JIMMA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
JIMMA INSTITUTE OF TECHNOLOGY
FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING
HIGHWAY ENGINEERING STREAM

STABILIZATION OF BLACK COTTON SOIL USING BAGASSE ASH MIXED WITH CEMENT FOR SUBGRADE IN ROAD CONSTRUCTION: A CASE STUDY ON KAFTA HUMERA

A final thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for Degree of Master of Science in Civil Engineering (Highway Engineering)

By:

Gidey Tsegay

March, 2020

JIMMA ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

STABILIZATION OF BLACK COTTON SOIL USING BAGASSE ASH MIXED WITH CEMENT FOR SUBGRADE IN ROAD CONSTRUCTION: A CASE STUDY ON KAFTA HUMERA

By:

Gidey Tsegay

A final thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for Degree of Master of Science in Civil Engineering (Highway Engineering)

Advisor: - Engr. Elmer C. Agon (Ass. Prof.)

Co- Advisor: - Engr. Yibas Mamuye (MSc.)

March, 2020

JIMMA ETHIOPIA

JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

STABILIZATION OF BLACK COTTON SOIL USING BAGASSE ASH MIXED WITH CEMENT FOR SUB GRADE IN ROAD CONSTRUCTION

By:

Gidey Tsegay

APPROVED BY BOARD OF EXAMINERS:

1. ______________________ ________________ _____/____/______
   External Examiner Signature Date
2. ______________________ ________________ ____/____/______
   Internal Examiner Signature Date

3. _______________________ _______________ ____/____/_____
   Chairman of Examiner Signature Date

4. Engr. Elmer C. Agon, Ass. Prof _____________ ____/____/____
   Main Advisor Signature Date

5. Engr. Yibas Mamuye (MSc) ____________ ____/____/_____
   Co – Advisor Signature Date

DECLARATION

I, the undersigned, declare that this thesis entitled “Stabilization of black cotton soil using bagasse ash mixed with cement for sub grade in road construction” is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for thesis have been duly acknowledged.

Candidate: Gidey Tsegay

Signature: ___________

As Master research Advisors, we hereby certify that we have read and evaluated this MSc research prepared under our guidance, by **Gidey Tsegay** entitled: “Stabilization of black cotton soil using bagasse ash mixed with cement for sub grade in road construction”.

We recommend that it can be submitted as fulfilling the MSc Thesis requirements.

1. Engr. Elmer C. Agon, Ass. Prof	_____________	____/____/____
Main Advisor	Signature	Date
2. Engr. Yibas Mamuye (MSc)	___________	____/____/_____
Co – Advisor	Signature	Date

ACKNOWLEDGEMENT

First of all, I would like to gratefully acknowledge the almighty God and his holly mother for their guidance through.

I express sincere appreciation to my Advisor Engr. Elmer C. Agon, (Ass. Prof.) for his whole hearted advice, encouragement, and invaluable professional guidance throughout this research. I would like to thank him for his help full comments, suggestions and giving me the chance to work with him.

I would like to express my genuine and heartfelt gratitude and deep appreciation to my co advisor, Engr. Yibas Mamuye (MSc) for his encouragement, patient guidance, and constant support throughout the study period of my research.

I would like to acknowledge to Sur Construction Plc. Lot Adiremets – Adihirdi - Beahker Laboratory staff and assistants for their patient discussion, support, and valuable cooperation throughout my research.

Finally, I want to express my deep gratitude to my family for providing me authentic support and continuous encouragement throughout my study. This achievement would not have been possible without them.

ABSTRACT

Expansive soils occurring in arid and semi-arid climate regions of the world cause serious problems on civil engineering structures. Such soils swell when exposed to water and shrink when they dry out. Several attempts are being made to control the swell-shrink behavior of these soils. The swell and shrinkage distinctiveness of expansive soil causes significant damage to structures such as buildings and pavements. The above problems are extensively occurring in Ethiopia. The aerial coverage of expansive soils in Ethiopia is estimated to be 9.9 million hectare. Since most soil which is found in Humera town are plastic clay soils. These clays are a consequence for expansive and unstable soil. As a result, they make pavement structure failure.

The general objective of this research was to assess on stabilizing black cotton soil using bagasse ash mixed with cement for sub grade in road construction. In this study moisture content, Atterberg Limits testing, particle size distribution, classification, free swell index, specific gravity, compaction and uncured and 3, 7 and 14 days cured CBR was determined. The research design was followed the experimental type of study which begins by collecting samples. The sampling technique used for this research was a purposive sampling which is non–probability method. Two black cotton soil samples were taken from three test pit for the study by observation and free swell index tests at a depth of 1.50 m to remove organic matter. Bagasse ash was taken from Arjjo Dedessa sugar factory.

The result of physical properties of Adiremets – Adihidi – Beahker Road project soil sample has plastic index 53.77%, free swell index 80%, and CBR value 0.98 % for uncured and 1.14%,1.37% and 1.72% for 3,7 and 14 days cured soil respectively. Similarly, Humera Airport soil sample has plastic index 63 %, free swell index 100 % and CBR value 0.81% for uncured and 1.11%, 1.30% and 1.65% for 3, 7 and 14days cured soil sample respectively. Since both the given soil samples were found with high degree of expansion, stabilization was made with different mix-ratio.

As the amount of cement ratio increases CBR strength also increase but amount of cement decreasing with increasing bagasse ash did not increase strength. 8% cement + 2% bagasse ash is an optimum ratio which has the plastic index 8%, free swell index 20%, and CBR value 10.7 % for uncured and 20.48%,28.93% and 40.95% for 3,7 and 14 days cured soil respectively. Similarly, Humera Airport soil sample has plastic index 12%, free swell index 23% and CBR value 10.1% for uncured and 19.83%, 28.6% and 38% for 3, 7 and 14days cured soil sample respectively. All the laboratory result was compared with ERA specifications. Additional curing time effect on all geotechnical laboratory tests should be performed. The high amount of bagasse ash should add to cement to get relevant strength and to reduce the cost of highway construction.

Keywords: *Black cotton soil, bagasse ash, Ordinary Portland cement stabilization, Sub grade Strength.*

TABLE OF CONTENTES

Contents

LIST OF TABLES

LIST OF FIGURES

ACRONOMY

AASHTO	American Association State Highway and Transportation Officials
AABRP	Adiremets – adihirdi – Beahker Road Project
ASR	Alkali-Silica Reaction
ASTM	American Society for Testing and Materials
CAH	Calcium Aluminate Hydrate
CBR	Californian bearing Ratio
CEC	Cation Exchange Capacity
CSA	Central Statistical Agency
CSH	Calcium Silicate Hydrates
ERA	Ethiopian Roads Authority
FSI	Free Swell Index
Gs	Specific Gravity
GSE	Geological Survey of Ethiopia
HAP	Humera Airport
IS	Indian Standard
JU	Jimma University
LL	Liquid Limit
M_3	Meter Cubic
Max	Maximum
Min	Minimum
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
UCS	Unified Compressive Strength
USCS	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 Background

Soils susceptible to expansion in volume are known as expansive soils. Such soils usually have high clay content. Expansive soils are the soils which swell significantly when come in contact with water and shrink when the water squeezes out. Because of this alternate swell- shrink behavior of the soil, damages occur to different civil engineering structures founded on them.

Expansive soils can be found anywhere in the world, but they are confined to semi-arid and arid regions. These areas are naturally characterized by marked dry and wet seasons with low rainfall, poor drainage, and exceedingly great heat. The climate condition is such that the annual evapotranspiration exceeds the precipitations (Chen, F.H., 1988).

Expansive soils are generally found in the highlands and low lands of the Ethiopia. These soils undergo volumetric changes upon wetting and drying, thereby causing ground heave and settlement problems. This characteristic causes considerable construction defects if not adequately taken care of. Expansive soils in construction sites have the significant influence on planning, structural design, construction, maintenance costs and performance, especially for shallow foundation structures (Reshid,M.,2014).

The swell and shrinkage distinctiveness of expansive soil causes significant damage to structures such as buildings and pavements. This damage can be attributed to moisture fluctuations caused by seasonal variations. Volumetric changes weaken the subgrade by inducing cracking which metes out damage to the overlying structures. A vast majority of the expansive soils are montmorillonite-rich clays, over consolidated clays and shales (Nelson and Miller, 1992).

Expansive soils are also referred to as ‟black cotton soil" in some parts of the world. They are so named because of their suitability for growing cotton. Black cotton soils have varying colors ranging from light gray to dark gray and black. The mineralogy of this soil is dominated by the presence of montmorillonite which is characterized by large volume change from wet to dry seasons and vice versa. Deposits of black cotton soil in the field show a general pattern of cracks during the dry season of the year. In some cases the cracks are seen to extend to as deep as 1.5m (Teferra, A., and Leikun, M., 1999).

Pavement design is based on the premise that minimum specified structural quality will be achieved for each layer of material in the pavement system. Each layer must resist shearing, avoid excessive deflections that cause fatigue cracking within the layer or in overlying layers, and prevent excessive permanent deformation. As the quality of a soil layer is increased, the ability of that layer to distribute the load over a greater area increases so that a reduction in the required thickness of the soil and surface layers may be permitted. Commonly, improvement attained from soil stabilization can be summarized as (Guyer, J. P.2011). Frequently used methods of stabilizing soils are stabilization by compaction (mechanical) or stabilization by chemical additives.

The over dependence on industrially manufactured soil improving additives (cement, lime etc.) have kept the cost of construction of stabilized road financially high. This hitherto have continued to deter the underdeveloped and poor nations of the world from providing accessible roads to meet the need of their rural dwellers who constitute large percentage of their population which are mostly rural farmers (Meron, W.2013).

The need to bring down the growing cost of soil stabilizers and the cost of waste disposal has leads to intense global research towards economic utilization of wastes for engineering purposes. The safe disposal of industrial and agricultural waste products demands urgent and cost effective solutions because of the debilitating effect of these materials on the environment and to the health hazards that these wastes constitute (Osinubi, K.J.2007).

Thus, the possible use of agricultural waste, such as bagasse ash, will considerably reduce the cost of construction and as well as reduce or eliminate the environmental hazards caused by such waste. Bagasse ash is an agricultural waste obtained from milling of sugarcane. In Ethiopia currently with sugar production of about 300,000 tons, the bagasse ash potential is about 72,000 tons annually. When the five years plan come into reality there is going to be 0.94 million tons of bagasse ash generated annually. Meanwhile, the ash from bagasse has been categorized under pozzolana with about 1.78% calcium oxide (CaO), 5.78% iron oxide (Fe_2O_3), 1.23% magnesium oxide (MgO), 65.58% silicon oxide (SiO_2) and 5.78% aluminum oxide (Al_2O_3) (Hailu, B., 2011).

The utilization of this pozzolana as a replacement for traditional stabilizers, such as cement and lime, will go a long way in actualizing the dreams of most developing countries of scouting for cheap and readily available construction materials. Bagasse ash has been used in concrete as a partial replacement material for cement (Hailu, B., 2011;Chusilp, N., et.al, 2009).

1.2 Statement of the problem

Expansive soils occur in many parts of the world. However, the problem of expansion and shrinkage is associated with high moisture changes. Hence, it is restricted in areas where the seasonal variation in climatic condition is high. The large volume change with the periodic cycle of wetting and drying can cause extensive damages in civil engineering infrastructures; mainly on small buildings, shallow foundation and other lightly loaded structures including roads and airport pavements, pipelines etc. (Chen, F.H., 1988; Gourley, C., et.al, 1993; Nelson, D.J. and Miller, J.D., 1992).

The above problems are extensively occurring in Ethiopia. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres (Lyon associates, 1971; as cited by Nebro, D., 2002). They are widely spread in the central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. Also, areas like Mekele, Gambella and Humera are covered by expansive soil.

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include the blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil (Guyer, J. P. 2011).

Since most soil which is found in North West Ethiopia in Humera Town are expansive soils as it shown from the map of distribution of expansive soil in Ethiopia. These soils are a consequence for black cotton and unstable subgrade soil. As a result, they make pavement structure failure. The aim of this study is to investigate the engineering properties of stabilizing black cotton soil using bagasse ash mixed with cement for subgrade construction.

1.3 Research Question

1. What are the physical properties of black cotton soils?

2. What are the effects of bagasse ash-cement on the engineering properties of the stabilized black cotton soil?

3. What are the effects of curing on CBR value of the stabilized soil?

4. Which of the stabilized black cotton soil satisfied the standard specification?

1.4 Objectives of the study

1.4.1 General objective

The general objective of this research was to asses on stabilizing black cotton soil using bagasse ash mixed with cement for sub grade construction.

1.4.2 Specific objectives

To support the above general objective, the following specific objectives should be sought:

- ✓ To identify the physical properties of black cotton soil.
- ✓ To determine the effect of bagasse ash-cement on the engineering properties of the stabilized black cotton soil.
- ✓ To determine the effects of curing on CBR value of the stabilized soil.
- ✓ To compare the laboratory result with standard specification.

1.5 Significance of the Study

The production of traditional stabilizers, such as cement and lime, is environmental unfriendly processes. So it is important to find another option which is environmentally friendly and Cost advantage. The following are some significant of the study:

- ➢ Cost saving Bagasse ash is typically by far cheaper than traditional stabilizers such as cement and lime.
- ➢ The extraction of substantial amounts of non-renewable natural resources for road construction creates significant damaging impacts on the local environment and its inhabitants;
- ➢ Waste management can be done economically.
- ➢ The ongoing establishment of huge sugarcane factories in the country.

Therefore, using bagasse ash for improving engineering properties of the soils is an economical solution for Ethiopia as it is available in large quantity.

1.6 Scope of the study

This study was supported by different types of literatures and a series of laboratory experiments. However, the findings of the research are limited to one soil sample considered in this research which is black cotton soil. The results were also specific to the type of additives was used and test procedures that was adopted in the experimental work. Therefore, findings should be considered indicative rather than definitive for field applications.

CHAPTER TWO
LITERATURE REVIEW

2.1 Expansive Soils

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. Expansive soils cause more damage to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods. It has been reported that the damage caused by these soils contribute significantly to the burden that the natural hazard pose on the economy of countries where the occurrence of these soils is significant (Nelson, D.J., and Miller, J.D., 1992).

The most well-known example of expansive soils is the black cotton soil which is dark grey to black in color and the name originated from India where locations of these soils are favorable for growing cotton. Black cotton soils are inorganic clays of medium to high compressibility. They are characterized by high shrinkage and swelling properties. Because of their high swelling and shrinkage characteristics, black cotton soils have been a challenge to the highway engineers. The Black cotton soil is very hard when dry, but loses its strength completely when in wet condition (NG.Pulling, 2005).

Black cotton soils can be found anywhere in the world, but they are confined to semi-arid and arid regions. These areas are naturally characterized by marked dry and wet seasons with low rainfall, poor drainage, and exceedingly great heat. The climate condition is such that the annual evapo-transpiration exceeds the precipitations (Chen, F.H., 1988). Ethiopia is amongst the list of countries where the occurrence and spatial distribution is recognized as significant

Black cotton soils owe their specific properties to the presence of swelling clay minerals, mainly montmorillonite. As a result of the wetting and drying, massive expansion and contraction of the clay minerals takes place. Contraction leads to the formation of the wide and deep cracks. These cracks can be wide enough to make the terrain treacherous for animals (NG.Pulling, 2005).

2.1.1 Source of Expansive Soils

The origin of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, expands. Variations in the conditions and processes may also form other clay minerals, most of which are non-expansive. The conditions or processes, which determine the clay mineralogy, include the composition of the parent material and degree of physical and chemical weathering to which the materials are subjected.

A. Parent Material

The constituents of the parent material during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time (Chen, F.H., 1988).

The parent materials that can be associated with expansive soils are classified into two groups. The first group comprises the basic igneous rocks and the second group comprises the sedimentary rocks that contain montimorillonite as a constituent (Chen, F.H., 1988).

The basic igneous rocks are comparatively low in silica, generally about 45 to 52 percent. Rocks that are rich in metallic base such as the pyroxenes, amphiboles, biotite and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glasses (Chen, F.H., 1988).

The sedimentary rocks that contain montimorillonite as constituent include shale and clay stones. Limestone and marls rich in magnesium can also weather to clay. These parent materials contain varying amounts of volcanic ash and glass, which can subsequently be weathered to montimorillonite. The volcanic eruptions sent up clouds of ash, which fell on the continents and sea. Some of fine grained sediments which accumulated to form these rocks also contain montimorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas (Chen, F.H., 1988).

The formation of morocntmorillonite was probably the weathering and erosion in the highlands and carried by streams to the coastal plains. And volcanic eruptions sending up clouds of ash felt on the plains and the seas with the ashes to be altered to morocntmorillonite (Chen, 1998).

The Presence of montmorillonite clay in these soils imparts them high swell–shrink potentials. Low rainfall has hindered the weathering of the active Montmorillonite mineral into low active clay types such as Illite and kaolinite. Further, the rainfall has not been sufficient to leach the clay particles far enough so that the overburden pressure can control the swell (Amer Ali-Rawas and matteus, 2006). The second group comprises of sedimentary rocks. The rock comprises of smectite (montmorillonite) as a constituent and breaks down physically to form expansive soils. Smectite (montmorillonite) is one of the main sources of clay materials that form expansive soils (Chen, 1998).

B. Weathering and Climate

The weathering process by which clay is formed includes physical, biological and chemical process. The most important weathering process responsible for the formation of montmorillonite is the chemical weathering of parent rock mineral. The parent material generally consists of ferromagnesium mineral, calcic feldspars, volcanic glass, volcanic rocks and volcanic ash. The formation is aided in alkaline environment, presence of magnesium ion and lack of leaching. Such condition is favorable in semi-arid regions with relatively low rain fall or seasonal moderate rainfall particularly where evaporation exceeds precipitation. Under these conditions enough water is available for the alteration process but the accumulated cations will not be removed by rain water (Chen, F.H., 1988).

2.1.2 Distribution of Expansive Soil

Expansive soils are widespread in African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria etc. In other parts of the world case of expansive soils have been widely reported in countries like USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela etc. (Teferra, A., and Leikun, M., 1999).

The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres (Lyon associates, 1971; as cited by Nebro, D., 2002). They are widely spread in central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis Gohatsion, Addis-Modjo are covered by expansive soils. Also areas like Mekele and Gambella are covered by expansive soil. The distributions are shown in Figure 2.1 (Tilahun, D., 2004; Teklu, D., 2003).

Figure 2. 1Distribution of expansive soil in Ethiopia {Tilahun, D., 2004; Teklu, D., 2003)

2.1.3 Identification of Expansive Soils

Investigation of expansive soils generally consists of two important phases. The first is the visual identification and recognition of the soil as expansive and the second is sampling and measurement of material properties to be used as the basis for design. The theme of this topic is to discuss different ways that are commonly used to identify expansive soils.

A. **Field Identification**

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include (Chen, F.H., 1988; Nelson, D.J., and Miller, J.D., 1992):

- Usually have a color of black or grey.
- Wide or deep shrinkage cracks.
- High dry strength and low wet strength.
- Stickiness and low traffic ability when wet.
- Cut surfaces have a shiny appearance.
- Appearance of cracks in nearby structures.

Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature.

B. **Laboratory Identification**

Laboratory identification of expansive soils can be categorized into mineralogical, indirect and direct methods.

a) Mineralogical Identification

Clay mineralogy is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques. The techniques that can be used are (Chen, F.H., 1988; Nelson, D.J. and Miller, J.D., 1992):

- ✓ X-ray diffraction
- ✓ Differential thermal analysis
- ✓ Dye adsorption
- ✓ Chemical analysis
- ✓ Electron microscope resolution

But these methods are not suitable for routine tests because of the following reason;

- They are time consuming;

- They require expensive test equipment; and
- The results can only interpreted by specially trained technicians.

b) **Indirect Methods**

In this method simple soil property tests can be used for the evaluation of swelling potential of expansive soils. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soils. Such tests may include (Chen, F.H., 1988; Nelson, D.J., and Miller, J.D., 1992):

I. Atterberg Limits

In this method, measurements of the atterberg limits of the soil are conducted for identification of all soils and provide a wide acceptable means of rating. Especially when they are combined with other tests they can be used to classify expansive soils. The relation between the swelling potential of clays and the plasticity index is shown in Table 2.1 below.

Table 2. 1: Relation between the swelling potential of clays and the plasticity index

(Chen, F.H., 1988)

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

While it may be true that high swelling soil will manifest high index property, the converse is not true (Chen, F.H., 1988).

II. **Free Swell Tests**

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. Experiments indicated that a good grade of high swelling commercial bentonite will have a free swell of from 1200 to 2000 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings. The free swell percentage can be computed using Equation (2.1) from the relationship between initial and swelled volume.(Chen, F.H., 1988; Nelson, D.J., and Miller, J.D., 1992; Teferra, A., and Leikun, M., 1999).

Free swell (%) = $\frac{vf - vi}{vi}$ 2.1

Where:

V_i = initial volume

V_f =final volume

III. **Free Swell Index**

Free swell index is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay. The procedure involves in taking two oven dried soil samples passing through 425μm sieve, 10cc each were placed separately in two 100ml graduated soil sample. Distilled water was filled in one cylinder and kerosene in the other cylinder up to 100ml mark. The final volume of soil is computed after 24hours to calculate free swell index. The free swell index is then calculated using Equation (2.2). (Amer, A., and Mattheus, F.A., 2006)

Free swell Index = $\frac{vw - vk}{vk}$*100……………………………………………………………2.2

Where

V_w = final volume in water

V_k = final volume in kerosene

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

Table 2. 2: Degree of expansion and differential free swell index

(Ranjan, G., and Rao, A.S.R., 2002)

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

IV. Free Swell Ratio test

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

Free swell ratio = $\frac{vw}{vk}$*100……………………………………………………………2.3

Where

V_w = final volume in water

V_k = final volume in kerosene

The relation between the degree of expansion and differential free swell ratio is given in Table

Table 2. 3: Classification of Soils based on free swell ratio (Sridharan and Prakash 2004)

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

V. Cation Exchange Capacity (CEC)

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

Table 2. 4: Typical CEC values of basic clay minerals after Mitchell, 1976(Nelson, D.J., and Miller, J.D., 1992)

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

c) Direct Methods

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

2.1.4. Classification of Expansive Soils

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

2.1.4.1. Classification Using General Methods

The most widely used general classification systems are:

i. AASHTO Classification

As shown on Table 2.5 soils rated A-6 or A-7 by AASHTO can be considered potentially expansive (Nelson, D.J., and Miller, J.D., 1992).

Table 2. 5: AASHTO soil classification chart

General classification	Granular Materials (35 percent or less of total sample passing No. 200)							Silt-clay Materials (More than 35 percent of total sample passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis percent passing											
No. 10	50 max										
No. 40	30 max	50 max	51 min								
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 max	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 subgrade	Excellent to good						Fair to poor				

ii. **Unified Soil Classification Systems**

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

Category	**Soil classification system**
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderate expansion	GW, SC, ML, MH
High volume change	CL OL, CH, OH
No rating	Pt.

The above classification system can be summarized as follow:

- All clay soil and organic soils exhibit high volume change.
- All clayey gravels and sands and all silts exhibit moderate volume changes.
- All sands and gravels exhibit little or no expansion.

2.1.4.2 Classification Specific to Expansive Soil

The above classification system may give an initial alert that the soil may have expansive character but it does not provide useful information. A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas (Nelson, D.J., and Miller, J.D., 1992). Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which provide reliable information.

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

I. **Method of Chen**

Chen (1988) presented a single index method for identifying expansive soils using only plasticity index. Chen suggested four classes of clays according to their plasticity indices shown in Table 2.6

Table 2. 6: Relation between the swelling potential of clays and the plasticity index (Chen 1988)

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

II. **Method of Daksanamurthy and Raman (1973)**

Daksanamurthy and Raman (1973) presented a single index method for identifying expansive soils using only liquid limit. They suggested four classes of clays according to their liquid limits as shown in Table 2.7 (Amer, A., and Mattheus, F.A., 2006).

Table 2. 7**:** Relation between the swelling potential of clays and the liquid limit *(*Amer, A., and Mattheus, F.A., 2006).

Swelling potential	Liquid limit
Low	$20 < LL \leq 35$
Medium	$35 < LL \leq 50$
High	$50 < LL \leq 70$
Very high	$LL > 70$

III. **USBR Method**

This method is developed by Holtz and Gibbs; it is based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit. The classification is as given in Table 2.8.

Table 2. 8: Classification based on bureau of reclamation method (Chen, F.H., 1988; Ranjan, G., and Rao, A.S.R., 2002)

Colloid content, (%)	Plasticity index, (%)	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

IV. **Method of Seed et al**

After an extensive study on swelling characteristics of remolded, artificially prepared and compacted clays, Seed et.al (Chen, F.H., 1988) have developed a chart based on activity and percent clay sizes as shown in Figure 2.2. The activity here is defined as:

$A_C = \frac{PI}{C-10}$……………………………………………………………………………2.4

Where

A"= activity

C= percentage of clay size finer than 0.002mm

PI= plasticity index

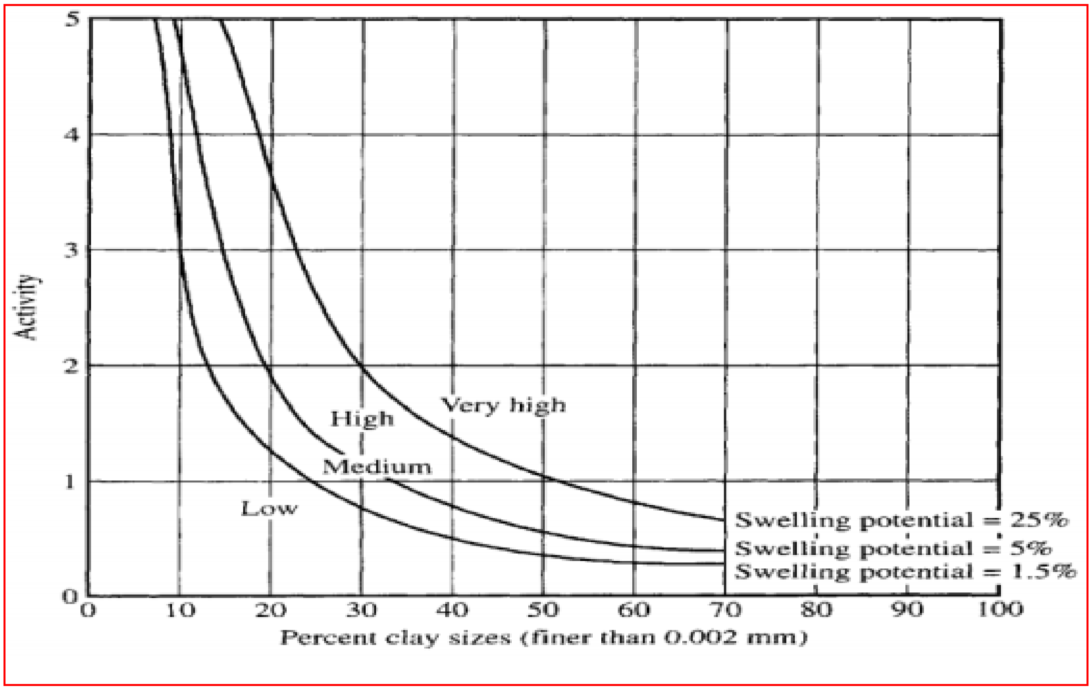


Figure 2. 2: Classification chart for swelling potential after Seed et.al, 1962(as cited by Chen, F.H., 1988)

V. **Method of Skempton**

This method is developed, by combining Atterberg limits and clay content into a single parameter called Activity. Activity is defined as:

A_C = PI /percentage by weight finer than 2μm ………………………………………………2.4

Where

A"= activity and PI= plasticity index

Skempton suggested three classes of clays according to their activity shown in Table 2.9.

Table 2. 9: Relation between clay activity and potential of expansion

Activity	Potential expansion
$A_C < 0.75$	Low (inactive)
$0.75 < A_C < 1.25$	Medium(normal)
$A_C > 1.25$	High (active)

2.2 Stabilization

The successful construction of highways requires the construction of a structure that is capable of carrying the imposed traffic loads. One of the most important layers of the road is the actual foundation, or subgrade. Subgrade soil form the integral part of the road pavement structure as it provides the support to the pavement from beneath. The main function of the subgrade is to give adequate support to the pavement and for this; the subgrade should possess sufficient stability under adverse climate and loading condition. If these structures are founded on soil with low bearing capacity, they are likely to fail either during or after construction, with or without application of wheel load on them. Where the pavement is founded in an inherently weak soil, this material will be typically then removed and replaced with a stronger granular material or improving the soil towards the desired property by addition of chemical(s) (Christopher, H., 2010).

This removal and replacement technique can be both costly and time consuming. Where aggregates are scarce, the use of these non-renewable resources is viewed as non-sustainable, particularly if haulage distances are significant. An alternative to the removal and replacement option is to chemically stabilize the host material. This eliminates the requirement to replace the material, and ensures the engineering characteristics and performance of the host material is enhanced to allow for its use within the pavement structure (Christopher, H., 2010).

2.3 Soil Stabilization

Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil (Guyer, J. P., 2011; US Army, 1994).

2.4 Uses of Stabilization

Pavement design is based on the premise that minimum specified structural quality will be achieved for each layer of material in the pavement system. Each layer must resist shearing, avoid excessive deflections that cause fatigue cracking within the layer or in overlying layers, and prevent excessive permanent deformation. As the quality of a soil layer is increase, the ability of that layer to distribute the load over a greater area generally increase so that a reduction in the required thickness of the soil and surface layers may be permitted. Commonly, improvement attained from soil stabilization can be summarized as;(Guyer, J. P., 2011; US Army, 1994).

• *Quality improvement:* the most common improvements achieved through stabilization include reduction of plasticity index or swelling potential, and increases in durability and strength with a better soil gradation. In wet weather, stabilization may also be used to provide a working platform for construction operations (Guyer, J. P., 2011; US Army, 1994).

• *Thickness reduction:* the strength and stiffness of a soil layer can be improved through the use of additives to permit a reduction in design thickness of the stabilized material compared with an unstabilized or unbound material. The design thickness can be reduced if the strength, stability and durability requirement of a base or subbase course is indicated to suitable by further analysis (Guyer, J. P., 2011; US Army, 1994).

2.5 Types of Soil Stabilization

The two frequently used methods of stabilizing soils are stabilization by compaction or stabilization by chemical additives

2.5.1 Mechanical Stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main methods of mechanical stabilization can be categorized in to compaction, mixing or blending of two or more gradations, applying geo-reinforcement and mechanical remediation (Guyer, J. P., 2011; Makusa, G.P., 2012).

2.5.2 Chemical Stabilization

Soil stabilization using chemical admixtures is the oldest and most widespread method of ground improvement. Chemical stabilization is mixing of soil with one or a combination of admixtures of powder, slurry or liquid to improve or control its stability, strength, swelling, permeability and durability.

Soil improvement by means of chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation-agglomeration pozzolanic reactions.

a) **Cation Exchange**

The excess ions of opposite charge that of the surface of clay, over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge, by altering the chemical composition of the equilibrium electrolyte solution.

Negatively charged clay particles adsorb cations of specific type and amount. The ease of replacement or exchange of cations depends on several factors, primarily the valence of the cation. Higher valence cations easily replace cations of lower valence. For ions of the same valence, the size of the hydrated ion becomes important; the larger the ion, the greater the replacement power. If other conditions are equal, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations. A typical replace ability series is:

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

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers.

An example of the cation exchange;

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

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

b) **Flocculation and Agglomeration**

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

c) **Pozzolanic Reactions**

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

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

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

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

2.6 Cement Stabilization

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers, cement probably is the most routinely used. Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products (AASHTO, 2008). Portland cement stabilization, commonly referred to as soil cement, is a mixture of Portland cement, water and soil compacted to a high density. Soil cement is sometimes referred to as a cement treated subgrade or cement stabilized sub base. When cured, the soil cement mixture becomes a hard, rigid base material. A flexible or rigid pavement surface is placed on top of the soil cement to complete the pavement structure.

Cement stabilization differs from other forms of chemical stabilization in such a way that structural strength is primarily obtained from the cementing action rather than from internal friction, cohesion, chemical ion exchange and/or waterproofing of the materials. Almost all types of soils can be used for cement stabilization except highly organic soils and heavy clay soils.

The four fundamental control factors for the design and construction of soil cement are moisture content, curing procedure and duration, compaction and cement content. Cement stabilization is generally considered to be too expensive for workability improvements alone (The Tensar Coporation, TTN: BR10, 1998)

2.6.1 Mechanisms of Cement Stabilization

Hydration reaction is the primary mode of strength gain in soil cement. Free lime, Ca(OH)2, produced during the hydration process can comprise up to about 25 percent of the cement and water mix on a weight basis. This free lime can produce pozzolanic reaction between the lime and soil, which can continue as long as the pH is high enough to solubilize the soil minerals (AASHTO, 2008)

The hydration product obtained from cement stabilization occurs through the same type of pozzolanic reactions as lime stabilization. It is the origin of silica required for pozzolanic reaction that differs. With cement stabilization, the cement already contains the silica, unlike lime stabilization where silica needs to be broken down from clay. Therefore, unlike lime stabilization, cement stabilization is fairly independent of soil properties (Chrisopher M. Geiman, George M. Filz and Thomas L. Brandon, 2005)

According to The tensar Corporation (1998), mechanisms of cement stabilization are classified in to four major groups.

Hydration of cement (highest importance): continuous skeleton of hard, strong material forms and encloses a matrix of unaltered soil. Strengthening of treated material and filling some of the voids occurs. Permeability and shrink/ swell tendencies are reduced and resistance to changes in moisture content is increased.

Cation Exchange (high importance): Cation exchange alters electric charge, reducing plasticity and resulting in flocculation and aggregation of soil particles.

Carbonation (minor): Lime generated during hydration of cement reacts with carbon dioxide in air to from cementing agents.

Pozzolanic Reactions (minor): Free lime liberated during hydration reacts with silica or alumina from clay particles in the presence of moisture to form cementing agents.

Cement hydration is rapid and causes immediate strength gain in stabilized layers. Therefore a mellowing period is not typically allowed between mixing and compaction. The general practice is to compact soil cement before or shortly after initial set, preferably within 2 hours of mixing (The Tensar Coporation, TTN: BR10, 1998).

2.6.2 Mix Design and Strength Characteristics

The goal of mixture design using cement stabilization is to find the lowest cement content that will produce the desired strength. The cement content determines whether the characteristics of the mixtures are dominated by the properties of original soil or hydration products. The strength of soil cement increases linearly with the quantity of cement added to the soil (ERA Vol.2.2002).

Cement stabilization is ideally suited for well graded aggregates with sufficient amount of fines to effectively fill void space and float the coarse aggregate particles. General guidelines for stabilization are that the plasticity index should be less than 30 for sandy materials. For fine-grained soils, soils with more than 50 percent by weight passing No.200, the general consistency guidelines are that the plasticity index should be less than 20 and the liquid limit should be less than 40 in order to ensure proper mixing. A more specific guideline based on the fines content is given in the equation (2.5) defining the upper limit Plasticity Index (PI)

$$PI \leq \left(\frac{20 + 50 - (\%\text{smaller than } 0.075\text{mm})}{4}\right) \qquad \ldots\ldots 2.5$$

Cement content requirements vary depending on the soil type and desired properties of the soil cement. For most soil cement applications, Type I or Type II Portland cement conforming to ASTM C150 is used. Generally, as the clay content of the soil increases, the quantity of cement required increases. AASHTO soil classification system is used for pavement soil classification in Ethiopia. AASHTO's cement requirement for different types of soils is summarized in Table 2.10.

Table 2. 10: Cement requirement for AASHTO soil Groups

AASHTO Soil Group	Usual Range in Cement Requirement in percent by		Estimated Cement Content, Percent by Weight
	Volume	Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

Water is necessary in soil cement to help obtain maximum compaction and cement hydration. Moisture contents of soil cement usually range from 10 to 13 percent by weight of oven dry soil cement. Water should be potable or relatively clean, free from harmful amount of alkalis, acids or organic matter matrix (Dallas N.Little and Syam Nair, 2009)

2.6.3. Moisture Density Relations of Soil-Cement Mixtures

Changes in optimum moisture content and dry density with addition of cement are not always predictable (*ACI Materials Journal*, vol. 87, no. 4, p. 23, 1990). Flocculation of clay particles by cement can cause an increase in optimum moisture content and decrease in maximum dry density for cement-soil mixes whereas the higher density of cement relative to soil can result in a higher density for mixes. Therefore, it is appropriate to use the median cement content as estimated in Table 2.10 for determination of moisture density relationships as the maximum dry density varies only slightly with modest changes in percent cement content(Portland Cement Association, 1992). However, as previously discussed, if it is expected that acceptable treatment can be achieved with considerably lower cement contents than those in Table 2.10, then that cement content should be used to determine the moisture-density relationship. After the required amount of cement is added to the soil, the blend should be mixed thoroughly until the color of the mixture is uniform. Fabrication and testing of samples for moisture density relationship should be done in accordance with AASHTO T 134.

2.6.3.1 Curing Time

There was an increase in the CBR value as percentage of coir increased. It is because; the addition of coir imparts some amount of shear resistance to the soil. The increase in the strength was less, due to lack of chemical reaction taking place between Arecanut coir and lateritic soil. Good improvement in CBR value was observed with constant dosage of 3 per cent cement. As the curing period increased, the CBR values also increased and the maximum CBR value was obtained at 0.6% replacement of soil by coir, and then it decreased. Increase in CBR value may be because the coir offered better resistance to the penetration of the plunger. The resistance may be made up of bond between soil mixes. The increase in CBR value can also be attributed to the better packing of different fractions (B.M. Lekha, S. Goutham, A.U.R. Shankar, 2014).

2.6.4 Previous Studies

Tesfaye, A., (2001) studied improvement of expansive soil by addition of lime and cement on black cotton soil from different parts of Addis Ababa. Index properties, compaction characteristics and swelling pressure of soil-cement and soil-lime were determined using Atterberg limit test, moisture-density relations, free swell and swelling pressure tests. The conclusions and findings drawn from the study are;

- Expansive soil becomes moderately active to inactive based on the amount of lime and cement added.
- Swelling pressure of expansive soil decreases with increasing lime, cement and molding water content.

4-6% of lime and 9-12% of cement yielded significant improvement on plasticity and swelling properties of expansive soils.

2.7. Industrial and Agricultural Waste as a Soil Stabilizing Material

Recent research works in the field of geotechnical engineering and construction materials focuses more on the search for cheaper and locally available materials, agricultural and industrial wastes, for use in construction industry. The use of different industrial and agricultural wastes has become a common practice in the construction industry. Fly ash, sugarcane bagasse ash, coconut husk ash and rice husk can be sited as an example. Those by-products are increasingly playing a part in road construction and concrete technology, hence minimizing the problem of resource depletion, environmental degradation and energy consumption.

This research focuses on the potential utilization of bagasse ash in soil stabilization, specifically black cotton soil.

2.7.1 Bagasse Ash

Bagasse is the fibrous residue obtained from sugarcane after the extraction of sugar juice at sugar cane mills. Bagasse ash is the residue obtained from the incineration of bagasse. Previously, bagasse was burnt as a means of solid waste disposal. However, as the cost of fuel oil, natural gas and electricity has increased, bagasse has come to be regarded as a fuel rather than refuse in the sugar mills. The fibrous residue used for this purpose leaves behind about 8-10% ash, known as bagasse ash (Aigbodion, V. S., 2010; Hailu, B., 2011).

2.7.1.1 Bagasse Ash as a Soil Stabilizing Material

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

Bagasse ash contains large amount of silica which is the most important component of cement replacing materials. It is also found in large amount as a byproduct in sugar factories. Despite this abundance and silica content, relatively little has been done to examine the potential of this material for soil stabilization. Even though little, the conducted researches conform the suitability of this material for soil stabilization as an admixture with lime and cement. But still its suitability as a standalone material is still questionable.

2.7.1.2 Previous Studies

a) Bagasse Ash as a Standalone Stabilizer

Gandhi, K. S., (2012) investigated on stabilization of expansive soil using bagasse ash. The experimental study involved Atterberg limit test, free swell index and swelling pressure tests. The findings and conclusions of the study can be summarized as follows:

- The results show that when the percentage of bagasse ash is increases in the soil sample, the plasticity of the soil decreases as shown in Table 2.11.

Table 2. 11**:** Variations of Atterberg limits with addition of bagasse ash

(Gandhi, K. S., 2012)

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

- As shown in Figure 2.3 free swell Index decreases as percentage of bagasse ash increases; and shows some linearity between them.

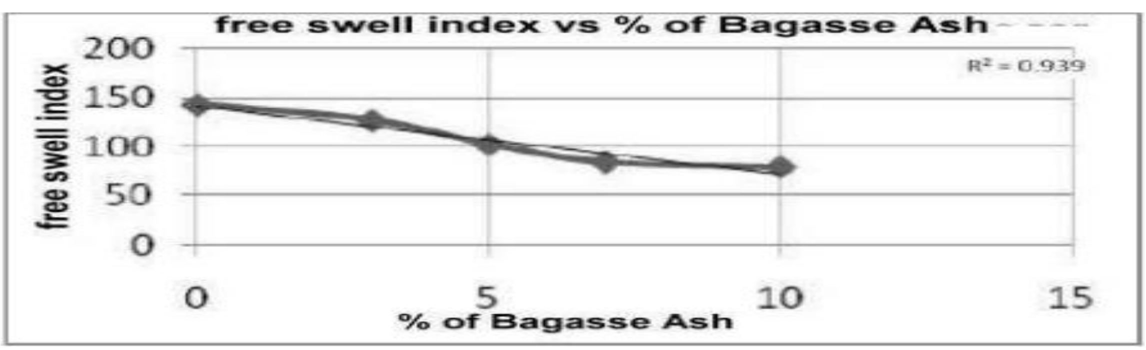


Figure 2. 3**:** Relation between free swell index & percentage of bagasse ash

- Swelling pressure decreases as percentage of bagasse ash increases. Summary of swelling pressure is shown in Figure 24.

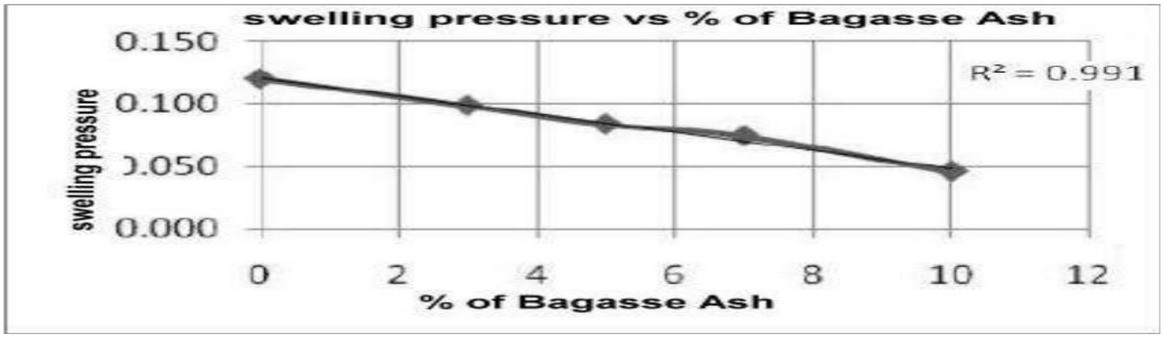


Figure 2. 4**:** Relation between swelling pressure and percentage of bagasse ash.

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

• **Chittaranjan, M., et al., (2011)** evaluated agricultural wastes as soil stabilizers on clay of medium plasticity. Hence, in their investigation an attempt has been made to utilize certain agricultural wastes such as rise husk ash (RHA), groundnut shell ash (GSA) and sugarcane bagasse ash (SCBA) to stabilize weak subgrade soil. Use of these agricultural wastes improves the subgrade strength of the weak soil. The findings of this study are summarized in Table 2.12

Table 2. 12**:** CBR with increase in percentage of bagasse ash

(Chittaranjan, M., et al., 2011)

Agricultural Waste (%)	CBR value For RHA	CBR value for SCBA	CBR value for GSA
0	7.66	7.66	7.66
3	10.63	11.63	9.35
6	14.33	17.54	12.69
9	19.62	21.26	18.36
12	23.96	25.54	20.87
15	24.23	26.68	21.23

• **Osinubi, K.J., and Thomas, S.A., (2007)** evaluated the influence of compactive efforts on bagasse ash treated black cotton soils. The experimental study involved Atterberg limit test, moisture-density relation, UCS and CBR. The findings and conclusions of the study can be summarized as follows:

- ➢ The unconfined compressive strength increased with the addition of bagasse ash content.
- ➢ The CBR increased with the addition of bagasse ash content. But, these values obtained barely meet the minimum specification by the Nigerian general specification (1997) of 15% CBR for a subgrade material.
- ➢ The research concluded that bagasse ash stabilized soil cannot be used as a pavement material. But it can be more profitably used as an admixture with a conventional stabilizer such as cement or lime.

• **Osinubi, K.J., et.al, (2009)** evaluated bagasse ash stabilization of lateritic soil. The experimental study involved moisture-density relation, UCS and CBR. The findings and conclusions of the study can be summarized as follows:

- ➢ The MDD and OMC of the treated generally showed trends of decrease and increase, respectively, with higher bagasse ash content as shown in Figure 2.5

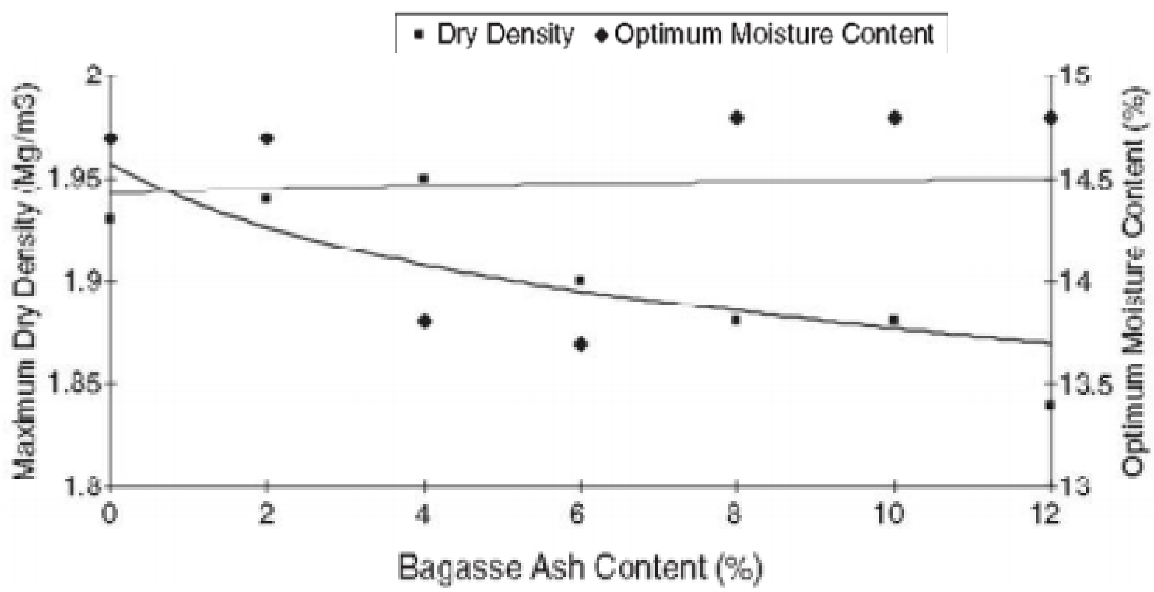


Figure 2. 5**:** Variation of maximum dry density and optimum moisture content with bagasse ash content.

- ➢ CBR and UCS values generally increased with the addition of bagasse ash content as shown in Figure 2.6 and 2.7 respectively.

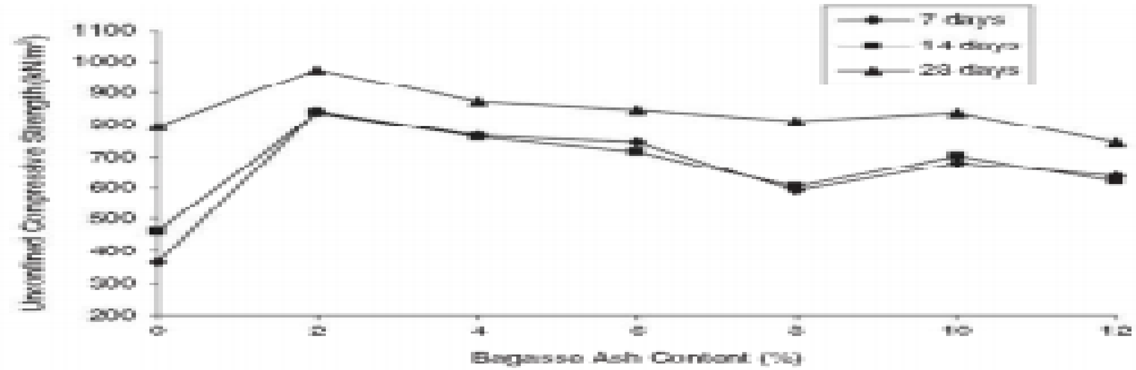


Figure 2. 6**:** Variation of unconfined compressive strength with bagasse ash content

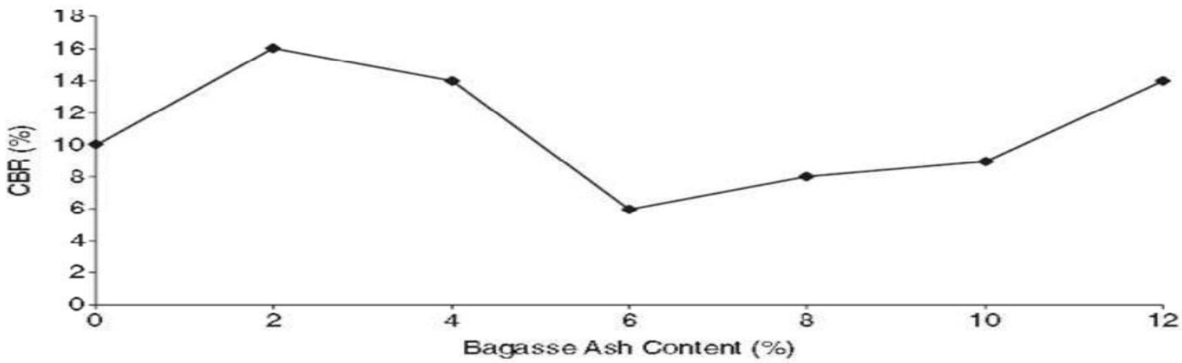


Figure 2. 7**:** Variation of California bearing ratio with bagasse ash content

- Generally the research concluded that bagasse cannot be used as a standalone stabilizer but should be employed in admixture stabilization.

b) Bagasse Ash as an Admixture with Lime

- **Sabat, A.K., (2012)** studied the utilization of bagasse ash and lime sludge for construction of flexible pavements in expansive soil areas. The experimental study involved compaction characteristics, unconfined compressive strength, CBR and swelling pressure. The following conclusions are drawn from the study:
  - ✓ The addition of bagasse ash to expansive soil decreases the MDD and increases the OMC of the expansive soil irrespective of the percentage of addition of bagasse ash. Addition of lime sludge to each expansive soil-bagasse ash mixes decreases the MDD and increases the OMC of the expansive soil irrespective of the percentage of addition of lime sludge.

Generally the research concluded that the industrial wastes, such as bagasse ash and lime sludge can be utilized for strengthening the subgrade soil with a significant saving in cost of construction.

- **Ochepo, J. and Osinubi, K. J., (2013)** studied the effect of compactive effort and elapse time on the strength of lime-bagasse ash stabilized expansive clay from Gombe, Nigeria. The experimental study involved unconfined compressive strength. The following conclusions are drawn from the study:
  - ✓ The results obtained indicate that UCS values increase with lime and bagasse ash treatment, curing periods.

c) **Bagasse Ash as an Admixture with Cement**

- ❖ **Mu'azu, M.A., (2007)** evaluated the plasticity and particle size distribution characteristics of bagasse ash on cement treated lateritic soil. The experimental study involved Atterberg limit test and particle size distribution. The findings and conclusions of the study can be summarized as follows:
- ➢ The liquid limit reduced while the plastic limit increased and consequently the plasticity index reduced with addition of bagasse ash and cement.
- ➢ With respect to particle size distribution, there was a general reduction in the percentage of fines with addition of bagasse ash and cement.
- ➢ The research concluded that from the consideration of the plasticity characteristic and grain size, the Nigerian general specification on road and bridgeworks for direct use of the materials as pavement material are not met for any of the bagasse ash and cement contents considered, but the grain size requirement were met with the respect to the percentage passing BS sieve No. 200.
- **Onyelowe, K. C., (2012)** studied cement stabilized Akwuete, Nigeria, lateritic soil and the use of bagasse ash as admixture. The experimental study involved moisture-density relation and CBR. The findings and conclusions of the study can be summarized as follows:
  - ➢ At 4% cement content, with addition of bagasse ash as an admixture, there is a general reduction in the MDD increment in OMC and CBR as shown in Table 2.13 while there is an increase in the MDD and CBR and reduction in OMC with increase in bagasse ash content from 2-10% at 6% cement content as shown in Table2.14
  - ➢ From the foregoing tests carried out, bagasse ash is proved to be a good pozzolana in soil stabilization and modification.

Table 2. 13**:** Variation of OMC, MDD and CBR with addition of bagasse ash contents at 4% cement (Onyelowe, K. C., 2012)

Bagasse ash (%)	OMC (%)	MDD (g/cm3)	CBR (%)
0	20.51	1.78	58.77
2	20.71	1.79	85.66
4	21.20	1.74	86.10
6	21.80	1.69	94.20
8	22.64	1.50	112.18
10	22.88	1.50	128.05

Table 2. 14: Variation of OMC, MDD and CBR with addition of bagasse ash contents at 6% cement (Onyelowe, K. C., 2012)

Bagasse ash (%)	OMC, (%)	MDD, (g/cm3)	CBR, (%)
0	17.90	1.88	85.30
2	14.02	1.96	98.60
4	15.96	1.98	106.70
6	16.01	2.01	119.02
8	16.22	2.30	126.75
10	16.40	2.50	137.62

2.7.1.3 Availability of Bagasse ash in Ethiopia

In order to assess the potential of bagasse ash production in Ethiopia, it is imperative to evaluate the sugarcane crop yield in the country. There are three state owned sugar factories functioning in the country in 2013. Their annual production capacity is about 300,000 tons, the sugarcane covering about 10,000 hectares of land. This annual production is not sufficient to the local sugar demand forcing the government to annually import 1.5 million quintals from abroad.

To avoid this shortage of sugar in the country the government plans to establish eight new sugar factories in the coming five years with a total estimated capacity of 2.250 million tons at the start of their production according to the strategic plan and covering about 225,000 hectares.

Beside this the government is undertaking expansion projects on the existing factories to increase their production capacity. At the end of this expansion projects on Fincha, Methara and Wonji-Shoa sugar factories the additional total production capacity is expected to be around 365,000 tons of sugar annually. In detail, Fincha found in the western part of the country planned to increases its production to 270,000 tons; Wonji-Shoa found 125km east of Addis Ababa plans to increase their production to 350,000 tons; Methara sugar factory found 200kms east of Addis Ababa, is also expected to increase its annual production to 190,000 tons according to the sugar development study paper. Tendaho sugar factory is expected to have an annual production capacity of 600,000 tons is expected to be completed at the end of 2014.

As can be seen from the above discussion the sugar production in the country is boosting at a high rate, even planning to hold 2.5% of the world sugar market in the coming years according to the strategic plan. This boosting in sugar production will also result in high amount of bagasse and bagasse ash. When all the factories become fully operational, the bagasse ash from all these factories will be expected to be in thousands of tones. As per the information from Ethiopian Sugar Corporation, all of the factories that are operating currently are now using bagasse as a fuel for boiler. Not only the current factories but the future intended projects will also operate in the same manner as this method reduces energy consumption. When all the factories start to operate with their full capacity, the respective bagasse ash that will be produced by that time will reach up to two million ton per annum. Bagasse ash of this amount can substantially contribute to both technical and environmental advantage to the cement industry. Table 3.5 summarizes the expected future sugar production of the country and the respective bagasse ash potential.

Table 2. 15**:** Estimated bagasse ash potential of Ethiopia

(Ethiopian Sugar Corporation (Communication department), unpublished, 2016).

S.No	**Sugar factories**	**Tone of cane per day (TCD**	**Annual crushing capacity (Ton)**	**Bagasse (Ton)**	**Bagasse ash (Ton)**
1	Wonji Shoa	12,500	3,000,000	870,000	108,75
2	Metehara	5,000	1,200,000	348,000	43,500
3	Fincha	12,000	2,880,000	835,200	104,400
4	Tendahu	26,000	6,240,000	1,809,600	226,200
5	Beles I	12,000	2,880,000	835,200	104,400
6	Beles II	12,000	2,880,000	835,200	104,400
7	Beles III	12,000	2,880,000	835,200	104,400
8	Kuraz I	12,000	2,880,000	835,200	104,400
9	Kuraz II	12,000	2,880,000	835,200	104,400
10	Kuraz III	12,000	2,880,000	835,200	104,400
11	Kuraz IV	24,000	5,760,000	1,670,400	208,800
12	Kuraz V	24,000	5,760,000	1,670,400	208,800
13	Kesem	11,000	2,640,000	765,600	95,700
14	Arjo dedesa	8,000	1,920,000	556,800	69,600
15	Wolkayte	24,000	5,760,000	1,670,400	208,800
		218,500	**52,440,000**	**15,207,600**	**1,900,950**

The annual production capacity data was taken from Ethiopian Sugar Corporation; communication department. As per the combined information taken from the corporation's technical office and chemical laboratory technicians in Wonji sugar factory, the bagasse that will be extracted from the cane accounts 28-30% of the total cane production. And the bagasse ash that will be obtained from the bagasse was estimated to be 11-14% of the bagasse produced. The above calculation in the table was done using the average percentage of the range and an annual operational period of 240days.

From the table it can be seen that around 2 million tons of bagasse ash per year will be disposed from the sugar industry when the intended sugar factories will start to operate with their full production capacity. This amount can significantly support and strengthen the cement industry if there will be a system and mechanism that makes the proper utilization of this material into practice.

The results of chemical tests carried out on bagasse ash in Ethiopia and elsewhere are shown in Table 2.16.The results indicate pozzolanicity of the bagasse ash. The combined percent composition of Silica, Al2O3 and Fe requirement of ASTM C618 standard

Table 2. 16**:** Oxide composition of bagasse ash

(Hailu, B., 2011*, Osinubi, K.J. and Thomas, S.A.,2005**)

Constituents	**Chemical composition (%)**		ASTM (C618) Requirement in (%)	Remark
	Wonji shoa*	**Nigeria****		
SiO_2	65.58	57.12	35 Min	Satisfied
Al_2O_3	5.87	29.73		
Fe_2O_3	4.32	2.75		
CaO	1.78	3.23		
MgO	1.23	-	5 Mix	Satisfied
K_2O	6.41	8.75		
Na_2O	1.02	-		
P_2O_5	1.35	-		
SO_3	0.18	0.02		
Cl_2	< 0.1			
MnO	0.05			
TiO_2	0.25	1.1		
L.O.I	10.48	17.57		
SiO_2 + $Al2O_3$ + Fe_2O_3	75.77	89.6	70 Min	Satisfied

According to table 4.9 all chemical properties of bagasse ash are satisfied the requirements when compared ASTM C618.

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1 Study Area

Humera is located in northwestern Ethiopia. By road it is 984 km northwest of Addis Ababa, 515 km west of Mekelle, and 267 km east of Shire. The Tekezé river runs to the west of Humera. The city is spread on the east bank of the river. The Geographical condition of the town approximately at 14°18′N and 13°55′N latitude and 36° 37'E and 37° 7'60" E longitude Humera has a Mediterranean climate with the temperature of 33°C. The overall climate throughout the year is mild and dry. The annual rainfall ranges between 400 to 600 mm, with most of the rain falling in the rainy season (June up to September). The town is found in an area of the altitude of 585 m above sea level (GSE). It lies in the climatic zone locally known as bereha which is considered ideal for agriculture as well as human settlement. (source Google map 2018).

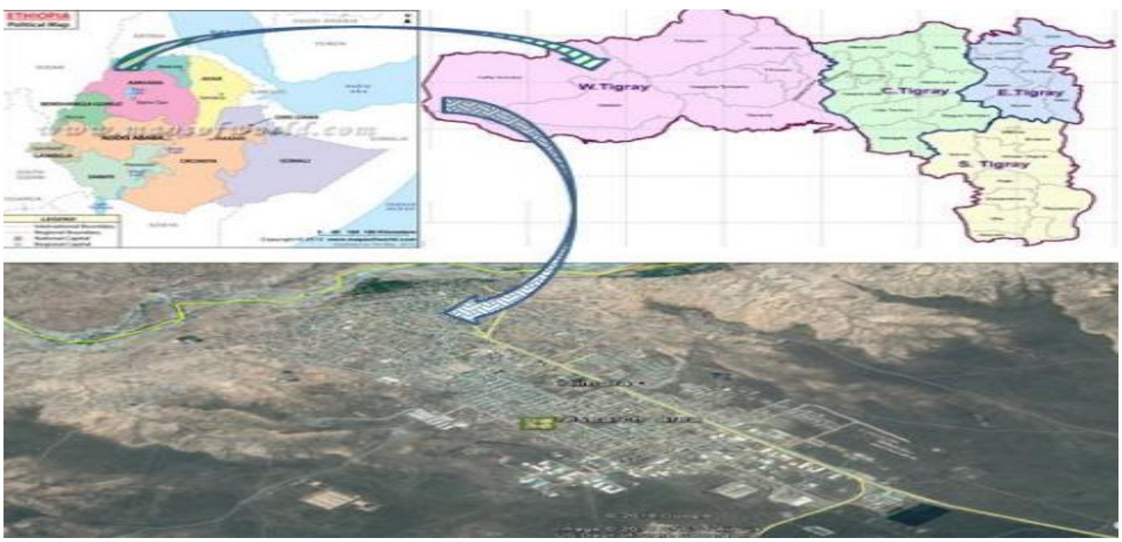


Figure 3. 1: Location map of the study area (Source: Ethio-GIS (1984) and Google Map)

3.2 Materials

In this area, laboratory works and analysis of collected samples and soil stabilizer materials are presented. Laboratory works was done in Sur Construction PLC lot Adiremets – Adihirdi – Beahker Laboratory, Woldia University Civil Engineering Laboratory. The chemicals which were used for this study were ordinary Portland cement taken from Sur Construction PLC and bagasse ash which were taken from Arjjoo Dedesa sugar Factory. Besides, secondary data were also obtained from Geological Survey of Ethiopian (GSE)

Bagasse Ash: Waste bagasse ash was obtained from Arjo Dedesa Sugar Factory which is located in the Western Ethiopia in Oromiya Regional State. For this research, fresh bagasse ash is taken from the furnace cooled in air by applying a small quantity of water. Then, it is properly packed in sacks and transported to the laboratory.

Figure 3. 2: views of bagasse ash disposal site and fresh bagasse ash (source Gidey T.1/102018)

Cement: Mesebo - Ordinary Portland cement (OPC) whose Cement Grade 42.5R and specific gravity of 3.15 which was commercially available and used as stabilizer mixed with bagasse ash. There are different types of cement however for this study OPC was used, because it has a high sulfate resistance capacity (Guyer, J. P., "Introduction to Soil Stabilization in Pavements", New York, 2011.

Table 3. 1: oxide content of ordinary Portland cement.

(A. M. Neville, 1996)

Oxides	CaO	SiO_2	AL_2O_3	Fe_2O_3	SO_3	MgO	Na_2O	K_2O
Ranges (%)	60-70	17-25	3-8	0.5-6	1-3	0.1-4	0.5-1.3	0.5-1.3

Black cotton Soil: The black cotton soil samples used for this research work were collected from around Humera Town, in Kafta Humera from Adiremets –Adihirdi – Bahker road project at station 45+60 and humera Airport. The soils were black cotton in colour and highly plastic clay. The excavation was made manually using the shovel. The collected samples for this study were disturbed samples at a depth of below 1.5 m to remove organic matter.

3.3 Sample collection

For this study, 3 subgrade soils were taken from different place of Kafta Humera town. From those two most clayey soils were selected because of time constrain by observations and free swell index tests. Those are Adiremets - Adihirdi – Behaker upgrade road project approximately found at 13°55' N Latitude and 37°7'59" E Longitude and Humera Airport approximately found at 13°49'49" N Latitude and 36°52'54" E Longitude ("www.Google map.com," 2018). The excavation was made manually using the shovel. The collected samples for this study were disturbed samples at a depth of below 1.5 m to remove organic matter.

Figure 3. 3:Photo of black cotton soil taken from AABRP and HAP site (source: Gidey T.,2018).

3.4 Study Design

The research was following the experimental type of study which begins collecting samples. The stages involved in the study include:-

- Taking sample
- Preparation of sample for each laboratory tests
- Process of mixing bagasse ash with cement

- Find out maximum replacement amount that satisfies requirement of the standard specification.
- Process of chemical stabilization to determine the amount of cement needed to be added to bagasse ash to satisfy strength requirements.

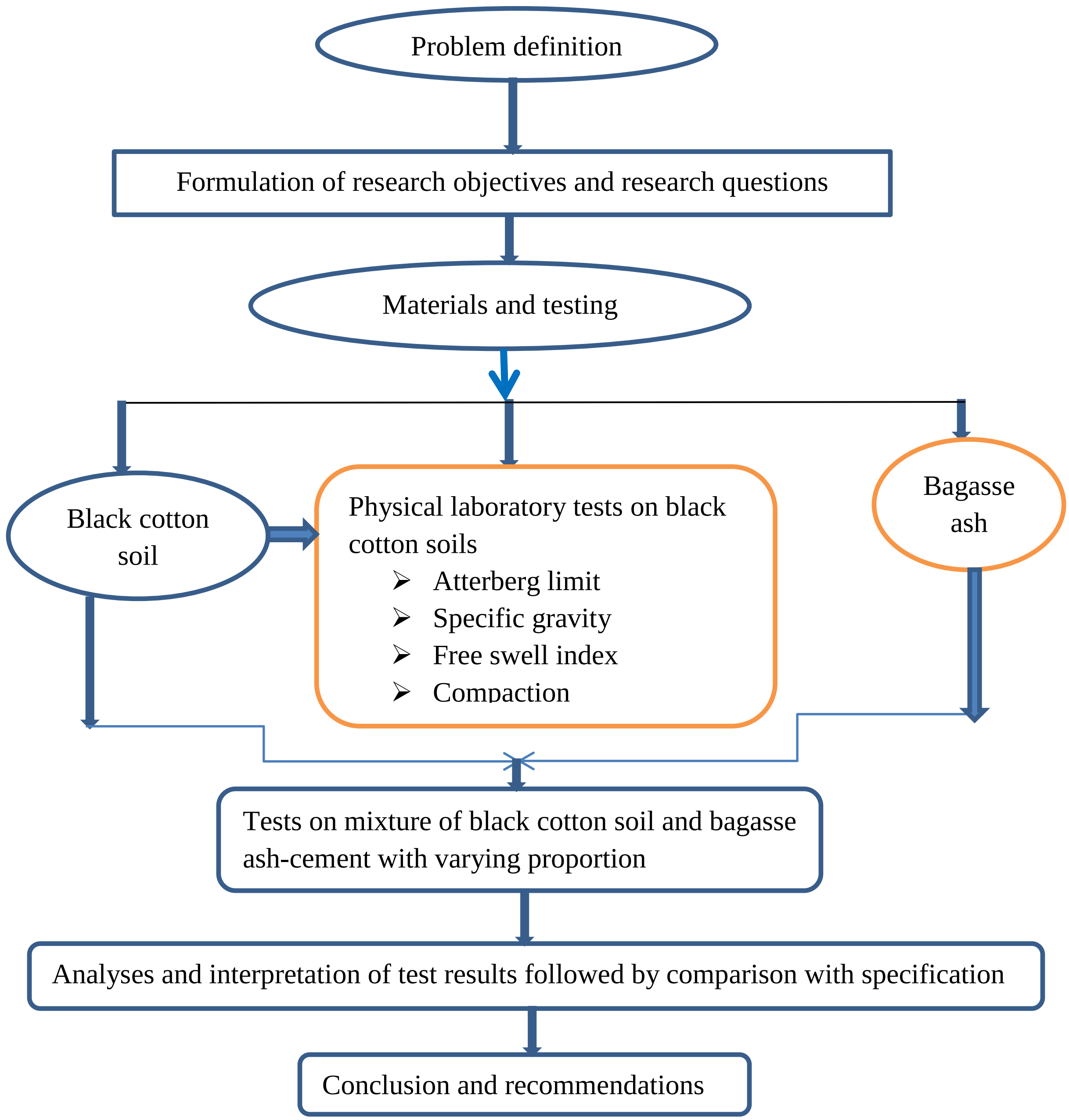


Figure 3. 4: Flow chart showing general outline of the study area.

3.5 Study variables

Table 3. 2: Study variables

Independent variables	dependent variables
➢ Atterberg Limits ➢ Specific gravity ➢ Compaction ➢ Free swelling index ➢ Particle size distribution	➢ Performance of stabilized black cotton soil with bagasse ash and cement.

3.6 Sources of data

Both primary data and secondary data sources were used. Primary data for this study were a laboratory experiment output. Secondary data needed for this research was collected from different journals, book, website and manuals.

3.7 Sampling techniques

The sampling technique used for this research was a purposive sampling which is nonprobability method, because the experimental investigation of the study was executed particularly on the most black cotton soil samples, Since these study pick out the sample in relation to some criterion, It was considered important for the particular study. This sampling technique was proposed based on the target to perform laboratory test on the selected sub grade soils to investigate the extent/performance of bagasse ash on black cotton sub grade soils.

3.8 Methods and standard testing procedure

Table 3. 3: Standards and specifications for this study Source: AASHTO, ASTM and IS

No.	Laboratory test	Standards		
		AASHTO	ASTM	IS
1	Moisture Content	T265		
2	Grain size analysis		D422-63	
3	Atterberg limit		D4318-98	
4	Soil classification		D2487-98	
5	Specific gravity		D854-83	
6	Free swell index			2720 (part 40)
7	Standard proctor test		D698-98	
8	CBR test	T193-93		

3.9 Laboratory Experiment

3.9.1 Black Cotton Soil

3.9.2 Sample Preparation

Before treating and testing, the sample was prepared in accordance with the described AASHTTO T87-86. This method includes:

1) Soil samples as received from the field dried in air/sun.
2) The soil clods were broken by wooden mallet to hasten drying
3) The organic matters were removed from sample then the sample kept in oven for drying at 60^{o}C for 24 hr.
4) For test like liquid limit, plastic limit and light compaction the sample were air dried. Using the above soil sample for bask laboratory investigation by blending it with bagasse ash and cement.
5) Before mixing the prepared soil sample with bagasse ash and cement the original bagasse ash dried in an oven to form dry ash. Then all the materials are ready for mix.
6) Now the concern of uniformity of mix to attain the uniform mix, adding little water to mix then dried in oven. Now the blended sample ready for detail laboratory investigation.

Figure 3. 5: sample quartering and mixing of black cotton soil with bagasse ash and cement (source: Gidey T., 17/112018).

3.9.3 Moisture Content

The test is conducted in accordance with AASHTO T265. Small representative sample of the natural soil and soil-bagasse ash-cement mixture specimens are obtained and oven-dried at 105 ± 5°C for at least 16hours. The samples were then reweighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content of the soil a dry weight basis.

3.9.4 Grain Size Analysis

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and hydrometer method is used to determine the distribution of finer particles. For this study both wet sieve analysis and hydrometer analysis was done according to ASTM D422-63.lastly the analysis was combined particle size distribution curve was plotted as figure 4.1

3.9.5 Atterberg Limits Testing

The test includes the determination of the liquid limits, plastic limits and the plasticity index for the natural soil and the soil-bagasse ash-cement mixtures.

Liquid Limit: The soil sample for liquid limit is air dried and 200g of the material passing through No. 40 sieve (425µm aperture) was obtained and thoroughly mixed with water to form a homogeneous paste on a flat glass plate. A portion of the soil water mixture is then placed in the cup of the Casagrande apparatus, leveled off parallel to the base and divided by drawing the grooving tool along the diameter through the center of the hinge. The cup is then lifted up and dropped by turning the crank until the two parts of the soil come into contact at the bottom of the groove. The number of blows at which that occurred was recorded and a little quantity of the soil was taken and its moisture content determined. The test is performed for well–spaced out moisture content from the drier to the wetter states. The values of the moisture content (determined) and the corresponding number of blows is then plotted on a semi–logarithmic graph and the liquid limit is determined as the moisture content corresponding to 25 blows. The same procedure is also carried out for the treated soil with increment of bagasse ash-cement content.

Plastic Limit: A portion of the natural soil and the soil–bagasse ash-cement mixture used for the liquid limit test is retained for the determination of plastic limit. The ball of the natural soil and the soil– bagasse ash-cement mixture is molded between the fingers and rolled between the palms of the hand until it dried sufficiently, even though the soil is already relatively drier than the ones used for liquid limit. The sample is then divided into approximately two equal parts. Each of the parts is rolled into a thread between the first finger and the thumb. The thread is then rolled between the tip of the fingers of one hand and the glass. This continued until the diameter of the thread is reduced to about 3mm. The movement continued until the thread shears both longitudinally and transversely. The crumbled natural soil and soil–bagasse ash-cement mixture is then put in the moisture container and the moisture content determined. The same procedure is also carried out for the treated soil with increment of bagasse ash-cement content.

Plasticity Index: The plasticity index of the natural soil and the soil–bagasse ash-cement mixture is the difference between the liquid limits and their corresponding plastic limits. The plasticity indexes of the samples are calculated as:

$$PI = LL - PL \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.3.1$$

3.9.6 Soil Classification

The most widely used soil classification systems are AASHTO and USCS systems. The AASHTO classification system classifies soils into seven major groups from A-1to A- 7 with 12 subgroups. The system is based on particle size, liquid limit and, plasticity index of the soil. The USCS is based on grain size, gradation, plasticity, and compressibility. It divides soils into three major groups: coarse-grained soils, fine grained soil and organic soils.

3.9.7 Free Swell Index

The test includes the determination of the free swell index of the natural soil and the soil-bagasse ash-cement mixture. The tests are conducted in accordance with IS: 2720 (Part40) 1977 testing procedure.

Two samples of oven dried soil 10cc each, passing through 425 micron sieve are taken. One is put in a 100cubic centimeters graduated glass cylinder containing kerosene. The other sample is put in a similar cylinder containing distilled water. Both the samples are left undisturbed for 24 hours and then their volumes are noted. Then free swell index is determined using Equation (2.2). The same procedure is also carried out for the treated soil with increment of bagasse ash-cement content.

Figure 3. 6: Free Swell Index test (Source: Gidey T., 20/11/2018)

3.9.8 Free Swell Ratio Test

In this study, recommended procedure of Sridharan and Prakash is adopted.10gm oven dried soil passing through 425 micron is added to 100ml of distilled water in a jar and another 10gm of same sample is added to 100ml of Kerosene. After 24hours, sediment volumes of samples are measured to determine free swell ratio. Free swell ratio is the ratio of change in volume in water to change in volume in kerosene after 24 hours. Then free swell ratio is determined using Equation (2.3). The same procedure is also carried out for the treated soil with increment of bagasse ash-cement content.

3.9.9 Free Swell Tests

The test includes the determination of the free swell for the natural soil and soil-bagasse ash-cement mixture. This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs, (1956) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample. The procedure consists of pouring very slowly of 10 cubic centimeters of that part of the dry soil passing No. 40 sieve in to a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately 24 hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted. Finally, free swell value is calculated using Equation (2.1).

3.9.10 Specific Gravity

This lab is performed to determine the specific gravity of soil by using a pycnometer. Specific gravity is the ratio of the mass of the unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The importance of determining the specific gravity in this study is to determine particle sizes in hydrometer analysis. The specific gravity of the samples was determined using ASTM D 854-83, and the result of the test is as tabulated at Table 4.5.

3.9.11 Compaction

This laboratory test is performed to determine the maximum dry density and the optimum moisture content for the natural soil and the soil-bagasse ash-cement mixture. The compact effort is the amount of mechanical energy that is applied to the Soil mass and soil-bagasse ash-cement mixture. For road construction, the CBR value is obtained using the compaction test results. And these CBR results used to determine the thickness of the sub-grade layer of a road section. The test for this study was done according to ASTM D698-98 test procedure.

Maximum Dry Density: The maximum dry density is conducted for both the natural and soil-bagasse ash-cement mixture of about 2.5kg, by varying the moisture content. The sample is then compacted into the 944 cubic centimeters (of mass m1); in three layers of approximately equal mass with each layer receiving 25 blows. The blows are uniformly distributed over the surface of each layer. The collar is then removed and the compacted sample leveled off at the top of the mould with a straight edge. The mould containing the leveled sample is then weighed to the nearest 1g; m2. One small representative sample is then taken from the compacted soil for the determination of moisture content. The same procedure is repeated until minimum of five sets of samples are taken for moisture content determination. The bulk density is then calculated for each compacted specimen using:

$$p = \frac{m1-m2}{944} \quad \text{………………………………………………………………………………..………3.2}$$

The dry density is also calculated using the following equation:

$$pd = \frac{100*p}{(100+p)} \quad \text{…………………………………………………………………………..……3.3}$$

Where; w is the moisture content of each compacted specimen. The values of the dry densities as obtained from equation above are plotted against their respective moisture contents and the dry densities; MDD is deduced as the maximum point on the resulting curves.

Optimum Moisture Content: The corresponding value of moisture contents at maximum dry densities, which is deduced from the graph of dry density against moisture content, gives the optimum moisture content.

3.9.12 CBR and CBR-swell

The CBR and CBR-swell tests are conducted in accordance with AASHTO T193-93 for the natural soils and soil-bagasse ash-cement mixture. For stabilized soil samples tests were conducted for uncured and 3, 7 and 14 days cured soil samples. The CBR is expressed by the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration.

6.0kg of the natural soil and the soil-bagasse ash-cement mixture are mixed at their respective optimum moisture contents in 2124 cubic centimeters mould. The samples are compacted in three layers with 56 blows from the 2.5kg rammer.

The CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The CBR is obtained as the ratio of load required to affect a certain depth of penetration of a standard penetration piston into a compacted specimen of the soil at some water content and density to the standard load required to obtain the same depth of penetration on a standard sample of crushed stone.

In equation form, this is:

CBR= (test load on the sample / standard load of the crushed stone)*100……………………..3.4

CBR tests were conducted on the compacted specimens at the optimum moisture content using standard compaction test. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value and the CBR swell of the soil. The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold is taken just after soaking the sample. At the end of 96 hours the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is given by:

CBR Swell = (Change in Length in mm during Soaking/116.3mm)*100 % ……………… (4.5)

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Properties of Natural Soil Used in the Study

This chapter presents the results of laboratory tests and a discussion pertinent to the results. The relevant engineering property of the soil is evaluated both for natural and stabilized soil samples separately. The tests include Atterberg limits, free swell index, moisture density relationship (compaction) and California bearing ratio (CBR). CBR are conducted for uncured and 3, 7 and 14 days cured soil samples.

The results of the tests conducted for identification and/or determination of properties of the natural soil before applying bagasse ash for AABRP and HAP are presented in Table 4.1. The soils are dark black in color. The soil sample of AABRP has the following physical properties. As shown in Figure 4.1 on the particle size distribution curve almost 90.38% of the soil is passing through No. 200 sieve; it exhibits a liquid limit of 90.34 %, a plastic limit of 36.57 % and plasticity index of 53.77%. Liquid limit less than 35% indicates low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity (Whitlow, R., 1995). Hence, these values indicate that the soil is highly plastic clay. Accordingly the soil falls under the A-7-5 soil class based on AASHTO soil classification system. Soils under this class are generally classified as a material of poor engineering property to be used as a sub-grade material.

Results that are related to swelling characteristics of the soil are also indicating that the soil is highly expansive clay with a free swell index of 80%.

The soil has a maximum dry density of 1.35g/cm3, optimum moisture content of 21.4% and soaked CBR value of 0.98%. The general relationship between CBR values and the quality of the subgrade soils used in pavement applications is as follows (Bowles, J., 1992).

CBR-values	**Quality of subgrade**
0 – 3 %	very poor sub grade
3 – 7 %	poor to fair sub grade
7 – 20 %	fair sub grade
20 – 50 %	good sub grade
Above 50	Excellent sub grade

Table 4. 1:Geotechnical properties of the natural soil

Parameter	Results	
	AABRP	HAP
Natural Moisture Content	43	37
Percentage of passing No.200sieve	90.38	96.5
Liquid limit	90.34	101
Plastic limit	36.57	36.68
Plasticity index	53.77	64.32
Grain size		
Gravel fraction	0%	0%
Sand fraction	9.6%	3.5%
Silt fraction	35%	30.88%
Clay fraction	54.71	65.62%
AASHTO	A – 7 - 5	A – 7 – 5
USCS	CH	CH
Specific Gravity	2.71	2.70
Free swell index	80	100
Maximum dry density, g/cm3	1.351	1.315
Optimum moisture content	21.4	24.5
Soaked CBR value	0.98	0.81
CBR-swell %	16.66	18.29
Color	black	Dark black

Hence, the soils were found to be highly plastic black cotton soil with low bearing capacity when it is soaked and high swelling potential and fell below the standard recommendations for most geotechnical construction works especially highway construction. Therefore, the subgrade shrink and swell easily and does not resist internal and external load. Finally the structure make crack and easily demolish. To protect this failure stabilization using different additives should be required.

4.1.1Particle size distribution

The results of the natural soil sample before any stabilizers are added listed above in Table 4.1. The soil for sample BAARP is black, and almost 90.38% of the soils are passing through No.200 sieve as shown in Figure 4.1.

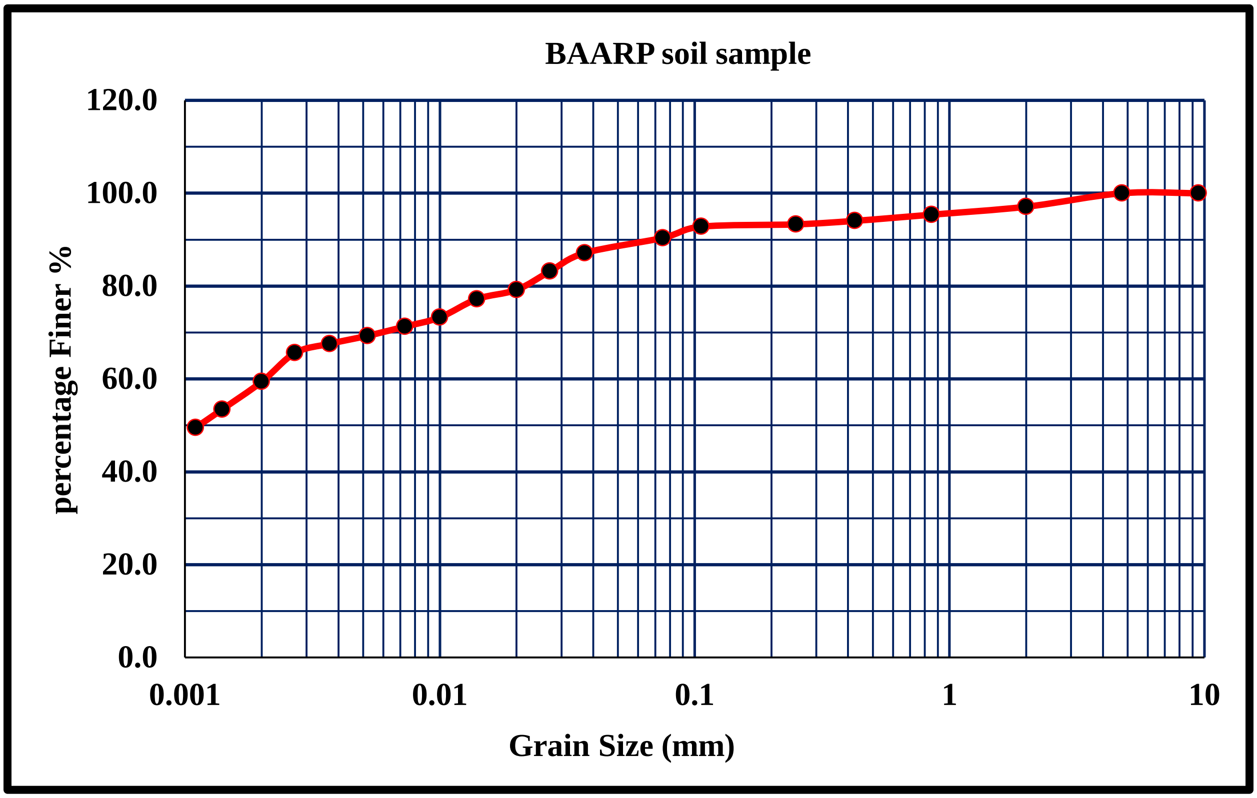


Figure 4. 1: Particle size distribution curve of the BAARP sample black cotton soil

The distribution of different grain sizes affects the engineering properties of the given soil. Grain size analysis provides the grain size distributions, and it is required in classifying the soil. The soil for sample AABRP is dark black and almost 90.38 of the soil is passing through No.200 sieve as shown in figure 4.1. Almost the given soil sample were a fine expansive soil. This assists to know its grain size distribution of the selected area. Mechanical analysis was used for coarse sized soils by using set of sieve and whereas hydrometer analysis was used for fined grained soils. Sodium hexameta phosphate is used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods were performed.

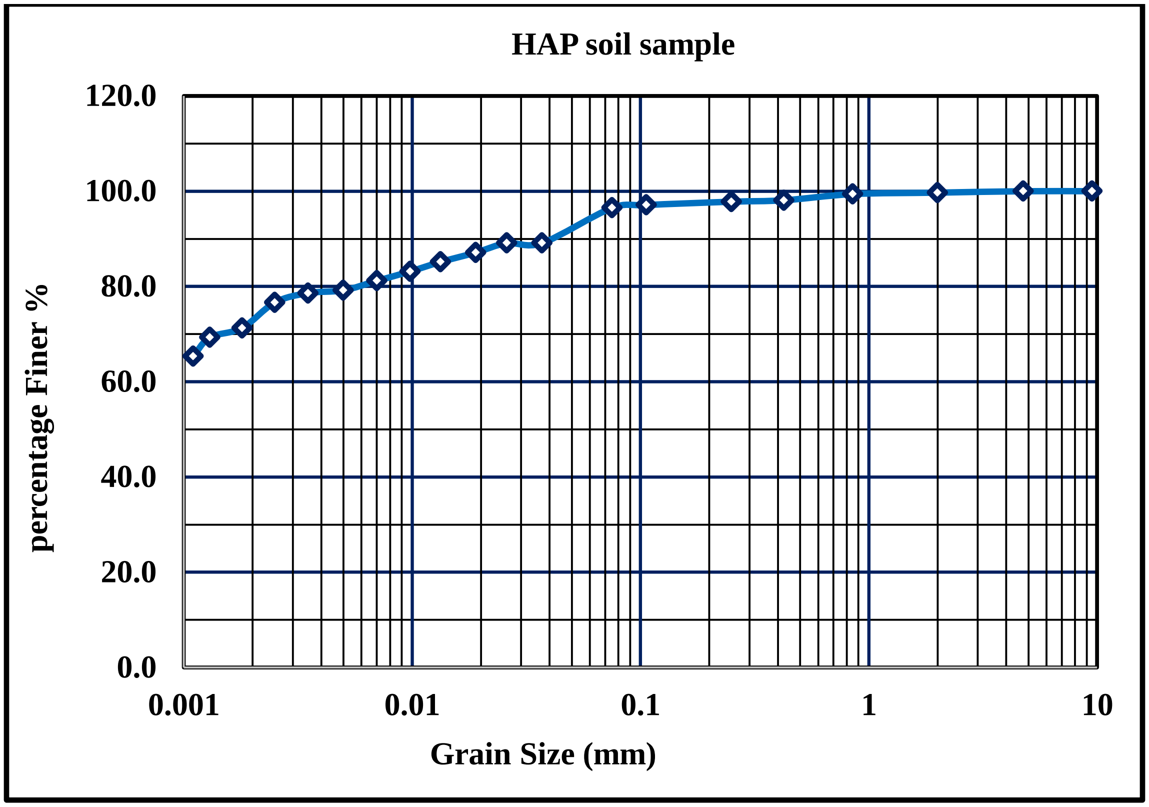


Figure 4. 2: Particle size distribution curve of the HAP sample expansive soil.

The soil for sample HAP is dark black and almost 96.5% of the soil is passing through No.200 sieve as shown in figure 4.2. Almost the given soil sample were a fine black cotton soil. This assists to know its grain size distribution of the selected area. Mechanical analysis was used for coarse sized soils by using set of sieve and whereas hydrometer analysis was used for fined grained soils. Sodium hexameta phosphate is used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods were performed.

4.1.2 Atterberg test result

As tabulated in Table 4.2 below the soil samples shows that the liquid limit of 90.34%, plastic limit of 36.57 % and plasticity index of 53.77 for Adihirdi – Adiremets - Beahker road project at station of 45 + 60 test pit and the liquid limit of 101 %, plastic limit of 36.68 and plastic index of 64.32 for the soil of Humera Airport test pit.

Table 4. 2:Atterberg test result of the AABRP and HAP sample soil

100 % soil sample			
Sample name	**Average Liquid limit % (LL)**	**Average Plastic limit % (PL)**	**Plastic index (PI)**
AABRP	90.34	36.57	53.77
HAP	101	36.68	64.32

From Table 4.2 the AABRP and HAP soil sample changed from liquid state to plastic state and got an average liquid limit of 90.34% and 101% respectively. As a result, at this stage all the soils possess certain small shear strength. This arbitrary chosen shear strength is probably the smallest value that is feasible to measure in standardized procedure. The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 36.57 % and 36.68 % for AABRP and HAP soil sample respectively. At this state the soil rolled into threads. Further decrease of water contents of the same will lead finally to the point where the samples can decrease in volume no further. At this point the sample begins to dry at the surface, saturation is no longer complete, and further decrease in water in the voids occurs without change in the void volume. The difference between the liquid lime and plastic limit is called Plastic Index. The soil sample also has called Plastic Index of 53.77% and 64.32% for both soil samples respectively.

4.1.3 Soil Classification

A. AASHTO Classification system

From AASHTO Classification system results shown in table 4.3 and Figure 4.3 it can be concluded that both AABRP and HAP soil samples fall under A-7-5, which were clayey soils with group index of 58 and 75.3 respectively. The group index results indicate that generally the soils of the study area were very poor for subgrade material.

Table 4. 3: Classification of soils based on AASHTO classification system

Sample Name	Sieve Analysis Percentage of Passing			LL (%)	PI (%)	LL-30	Group Index	Soil Group	Material Type
	No. 10	No. 40	No. 200						
AABRP	97.1	94.1	90.4	90	54	50	58	A – 7 - 5	Clay
HAP	99.7	98.1	96.5	101	64	71	75.3	A – 7 - 5	Clay

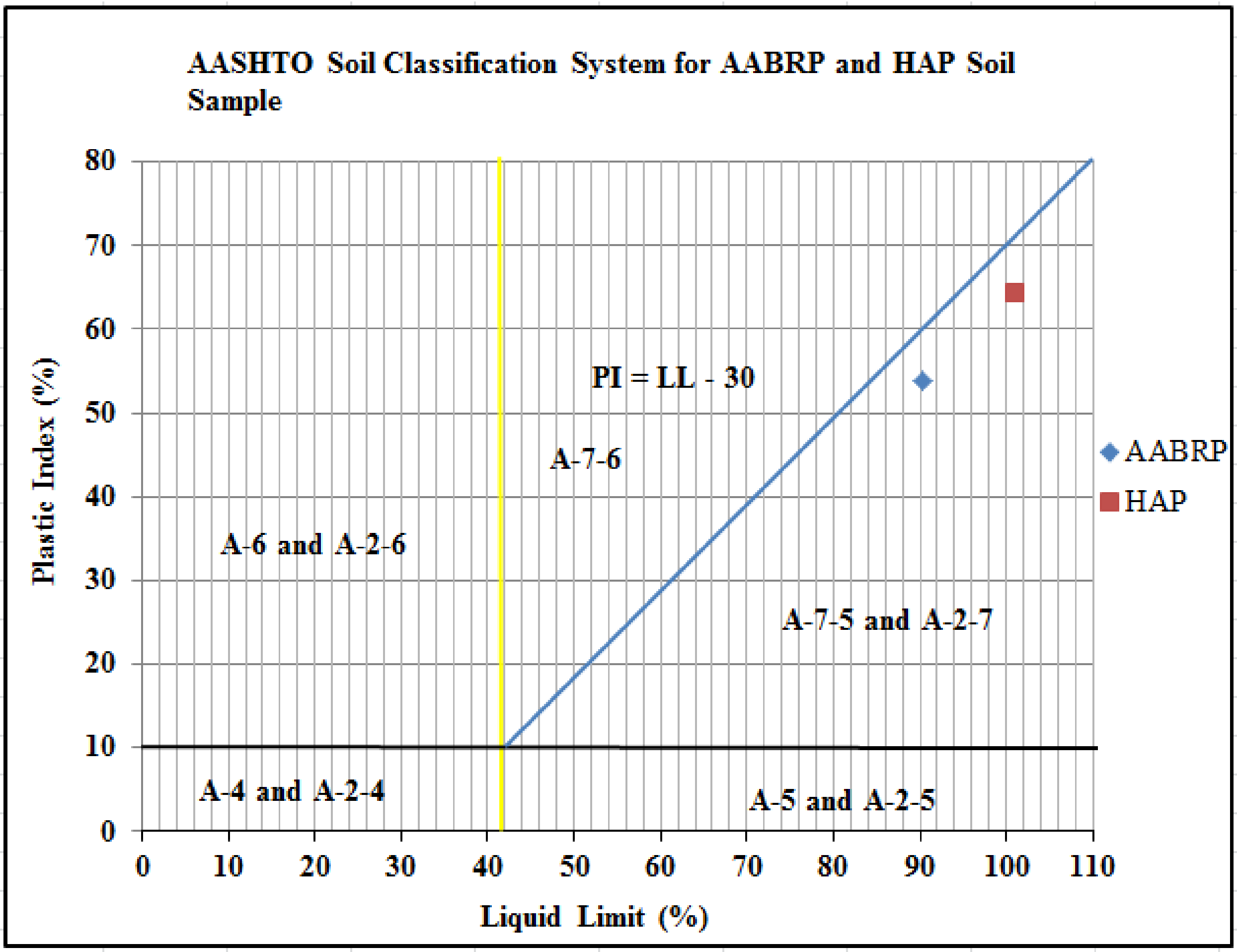


Figure 4. 3: Plasticity chart of untreated soil samples according to AASHTO

B. Unified soil classification system (USCS)

According to unified soil classification system as shown in table 4.4, and figure 4.4, both AABRP and HAP soil sample lie above the A- line in CH region, which means clayey soil with high plasticity.

Table 4. 4: Classification of soils based on USCS classification system

Sample Name	Quantity of grain size (%)				LL (%)	PI (%)	USCS Classification
	Gravel	Sand	Silt	Clay			
AABRP	0 %	9.6 %	35%	54.71%	90	54	CH
HAP	0%	3.5%	30.88%	65.62%	101	64	CH

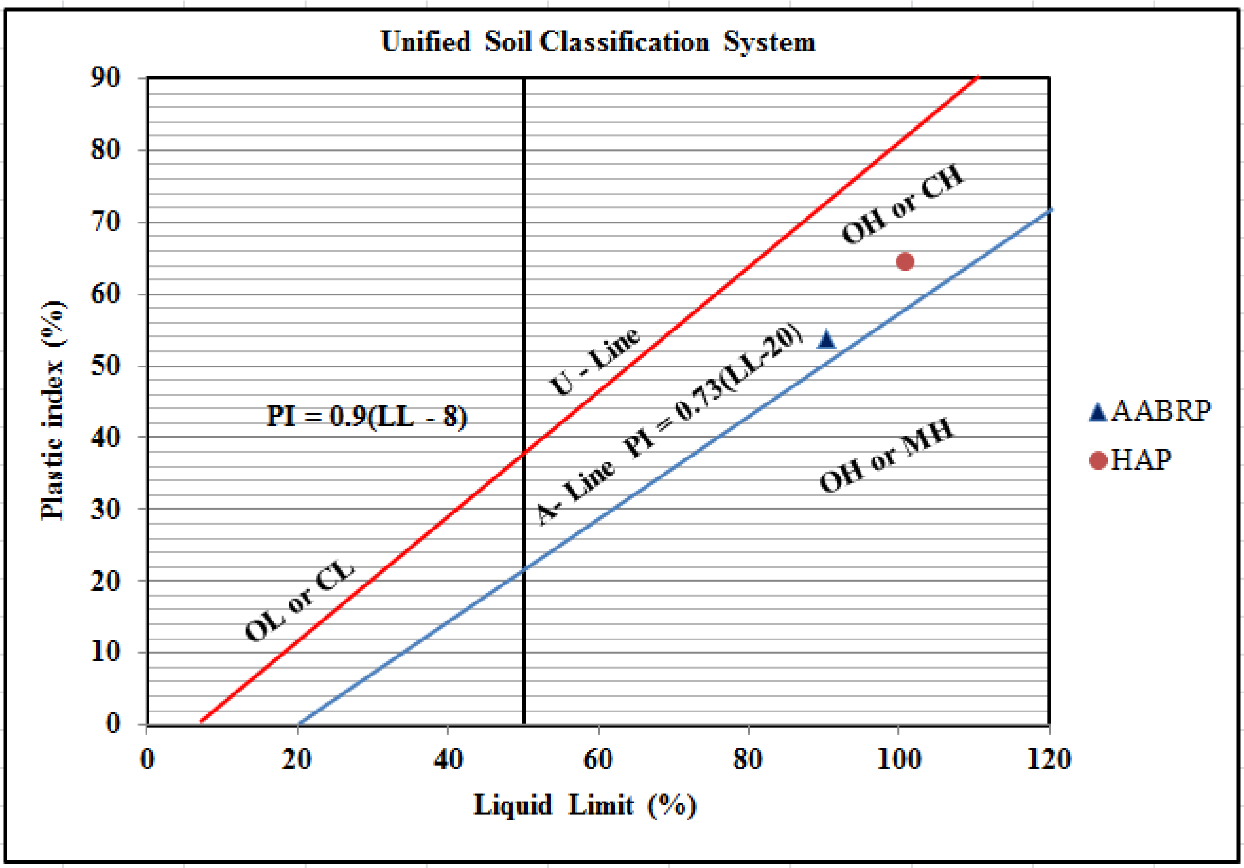


Figure 4. 4: USCS plasticity chart of AABRP and HAP soil samples

4.1.4 Specific Gravity

This test was conducted on fined grained particles of materials used for the study and summary of the test results are tabulated as followed in Table 4.5.

Table 4. 5: The Specific Gravity of AABRP and HAP Soil Samples

Sample Name	AABRP			HAP		
Specimen number	1	2	3	1	2	3
Pycnometer bottle number	A	B	C	A	B	C
WP = Mass of empty, clean pycnometer (grams)	47.1	27.1	47.1	27.1	47.1	47.1
WPS = Mass of empty pycnometer + dry soil (grams	57.3	37	57.3	37	57.3	57.3
WB = Mass of pycnometer + dry soil + water (grams)	103.95	82.25	103.9	82.23	103.95	103.9
WA = Mass of pycnometer + water (grams)	97.5	76	97.5	76	97.5	97.5
Specific gravity (GS)	2.72	2.71	2.68	2.70	2.71	2.68
Average GS	**2.71**			**2.70**		

Table 4.5 showed that BAARP sample has an average specific gravity of 2.71 and 2.70 for HAP sample. The specific gravity of solid particles of most soils varies from 2.5 to 2.9.For most of the calculations specific gravity (Gs) can be assumed as 2.65 for cohesion less soils and 2.70 for clay soils. This result indicated both sample is under clay soil.

4.1.5 Free swell index

Table 4. 6: Free swell index test result of the AABRP and HAP Soil Samples.

Sample Name	**Free swell index**
AABRP	**80**
HAP	**100**

This result indicated that the two soils were highly expansive soils. Soils are called highly expansive when the free swell index exceeds 50%, and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying (Ranjan, G., and Rao, A.S.R., 2002).

4.1.6 Compaction test result of soil sample

The AABRP soil sample has a maximum dry density of 1.353 g/cm3 and the optimum moisture content of 21.4 %.Besides, The HAP soil sample has a maximum dry density of 1.317 g/cm3 and the optimum moisture content is 24.5%.

Table 4. 7: Compaction test results of the AABRP and HAP Soil Samples.

Sample Name	AABRP				HAP			
Trial number	1	2	3	4	1	2	3	4
Water added (%)	8	10	12	14	10	12	14	16
NMC	9.70				10.5			
Average water content, (%)	17.47	19.33	21.10	23.77	20.54	22.51	24.26	26.42
Dry unit weight, (%)	1.224	1.315	1.351	1.316	1.2	1.27	1.315	1.28
MDD (g/cm3) from graph	1.353				1.317			
OMC (%) from graph	21.4				24.5			

The maximum dry density and optimum moisture content obtained are used to determine the strength to be attained during construction of a road especially subgrade layer. During road construction the CBR value is obtained using the compaction test result. And these CBR results used to determine the thickness of the sub-grade layer of a road construction.

4.1.7 Soaked CBR and CBR Swell of soil sample

According to OMC and MDD of the AABRP soil sample the Soaked CBR value is 0.98 % and CBR Swell value of 16.66 %.The HAP soil sample has also Soaked CBR value of 0.81 % and CBR Swell value of 18.29 %.

Table 4. 8: CBR and CBR Swell test result of the BAARP and HAP soil sample

Sample type	No. of Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		CBR at100% MDD	Swell (%)
			2.54mm	5.08mm	2.54mm	5.08mm		
AABRP	56	1.352	0.13	0.18	0.98	0.91	0.98	16.66
HAP	56	1.315	0.11	0.16	0.81	0.78	0.81	18.29

As Table 4.8 showed CBR test was determined at 2.54 and 5.08 penetration at the given maximum dry density and optimum moisture content in both soil sample. AABRP soil sample had 0.98 % soaked CBR value at maximum dry density with 16.66 % CBR swell and HAP soil sample had 0.81% CBR value with 18.29 % CBR swell. Therefore, based on the ERA requirement both the soils have lower CBR value and they are not suitable for subgrade in road construction. From test result of Table 4.8 both soil samples were black cotton soil so they required additives to stabilize. To achieve the objective of this study both soil sample should stabilize chemically using bagasse ash and cement.

4.2 Laboratory test results of stabilized back cotton soil

4.2.1 Atterberg limits

The amount of bagasse ash mixed with cement is added at the different ratio to the black cotton soil to reduce the plasticity index of the soil. The pozzolanic reaction is dependent on both access to water, pozzolanic materials and content of calcium ions, and there is no enough calcium in bagasse ash. This shows that certain amount of calcium from cement is necessary. The test result of Atterberg limit are tabulated in table 4.9 below and are illustrated in figure 4.5 and figure 4.6 below for AABRP and HAP soil samples respectively.

Table 4.9: Atterberg's limit test result of the stabilized black cotton soils.

Sample Name	Percent of Soil, Cement and Bagasse ash			LL	PL	PI	Percentage of redaction	ERA(2002) requirement of PI for subgrade	Compare result
	Soil	Bagasse ash	Cement						
AABRP	100	0	0	90	37	54	0	< 30	Control
	90	10	0	80	38	43	20.37		Slight reduction
	90	8	2	76	39	37	31.48		Slight reduction
	90	6	4	71	41	30	44.44		In range
	90	5	5	65	42	23	57.4		In range
	90	4	6	62	45	17	68.52		In range
	90	2	8	57	49	8	85.2		In range
	90	0	10	55	51	4	92.6		OK
HAP	100	0	0	101	37	64	0	< 30	Control
	90	10	0	93	38	55	14		Slight reduction
	90	8	2	84	42	42	34.4		Slight reduction
	90	6	4	79	43	36	43.8		Slight reduction
	90	5	5	73	46	27	57.8		In range
	90	4	6	66	50	16	75		In range
	90	2	8	64	52	12	81.3		In range
	90	0	10	60	54	6	90.6		OK

From Table 4.9, plastic index of both soil samples has improved through stabilization. From ERA manual specification the plasticity index of subgrade soil have been stated. The untreated plasticity index values determined in the laboratory were 54 % and 64 % for samples at BAARP and HAP respectively. Both samples have plastic index values which fall in the range of high plasticity index. With cement 10% treatment, the plastic index values becomes 4% and 6% respectively for BAARP and HAP samples. This lab result indicts high reduction occur at this ratio in both samples.

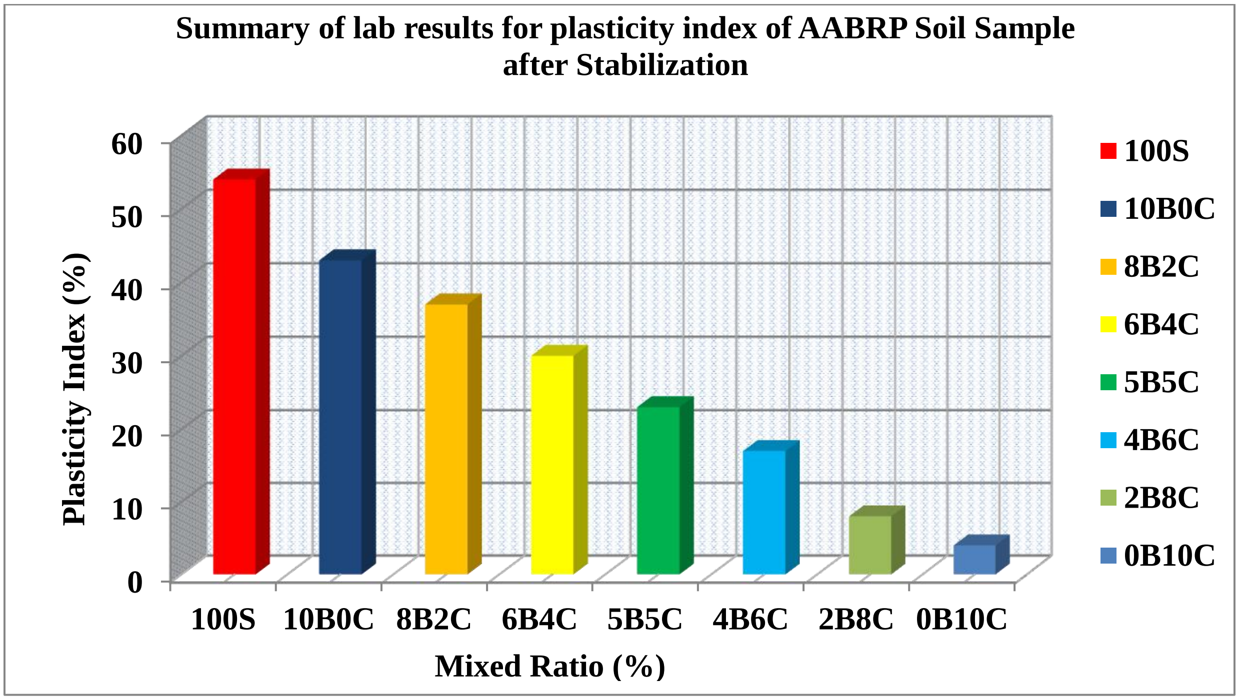


Figure 4. 5: Effect of addition of bagasse ash and cement on plasticity index of BAARP soil sample

From figure 4.5 it showed that highest reduction in plasticity index is observed while adding the maximum ratio of cement than the maximum ratio of bagasse ash. For cement mixed with bagasse ash stabilization the following observation has been made:

- Liquid limit decreases with increasing percentage of cement ratio (0%-10%) while the percentage of bagasse ash decreased from (10% - 0%) proportionally
- As the percentage of cement in cement – bagasse ash mix increased from 0% - 10% the percentage of bagasse ash in cement – bagasse ash mix reduced from 10% - 0%, this

increasing percentage of cement content and decreasing bagasse ash in total mix design 10% decreased the liquid limit.

- Generally, liquid limit decreased while plastic limit increased and consequently plastic index reduced with the addition of bagasse ash and cement.

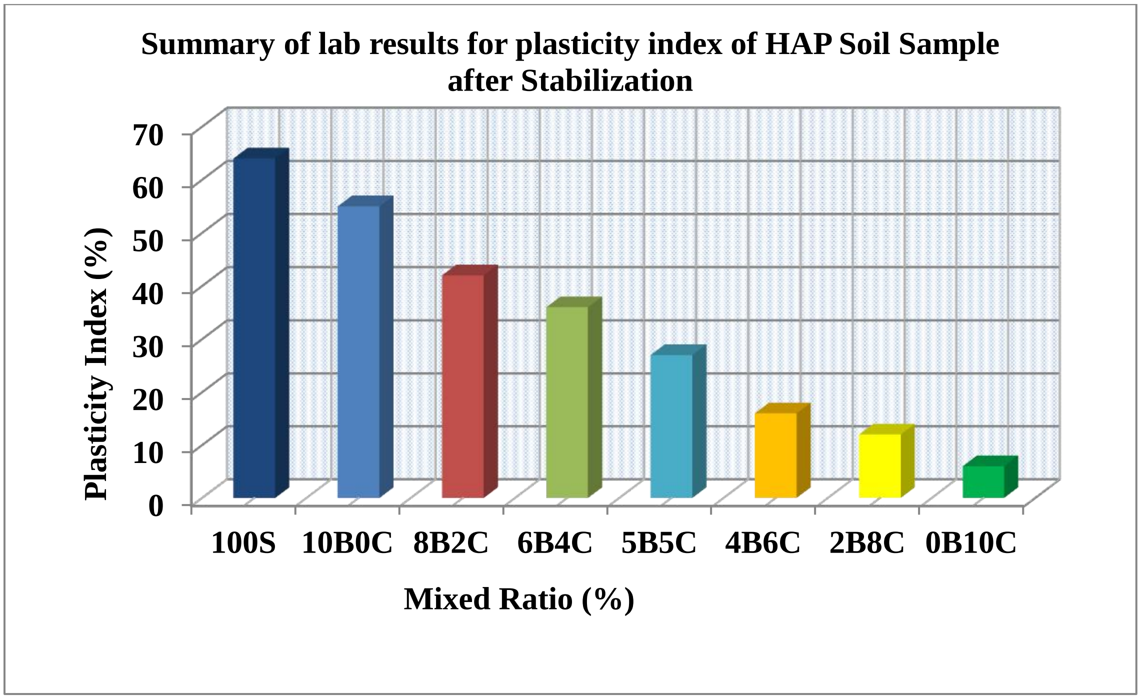


Figure 4. 6: Effect of addition of cement and bagasse ash on plasticity index of HAP soil sample.

In Figure 4.6 there is a significant reduction in plasticity associated with addition of bagasse ash in combination with cement is expected as more calcium is available for cation exchange to take place and also this effect could be attributed to the combine action of partial replacement of plastic soil particles with non-plastic particles of bagasse ash, and the ionic exchange of cement and clay minerals of the soils. These led to flocculation and agglomeration of the clay particles which in turn reduces the plasticity of the treated soil. Generally, both the additives reduce the clay particle or plasticity of the given soil sample in all mix ration. However, the percent reduction is different since the amount of chemical reactions in the stabilizer is decrease as well as increase.

4.2.2 Free swell index

The free swell index of black cotton soil decrease when the ratios of cement additives are added. The free swell index results of stabilized soil is presented in Table 4.10 below and are illustrated in figure 4.7 below for sample BAARP and HAP.

Table 4.10: Free swell test result of stabilized black cotton soil.

Sample Name	Percent of Soil, Bagasse ash and Cement			V k	V w	FSI (%)	Percentage Of reduction	IS 2720 (Part XL) requirement	Compare result
	Soil	Bagasse ash	Cement						
BAARP	100	0	0	10	18	80	0	FSI > 50 very high expansive	Control
	90	10	0	10	17	70	12.5		Slight reduction
	90	8	2	10	15.5	55	31.25		Slight reduction
	90	6	4	10	14	40	50		In range
	90	5	5	10	13.5	35	56.25		In range
	90	4	6	10	12.5	25	68.75		In range
	90	2	8	10	11.9	19	76.25		In range
	90	0	10	10	12	20	75		OK
HAP	100	0	0	10	20	100	0	FSI > 50 very high expansive	Control
	90	10	0	10	18.5	85	15		Slight reduction
	90	8	2	10	17	70	30		Slight reduction
	90	6	4	10	14.5	45	55		In range
	90	5	5	10	14	40	60		In range
	90	4	6	10	13	30	70		In range
	90	2	8	10	12.5	25	75		In range
	90	0	10	10	12.5	25	75		OK

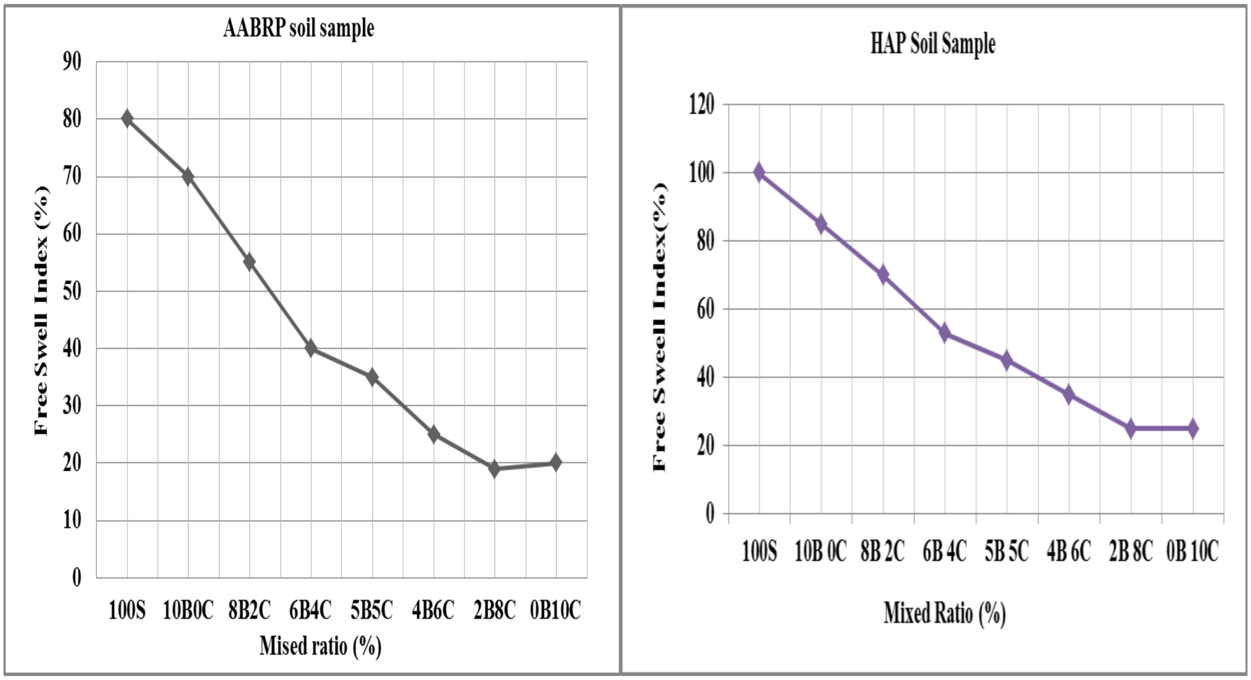


Figure 4. 7: Free swell index for AABRP and HAP soil samples at different stabilizer ratio.

The free swell index of the AABRP and HAP soil samples have very high swelling and expansiveness properties. However mixing the poor soil with different additives which are rich in calcium and silicate reduce the potential of swelling and expansiveness of the black cotton soils. As shown in Figure 4.7, the free swell index of the samples has decreased with increase in cement ration for the soil samples. But slight reduction is observed with higher ratio of bagasse ash content and lower ratio of cement content added. Except 10% bagasse ash alone and 2% cement with 8% bagasse ash all the mix ratios were under the specification. In addition, 10% cement and 2% bagasse ash mixed with 8% cement has got maximum free swell index value. This reduction in free swell index indicated that removing potentially expansive soil is important especially to the subgrade soil to stay for long period of time without failure.

4.2.3 Compaction characteristics

The effects of the stabilizers on maximum dry density and optimum moisture content of the expansive soils are illustrated in Table 4.11, Figure 4.8 and Figure 4.9 below.

Table 4.11: Effect of bagasse ash -cement content addition on Moisture Density Relation

AABRP Soil Sample				HAP Soil Sample			
Mix-Proportion of additive (%)		MDD (gm/cm3)	OMC (%)	Mix-Proportion of additive (%)		MDD (gm/cm3)	OMC (%)
bagasse as	cement			bagasse as	cement		
0	0	1.353	21.4	0	0	1.317	24.5
10	0	1.35	23	10	0	1.3	25.05
8	2	1.332	25.41	8	2	1.287	26.56
6	4	1.297	26.5	6	4	1.26	28.6
5	5	1.27	27	5	5	1.252	29.74
4	6	1.262	28.9	4	6	1.241	31.22
2	8	1.231	28.9	2	8	1.229	32.1
0	10	1.228	30.2	0	10	1.22	38.15

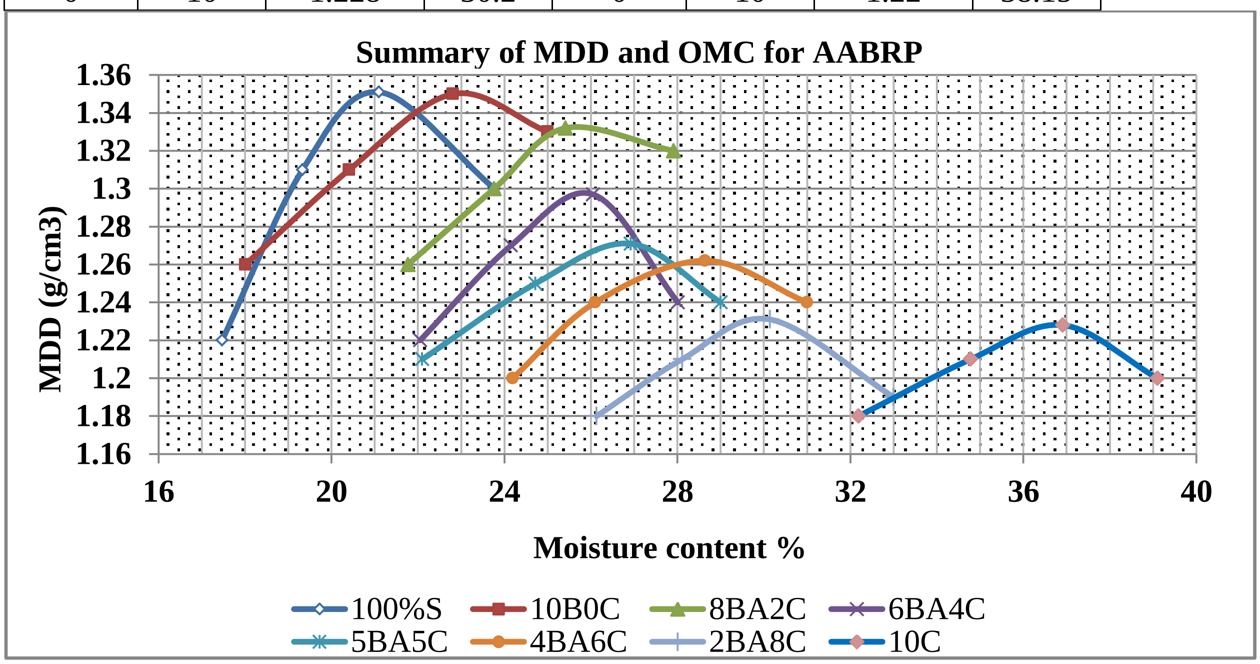


Figure 4. 8: Summary of OMC and MDD of cement mixed with bagasse ash for AABRP.

The result of optimum moisture content and maximum dry density showed that, as the stabilization cement has to increase, the optimum moisture content increase but the maximum dry density decrease.

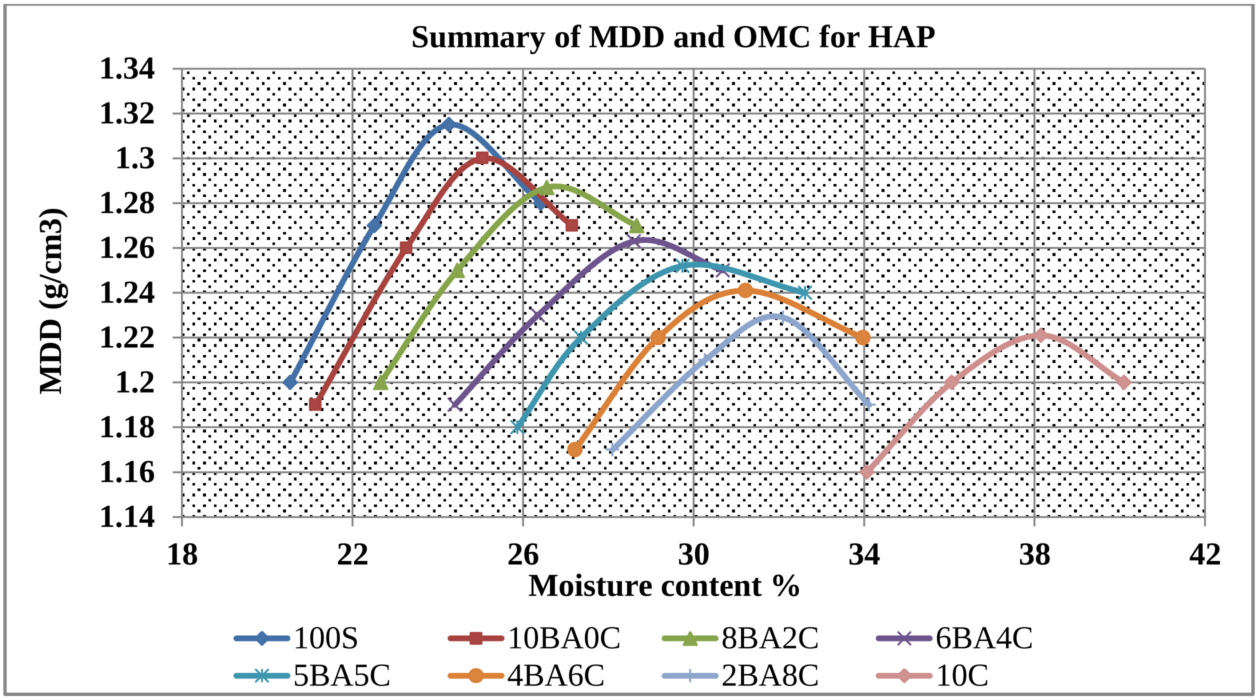


Figure 4. 9: Summary of MDD and OMC of cement mixed with bagasse ash for HAP

The amount of bagasse ash added did not bring a significant change in the optimum moisture content and maximum dry density. The addition of cement and bagasse ash changes the optimum moisture content and maximum dry density of black cotton soils because the effects of cation exchange and short-term pozzolanic reactions between cement and the soil results in flocculation and agglomeration of clay particles leading to textural changes. These results are in consistent obtained by Afes and Didier (2000).

The increase in the optimum moisture content was mainly due to;

- The optimum moisture content of the soil increases with an increase cement and bagasse ash, because cement and bagasse ash is finer than the soil. The more fines the more surface area, so more water is required to provide well lubrication.
- The increase in OMC due to addition of cement and bagasse ash caused by the absorption of water by cement and bagasse ash this implies that more water is needed in order to compact the black cotton soil with cement and bagasse ash mixture. So cement and bagasse ash effectively dries wet soils and provides an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions. In general it can be utilized in improving the workability of wet soils.

4.2.4 CBR Test Result

CBR is a parameter which is used to measure the strength of subgrade soil. The soil stabilized by hydrated cement showed more improvement than bagasse ash. The CBR Test Result of stabilized soil and ERA requirement is presented in Table 4.12 below for sample BAARP and HAP.

Table 4.12: Soaked CBR test results of the stabilized two black cotton soil samples.

Sample Name	Percent of Soil, Bagasse ash and cement			Un cured CBR (%)	3 day cured CBR (%)	7 day cured CBR (%)	14day cured CBR (%)	ERA(2002) requirement	Compare result
	soil	Bagasse ash	Cement						
BAARP	100	0	0	0.98	1.14	1.37	1.72	CBR > 3%	Control
	90	10	0	1.24	1.43	1.63	2.67		Slight increased
	90	8	2	2.93	4.55	7.80	12.35		Slight increased
	90	6	4	5.20	9.75	17.55	23.4		In range
	90	5	5	5.53	11.38	20.48	30.88		In range
	90	4	6	6.18	12.35	22.10	33.48		In range
	90	2	8	10.7	20.48	28.93	40.95		In range
	90	0	10	10.4	20.15	28.60	40.63		OK
HAP	100	0	0	0.81	1.11	1.30	1.65	CBR < 3 %	Control
	90	10	0	1.14	1.30	1.56	2.44		Slight increased
	90	8	2	2.6	3.58	7.15	10.08		Slight increased
	90	6	4	5.04	8.78	16.25	20.15		Slight reduction
	90	5	5	5.36	10.40	19.50	26.65		In range
	90	4	6	5.85	11.70	20.80	29.25		In range
	90	2	8	10.1	19.83	28.60	38.03		In range
	90	0	10	9.75	19.50	28.28	37.39		OK

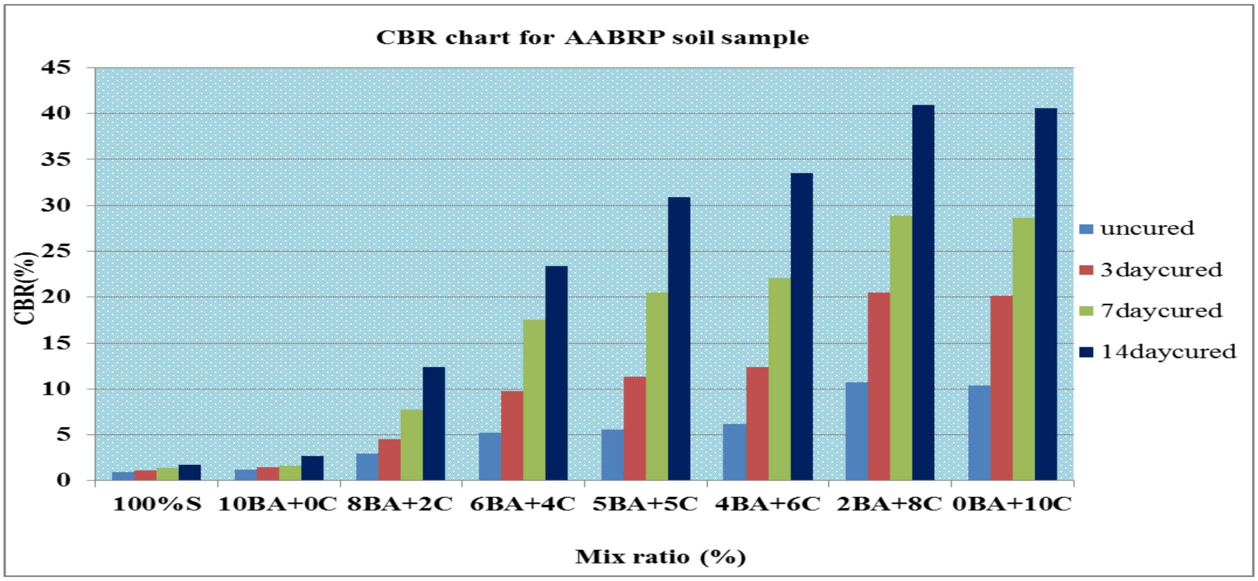


Figure 4. 10: Summary of uncured and cured CBR test results of AABRP soil Samples

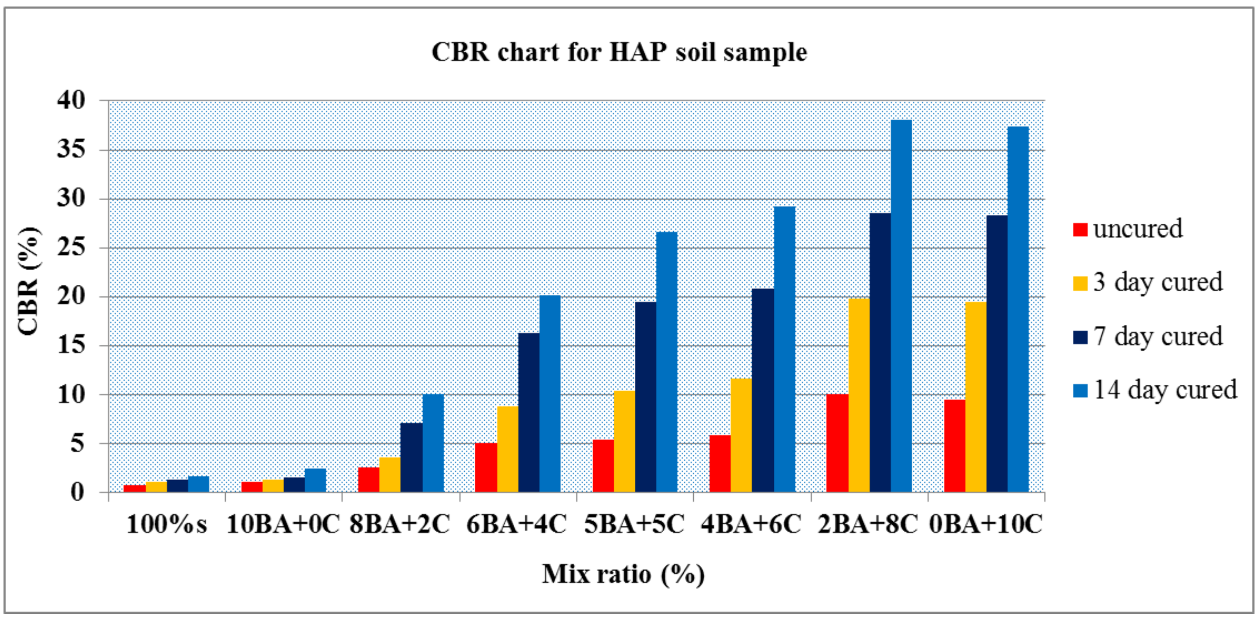


Figure 4. 11: Summary of uncured and cured socked CBR test results of HAP soil Samples

The CBR increased with increasing cement content than increasing bagasse ash content. A slight improvement has been observed when bagasse ash alone has added to the soil the reason is due to inadequate amount of calcium required for the formation of Calcium – Silicate – Hydrates (CSH) which is the major element for strength gain. Bagasse ash could be used in admixture stabilization with more potent stabilizers, such as cement to reduce the cost of stabilization and climate pollution. Results also shows that curing enhances the strength development of black cotton soil treated with cement plus bagasse ash but curing has an insignificant change when black cotton soil is treated with bagasse ash only. As shown in the Table 4.12 the cured CBR value of the AABRP natural soil increases from a value 0.98 to 1.14, 1.37 and 1.72 for 3, 7 and 14 cured days respectively. The result of 10% bagasse ash alone treated AABRP soil sample had the cured CBR values increases from 1.24 to 1.43, 1.63 and 1.67 for 3,7 and 14 cured days respectively. Detailed results of the stabilized soil samples are given in the appendix.

4.2.5 CBR Swell Test Result

The cement and bagasse ash mixtures compacted in CBR molds at Optimum moisture content and maximum dry density gauged for swelling properties before and after soaking for four days to evaluate the percent of swell. The test result at different ratios was illustrated in Figure 4.12 and Figure 4.13 below.

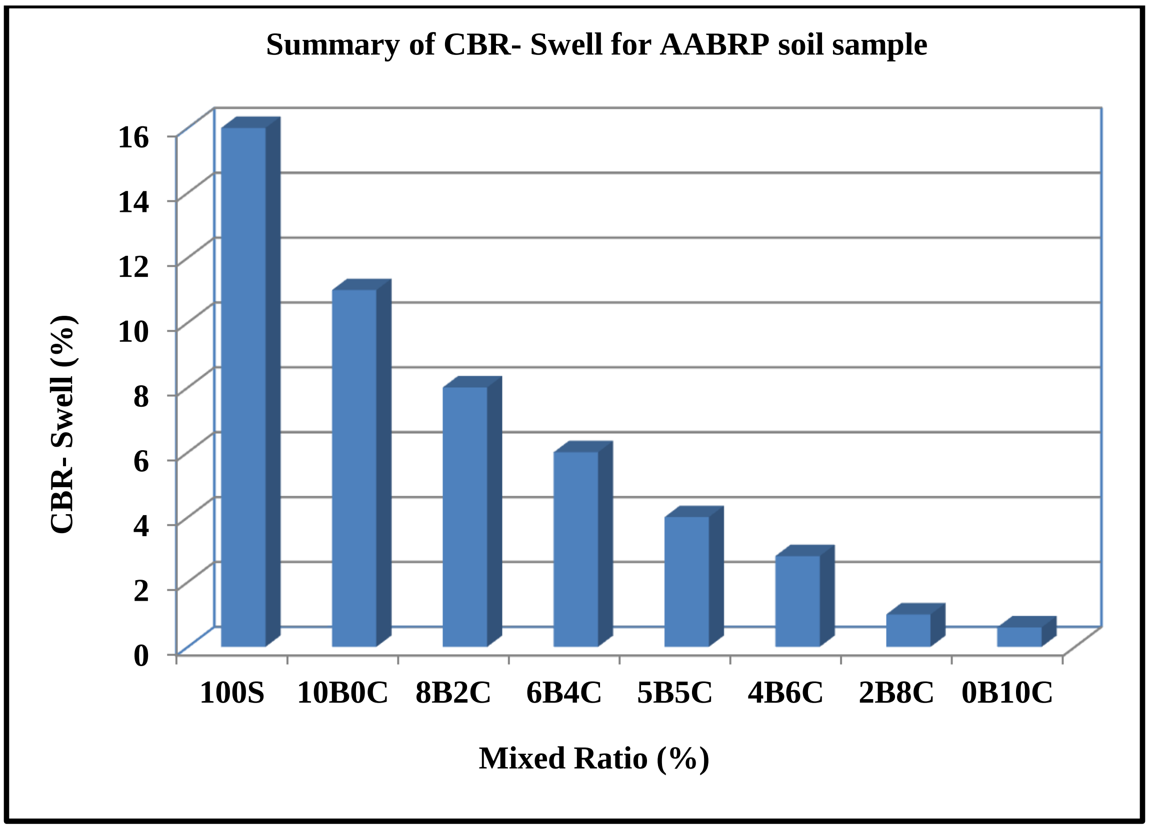


Figure 4. 12: CBR Swell test result of stabilized and natural soil sample for AABRP

The Figure 4.12 shows 100% soil has the properties of swelling and potentially expansive soil. However, when bagasse ash and cement mix with different ratio the CBR swell reduce. Cation reactions from the additives with the clay particles make a strong bond and don not make crack and shrinkage. Soil sample had 17% value of CBR swell but when 10% bagasse ash added it reduce to 11%.This indicate slight reduction in CBR swell. When it mixed with cement it improve the black cotton soil strongly. At cement 8% and bagasse ash 2% the CBR swell is 1% this indicate the subgrade is strong and suitable for road construction.

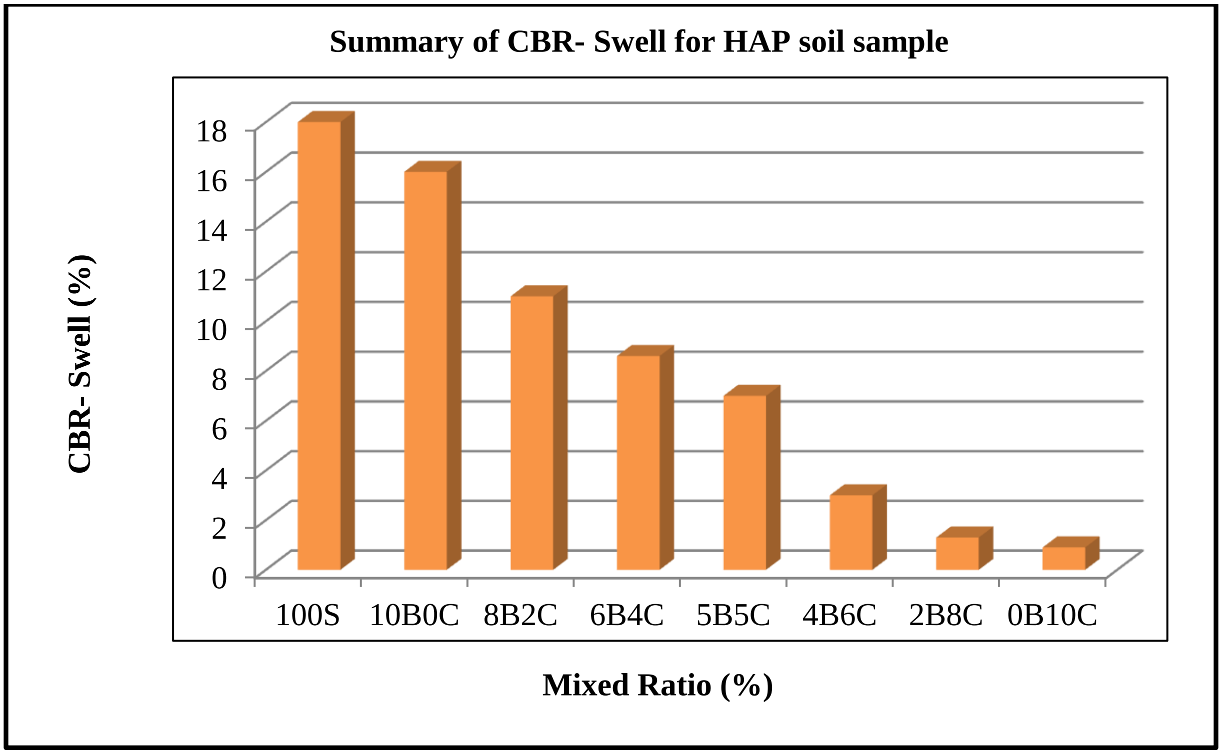


Figure 4. 13: CBR Swell test result of stabilized and natural soil sample for HAP

The decrease in CBR Swell of black cotton soil is due to cation exchange and flocculation and agglomeration of the soil particles. This is also due to replacement of some the volume that is previously occupied by expansive clay minerals by bagasse ash-cement. HAP Soil sample had 18% value of CBR swell but when 10% bagasse ash added it reduce to 16%. This indicates slight reduction in CBR swell. When it mixed with cement it improve the black cotton soil strongly. At cement 8% and bagasse ash 2% the CBR swell is 1.3% this mix ratio indicates the swell potential of the black cotton soil is reduced and make a strong bond with each other. Using both the stabilizers improve the stability and strength of the subgrade soils. The strength of subgrade is the principle factor in determining the thickness of the pavement, but deterioration due to frost action must also be taken into account. The strength of subgrade is associated on CBR scale.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

According to laboratory test results, specification, manuals and interpretation the following conclusions are made.

- ✓ The test result showed that both subgrade soil considered for this study were A- 7-5 as per AASHTO soil classification system and CH as USCS. The plastic index for BAARP soil sample is 53.77 % and 64.32 % for HAP soil sample. And also the laboratory result showed that MDD of 1.351 g/cm^3 and 1.315 g/cm^3 on AABRP and HAP samples respectively. Besides, soaked CBR value of AABRP soil sample 0.98 % and 0.81% of HAP soil sample. The Engineering properties of the natural soil sample were black cotton soil. Both soil samples have high plasticity index, very low load bearing capacity and high swelling potential and those properties make the subgrade unsuitable without additives and stabilizers.
- ✓ Adding bagasse ash 10% alone has 43% plastic index for AABRP soil sample and 55% for HAP which is small reduction. Bagasse ash alone has slight reduction in plasticity index, CBR, free swell Index and does not improve the CBR value of soils due to the presence of only reactive silica with the low amount of calcium content in bagasse ash material. The optimum moisture content increased while the maximum dry density values decreased with increasing cement and decreasing bagasse ash content. The addition of cement mixed with bagasse ash improves the CBR value
- ✓ The addition of bagasse ash in combination with cement improved the CBR value. The improvement is more significant when the sample is cured. Hence, combination of bagasse ash and cement can strongly improve the strength of the black cotton soil.
- ✓ Bagasse ash is not effective standalone stabilizer for highly plastic black cotton soil. However, bagasse ash mixed with cement can effectively stabilize this poor soil. There for mixing of the two stabilizers can effectively treat the poor geotechnical properties of the black cotton soil in road construction, reduce pollution and reduce the amount of cement required for stabilization purpose, as a result, it minimizes the cost of the construction.

5.2. Recommendation

According to the findings of this research, the following recommendations are forwarded to next researcher:-

- Sugarcane bagasse ash as investigated in this research work can be used as a soil stabilizing material in combination with cement with economic and environmental benefits. Therefore concerned bodies like sugar industries and government entities should be made aware about this potential soil stabilizing material and promote its standardized production and usage.
- The sugar factories in collaboration with higher education organizations in the country should work together and establish a research team to further study the use of bagasse ash as a soil stabilizing material on different types of soils.
- Additional curing time effect on all geotechnical laboratory tests should be performed.
- This study was conducted by taking the small ratio of bagasse ash material. It is recommended to take the high ratio of bagasse ash up to 50% to increase the strength of subgrade soil and reduce the cost of cement content.
- This study was taken only two high expansive soil sample. It is recommended to take a large number of soil samples which characterizes the whole study area
- The present study was conducted by taking limited parameter such as atterberg limit, free swell index, moisture density relation, CBR and CBR swell potential on stabilization by cement and bagasse ash. It is recommended to test additional parameter like unconfined compressive strength and mineralogical tests should also be performed to have more realistic test results.
- Studies should be made to check the secondary reaction of the bagasse ash using more advanced methods like X-ray Diffraction (XRD) Analysis, Thermal Analysis (TGA) and Scanning Electron Microscopy (SEM).
- The effects of different fineness of the bagasse ash should be studied

REFERENCES

1. AASHTO, "Standard specification for transportation materials and methods of sampling and testing," America, 1995.
2. Al-Rawas, A.A., and Mattheus, F.A., "Expansive Soils Recent Advanced in Characterization and treatment*," Proceedings and Monographs in Engineering, Water and Earth Sciences,* Netherlands, 2006.
3. Amu, O.O., et al., "Geotechnical properties of lateritic soil stabilized with sugarcane straw ash,"American Journal of Scientific and Industrial Research, 2011.
4. ASTM C618 – 03, "Specification for Fly Ash and Raw or Calcined Natural Pozzolanas for Use as a Mineral Admixture in Portland Cement Concrete," *American Society for Testing and Materials*, Vol.04.02, Philadelphia, 2003.
5. Bell, F.G, "Lime Stabilization of Clay Minerals and Soils," Journal of Engineering Geology, Vol. 42, pp. 223-227, 1996. As cited by Prof. Khairul, A.B., "The Effect of Catalyst on Soil Stabilization by Application of Lime," University of Malaysia, 2009.
6. Bell, F.G, "Lime Stabilization of Clay Minerals and Soils," Journal of Engineering Geology, Vol. 42, pp. 223-227 1996. As cited by Thian, S.Y, and Lee C.Y, "Shear Strength Behaviour of Mining Sand Treated with Clay and Hydrated Lime," International Journal of Civil and Structural Engineering Volume 3, No 1, 2012.
7. Bowles, J., "Engineering properties of soil and their measurements," McGraw-Hill Boston, 4th edition, 1992.
8. Brooks, R.M., "Soil Stabilization with Fly ash and Rice Husk Ash, *"International Journal of Research and Reviews in Applied Sciences*, Volume 1, Issue 3, 2009.
9. Chen, F.H., "Foundation on expansive soils," Elsevier, Amsterdam, 1988.
10. Chittaranjan, M., et al., "Agricultural wastes as soil stabilizers, "*International Journal of Earth Sciences and Engineering*, Volume 04, No 06 SPL, pp 50-51, October 2011.
11. Chou, L., "Lime Stabilization: Reactions, Properties, Design, and Construction," *State of the Art Report 5*, Transportation Research Board, Washington, DC, 1987.
12. Christopher, M.G., (2005), "Stabilization of Soft Clay Subgrades in Virginia Phase I Laboratory Study," MSc. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, 2005.

13. Chusilp, N., et.al, “Development of bagasse ash as a pozzolanic material in concrete,” *Asian Journal on Energy and Environment*, p. 149-159, 2009.
14. Dallas, N. Little, & Syam, N., “Recommended Practice for Stabilization of Subgrade Soils and Base Materials, “Texas Transportation Institute Texas A&M University College Station, Texas, 2009.
15. Department of the Army, the Navy, and the Air Force, “Technical Manual Soil Stabilization for Pavements,” October 1994.
16. Ethiopia plans to take 2.5pc of the world sugar market, 30 December 2010, 19:54,http://www.ethiopiafirst.info/news/index.php?option=com_content&view=article&id=412:ethiopia-plans-to-take-25pc-of-the-world-sugar-market&catid=1:latestnews&Itemid=1. As cited by Hailu, B., “Bagasse ash as cement replacing material,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2011.
17. Ethiopian Roads Authority, “Pavement Design Manual,” Addis Ababa, 2002.
18. Gourley, C., et.al, 1993 “Expansive Soils,” first international symposium on engineering characterization of arid soils, transport research, UK.
19. Govindarajan., D., and Jayalakshmi, G., “XRD, FTIR and SEM studies on calcined sugarcane bagasse ash blended cement,” Scholars Research Library, Department of Physics, Annamalai University, Annamalai Nagar, Tamilnadu, India, 2011.
20. Guyer, J. P.,”Introduction to Soil Stabilization in Pavements,” New York, 2011.
21. Hailu, B., “Bagasse ash as cement replacing material,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2011.
22. Laxmikant, Y., et al., **“**Comparison of Fly Ash and Rice Husk Ash Stabilized Black Cotton Soil, ”*International Journal of Earth Sciences and Engineering,* Volume 04, No 06 SPL, pp. 42-45, 2011.
23. Lees, G., et al., “Sodium Chloride as an Additive in Lime-Soil Stabilisation,” *Highway Engineer*, Vol. 29, No. 12, pp. 2-8, 1982.
24. Little, D.N., “Handbook for Stabilization of Pavement Subgrades and Base Courses with Lime,” Kendall/Hunt, Iowa, 1995.
25. Lyon Assoiciates, Inc, “Laterite and Lateritic soils and other problems soils of Africa,” Baltimore, 1971. As cited by Nebro, D., “Stabilization of Potentially Expansive Subgrade Soil Using Lime and Con-Aid,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2002.
26. Makusa, G.P., “Soil Stabilization Methods and Materials in Engineering Practice,” Department of Civil, Environmental and Natural resources engineering, Luleå University of Technology, Luleå, Sweden, 2012.

27. Medjo, E., and Riskowiski, G., “A Procedure for processing mixtures of soil, cement and sugar cane bagasse,” *Agricultural Engineering International Journal of Scientific Research and Development,* Vol.III, pp 1-6, 2004.
28. Mu’azu, M.A., “Evaluation of Plasticity and Particle Size Distribution Characteristics of Bagasse Ash on Cement Treated Lateritic Soil,”*Leonardo Journal of Sciences,* p. 137-152, January-June 2007.
29. Mu’azu, M.A., “Evaluation of Plasticity and Particle Size Distribution Characteristics of Bagasse Ash on Cement Treated Lateritic Soil,”*Leonardo Journal of Sciences,* p. 137-152, January-June 2007.
30. Murthy, V.N.S., “principles and practices of soil mechanics and foundation engineering,” New York,
31. Nebro, D., “Stabilization of Potentially Expansive Subgrade Soil Using Lime and Con-Aid,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2002.
32. Nelson, D., and Miller, J., “Expansive Soils: Problems and Practices in Foundation and Pavement Engineering,” New York, 1992.
33. Osinubi, K. J., “Lime modification of black cotton soils,” *Spectrum Journal,* Vol. 2, pp. 112 – 122, 1995.
34. Osinubi, K. J., and Thomas, S. A., *“*Influence of Compactive Effort on Bagasse Ash Treated Soils,” *Nigerian Journal of Soil and Environmental Research,* Vol. 7, pp. 92-101, 2007.
35. Osinubi, K.J., et.al, “Bagasse Ash Stabilization of Lateritic Soil,” *Appropriate Technologies for Environmental Protection in the Developing World,* p. 281-290, 2009.
36. Ranjan, G., and Rao, A.S.R., “Basic and Applied Soil Mechanics,” New Delhi, 2002.
37. Sabat, A.K., “Utilization of Bagasse Ash and Lime Sludge for Construction of Flexible Pavements in Expansive Soil Areas, *“Electronic Journal of Geotechnical Engineering, (EJGE),* Vol. 17, 2012.
38. Teklu, D., “Examining the Swelling Pressure of Addis Ababa Expansive Soil,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2003.
39. Tesfaye, A., “Chemical Treatment of “Black Cotton” Soil to make it Usable as a Foundation Material,” MSc. Thesis, Addis Ababa University, Addis Ababa, October 2001.
40. Tilahun, D., “Influence of Drainage Condition on Shear Strength Parameters of Expansive Soils,” MSc. Thesis, Addis Ababa University, Addis Ababa, 2004.
41. Veisi, M., et al., “Accelerated Stabilization Design of Subgrade Soils,” Center for Transportation Infrastructure Systems (CTIS) the University of Texas, Research Report 0-55691, 2010.
42. Whitlow, R., “Basic Soil Mechanics,”3rd ed. Edinburgh Gate: Addison Wesley Longman Limited, 1995.

APPENDIX

Appendix A: Laboratory Test Result of AABRP Soil sample

1) Natural Moisture Content

Sample location	AABRP		
Can number	A	B	C
Mass Of Moisture can (Mc)	20	20	21
Mass of moisture can + Mass of moist soil (Mcms)	100	105	110
Mass of Moisture can + mass of oven dried soil(Mcds)	75	79	85
Mass of water (Mw)	25	26	25
Mass of dry soil (Ms)	55	59	64
Water Content(w) %	45	44	39
Average water content(w) %	43		

2) wet sieve analysis

Sieve size in mm	Mass of retained (gm.)	Percentage retained (%)	Percentage of commutative retained %	percentage of finer particle %
4.75	0.0	0	0	100.00
2.000	14.53	2.91	2.91	97.09
0.850	8.56	1.71	4.62	95.38
0.425	6.68	1.34	5.96	94.05
0.30	5.12	1.02	6.98	93.02
0.15	7.65	1.53	8.51	91.49
0.075	5.58	1.12	9.63	90.38
PAN	451.9	90.38	100	0.00
Total	500	**100.0**		

3) Hydrometer Analysis

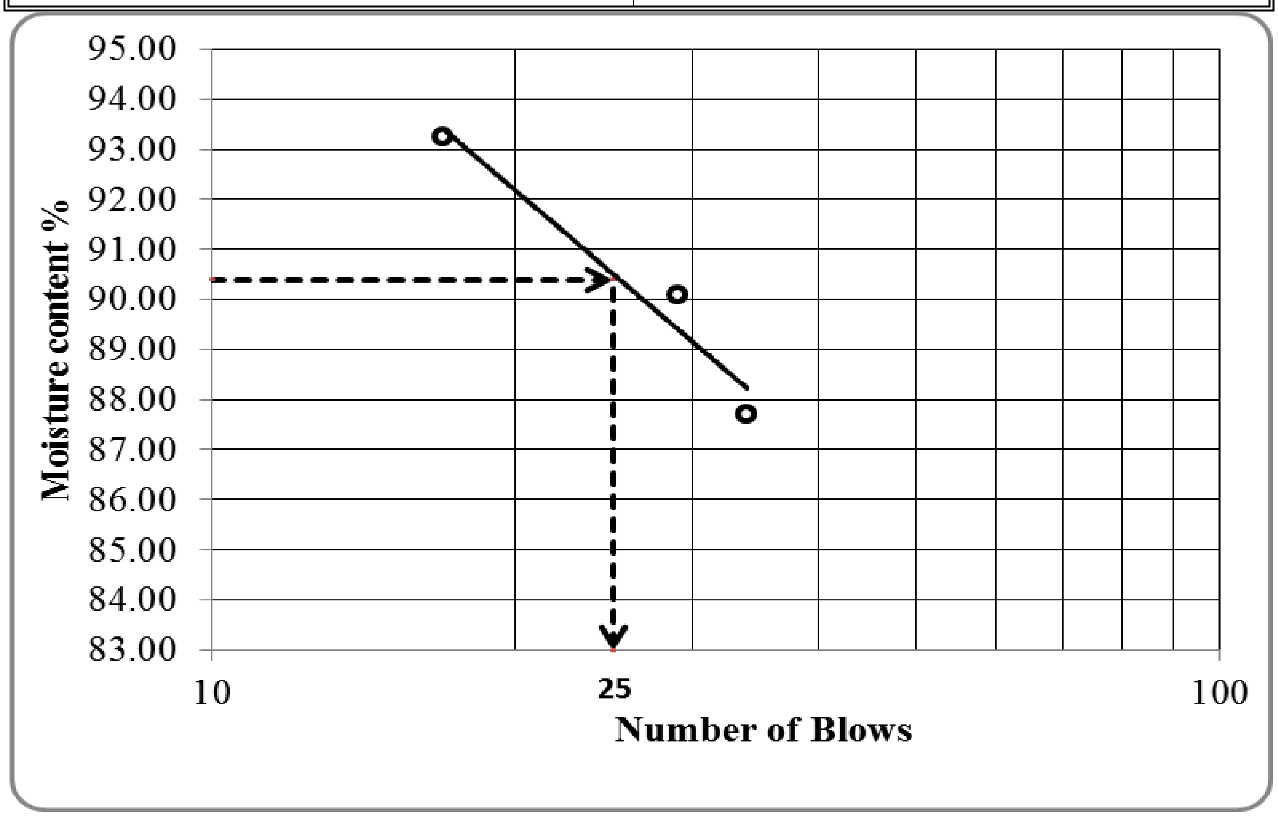
Time	3:00	3:02	3:06	3:14	3:29	3:59	4:59	6:59	8:59	16:59		
Elapsed time t in, min	1	2	4	8	15	30	60	120	240	480	960	1440
Actual hydrometer reading	50	48	46	45	43	42	41	39	38	36	33	31
Eff. depth, L (cm)	8.1	8.4	8.8	8.9	9.2	9.4	9.6	9.9	10.1	10.4	10.9	11.2
Temperature (Co)	20	20	20	20	20	20	20	25	25	20	20	20
K	0.01344	0.01344	0.01344	0.01344	0.01344	0.01344	0.01344	0.01267	0.01267	0.01344	0.01344	0.01344
CT	0	0	0	0	0	0	0	1.13	1.13	0	0	0
Par. diameter D (mm)	0.038	0.027	0.02	0.014	0.011	0.008	0.005	0.004	0.003	0.002	0.0014	0.0011
Value of 'a' for G_S	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Corrected hydrometer reading for present particle	44	42	40	39	37	36	35	34.13	33.13	30	27	25
% Finer D,P(%)= Rd+a/ Mo	87.12	83.16	79.2	77.22	73.26	71.28	69.3	67.577	65.597	59.4	53.46	49.5
corrected (Pa)	80.24	76.59	72.94	71.12	67.47	65.65	63.83	62.24	60.42	54.71	49.24	45.59

4) Combined sieve analyses

Grain Size (mm)	% passing	Combined % passing
9.5	100.0	100.0
4.75	100.0	100.0
2	97.1	97.1
0.85	95.4	95.4
0.425	94.1	94.1
0.25	93.3	93.3
0.106	92.8	92.8
0.075	90.4	90.4
0.037	87.12	80.24
0.027	83.16	76.59
0.02	79.2	72.94
0.014	77.22	71.12
0.01	73.26	67.47
0.0073	71.28	65.65
0.0052	69.3	63.83
0.0037	67.577	62.24
0.0027	65.597	60.41
0.002	59.4	54.71
0.0014	53.46	49.24
0.0011	49.5	45.59

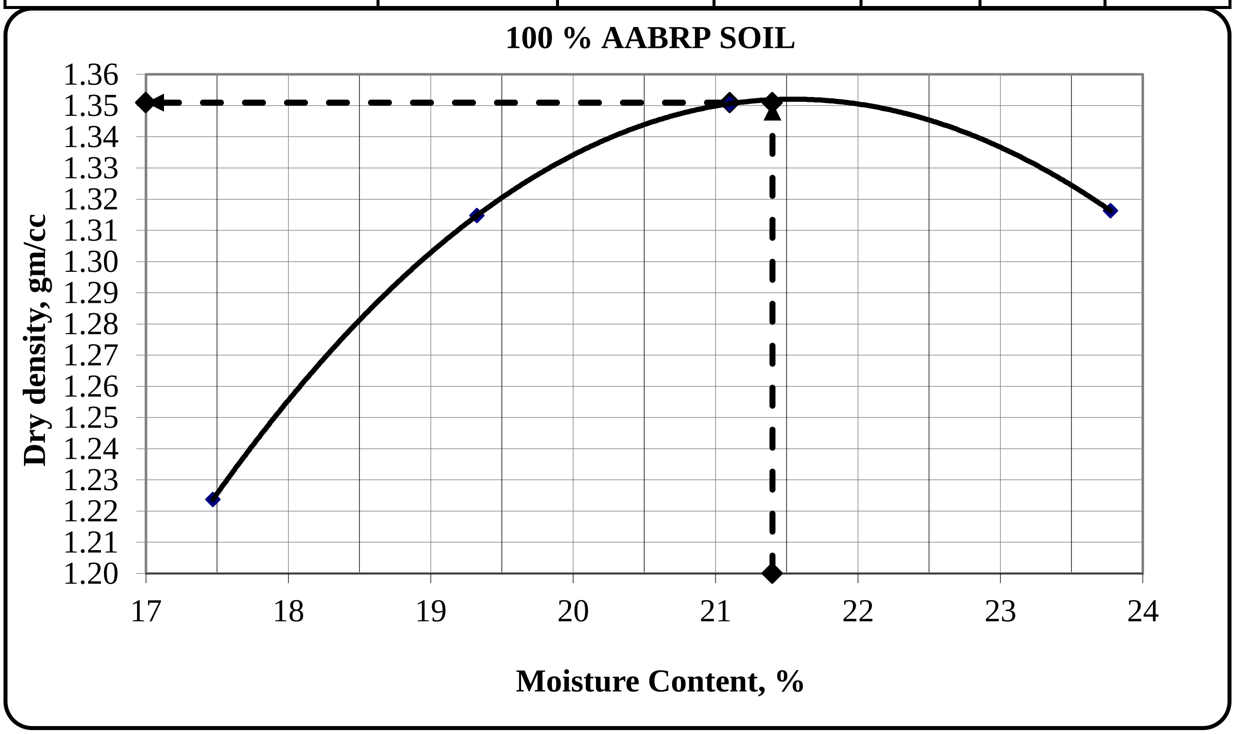
5) Atterberg Test Result

description		100 % AABRP Soil				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	29	17		
Container No.		C-4	H	F	G	N
Mass of wet soil + container (a)	g	37.44	37.79	38.88	21.03	21.03
Mass of dry soil + container (b)	g	27.75	27.70	28.30	19.85	19.83
Mass of container (c)	g	16.7	16.50	16.58	16.66	16.51
Mass of moisture (a-b)	g	9.69	10.09	10.76	1.18	1.20
Mass of dry soil (b-c)	g	11.05	11.20	11.54	3.19	3.32
Moisture content (w=a-b/b-c x 100)	%	87.69	90.09	93.24	36.99	36.14
Average moisture content (wa)		90.34			36.57	
Plastic Index		53.77				



6) Compaction laboratory result

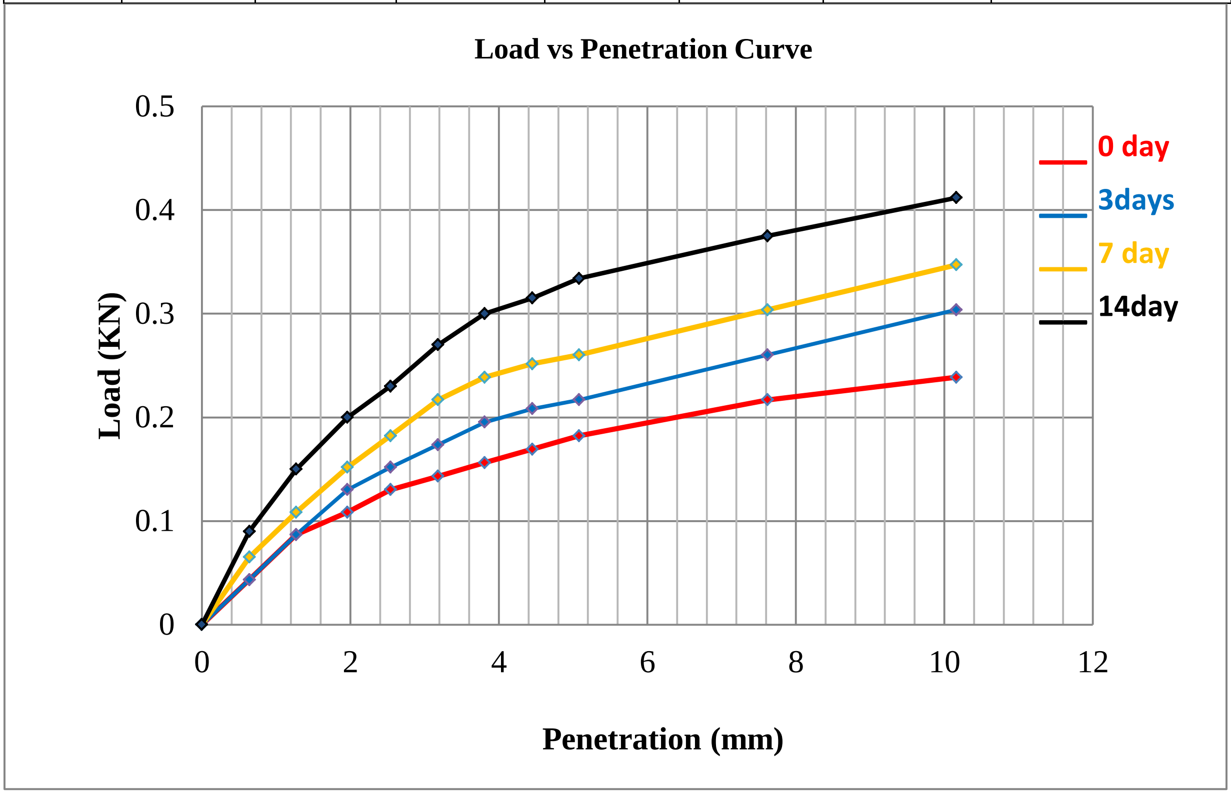
Trial No.		1	2	3	4	
Assumed Water Content	%	8	10	12	14	
Wt. of Mold + Wet Soil	Gm	5174	5298	5361	5355	
Wt. of Mold	Gm	3817	3817	3817	3817	
Wt. Wet Soil	Gm	1357	1481	1544	1538	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.438	1.569	1.636	1.629	
						NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	Gm	191.27	162.22	211.06	187.7	213.55
Wt. Cont + Dry soil	Gm	166.50	140.00	178.81	156.29	196.81
Weight of Water	Gm	24.8	22.2	32.3	31.4	16.7
Weight of Container	Gm	24.72	25.02	25.97	24.17	24.25
Weight of Dry Soil	Gm	141.8	115.0	152.8	132.1	172.6
Moisture Content	%	17.47	19.33	21.10	23.77	9.70
Dry Density	gr/cm^3	1.22	1.31	1.351	1.32	



7) CBR and CBR Swelling

DENSITY DETERMINAITON								
No. of Blows					56 Blows			
Mould No.					E			
Soaking Condition					BEFORE	AFTER		
WEIGHT OF SOIL + MOLD g			W2		11095	11710		
WEIGHT OF MOLD g			W1		7610	7610		
WEIGHT OF SOIL g			W2 - W1		3485	4100		
VOLUME OF MOLD cc			V		2124	2124		
WET DENSITY OF SOIL g/cc			(W2 - W1)/V		1.64	1.93		
DRY DENSITY OF SOIL g/cc			(W2 - W1)/V(1+w)		1.352	1.42		
MOISTURE DETERMINATION								
Mould No.					E			
Soaking Condition					BEFORE	AFTER		
CONTAINER NUMBER			NO.		H	E		
WET SOIL + CONTAINER g			W3		165.71	124.16		
DRY SOIL + CONTAINER g			W2		140.95	97.56		
WEIGHT OF WATER g			W3- W2		24.76	26.60		
WEIGHT OF CONTAINER g			W1		25.23	24.19		
WEIGHT OF DRY SOIL g			W2 - W1		115.72	73.37		
MOISTURE CONTENT %			W3 - W2/W2 - W1		21.40	36.25		
Ring factor	0.04339	KN/div.						
PENETRATION TEST DATA								
Pentration in mm	load (KN)	CBR %	Load (KN	CBR %	load (KN)	CBR %	load (KN)	CBR %
	0 day		3 days		7 day		14 day	
0	0.00		0		0.00		0	
0.64	0.04		0.04		0.07		0.09	
1.27	0.09		0.09		0.11		0.15	
1.96	0.01		0.013		0.15		0.2	
2.54	0.13	0.98	0.15	1.14	0.18	1.34	0.23	1.72
3.18	0.14		0.17		0.22		0.27	
3.81	0.16		0.2		0.24		0.3	
4.45	0.17		0.21		0.25		0.315	
5.08	0.18	0.90	0.22	1.09	0.26	1.30	0.34	1.67
7.62	0.22		0.26		0.304		0.375	
10.16	0.24		0.304		0.347		0.412	
No. of Blows	Intial Reading	Final Reading	% Swell	Proctor Data	OMC	21.400	MDD	1.351
56	1.25	20.650	16.66					

Curing day	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0	56	1.35	0.13	0.18	0.98	0.91	16.66
3	56	1.35	0.15	0.22	1.14	1.09	16.66
7	56	1.35	0.18	0.26	1.37	1.30	16.66
14	56	1.35	0.23	0.334	1.72	1.67	16.66



Appendix B: Laboratory Test Result of HAP soil sample

1. Natural Moisture Content

Sample location	HAP		
Can number	B1	B2	B3
Mass Of Moisture can (Mc)	25	25	25
Mass of moisture can + Mass of moist soil (Mcms)	92	93	95
Mass of Moisture can + mass of oven dried soil (Mcds.)	75	76	78
Mass of water (Mw)	17	19	21
Mass of dry soil (Ms.)	50	51	53
Water Content(w) %	34	37	40
Average water content(w) %	37		

2. Wet sieve analysis

Sieve size in mm	Mass of retained (gm.)	Percentage retained (%)	Percentage of commutative retained %	percentage of finer particle %
4.75	0.0	0	0	100.00
2.000	1.5	0.3	0.3	99.7
0.850	1.5	0.3	0.6	99.4
0.425	6.5	1.3	1.9	98.1
0.30	1.5	0.3	2.2	97.8
0.15	3.5	0.7	2.9	97.1
0.075	3	0.6	3.5	96.5
PAN	482.5	96.5	100	0.00
Total	500	**100.0**		

3. Hydrometer Analysis

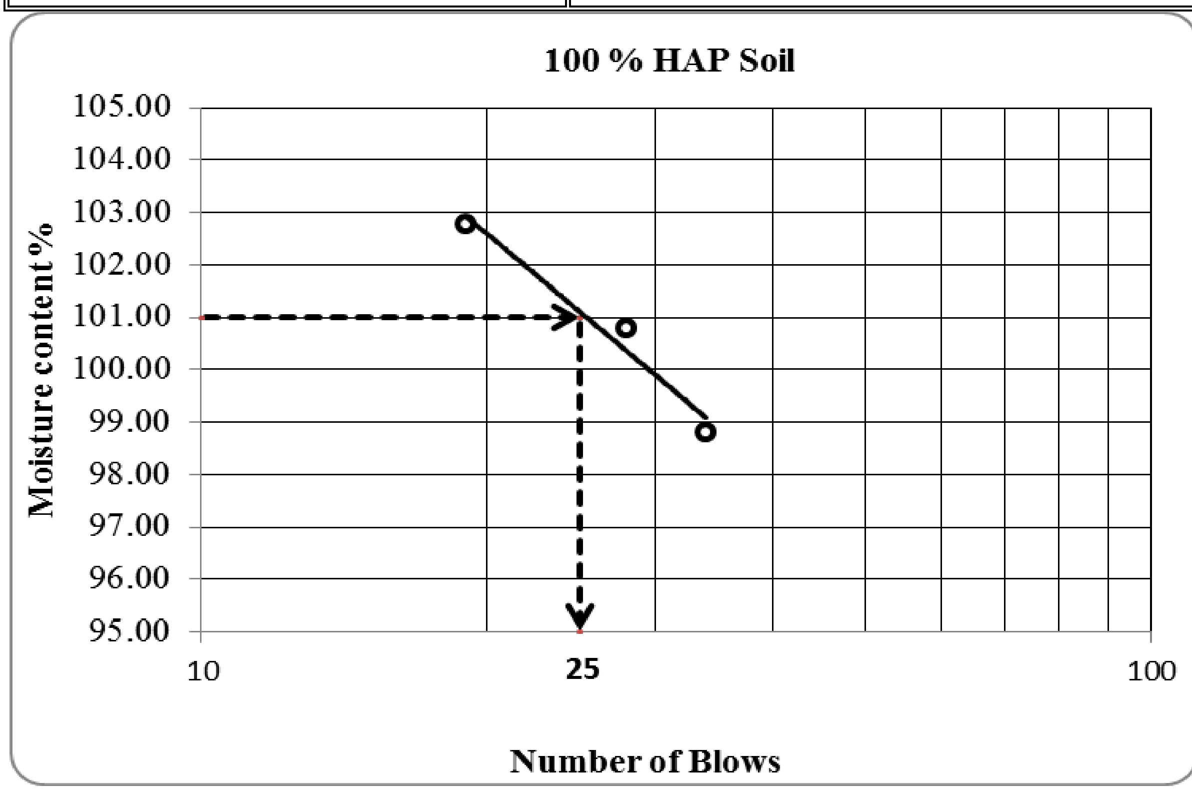
Time	3:00	3:02	3:06	3:14	3:29	3:59	4:59	6:59	8:59	16:59		
Elapsed time t in, min	1	2	4	8	15	30	60	120	240	480	960	1440
Actual hydrometer reading	51	51	50	49	48	47	46	45	44	42	41	39
Eff. depth, L (cm)	8.1	8.1	8.3	8.4	8.6	8.8	8.9	9.1	9.2	9.6	9.7	10.1
Temperature (Co)	20	20	20	20	20	20	20	23	23	20	20	20
K	0.01344	0.01344	0.01344	0.01344	0.01344	0.01344	0.01344	0.01267	0.01267	0.01344	0.01344	0.01344
CT	0	0	0	0	0	0	0	0.7	0.7	0	0	0
Par. diameter D (mm)	0.038	0.027	0.019	0.0138	0.01	0.007	0.005	0.0035	0.0025	0.0019	0.0014	0.0011
Value of 'a' for G_S	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Corrected hydrometer reading for present particle	45	45	44	43	42	41	40	39.7	38.7	36	35	33
%Finer D,P(%)= Rd+a/ Mo	89.10	89.10	87.12	85.14	83.16	81.18	79.20	78.61	76.63	71.28	69.30	65.34
corrected (Pa)	81.79	81.79	79.98	78.16	76.34	74.52	72.71	72.16	70.34	65.44	63.62	59.98

4. Combined sieve analyses

Grain Size (mm)	% passing	Combined % passing
9.5	100.0	100.0
4.75	100.0	100.0
2	99.7	99.7
0.85	99.4	99.4
0.425	98.1	98.1
0.3	97.8	97.8
0.15	97.1	97.1
0.075	96.5	96.5
0.037	89.10	81.79
0.026	89.10	81.79
0.019	87.12	79.98
0.0133	85.14	78.16
0.0098	83.16	76.34
0.007	81.18	74.52
0.005	79.20	72.71
0.0035	78.61	72.16
0.0025	76.63	70.35
0.0018	71.28	65.44
0.0013	69.30	63.62
0.0011	65.34	59.98

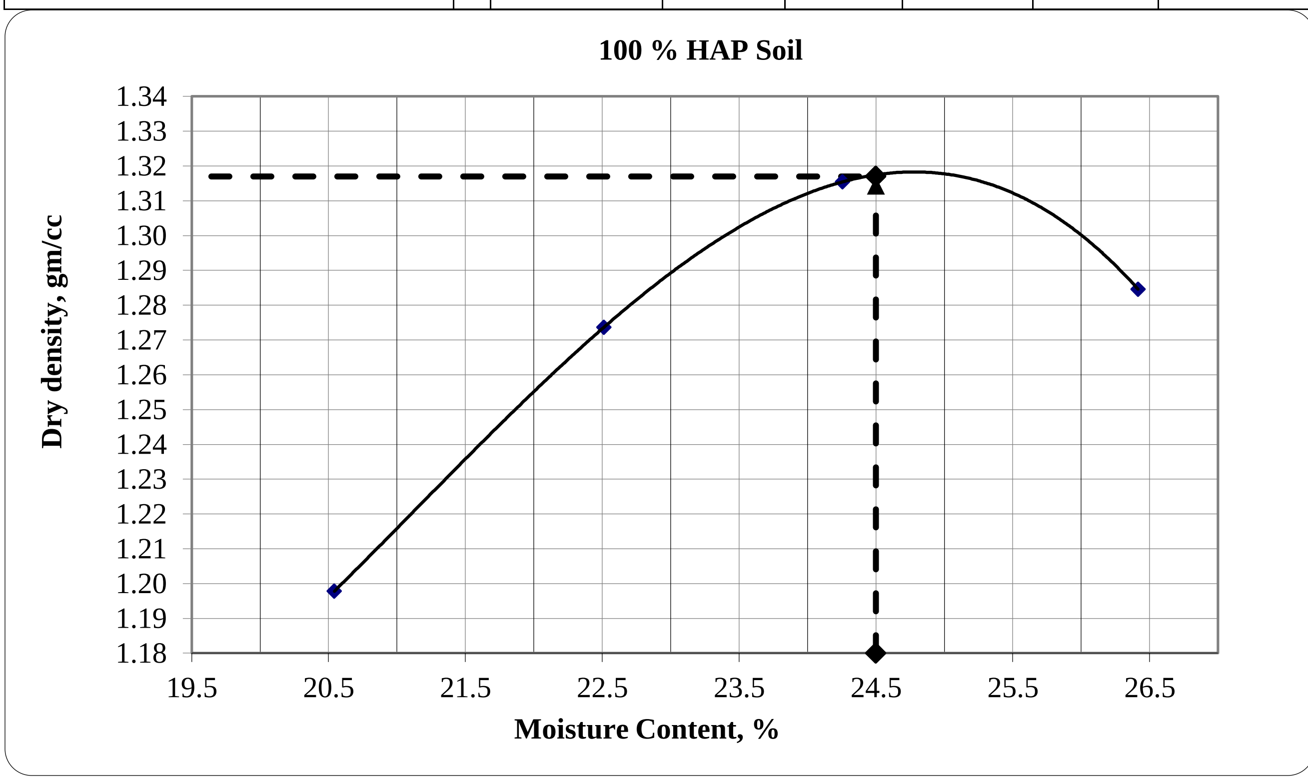
5. Atterberg Test Result

Descriptions		100 % HAP Soil				
		Liquid limit			plastic limit	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	19		
Container No.		B - 10	G	A	F	C
Mass of wet soil + container (a)	g	38.45	38.95	38.90	22.11	22.11
Mass of dry soil + container (b)	g	27.85	27.61	27.81	20.62	20.63
Mass of container (c)	g	17.12	16.36	17.02	16.65	16.5
Mass of moisture (a-b)	g	10.60	11.34	11.09	1.49	1.48
Mass of dry soil (b-c)	g	10.73	11.25	10.79	3.97	4.13
Moisture content (w=a-b/b-c x 100)	%	98.79	100.80	102.78	37.53	35.84
Average moisture content (wa)		**100.79**			**36.68**	
Plastic Index		**64.32**				



6. Compaction laboratory result

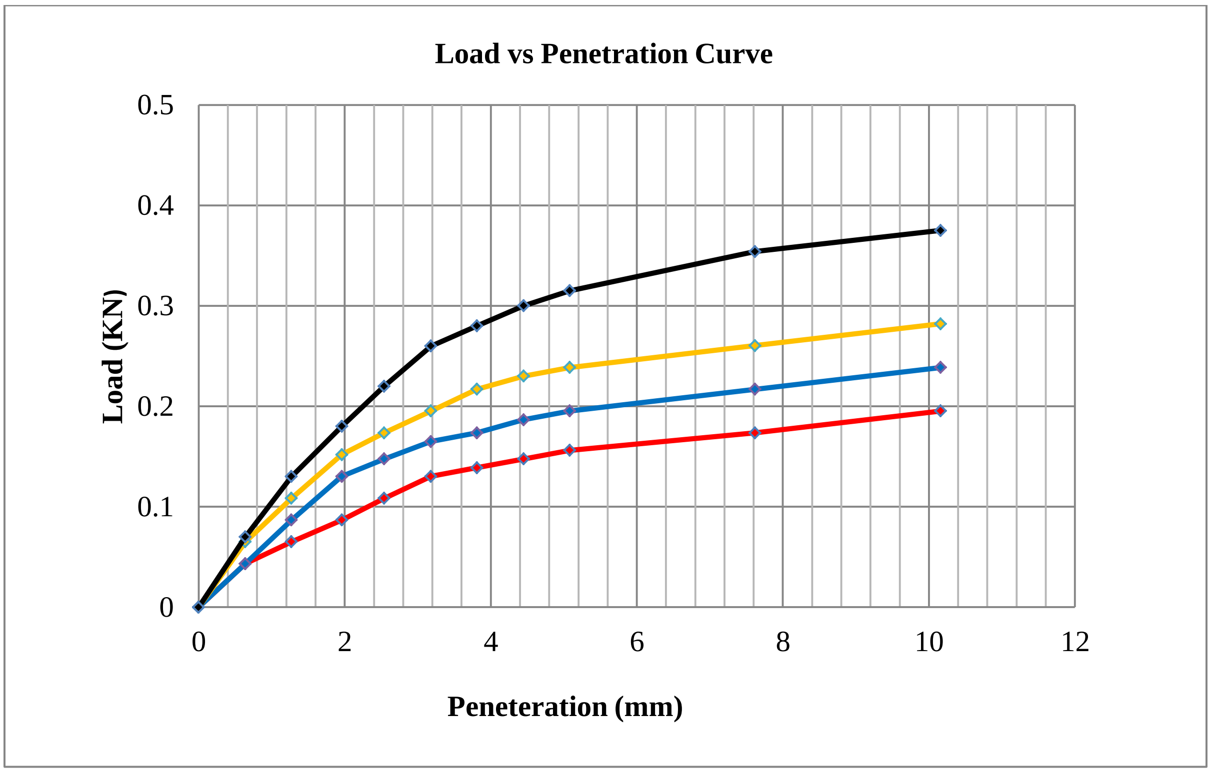
Trial No.		1	2	3	4	
Assumed Water Content	%	10	12	14	16	
Wt. of Mold + Wet Soil	gm.	5180	5290	5360	5350	
Wt. of Mold	gm.	3817	3817	3817	3817	
Wt. Wet Soil	gm.	1363	1473	1543	1533	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.444	1.560	1.635	1.624	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	195.1	160.23	215.50	190.23	208.74
Wt. Cont + Dry soil	gm.	165.95	135.50	178.50	155.53	192.50
Weight of Water	gm.	29.2	24.7	37.0	34.7	16.2
Weight of Container	gm.	24.05	25.65	25.97	24.17	25.02
Weight of Dry Soil	gm.	141.9	109.9	152.5	131.4	167.5
Moisture Content	%	20.54	22.51	24.26	26.42	9.70
Dry Density	gr/cm^3	1.20	1.27	1.315	1.28	



7. CBR laboratory result

DENSITY DETERMINAITON									
No. of Blows						56 Blows			
Mould No.						E			
Soaking Condition						BEFORE	AFTER		
WEIGHT OF SOIL + MOLD g				W2		11090	11450		
WEIGHT OF MOLD g				W1		7610	7610		
WEIGHT OF SOIL g				W2 - W1		3480	3840		
VOLUME OF MOLD cc				V		2124	2124		
WET DENSITY OF SOIL g/cc				(W2 - W1)/V		1.64	1.81		
DRY DENSITY OF SOIL g/cc				(W2 - W1)/V(1+w)		1.316	1.31		
MOISTURE DETERMINATION									
Mould No.						56 Blows			
Soaking Condition						BEFORE	AFTER		
CONTAINER NUMBER				NO.		H	E		
WET SOIL + CONTAINER g				W3		170.12	125.62		
DRY SOIL + CONTAINER g				W2		141.61	97.56		
WEIGHT OF WATER g				W3- W2		28.51	28.06		
WEIGHT OF CONTAINER g				W1		25.3	24.2		
WEIGHT OF DRY SOIL g				W2 - W1		116.31	73.36		
MOISTURE CONTENT %				W3 - W2/W2 - W1		24.51	38.25		
Ring factor =		0.04339	KN/div.						
PENETRATION TEST DATA									
Penetrati on in mm	Load(KN)	CBR %	Load(KN)	CBR %	Load(KN)	CBR %	Load(KN)	CBR %	
	0 day		3 days		7 days		14 days		
0	0.00		0.00		0.00		0		
0.64	0.04		0.04		0.07		0.07		
1.27	0.07		0.09		0.11		0.13		
1.96	0.09		0.13		0.15		0.18		
2.54	0.11	0.82	0.15	1.11	0.17	1.30	0.22	1.65	
3.18	0.13		0.16		0.20		0.26		
3.81	0.14		0.17		0.22		0.28		
4.45	0.15		0.9		0.23		0.3		
5.08	0.16	0.8	0.20	0.98	0.24	1.19	0.32	1.58	
7.62	0.174		0.217		0.26		0.354		
10.16	0.195		0.239		0.28		0.375		
No. of Blows	Intial Reading	Final Reading	% Swell	Proctor Data		CBR at 100 MDD			
				OMC	MDD	0 Day	3 days	7 days	14 days
56	1.25	22.54	18.29	24.5	1.315	0.82	1.11	1.3	1.65

curing days	NO. OF Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.32	0.11	0.16	0.81	0.78	18.29
3 days			0.15	0.20	1.11	0.98	
7 days			0.17	0.24	1.30	1.19	
14 days			0.22	0.32	1.65	1.58	

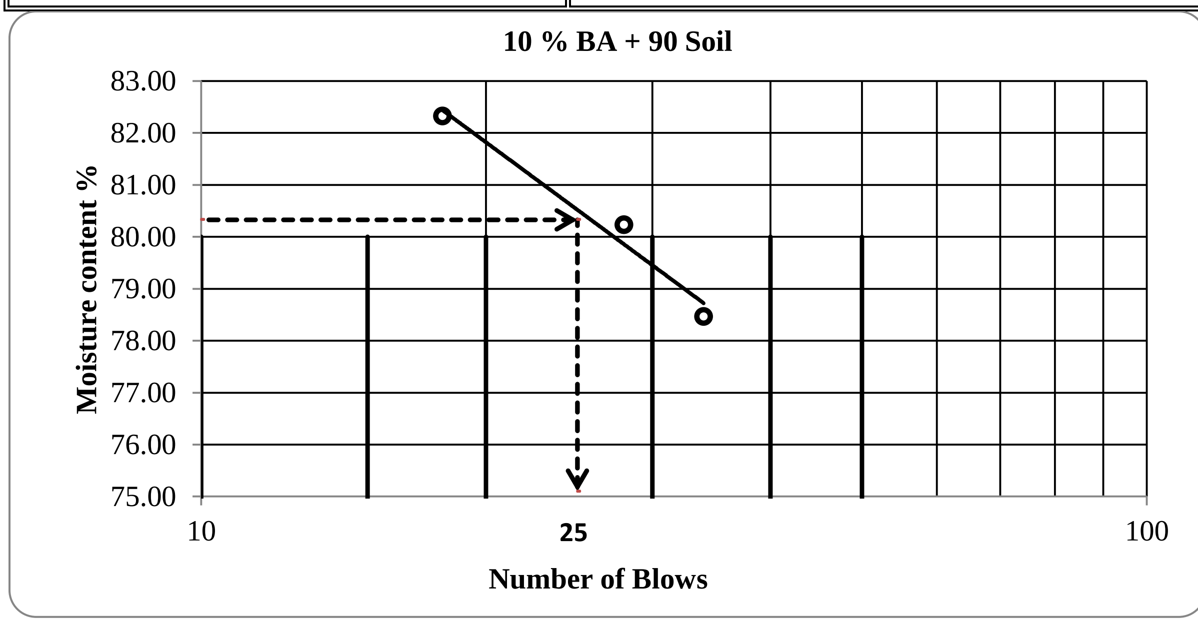


Appendix C: Laboratory test result of stabilizing AABRP soil sample using bagasse ash mixed with cement

1. **Atterberg limit test**

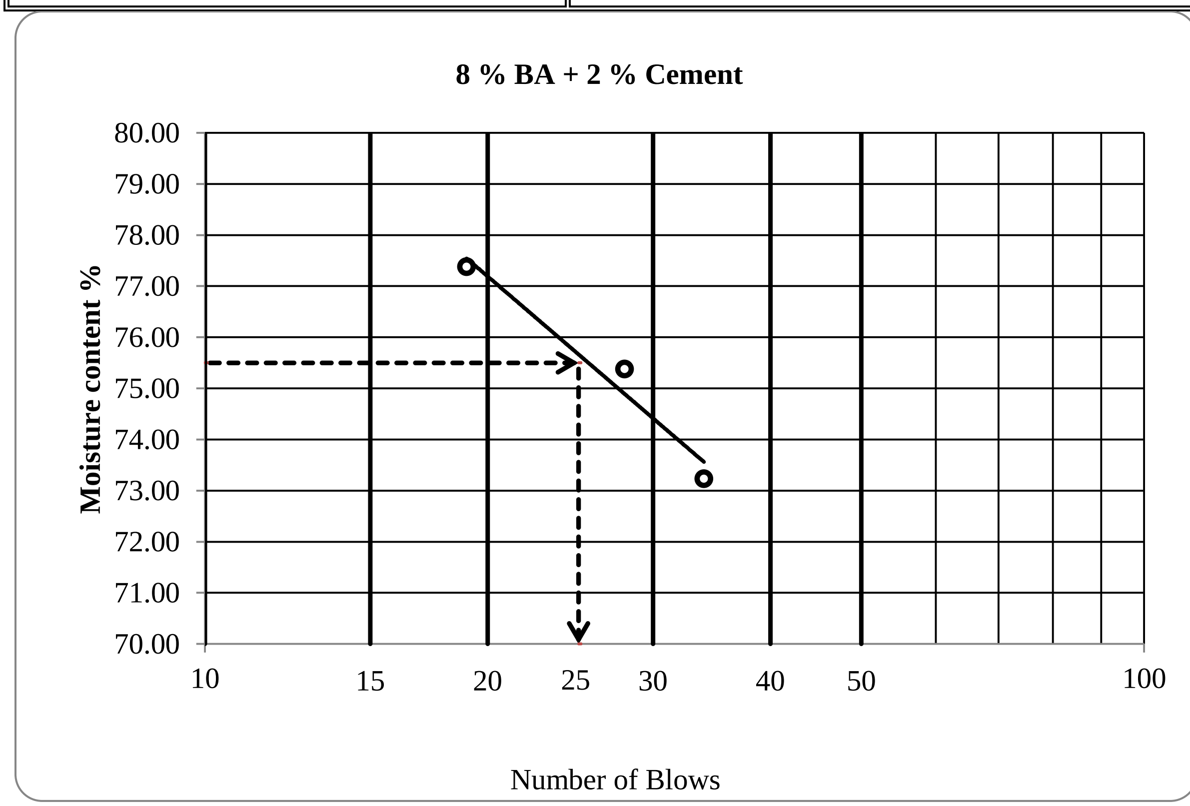
1.1 Bagasse ash 10 % + 0 % Cement

determination		10 % BA + + 0 % Cement				
		Liquid limit			plastic limit	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	18		
Container No.		E	B	H	M	C-4
Mass of wet soil + container (a)	g	39.11	40.14	41.77	20.21	20.22
Mass of dry soil + container (b)	g	29.20	29.67	30.36	19.17	19.26
Mass of container (c)	g	16.57	16.62	16.5	16.43	16.7
Mass of moisture (a-b)	g	9.91	10.47	11.41	1.04	0.96
Mass of dry soil (b-c)	g	12.63	13.05	13.86	2.74	2.56
Moisture content (w=a-b/b-c x 100)	%	78.46	80.23	82.32	37.96	37.50
Average moisture content (wa)		**80.34**			**37.73**	
Plastic Index		**42.60**				



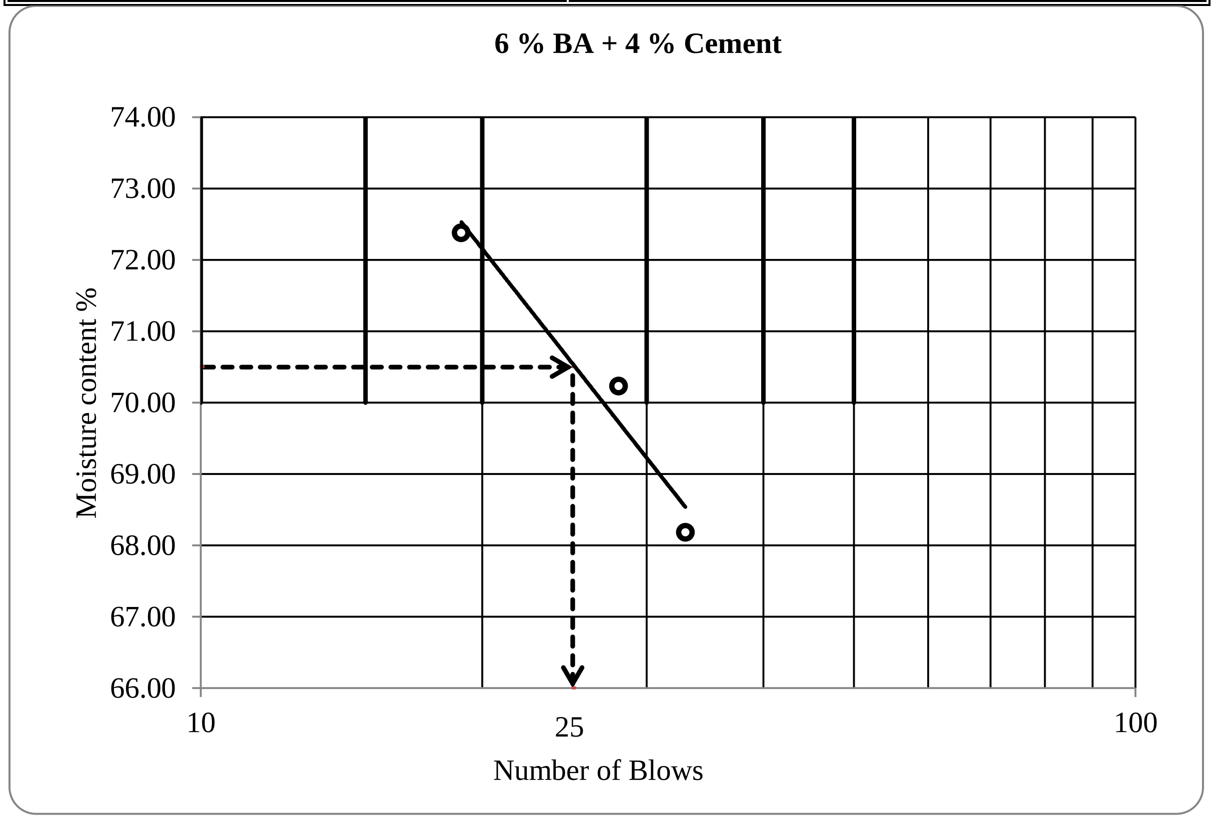
1.2 Bagasse ash 8% + 2 % Cement

determination	8% BA + 2% C					
	Liquid limit			**plastic limit**		
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	19		
Container No.		F	R	H	C-13	K
Mass of wet soil + container (a)	g	39.09	40.74	41.65	22.1	22.1
Mass of dry soil + container (b)	g	29.57	30.30	30.67	20.56	20.55
Mass of container (c)	g	16.57	16.45	16.48	16.54	16.6
Mass of moisture (a-b)	g	9.52	10.44	10.98	1.54	1.55
Mass of dry soil (b-c)	g	13.00	13.85	14.19	4.02	3.95
Moisture content (w=a-b/b-c x 100)	%	73.23	75.38	77.38	38.31	39.24
Average moisture content (wa)		**75.33**			**38.77**	
Plastic Index		**36.55**				



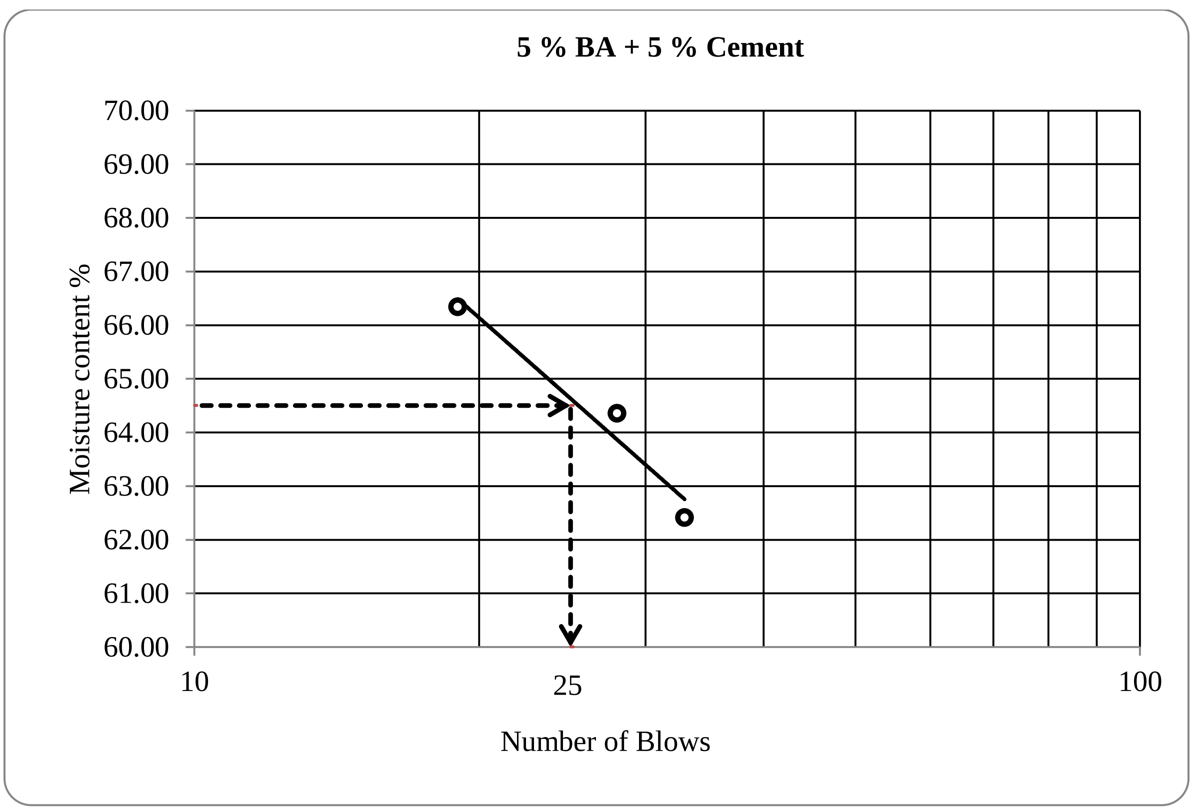
1.3 Bagasse ash 6 % + 4 % Cement

determination		**6 % BA + 4 % C**				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		33	28	19		
Container No.		C-13	Q	F	B	R
Mass of wet soil + container (a)	g	35.80	36.53	38.11	21.55	21.57
Mass of dry soil + container (b)	g	28.00	29.50	29.07	20.08	20.10
Mass of container (c)	g	16.56	19.49	16.58	16.54	16.46
Mass of moisture (a-b)	g	7.80	7.03	9.04	1.47	1.47
Mass of dry soil (b-c)	g	11.44	10.01	12.49	3.54	3.64
Moisture content (w=a-b/b-c x 100)	%	68.18	70.23	72.38	41.53	40.38
Average moisture content (wa)		**70.26**			**40.96**	
Plastic Index		**29.31**				



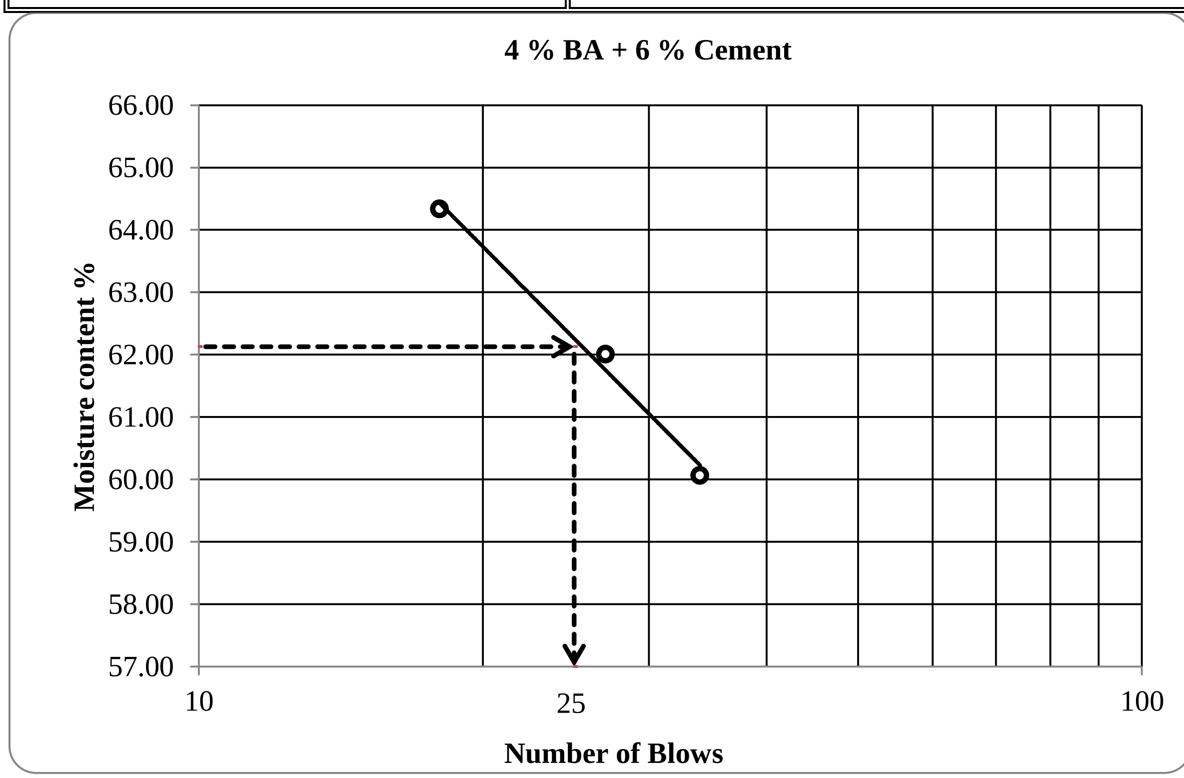
1.4 **Bagasse Ash 5 % +5 % Cement**

determination						
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		33	28	19		
Container No.		D	L	N	C-15	C-17
Mass of wet soil + container (a)	g	36.54	37.04	38.64	21.14	21.14
Mass of dry soil + container (b)	g	30.48	29.06	29.81	19.81	19.75
Mass of container (c)	g	20.77	16.66	16.5	16.67	16.44
Mass of moisture (a-b)	g	6.06	7.98	8.83	1.33	1.39
Mass of dry soil (b-c)	g	9.71	12.40	13.31	3.14	3.31
Moisture content (w=a-b/b-c x 100)	%	62.41	64.35	66.34	42.36	41.99
Average moisture content (wa)		**64.37**			**42.18**	
Plastic Index		**22.19**				



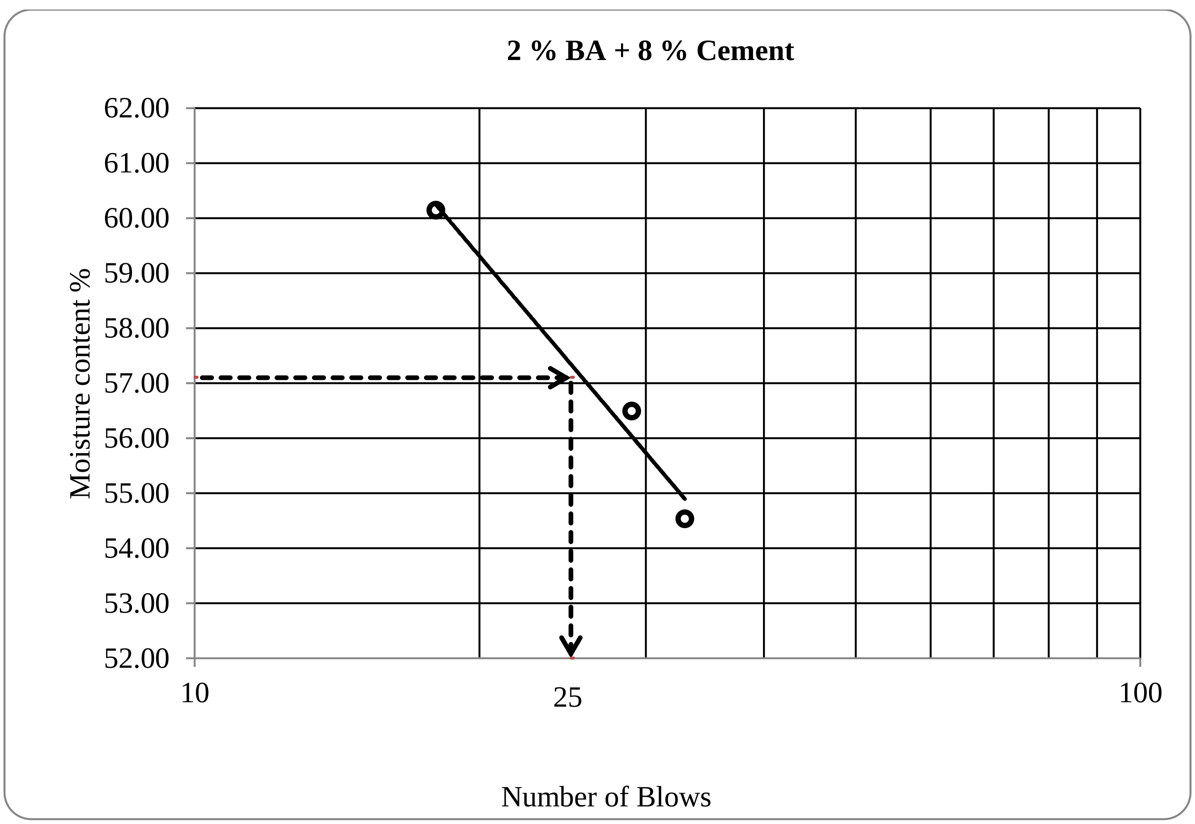
1.5 Bagasse Ash 4 % + Cement 6 %

determination						
		Liquid limit			**plastic limit**	
trial namber		**1**	**2**	**3**	**1**	**2**
No. of blows		34	27	18		
Container No.		M	Q	C-24	C-22	M-5
Mass of wet soil + container (a)	g	36.39	37.58	38.92	22.77	22.77
Mass of dry soil + container (b)	g	28.90	30.66	31.00	21.26	21.40
Mass of container (c)	g	16.43	19.50	18.69	17.93	18.39
Mass of moisture (a-b)	g	7.49	6.92	7.92	1.51	1.37
Mass of dry soil (b-c)	g	12.47	11.16	12.31	3.33	3.01
Moisture content (w=a-b/b-c x 100)	%	60.06	62.01	64.34	45.35	45.51
Average moisture content (wa)		**62.14**			**45.43**	
Plastic Index		**16.71**				



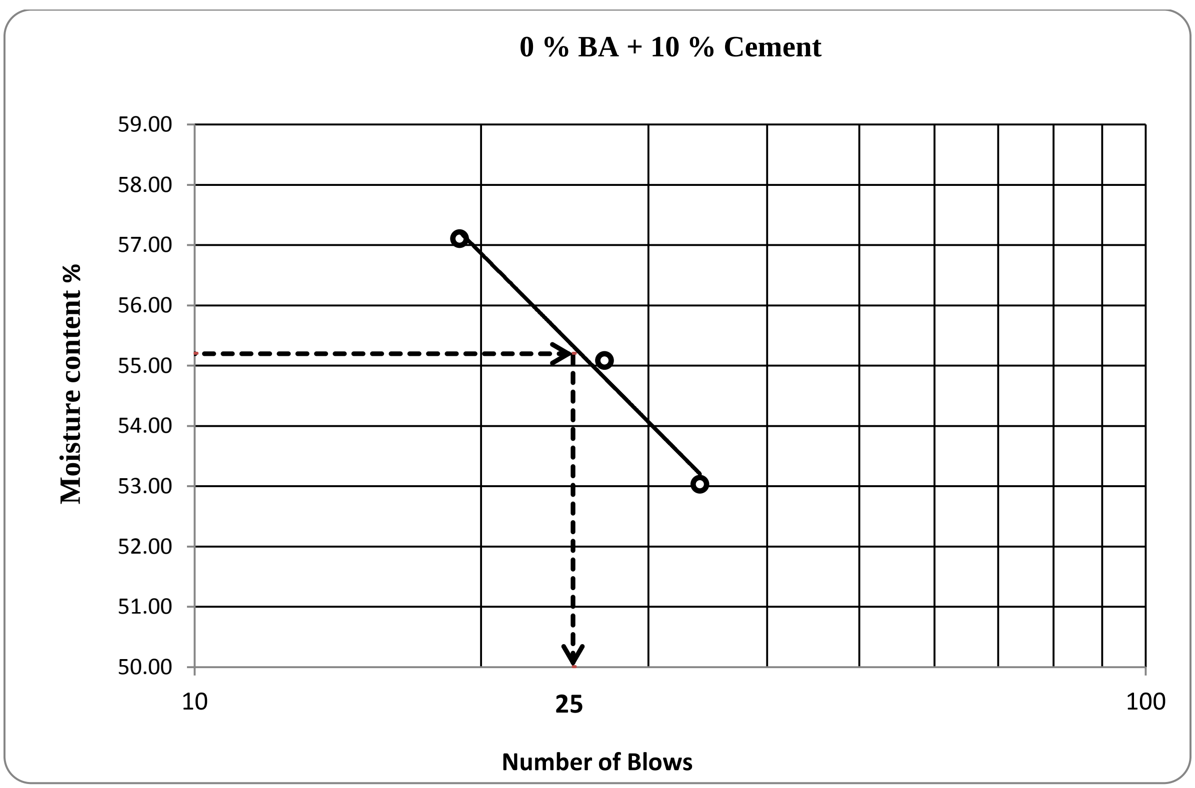
1.6 Bagasse Ash 2 % + Cement 8 %

determination						
		Liquid limit			**plastic limit**	
trial namber		**1**	**2**	**3**	**1**	**2**
No. of blows		33	29	18		
Container No.		E	K	C-12	C-15	C-17
Mass of wet soil + container (a)	g	36.86	37.23	38.99	21.68	21.68
Mass of dry soil + container (b)	g	29.70	29.79	30.57	20.05	19.97
Mass of container (c)	g	16.57	16.62	16.57	16.7	16.48
Mass of moisture (a-b)	g	7.16	7.44	8.42	1.63	1.71
Mass of dry soil (b-c)	g	13.13	13.17	14.00	3.35	3.49
Moisture content (w=a-b/b-c x 100)	%	54.53	56.49	60.14	48.66	49.00
Average moisture content (wa)		**57.06**			**48.83**	
Plastic Index		**8.23**				



1.7 Bagasse Ash 0 % + 10 % Cement

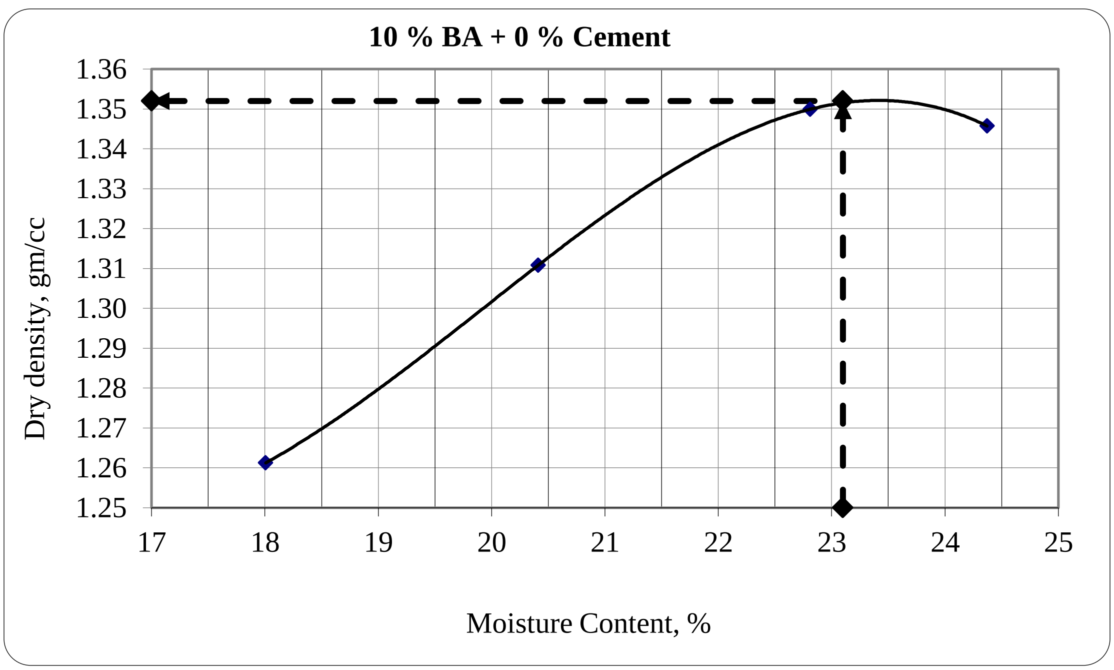
determination						
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	27	19		
Container No.		B	C	E	M	Q
Mass of wet soil + container (a)	g	38.99	39.46	40.20	22.57	22.57
Mass of dry soil + container (b)	g	31.20	32.20	31.60	20.45	21.53
Mass of container (c)	g	16.51	19.02	16.54	16.4	19.46
Mass of moisture (a-b)	g	7.79	7.26	8.60	2.12	1.04
Mass of dry soil (b-c)	g	14.69	13.18	15.06	4.05	2.07
Moisture content (w=a-b/b-c x 100)	%	53.03	55.08	57.10	52.35	50.24
Average moisture content (wa)		**55.07**			**51.29**	
Plastic Index		**3.78**				



2 Compaction test result

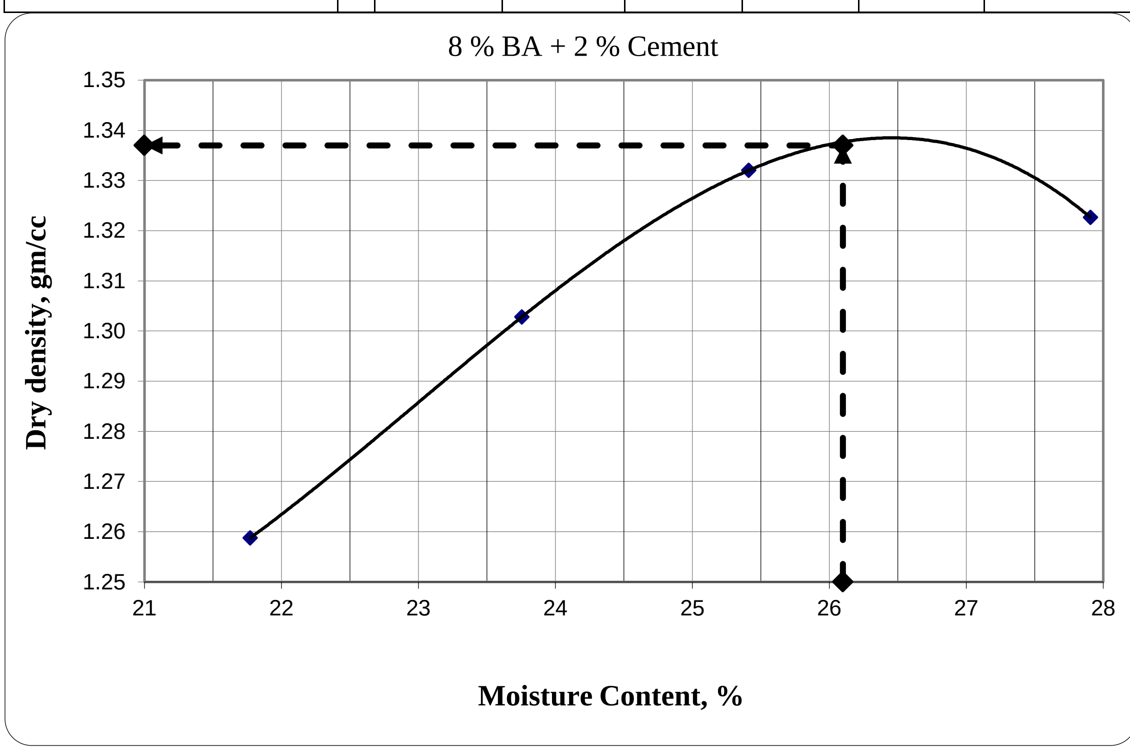
2.1 Bagasse Ash 10 % + Cement 0 %

Trial No.		1	2	3	4	
Assumed Water Content	ml	300	400	500	600	
Wt. of Mold + Wet Soil	gm.	5220	5305	5380	5395	
Wt. of Mold	gm.	3815	3815	3815	3815	
Wt. Wet Soil	gm.	1405	1490	1565	1580	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.488	1.578	1.658	1.674	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	148.1	171.47	179.93	176.48	203.45
Wt. Cont + Dry soil	gm.	129.20	146.50	151.10	146.80	191.46
Weight of Water	gm.	18.9	25.0	28.8	29.7	12.0
Weight of Container	gm.	24.24	24.17	24.71	25.02	25.96
Weight of Dry Soil	gm.	105.0	122.3	126.4	121.8	165.5
Moisture Content	%	18.01	20.41	22.81	24.37	7.24
Dry Density	gr/cm^3	1.26	1.31	1.350	1.35	



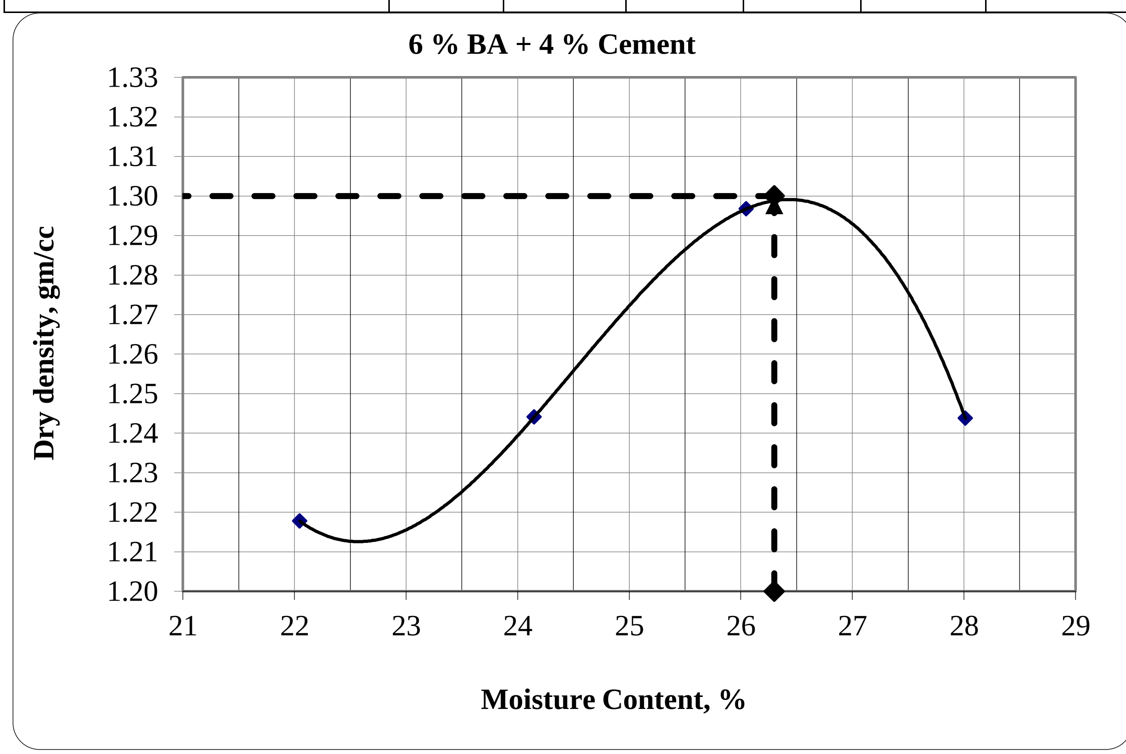
2.2 Bagasse Ash 8 % + 2 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	12	14	16	18	
Wt. of Mold + Wet Soil	gm.	5260	5335	5390	5410	
Wt. of Mold	gm.	3813	3813	3813	3813	
Wt. Wet Soil	gm.	1447	1522	1577	1597	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.533	1.612	1.671	1.692	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont. + Wet soil	gm.	140.93	135.72	114.38	120.56	224.17
Wt. Cont.+ Dry soil	gm.	120.38	114.39	95.94	99.71	206.28
Weight of Water	gm.	20.6	21.3	18.4	20.9	17.9
Weight of Container	gm.	26.00	24.60	23.38	25.00	24.17
Weight of Dry Soil	gm.	94.4	89.8	72.6	74.7	182.1
Moisture Content	%	21.77	23.76	25.41	27.91	9.82
Dry Density	gr/cm^3	1.26	1.30	1.332	1.32	



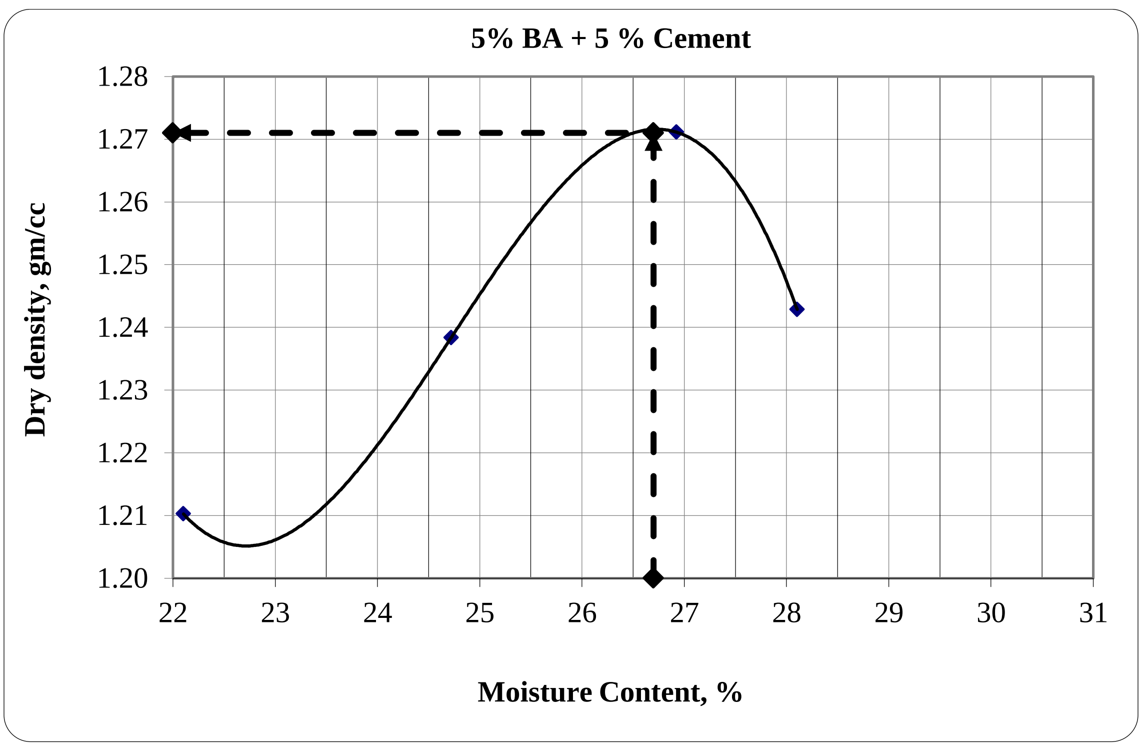
2.3 bagasse Ash 6 % + 4% Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	14	16	18	20	
Wt. of Mold + Wet Soil	gm.	5220	5275	5360	5320	
Wt. of Mold	gm.	3817	3817	3817	3817	
Wt. Wet Soil	gm.	1403	1458	1543	1503	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.486	1.544	1.635	1.592	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	187.83	146.55	131.42	112.36	126.2
Wt. Cont + Dry soil	gm.	158.71	124.10	109.65	93.23	118.02
Weight of Water	gm.	29.1	22.5	21.8	19.1	8.2
Weight of Container	gm.	26.63	31.13	26.08	24.94	26.20
Weight of Dry Soil	gm.	132.1	93.0	83.6	68.3	91.8
Moisture Content	%	22.05	24.15	26.05	28.01	8.91
Dry Density	gr/cm^3	1.22	1.24	1.297	1.24	



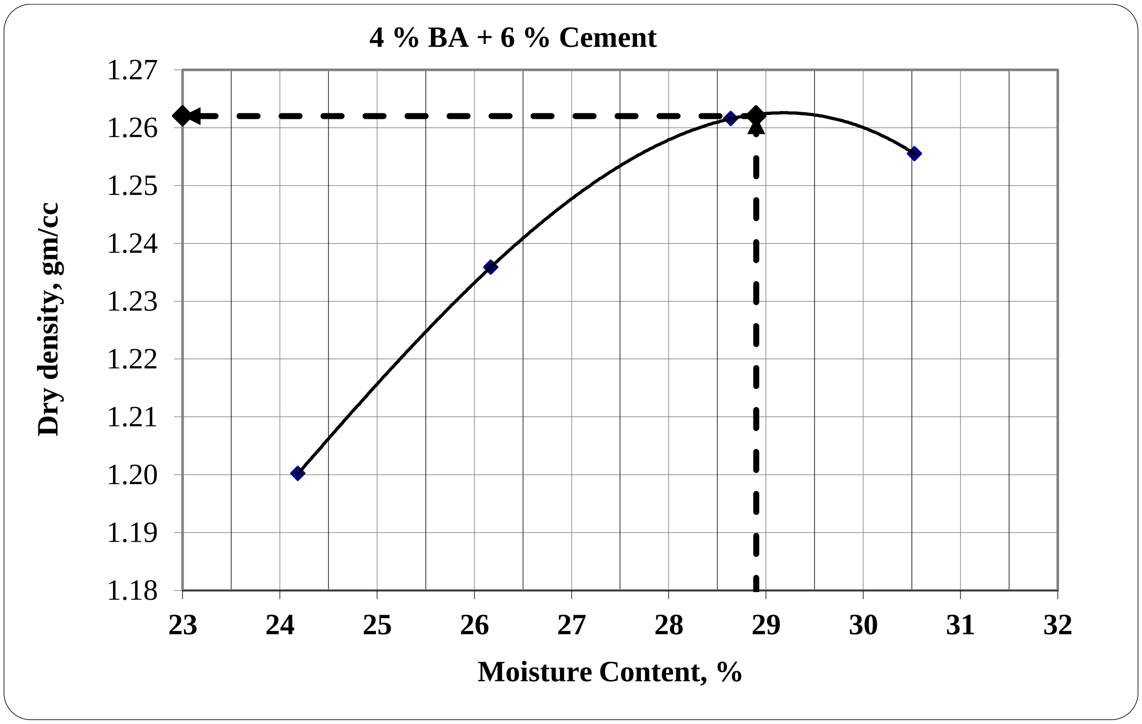
2.4 Bagasse Ash 5 % + 5 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	14	16	18	20	
Wt. of Mold + Wet Soil	gm.	5212	5275	5340	5320	
Wt. of Mold	gm.	3817	3817	3817	3817	
Wt. Wet Soil	gm.	1395	1458	1523	1503	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.478	1.544	1.613	1.592	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont. + Wet soil	gm.	128.11	115.05	100.03	125.26	125.04
Wt. Cont. + Dry soil	gm.	109.10	97.10	84.00	103.10	117.33
Weight of Water	gm.	19.0	18.0	16.0	22.2	7.7
Weight of Container	gm.	23.09	24.49	24.46	24.25	25.04
Weight of Dry Soil	gm.	86.0	72.6	59.5	78.9	92.3
Moisture Content	%	22.10	24.72	26.92	28.10	8.35
Dry Density	gr/cm^3	1.21	1.24	1.271	1.24	



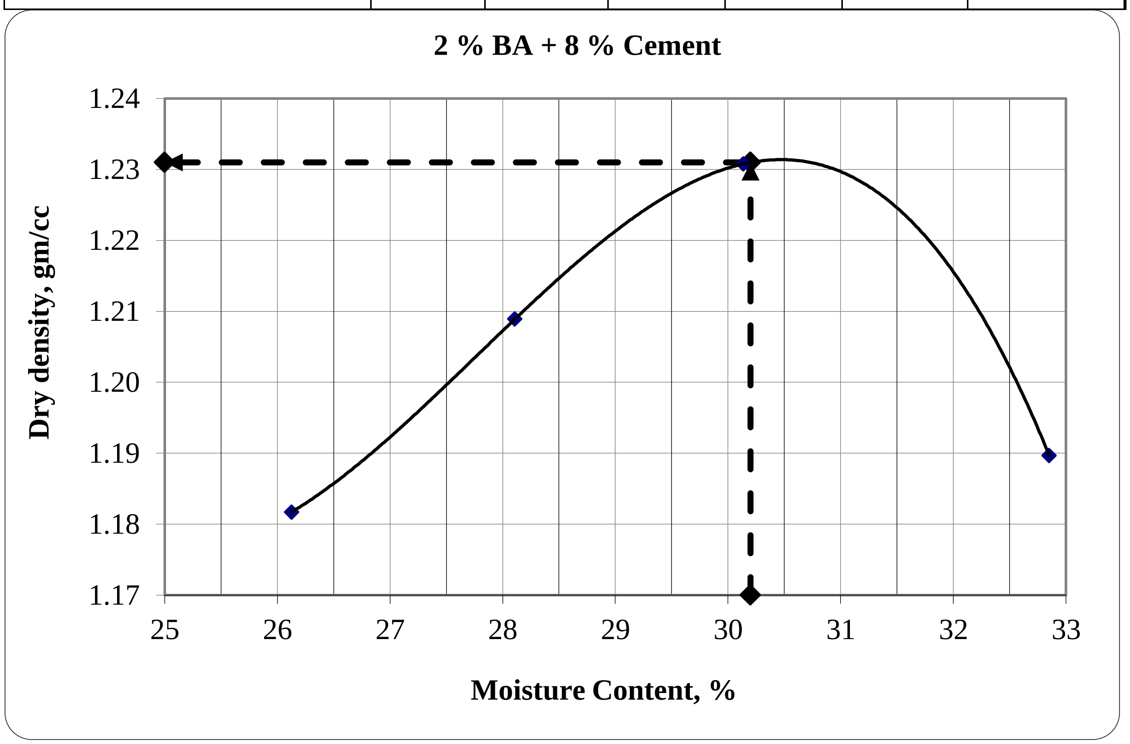
2.5 Bagasse Ash 4 % + 6 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	14	16	18	20	
Wt. of Mold + Wet Soil	gm	5225	5290	5350	5365	
Wt. of Mold	gm	3818	3818	3818	3818	
Wt. Wet Soil	gm	1407	1472	1532	1547	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.490	1.559	1.623	1.639	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm	198.00	190.00	179	153	131
Wt. Cont + Dry soil	gm	165.50	156.00	144.70	124.70	121.80
Weight of Water	gm	32.5	34.0	34.3	28.3	9.2
Weight of Container	gm	31.13	26.08	24.94	32.00	31.00
Weight of Dry Soil	gm	134.4	129.9	119.8	92.7	90.8
Moisture Content	%	24.19	26.17	28.64	30.53	10.13
Dry Density	gr/cm^3	1.20	1.24	1.262	1.26	



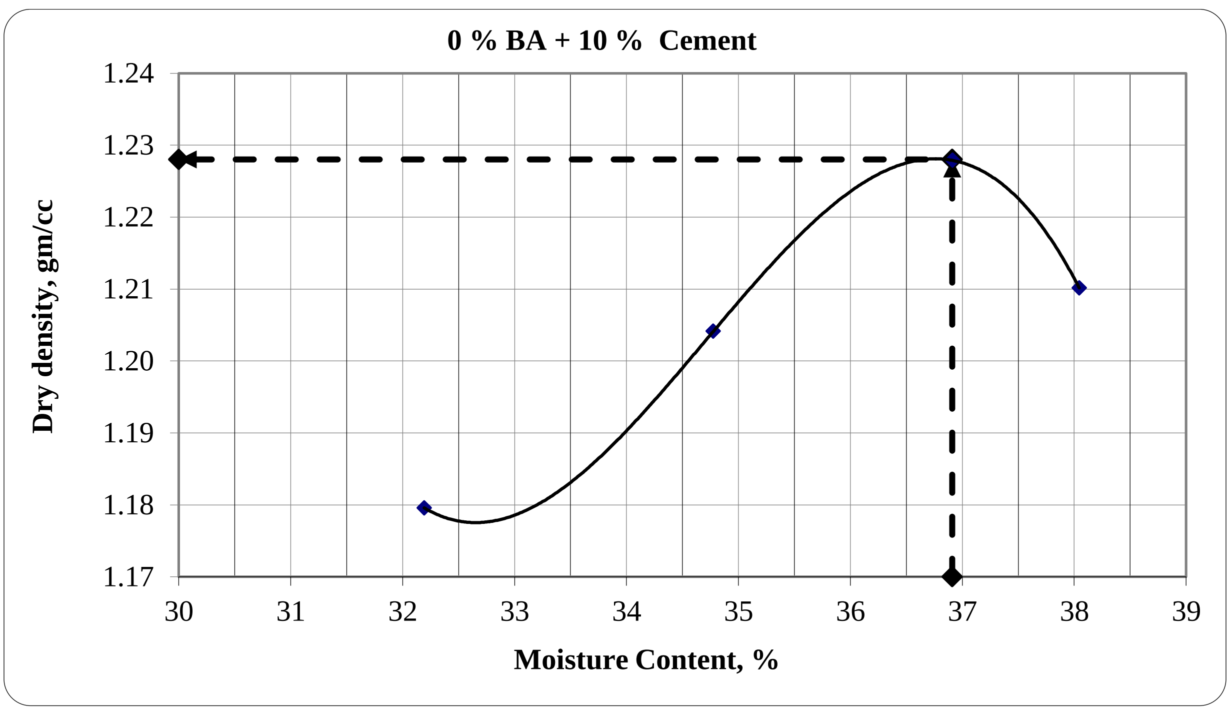
2.6 Bagasse Ash 2 % + 8 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	ml	18	20	22	24	
Wt. of Mold + Wet Soil	gm	5225	5280	5330	5310	
Wt. of Mold	gm	3818	3818	3818	3818	
Wt. Wet Soil	gm	1407	1462	1512	1492	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.490	1.549	1.602	1.581	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm	236.43	136.44	142.89	154.36	170.35
Wt. Cont + Dry soil	gm	200.80	113.50	115.70	122.30	162.43
Weight of Water	gm	35.6	22.9	27.2	32.1	7.9
Weight of Container	gm	64.43	31.89	25.48	24.71	70.35
Weight of Dry Soil	gm	136.4	81.6	90.2	97.6	92.1
Moisture Content	%	26.13	28.11	30.14	32.85	8.60
Dry Density	gr/cm^3	1.18	1.21	1.231	1.19	



2.7 Bagasse Ash 0 % +10 % Cement

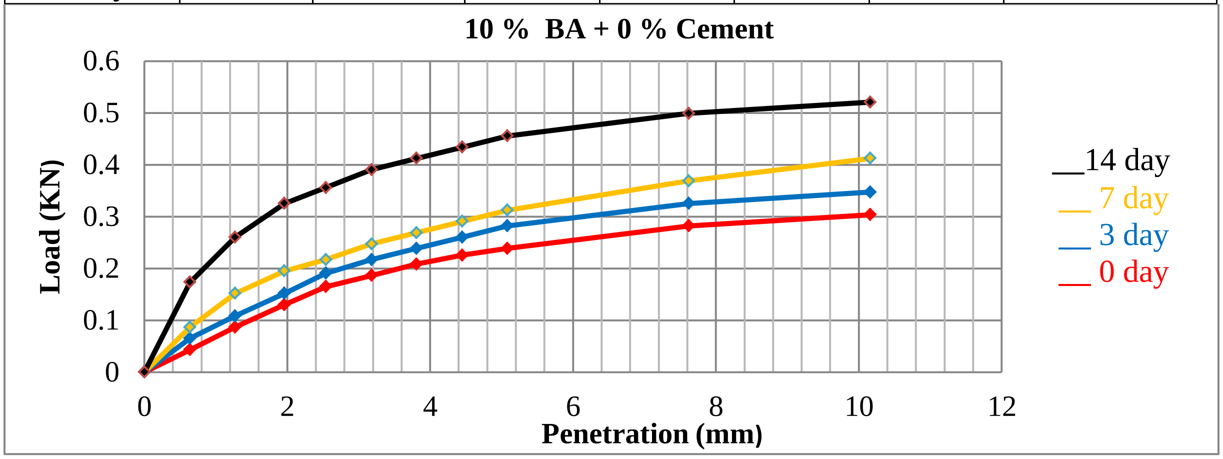
Trial No.		1	2	3	4	
Assumed Water Content	%	24	26	28	30	
Wt. of Mold + Wet Soil	gm	5290	5350	5405	5395	
Wt. of Mold	gm	3818	3818	3818	3818	
Wt. Wet Soil	gm	1472	1532	1587	1577	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.559	1.623	1.681	1.671	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm	146.22	245.93	208.51	177.83	170.35
Wt. Cont + Dry soil	gm	116.78	199.10	171.30	137.40	162.43
Weight of Water	gm	29.4	46.8	37.2	40.4	7.9
Weight of Container	gm	25.33	64.44	70.50	31.13	70.35
Weight of Dry Soil	gm	91.5	134.7	100.8	106.3	92.1
Moisture Content	%	32.19	34.78	36.91	38.04	8.60
Dry Density	gr/cm^3	1.18	1.20	1.228	1.21	



3. CBR and CBR Swell test result

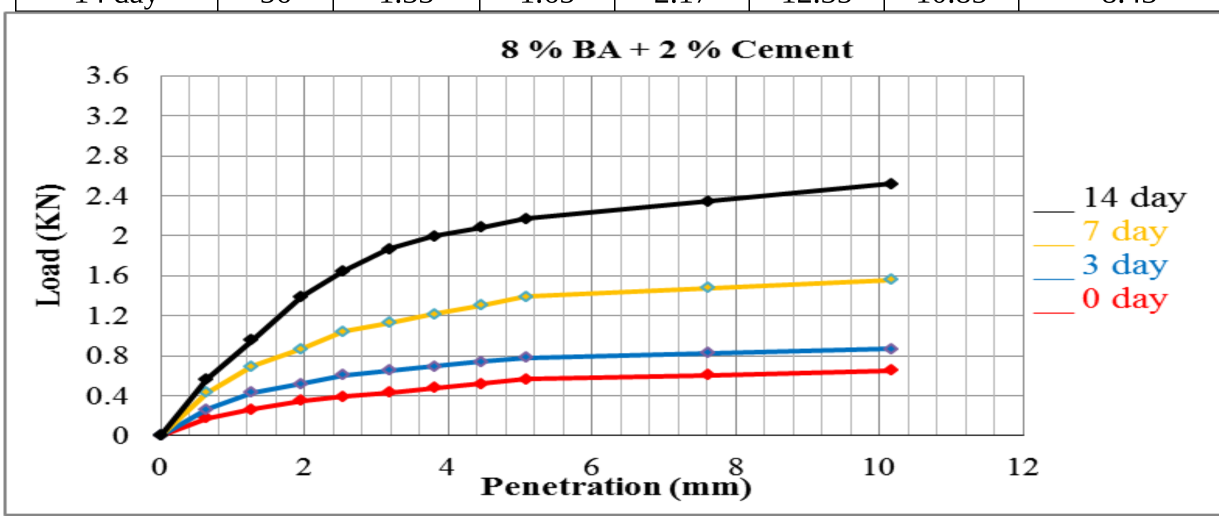
3.1 Bagasse Ash 10 % + 0 % Cement

curing day	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.35	0.16	0.24	1.24	1.19	11.29
3 days	56	1.35	0.19	0.28	1.43	1.41	11.29
7 days	56	1.35	0.22	0.31	1.63	1.56	11.29
14 days	56	1.35	0.36	0.46	2.67	2.28	11.29



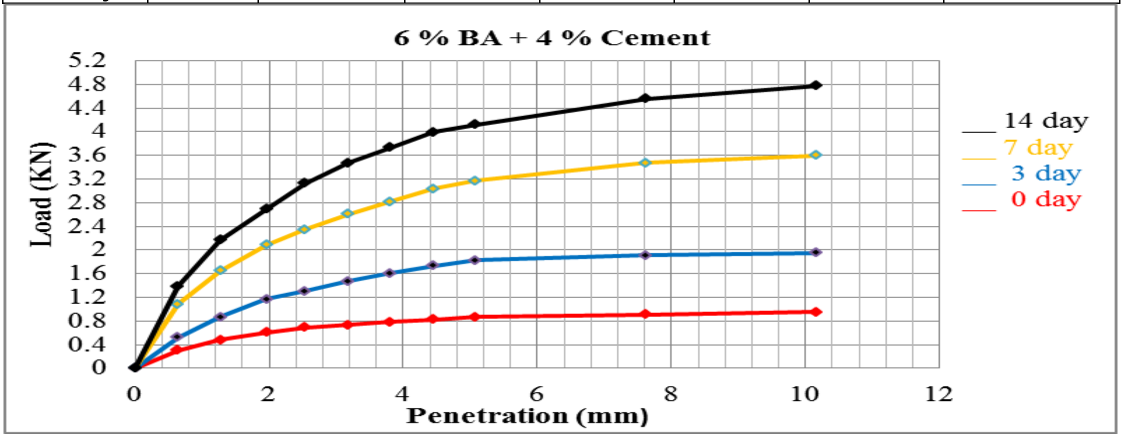
3.2 Bagasse Ash 8 % + 2 % Cement

curing day	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.33	0.39	0.56	2.93	2.82	8.43
3 day	56	1.33	0.61	0.78	4.55	3.91	8.43
7 day	56	1.33	1.04	1.39	7.80	6.95	8.43
14 day	56	1.33	1.65	2.17	12.35	10.85	8.43



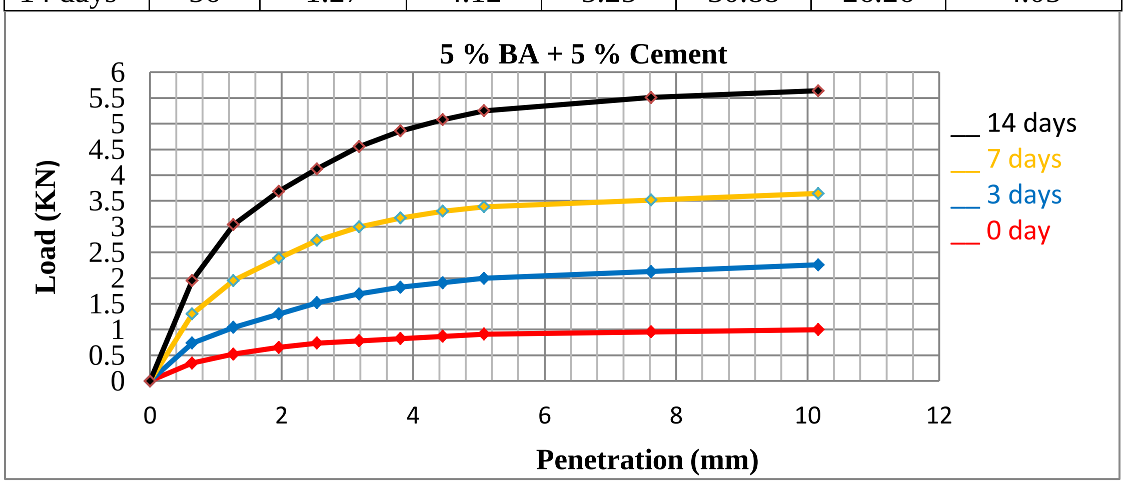
3.3 bagasse ash 6 % + 4 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.30	0.69	0.87	5.20	4.34	6.50
3 day	56	1.30	1.30	1.82	9.75	9.12	6.50
7 day	56	1.30	2.34	3.17	17.55	15.85	6.50
14 day	56	1.30	3.12	4.12	23.4	20.62	6.50



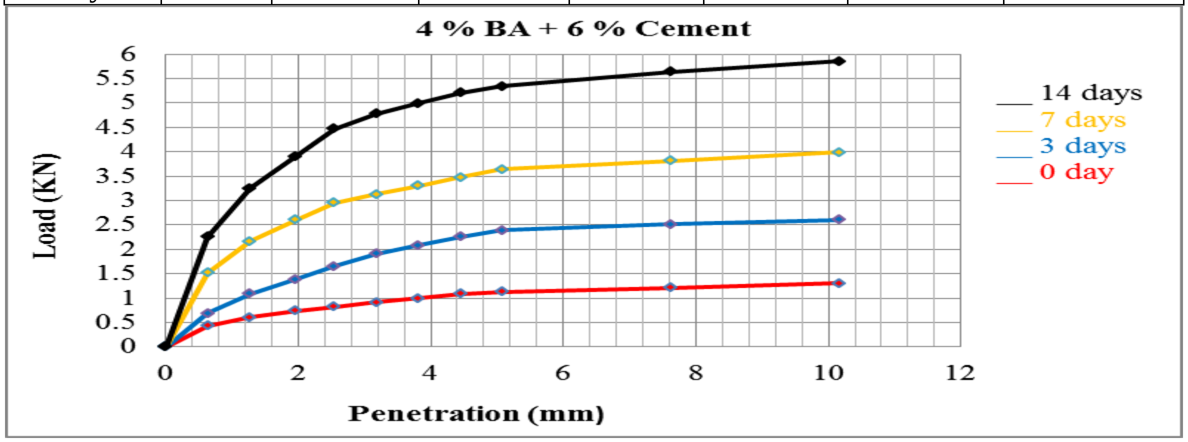
3.4 bagasse ash 5 % + 5 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.27	0.74	0.91	5.53	4.56	4.05
3 days	56	1.27	1.52	2.00	11.38	9.98	4.05
7 days	56	1.27	2.73	3.38	20.48	16.93	4.05
14 days	56	1.27	4.12	5.25	30.88	26.26	4.05



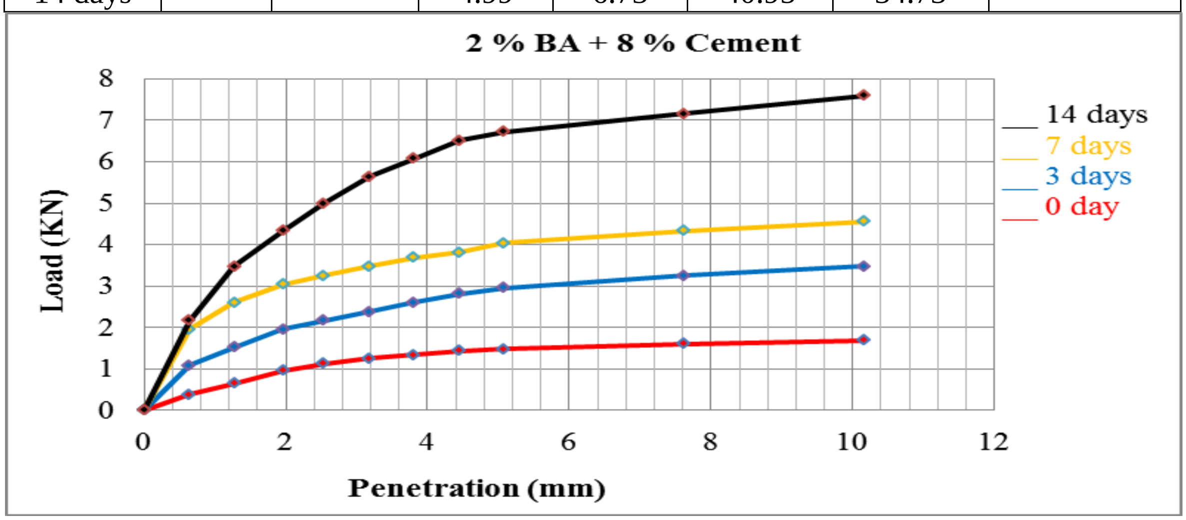
3.5 bagasse ash 4 % + 6 % Cement

curing day	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.26	0.82	1.13	6.18	5.64	2.79
3 days	56	1.26	1.65	2.39	12.35	11.94	2.79
7 days	56	1.26	2.95	3.64	22.10	18.23	2.79
14 days	56	1.26	4.47	5.34	33.48	26.7	2.79



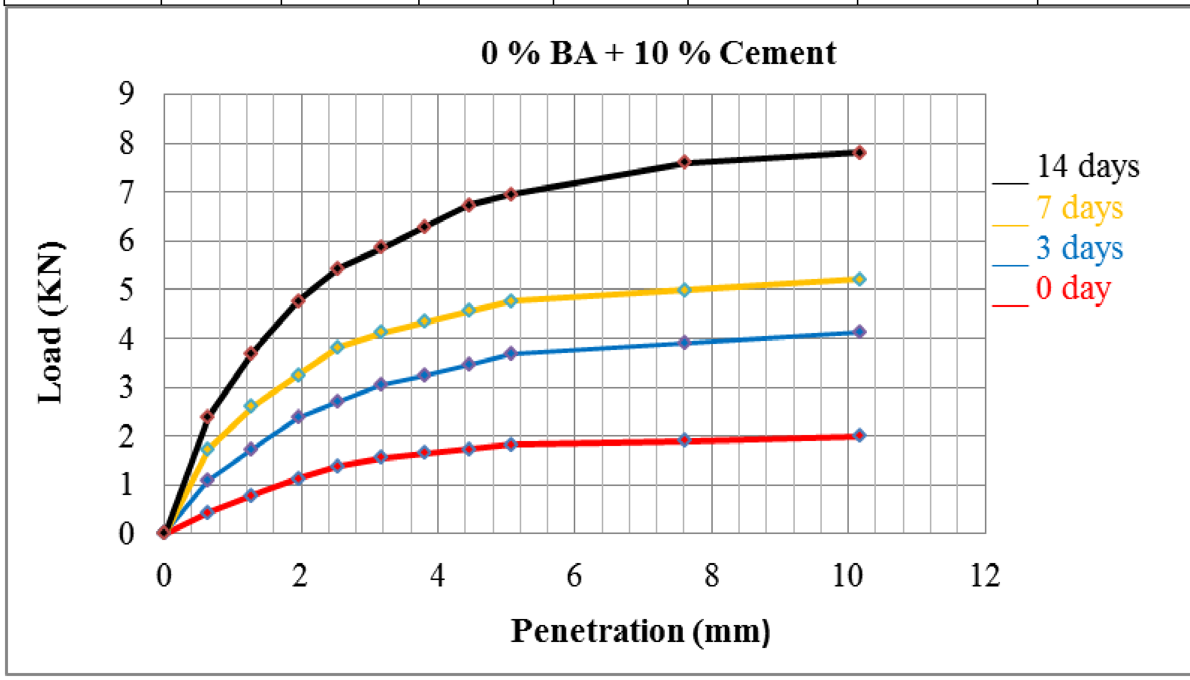
3.6 bagasse ash 2 % + 8 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.23	1.13	1.48	10.7	9.12	1.40
3 days	56	1.23	2.17	2.95	20.48	18.45	1.40
7 days	56	1.23	3.25	4.04	28.93	23.88	1.40
14 days	56	1.23	4.99	6.73	40.95	34.73	1.40



3.7 bagasse ash 0 % + 10 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.23	1.39	1.82	10.40	9.12	0.60
3 days	56	1.23	2.69	3.69	20.15	18.45	0.60
7 days	56	1.23	3.82	4.77	28.60	23.88	0.60
14 days	56	1.23	5.42	6.94	40.63	34.73	0.60

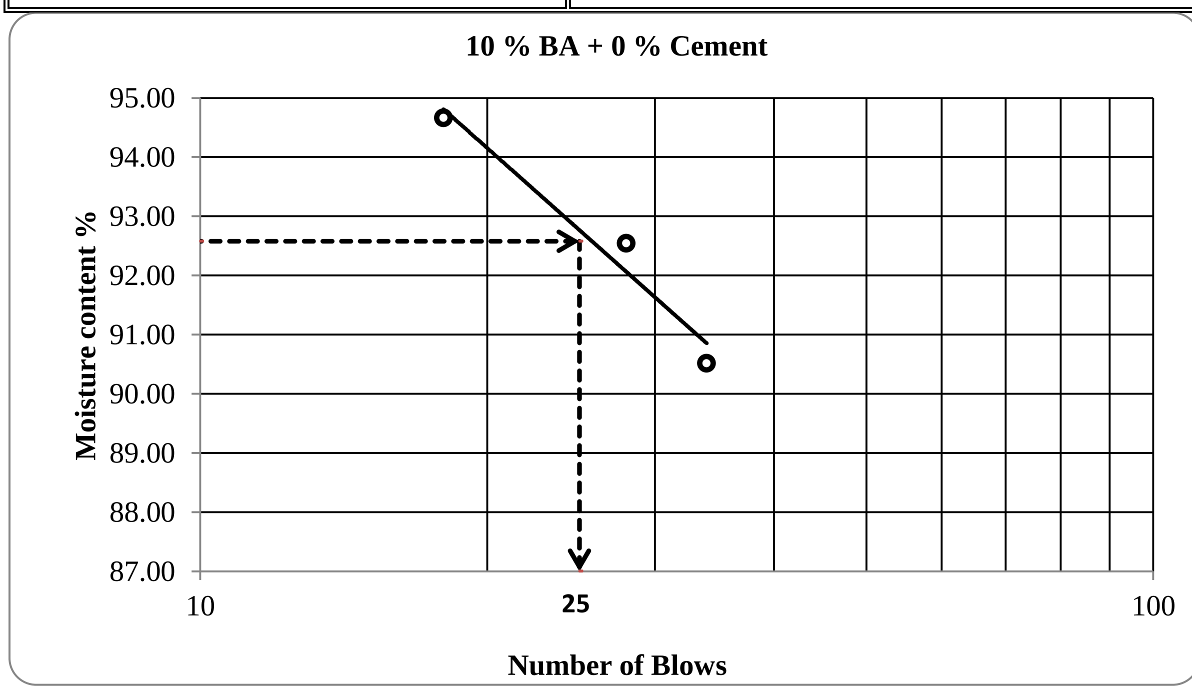


Appendix D: Laboratory Test Result of stabilizing HAP soil sample using bagasse ash mixed with cement

1. Atterberg limit

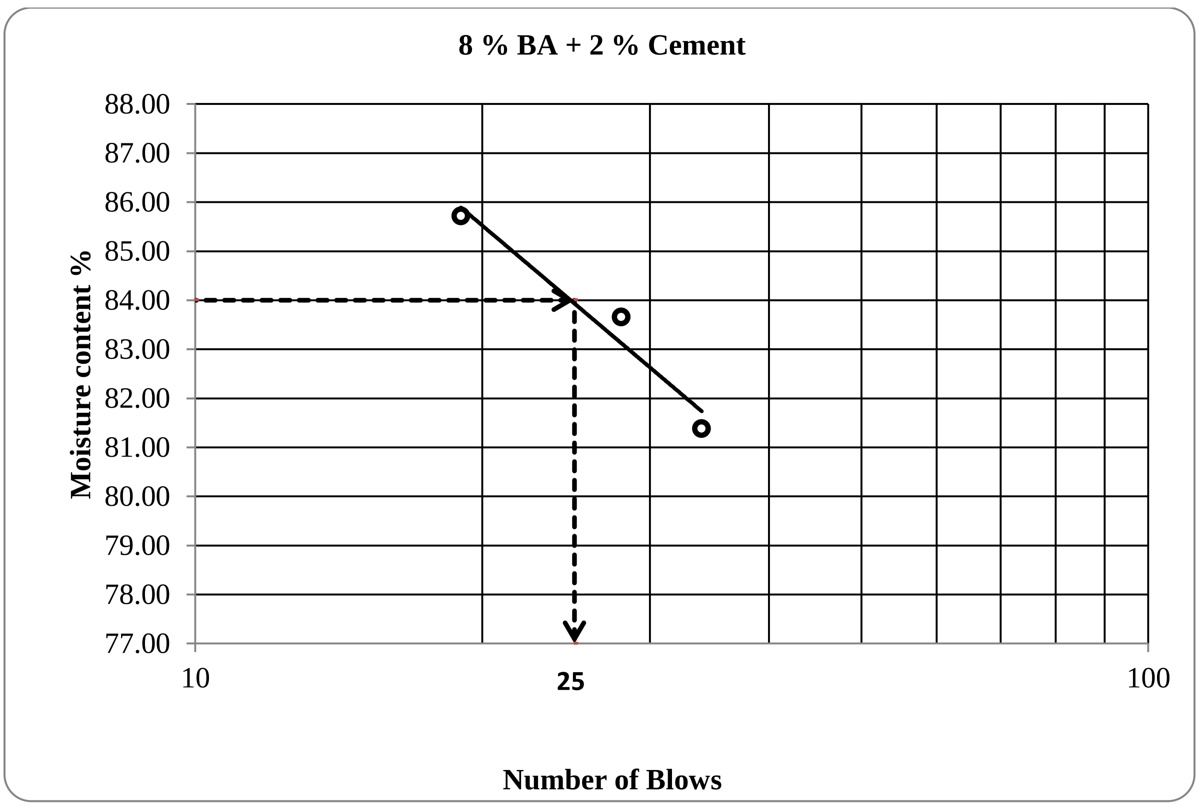
1.1 Bagasse Ash 10 % + 0 % Cement

determination		10 % BA + 0 % C				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	18		
Container No.		T	K	G	B - 10	C-4
Mass of wet soil + container (a)	g	40.12	41.16	42.78	20.22	20.23
Mass of dry soil + container (b)	g	28.95	29.37	30.01	19.17	19.27
Mass of container (c)	g	16.61	16.63	16.52	16.44	16.7
Mass of moisture (a-b)	g	11.17	11.79	12.77	1.05	0.96
Mass of dry soil (b-c)	g	12.34	12.74	13.49	2.73	2.57
Moisture content (w=a-b/b-c x 100)	%	90.52	92.54	94.66	38.46	37.35
Average moisture content (wa)		**92.57**			**37.91**	
Plastic Index		**54.67**				



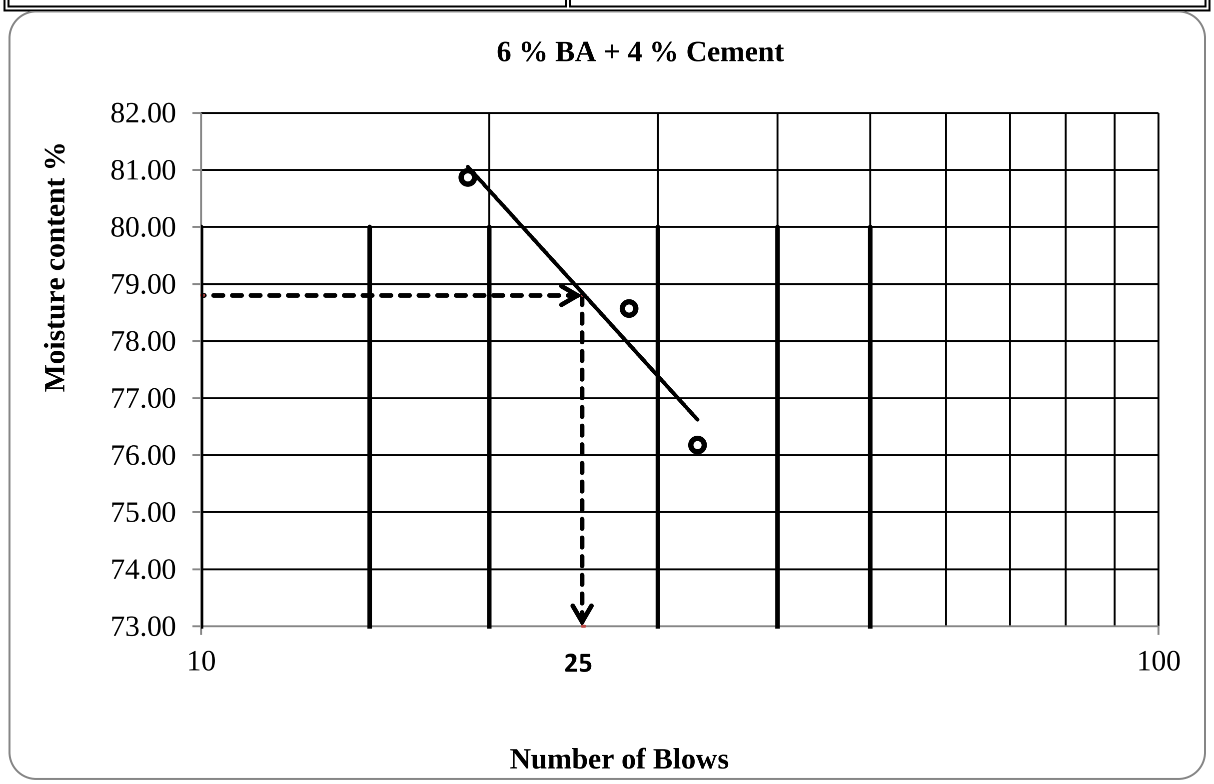
1.2 Bagasse Ash 8 % + 2 % Cement

determination		8 % BA + 2 % C				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	19		
Container No.		F	R	H	C-13	K
Mass of wet soil + container (a)	g	39.50	40.82	41.80	22.21	22.21
Mass of dry soil + container (b)	g	29.23	29.71	30.10	20.55	20.56
Mass of container (c)	g	16.61	16.43	16.45	16.6	16.61
Mass of moisture (a-b)	g	10.27	11.11	11.70	1.66	1.65
Mass of dry soil (b-c)	g	12.62	13.28	13.65	3.95	3.95
Moisture content (w=a-b/b-c x 100)	%	81.38	83.66	85.71	42.03	41.77
Average moisture content (wa)		**83.58**			**41.90**	
Plastic Index		**41.69**				



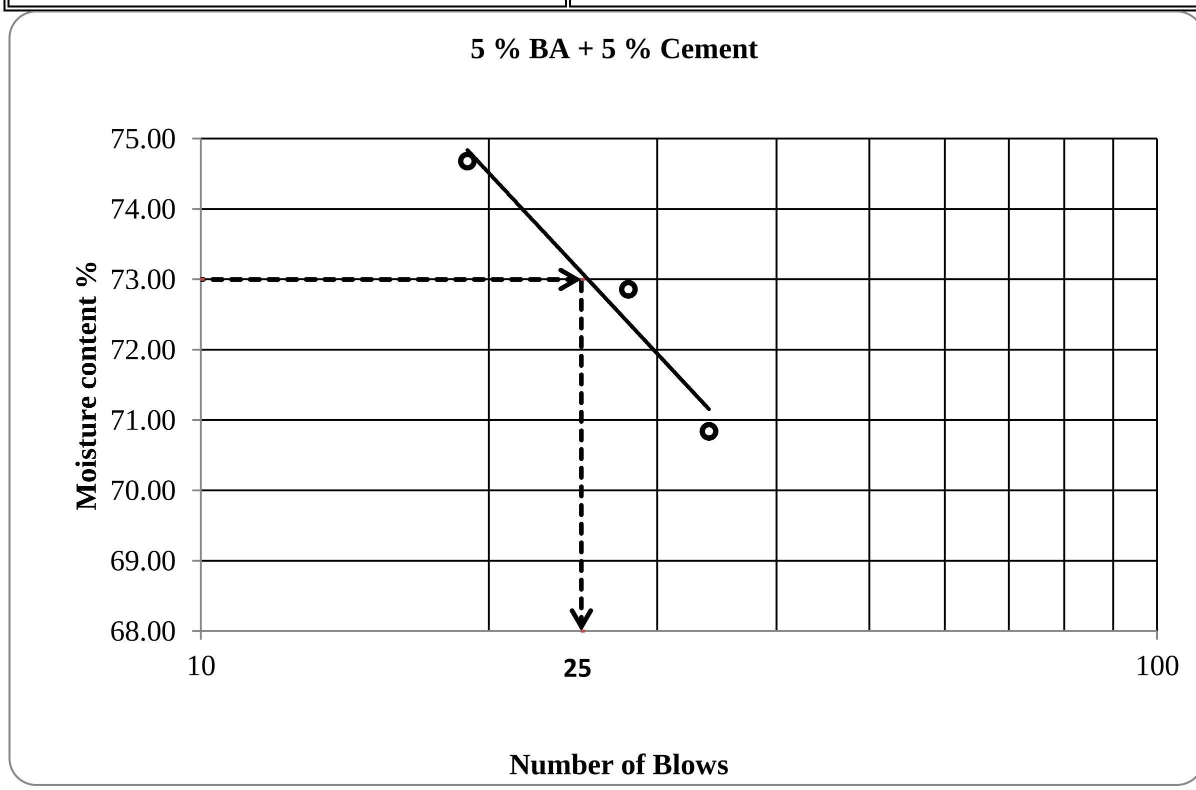
1.3 Bagasse Ash 6 % + 4 % cement

determination		6 % BA + 4 % C				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		33	28	19		
Container No.		C-13	Q	F	B	R
Mass of wet soil + container (a)	g	36.01	37.56	38.32	22.15	22.23
Mass of dry soil + container (b)	g	27.60	29.20	28.60	20.50	20.45
Mass of container (c)	g	16.56	18.56	16.58	16.54	16.46
Mass of moisture (a-b)	g	8.41	8.36	9.72	1.65	1.78
Mass of dry soil (b-c)	g	11.04	10.64	12.02	3.96	3.99
Moisture content (w=a-b/b-c x 100)	%	76.18	78.57	80.87	41.67	44.61
Average moisture content (wa)		**78.54**			**43.14**	
Plastic Index		**35.40**				



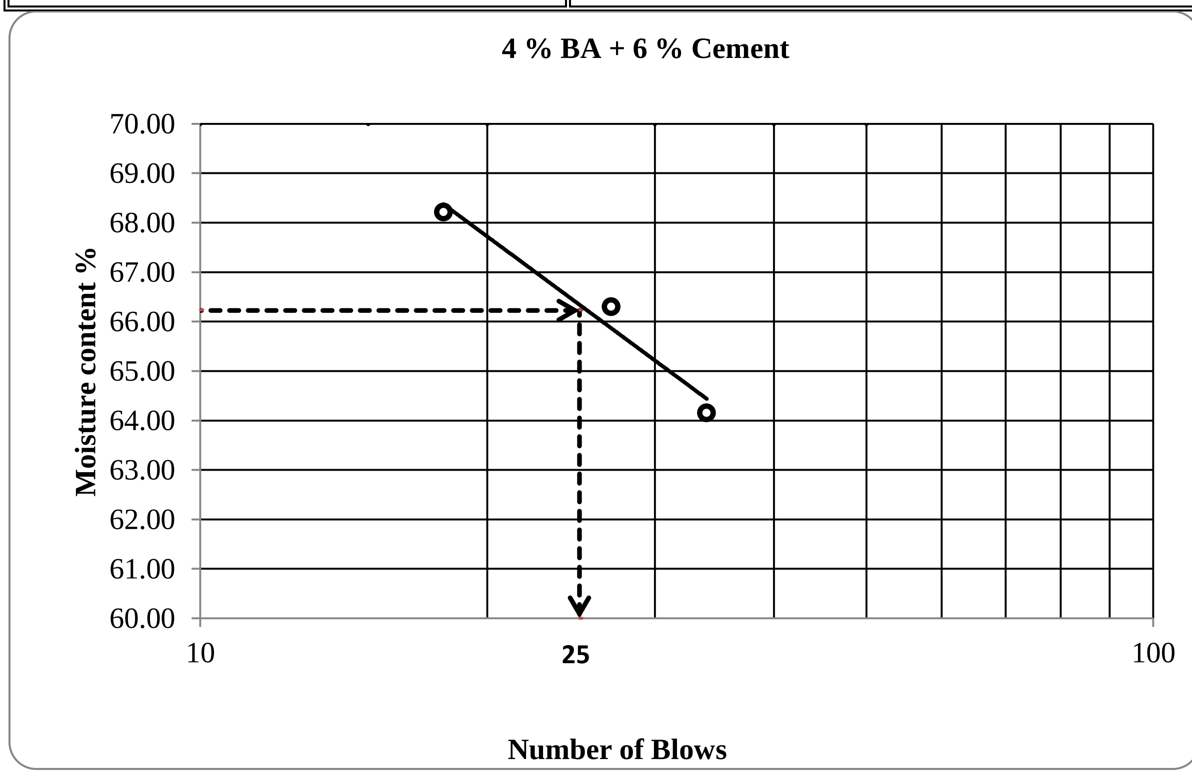
1.4 Bagasse Ash 5 % +5 % Cement

determination		5 % BA + 5% C				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	28	19		
Container No.		D	L	N	C-15	C-17
Mass of wet soil + container (a)	g	36.12	37.32	38.50	22.23	22.31
Mass of dry soil + container (b)	g	29.78	28.65	29.12	20.50	20.48
Mass of container (c)	g	20.83	16.75	16.56	16.7	16.55
Mass of moisture (a-b)	g	6.34	8.67	9.38	1.73	1.83
Mass of dry soil (b-c)	g	8.95	11.90	12.56	3.80	3.93
Moisture content (w=a-b/b-c x 100)	%	70.84	72.86	74.68	45.53	46.56
Average moisture content (wa)		**72.79**			**46.05**	
Plastic Index		**26.75**				



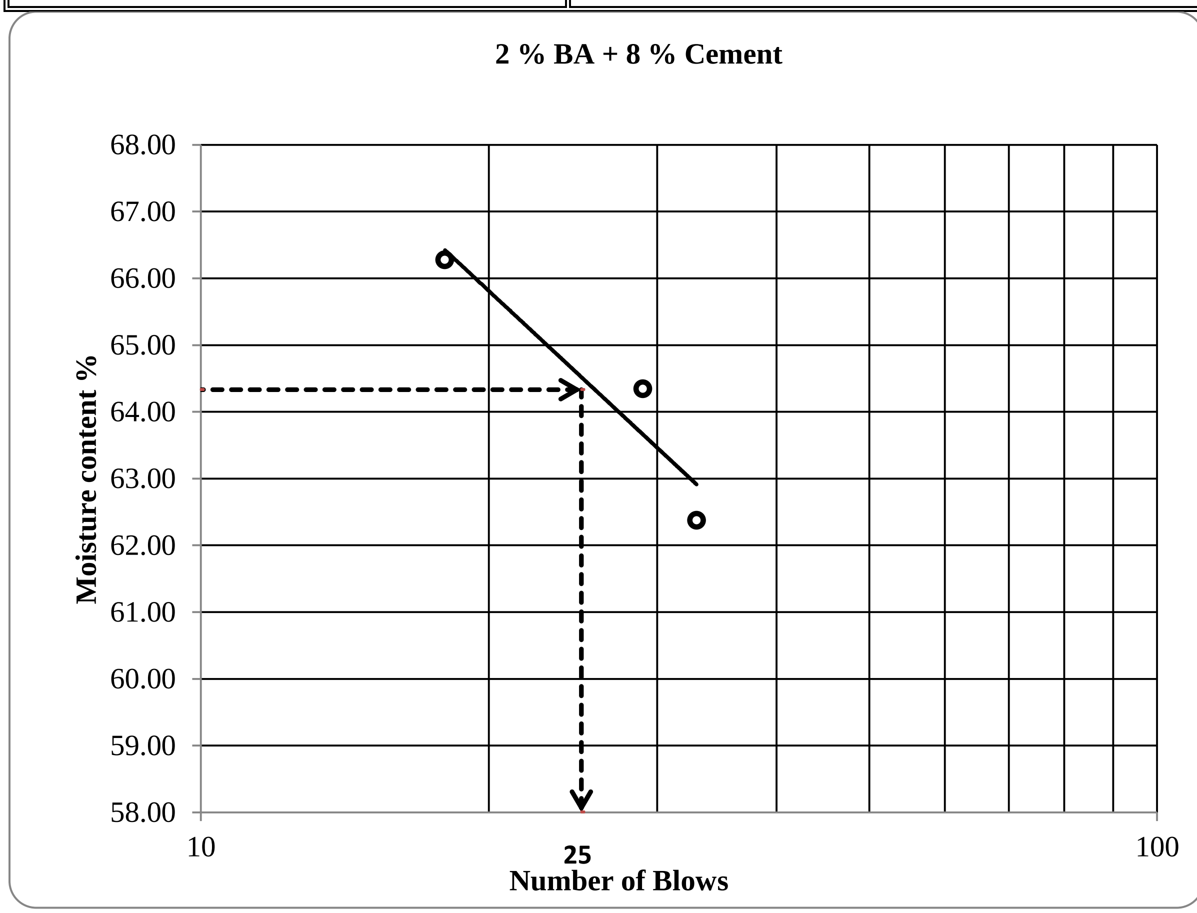
1.5 Bagasse Ash 4 % + 6 % Cement

determination		4 % BA + 6 % C				
		Liquid limit			plastic limit	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	27	18		
Container No.		M	Q	C-24	C-22	M-5
Mass of wet soil + container (a)	g	34.23	35.55	36.70	23.24	23.75
Mass of dry soil + container (b)	g	27.75	29.45	28.48	21.50	21.94
Mass of container (c)	g	17.65	20.25	16.43	18.01	18.4
Mass of moisture (a-b)	g	6.48	6.10	8.22	1.74	1.81
Mass of dry soil (b-c)	g	10.10	9.20	12.05	3.49	3.54
Moisture content (w=a-b/b-c x 100)	%	64.16	66.30	68.22	49.86	51.13
Average moisture content (wa)		**66.23**			**50.49**	
Plastic Index		**15.73**				



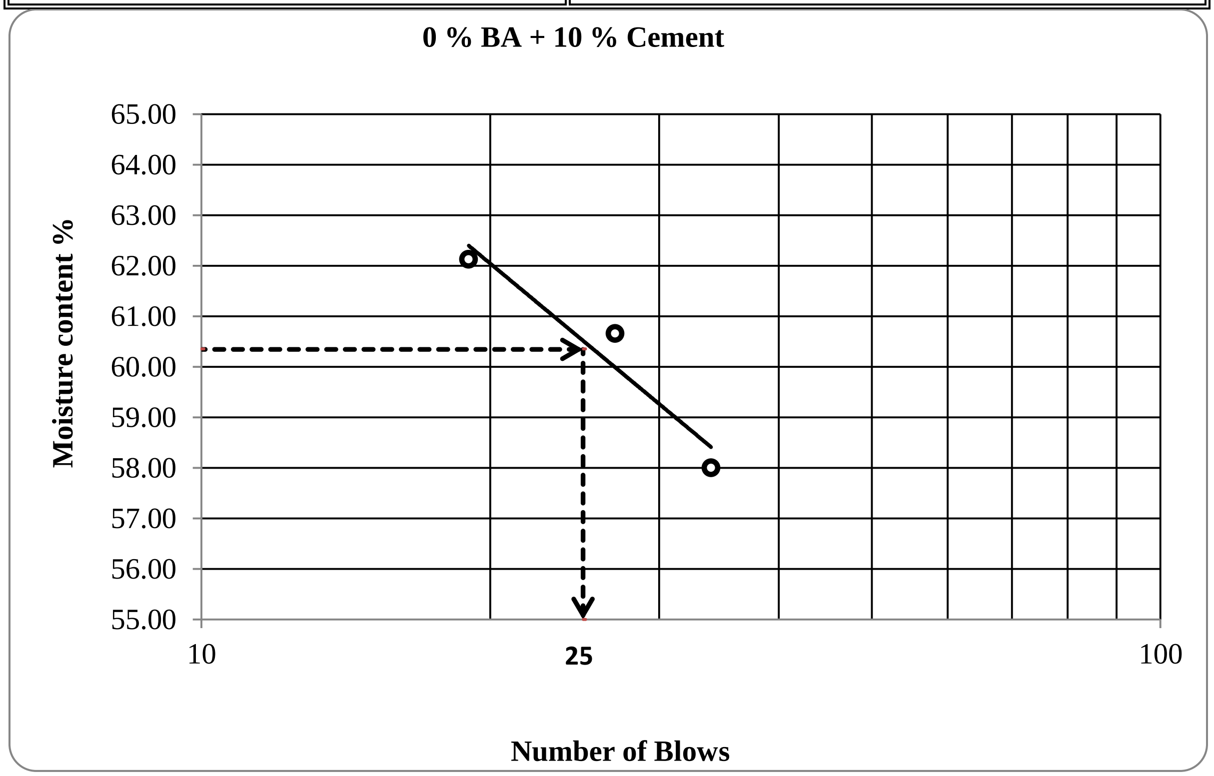
1.6 Bagasse Ash 2 % + 8 % Cement

determination		2 % BA + 8 % C				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		33	29	18		
Container No.		E	K	C-12	C-15	C-17
Mass of wet soil + container (a)	g	37.41	38.12	39.45	22.01	22.02
Mass of dry soil + container (b)	g	29.42	29.71	30.35	20.18	20.13
Mass of container (c)	g	16.61	16.64	16.62	16.68	16.51
Mass of moisture (a-b)	g	7.99	8.41	9.10	1.83	1.89
Mass of dry soil (b-c)	g	12.81	13.07	13.73	3.50	3.62
Moisture content (w=a-b/b-c x 100)	%	62.37	64.35	66.28	52.29	52.21
Average moisture content (wa)		**64.33**			**52.25**	
Plastic Index		**12.08**				



1.7 Bagasse Ash 0 % + 10 % Cement

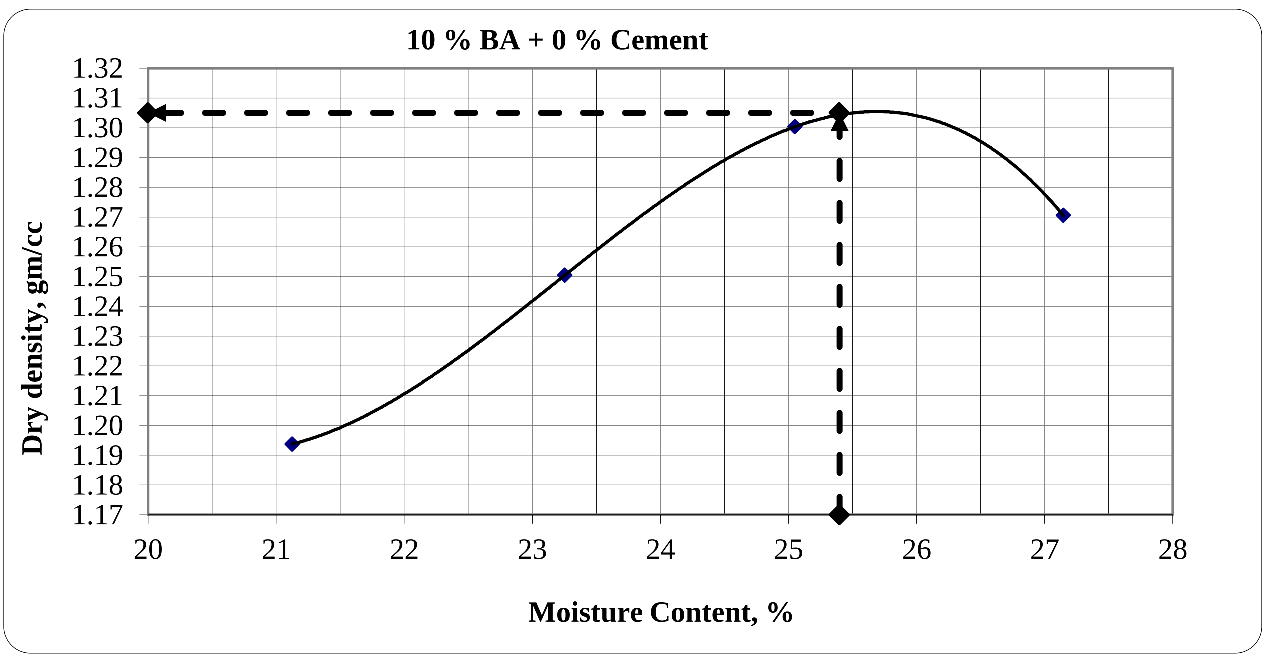
determination		0 % BA + 10 %				
		Liquid limit			**plastic limit**	
trial number		**1**	**2**	**3**	**1**	**2**
No. of blows		34	27	19		
Container No.		B	C	E	M	Q
Mass of wet soil + container (a)	g	39.25	40.01	40.20	23.01	23.01
Mass of dry soil + container (b)	g	30.99	32.50	31.16	20.75	21.76
Mass of container (c)	g	16.75	20.12	16.61	16.52	19.51
Mass of moisture (a-b)	g	8.26	7.51	9.04	2.26	1.25
Mass of dry soil (b-c)	g	14.24	12.38	14.55	4.23	2.25
Moisture content (w=a-b/b-c x 100)	%	58.01	60.66	62.13	53.43	55.56
Average moisture content (wa)		**60.27**			**54.49**	
Plastic Index		**5.77**				



2. Compaction test results

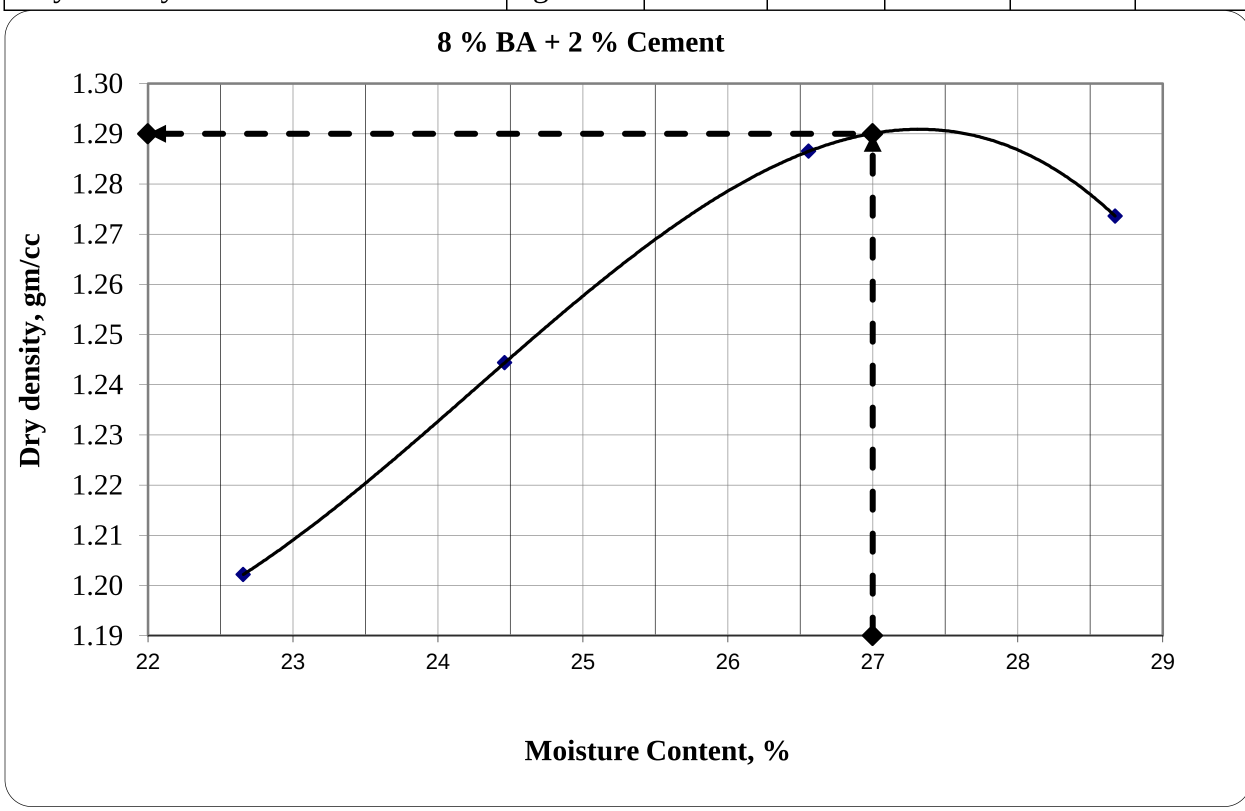
2.1 Bagasse Ash 10 % + 0 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	12	14	16	18	
Wt. of Mold + Wet Soil	gm.	5180	5270	5350	5340	
Wt. of Mold	gm.	3815	3815	3815	3815	
Wt. Wet Soil	gm.	1365	1455	1535	1525	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.446	1.541	1.626	1.615	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	151.32	172.16	183.45	179.53	200.56
Wt. Cont + Dry soil	gm.	129.20	144.23	151.65	146.54	185.95
Weight of Water	gm.	22.1	27.9	31.8	33.0	14.6
Weight of Container	gm.	24.50	24.13	24.71	25.02	25.96
Weight of Dry Soil	gm.	104.7	120.1	126.9	121.5	160.0
Moisture Content	%	21.13	23.26	25.05	27.15	9.13
Dry Density	gr/cm^3	1.19	1.25	1.300	1.27	



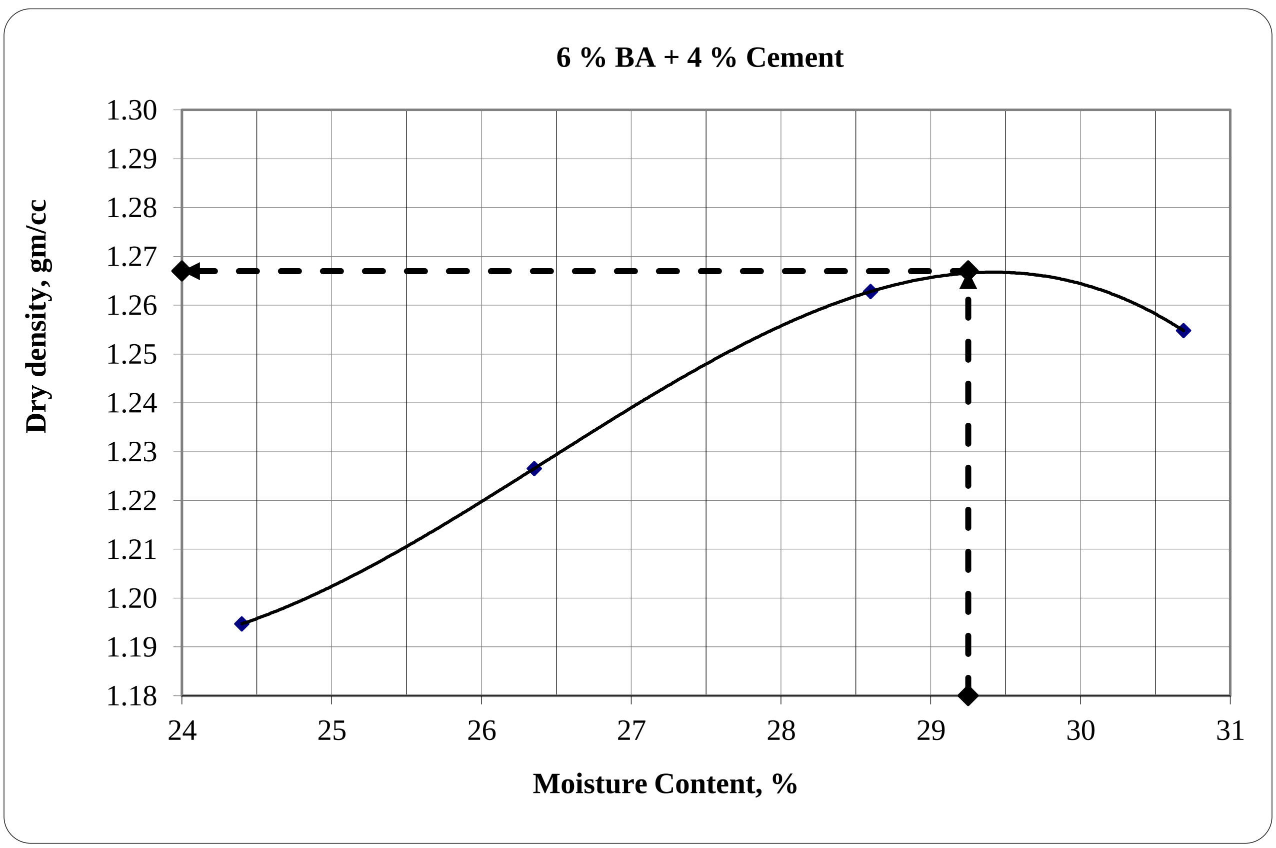
2.2 Bagasse ash 8 % + 2 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	12	14	16	18	
Wt. of Mold + Wet Soil	gm.	5205	5275	5350	5360	
Wt. of Mold	gm.	3813	3813	3813	3813	
Wt. Wet Soil	gm.	1392	1462	1537	1547	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.475	1.549	1.628	1.639	NMC
Container	No.	C	H	G	I	X-2
Wt. Cont. + Wet soil	gm.	142.01	136.54	121.45	125.8	224.17
Wt. Cont. + Dry soil	gm.	120.58	114.54	100.87	103.45	206.28
Weight of Water	gm.	21.4	22.0	20.6	22.4	17.9
Weight of Container	gm.	26.00	24.60	23.38	25.50	24.17
Weight of Dry Soil	gm.	94.6	89.9	77.5	78.0	182.1
Moisture Content	%	22.66	24.46	26.56	28.67	9.82
Dry Density	gr/cm^3	1.20	1.24	1.287	1.27	



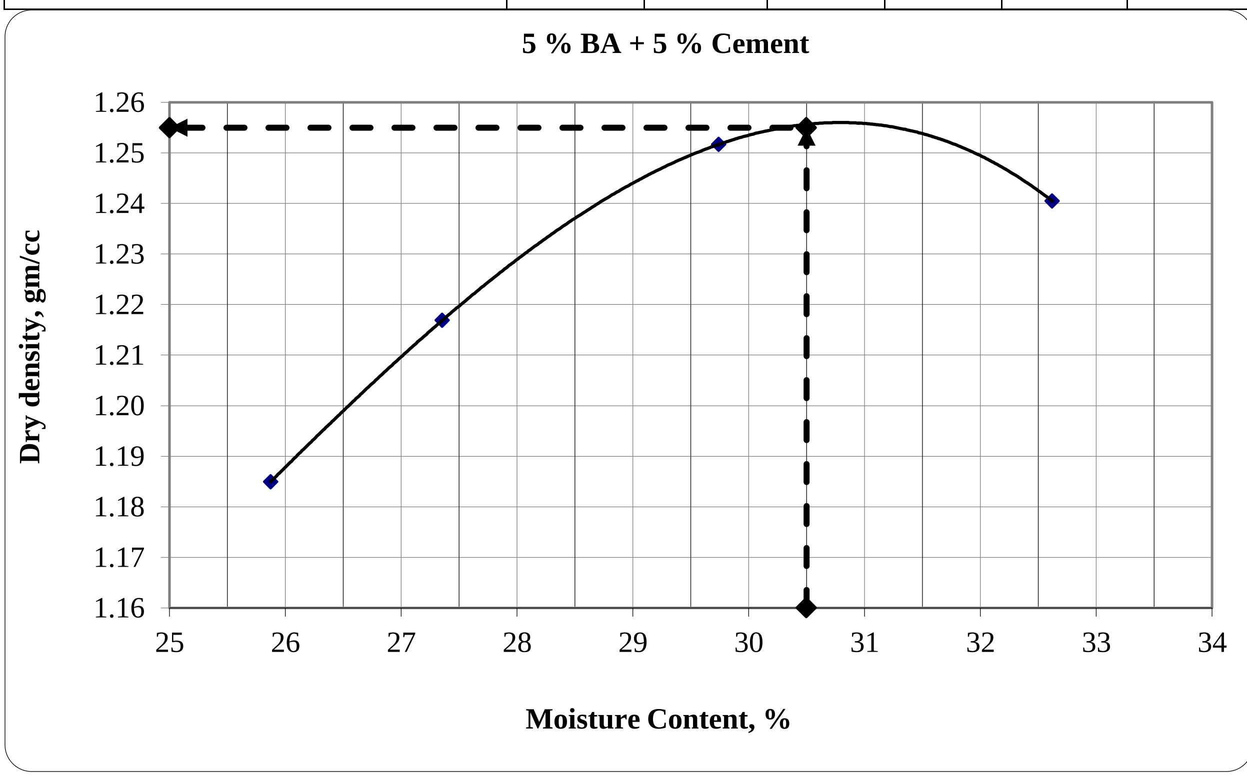
2.3 Bagasse Ash 6 % + 4 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	15	17	19	21	
Wt. of Mold + Wet Soil	gm.	5220	5280	5350	5365	
Wt. of Mold	gm.	3817	3817	3817	3817	
Wt. Wet Soil	gm.	1403	1463	1533	1548	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.486	1.550	1.624	1.640	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont. + Wet soil	gm.	193.01	148.60	134.00	115.01	126.2
Wt. Cont.+ Dry soil	gm.	160.30	124.10	110.00	93.86	118.02
Weight of Water	gm.	32.7	24.5	24.0	21.2	8.2
Weight of Container	gm.	26.24	31.13	26.08	24.94	26.20
Weight of Dry Soil	gm.	134.1	93.0	83.9	68.9	91.8
Moisture Content	%	24.40	26.35	28.60	30.69	8.91
Dry Density	gr/cm^3	1.19	1.23	1.263	1.25	



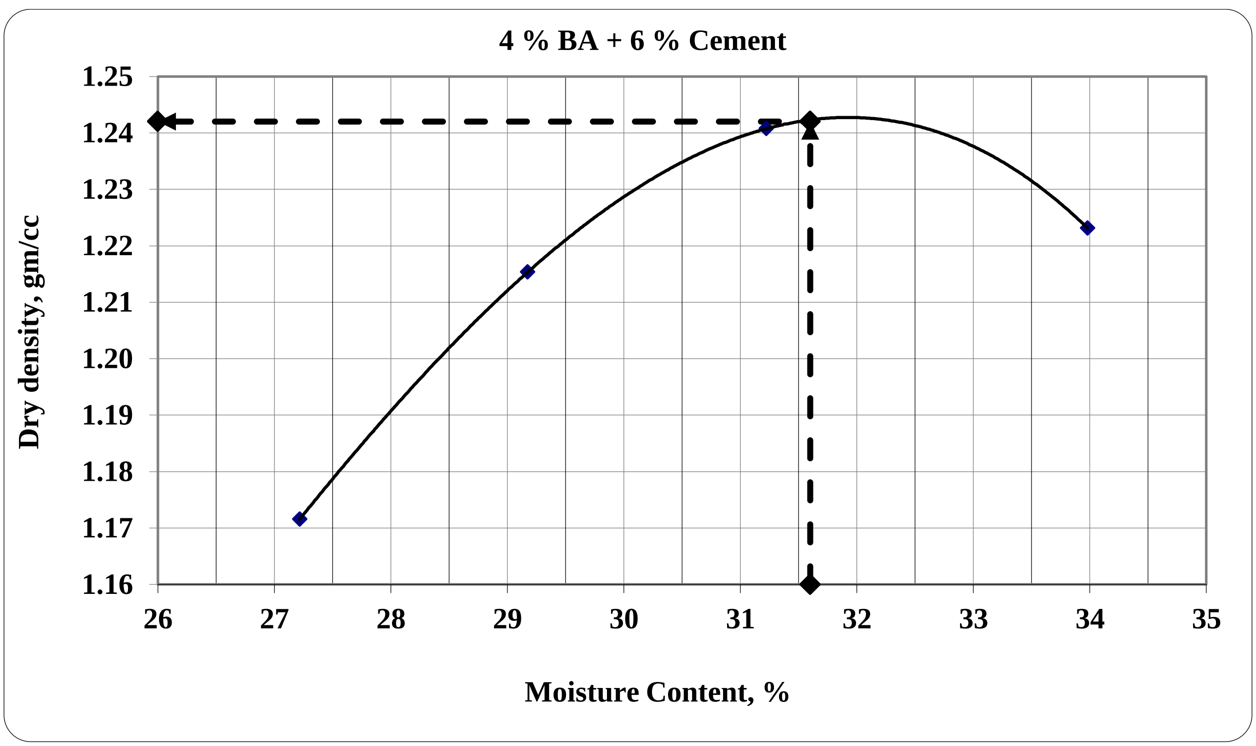
2.4 Bagasse Ash 5 % + 5 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	17	19	21	24	
Wt. of Mold + Wet Soil	gm.	5225	5280	5350	5370	
Wt. of Mold	gm.	3817	3817	3817	3817	
Wt. Wet Soil	gm.	1408	1463	1533	1553	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.492	1.550	1.624	1.645	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	132.36	120.02	135.70	135.32	125.04
Wt. Cont + Dry soil	gm.	109.90	99.50	110.20	108.00	117.33
Weight of Water	gm.	22.5	20.5	25.5	27.3	7.7
Weight of Container	gm.	23.09	24.49	24.46	24.25	25.04
Weight of Dry Soil	gm.	86.8	75.0	85.7	83.8	92.3
Moisture Content	%	25.87	27.36	29.74	32.62	8.35
Dry Density	gr/cm^3	1.18	1.22	1.252	1.24	



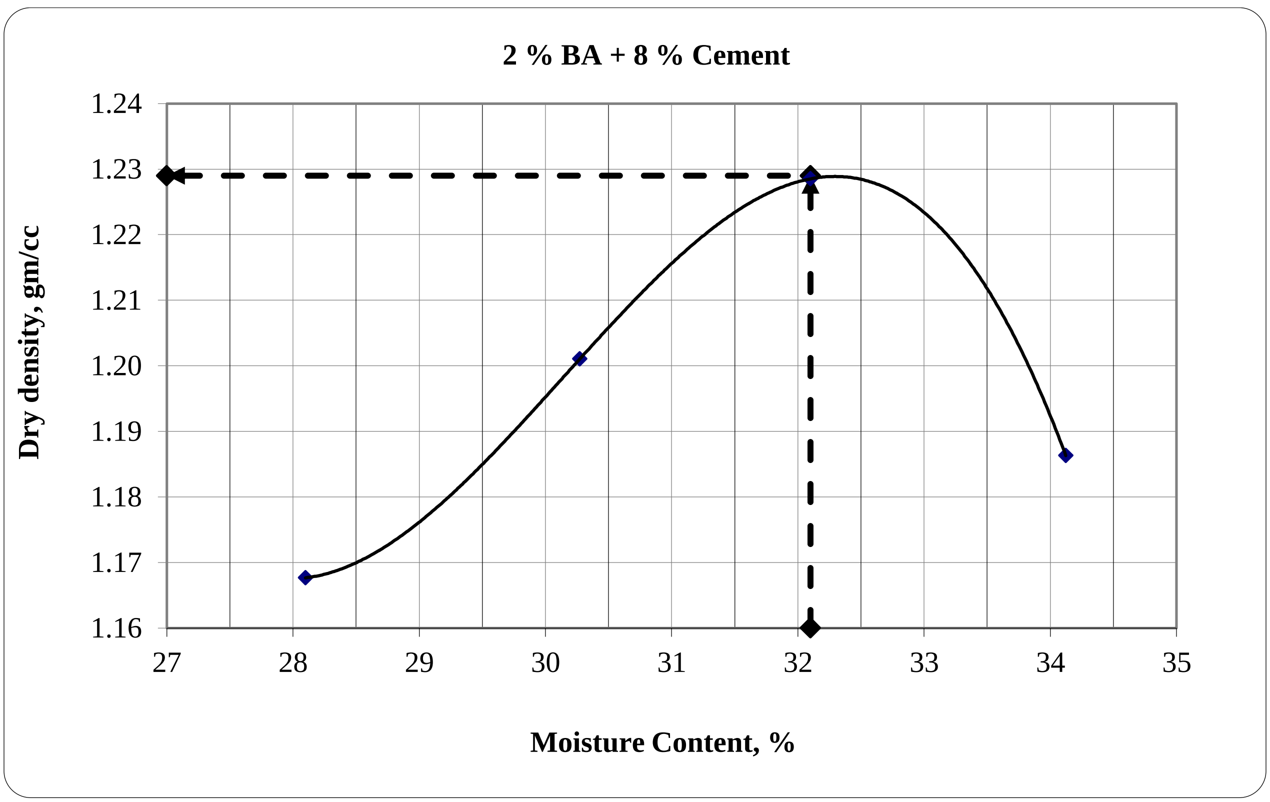
2.5 Bagasse Ash 4 % + 6 % Cement

Trial No.		1	2	3	4	
Assumed Water Content	%	17	19	21	23	
Wt. of Mold + Wet Soil	gm.	5225	5300	5355	5365	
Wt. of Mold	gm.	3818	3818	3818	3818	
Wt. Wet Soil	gm.	1407	1482	1537	1547	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.490	1.570	1.628	1.639	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont. + Wet soil	gm.	200.32	196.10	182.01	155.8	131
Wt. Cont. + Dry soil	gm.	164.00	157.70	144.70	124.02	121.80
Weight of Water	gm.	36.3	38.4	37.3	31.8	9.2
Weight of Container	gm.	30.56	26.08	25.21	30.50	31.00
Weight of Dry Soil	gm.	133.4	131.6	119.5	93.5	90.8
Moisture Content	%	27.22	29.17	31.22	33.98	10.13
Dry Density	gr/cm^3	1.17	1.22	1.241	1.22	



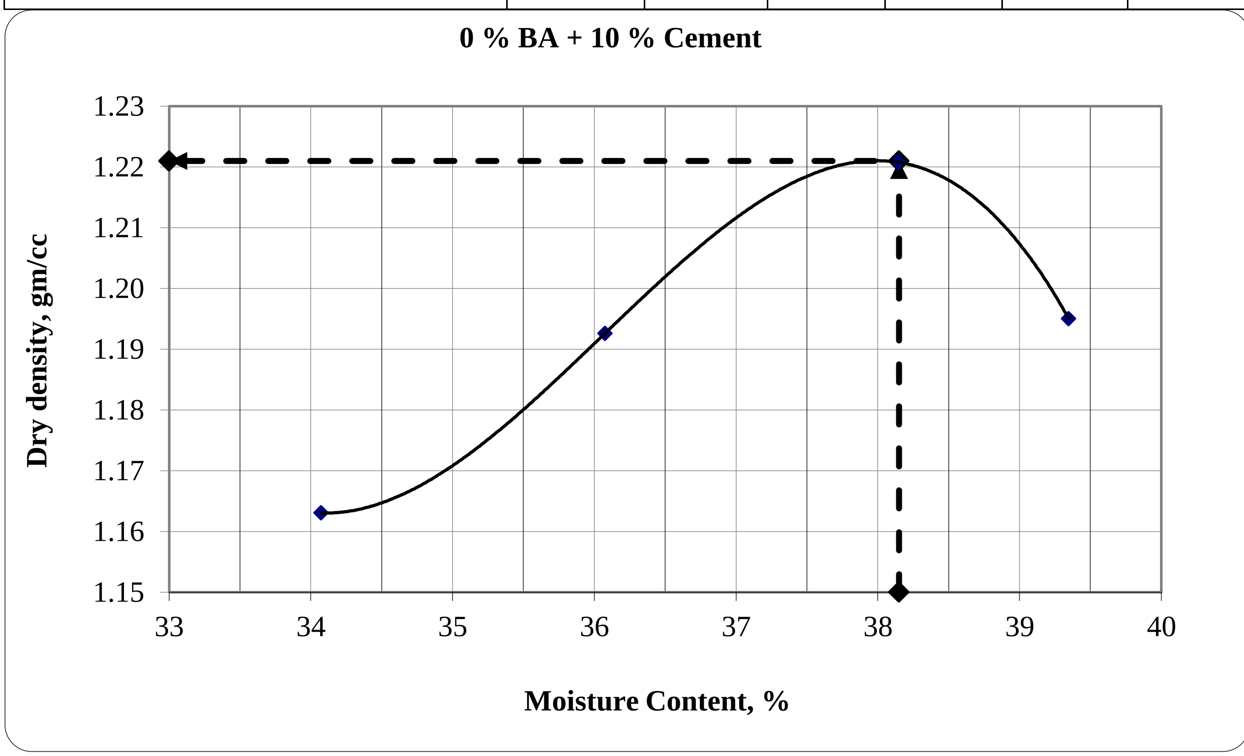
2.6 Bagasse Ash 2 % + 8 % Cement

Assumed Water Content	%	20	22	24	26	
Wt. of Mold + Wet Soil	gm.	5230	5295	5350	5320	
Wt. of Mold	gm.	3818	3818	3818	3818	
Wt. Wet Soil	gm.	1412	1477	1532	1502	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.496	1.565	1.623	1.591	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont.+ Wet soil	gm.	200.12	140.35	144.12	151.21	170.35
Wt. Cont. + Dry soil	gm.	167.30	115.23	116.52	119.10	162.43
Weight of Water	gm.	32.8	25.1	27.6	32.1	7.9
Weight of Container	gm.	50.50	32.25	30.54	25.00	70.35
Weight of Dry Soil	gm.	116.8	83.0	86.0	94.1	92.1
Moisture Content	%	28.10	30.27	32.10	34.12	8.60
Dry Density	gr/cm^3	1.17	1.20	1.229	1.19	



2.7 Bagasse Ash 0 % + 10 % Cement

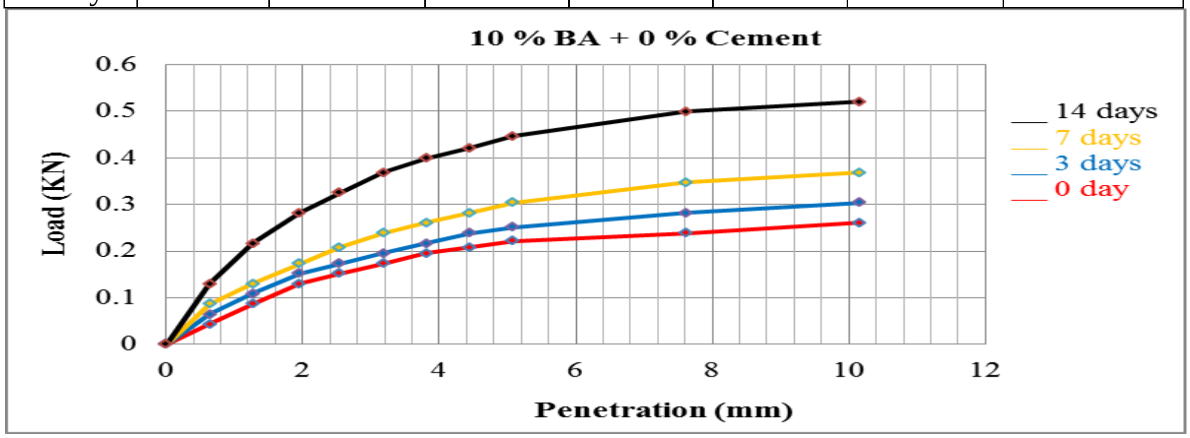
Trial No.		1	2	3	4	
Assumed Water Content	%	26	28	30	32	
Wt. of Mold + Wet Soil	gm.	5290	5350	5410	5390	
Wt. of Mold	gm.	3818	3818	3818	3818	
Wt. Wet Soil	gm.	1472	1532	1592	1572	
Volume of Mold	cm^3	944	944	944	944	
Wet Density	gr/cm^3	1.559	1.623	1.686	1.665	NMC
Container	No.	E	I	D	F	X-2
Wt. Cont + Wet soil	gm.	150.24	241.12	220.64	180.83	170.35
Wt. Cont + Dry soil	gm.	119.95	194.56	179.65	138.56	162.43
Weight of Water	gm.	30.3	46.6	41.0	42.3	7.9
Weight of Container	gm.	31.05	65.50	72.20	31.13	70.35
Weight of Dry Soil	gm.	88.9	129.1	107.5	107.4	92.1
Moisture Content	%	34.07	36.08	38.15	39.35	8.60
Dry Density	gr/cm^3	1.16	1.19	1.221	1.20	



3. CBR and CBR Swell test result

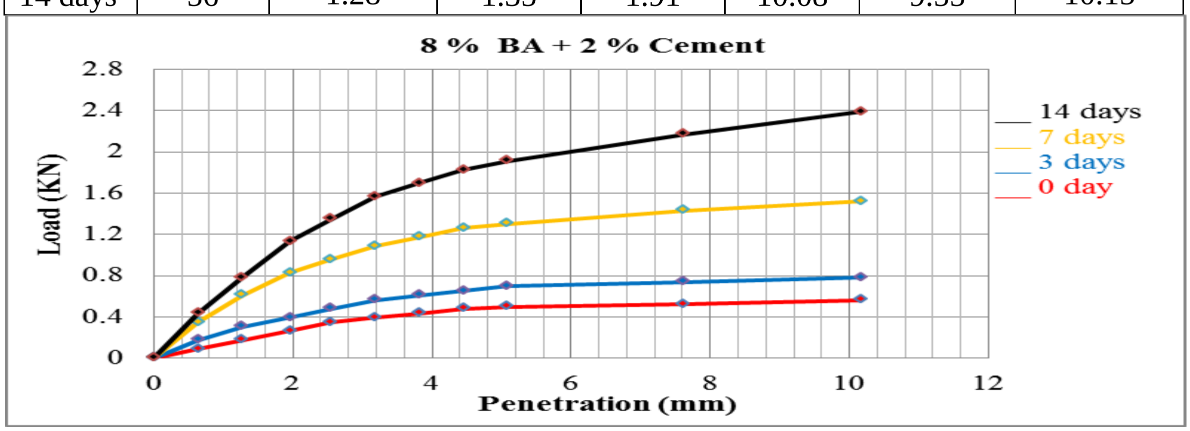
3.1 Bagasse Ash 10 % + 0 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.30	0.15	0.22	1.14	1.11	15.92
3days	56	1.30	0.17	0.25	1.30	1.26	15.92
7 days	56	1.30	0.21	0.30	1.56	1.52	15.92
14 days	56	1.30	0.33	0.45	2.44	2.24	15.92



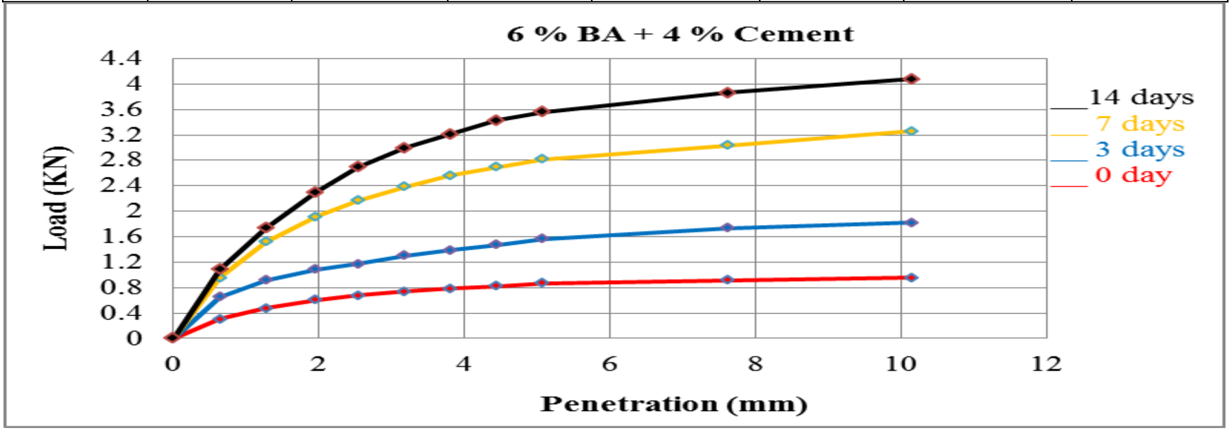
3.2 Bagasse Ash 8 % + 2 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.28	0.35	0.50	2.60	2.50	10.15
3 days	56	1.28	0.48	0.69	3.58	3.47	10.15
7 days	56	1.28	0.95	1.30	7.15	6.51	10.15
14 days	56	1.28	1.35	1.91	10.08	9.55	10.15



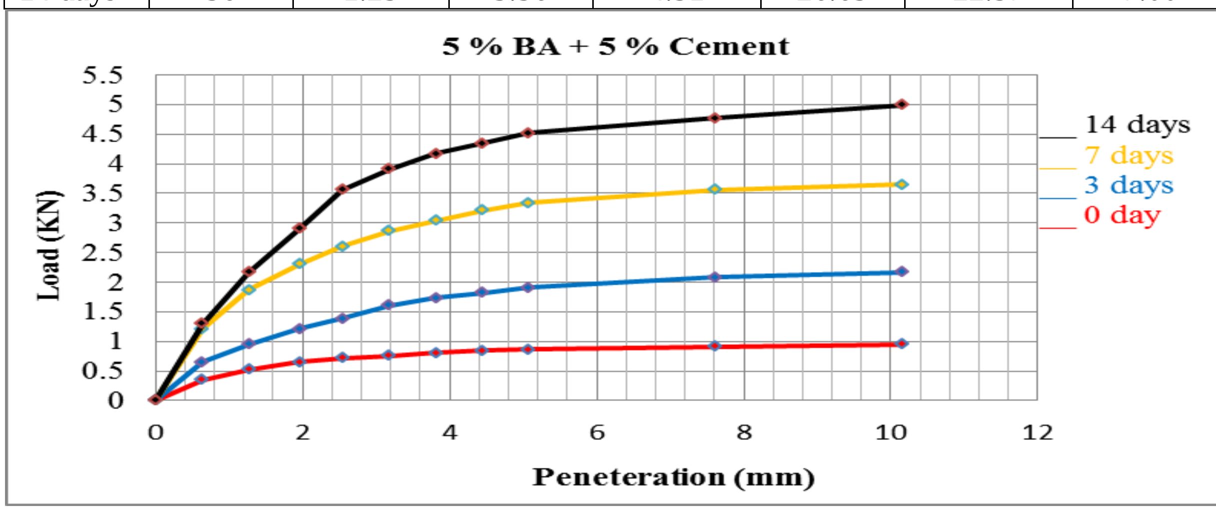
3.3 Bagasse Ash 6 % + 4 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.26	0.67	0.87	5.04	4.34	8.63
3 days	56	1.26	1.17	1.56	8.78	7.81	8.63
7 days	56	1.26	2.17	2.82	16.25	14.11	8.63
14 days	56	1.26	2.69	3.56	20.15	17.8	8.63



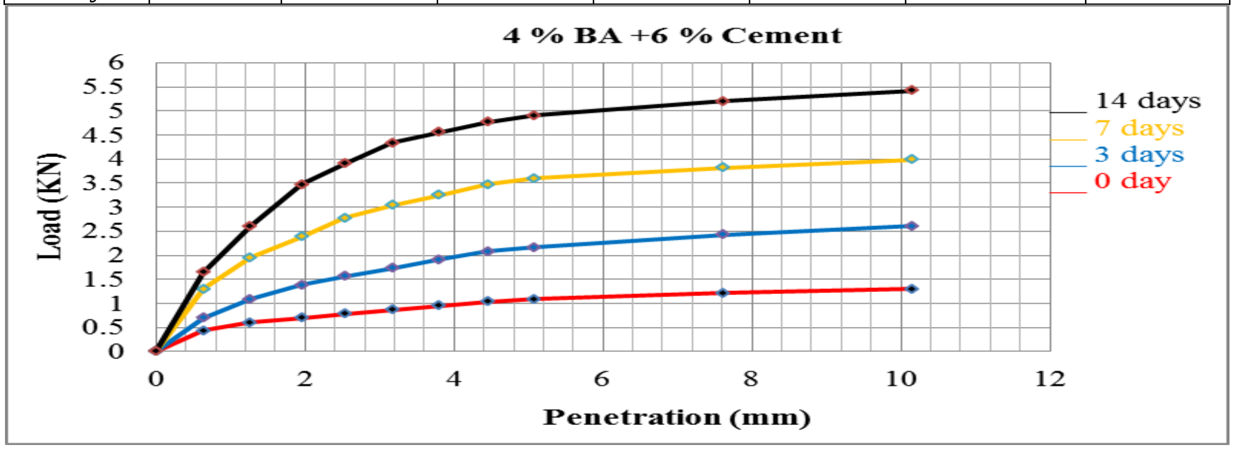
3.4 Bagasse Ash 5 % + 5 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.25	0.72	0.87	5.36	4.34	7.00
3 days	56	1.25	1.39	1.91	10.40	9.55	7.00
7 days	56	1.25	2.60	3.34	19.50	16.71	7.00
14 days	56	1.25	3.56	4.51	26.65	22.57	7.00



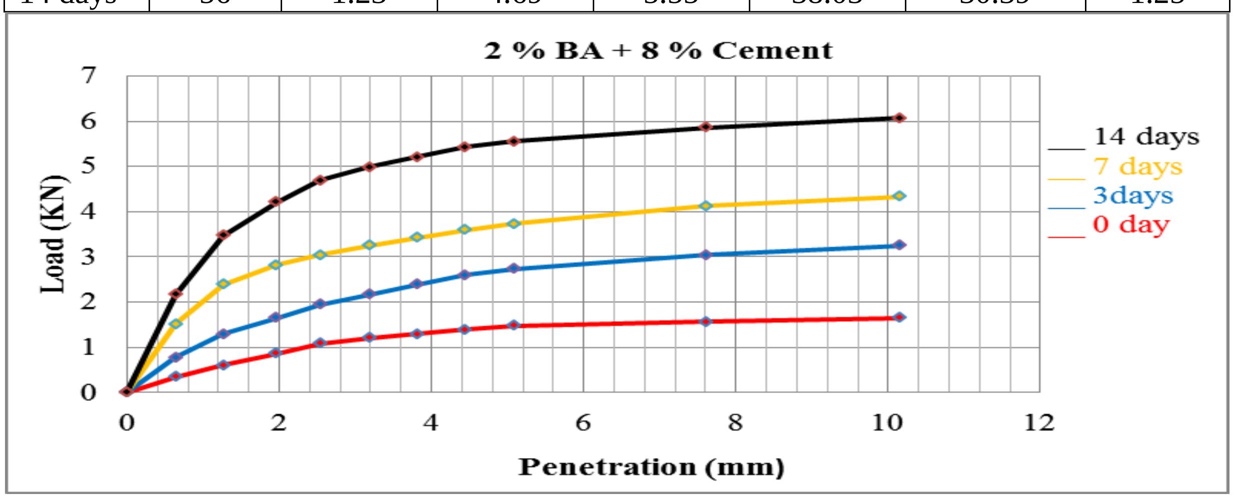
3.5 Bagasse Ash 4 % + 6 % Cement

curing says	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.24	0.78	1.08	5.85	5.43	3.18
3 days	56	1.24	1.56	2.17	11.70	10.85	3.18
7 days	56	1.24	2.78	3.60	20.80	18.02	3.18
14 days	56	1.24	3.9	4.9	29.25	24.53	3.18



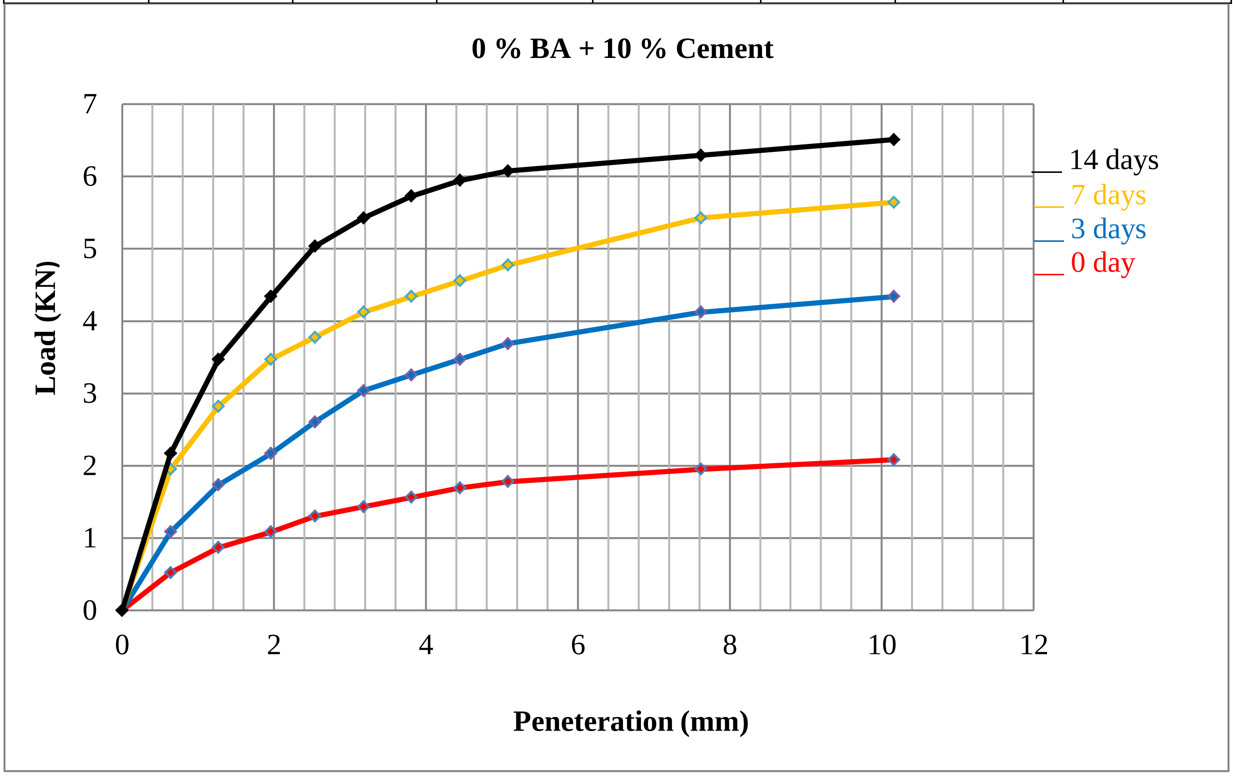
3.6 Bagasse Ash 2 % + 8 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR (%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.23	1.08	1.48	10.07	8.90	1.25
3 days	56	1.23	1.95	2.73	19.83	18.45	1.25
7 days	56	1.23	3.04	3.73	28.60	23.88	1.25
14 days	56	1.23	4.69	5.55	38.03	30.39	1.25



3.7 Bagasse Ash 0 % +10 % Cement

curing days	Blows	Dry Density, g/cc	LOAD (KN)		CBR(%)		Swell
			2.54mm	5.08mm	2.54mm	5.08mm	%
0 day	56	1.22	1.30	1.78	9.75	8.90	0.93
3 days	56	1.22	2.60	3.69	19.50	18.45	0.93
7 days	56	1.22	3.77	4.77	28.28	23.88	0.93
14 days	56	1.22	5.03	6.07	37.7	30.39	0.93



Appendix E: Photos taken during the research work