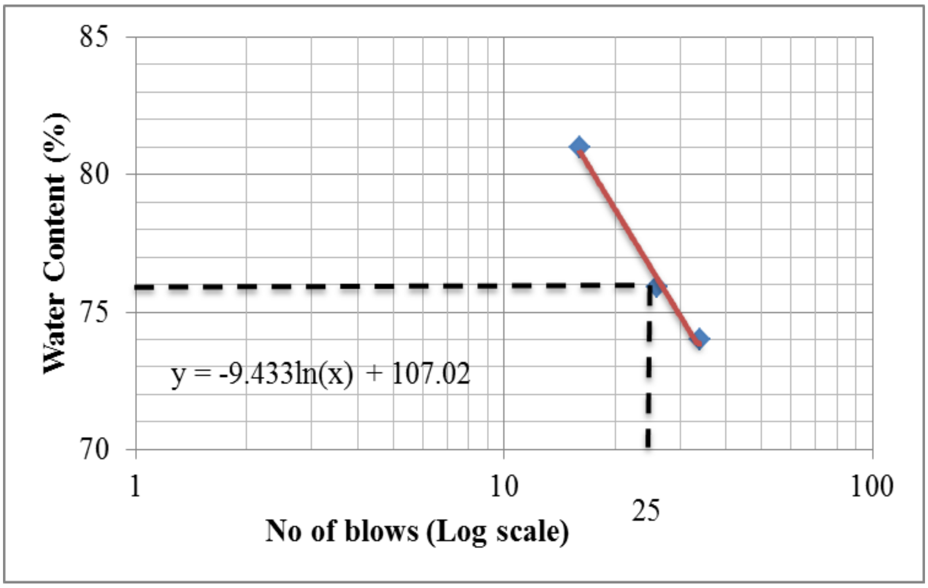


Figure 3.7 Liquid Limit test for 30% bottom ash

Appendix v: CBR (%) and CBR swell(%) determination

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	Pure Black Cotton Soil	DATE AND TIME SOAKED	14/04/22
PURPOSE	Sub-grade	DATE AND TIME TESTED	18/04/22
DEPTH	-	LAB.NO.	181/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows	
		BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		58		57	
WEIGHT OF SOIL + MOLD	g	9680	10370	9925	10530
WEIGHT OF MOLD	g	6106	6106	6176	6176
WEIGHT OF SOIL	g	3574	4264	3744	4354
VOLUME OF MOLD	cc	2091.27	2091.27	2078.00	2078.00
WET DENSITY OF SOIL	g/cc	1.709	2.039	1.802	2.095
DRY DENSITY OF SOIL	g/cc	1.381	1.366	1.450	1.412

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows		
		BEFORE	AFTER		BEFORE	AFTER	
CONTAINER NUMBER		L3	Bottom	Bottom	BN	Bottom	Bottom
WET SOIL + CONTAINER	g	427.0	853.9	1117.7	531.5	961.6	1960.5
DRY SOIL + CONTAINER	g	358.5	661.7	763.8	441.0	746.6	1563.7
WEIGHT OF WATER	g	68.5	192.2	353.9	90.5	215.0	396.8
WEIGHT OF CONTAINER	g	70.2	165.8	170.8	68.2	169.1	897.1
WEIGHT OF DRY SOIL	g	288.3	495.9	593.0	372.8	577.5	666.6
MOISTURE CONTENT	%	23.8	38.8	59.7	24.3	37.2	59.5
AVG. MOISTURE CONTENT	%		49.2			48.4	

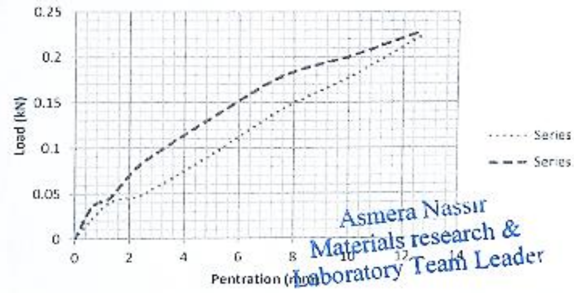
PENETRATION TEST DATA

Ring Factor 0.04489 KN/Div Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %
0	0	0			0	0		
0.64	0.5	0.022445			0.8	0.035912		
1.27	0.9	0.040401			1	0.04489		
1.91	1	0.04489			1.5	0.067335		
2.54	1.1	0.049379		0.37	1.9	0.085291		0.64
5.08	2.1	0.094269		0.47	3	0.13467		0.68
7.62	3.2	0.143648			4	0.17956		
10.16	4	0.17956			4.5	0.202005		
12.7	5	0.22445			5.1	0.228939		

SWELL %

Height of Specimen (mm)		116	115.4
No OF BLOWS		56-Tril-1	56-Tril-2
RDG (BEFORE SOAKING)		0.007	0.008
RDG(AFTER SOAKING)		2999	2608
PERCENT SWELL		25.85	22.61
AVERAGE PERCENT SWELL		24.23	



Asmera Nassir
Materials research &
Laboratory Team Leader

REMARKS:-	Bottom pentrated		
Lead Lab.Technician		Lead Research Engineer (Materials)	Team Leader
Name	Marta G/Hana	Name Negash Abido	Name Asmera Nassir
Sign		Sign	Sign
Date	11/05/2022	Date 11/05/22	Date 11/05/22

Figure 4.1 CBR (%) and CBR swell (%) for 0 % or pure soil

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	95% Black Cotton Soil mixed with 5% Bottom Ash	DATE AND TIME SOAKED	6/5/2022
PURPOSE	Sub-grade	DATE AND TIME TESTED	10/5/2022
DEPTH	-	LAB.NO.	222/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows			
		BEFORE	AFTER	BEFORE	AFTER		
MOLD NUMBER		62		59			
WEIGHT OF SOIL + MOLD	g	10010	10740	9970	10440		
WEIGHT OF MOLD	g	6431	6431	6178	6178		
WEIGHT OF SOIL	g	3579	4309	3792	4262		
VOLUME OF MOLD	cc	2120.175	2120.175	2100.142	2100.142		
WET DENSITY OF SOIL	g/cc	1.688	2.032	1.806	2.029		
DRY DENSITY OF SOIL	g/cc	1.353	1.344	1.429	1.347		

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows				
		BEFORE	AFTER		BEFORE	AFTER			
CONTAINER NUMBER		B7	TOP	Bottom	B100	TOP	Bottom		
WET SOIL + CONTAINER	g	495	462.9	515.0	448.6	467.1	485.3		
DRY SOIL + CONTAINER	g	414.6	348.6	346.6	370.5	349.9	330.6		
WEIGHT OF WATER	g	83.4	114.3	168.4	78.1	117.2	154.7		
WEIGHT OF CONTAINER	g	77.5	78.8	65.8	74.6	69.1	70.6		
WEIGHT OF DRY SOIL	g	337.1	269.8	280.8	295.9	280.8	260.0		
MOISTURE CONTENT	%	24.7	42.4	60.0	26.4	41.7	59.5		
AVG. MOISTURE CONTENT	%		51.2		50.6				

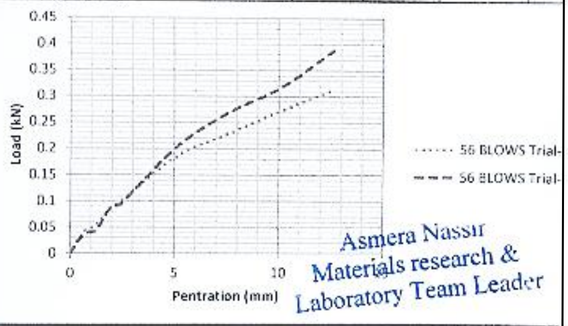
PENETRATION TEST DATA

Ring Factor 0.04483 KN/Div — Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %
0	0	0			0	0		
0.64	0.9	0.040347			0.8	0.035864		
1.27	1.2	0.053796			1	0.04483		
1.91	1.9	0.085177			1.9	0.085177		
2.54	2.3	0.103109		0.77	2.2	0.098626		0.74
5.08	4.1	0.183803		0.92	4.5	0.201735		1.01
7.62	5.1	0.228633			6	0.26898		
10.16	6.1	0.273463			7.1	0.318293		
12.7	7	0.31381			8.7	0.390021		

SWELL %

Height of Specimen (mm)	116		
No OF BLOWS		56 Trial-1	56 Trial-2
RDG (BEFORE SOAKING)		0.006	0.006
RDG(AFTER SOAKING)		2617	2395
PERCENT SWELL		22.55	20.64
AVERAGE PERCENT SWELL	21.59		



Asmera Nassir
Materials research &
Laboratory Team Leader

REMARKS:-	Bottom pentrated		
Lead Lab.Technician		Lead Research Engineer (Materials)	Team Leader
Name	Marta G/Hana	Name Negash Abido	Name Asmera Nassir
Sign		Sign	Sign
Date	12/05/2022	Date 12/05/22	Date 12/05/22

Figure 4.2 CBR (%) and CBR swell (%) for 5% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	90% Black Cotton Soil mixed with 10% Bottom Ash	DATE AND TIME SOAKED	14/04/22
PURPOSE	Sub-grade	DATE AND TIME TESTED	18/04/22
DEPTH	-	LAB.NO.	164/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows	
		BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		ERA156		62	
WEIGHT OF SOIL + MOLD	g	10050	10500	10340	10810
WEIGHT OF MOLD	g	6226	6226	6431	6431
WEIGHT OF SOIL	g	3824	4274	3909	4379
VOLUME OF MOLD	cc	2111.56	2111.56	2120.18	2120.18
WET DENSITY OF SOIL	g/cc	1.811	2.024	1.844	2.065
DRY DENSITY OF SOIL	g/cc	1.400	1.397	1.426	1.412

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows		
		BEFORE	AFTER		BEFORE	AFTER	
CONTAINER NUMBER		B8	Bottom	Bottom	TE	Bottom	Bottom
WET SOIL + CONTAINER	g	518.4	1018.4	1039.4	513.8	1060.4	1014.8
DRY SOIL + CONTAINER	g	416.5	798.2	728.2	423.9	817.6	713.7
WEIGHT OF WATER	g	101.9	220.2	311.2	89.9	242.8	301.1
WEIGHT OF CONTAINER	g	69.4	158.5	166.2	116.9	162.7	170.0
WEIGHT OF DRY SOIL	g	347.1	639.7	562.0	307.0	654.9	543.7
MOISTURE CONTENT	%	29.4	34.4	55.4	29.3	37.1	55.4
AVG. MOISTURE CONTENT	%		44.9			46.2	

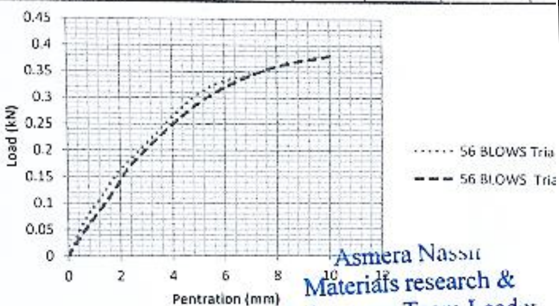
PENETRATION TEST DATA

Ring Factor 0.04489 KN/Div   Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %
0	0	0			0	0		
0.64	1.5	0.067335			1.1	0.049379		
1.27	2.5	0.112225			2	0.08978		
1.91	3.5	0.157115			3	0.13467		
2.54	4.2	0.188538		1.41	4	0.17956		1.35
5.08	6.9	0.309741		1.55	6.5	0.291785		1.46
7.62	7.9	0.354631			7.9	0.354631		
10.16	8.2	0.368098			8.5	0.381565		
12.7	9	0.40401			9	0.40401		

SWELL %

Height of Specimen (mm)		117	115.8
No OF BLOWS		56-Tril-1	56-Tril-2
RDG (BEFORE SOAKING)		0.005	0.006
RDG(AFTER SOAKING)		1795	2148
PERCENT SWELL		15.37	18.40
AVERAGE PERCENT SWELL		16.89	



Asmera Nassir
Materials research &
Laboratory Team Leader

REMARKS:-	Bottom pentrated	
Lead Lab.Technician	Lead Research Engineer (Materials)	Team Leader
Name Marta G/Hana	Name Negash Abido	Name Asmera Nassir
Sign	Sign	Sign
Date 6/05/2022	Date 06/05/22	Date 06/05/22

Figure 4.3 CBR (%) and CBR swell (%) for 10% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	85% Black Cotton Soil mixed with 15% Bottom Ash	DATE AND TIME SOAKED	29/04/22
PURPOSE	Sub-grade	DATE AND TIME TESTED	3/5/2022
DEPTH	-	LAB.NO.	209/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows			
		BEFORE	AFTER	BEFORE	AFTER		
MOLD NUMBER		ERA20	5	56			
WEIGHT OF SOIL + MOLD	g	9670	10130	9880	10340		
WEIGHT OF MOLD	g	5879	5879	6178	6178		
WEIGHT OF SOIL	g	3791	4251	3702	4162		
VOLUME OF MOLD	cc	2076.686	2076.686	2100.142	2100.142		
WET DENSITY OF SOIL	g/cc	1.826	2.047	1.763	1.982		
DRY DENSITY OF SOIL	g/cc	1.460	1.413	1.437	1.359		

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows				
		BEFORE	AFTER		BEFORE	AFTER			
CONTAINER NUMBER		BX	TOP	Bottom	B-7	TOP	Bottom		
WET SOIL + CONTAINER	g	547.6	676.2	673.5	427.7	559.9	536.8		
DRY SOIL + CONTAINER	g	453.1	512.9	468.4	362.9	429.5	375.4		
WEIGHT OF WATER	g	94.5	163.3	205.1	64.8	130.4	161.4		
WEIGHT OF CONTAINER	g	75.9	71.5	79.0	77.5	81.7	77.8		
WEIGHT OF DRY SOIL	g	377.2	441.4	389.4	285.4	347.8	297.6		
MOISTURE CONTENT	%	25.1	37.0	52.7	22.7	37.5	54.2		
AVG. MOISTURE CONTENT	%		44.8		45.9				

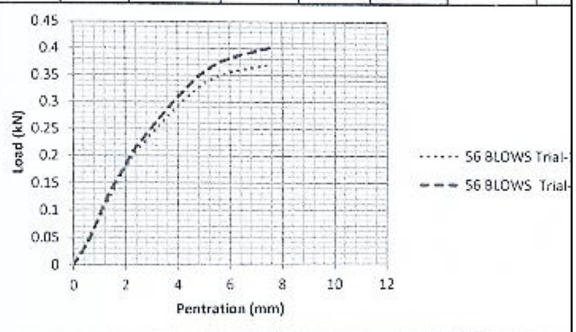
PENETRATION TEST DATA

Ring Factor 0.04483 kN/Div   Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows						
	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %			
0	0	0			0	0					
0.64	1.2	0.053796			1.1	0.049313					
1.27	2.5	0.112075			2.7	0.121041					
1.91	3.8	0.170354			3.9	0.174837					
2.54	4.8	0.215184		1.61	5	0.22415		1.68			
5.08	7.5	0.336225		1.69	8	0.35864		1.80			
7.62	8.3	0.372089			9	0.40347					
10.16											
12.7											

SWELL %

Height of Specimen (mm)		116	
No OF BLOWS		56 Trial-1	56 Trial-2
RDG (BEFORE SOAKING)		4	8
RDG(AFTER SOAKING)		1810	1711
PERCENT SWELL		15.52	14.64
AVERAGE PERCENT SWELL		15.08	



REMARKS:-	Bottom pentrated		
Lead Lab.Technician		Lead Research Engineer (Materials)	Team Leader
Name Marta G/Hana		Name Negash Abido	Name Asmera Nassir
Sign		Sign	Sign
Date 6/05/2022		Date 06/05/22	Date 06/05/22

Figure 4.4 CBR (%) and CBR swell (%) for 15% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	80% Black Cotton Soil mixed with 20% Bottom Ash	DATE AND TIME SOAKED	29/04/22
PURPOSE	Sub-grade	DATE AND TIME TESTED	3/5/2022
DEPTH	-	LAB.NO.	211/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows	
		BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		62		58	
WEIGHT OF SOIL + MOLD	g	10120	10690	9640	10320
WEIGHT OF MOLD	g	6431	6431	6106	6106
WEIGHT OF SOIL	g	3689	4259	3534	4214
VOLUME OF MOLD	cc	2120.175	2120.175	2091.272	2091.272
WET DENSITY OF SOIL	g/cc	1.740	2.009	1.690	2.015
DRY DENSITY OF SOIL	g/cc	1.463	1.402	1.459	1.406

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows		
		BEFORE	AFTER		BEFORE	AFTER	
CONTAINER NUMBER		Bz	TOP	Bottom	GW	TOP	Bottom
WET SOIL + CONTAINER	g	525.4	516.8	668.7	418.3	747.7	663.0
DRY SOIL + CONTAINER	g	454.6	398.0	483.9	370.9	571.8	456.6
WEIGHT OF WATER	g	70.8	118.8	184.8	47.4	175.9	195.4
WEIGHT OF CONTAINER	g	81.2	69.7	116.8	71.2	81.4	79.3
WEIGHT OF DRY SOIL	g	373.4	328.3	367.1	299.7	490.4	387.3
MOISTURE CONTENT	%	19.0	36.2	50.3	15.8	35.9	50.7
AVG. MOISTURE CONTENT	%		43.3		43.3		

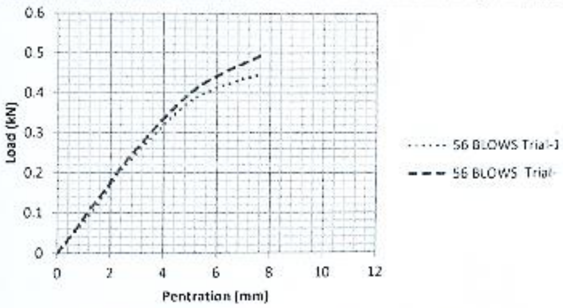
PENETRATION TEST DATA

Ring Factor 0.04483 kN/Div — Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %	DIAL RDG	LOAD (kN)	COR LOAD(kN)	CBR %
0	0	0			0	0		
0.64	1.1	0.049313			1.2	0.053796		
1.27	2.2	0.098626			2.5	0.112075		
1.91	3.5	0.156905			3.7	0.165871		
2.54	4.8	0.215184		1.61	5	0.22415		1.68
5.08	8.5	0.381055		1.91	9	0.40347		2.02
7.62	10	0.4483			11	0.49313		
10.16								
12.7								

SWELL %

Height of Specimen (mm)	116		
№ OF BLOWS		56 Trial-1	56 Trial-2
RDG (BEFORE SOAKING)		19	66
RDG(AFTER SOAKING)		1736	1941
PERCENT SWELL		14.81	16.18
AVERAGE PERCENT SWELL	15.49		



REMARKS:-	Bottom pentrated		
Lead Lab.Technician		Lead Research Engineer (Materials)	Team Leader
Name Marta G/Hana		Name Negash Abido	Name Asmera Nassir
Sign		Sign	Sign
Date 6/05/2022		Date 06/01/22	Date 06/10/22

Figure 4.5 CBR (%) and CBR swell (%) for 20% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	75% Black Cotton Soil mixed with 25% Bottom Ash	DATE AND TIME SOAKED	29/04/22
PURPOSE	Sub-grade	DATE AND TIME TESTED	3/5/2022
DEPTH	-	LAB.NO.	216/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows	
MOLD NUMBER		BEFORE	AFTER	BEFORE	AFTER
WEIGHT OF SOIL + MOLD	g	ERA60		ERA22	
WEIGHT OF MOLD	g	9780	10300	9870	10380
WEIGHT OF SOIL	g	5038	6038	6145	6145
VOLUME OF MULD	cc	3742	4262	3725	4235
WET DENSITY OF SOIL	g/cc	2111.268	2111.268	2114.473	2114.473
DRY DENSITY OF SOIL	g/cc	1.772	2.019	1.762	2.003
		1.485	1.412	1.471	1.390

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows		
		BEFORE	AFTER		BEFORE	AFTER	
CONTAINER NUMBER		B3	TOP	Bottom	BN	TOP	Bottom
WET SOIL + CONTAINER	g	499.4	620.2	516.6	553.2	560.5	605.8
DRY SOIL + CONTAINER	g	431.7	473.9	434.4	473.3	424.9	429.0
WEIGHT OF WATER	g	67.7	146.3	182.2	79.9	135.6	176.8
WEIGHT OF CONTAINER	g	81.6	71.4	67.2	68.9	74.8	71.7
WEIGHT OF DRY SOIL	g	350.1	402.5	367.2	404.4	350.1	357.3
MOISTURE CONTENT	%	19.3	35.3	49.6	19.8	38.7	49.5
AVG. MOISTURE CONTENT	%		43.0		44.1		

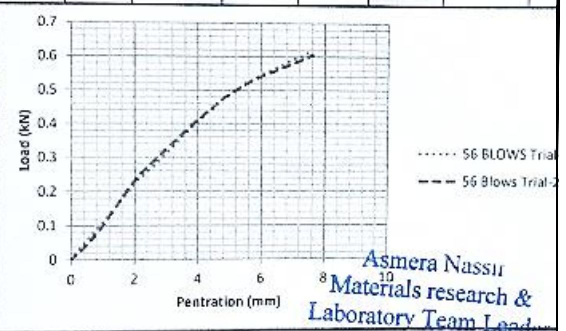
PENETRATION TEST DATA

Ring Factor 0.04483 KN/Div Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %
0	0	0			0	0		
0.64	1.6	0.071728			1.3	0.058279		
1.27	3.1	0.138973			3	0.13449		
1.91	4.7	0.210701			4.9	0.219667		
2.54	6	0.26898		2.01	6.3	0.282429		2.12
5.08	11	0.49313		2.47	11	0.49313		2.47
7.62	13.8	0.618654			13.5	0.605205		
10.16								
12.7								

SWELL %

Height of Specimen (mm)		116	
No OF BLOWS		56 Trial-1	56 Trial-2
RDG (BEFORE SOAKING)		146	8
RDG(AFTER SOAKING)		1791	1588
PERCENT SWELL		14.13	13.57
AVERAGE PERCENT SWELL		13.85	



Asmera Nassir
Materials research & Laboratory Team Lead

REMARKS:- Bottom pentrated		
Lead Lab.Technician	Lead Research Engineer (Materials)	Team Leader
Name Marta G/Hana	Name Negash Abido	Name Asmera Nassir
Sign	Sign	Sign
Date 6/05/2022	Date 06/05/22	Date 06/05/22

Figure 4.6 CBR (%) and CBR swell (%) for 25% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	70% Black Cotton Soil mixed with 30% Bottom Ash	DATE AND TIME SOAKED	6/5/2022
PURPOSE	Sub-grade	DATE AND TIME TESTED	10/5/2022
DEPTH	-	LAB.NO.	221/22

CALIFORNIA BEARING RATIO WORK SHEET

TEST MATHOD : ASTM - D1883

DENSITY DETERMINATION

4 DAY SOAKING CONDITION		56 Blows		56 Blows	
		BEFORE	AFTER	BEFORE	AFTER
MOLD NUMBER		ERA22		50	
WEIGHT OF SOIL + MOLD	g	9730	10330	9760	10330
WEIGHT OF MOLD	g	6145	6145	6106	6106
WEIGHT OF SOIL	g	3585	4185	3654	4224
VOLUME OF MOLD	cc	2114.473	2114.473	2091.272	2091.272
WET DENSITY OF SOIL	g/cc	1.695	1.979	1.747	2.020
DRY DENSITY OF SOIL	g/cc	1.416	1.399	1.468	1.437

MOISTURE DETERMINATION

4 DAYS SOAKING CONDITION		56 Blows			56 Blows		
		BEFORE	AFTER		BEFORE	AFTER	
CONTAINER NUMBER		B15	TOP	Bottom	C7	TOP	Bottom
WET SOIL + CONTAINER	g	460.8	552.9	412.2	404.8	488.2	462.6
DRY SOIL + CONTAINER	g	396.5	442.3	300.0	351.4	384.7	336.1
WEIGHT OF WATER	g	64.3	110.6	112.2	53.4	103.5	126.5
WEIGHT OF CONTAINER	g	71.1	117.3	79.2	71.2	68.5	75.0
WEIGHT OF DRY SOIL	g	325.4	325.0	220.8	280.2	316.2	261.1
MOISTURE CONTENT	%	19.8	34.0	50.8	19.1	32.7	48.4
AVG MOISTURE CONTENT	%		42.4		40.6		

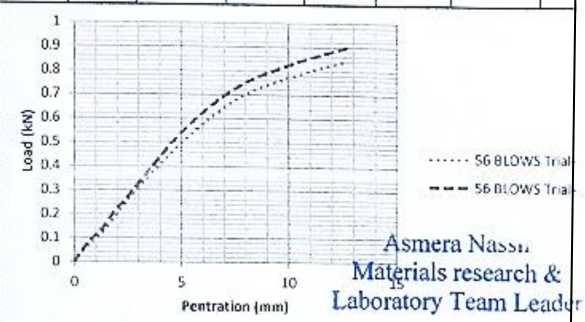
PENETRATION TEST DATA

Ring Factor 0.04483 KN/Div — Planger Area 1962.5

PENETRATION (mm)	56 Blows				56 Blows			
	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %	DIAL RDG	LOAD (KN)	COR LOAD(KN)	CBR %
0	0	0			0	0		
0.64	1.6	0.071728			1.8	0.080694		
1.27	2.7	0.121041			3	0.13449		
1.91	4.2	0.188286			4.8	0.215184		
2.54	5.9	0.264497		1.98	6.2	0.277946		2.08
5.08	11.2	0.502096		2.52	12.2	0.546926		2.74
7.62	15.2	0.681416			16.3	0.730729		
10.16	17.3	0.775559			18.5	0.829355		
12.7	18.8	0.842804			20.1	0.901083		

SWELL %

Height of Specimen (mm)	116	
No OF BLOWS	56 Trial-1	56 Trial-2
RDG (BEFORE SOAKING)	10	0.007
RDG(AFTER SOAKING)	1532	1625
PERCENT SWELL	13.09	13.98
AVERAGE PERCENT SWELL	13.54	



Asmera Nassir
Materials research &
Laboratory Team Leader

REMARKS:- Bottom pentrated

Lead Lab.Technician	Lead Research Engineer (Materials)	Team Leader
Name Marta G/Hana	Name Negash Abido	Name Asmera Nassir
Sign	Sign	Sign
Date 17/05/2022	Date 12/05/22	Date 12/05/22

Figure 4.7 CBR (%) and CBR swell (%) for 30% bottom ash

Appendix vi: Sieve diameter and size from corbel to clay

Sieve No	Size	Size (mm)
3 in		75.000
2 in		50.000
1-1⁄2-in		37.500
1 in	75 mm	25.000
3⁄4 in	50 mm	19.000
3⁄8 in	37.5 mm	9.500
No. 4	4.75 mm	4.750
No. 10	2.00 mm	2.000
No. 16	1.18-mm	1.180
No. 30	600-µm	0.600
No. 40	425 µm	0.425
No. 50	300-µm	0.300
No. 60	250 µm	0.250
No. 100	150 µm	0.150
No. 200	75 µm	0.075

Figure 5.1 Sieve diameter

	Inch	mm
Cobbles	> 3" sieve	> 75
Coarse gravel	3"sieve - 3/4"sieve	75.00 - 19.00
Fine gravel	3/4"sieve -No. 4 sieve	19.00 - 4.75
Coarse sand	No. 4 sieve - No. 10 sieve	4.75 - 2.00
Medium sand	No. 10 sieve - No. 40 sieve	2.00 - 0.425
Fine sand	No. 40 sieve - No. 200 sieve	0.425 - 0.075
Silt	No. 200 sieve - 0.005mm size sieve	0.075 - 0.005
Clay	< 0.005 mm	< 0.005

Figure 5.2 Size from corbel to clay

Appendix vii: Compaction (Maximum Dry density (MDD) verses Optimum Moisture Content (OMC) determination

ETHIOPIAN ROADS AUTHORITY

ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION		DATE SAMPLED	04/04/22
MATERIAL DESCRIBTION	Pure Black Cotton Soil	DATE TESTED	11/04/22
PURPOSE	Sub-grade	LAB.No.	162/22

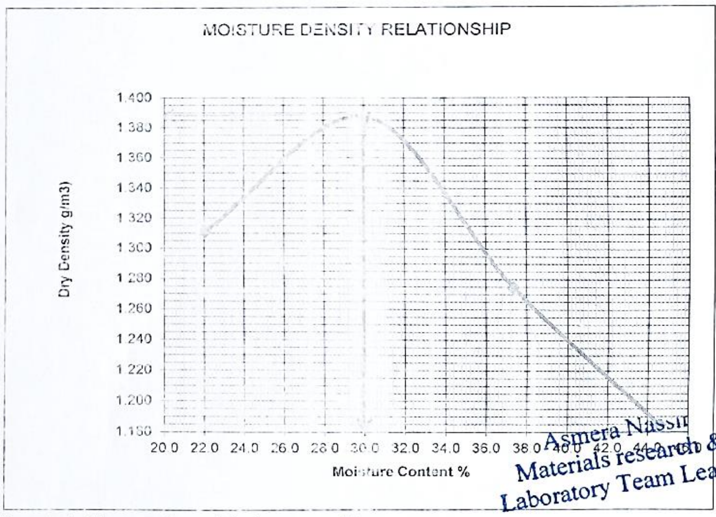
MODIFIED AASHTO DENSITY TEST

TEST METHOD : AASHTO T-180 METHOD D

DENSITY	TRIAL NUMBER	1	2	3	4			
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100			
	WATER ADDED (%)	150	300	450	600			
	WEIGHT OF SOIL + MOLD (g)	4654.4	4848	4797	4761.1			
	WEIGHT OF MOLD (g)	3152	3152	3152	3152			
	WEIGHT OF SOIL (g)	1502.4	1696	1645	1609.1			
	VOLUME OF MOLD (cc)	939.633	939.633	939.633	939.633			
	Wet DENSITY OF SOIL (g/cc)	1.599	1.805	1.751	1.712			NMC
MOISTURE	CONTAINER NUMBER	B2	BO	R-8	C-1			B-7
	WET SOIL + CONTAINER (g)	562.3	753.8	573.1	643.0			474.4
	DRY SOIL + CONTAINER g	475.5	595.5	435.7	466.2			416.0
	WEIGHT OF WATER g	86.8	158.3	137.4	176.8			58.4
	WEIGHT OF CONTAINER g	81.2	70.2	67.9	69.8			77.4
	WEIGHT OF DRY SOIL g	394.3	525.3	367.8	396.4			338.6
	MOISTURE CONTENT (%)	22.0	30.1	37.4	44.6			17.2
DRY DENSITY OF SOIL (g/cc)		1.310	1.387	1.275	1.184			

MDD :	1.388	g/cm3
OMC :	30.0	%

REMARK:-



Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name Marta G/Hana	Name :- Negash Abido	Name :- Asmera Nassir
Sign	Sign	Sign
Date 18/04/2022	Date 18/04/22	Date 18/04/22

Figure 6.1 MDD verses OMC at 0% bottom ash

ETHIOPIAN ROADS ADMINSTRATION
ROAD RESEARCH CENTER

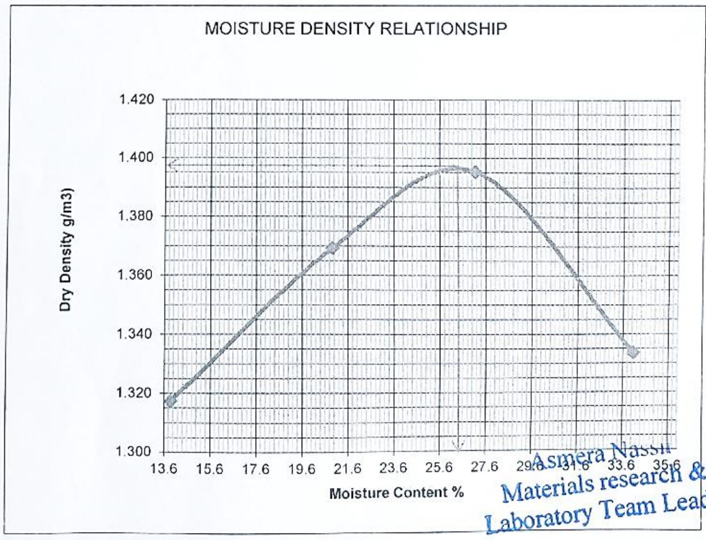
PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	4/4/2022
MATERIAL DESCRIBTION	95% Black Cotton Soil +5% Bottom Ash	DATE TESTED	19/04/22
PURPOSE	Sub-grade	LAB.No.	174/22

MODIFIED DENSITY TEST
TEST METHOD : ASTM D1557 Procedure A

		1	2	3	4			
DENSITY	TRIAL NUMBER	1	2	3	4			
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100			
	WATER ADDED (%)	75	200	325	450			
	WEIGHT OF SOIL + MOLD (g)	4567.2	4714	4825.6	4838.3			
	WEIGHT OF MOLD (g)	3152.6	3152.6	3152.6	3152.6			
	WEIGHT OF SOIL (g)	1414.6	1561.4	1673	1685.7			
	VOLUME OF MOLD (cc)	943	943	943	943			
	Wet DENSITY OF SOIL (g/cc)	1.500	1.656	1.774	1.788			NMC
MOISTURE	CONTAINER NUMBER	Z-8	B100	B10	B8			RF
	WET SOIL + CONTAINER (g)	501.3	640.6	720.0	559.5			481.3
	DRY SOIL + CONTAINER g	449.8	542.6	583.0	435.0			439.8
	WEIGHT OF WATER g	51.5	98.0	137.0	124.5			41.5
	WEIGHT OF CONTAINER g	78.8	74.4	78.5	69.4			68.0
	WEIGHT OF DRY SOIL g	371.0	468.2	504.5	365.6			371.8
	MOISTURE CONTENT (%)	13.9	20.9	27.2	34.1			11.2
DRY DENSITY OF SOIL (g/cc)		1.317	1.369	1.395	1.333			

MDD :	1.396	g/cm3
OMC :	26.4	%

REMARK:-



Asmera Nassir
Materials research &
Laboratory Team Leader

Materials Research and Laboratory Team

Lead Lab.Technician		Lead Materials Engineer	Team Leader
Name	Marta G/Hana	Name :- Negash Abido	Name :- Asmera Nassir
Sign		Sign	Sign
Date	3/05/2022	Date 03/05/22	Date 03/05/22

Figure 6.2 MDD verses OMC at 5% bottom ash

ETHIOPIAN ROADS AUTHORITY
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION		DATE SAMPLED	04/04/22
MATERIAL DESCRIBTION	90% Black Cotton Soil +10% Bottom Ash	DATE TESTED	12/04/22
PURPOSE	Sub-grade	LAB.No.	161/22

MODIFIED AASHTO DENSITY TEST

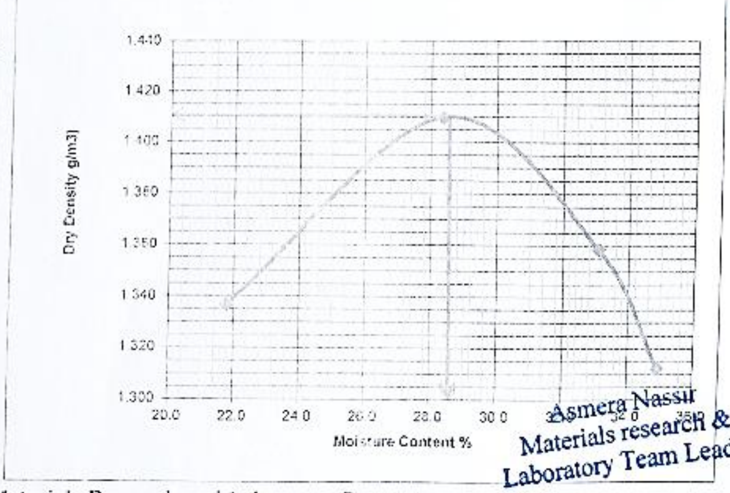
TEST METHOD : AASHTO T-180 METHOD D

DENSITY	TRIAL NUMBER	1	2	3	4			
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100			
	WATER ADDED (%)	200	325	450	575			
	WEIGHT OF SOIL + MOLD (g)	4681.3	4852.5	4850.7	4815.6			
	WEIGHT OF MOLD (g)	3152	3152	3152	3152			
	WEIGHT OF SOIL (g)	1529.3	1700.5	1698.7	1663.6			
	VOLUME OF MOLD (cc)	939.633	939.633	939.633	939.633			
	Wet DENSITY OF SOIL (g/cc)	1.628	1.810	1.808	1.770			NMC
MOISTURE	CONTAINER NUMBER	C-7	W-1	B-7	C-1			BO
	WET SOIL + CONTAINER (g)	635.0	562.7	525.0	502.1			385.0
	DRY SOIL + CONTAINER g	534.2	453.7	413.8	390.2			353.0
	WEIGHT OF WATER g	100.8	109.0	111.2	111.9			32.0
	WEIGHT OF CONTAINER g	71.3	69.5	77.7	69.8			70.1
	WEIGHT OF DRY SOIL g	462.9	384.2	336.1	320.4			282.9
	MOISTURE CONTENT (%)	21.8	28.4	33.1	34.9			11.3
DRY DENSITY OF SOIL (g/cc)		1.337	1.410	1.358	1.312			

MDD : 1.410 g/cm3

OMC : 28.6 %

REMARK:-



Asmera Nassir
Materials research &
Laboratory Team Leader

Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name Marta G/Hana	Name :- Negash Abido	Name :- Asmera Nassir
Sign	Sign	Sign
Date 18/04/2022	Date 18/04/22	Date 18/04/22

Figure 6.3 MDD verses OMC at 10% bottom ash

ETHIOPIAN ROADS AUTHORITY

ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION	-	DATE SAMPLED	04/04/22
MATERIAL DESCRIBTION	85% Black Cotton Soil mixed with 15% Bottom Ash	DATE TESTED	15/4/2022
PURPOSE	Sub-grade	LAB.No.	166/22

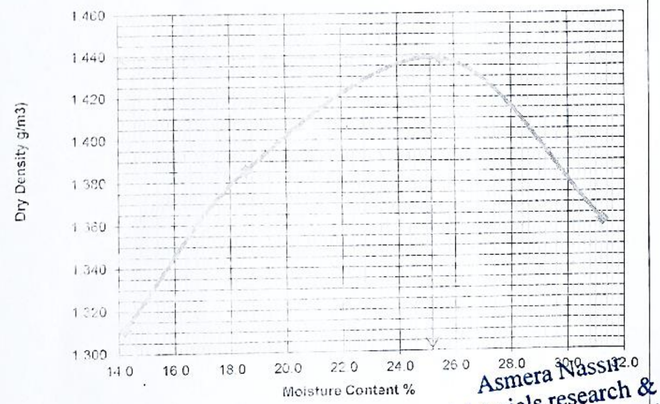
MODIFIED DENSITY TEST

TEST METHOD : ASTM D1557 Procedure A

DENSITY	TRIAL NUMBER	1	2	3	4			
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100			
	WATER ADDED (%)	75	200	325	450			
	WEIGHT OF SOIL + MOLD (g)	4565	4701.7	4855.1	4837.5			
	WEIGHT OF MOLD (g)	3152.6	3152.6	3152.6	3152.6			
	WEIGHT OF SOIL (g)	1412.4	1549.1	1702.5	1684.9			
	VOLUME OF MOLD (cc)	943	943	943	943			
	Wet DENSITY OF SOIL (g/cc)	1.498	1.643	1.805	1.787			NMC
MOISTURE	CONTAINER NUMBER	B-10	Z-8	B-3	GW			B-6
	WET SOIL + CONTAINER (g)	452.0	668.4	678.6	566.6			400.2
	DRY SOIL + CONTAINER g	405.5	576.1	557.2	448.7			376.6
	WEIGHT OF WATER g	46.5	92.3	121.4	117.9			23.6
	WEIGHT OF CONTAINER g	78.5	78.9	81.6	71.3			68.9
	WEIGHT OF DRY SOIL g	327.0	497.2	475.6	377.4			307.7
	MOISTURE CONTENT (%)	14.2	18.6	25.5	31.2			7.7
DRY DENSITY OF SOIL (g/cc)		1.311	1.386	1.438	1.361			

MDD :	1.438	g/cm3
OMC :	25.2	%

REMARK:-



Asmera Nassir
Materials research & Laboratory Team Leader

Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name martha Gllana	Name :- Negash Abido	Name :- Asmera Nassir
Sign	Sign	Sign
Date 19/04/2022	Date 19/04/22	Date 19/04/22

Figure 6.4 MDD verses OMC at 15% bottom ash

ETHIOPIAN ROADS AUTHORITY
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION		DATE SAMPLED	04/04/22
MATERIAL DESCRIBTION	80% Black Cotton Soil mixed with 20% Bottom Ash	DATE TESTED	15/4/2022
PURPOSE	Sub-grade	LAB.No.	165/22

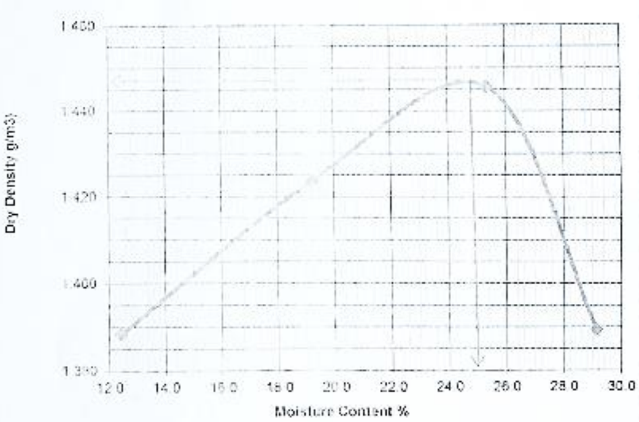
MODIFIED DENSITY TEST

TEST METHOD : ASTM D1557 Procedure A

DENSITY	TRIAL NUMBER	1	2	3	4			
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100			
	WATER ADDED (%)	100	200	300	400			
	WEIGHT OF SOIL + MOLD (g)	4624.5	4753.3	4861.5	4844.4			
	WEIGHT OF MOLD (g)	3152.6	3152.6	3152.6	3152.6			
	WEIGHT OF SOIL (g)	1471.9	1600.7	1708.9	1691.8			
	VOLUME OF MOLD (cc)	943	943	943	943			
	Wet DENSITY OF SOIL (g/cc)	1.561	1.697	1.812	1.794			NMC
MOISTURE	CONTAINER NUMBER	B-9	B-15	B-X	B-O			BN
	WET SOIL + CONTAINER (g)	519.1	576.0	594.9	512.9			333.8
	DRY SOIL + CONTAINER g	469.3	494.6	489.9	413.0			312.5
	WEIGHT OF WATER g	49.8	81.4	105.0	99.9			21.3
	WEIGHT OF CONTAINER g	63.7	71.5	75.9	70.1			68.2
	WEIGHT OF DRY SOIL g	405.6	423.1	414.0	342.9			244.3
	MOISTURE CONTENT (%)	12.4	19.2	25.4	29.1			8.7
DRY DENSITY OF SOIL (g/cc)		1.388	1.424	1.446	1.389			

MDD :	1.447	g/cm3
OMC :	25.0	%

REMARK:-



Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name Martha Gebane	Name :- Negash Abido	Name :- Asmera Nassir
Sign	Sign	Sign
Date 19/04/2022	Date 19/04/22	Date 19/04/22

Figure 6.5 MDD verses OMC at 20% bottom ash

ETHIOPIAN ROADS AUTHORITY

ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION		DATE SAMPLED	04/04/22
MATERIAL DESCRIPTION	75% Black Cotton Soil mixed with 25% Bottom Ash	DATE TESTED	19/4/2022
PURPOSE	Sub-grade	LAB.No.	175/22

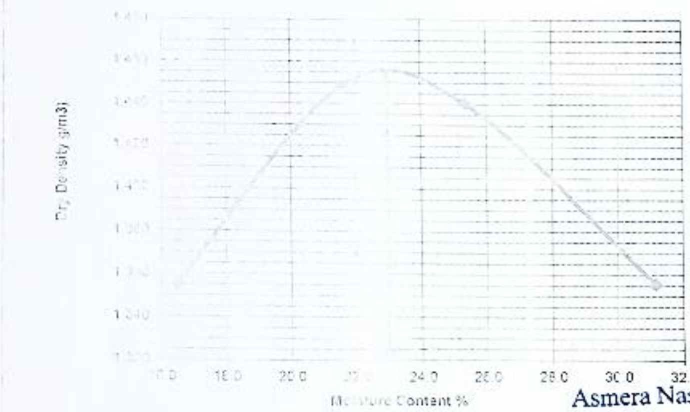
MODIFIED DENSITY TEST

TEST METHOD : ASTM D1557 Procedure A

DENSITY	TRIAL NUMBER		1	2	3	4			
	WEIGHT OF SAMPLE	(g)	2100	2100	2100	2100			
	WATER ADDED	(%)	150	250	350	450			
	WEIGHT OF SOIL + MOLD (g)		4641.1	4812.9	4855.1	4829.3			
	WEIGHT OF MOLD (g)		3152.6	3152.6	3152.6	3152.6			
	WEIGHT OF SOIL (g)		1488.5	1661.3	1702.5	1676.7			
	VOLUME OF MOLD (cc)		943	943	943	943			
	Wet DENSITY OF SOIL (g/cc)		1.578	1.762	1.805	1.778			NMC
MOISTURE	CONTAINER NUMBER		BL	BG	Z-3	[illegible]			B-7
	WET SOIL + CONTAINER	(g)	539.9	513.0	660.6	561.0			468.3
	DRY SOIL + CONTAINER	g	474.4	438.4	543.0	449.1			434.8
	WEIGHT OF WATER	g	65.5	79.6	117.6	111.9			33.5
	WEIGHT OF CONTAINER	g	76.8	70.2	73.9	77.4			68.2
	WEIGHT OF DRY SOIL	g	397.6	368.2	469.1	[illegible]			366.6
	MOISTURE CONTENT (%)		16.5	[illegible]	25.3	[illegible]			9.1
DRY DENSITY OF SOIL (g/cc)			1.355	[illegible]	1.440	1.356			

MDD :	1.456	g/cm3
OMC :	23.0	%

REMARK:-



Asmera Nassir
Materials research &
Laboratory Team Leader

Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name Marta G/Hana	Name :- Negash A	Name :- Asmera Nassir
Sign	Sign	Sign
Date 26/04/22	Date 26/04/22	Date 26/04/22

Figure 6.6 MDD verses OMC at 25% bottom ash

ETHIOPIAN ROADS AUTHORITY
ROAD RESEARCH CENTER

PROJECT	Stabilization of Black Cotton Soil Using Bottom Ash	SAMPLE TAKEN FROM	Gellan
SAMPLING STATION		DATE SAMPLED	04/04/22
MATERIAL DESCRIBTION	70% Black Cotton Soil mixed with 30% Bottom Ash	DATE TESTED	20/4/2022
PURPOSE	Sub-grade	LAB No.	172/22

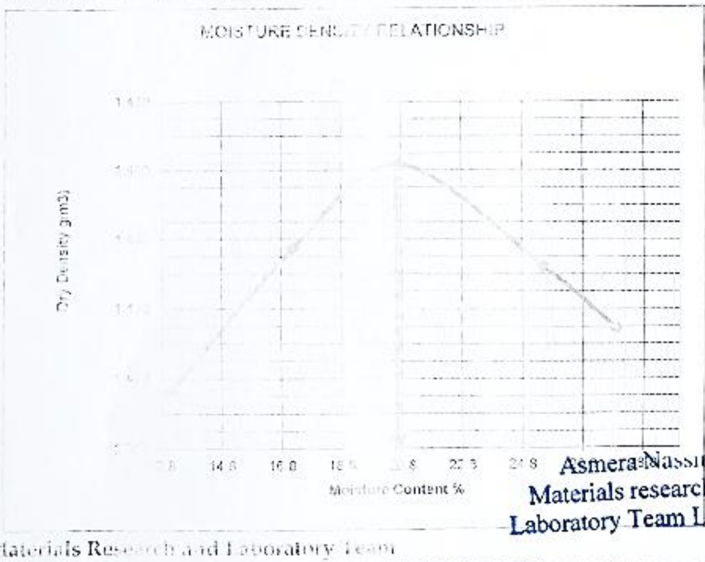
MODIFIED DENSITY TEST

TEST METHOD : ASTM D1557 Procedure A

DENSITY	TRIAL NUMBER	1	2	3	4	5		
	WEIGHT OF SAMPLE (g)	2100	2100	2100	2100	2100		
	WATER ADDED (%)	100	175	250	325	400		
	WEIGHT OF SOIL + MOLD (g)	4641.5	4740.8	4816.8	4847.4	4858.5		
	WEIGHT OF MOLD (g)	3152.6	3152.6	3152.6	3152.6	3152.6		
	WEIGHT OF SOIL (g)	1488.9	1588.2	1664.2	1694.8	1705.9		
	VOLUME OF MOLD (cc)	943	943	943	943	943		
	Wet DENSITY OF SOIL (g/cc)	1.579	[illegible]	1.765	1.797	1.809		NMC
MOISTURE	CONTAINER NUMBER	BO	[illegible]	B-L	[illegible]	B-X		R-F
	WET SOIL + CONTAINER (g)	591.9	[illegible]	592.0	[illegible]	653.8		477.9
	DRY SOIL + CONTAINER g	531.5	[illegible]	502.5	458.8	527.8		449.8
	WEIGHT OF WATER g	60.4	73.2	88.5	[illegible]	126.0		28.1
	WEIGHT OF CONTAINER g	70.1	83.4	76.8	78.8	75.9		67.8
	WEIGHT OF DRY SOIL g	461.4	[illegible]	425.7	[illegible]	451.9		382.0
	MOISTURE CONTENT (%)	13.1	[illegible]	20.7	[illegible]	27.9		7.4
DRY DENSITY OF SOIL (g/cc)		1.396	1.437	1.462	1.432	1.415		

MDD :	1.461	g/cm3
OMC :	20.6	%

REMARK:-



Asmera Nassir
Materials research &
Laboratory Team Leader

Materials Research and Laboratory Team

Lead Lab.Technician	Lead Materials Engineer	Team Leader
Name Marta G/Hana	Name : Negash Alemu	Name : Asmera Nassir
Sign	Sign	Sign
Date 26/04/22	Date 26/04/22	Date 26/04/22

Figure 6.7 MDD verses OMC at 30% bottom ash

Appendix viii: Major and minor oxides of bottom ash (Yigzaw, 2021)

	GEOLOGICAL SURVEY OF ETHIOPIA	Doc.Number: GLD/F5.10.2	Version No: 1
	GEOCHEMICAL LABORATORY DIRECTORATE		Page 1 of 1
Document Title:	Complete Silicate Analysis Report	Effective date:	May, 2017

Issue Date: -29/05/2020

Customer Name: -Tefera Getie Yigzaw.

Request No:- GLD/RQ/305/20

Report No:- GLD/RN/340/20

Sample type:- Soil

Sample Preparation: - 200 Mesh

Date Submitted: - 17/03/2020

Number of Sample:- Two (2)

Analytical Result: In percent (%) Element to be determined Major Oxides & Minor Oxides

Analytical Method: LiBO, FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
T1B	45.50	9.81	9.42	17.94	2.32	3.94	2.56	0.22	3.95	0.40	0.31	3.82
T2F	28.24	6.80	3.79	26.76	2.20	4.90	10.40	0.20	1.65	0.26	0.17	13.85

Note: - This result represent only for the sample submined to the laboratory.

Analysts

Yirgalem Abriham

Tizita Zemene

Nigist Fikadu

Checked By

for Yohannes Getachew

Approved By

Gosa Haile

Quality Control

Negash Worku

Appendix ix: Base metal analysis of bottom ash (Yigzaw, 2021)

		Doc.Number	
	GEOLOGICAL SURVEY OF ETHIOPIA	GLD/F5.10-2	Version No: 1
	GEOCHEMICAL LABORATORY DIRECTORATE		Page 1 of 1
Document Title:	Base Metal Analysis Report	Effective Date:	May, 2017

Issued Date: -25/03/2020

Request No: - GLD/RN/304/20

Originator: - Tefera Getie Yigzaw.

Report No: - GLD/TR/239/20

Sample type: - Soil

Sample Preparation: - 200 Mesh

Date Submitted: -17/03/2020

Number of Sample: - One (1)

Analytical Result: - ppm

Analytical Method: - Four acid attack (HCl, HNO_3, $HClO_4$, and HF) and AAS finish.

Collector's Code	Cu(ppm)	Zn(ppm)	Pb(ppm)	Co(ppm)	Ni(ppm)
T1B	1,120.00	2,540.00	262.20	184.20	155.20
T2F	380.00	42.00	1,077.40	144.20	127.40

-*Note:* - This result represent only for the sample submitted to the laboratory

- *ppm: parts* per million
- *Cu* for Copper, *Zn for* Zinc, Pb for Lead, Co for Cobalt and Ni for Nickel

Analysts

Mulumebet Luelseged

Almaz Eticha

Habtamu Alehegn

Checked By

Dessalew Bitew

Quality Control

Negash Worku

Geological Survey ... Central Geological ...