

Stabilization of Black Cotton Soil Using Fly Ash of Municipal Waste Byproduct



Sime Wadajo Bejiga

A Thesis Submitted to the department of Civil Engineering

School of Civil Engineering and Architecture

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

**Office of Graduate Studies
Adama Science and Technology University**

September, 2023
Adama, Ethiopia

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Advisor: Argaw Asha (PhD)

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DECLARATION

I declare that this thesis paper entitled “**Stabilization of black cotton soil using fly ash of municipal waste byproduct**” is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

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LIST OF ABBREVIATIONS AND SYMBOLS

MT -----	Metric Tone
AASHTO-----	American Association of Highway and Transportation Officials
ASTM -----	American Society for Testing and Materials
FA-----	Fly Ash
BC-----	Black cotton
CBR-----	California bearing ratio
UCS-----	unconfined compressive strength
OMC -----	Optimum moisture content
MDD -----	Maximum dry density
RHA -----	Rice Husk ash
MSWIFA-----	Municipal Solid Waste Incinerated Fly Ash
IS-----	Indian Standard
LL-----	Liquid Limit
PL-----	Plastic Limit
PI-----	Plastic index
USCS-----	Unified Soil Classification System,

ABSTRACT

Black cotton soil, formed by the decomposition of lava rocks, is a global problem affecting civil engineers. Its expansive nature negatively affects its bearing capacity and can cause damage to infrastructure, such as roadways, pavement structures, building foundations, reservoir linings, water lines, irrigation systems, and sewer lines. The main focus of this research paper was to investigate the effect of Dukem Town black cotton soil (expansive soil) and reducing their expansiveness property by using fly ash of Reppie municipal waste byproduct. Reppie municipal waste byproduct (municipal waste fly ash) has been disposed in traditional methods from Addis Ababa city which led to environmental, social, and economic problems. For the purpose of this research, this Fly Ash has been used to reduce the impact of black cotton soils was collected from Reppie-Waste-to-Energy Plant" which is found in Addis Ababa. Therefore, this research was proposed to evaluate the index properties, compaction and Unconfined compressive strength (UCS) of the 0% black cotton soils and their behavior after stabilizing by Fly Ash. The result shows that the natural water content measurements were in the range of 30.76 % to 32.67 %. The gradation result shows that the content of gravel soil ranges from 0 to 0.10%, sandy soils range from 3.50 to 10.30%, silt soil range from 64.24 to 92.327% and clay soil range from 4.07-23.87. The specific gravity of the Dukem town black cotton soil samples was ranges 2.55-2.66. Plasticity index of the soil was ranges 14.14-48.11. Compaction dry density with moisture content ranges BH-1 (1.1895g/cm³, 36%) BH-2(1.4174g/cm³, 30%) BH-3(1.4117g/cm³, 28%). For Add 4% to 20% Fly Ash compaction dry density with moisture content ranges BH-1 (1.333g/cm³, 27% to 1.58g/cm³, 30%) BH-2(1.2887 g/cm³, 28% to 1.58g/cm³, 30%) BH-3(1.2753g/cm³, 27% to 1.58g/cm³, 30%). unconfined strength (0%) BH-1 (154.92Kpa) BH-2(151.48Kpa) BH-3(154.745Kpa) For Add 4% to 20% Fly Ash add unconfined strength BH-1 (156.411Kpa to 211.59Kpa), BH-2(156.411Kpa to 211.23Kpa), BH-3(155.523Kpa to 210.03Kpa) of Dukem Town. Based on this experimental work one can proceed and check that the properties of this black cotton (expansive soils) by further adding the percentage amount of this fly ash of municipal waste by product stabilizer alone and also use additional stabilizers and mix to this municipal waste by product fly ash to stabilize the black cotton soil (expansive soil).

Key words: - Stabilization of black cotton soil, Fly Ash.

CHAPTER ONE

1. INTRODUCTION

1.1 Background

Expansive soils, which are also called as swell-shrink soil, have the tendency to shrink and swell with variation in moisture content. As a result of this variation in the soil, significant distress occurs in the soil, which is subsequently followed by damage to the overlying structures. During periods of greater moisture, like monsoons, these soils imbibe the water, and swell; subsequently, they become soft and their water holding capacity diminishes. As opposed to this, in drier seasons, like summers, these soils lose the moisture held in them due to evaporation, resulting in their becoming harder. Generally found in semi-arid and arid regions of the globe, these type of soils are regarded as potential natural hazard – if not treated, these can cause extensive damage to the structures built upon them, as well causing loss in human life. Soils whose composition includes presence of montmorillonite, in general, display these kinds of properties. Tallied in billions of dollars annually worldwide, these soils have caused extensive damage to civil engineering structures (Mohanty et al, 2015).

Every structure must rest upon soil and the soil at that particular site should be satisfactory for the intended use. But unfortunately now a days finding such satisfactory soils is of rare occurrence. Generally the soils, which are unsatisfactory, and are not suitable for the intended uses are mostly expansive clayey soils. Expansive soils are swelling soils and are most problematic because of their drastic changes in volume with addition or removal of water. Hence stabilization of these soils is very essential. It is highly difficult to work on such soils. Since construction on these soils cannot be avoided, we have to identify and modify them to be suitable for our construction works. 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. Soils may be stabilized to increase strength and durability or to prevent erosion and dust generation. Regardless of the purpose for stabilization, the desired result is the creation of a soil material or soil system that will remain in place under the design use conditions for the design life of the project (M. Siva Parvathi1 et al, 2019).

Soil is a mixture of various sizes of particles like gravel, sand, silt and clay. Gravel and sand are among the coarse fractions and they are considered inert materials because of their significant surface activity. In contrast, clay and silt clays are particles of ultra-fine size in the form of platelets. They carry an unbalanced negative electric charge on their surface. This electric charge and large specific surface they possess render them highly active. They can absorb water as well as the positively-charged ions from the salts in water to neutralize the electric charge they carry on their surface. The amount of water absorbed depends on the type of the clay mineral present in the soil. Three most common minerals present in clay are Kaolionite, Illite and Montmorillonite and their capacity to adsorb water increases in that order, therefore, the greater the percentage of Montmorillonite mineral present, the greater would be the expansive nature of the soil. (Amsalu, et al, 2021)

1.2 Statement of the problem

Dukem town is the most affected area with black cotton soil that even very simple construction work is not simply done without shear wall structure due to very Swell and Expansiveness of this soil type, this is why this research is needed and focused on Dukem town

1.3 Research Questions

- What are the effects of fly ash on expansive (black cotton) soil?
- What are the ambit of reducing expansiveness and improving bearing capacity value by adding fly ash?
- How to establish the usage of Fly Ash of municipal waste by product as an additive, thereby helping utilize it which otherwise always lays as fine waste product from thermal power plants?

1.4 Objectives

1.4.1 General Objective

The main objective of this research work was to investigate the effect of fly ash on the bearing capacity of expansive (black cotton) soil.

1.4.2 Specific Objectives

- To determine effects of fly ash on expansive (black cotton) soil
- To determine the ambit of reducing expansiveness and improving bearing capacity value by adding fly ash.

- To establish the usage of Fly Ash of municipal waste by product as an additive, thereby helping utilize it which otherwise always lays as fine waste product from thermal power plants

1.5 Significance of the study

The importance of this study is stabilizing the black cotton soil using **fly ash of municipal waste byproduct** material which gate after the electric power plant generates the electric energy. The disposal of this ash in the form of particulate matter is already causing series environmental problem to the city. This byproduct is deposited to the land fill site around Addis Ababa. The area of land fill side (i.e. where this byproduct dumped) is very crowded with this byproduct (fly ash) and the city municipality is also facing different problems this day.

Therefore; after this research is carry out, it will be expect that the problems of soils expansiveness, problems of city land fill site will be solved and also it will generate income for the city as well by using this residue material as stabilizer.

1.6 Delimitations and limitations of the study

Even though, the findings of the research were limited to one soil sample considered in this research which is black cotton soil, the study was supported by different types of literatures and a series of laboratory experiments. The results were also specifying to the type of additives used and test procedures that were carried out adopted in the experimental work.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

This paper was investigating the feasibility of using fly ash as an alternative source for the stabilization of soil. It was supported with the related research papers. Although earlier studies have been conducted on using these waste in soil. But in those studies municipal waste by-product materials have not been used to stabilize soils yet. Some of the related studies are discussed under this chapter:

2.1.1 Origin of expansive soil

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

2.2 Nature of expansive soil

Swelling in clays can be sub-categorized into two distinctive types, namely:

- Elastic rebound in the compressed soil mass due to reduction in compressive force.
- Imbibing of water resulting in expansion of water-sensitive clays.

Swelling clays are the clays that exhibit latter type of swelling, where the clay minerals with largely inflating lattice are present. One of the fundamental characteristics of clayey soil is that they display little cohesion and strength when wet, but they become hard when devoid of water. However, all of them do not swell due to wetting action. Decrease in ultimate bearing capacity at saturation, and large differential settlement due to this occurs. Thus, clayey soils exhibit foundation problems.

2.3 Methods of recognizing expansive soils

Grouped into three categories, following are the methods of recognizing expansive soils:

- ❖ Mineralogical identification

- ❖ Indirect methods, such as soil suction, activity and index properties
- ❖ Direct measurement.

Impractical and uneconomical in practice, methods of mineralogical identification still hold importance in exploring basic properties of clay minerals. Direct measurement, out of the remaining two categories, offers the most useful data. By their shattered or fissured condition, or obvious structural damage to existing buildings caused by such soils, potentially expansive soils are usually identified in the field. To classify expansive soil, potential swell, or potential expansion, or the degree of expansion is a favoured term used; from this, geotechnical engineers establish how good or bad the expansive soils are.

2.3.1 Clay Mineralogy

On the basis of their crystalline arrangement, clay minerals can be categorized into three general groups, namely: Kaolinite group, Illite group and Montmorillonite group.

Clay minerals are complex aluminum silicates composed of two basic units these are silica tetrahedron and alumina octahedron. Each tetrahedron unit consists of four oxygen atoms surrounding a silicon atom. Clay minerals are formed by sandwiching tetrahedral and octahedral layer sand sheets together. A tetrahedron sheet can be considered as a layer of silicon ions between a layer of oxygen and a layer of hydroxyl ions (Das and Roy, 2014). This also is called a gibbsite sheet sometimes magnesium replaces the aluminum atoms in the octahedral units; in this case, the octahedral sheet is called a Brucite sheet. Absorption of water by clays leads to expansion. From the mineralogical standpoint, the magnitude of expansion depends on the kind and amount of clay minerals present, their exchangeable ions, electrolyte content of the aqueous phase, and the internal structure (Chen, 1975).

In order of increasing their expansiveness, they are listed as Kaolinite, Illite, and Montmorillonite.

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

Montmorillonite: - Montmorillonite is the clay mineral that presents most of the expansive soil problems. Its basic structure consists of an alumina sheet sandwiched between two silica sheets. The basic Montmorillonite units are stacked with one on top of the other but the bond

between individual units is relatively weak so water can easily penetrate between the sheets and separation of them produces swelling. It is extremely active and its activity decreases as the adsorbed cation exchanges in the order of Sodium, Lithium, Potassium, Calcium, Magnesium, and Hydroxyl (Mesfin, 1999).

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

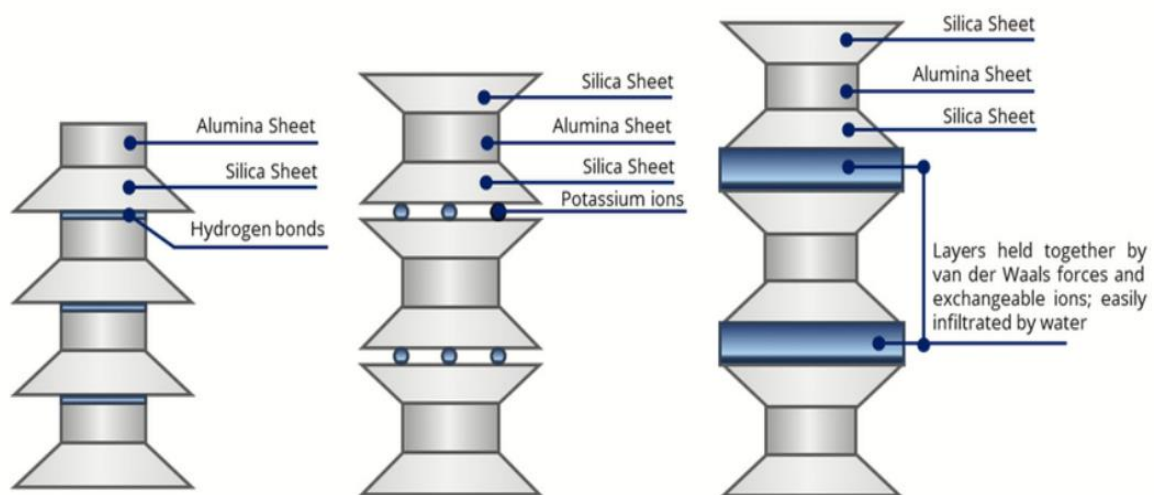


Figure 2. 1 Main group of clay minerals.(a) kaolinite(1:1 non-expanding);(b)Illite (2:1 non-expanding);(c)montmorillonite(2:1 expanding) **(Al-Atroush, 2021)**

i. Kaolinite group

A clay mineral which has a chemical composition $Al_2Si_2O_5(OH)_4$ is called Kaolinite. This type of clay mineral has a layered silicate, with linkage to one octahedral sheet of alumina through oxygen atoms. Kaolinite is the purest of clays, meaning that it varies little in composition. It also does not absorb water and does not expand when it comes in contact with water. Thus, kaolinite is the preferred type of clay for the ceramic industry.

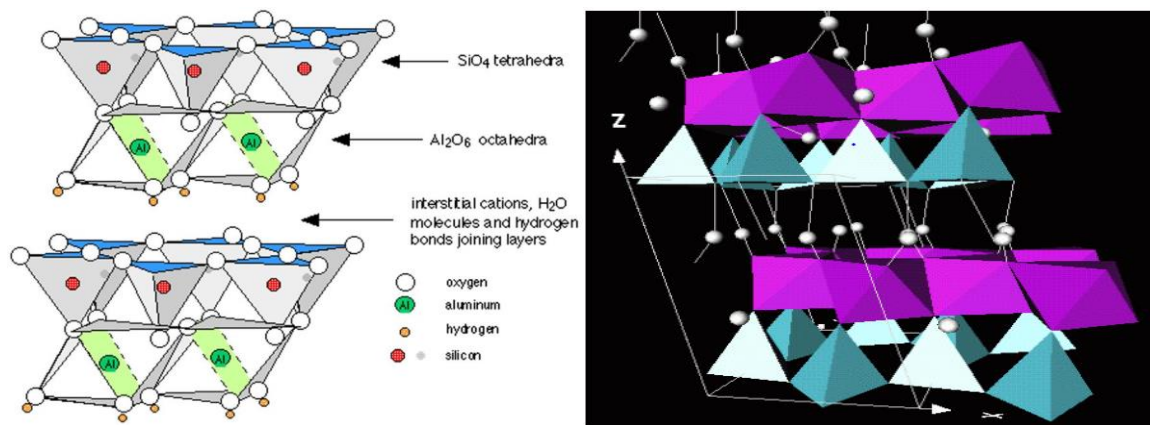


Figure 2. 2 Atomic structure of kaolinite

ii. Illite group

Illite is essentially a group name for non-expanding, clay-sized, dioctahedral, micaceous minerals. It is structurally similar to muscovite in that its basic unit is a layer composed of two inward-pointing silica tetragonal sheets with a central octahedral sheet.

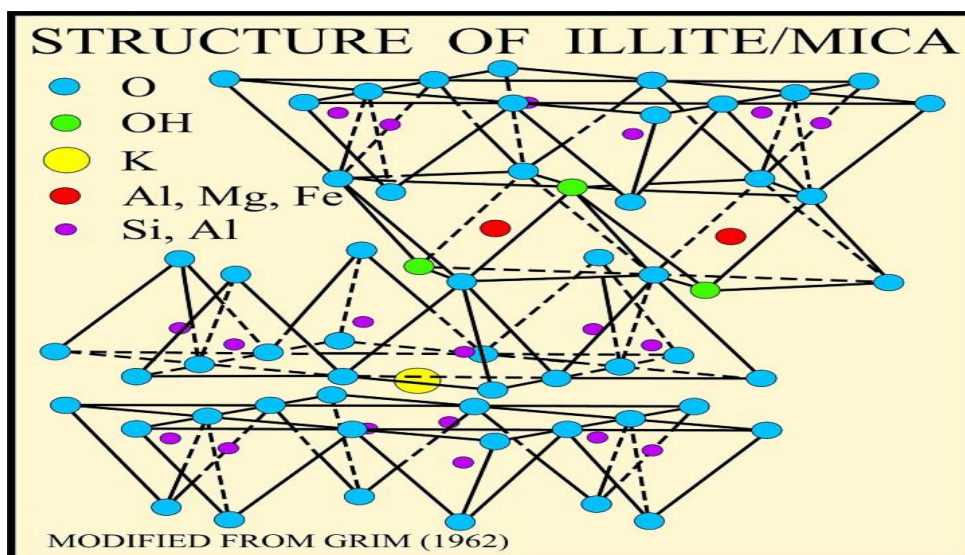


Figure 2. 3 Atomic structure of Illite

iii. Montmorillonite group

The structural arrangement of the montmorillonite mineral consists of two tetrahedral silicon dioxide layer units with a central aluminum oxide octahedral layer. Water can penetrate between the panels, causing the panels to expand significantly and the structure to be divided into units. Soils containing a significant proportion of montmorillonite minerals exhibit high swelling and contraction behaviour (V.N.S. MURTHY 2002). (Informa, 2017)

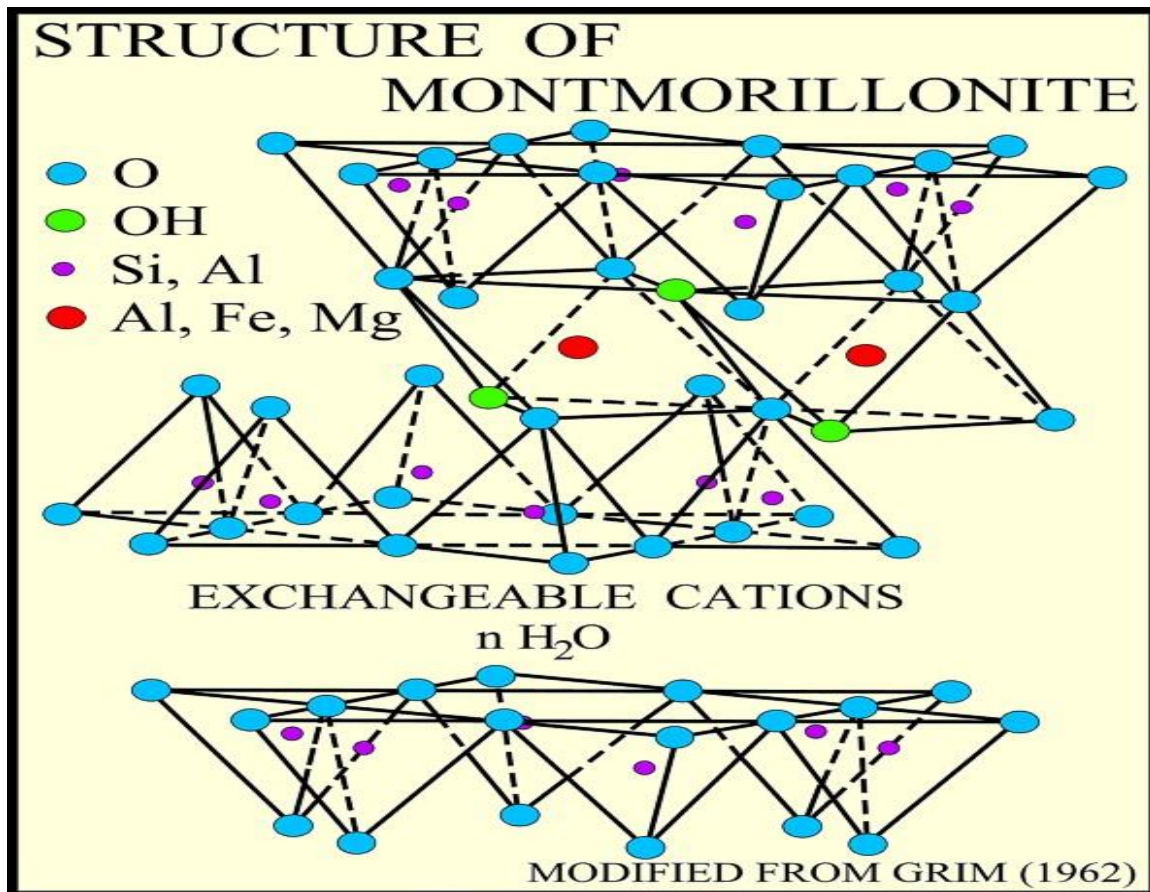


Figure 2. 4 Atomic structure of Montmorillonite

Identification and classification of expansive soils

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

expansive and the percentage of montmorillonite present in the black and gray soils ranges from 45 to 87 percent (Solomon, 1986).



Figure 2. 5 Typical crack on expansive soil during the dry season (a) Diagonal cracks that develop below windows (b) (**Kassahun, 2006**)

Expansive soil can be identified by conducting several laboratory tests. These can be categorized as mineralogical identification and direct and indirect methods. Mineralogical identification can be useful in the evaluation of the material but is not sufficient in it when dealing with natural soils. The various method of mineralogical identification is important in a research laboratory in exploring the basic properties of clays, but it is impractical and uneconomical for practicing engineers. The five techniques which may be used are for mineralogical identification are differential thermal analysis-ray diffraction, dye adsorption. Chemical analysis and Electron microscope resolution. Due to their property of time consumption, expensiveness, and need for skilled technicians for result interpretation, these tests cannot be considered routine tests (**Chen, 1988**)

2.3.2 Indirect Methods for the identification of expansive soil

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

A. Atterberg limit tests

The Atterberg limit defines the moisture content boundaries between states of consistency of fine-grained soils. The plasticity index and liquid limit are used to characterize soils and useful indices for the identification of swelling of expansive soils. The plasticity index can be used as a preliminary indication of the swelling characteristics of most clay. Expansive soils are characterized by a $PI > 30\%$, a liquid limit $> 50\%$, and a high swelling potential

(Chen, 1975). The relation between the degree of expansion and the plasticity index is given in Table 2.1.

Table 2. 1 Relation between degree of expansion and Atterberg limits (**Mesfin, 1999**)

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

Daksanamurthy and Raman (1973) also presented a single index method for identifying expansive soils using only liquid limits. They suggested four classes of clays according to their liquid limits as shown in Table 2.2. USBR Method is developed by Holtz and Gibbs; it is based on a direct correlation of observed volume change (degree of expansion) with colloid content, plastic index, and shrinkage limit. The classification is as given in Table 2.3.

Table 2. 2 Relation between the swelling potential of clays and the liquid limit
Daksanamurthy and Raman (1973)

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

Table 2. 3 Classification based on bureau of reclamation method (Chen, 1988)

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 hig

B. Linear shrinkage

Linear shrinkage is the change in length of a soil sample as it dries to the shrinkage limit, SL, expressed as a percentage of the original length. This test also in combination with other tests is used for the classification of expansive soil (Mesfin, 1999). The swell potential is presumed to be related to the opposite property of linear shrinkage measured in a very simple

test. In theory, it appears that the shrinkage characteristics of the clay should be a consistent and reliable index to the swelling potential (Chen, 1975)

Table 2. 4 Potential expansiveness for various values of shrinkage limits and linear shrinkage (Altmeyer, 1955)

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

C. Free swell

This test is a simple test conducted in the laboratory for the determination of the degree of expansiveness. Free swell tests consist of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. The difference between the final and initial volume, expressed as a percentage of the initial volume, is the free swell value. The swell test is very crude and was used in the early days when refined testing methods were not available. Experiments conducted by (Holtz & Gibbs, 1956) indicated that a good grade of high swelling commercial bentonite will have a free swell.

Some laboratory tests are available for the identification purposes of swelling soils. By differential thermal analysis, Microscopic examination, and X-ray diffraction. The presence of Montmorillonite in clay minerals allows the judgment of the expansiveness of the soil. This aspect is however very technical in nature. A simple aspect, as opposed to the aforementioned methods, is the free-swell test, that's done in the laboratory. This test is conducted by adding 10 gm of dry soil, passing through a 425 μ sieve into two separate 100 cc graduated jar – one filled with water, and the other with kerosene. Swelling occurs in the jar containing water. The swelled volume of the soil is then noted (after 24 hours period), and subsequently, the free swell index values, in percentage, are calculated. IS: 2720-II was followed for free swell index test.

$$\text{Free swell value } [I_n] \text{ (in \%age)} = \frac{(\text{Final volume} - \text{initial volume})}{\text{initial volume}}$$

Good grade, high swelling, commercial Bentonite has been reported to have free swell values varying from 1200% to 2000%. In general, the swelling potential of a soil is related to plasticity index. With corresponding range of plasticity index, various degrees of swelling capacities are as indicated through the following table:

Table 2. 5 Swelling potential vs. Plasticity Index

Swelling potential	Plasticity Index
Low	0-15
Medium	15-24
High	24-46
Very High	>46

D. Causes of swelling

There are different theories, but the mechanism of swelling is still unclear. No conclusion to the mechanism has been reached. Soil consisting high percentage of clay or colloid, with Montmorillonite mineral present as the chief mineral is one of the most universally accepted reasons for the swelling of soils.

E. Swell Pressures

The pressure exerted by expansive soil when they swell, owing to their contact with water, is called swell pressure. The estimation of this swell pressure and likely becomes a very important task for designing a structure on such soils, or building the core of a dam, or constructing a road embankment, or taking a canal through such soils.

F. Factors affecting swelling

Initial moisture content or the molding water in case of a re-molded sample is the most influencing factor. “The behaviour of re-molded clays is much as undisturbed clays”, as per Holts’ and Gibbs’ findings. For a given dry density, the value of initial water content will be a key factor in determining the water affinity of a given sample, as well as its swell pressure. Minimum moisture content (w_n) required by clay for swelling to begin beneath a pre-paved sub-grade is given by:

$$w_n = 0.2w_1 + g$$

where, w_1 = liquid limit

The factors that affect the swelling aspect of a soil largely depend on the soils environmental conditions. With the intake of water, swelling is more in a soil element which is close to the surface, but if below the surface, the same soil exhibit negligible swelling because the overburden pressure neutralizes the developing swelling pressure of the dry soil.

Generally responsible for swelling are the following factors:

- ❖ Location of the soil sample from the ground surface
- ❖ Thickness, as well as shape of the sample
- ❖ Change in volume
- ❖ Temperature
- ❖ Nature of pore fluid
- ❖ Time
- ❖ Stress history
- ❖ Unit weight of the sample taken, etc.

2.3.3 Direct Methods

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

Figure 2. 6 Relationship between swelling potential and % of expansion in oedometer

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

2.4 Classification of expansive Soils

Most of the soil classification systems that have been developed for engineering purposes are based on simple index properties such as particle-size distribution and plasticity. Although several classification systems are now in use, none is totally definitive of any soil for all possible applications because of the wide diversity of soil properties. In general, there

are two major categories into which the classification systems developed in the past can be grouped.

1. The textural classification is based on the particle-size distribution of the percent of sand, silt, and clay-size fractions present in a given soil.
2. The other major category is based on the engineering behavior of soil and takes into consideration the particle-size distribution and the plasticity (i.e., liquid limit and plasticity index). Under this category, there are two major classification systems in extensive use now:
 - a. The AASHTO classification system, and
 - b. The Unified classification system.

2.4.1 The textural classification

By making use of the grain size limits for sand, silt, and clay, a triangular classification chart has been developed as shown in Figure 2.3 for classifying mixed soils. This method of classification does not reveal any properties of the soil other than grain-size distribution. Because of its simplicity, it is widely used by workers in the field of agriculture. The gradation of the black cotton soils varies considerably. There is little or no gravel size in most black cotton soils and those that have shown percentages of less than 8 %. The amount of sand varies between 2 % and 50 % and silt from 11 % to 58 %. The clay size contents also range from 40 % to 75 %. The soils classify as clay, sandy clay, and silty-clay in the U.S.

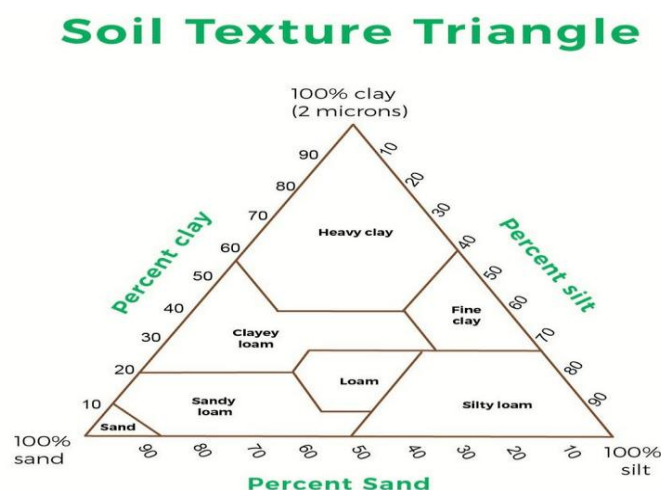


Figure 2. 7 U.S department of agriculture textural classification (Murthy, 2002)

2.4.2 Classification based on the engineering behavior of soil

a. The AASHTO classification system

This system is known as the AASHTO (American Association of State Highway and Transportation Officials) System. The revised system comprises seven groups of inorganic soils, A-1 to A-7 with 12 subgroups in all. The system is based on Particle-size distribution, Liquid Limit, and Plasticity Index. To evaluate the quality of soil as a highway subgrade material group index (GI) is an important parameter and it can be given by the equation below.

$GI = (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 35)(PI - 10)$ (2,3) Where F_{200} is percentages passing through sieve No.200 and LL & PI are liquid limit and plasticity index respectively.

Table 2. 6 Chart for use in the AASHTO soil classification system (Murthy, 2002)

General classification	Granular materials (35 percent or less of total sample passing No,200						Silt –clay materials (more than 35 percent of total samples passing No,200)				
Group classification	A-1		A3	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	50max	51 min								
No.200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	35 max	35 max	35 max	35 max
Characteristics of fraction passing No.40											
liquid limit											
plastic index											
	6 max		N,P	40max 40 min 10 max 10max		40 max 41 min 11 min 11 max		40 max 41 min 10 max 10 max		40 max 41 min 11 min 11 min	
Usual types of significant constituent	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							

b. Unified soil classification system

The USCS is the most widely used system for determining the suitability of soils for general use based on particle size distribution and plasticity. According to this system, the soils are classified into three major groups; coarse-grained, fine-grained, and highly organic (peat). The soils are grouped as coarse-grained if more than 50% of the soil is retained on the No.200 (0.075mm) sieve and fine-grained if more than 50% of the soil passes the No.200 sieve.

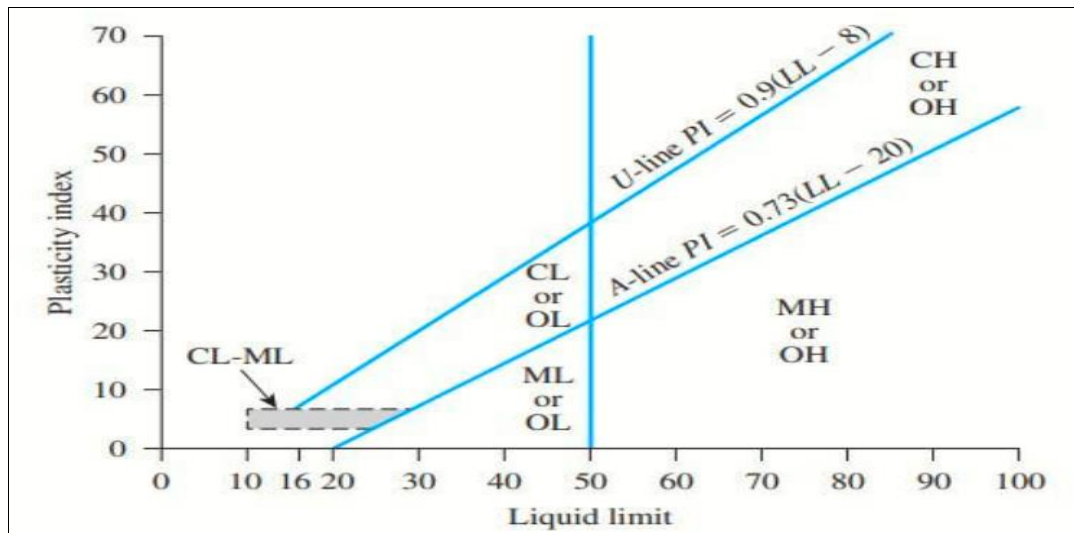


Figure 2. 8 Plasticity chart for fine-grained soils (Murthy, 2002)

2.5 Problems associated with the expansive soil

Generation of problems for all kinds of construction over expansive soils is common, leading us to believe that such types of soil are not suitable for these purposes. However, given the placement of these kinds of soil over the country, it leaves engineers no other choice but to develop different structures on the soil, well aware of the risk. These structures chiefly are a part of irrigation projects. Buildings, and other kinds of structures constructed over these soils are subjected to differential deflections. These deflections cause distressing, and in turn leads to damage of the structure.

Moreover, the reduction in moisture content due to the evaporation of water in soil causes shrinkage, and heaving of soil occurs when there is a disproportionate increase in moisture content. The level of ground water table also has a significant impact on the moisture content of these soils, which in return affect the shrinkage-swelling cycles. In seasons which are dry in nature, the surface of clayey soil shrinks, however, little evaporation is there on the clayey soil on which the building stands. This causes differential settlement at plinth level, posing danger to the structure.

If the construction of a building on such type of soil is done in its dry season, the base of the structure's foundation would experience swelling pressures when the partially saturated soil underneath starts imbibing water in the wet season, developing swelling pressures. When the pressure imposed by the structure on the foundation is less than the swelling pressure developed, upliftment of such a structure occurs, which would lead to formation of cracks.

The 22 imposed bearing pressures if the building is constructed in the wet season should be within the permissible limits of bearing pressure for the soil. A better practice is to construct a building during dry season, and completing it before the onset of wet seasons.

One of the methods of treatment of expansive soil to make them fit for the construction purposes is called stabilization. According to Petry (2002), assortment of stabilizers can be grouped into:

- ❖ By-product stabilizers (Quarry dust, Fly ash, Slag, Phosphor-gypsum, etc.)
- ❖ Traditional stabilizers (Cement, Lime, etc.)
- ❖ Non-traditional stabilizers (Sulfonated oils, Potassium compounds, Polymer, Enzymes, etc.)

Lots of geo-environmental problems are a result of industrial by-products whose disposal as fills in disposal sites adjacent to the industries demand large chunks of land, which can otherwise be utilized for construction, growing of vegetation, etc. purposes. Various attempts by different researchers and organizations have been made to utilize these by-products.

Stabilization of expansive soil is one of the ways of fulfilling such a thing.

2.6 Stabilization using fly ash

Studied the effect of Bagasse ash as a Sub-grade Soil Stabilizer Material and the following conclusions have been drawn from the results of the study/investigation carried out within the scope of the study.

The plasticity index slightly reduced with increased in bagasse ash content and curing has also an insignificant effect on the plasticity of the expansive soil. The optimum moisture content increased while the maximum dry density values decreased with increment of bagasse ash content. Free swell, free swell index and free swell ratio of the stabilized samples decreased with increasing bagasse ash content. CBR values slightly increased with the addition of bagasse ash. The change in CBR value is not significant for both cured and uncured samples. Addition of bagasse ash alone does not improve the strength of soils due to presence of only reactive silica with low amount of calcium content in bagasse ash. The plasticity index significantly decreased with addition of bagasse ash combined with lime and increased curing period. However, the addition of bagasse ash alone has a minor effect on the plasticity index of expansive soil. The addition of lime and bagasse ash together led to a more decrease of the maximum dry density and increase in optimum moisture content

compared to the addition of lime and bagasse ash separately. The addition of bagasse ash in combination with lime improved the CBR value. The improvement is more significant when the sample is cured. Hence, combination of bagasse ash and lime can strongly improve the strength of the expansive soil.

Unlike lime in combination with bagasse ash the improvement achieved by bagasse ash alone on the poor geotechnical properties of expansive soil was limited because lower amount of calcium in the bagasse ash. Hence, improvements achieved with up to 30% bagasse ash content were not satisfactory. However, the rate of swelling and heave decreased with increasing bagasse ash content of stabilized expansive soil. In this investigation bagasse ash stabilized expansive soil does not bring significant change for use it as a sub-grade material. Therefore, bagasse ash is not an effective standalone stabilizer for highly plastic expansive soils. However, bagasse ash plus/in combination with lime can effectively stabilize this soil. The expansive soil stabilized with bagasse ash plus/in combination with lime can be used as a good sub grade material. So, combining two local materials (bagasse ash and lime) can effectively improve the poor geotechnical properties of this soil and help in increasing land resources availability for construction projects and reduce the amount of lime needed for the stabilization purpose. (Wubshet, 2013)

A study was undertaken to investigate the effects of Quarry Dust on some geotechnical characteristics of expansive soil. The following conclusions can be drawn from the results of the study.

Plasticity index showed a slight decrease with increasing amount of quarry dust. Free swell and swelling pressure also showed decrease with increasing amount of quarry dust. Swelling pressure showed reduction to the level in which the uplift pressure of expansive soil can be controlled by the counter pressure of light weight structures. UCS of the treated soil decreased with increasing amount of quarry dust. This revealed that, both quarry dust samples (i.e. Basaltic and Trachyte samples from the two quarry sites) do not exhibit any pozzolanic property. OMC and CBR-swell values decreased while MDD and CBR of the treated soil showed a very slight increase for all increased concentrations of quarry dust. Addition of quarry dust did not bring significant improvement in strength characteristics of the soil. This is due to the presence of reactive silica in greater amount with low amount of calcium content (for the formation of elements responsible for strength gain) in quarry dust.

In almost all test results a slight difference has been observed between soils treated with Sululta quarry dust (Basaltic quarry dust) and Legetafo quarry dust (Trachyte quarry dust). Soil treated using basaltic quarry dust has showed relatively better improvement than the soil treated with Legetafo quarry dust. This may be due to the greater amount of alumina in the oxide composition of Sululta quarry waste; which replaces the water sheet sandwiched between two Structural units of Montmorillonite.

Based on the results discussed above, quarry dust is not a very effective chemical stabilizer for improving the geotechnical properties of highly plastic expansive soils; although some improvement is observed. However, it can be utilized for building construction purposes; i.e. to construct light weight building structures, taking care of the determination of the load of the structure to be built on a specific soil.

For the other perspective of the study i.e. tests have been performed to check the mechanical stabilizing ability of the quarry waste by using granular quarry waste and based on the test results the following conclusions have been made. The optimum moisture content decreased while the maximum dry density values increased with percentage increment of granular quarry waste. The CBR-swell and CBR values showed a slight decrement and increment respectively.

The treated soil cannot satisfy the minimum requirement to be used as a subgrade material. In the meantime mechanically treated soil samples in this case have not showed any significant improvement in properties of the soil than the already observed improvement in the case of chemical stabilization. (Asres, 2017)

A study was undertaken to investigate the effects of using different Stabilizers on some geotechnical characteristics of expansive soil. The following conclusions can be drawn from the results of the stud.

Specific gravity increased from 1.98 to 2.34. It is observed that by the addition of 5% lime and 30% of demolished concrete Liquid limit decreased from 55.7% to 43.33%. And Plastic limit decreased from 26.11% to 18.56%. It is seen that by the addition 5% lime and bagasse ash (30%) liquid limit decreased to 47.98 % and plastic limit decreased to 20.16%

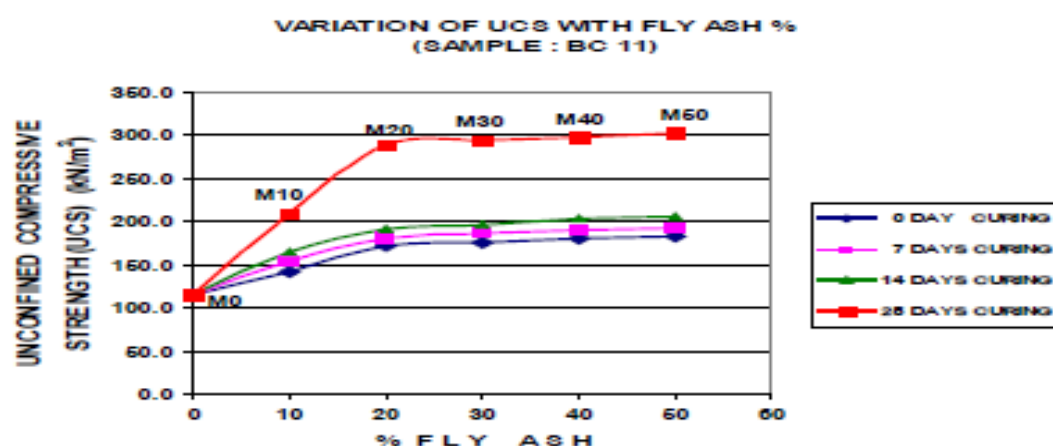
An addition of 5% lime and 30% demolished waste MDD increased from 1.429g/cc to 1.776g/cc. An Addition of 5% lime and 30% demolished waste OMC decreased from 27.2% to 18.25%. Unconfined compressive strength increased from 1.618 kg/cm² to 1.9422 kg/cm². CBR value increased from 1.28 % to 12.34% for soaked condition. And for unsoaked condition CBR value increased from 2.38% to 16.23%

From the above study it is shown that the Bagasse ash waste is used to improving geotechnical properties like CBR value of the soil while demolished waste concrete is more effective than bagasse ash in the improvement of index properties (liquid limit, plastic limit of the black cotton soil). (H S Jayatheertha1, 2019)

It was concluded that there is a considerable improvement in California Bearing Ratio (CBR) of black cotton soil with addition of 14% of rubber chips as soil stabilization when compared to only black cotton soil. Based on the result from standard proctor test it can be stated that with increase in percentage of rubber chips the compaction parameters also increased. Up to the range of 14% the CBR value gets increased and at 16% it shows a decrease in the value. Observing its economic cost and quality of stabilization improvement, it is clear that this type of stabilization may be applicable in stabilization of black cotton soil in construction of road or in shoulder portion of highways. (Sadhasivam M 1, 2020)

Concluded that: Stabilization of Black Cotton soil can be effectively achieved by using Fly ash (FA) and Rice Husk ash (RHA). Initially Black cotton soil having 21.5% of Plasticity Index and 1480Kg/m³ of Max dry density and 0.27(N/mm²) of unconfined compressive strength (UCS). By addition of different % of Fly Ash plasticity index is reduced to 19% and MDD increased to 1565(Kg/m³) at 15% of Fly Ash. And UCS value increased to 0.37(N/mm²) at 25% of Fly Ash. By addition of different percentages of RHA plasticity index is reduced to 19.5% and MDD increased to 1542(Kg/m³) and UCS value increased to 0.315(N/mm²) at 6% RHA. (NarasimhaRao .Battina1, 2020)

Based on the results of the investigation, following conclusions are drawn. Dandeli fly ash is used as a stabilizer for improving the geotechnical characteristics of Hubballi-Dharwad



black cotton soils. Addition of Dandeli fly ash significantly improves the index properties, compaction and strength characteristics of black cotton soils under study and the effects of fly ash treatment vary depending upon the quantity of fly ash that is mixed with the study black cotton soil samples.

Figure 2. 9 Variation of UCS with DFA percentages for different curing periods BC 11

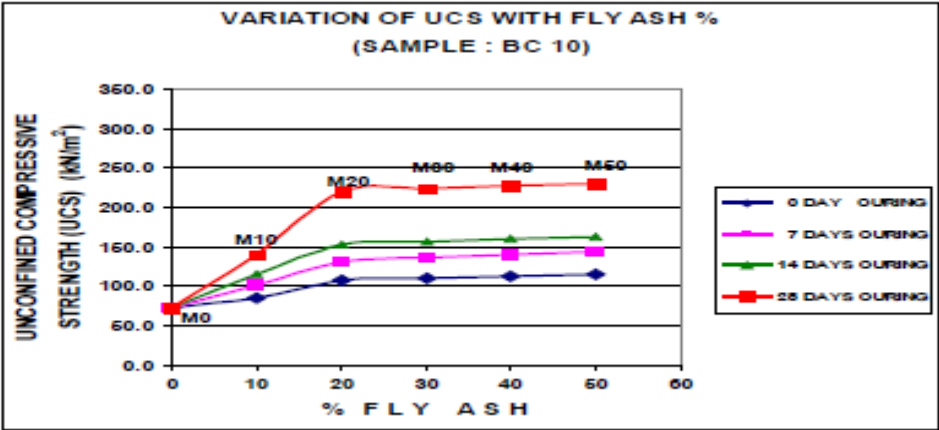


Figure 2. 10 Variation of UCS with DFA percentages

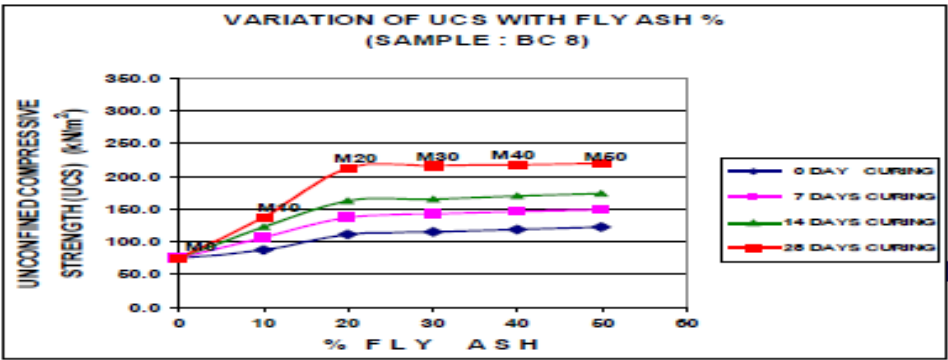


Figure 2. 11 Variation of UCS with DFA percentages for different curing periods BC 10

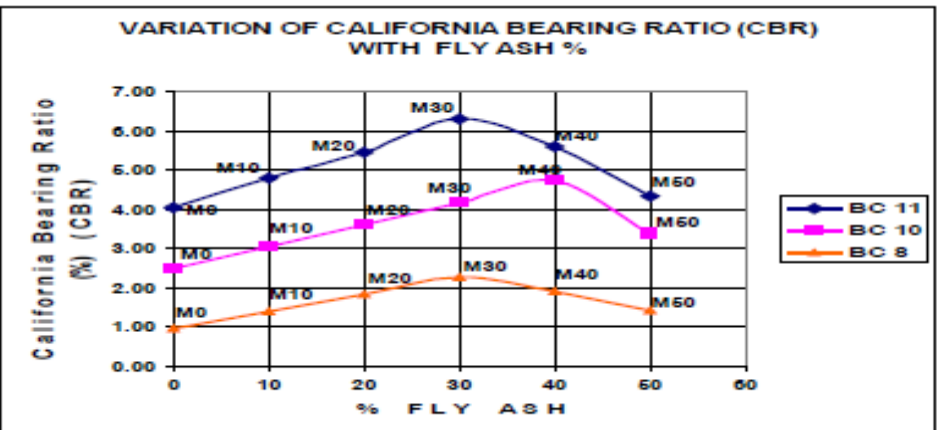


Figure2. 1 Variation of CBR with DFA percentage

The liquid limit and plastic limit of the soils decrease with the addition of Dandeli fly ash which indicates a desirable change as the soil + fly ash mix can gain shear strength at an early stage than the virgin soil with the change in the water content. The relative decrease in the plasticity index of the soils is another favorable change since it increases the workability of these soils. The shrinkage limit of the soils increases with the addition of Dandeli fly ash, which facilitates in checking the volume change behavior of the soils over a large variation in the moisture content as the season changes.

Addition of Dandeli fly ash brings in an improvement in the compaction parameters of the study soils, by increasing the maximum dry density of soils with decrease in the corresponding values of optimum moisture content.

The unconfined compressive strength of these soils increases upon the addition of Dandeli fly ash. The trend of improvement in the unconfined compressive strength is observed to be more pronounced with the curing of the soil + fly ash mix. A curing period of 28 days is observed to yield the maximum enhancement in the unconfined compressive strength.

The California bearing ratio of the study soils increase gradually with the addition of Dandeli fly ash up to a certain percentage of Dandeli fly ash, beyond which, further increase in Dandeli fly ash percentage is observed to cause a decreasing trend in the California bearing ratio values. The improvement in the California bearing ratio value of the black cotton soil upon the addition of Dandeli fly ash suggests that, it can be effectively used in bulk as sub-base material in combination with the study soils, for the road construction works.

The study of variations of different parameters viz. liquid limit, plastic limit, plasticity index, shrinkage limit, maximum dry density, optimum moisture content, unconfined compressive strength and California bearing ratio with the addition of Dandeli fly ash suggest that, for each parameter of the study soil samples, there exists an optimum Dandeli fly ash percentage for mixing with the soil under consideration; at which the respective parameter attains its most desirable value from geotechnical point of view. The geotechnical properties of Hubballi-Dharwad black cotton soils can be favorably changed using the Dandeli fly ash and an optimum quantity between 30-40% can yield the best possible results. (UdayashankarD.Hakari, 2012)

2.7 Research gaps

From the reviewed literature, the following conclusion and recommendation were made. Expansive soils have been reported all over the world and considerable areas of Ethiopia are also covered by these expansive soils. The engineering and physical properties of expansive soil can be improved by the addition of different additive like, bagasse ash, different fly ash's, lime stone, industrial waste materials and fiber reinforcement.

Even though; earlier studies have been conducted on using different additives in soil stabilization, they have not been used **fly ash of municipal waste by-product** materials as soils stabilizer yet.

Therefore, it is necessary to conduct an experimental study to investigate the effects of **fly ash of municipal waste by-product** on the strength properties of expansive soils to mitigate the problems that come from expansive soil and to recommend this material as an alternative source for soil stabilization.

CHAPTER THREE

3 MATERIALS AND METHODS

3.1 Introduction

In this chapter, the materials used and the methods adopted for the research are described with respect to their source.

To address the objectives of this study, data were collected from different organizations; Black cotton Soil sample from Dukem town and Fly ash was collected from Reppie waste to energy incinerator plant.

Additional data to implement the research were collected from secondary sources. To collect the primary data; samples like MSWIFA, was collected physically and brought to the laboratory then prepared for specific purpose of the research. The secondary data like chemical composition of Fly ash material, different standard determination (i.e. standards for workability) of fresh fly ash were collected from different literature reviews.

3.2 Description of the Study Area

The study area was Dukem town located along with the Addis Ababa -Adama express way 37 km south east of Addis Ababa along with Adama, Dire Dawa - Djibouti transport axis and its geographical location is $8^{\circ}47'40''$ north, $38^{\circ}54'1''$ east and its elevation is 1959 meter. The town is planned to be one of the largest industrial areas in Ethiopia. In the town, different huge factories are found;like, Elsewedy cable factory, East Africa Bottling, , marble factory, etc. have been constructed and many more are planned in this town.

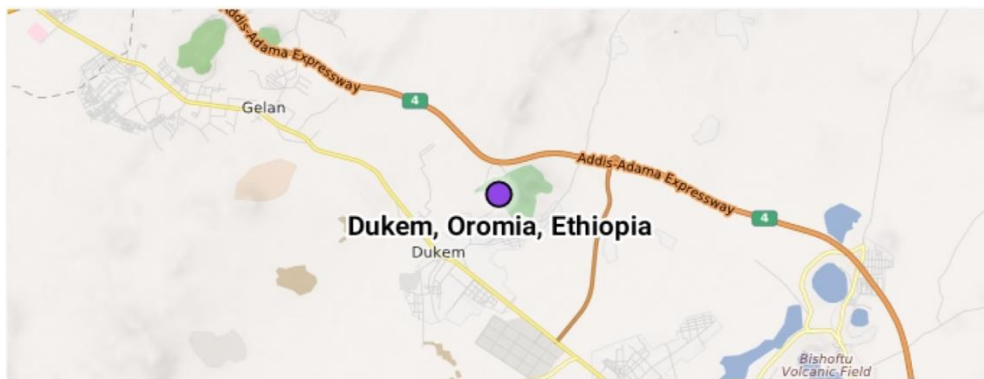


Figure 3. 1 Map shows study area

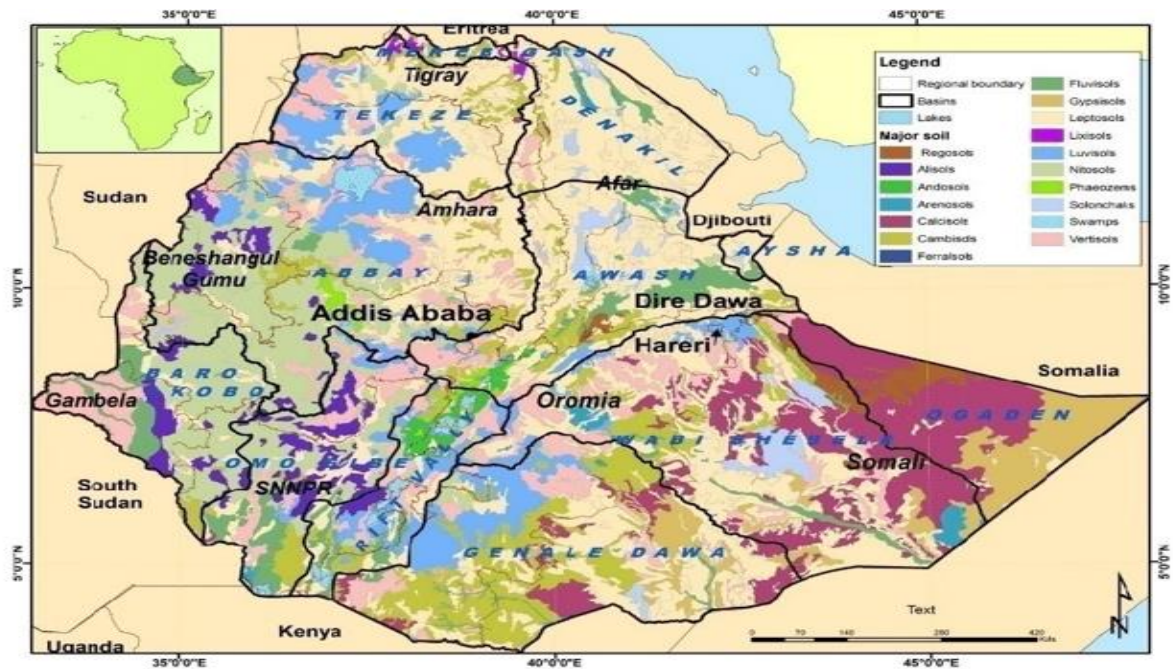


Figure 3. 2 Photograph from Google earth shows Major soil type in Ethiopia.

3.3 Materials

3.3.1 Expansive Soil (Black cotton soil)

As a part of this investigation, the expansive (black cotton soil) was acquired from the site, Dukem town, Oromia, Ethiopia. A disturbed sample was collected from test pit at a depth below 1.5m in order to avoid the inclusion of organic matter as shown in the following picture.



Figure 3. 3 Photograph taken during soil sample collecting

The black cotton soil thus obtained was carried to the laboratory in sacks. Besides this, small amount of soil sample was packed with plastic materials and taken to laboratory as well in order to check the natural moisture content of the soils. And then, a small amount of soil was taken, sieved through 4.75 mm sieve, weighed, and air-dried before weighing again to

determine the natural moisture content of the same soil sample. The various geotechnical properties of the procured soil are as follows:



Figure 3. 4 Photograph taken during laboratory testing

Sample was collected twice for full silicate analysis and other associated physical tests to the Geological Survey Center in Ethiopia. After the silicate test, the second sample was again collected from Reppie and taken to the Adama Science and Technology University Laboratory to verify basic parameters of blended fly ash quality.

The chemical composition of MSWI fly ash or Silica analysis (Si, Al, Fe, Mg, Ca, K, Na, and S) was characterized at laboratory of Geological Survey of Ethiopia using Atomic Absorption Spectrometer and UV- visible Spectrophotometer as per BGS standard (BGS,1993). The Milling, specific gravity and sieving of MSWI fly ash were also carried out in the mechanical unit operation laboratory of Geological Survey of Ethiopia as well. Whereas all other necessarily experimental civil works such as compressive strength, tensile strength, absorption capacity, specific gravity and etc. was conducted in material test laboratory of College of Civil and Architectural Engineering, Adama Science and Technology University .

3.3.2 Fly ash

In worldwide concept, a waste material extracted from the gases emanating from municipal waste fired furnaces, generally of a thermal power plant, is called fly ash. It is the fly ash that the mineral residue left behind after generating the electric power which accumulated in the cello.

Size and Shape.

Fly ash is typically finer than Portland cement and lime. Fly ash consists of silt-sized particles which are generally spherical, typically ranging in size between 10 and 100 micron.

These small glass spheres improve the fluidity and workability of fresh concrete. Fineness is one of the important properties contributing to the pozzolanic reactivity of fly ash.

There are two major classes of fly ash, class C and class F. The former is produced from burning anthracite or bituminous coal and the latter is produced from burning lignite and sub bituminous coal. Both the classes of fly ash are pozzolans, which are defined as siliceous and aluminous materials (Cokca, 2001). The micro sized fly ash mainly consists of silica, alumina and iron and the particles are generally spherical in size which makes them easy to blend and make a suitable mixture. In order to utilize fly ash in bulk quantities, ways and means are being explored all over the world. In spite of continuous efforts made by the government hardly 5-10% of the fly ash is being used for construction purpose like brick making, cement manufacturing, soil stabilization and as filling material. As the properties of fly ashes vary from place to place, there is a need to check the variability of properties for its effective utilization. Hence, before the utilization of fly ash as a construction material, it is necessary to study properties of fly ash from different sources, so that it can be used beneficially. From different studies obtained it can be seen that silica content in the fly ash is very high. Such high content of silica is reason for the pozzolonic activity

For the purpose of this study, the fly ash was obtained by processing fly ash gotten from Addis Ababa city with geographical coordinates; 8° 58' 32" North, 38° 42' 35" East at the reppie waste- to- energy plant, reppie koshe in a furnace at a temperature of 850 °c. The chemical ash elemental analysis was done at Geological survey of Ethiopia of geochemical laboratory Directorate, Addis Ababa, Ethiopia to adequately determine the class of fly ash based on the percentage composition of the various chemical compounds contained in it and was found to be a brownish Class C fly ash with the following chemical makeup:

Table 3. 1 Chemical properties of FA of Reppie waste to energy according to Geological Survey of Ethiopia 2020.

Chemical Composition (%)	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	Na ₂ O	K ₂ O	TiO ₂	LOI	H ₂ O
Fly ash	28.24	6.8	3.79	26.76	2.2	4.9	10.40	0.76	13.85	0.17

Disposal of fly aash

Generally, in our cases, Fly Ash is an Industrial waste product from thermal power plants, which uses municipal waste as fuel. It is estimated that 12,373.340 tons of fly ash is being produced from thermal power plants in Addis Abeba, Ethiopia consuming acres of precious land for its disposal causing severe health and environmental hazards.

The Electro Static Precipitator (ESP) of the power plants collects these fly ashes. Essentially consisting of alumina, silica and iron, fly ashes are micro-sized particles. Fly ash particles are generally spherical in size, and this property makes it easy for them to blend and flow, to make a suitable concoction. Both amorphous and crystalline natures of minerals are the content of fly ash generated. Its content varies with the change in nature of the municipal waste used for the burning process, but it basically is non-plastic silt.

For the purpose of investigations in this study, fly ash was obtained from Reppie waste to energy plant Addis Ababa Ethiopia.



Figure 3. 5 photographs taken during fly ash preparing

After preparing the fly ash materials from its source and taken to the laboratory in sacks, it was very important to separate out the vegetation and foreign material from the fly ash, this fly ash was screen through a 2 mm sieve. Then the samples were dried in the oven for about 24 hours before further usage.

Usage of municipal waste in thermal power plants for the generation of steam is a common practice. A method that was proved to be non-energy efficient was used in the past, where municipal waste in form of lumps was expended in the furnaces of the boilers to generate the evaporated content: steam. Thus, in order to optimize the production of energy from

municipal waste mass, the thermal power plants began to use pulverized municipal waste mass instead of the aforementioned content. In this process, firstly, this pulverized municipal waste is infused into the combustion chamber, where the instant but efficient burning of fuel happens. The mineral residue left behind after the burning of municipal waste is the bottom ash. The steam around this molten mass, when the municipal waste ash travels with the flue gases, results in the spherical shape of the fly ash particle. Next, the employment of the economizer recovers the heat from the steam gases and fly ash. As a result of this process, the temperature of the fly ash shows a sudden reduction in value. If this temperature fall is rapid, then the resulting structure of the fly ash material is amorphous. However, if the temperature drop during this cooling process is gradual, then the fly ash assumes a more crystalline in nature. This shows the implementation of the economizer, and how it improves the reactivity process.

In the process where fly ash is not subjected to the economizer, it forms a 4.3% soluble matter, and its pozzolanic activity index clocks to 94%. Whereas, during the process where the fly ash exposed to the economizer, its pozzolanic activity clocks to 103% and it forms 8.8% soluble matter. In conclusion, fly ashes are separated from the flue gases by a mechanical dust collector, which is commonly referred to as Electro Static Precipitator (ESP), or scrubbers. Free of fly ashes, the rest of the flue gases are liberated into the atmosphere via the chimney.

With about 90%-98%, the efficiency of ESPs for the separation of finer and lighter fly ash particles is high. In general, the fly ash consists of four to six hoppers, named as field. The fineness of the fly ash particles collected are thus proportional to the number of fields available in an ESP. Therefore, the fly ashes that are collected from the first hopper have a specific surface area of about $2800 \text{ cm}^2/\text{gm}$, whereas the fly ash collected from the last hopper exhibit a greater specific surface area, that is, $8200 \text{ cm}^2/\text{gm}$. With the scorching of pulverized fly ash, the resulting ash content forming during the process is either collected as fly ash or bottom ash. 80% of municipal waste ashes that are removed from the flue gases are recovered as fly ash, whereas the remaining 20%, that are generally coarser in size, are collected at the bottom of the furnace as bottom ash. Either in dry form, or its collection from a water-filled hopper, bottom ash is taken from the bottom of the furnace. When there is a sufficient amount of bottom ash in the water-filled hopper, beyond which its disposal becomes imminent before moving on to the next process, the transference can occur by water jets or water sluice to a disposal pond which this disposed waste is then called as pond ash.

Classification of fly ash

The extracted ash from the flue gases via an Electro Static Precipitator, after the process of pulverization, is called fly ash. It is the finest of particles among bottom ash, pond ash and fly ash. With some unburned carbon, the fly ash chiefly consists of non-combustible particulate matter. These generally consist of silt-sized particles. On the basis of a lime reactivity test, fly ashes have been classified into four different types, as given:

- Cementations fly ash
- Cementations and pozzolanic fly ash
- Pozzolanic fly ash
- Non-pozzolanic fly ash

With free lime content and negligible reactive silica, this fly ash is called as cementations. As opposed to this, with negligible free lime content, and chiefly reactive silica, this fly ash is called pozzolanic fly ash. Both reactive silica and free lime are predominant in cementitious and pozzolanic fly ash. Neither free lime, nor reactive silica are present in non-pozzolanic fly ash. The distinguishable difference between cementitious fly ash and pozzolanic fly ash is that the cementitious fly ash hardens when it comes in connection with water, whereas the pozzolanic fly ash hardens only after the activated lime reacts with water. Cementations & Cementitious and Pozzolanic Fly Ash are the types that are found widely. Based on the chemical composition of fly ash, fly ash has been categorized into two categories, as given:

- Class C fly ash
- Class F fly ash

Burning of municipal solid waste, which contains more than 20% Calcium Oxide, gives the Class C fly ash. By ignition of anthracite and bituminous type of coal, Class F fly ash comes into the picture. This fly ash contains less than 20% Calcium Oxide.

Therefore; in our case of study, the fly ash used is class C type

The chemical configuration of Class C and Class F fly ashes are as follows, in the given table

Table 3. 2 Chemical requirement of class C and class F fly ashes (ASTM C618-94a)

Particulars		Fly ash	
		Class F	Class C
<i>SiO₂+Al₂O₃+ Fe₂O₃</i>	% minimum	70.0	50.0
<i>SO₂</i>	% maximum	5.0	5.0
<i>MC</i>	% maximum	3.0	3.0
<i>LOI</i>	% maximum	6.0	6.0

3.3.3 Utilization of Fly Ash

The utilization of fly ash can be largely grouped into the following three classes:

- The Low Value Utilizations, which includes back filling, structural fills, road construction, soil stabilization, embankment and dam construction, ash dykes, etc.
- The Medium Value Utilizations, which includes grouting, cellular cement, pozzolana cement, bricks/blocks, soil amendment agents, prefabricated building blocks, fly ash concrete, weight aggregate, etc.
- The High Value Utilizations, which includes, fly ash paints, ceramic industry, extraction of magnetite, distempers, metal recovery, acid refractory bricks, floor and wall tiles, etc.

After these, there is still a large wastage of fly ash material observed; however, this has led to evolution of large number of technologies for the management of fly ashes.

So far there is no a wide acceptance of use of fly ash as a product that uses for construction materials in Ethiopia.

Table 3. 3 Production & Utilization of fly ashes in different countries (Akhtar, 2011)

Country	Annual Ash Production (MT)	Ash Utilization in %
India	131	56
China	100	45
Germany	40	85
Australia	10	85
France	3	85
Italy	2	100
USA	75	65
UK	15	50
Canada	6	75
Denmark	2	100
Netherland	2	100

Table 3. 4 Utilization of fly ash for different purposes. Data source: (Ministry of Environment & Forests)

Mode of Fly ash applications	% Utilization
Dykes	35
Cement	30
Land development	15
Building	15
Others	5

3.3.4 Reaction mechanism of Fly ash and expansive soil

By itself, fly ash has little cementitious value, however, this changes in presence of moisture, with which it reacts chemically, and forms cementitious compounds. These compounds attributes to the improvement of compressibility and strength characteristics of a soil. Both classes of Fly ash (C & F) are pozzolans i.e. they contain siliceous and aluminous materials. Fly ash can thus produce an assortment of divalent and trivalent cations (Ca^{2+} , Fe^{3+} , Al^{3+} etc.) under conditions that are ionized in nature, which in return can encourage flocculation of dispersed clay particles. Expansive soils can thus be theoretically stabilized in an effective manner by cationic exchange with fly ash.

3.3.5 Water

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

3.4 Methods

3.4.1 Sample Preparation

Prior to treatment and testing, the sample was prepared in accordance with the method described in AASHTO T87-86. This method involves:

- Air drying of samples and/or oven drying at 105°C or less;
- Breaking up the soil aggregates by rubber covered mallet. Then, sieve analysis were performed to separate the dried soils into two groups. The first group involves preparing uniform samples for Atterberg limits, free swell, free swell index and free swell ratio tests.
- Then, soil and fly ash was mixed manually to get uniform mix ratio for each test.

3.4.2 Methodology Adopted

To evaluate the effect of fly ash as a stabilizing additive in expansive soils, series of tests were done. For this purpose the content of fly ash used to stabilize expansive soil was varied in values of 4% to 20% (multiples of 4) by weight of the total quantity taken. The Indian Standard codes were followed during the conduction of the following experiments:

- Soil Moisture content-IS:2720(part 2)-2018
- Liquid & Plastic limit test (AASHTO T 89 – 96 and AASHTO T 90 - 00)
- Standard proctor test – IS : 2720 (Part 7) – 1980
- Free swell index test – IS 2720 (Part 40) - 1977
- California bearing ratio (CBR) test – IS : 2720 (Part 16) - 1987

- Unconfined compressive strength (UCS) test – IS: 2720 (Part 10) - 1991



Figure 3. 6 photograph taken from power plant

3.5 Laboratory Tests

3.5.1 Determination of Moisture Content

The moisture content of a soil sample is defined as the mass of water in the sample expressed as a percentage of the dry mass, usually heating at 105°C .

The test was conducted in accordance with AASHTO T265. Small representative sample of the natural soil and soil-fly ash mixture specimens was prepared accordingly and oven-dried at a standard temperature. The samples were then reweighed, and the difference in weight was determined, the weight of the water driven off during drying. The difference in weight divided by the weight of the dry soil, gives the water content of the soil a dry weight basis.



Figure 3. 7 Shows soil natural moisture content lab.determination.

$$\text{i.e Moisture content, } w(\%) = \frac{M_w}{M_D} \times 100$$

Where,

W= water content

M_w = mass of water

M_D = dry mass of sample

Oven-drying method (standard method)

Apparatus

- ✓ Thermostatically controlled drying oven capable of operating to 105⁰C.
- ✓ Glass weighing bottles
- ✓ Balance (with the required sensitivity).
- ✓ Desiccator containing anhydrous silica gel.
- ✓ Scoop, other small tools as appropriate.

Test procedure

- ✓ One clean container with the lid was taken and the mass in grams was recorded (m1) together with container number.
- ✓ The sample of wet soil was crumbled and placed in the container. The container with the lid weighed in grams (m2).
- ✓ The lid was removed and both lid and container are placed in the oven. The sample was then dried in a thermostatically controlled drying oven which was maintained at a temperature of 105⁰C. A period of 16 to 24 hours was used sufficiently. It was also vary if the oven contains a large number of samples or very wet samples.

Therefore; consideration was taken during laboratory session so that expected values to be gain. The soil was considered dry when the differences in successive weighings of the cooled soil at 4 hour intervals.

- ✓ For fine-grained soils, the container and lid should preferably be placed in a dessicator and allowed to cool. After cooling, the lids or stoppers should be replaced and the container plus dry soil weighed in grams (m3).

3.5.2 Atterberg Limits Testing

Introduction

Atterberg limit tests are conducted on fine-grained soils to determine the amount of water required to attain different behavioral states. For AASHTO and USCS, soil classification systems value of Atterberg tests is required. The Atterberg limit test that was done in this study includes the determination of the liquid limits, plastic limits, and the plasticity index for the natural soil and soil-fly ash mixtures. The tests were conducted following AASHTO Designation T89. Air-dried samples passing the No.40 (0.425mm) sieve was used to conduct the test.

Liquid limit (LL): The liquid limit is defined as the moisture content at the boundary between the liquid and plastic state. To accomplish the test first, the soil or soil-fly ash sample was thoroughly mixed with water by repeatedly stirring, kneading, and chopping with the spatula and placed in the Casagrande cup. The cup containing the sample was lifted and dropped at a rate of 2 drops/sec until the two sides of the sample come in contact at the bottom of the groove along with a distance of about 13mm. The number of blows required to close the grove to this distance was recorded and its moisture content was determined. The relationship between the water content and the number blow is plotted. The water content corresponding to the intersection of the line with the 25-blow abscissa is the liquid limit of the soil.

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

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

3.5.3 Particle Size Distribution

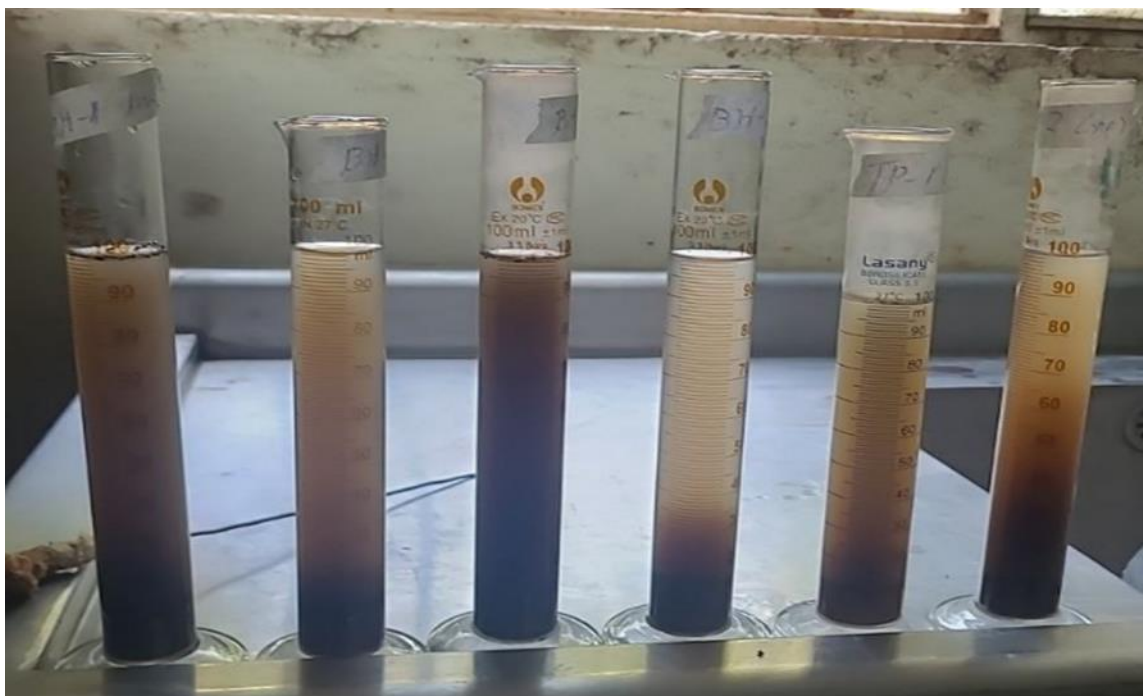
This test was conducted to determine the percentage of different grain sizes contained within a soil. Wet sieve and hydrometer analysis was conducted in this study according to AASHTO T88-93. The wet sieve analysis was used to determine the distribution of the coarse, larger-sized particles, and the hydrometer method was also used to determine the distribution of finer soil particles which were passed sieve size 0.075mm (N0.200) to measure the amount of silt and clay size particles. A hydrometer reading was conducted by using hydrometer 151H to compute the percent retained.

3.5.4 Classification

The soil was classified using the AASHTO soil classification system. Using the particle size distribution and the Atterberg limits, AASHTO designates a group name for each soil. A visual-manual procedure was also used to identify soils easily in the field; however, all classifications of soils in this research will be governed on the laboratory testing procedure.

3.5.5 Free Swell Index

The test was including determination of the free swell index of the natural soil and the soil fly ash mixture and conducted in accordance with IS: 2720 (Part 40) 1977 testing procedure.



This standard (Part 40) covers a test for the determination of free swell index of soil which helps to identify the potential of a soil to swell which might need further detailed investigation regarding swelling and swelling pressures under different field conditions.

Apparatus

- ✓ Sieve - 425-micron IS Sieve.
- ✓ Glass Graduated Cylinders - Two, 100-ml capacity (see IS: 878-1956.)

Procedure

Take two 10 g (see Note) soil specimens of oven dry soil passing through 425-micron IS Sieve.

NOTE -In the case of highly swelling soils, such as sodium bentonites, the sample size may be 5 g or alternatively a cylinder of 250 ml capacity may be used.

Each soil specimen shall be poured in each of the two glass graduated cylinders of 100 ml capacity. One cylinder shall then be filled with kerosene oil and the other with distilled water up to the 100 ml (see Note under 3.1) mark. After removal of entrapped air (by gentle shaking or stirring with a glass rod), the soils in both the cylinders shall be allowed to settle. Sufficient time (not less than 24 h) shall be allowed for the soil sample to attain equilibrium state of volume without any further change in the volume of the soils. The final volume of soils in each of the cylinders shall be read out.

Calculation

The level of the soil in the kerosene graduated cylinder shall be read as the original volume of the soil samples, kerosene being a non-polar liquid does not cause swelling of the soil. The level of the soil in the distilled water cylinder shall be read as the free swell level. The free swell index of the soil shall be calculated as follows:

$$\text{Percent} = \frac{V_d - V_k}{V_k} \times 100$$

Where, V_d = the volume of soil specimen read from the graduated cylinder containing distilled water, and

V_k , = the volume of soil specimen read from the graduated cylinder containing kerosene.

3.5.6 Specific Gravity

Specific gravity which is the measure of heaviness of the soil particles were determined by the method of pycnometer method using a soil sample passing No. 10 sieve and oven dried at 105 degree centigrade.

The test was includes the determination of the specific gravity for the natural soil and the soil fly ash mixture. The test was conducted in accordance with AASHTO T100-93 testing procedure.

Apparatus

- ✓ Two density bottles (pycnometers) of approximately 50 ml capacity with stoppers.
- ✓ A water-bath maintained at a constant temperature to within $\pm 0.2^\circ\text{C}$. (If standard density bottles are used this constant temperature is 27°C .)
- ✓ A vacuum desiccator (a convenient size is one about 200 mm to 250 mm in diameter).
- ✓ A desiccator (a convenient size is one about 200 mm to 250 mm in diameter) containing anhydrous silica gel.
- ✓ A thermostatically controlled drying oven, capable of maintaining a temperature of 105 to 110°C .
- ✓ A balance readable and accurate to 0.001 g.
- ✓ A source of vacuum, such as a good filter pump or a vacuum pump.
- ✓ A spatula (a convenient size is one having a blade 150 mm long and 3 mm wide; the blade has to be small enough to go through the neck of the density bottle), or a piece of glass rod about 150 mm long and 3 mm diameter.
- ✓ A wash bottle preferably made of plastics, containing air-free distilled water (see Note).
- ✓ A sample divider of the multiple slot type (riffle box) with 7 mm width of opening.
- ✓ A length of rubber tubing to fit the vacuum pump and the desiccator.



Figure 3. 8 Picture shows sample preparation



Figure 3. 9 Photograph taken during lab.test

Calculation

The specific gravity of the soil particles G was measured at room temperature. The water used was air-free liquid, and the following equation was used:

$$G = \frac{G_L(m_2 - m_1)}{m_4 - m_1 - (m_3 - m_2)}$$

Where

G_L = specific gravity of the liquid used, at the constant temperature;

m_1 = mass of density bottle in g;

m_2 = mass of bottle and dry soil in g;

m_3 =mass of bottle, soil and liquid in g; and

m_4 = mass of bottle when full of liquid only in g.

3.5.7 Compaction

The test was included determination of the maximum dry density and the optimum moisture content for the natural soil and the soil-fly ash mixture. It was also conducted for uncured and 7days cured stabilized soil samples in accordance with AASHTO T99-94 testing pro.

Apparatus

- ✓ Mold .
- ✓ Mold assembly
- ✓ Rammer Balance(s) or scale(s)
- ✓ Drying apparatus.
- ✓ Straightedge
- ✓ Sieve(s) – 4.75 mm (No. 4) and/or 19.0 mm (3/4 in.),
- ✓ Mixing tools
- ✓ Miscellaneous tools such as a mixing pan, spoon, trowel, spatula, Containers with close-fitting lids to prevent gain or loss of moisture in the sample.

Calculations

$$\text{Wet Density, } \rho_w = \frac{M_w}{V_m}$$

Where:

ρ_w =wet density, kg/m³ (lb/ft³)

M_w =wet mass

V_m = volume of the mold, Annex B

$$\text{Dry Density, } \rho_d = \left(\frac{\rho_w}{w+100} \right) \times 100 \quad \text{or} \quad \rho_d = \frac{\rho_w}{\left(\frac{M_w}{100} \right) + 1}$$

Where:

ρ_d = dry density, kg/m³ (lb/ft³)

w = moisture content, as a percentage

Moisture-Density Curve Development

When dry density was plotted on the vertical axis versus moisture content on the horizontal axis and the points were connected with a smooth line, a moisture-density curve was developed. The coordinates of the peak of the curve were the maximum dry density, or just “maximum density,” and the “optimum moisture content” of the soil.

3.5.8 CBR and CBR-swell

The CBR and CBR-swell tests were being conducted in accordance with AASHTO T193-93 for the natural soils and soil- fly ash mixture.

i. Apparatus

Molds—the cylindrical shape with standard dimension mold was used as shown in the following figure.

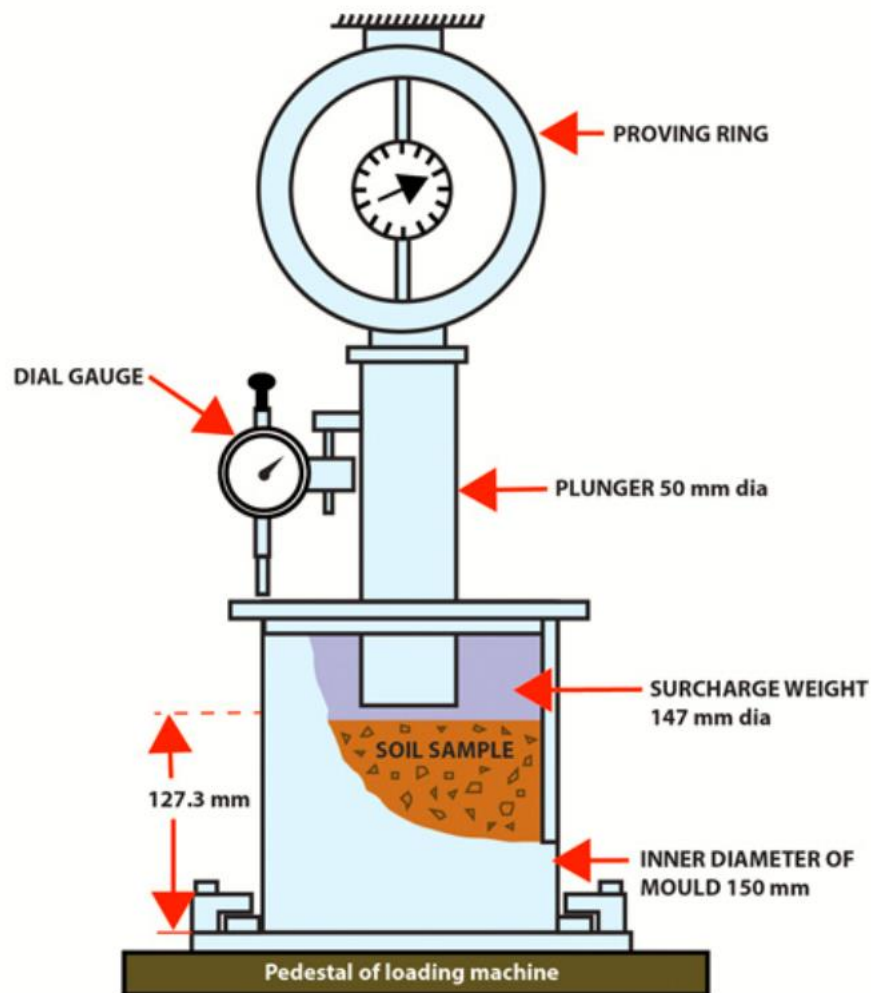


Figure 3. 10 California Bearing Ratio Apparatus set

Miscellaneous—miscellaneous tools such as mixing pans, spoons, straightedge, filter paper, balances, etc were also used.

A. Sample

The sample was handled and specimen(s) for compaction were prepared in accordance with the procedures given in T 99 or T 180 for compaction in a 152.4-mm mold except as follows:

B. Moisture-Density Relation

Bearing Ratio at Optimum Water Content—using the 11-kg (25-lb) portion prepared, as specified, T 99 or T 180.

C. Bearing Ratio at Optimum Water Content:

Normally, three specimens were compacted so that their compacted densities range from 95 percent (or lower) to 100 percent (or higher) of the maximum dry density determined in Section 6.1. Note 5 generally about 10, 30, and 65 blows per layer are suitable for compacting specimens 1, 2, and 3, respectively. 56 blows per layer were done. to mold a CB specimen to 100 percent of the maximum dry density determined by T 99 (Method D); this was due to the sample for the moisture-density test being reused, while the sample for the CBR specimen was mixed and compacted only once.

By having clamped the mold to the base plate, attached to the extension collar and weighed to 5 g (0.01 lb). Inserted the spacer disk into the mold and placed a coarse filter paper on top of the disk.

Optimum moisture content was determined by adding sufficient water, and then mixed each of the three 6.8-kg portions. sample in the mold

Soaking

swell plate was placed to the adjusted stem and sufficient annular weights was applied to sample in the mold and produce an intensity of loading equal to the mass of the sub base and base courses and surfacing above the tested material.

At the end of 96 hours, make a final dial reading on the soaked specimens and calculate the swell as a percentage of the initial sample length:

Change in length in mm during soaking

$$\text{Percent swell} = \frac{\text{Change in length in mm during soaking}}{116.43\text{mm}} \times 100$$

Calculations

California Bearing Ratio—The corrected load values was determined for each specimen at 2.54 and 5.08 mm (0.10 and 0.20 in.) penetration. California Bearing Ratio values were obtained in percent by divided the corrected load values at 2.54 and 5.08 mm (0.10 and 0.20 in.) by the standard loads of 6.9 and 10.3 MPa (1000 and 1500 psi), respectively, and multiplied the ratios by 100.010

$$\text{CBR} = \frac{\text{Corrected load value}}{\text{Standard load}} \times 100$$

3.5.9 Unconfined Compression (UCS) Testing

Under Unconfined compressive strength (UCS), tests were conducted on specimens prepared from the soils and soil-fly ash mixtures following ASTM D 5102.

Shear strength of a soil used was shown by the Mohr-Coulomb failure criteria as

$$s = c + \sigma + \tan \phi$$

Where s = shear strength

c = cohesion

σ = normal stress

ϕ = angle of friction.

For undrained tests of saturated clayey soils ($\phi = 0$)

$$s = c_u$$

Where c_u = untrained cohesion (or untrained shear strength).

The unconfined compression test is a quick method of determining the value of C_u for a clayey soil. The unconfined strength was given by the relation

$$q_u = \frac{c_u}{2}$$

Where q_u = unconfined compression strength.

The unconfined compression strength is determined by applying an axial stress to a cylindrical soil specimen with no confining pressure and observing the axial strains corresponding to various stress levels. The stress at which failure in the soil specimen occurs was referred to as the unconfined compression strength.

Equipment

- ✓ Unconfined compression testing device

- ✓ Specimen trimmer and accessories (if undisturbed field specimen is used)
- ✓ Harvard miniature compaction device and accessories (if a specimen is to be molded for classroom work)
- ✓ Scale
- ✓ Balance sensitive to 0.01 g
- ✓ Oven
- ✓ Porcelain evaporating dish

Unconfined Compression Test Machine

An unconfined compression test machine in which strain-controlled tests can be performed was shown in **Fig. 3.11**. The machine essentially consists of a top and a bottom loading plate. The bottom of a proving ring is attached to the top loading plate. The top of the proving ring is attached to a cross-bar which, in turn, is fixed to two metal posts. The bottom loading plate can be moved up or down.

Procedure

Obtain a soil specimen for the test. If it is an undisturbed specimen, it has to be trimmed to the proper size by using the specimen trimmer. For classroom laboratory work, specimens at various moisture contents can be prepared using a Harvard miniature compaction device.

The cylindrical soil specimen should have a height-to-diameter (LID) ratio of between 2 and 3. In many instances, specimens with diameters of 35.56 mm and heights of 88.9 mm are used.

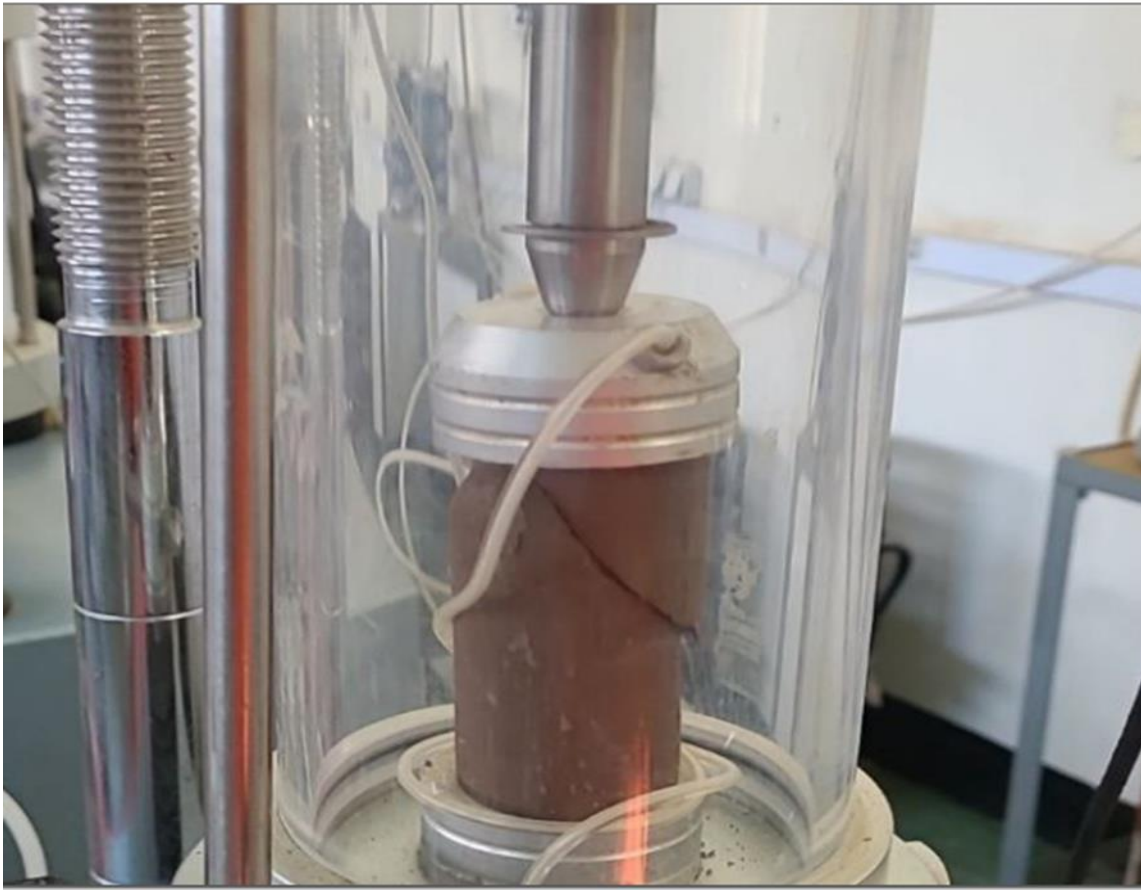


Figure 3. 11 A soil specimen after failure

Measure the diameter (D) and length (L) of the specimen and determine the mass of the specimen.

Place the specimen centrally between the two loading plates of the unconfined compression testing machine. Move the top loading plate very carefully just to touch the top of the specimen. Set the proving ring dial gauge to zero. A dial gauge [each small division of the gauge should be equal to 0.001 in. (0.0254 mm) of vertical travel] should be attached to the unconfined compression testing machine to record the vertical upward movement (i.e., compression of the specimen during testing) of the bottom loading plate. Set this dial gauge to zero.

Turn the machine on. Record loads (i.e., proving ring dial gauge readings) and the corresponding specimen deformations. During the load application, the rate of vertical strain should be adjusted to $\frac{1}{2}$ % to 2% per minute. At the initial stage of the test, readings are usually taken every 0.01 in. (0.254 mm) of specimen deformation. However, this can be varied to every 0.02 in. (0.508 mm) of specimen deformation at a later stage of the test when the load-deformation curve begins to flatten out.

1. Taking readings was continued until
 - i) Load reaches a peak and then decreases; or
 - ii) Load reaches a maximum value and remains approximately constant thereafter
(5 readings were taken after it was reached the peak value).
2. The specimen was unloaded by lowering the bottom loading plate.
3. The specimen was removed from the two loading plates.
4. A free-hand sketch of the specimen was drawn after failure.
5. The specimen was putted in a porcelain evaporating dish and determined the moisture content (after drying it in an oven to a constant weight).

Calculation

6. The vertical strain was Calculated by using the following formula:

$$\varepsilon = \frac{\Delta L}{L}$$

Where ΔL = total vertical deformation of the specimen

L = original length of specimen. ΔL

7. The vertical load on the specimen was Calculated by using the following formula:

Load = (proving ring dial reading, i.e. Column 3) x (calibration factor)

8. The corrected area of the specimen was Calculated by using the following formula:

$$A_c = \frac{A_o}{1 - \varepsilon}$$

where A_o = initial area of cross – section of the specimen $= \frac{\pi}{4} D^2$

- 3 The stress, σ , on the specimen was Calculated by using the following formula:

$$\sigma = \frac{\text{Load}_{\text{column4}}}{A_c \text{ column5}}$$

CHAPTER FOUR

4 RESULTS AND DISCUSSIONS

4.1 Introduction

In this Chapter the laboratory test results are briefly discussed and the effect of fly ash evaluated for each laboratory tests. The physical and engineering properties are summarized to discuss the results of laboratory tests of, natural moisture content, Atterberg limits, Particle Size Distribution analysis, Classification, Free Swell Index, Free Swell Tests, specific gravity, moisture density relation (compaction) California bearing ratio (CBR), swell (CBR) unconfined compressive strength (UCS),

4.2 Engineering properties of black cotton soil and fly ash

4.2.1 Result observed on Moisture Content

The natural moisture content determination of an expansive soil (black cotton soil) was done after Small representative sample of the natural soil brought from the three selected areas with the depth of 1.5m each, by using a plastic bag so that the soil not to lose its natural state and the test was conducted in accordance with AASHTO T265 and the result was taken the average of the three and was 32.78%

4.3 Effects of fly ash on Engineering Properties of expansive Soil

Fly ash samples were collected from Reppie waste to electric energy in Addis Ababa and expansive soil (black cotton soil) is excavated and collected from Dukem city and brought to laboratory for tests. The expansive soil (black cotton soil) in this research was treated with fly ash which is both passed sieve No. 4 (4.75mm) and for attereberg limit only both samples which passed through Sieve No 40 (0.425mm) with ratio of 0%, 4%, 8%, 12%, 16%, and 20% by weight of dry soil. The engineering properties of expansive soil (black cotton soil) and treated with fly ash are presented for different types of tests as follows.

4.4 Test results of expansive soil for both untreated and treated with fly ash.

4.4.1 Result observed on attereberg limits

The test was including the determination of the liquid limits, plastic limits and the plasticity index for both the natural Black cotton soil and treated soil with fly ash. The following test results were conducted in accordance with AASHTO T89-90 and T90-96 testing procedures. The result of liquid limit, plastic limit and plastic index decreased when treated with fly ash as shown in the table below

Table 4. 1 Liquid limit, Plastic limit and Plastic index verses % of fly ash.

S. No	Percent of fly ash added	Liquid limit (LL) %	Plastic limit (PL) %	Plastic index (PI) %
1	0%	60.02	32	28.02
2	4%	58.03	31.38	26.65
3	8%	57.02	31.08	25.94
4	12%	55.46	30.45	25.01
5	16%	54.5	29.86	24.64
6	20%	53.43	28.88	24.55

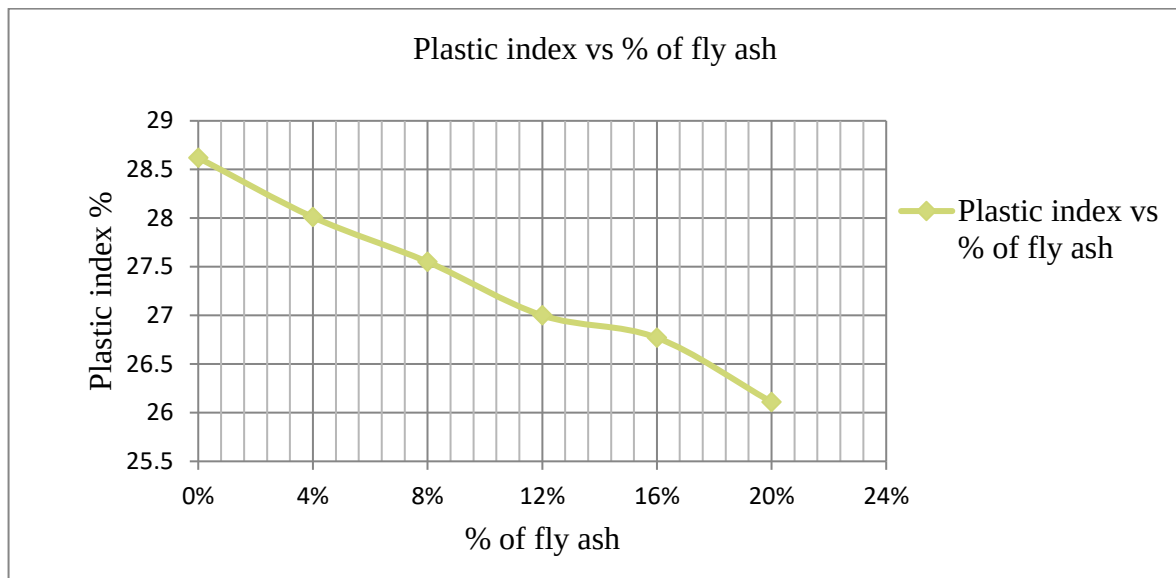


Figure 4. 1 Liquid limit vs. fly ash

4.4.2 Result observed on moisture density relation (compaction)

The result of expansive soil (black cotton soil) and fly ash passing No 40 (0.425 mm) sieve were used to determine moisture-density relation of the expansive soil (black cotton soil) mixed with varying proportions of bottom ash additives in accordance to AASHTO T99 and from the table and the graph of moisture content versus per cent of Fly ash the minimum optimum moisture content and from maximum dry density verses per cent of fly ash the maximum result was observed at 20% and lesser OMC better was soil as if achieve MDD sooner at low water content because of its pozzuolana property.

Table 4. 2 Tables of Moisture -Density Relation at BH1,BH2 and BH3

Types of Boreholes(BH)	Volume of Mold(m3)	Weight of soil in mold(kg)	Moist unit weight (g/cm3)	Moisture content (%)	Dry unit weight (g/cm3)
BH1	1000	1364	1.364	24.27	1.1
	1000	1373	1.373	24.21	1.11
	1000	1390	1.39	24.83	1.11
	1000	1419	1.419	25.97	1.13
	1000	1581	1.581	31.11	1.21
	1000	1683	1.683	35.55	1.24
	1000	1715	1.715	39.73	1.23
	1000	1700	1.7	42.6	1.19
BH2	1000	1364	1.364	24.27	1.13
	1000	1373	1.373	24.21	1.19
	1000	1390	1.39	24.83	1.26
	1000	1419	1.419	25.97	1.19
	1000	1581	1.581	31.11	1.37
	1000	1683	1.683	35.55	1.39
	1000	1715	1.715	39.73	1.19
	1000	1700	1.7	42.6	1.19
BH3	1000	1364	1.364	24.27	1.17
	1000	1373	1.373	24.21	1.23
	1000	1390	1.39	24.83	1.29
	1000	1419	1.419	25.97	1.37
	1000	1581	1.581	31.11	1.38
	1000	1683	1.683	35.55	1.18

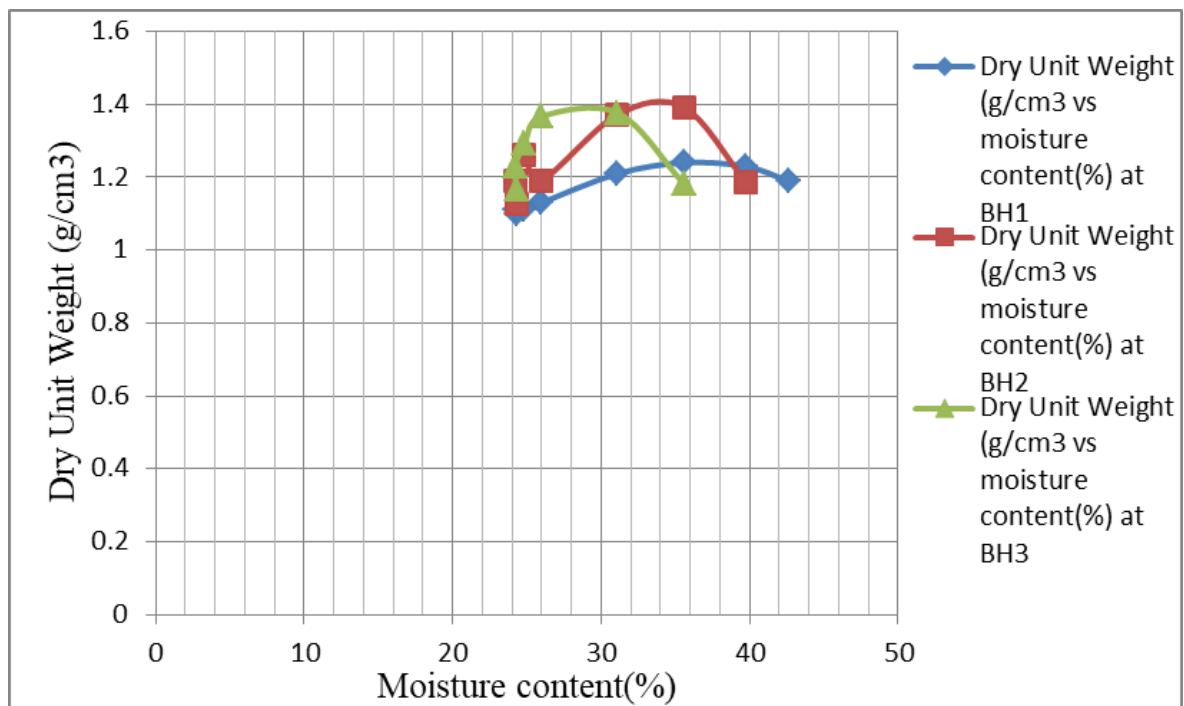


Figure 4. 2 Summary of figures of Moisture -Density Relation at BH1,BH2 and BH3

Table 4. 3 Tables of OMC (%) vs % of fly ash at BH1, BH2 and BH3

Borehole type	Fly ash added	OMC)	MDD(g/cm3)
BH1	4%	27%	1.33
	8%	25%	1.41
	12%	30%	1.43
	16%	32%	1.57
	20%	30%	1.58
BH2	4%	28%	1.29
	8%	30%	1.32
	12%	35%	1.42
	16%	32%	1.57
	20%	30%	1.58
BH3	4%	27%	1.28
	8%	28%	1.37
	12%	35%	1.42
	16%	34%	1.57
	20%	30%	1.58

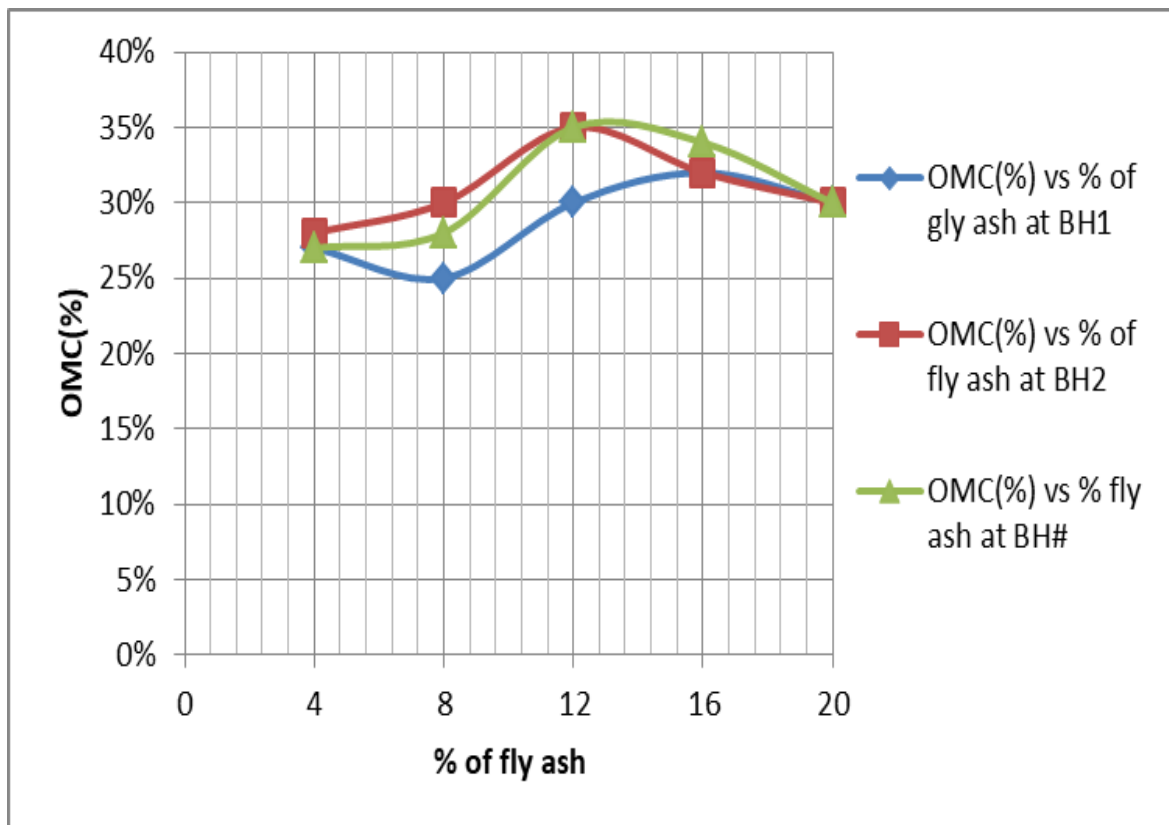


Figure 4. 3 Summary of figures of OMC vs % of fly ash at BH1,BH2 and BH3

Table 4. 4 Table of MDD and OMC For BH1,BH2 and BH3 at different percentage addition of fly ash

Fly ash added	MDD FOR BH1	MDD FOR BH2	MDD FOR BH3	MDD Average	OMC(%) For BH1	OMC(%) For BH1	OMC(%) For BH1	OMC(%) Average
4%	1.33	1.29	1.28	1.3	27%	28%	27%	27%
8%	1.41	1.32	1.37	1.37	25%	30%	28%	28%
12%	1.43	1.42	1.42	1.42	30%	35%	35%	33%
16%	1.57	1.57	1.57	1.57	32%	32%	34%	33%
20%	1.58	1.58	1.58	1.58	30%	30%	30%	30%

4.4.3 Result observed on Particle Size Distribution analysis test

The test was includes determination of the particle size distribution for the natural soil. It was conducted in accordance with AASHTO T88-93 testing procedures. And the following results were drawn from the test

Table 4. 5 Sieve Analysis at BH1

Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % $(WR/TW*100)$	Passing % $((Tw-\epsilon wr)/Tw)*100$	Spec's Requirements
3"	75.00				Gravel
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				Course Sand
No.4	4.75	1.0000	0.20	99.80	
No.8	2.36	2.0000	0.40	99.40	
No.10	2.00	0.0000	0.00	99.40	
No.16	1.18	2.0000	0.40	99.00	Medium Sand
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	3.0000	0.60	98.40	
No.40	0.425	0.0000	0.00	98.40	
No.50	0.300	4.0000	0.80	97.60	Fine Sand
No.100	0.150	4.0000	0.80	96.80	
No.200	0.075	2.0000	0.40	96.40	
Pan		482.0000	96.40	0	
Total Weight (g)		500.0000			
Loss (g)					

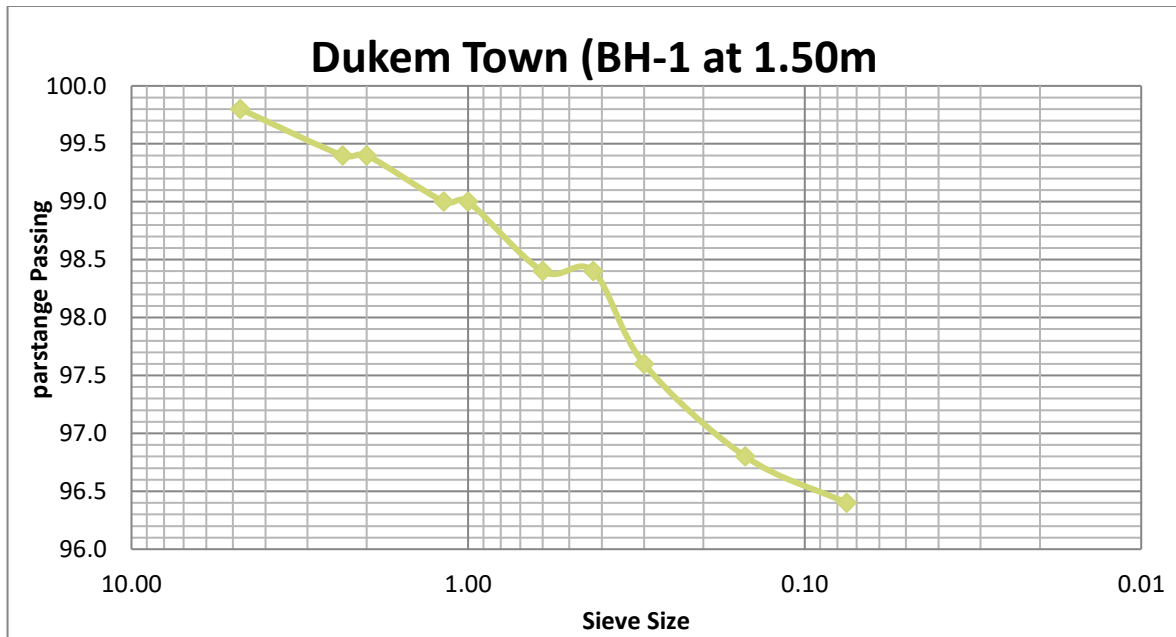


Figure 4. 4 Sieve Analysis at BH1

Table 4. 6 sieve analysis at BH2

Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % $(WR/TW*100)$	Passing % $((TW-\epsilon_{wr})/TW)*100$	Spec's Requirements
3"	75.00				
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				
No.4	4.75	1.0000	0.20	99.80	
No.8	2.36	2.0000	0.40	99.40	
No.10	2.00	1.0000	0.20	99.20	
No.16	1.18	1.0000	0.20	99.00	
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	0.0000	0.00	99.00	
No.40	0.425	0.0000	0.00	99.00	
No.50	0.300	2.0000	0.40	98.60	
No.100	0.150	4.0000	0.80	97.80	
No.200	0.075	5.0000	1.00	96.80	
Pan		484.0000	96.80	0.00	
Total Weight (g)		500.0000			
Loss (g)		0	100.0		

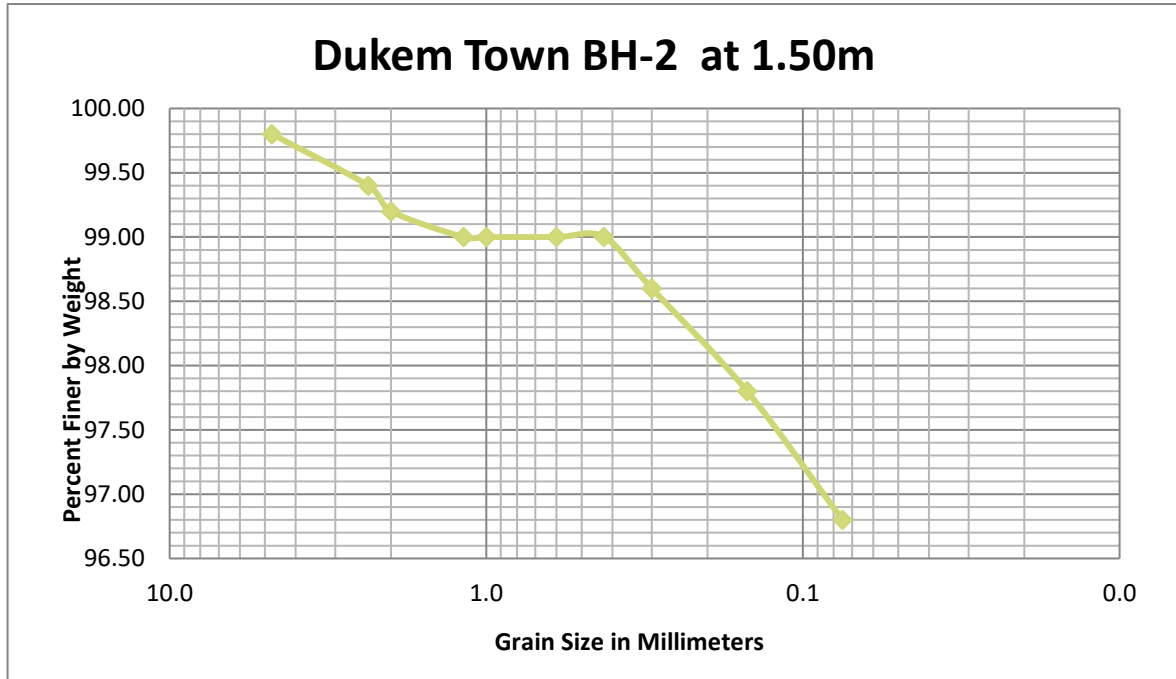


Figure 4. 5 sieve analysis at BH2

Table 4. 7 Sieve analysis at BH3

Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % $(W_R/TW \times 100)$	Passing % $((TW - \epsilon_{wr})/TW) \times 100$	Spec's Requirements
3"	75.00				
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				
No.4	4.75	0.0000	0.00	100.00	
No.8	2.36	1.0000	0.20	99.80	
No.10	2.00	1.0000	0.20	99.60	
No.16	1.18	3.0000	0.60	99.00	
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	4.0000	0.80	98.20	
No.40	0.425	0.0000	0.00	98.20	
No.50	0.300	6.0000	1.20	97.00	
No.100	0.150	10.0000	2.00	95.00	
No.200	0.075	9.0000	1.80	93.20	
Pan		466.0000	93.20	0	
Total Weight (g)		500.0000			
Loss (g)		0	100.0		

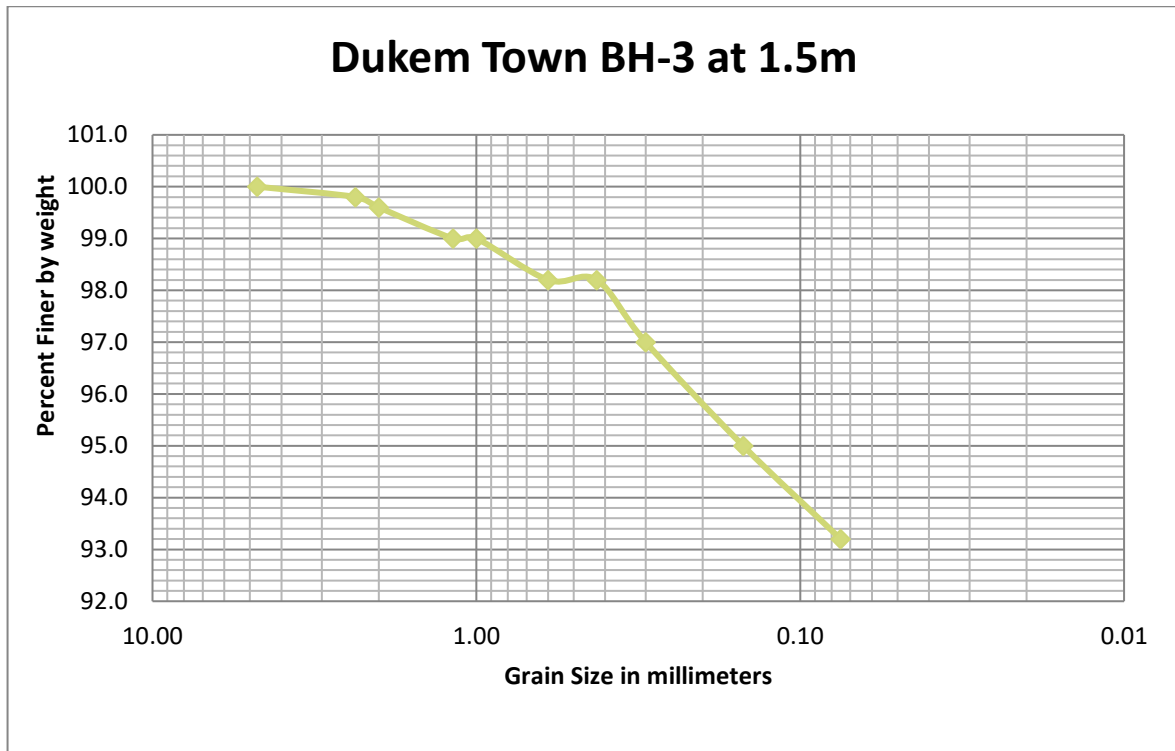


Figure 4. 6 Sieve analysis at BH3

4.4.4 Classification,

The soil was classified using the Unified Soil Classification System (USCS) soil classification system. Using the particle size distribution and the Atterberg limits, Unified Soil Classification System designated a group name for each soil. As test result shows in the figure 4.8 the soil type was MH.

Table 4. 8 Liquid limit vs. Plastic index

Fly ash added.	LL	PI
0	60.02	28.02
4%	58.03	27.02
8%	56.91	25.67
12%	55.46	25.01
16%	55	24.03
20%	54.09	23.54

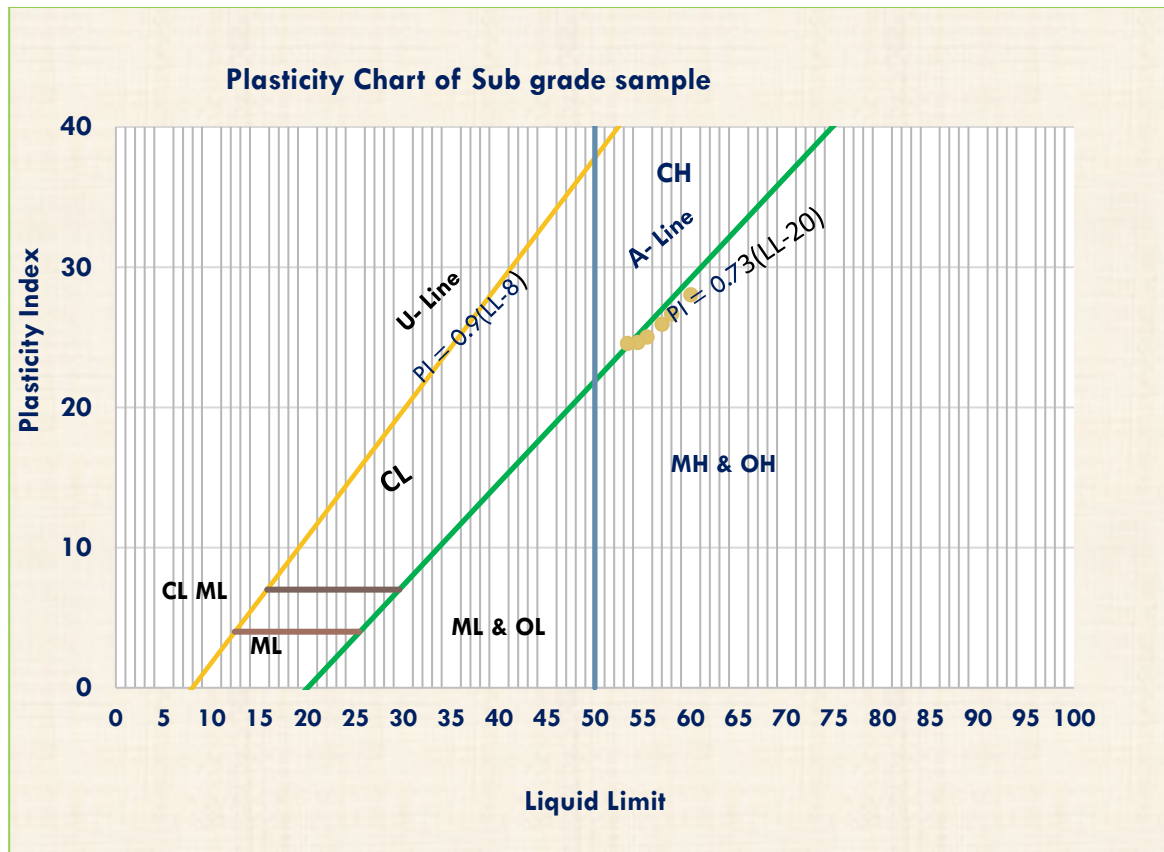


Figure 4. 7 Plasticity Chart

Effect of fly ash on the free swell, free swell index, and free swell ratio

The addition of fly ash to expansive soil slightly decreases the free swell, free swell index, and free swell ratios. The highest decrease is obtained when the soil is treated with 20% fly ash. The free swell index value decreased from 102% to 73 % with an increased amount of fly ash percentages from 0% to 20%. According to swell classification based on a free swell, the treated soil still has a high degree of expansion as discussed by (Chen, 1975). The decrease in free swell values is mainly due to the replacement of expansive clay minerals with fly ash which have less affinity for absorbing water as compared to expansive soil. The summary of the free swell, free swell index, and the free swell ratio is discussed in Table 4.10 and Figures 4.9 and 4.10.

Table 4. 9 The summary of the free swell, free swell index, and free swell ratio

Fly ash content	Free swell (%)	Free Swell Index (%)	Free swell ratio	%Decrease in free swell	%Decrease in free swell index	%Decrease in free swell ratio
0% FA	114	102	2.06	-	-	-
4%FA	109	90	1.89	4.38	11.76	8.25
8% FA	104	84	1.85	8.77	17.64	10.19
12%FA	90	81	1.81	21.05	20.59	12.13
16%FA	84	78	1.74	26.31	23.53	15.53
20%FA	76	73	1.68	33.33	28.43	18.44

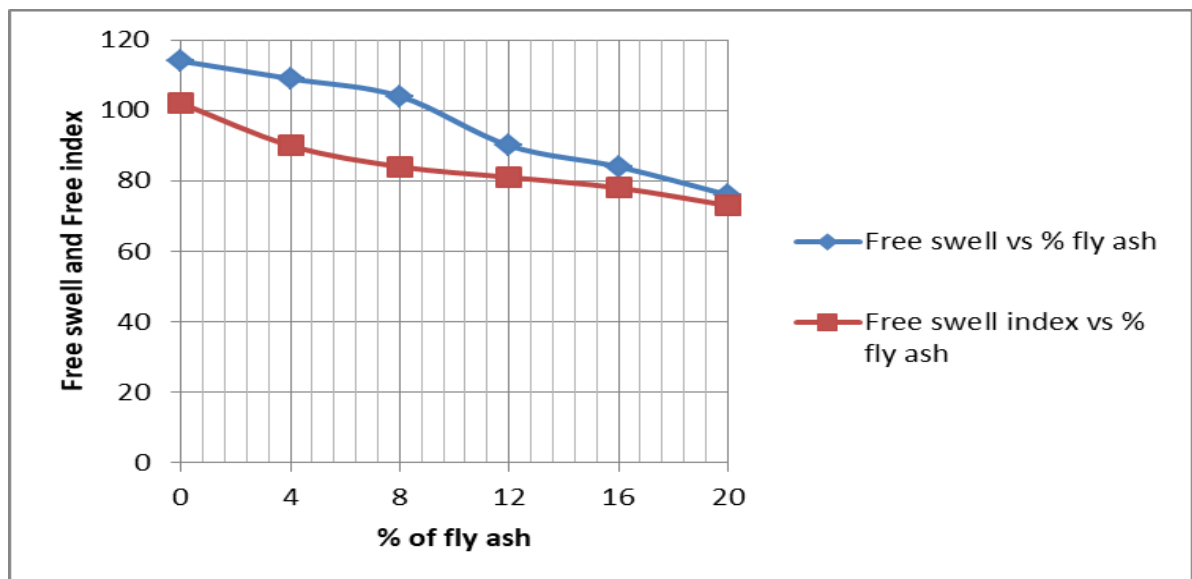


Figure 4. 8 Results of the free swell index and free swell

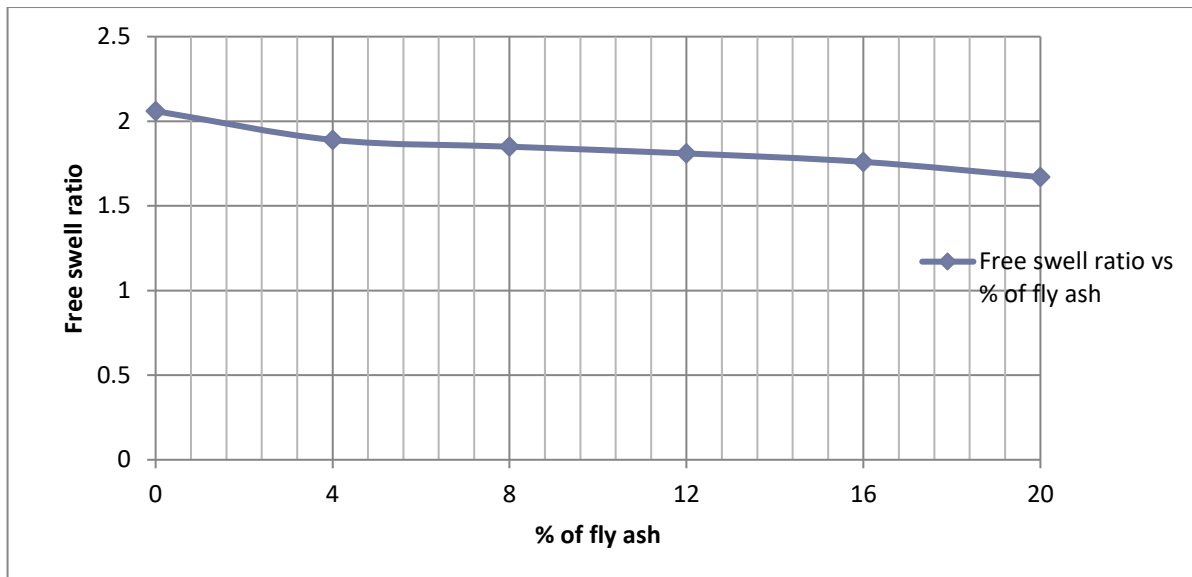


Figure 4. 9 Results of free swell ratio

Effect of Fly ash on linear shrinkage of expansive soil

The effect of different percentages of fly ash on linear shrinkage of expansive soil is Summarized in Table 4.11. The addition of fly ash decreases the linear shrinkage value. From 14.75 to 8.11 The soil is changed from critical stage to moderate shrinkage behavior of expansive soil. The increase in fly ash resulted in a decrease in linear shrinkage of soil. These could be due to the flocculation and aggregation phenomena of clay particles induced by the presence of fly ash contributing to the formation of coarser particles and enhancing the strength of the treated soil.

Table 4. 10 Effect of fly ash on linear shrinkage of expansive soil

Fly ash	Initial length(cm)	Final(cm)	Linear shrinkage (%)
0%	14	12.2	14.75
4%	14	12.5	12.00
8%	14	12.7	10.23
12%	14	12.9	8.53
16%	14	12.95	8.30
20%	14	12.99	8.11

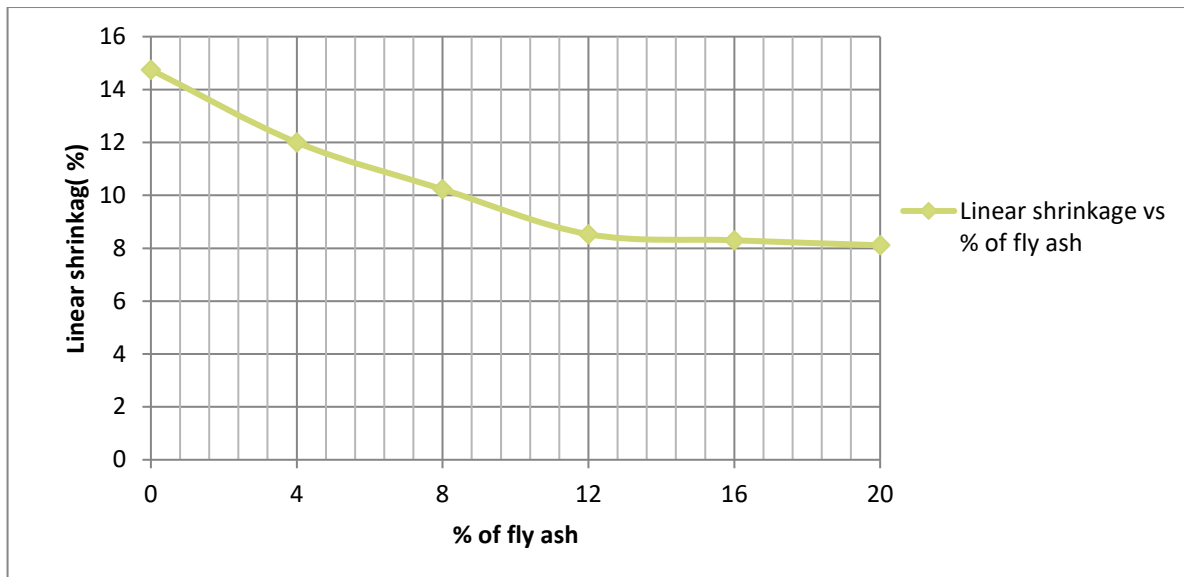


Figure 4. 10 Linear shrinkage of expansive soil mixed with various bagasse ash contents

4.4.5 Result observed on Specific gravity test results

According to the test method ASHTOT100-93, When different percentages of fly ash (0%,4%,8%,12%,16% and 20%) are added to the natural soil sample the specific gravity is increased from 2.60 to 2.68 as shown in Table 4.12. This increase in specific gravity is due to the partial replacement of lower specific gravity soil with fly ash which has higher specific gravity. It can be seen that the more the amount of stabilizer higher would be the specific gravity. It is due to the replacement of soil with the little bit thicker particles of stabilizers, which are denser in weight than soil particles. The effect of fly ash on the specific gravity of the soil is shown in Figure 4.12

Table 4. 11. Specific Gravity versus % of fly ash

S.No	% of fly ash added	Specific gravity
1	Pure or 0%	2.60
2	4%	2.64
3	8%	2.64
4	12%	2.65
5	16%	2.67
6	20%	2.68

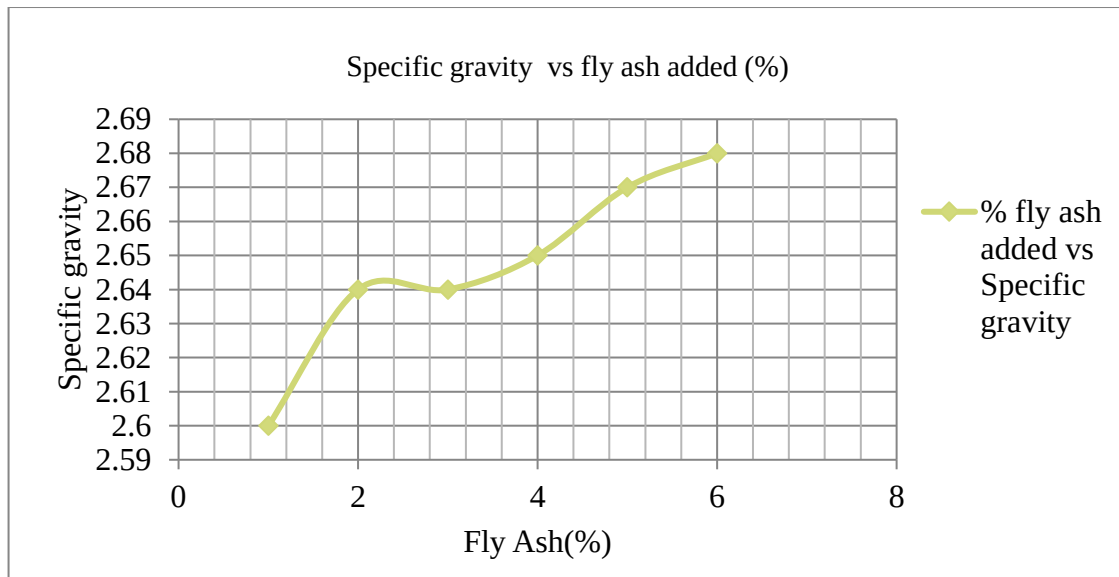


Figure 4. 11 Specific Gravity verses % of Fly Ash

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

The soaked CBR test was conducted according to ASTM D 1883 by taking the optimum moisture content for each per cent treatment of expansive soil (black cotton soil) from compaction result. It is observed that when the per cent of fly ash added increased, the value of CBR also increased, the reason was the chemical composition of the major constituents of MSWI ash are oxides of Calcium, Silicon, Iron and Aluminum and in addition it's pozzuolana property which impact to increase the strength of black cotton soil.

Table 4. 12 Summary of CBR (%) versus % of fly ash

S. No	% of Fly Ash Added	CBR (%)
1	0%	1.14
2	4%	2.5
3	8%	3.64
4	12%	6.00
5	16%	6.9
6	20%	8.05

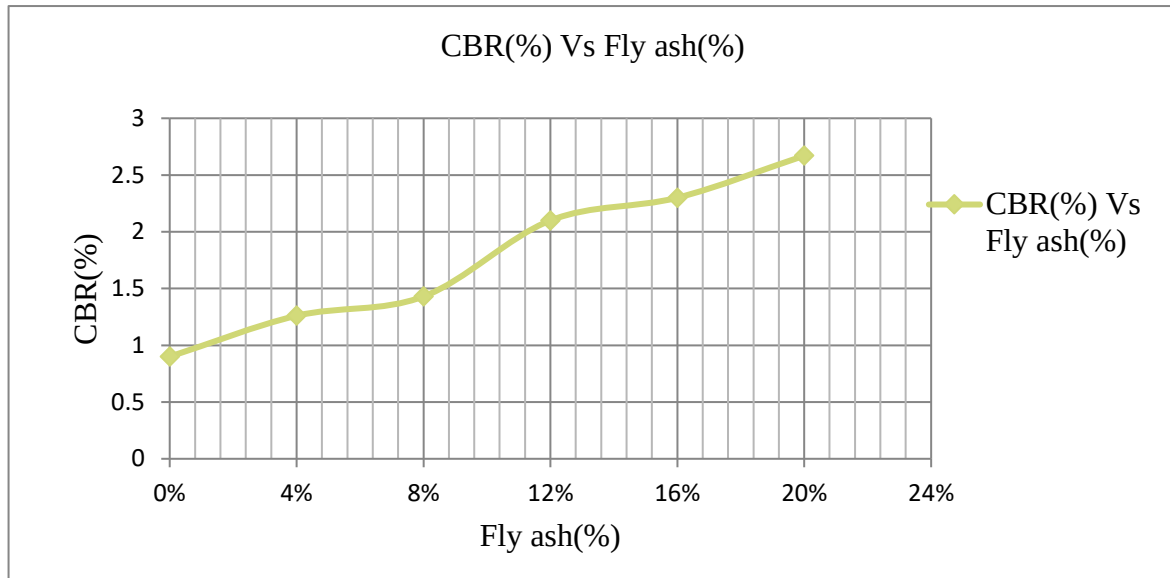


Figure 4. 12 CBR (%) versus % of fly ash

4.4.7 Result observed on unconfined compressive strength (UCS)

The test was performed by using the optimum moisture content that has observed from compaction test results for each treated and untreated soil samples which was done by replacement from the natural soil. The test was conducted according to IS 2720 (Part2): 1973 and the result showed that the unconfined compressive strength test increased as the addition of per cent of fly ash increased. The cause of increasing the unconfined compressive strength while increasing per cent of fly ash was its pozzolonic property.

Table 4. 13 UCS versus % of fly ash

NO	Fly Ash added	MDD	UCS
1	0	1.30	98.7
2	4	1.30	99.27
3	8	1.37	109.05
4	12	1.42	114.75
5	16	1.57	137.76
6	20	1.58	141.08

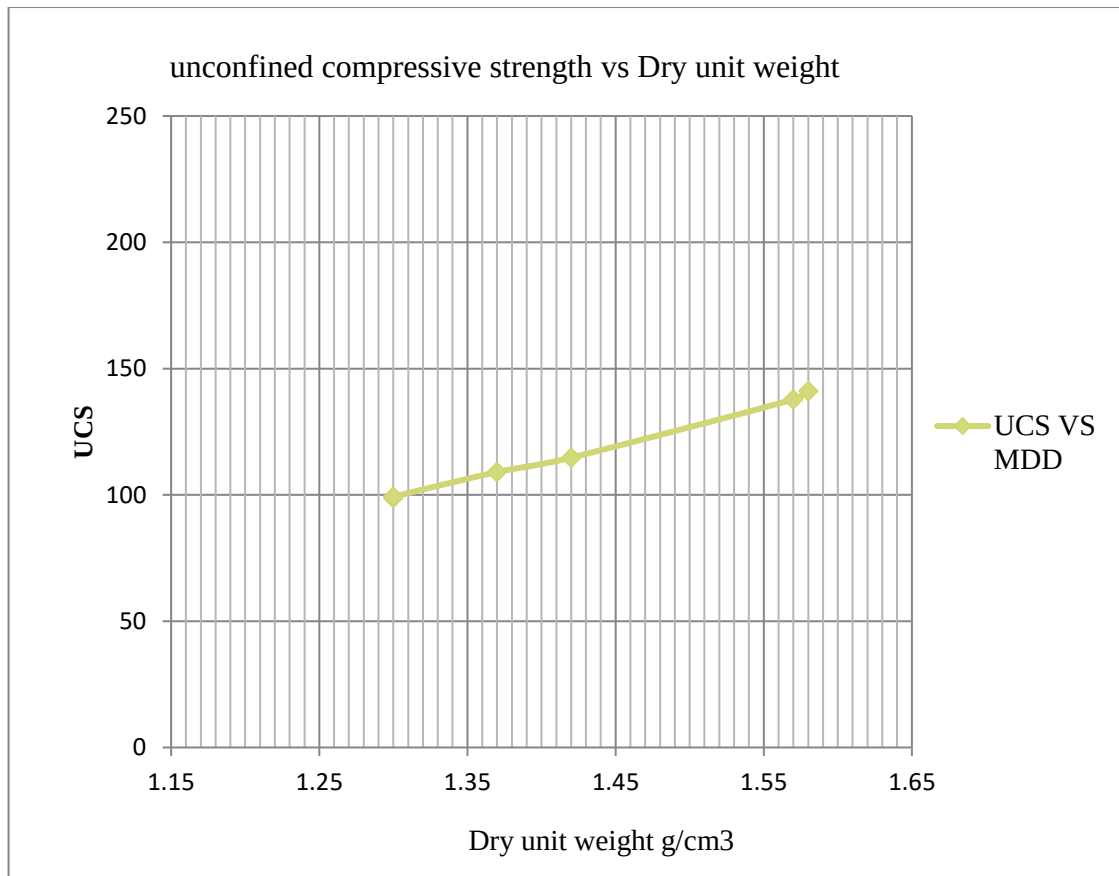


Figure 4. 13 UCS vs. Dry unit weight

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

UCS value of soil sample increases with the addition of fly ash content up to 20%. The maximum value of UCS is 141.08 kPa and 142.9kPa observed at 20% fly ash content for uncured and 7 days cured conditions respectively. The summary of the UCS test result for different percentages of fly ash is tabulated in Appendix H from Table H-1 to H-8 for uncured and 7 days cured samples. Generally, it can be expressed that the additions of fly ash expansive soils result in chemical reactions in the forms of cation exchange, pozzolanic reaction, and cementation. During cation exchange, flocculation of clay particles occurs giving rise to agglomeration into coarser particles helping resist the compressive stresses much more effectively in comparison with those of natural soil. In addition, the cementation process plays a key role in binding the clay particles together and consequently increasing the compressive stress.

Table 4. 14 UCS test results of soil with different fly ash percentage

% of Fly ash	UCS (kPa)(uncured)	Cu(kPa)(uncured)	UCS(kPa) (7 days cured)	Cu (kPa) (7days cured)	% Increase in uncured(cu)	% Increase 7days cured)(cu)
0	98.7	49.37	102.9	51.45	-	-
4	99.27	49.64	108.75	54.37	0.5	5.67
8	109.05	54.52	126.39	79.6	10.43	54.7
12	114.75	57.38	130.44	65.95	16.22	28.18
16	137.76	68.88	140	70	39.52	36.05
20	141.08	70.54	142.94	71.47	42.88	38.91

The substantial improvement in compressive strength could be attributed to cation exchange and pozzolanic reactions due to a high amount of alumina and silica available in fly ash. It is also seen that the unconfined compressive strength of treated expansive soil increased with increasing curing time as illustrated in Table 4.15. The compressive strength of treated soil increases significantly with curing time, which reveals the higher increase in additive content stabilized expansive soil, produced a larger impact on the compressive strength of treated soils. As shown in Figure 4.15 the curing time has a significant impact on the strength gain of stabilized soil.

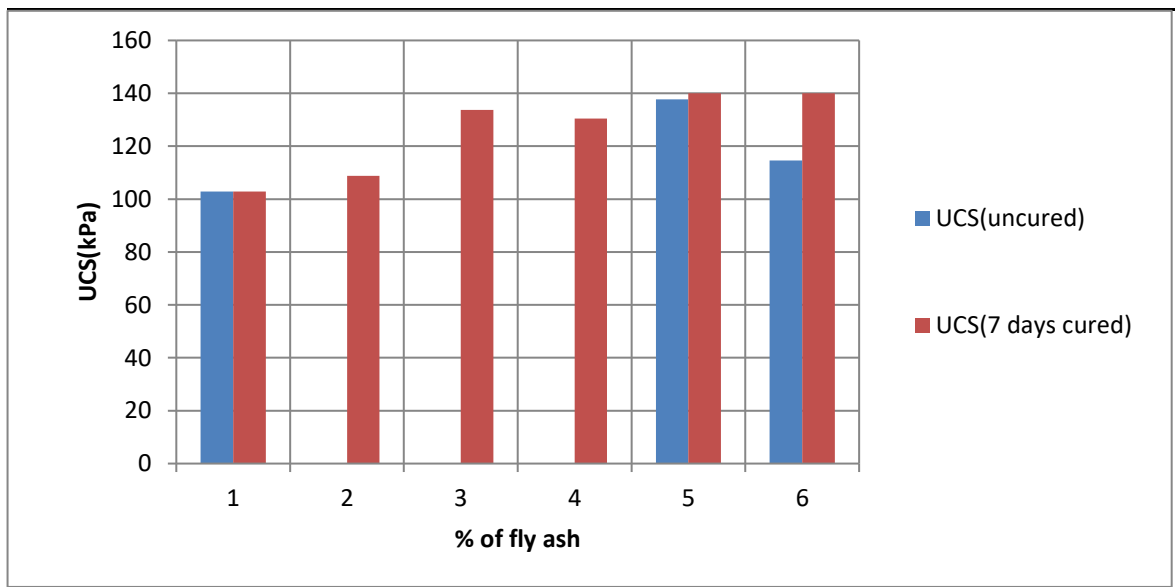


Figure 4. 14 Effect of curing time on stress-strain behavior of soil with different % of FA

Table 4. 15 Summary of laboratory test results

% of fly ash added	Liquid limit	Plastic Limit	Plastic index	OMC	MDD	Specific gravity	CBR	UCS
0	60.02	32.00	28.02	28.60	1.30	2.60	1.14	98.7
4	58.03	30.02	28.01	27.00	1.30	2.64	2.5	99.27
8	56.91	29.36	27.55	28.00	1.37	2.64	3.64	109.05
12	55.46	28.46	27	33.00	1.42	2.65	6.00	114.75
16	55.00	28.23	26.77	33.00	1.57	2.67	6.9	137.76
20	54.09	27.99	26.11	30.00	1.58	2.68	8.05	141.08

CONCLUSIONS

- ✓ The addition of fly ash showed a decrease in the plasticity index of expansive soil, Free swell, free swell index and free swell ratio also showed little decrement with an increasing amount of fly ash and OMC shows little increment with fly ash increment.
- ✓ The MDD, UCS, CBR and specific gravity were increased when the soil was mixed with fly ash. From the trial mixes addition of 20% fly ash gave a better result, when compared to the natural soil. The UCS at 20% fly ash stabilized soil increased by 141.08% and 142.92% for uncured and 7 days cured conditions respectively
- ✓ The coefficient of volume compressibility also decreases with increasing of fly ash used to stabilize the expansive soil.
- ✓ By considering the improvement of some geotechnical properties of expansive soil, this stabilizing material can be used for ground improvement.

RECOMMENDATIONS

- ✓ In this thesis the curing time of samples for UCS test was limited to 7 days so, Further study is recommended on the effect of curing time on fly ash stabilized soil.
- ✓ Adding of only fly ash of municipal waste by product to soil is not enough, finding other additional materials with fly ash is recommended in the future.
- ✓ The effect of kerosene coating on the strength properties of fly ash reinforced soil, and its effect on the environment should be studied.
- ✓ The utilization of fly ash of municipal waste by product in the strengthen of expansive soil (black cotton) should be further studied by increasing percentage added of fly ash.
- ✓ Fly ash with another stabilizing or additive materials like lime, cement, and other such materials can be used together, and may be varied in quantity to obtain the best possible stabilizing mixture.

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APPENDIXES

Appendix A: Natural Moisture Content

Table A. 1 Natural moisture content for BH1

Adama Science and Technology University Department Civil Engineering soil lab				
Source : Dukem Town				
Sample No:- BH-1 at 1.5 m				
Natural Moisture Content				
Test No 01				
No	particulars			
1	Container No	D	E	F
2	Weight of the cont. $W1(g)$	20	19	20
3	Weight of the cont+ wet soil $W2(g)$	89	113	103
4	Weight of the container + dry soil $W3(g)$	73	90	84
5	Weight of the moisture $Ww(g)=(w2-w3)$	16	23	19
6	Weight of dry soil $Ws(g)=(W3-W1)$	53	71	64
7	Moisture content $W(\%)$	30.19	32.39	29.69
	Average			30.76

Table A. 2 Natural moisture content for BH2

Source :-Dukem town				
Sample no:- BH-2 at 1.5 m				
Natural Moisture Content				
Test No 01				
No	particulars			
1	Container No	1	2	3
2	Weight of the cont. $W1(g)$	20	19	20
3	Weight of the cont+ wet soil $W2(g)$	85	106	99
4	Weight of the container + dry soil $W3(g)$	68	85	78
5	Weight of the moisture $Ww(g)=(w2-w3)$	17	21	21
6	Weight of dry soil $Ws(g)=(W3-W1)$	48	66	58
7	Moisture content $W(\%)$	35.42	31.82	36.21
	Average		34.48	

Table A. 3 Natural moisture content for BH3

Source :-Dukem town

Sample no:- BH -3 at 1.50 m

Natural Moisture Content				
Test No 01				
No	Particulars			
1	Container No	1	2	3
2	Weight of the cont. W_1 (g)	20	18	19
3	Weight of the cont+ wet soil W_2 (g)	85	89	82
4	Weight of the container + dry soil W_3 (g)	70	71	66
5	Weight of the moisture W_w (g)=(w_2-w_3)	15	18	16
6	Weight of dry soil W_s (g)=(W_3-W_1)	50	53	47
7	Moisture content W (%)	30.00	33.96	34.04
	Average	32.67		

Appendix B: Liquid limit and Plastic limit test results

Table B. 1 Liquid Limit WL @0% fly ash

Liquid Limit W_L			
Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	29	28	29
B. Weight of Dry Soil + Tare	25.221	23.83	24.6
C. Weight of Water (A – B)	3.779	4.17	4.4
D. Weight of Tare	18	17	18
E. Weight of Dry Soil (B -D)	7.221	6.83	6.6
Water Content % (C / E x 100)	52.33	61.05	66.67
Number of Blows	31	21	15
Liquid Limit %	60.02		

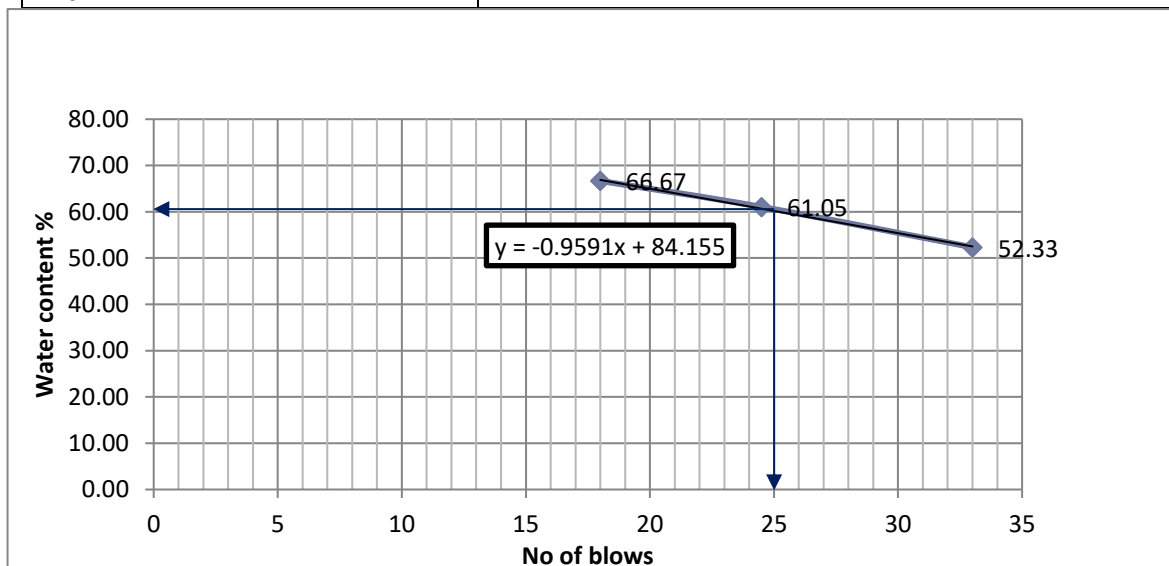


Figure 1.1 Liquid Limit at 0% of fly ash or pure soil

Table B. 2 Liquid Limit WL @4% of fly ash

Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	28.9	29.45	31.51
B. Weight of Dry Soil + Tare	25.8	25.9	26.45
C. Weight of Water (A – B)	3.1	3.55	5.06
D. Weight of Tare	19.09	19.77	19.22
E. Weight of Dry Soil (B -D)	6.71	6.13	7.23
Water Content % (C / E x 100)	46.20	57.91	69.99
Number of Blows	34	26	18
Liquid Limit %			

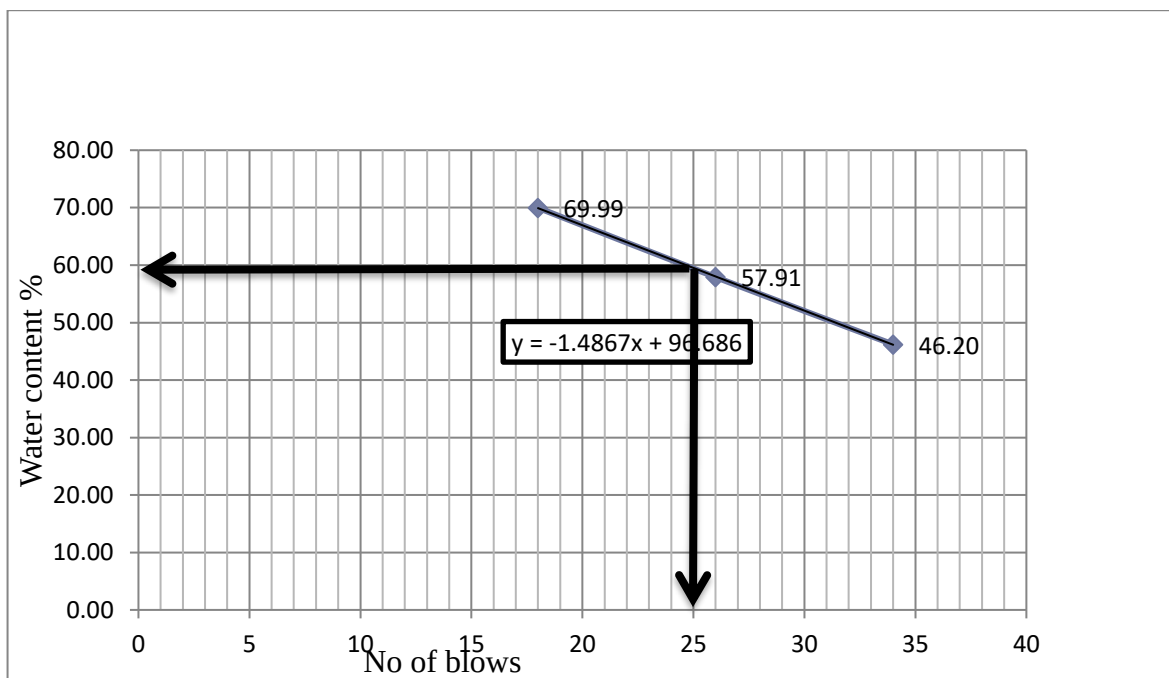


Figure 1.2 Liquid Limit at 4% of fly ash

Table B. 3 Liquid Limit WL @8% of fly ash

Liquid Limit WL			
Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	28.56	26.09	27.14
B. Weight of Dry Soil + Tare	25.4	23.58	24.25
C. Weight of Water (A – B)	3.16	2.51	2.89
D. Weight of Tare	19.42	19.2	19.48
E. Weight of Dry Soil (B -D)	5.98	4.38	4.77
Water Content % (C / E x 100)	52.84	57.31	60.59
Number of Blows	30	23	18
Liquid Limit %	56.91		

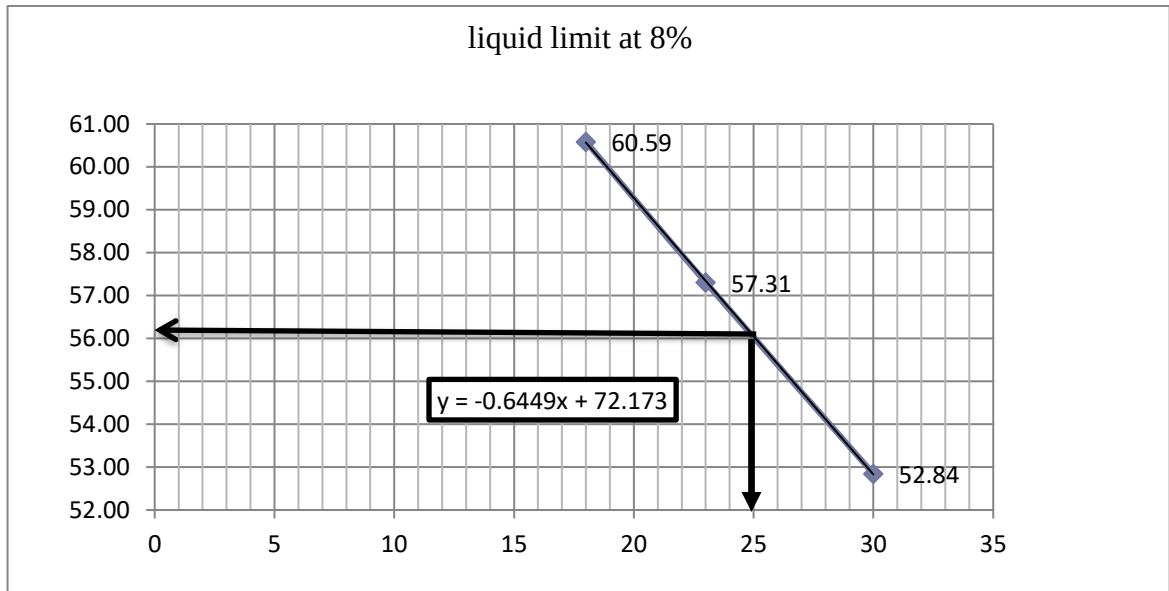


Figure 1.3 Liquid Limit at 8% of fly ash

Table B. 4 Liquid Limit WL @12% of fly ash

Liquid Limit W_L			
Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	31.01	32.55	27.84
B. Weight of Dry Soil + Tare	27.38	28.07	24.3
C. Weight of Water (A – B)	3.63	4.48	3.54
D. Weight of Tare	19.67	19.54	19
E. Weight of Dry Soil (B -D)	7.71	8.53	5.3
Water Content % (C / E x 100)	47.08	52.52	66.79
Number of Blows	30	26	16
Liquid Limit %	55.46		

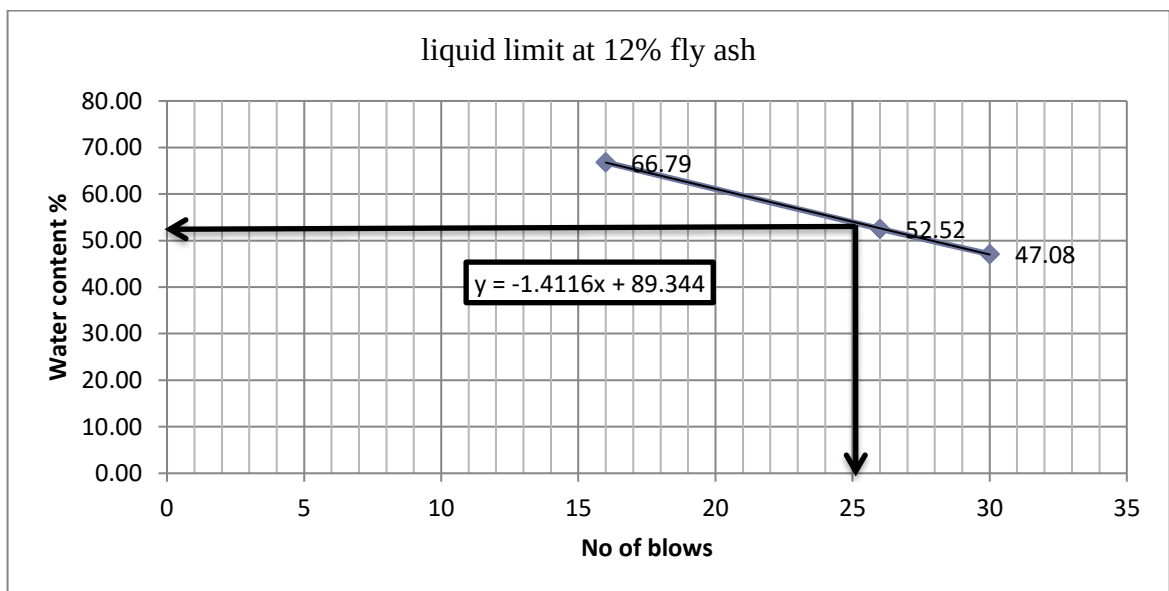


Figure 1.4 Liquid Limit at 16% of fly ash

Table B. 5 Liquid Limit WL @16 % of fly ash

Liquid Limit W _L			
Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	33.03	29.82	31.76
B. Weight of Dry Soil + Tare	28.58	26.21	27.149
C. Weight of Water (A – B)	4.45	3.61	4.611
D. Weight of Tare	19.58	19.65	19.53
E. Weight of Dry Soil (B -D)	9	6.56	7.619
Water Content % (C / E x 100)	49.44	55.03	60.52
Number of Blows	30	23	16
Liquid Limit %	55.00		

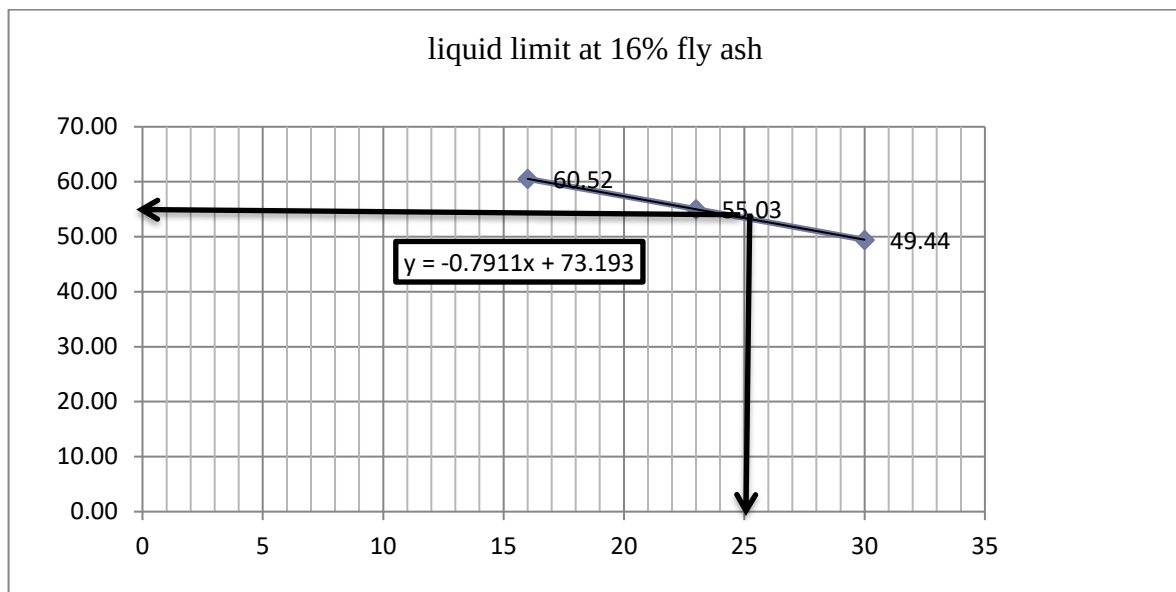


Figure 1.5 Liquid Limit at 16% of fly ash

Table B. 6 Liquid Limit WL @20 % of fly ash

Liquid Limit W _L			
Run Number			
Tare Number	A1	A2	A3
A. Weight of Wet Soil + Tare	29.58	30.12	30.24
B. Weight of Dry Soil + Tare	27.356	26.43	25.4
C. Weight of Water (A – B)	2.224	3.69	4.84
D. Weight of Tare	19.6	19.58	19.33
E. Weight of Dry Soil (B -D)	7.756	6.85	6.07
Water Content % (C / E x 100)	28.67	53.87	79.74
Number of Blows	31	24	17
Liquid Limit %	54.09		

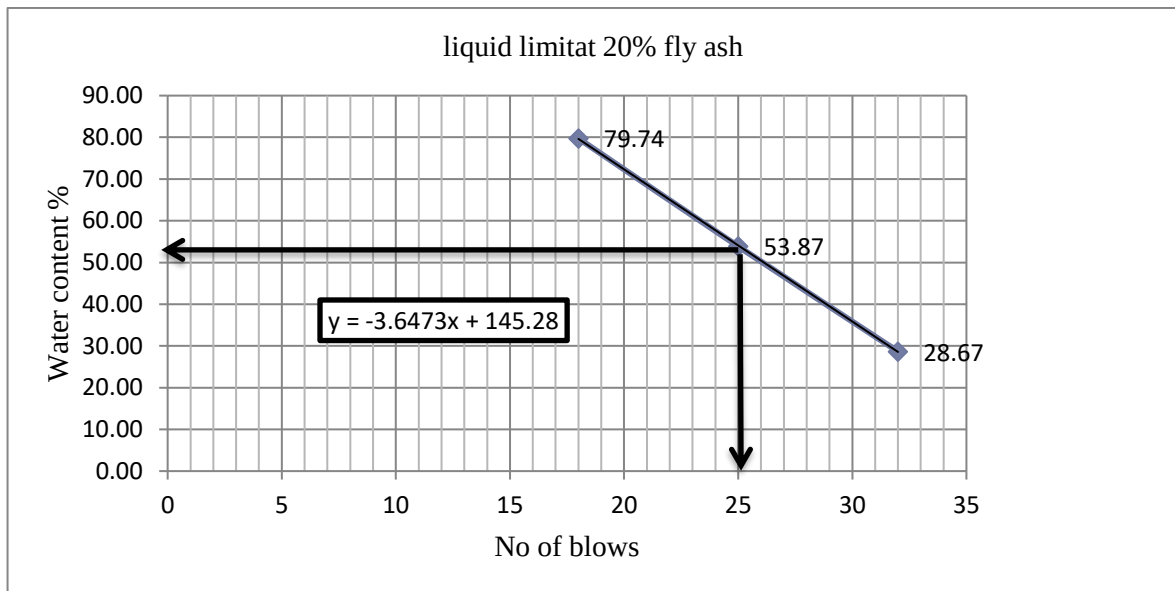


Figure 1.6 Liquid Limit at 20% of fly ash

Table B. 7 Plastic Limit @pure or 0% fly ash

Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	35.00	36.00	32
G. Weight of Dry soil + Tare	31.2	32	28.245
H. Weight of water (F – G)	3.80	4.00	3.755
I. Weight of Tare	17.00	19.00	18
J. Weight of Dry soil (G – I)	14.2	13.00	10.245
Water Content % (H - J x 100)	26.76	30.77	36.65
Plastic Limit % (Average)	31.39		

Plasticity Index ($PI = W_L - W_P$) = 28.62

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11
Diameter f mm	2.00	0.425	0.075	Silt Soils
% Passing	62.07	31.03	1.15	

GROUP INDEX = $(F - 35) [0.2 + 0.005 (LL - 40)] + 0.01 (F - 15) (PI - 10)$

Where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

Table B. 8 Plastic Limit @4% fly ash

Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	28.18	30.7	26.65
G. Weight of Dry soil + Tare	26.28	28.2	24.8
H. Weight of water (F – G)	1.90	2.53	1.85
I. Weight of Tare	19.63	19.70	18.97
J. Weight of Dry soil (G – I)	6.65	8.50	5.83
Water Content % (H - J x 100)	28.57	29.76	31.73
Plastic Limit % (Average)	30.02		

Plasticity Index ($PI = W_L - W_P$) =28.01

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11
Diameter f mm	2.00	0.425	0.075	Silt Soils
% Passing	62.50	29.35	0.54	

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15) (PI – 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

Plastic Limit W_P			
Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	27.93	31.04	32.29
G. Weight of Dry soil + Tare	25.99	28.5	29.3
H. Weight of water (F – G)	1.94	2.54	2.99
I. Weight of Tare	19.43	19.49	19.44
J. Weight of Dry soil (G – I)	6.56	9.01	9.86
Water Content % (H - J x 100)	29.57	28.19	30.32
Plastic Limit % (Average)	29.36		

Plasticity Index ($PI = W_L - W_P$) = 27.55

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11
Diameter f mm	2.00	0.425	0.075	Silt Soils
% Passing	62.07	31.03	1.15	

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15)
(PI – 10)

Where: F = Percentage passing 0.075 mm (No. 200) sieve.

Table B. 10 Plastic Limit @12% fly ash

Plastic Limit W_P			
Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	29.06	33.62	28.07
G. Weight of Dry soil + Tare	26.98	30.45	26.2
H. Weight of water (F – G)	2.08	3.17	1.87
I. Weight of Tare	19.47	19.59	19.64
J. Weight of Dry soil (G – I)	7.51	10.86	6.56
Water Content % (H - J x 100)	27.70	29.19	28.51
Plastic Limit % (Average)	28.46		

Plasticity Index ($PI = W_L - W_P$) = 27.00

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11
Diameter f mm	2.00	0.425	0.075	Silt Soils
% Passing	62.07	31.03	1.15	

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15)
(PI – 10)

Where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity

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Table B. 11 Plastic Limit @16%fly ash

Plastic Limit W_P			
Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	28.91	28.97	27.09
G. Weight of Dry soil + Tare	26.7	26.68	25.76
H. Weight of water (F – G)	2.21	2.29	1.33
I. Weight of Tare	19.71	19.37	19.64
J. Weight of Dry soil (G – I)	6.99	7.31	6.12
Water Content % (H - J x 100)	31.62	31.33	21.73
Plastic Limit % (Average)	28.23		

Plasticity Index ($PI = W_L - W_P$) = **26.77**

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11
Diameter f mm	2.00	0.425	0.075	Silt Soils
% Passing	62.07	31.03	1.15	

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F – 15)
(PI – 10)

Where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity

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Table B. 12 Plastic Limit @20% fly ash

Plastic Limit W_p			
Run Number			
Tare Number	P1	B	C
F. Weight of Wet soil + Tare	29.73	26.52	25.65
G. Weight of Dry soil + Tare	27.85	24.96	24.22
H. Weight of water (F – G)	1.88	1.56	1.43
I. Weight of Tare	19.39	19.61	19.83
J. Weight of Dry soil (G – I)	8.46	5.35	4.39
Water Content % (H - J x 100)	22.22	29.16	32.57
Plastic Limit % (Average)	27.99		

Plasticity Index ($PI = W_L - W_p$) = 26.11

Grading				
ASTM SIEVE No.	10	40	200	ASTM D 2487-11 Silt Soils
Diameter f mm	2.00	0.425	0.075	
% Passing	62.07	31.03	1.15	

GROUP INDEX = (F - 35) [0.2 + 0.005 (LL - 40)] + 0.01 (F - 15) (PI - 10)

where: F = Percentage passing 0.075 mm (No. 200) sieve.

LL = Liquid Limit & PI = Plasticity Index

Table B. 13 Summary of Liquid limit of soil after mixing different percentage of fly ash.

CONTENT	Liquid limit%
Black cotton soil pure or 0%	60.02
4% fly ash	58.03
8% fly ash	56.91
12% fly ash	55.46
16% fly ash	55.00
20% fly ash	54.09

Here results can be seen that with addition of fly ash up to 16% in soil sample, liquid limit value decreases from 60.02% to 55.00%.

Table B. 14 Summary of Plastic limit of soil after mixing different percentage of fly ash.

Content	Plastic Limit (%)
Pure Black Cotton Soil (0%)	32.00
4%	30.02
8%	29.36
12%	28.46
16%	28.10
20%	27.99

Here results can be seen that with addition of fly ash up to 16% in soil sample, plastic limit value decreases from 32% to 28.10%.

Table B. 15 Plasticity index after mixing with different percentage of fly ash

Fly ash added	Plasticity Index (%)
Pure Black Cotton Soil (0%)	28.02
4%	28.01
8%	27.55
12%	27
16%	26.90
20%	26.11

Table C. 1 Particle Size Distribution at BH1

PARTICLE SIZE ANALYSIS					
Source: <u>Dukem Town</u>			Wgt. Before Washing (g) : 500		
Sample No. : BH-1 at 1.50			Wgt. After Washing (g):		
Test No: 01			Material Type: _____		
			Date: _____		
SIEVE ANALYSIS					
☒ Wet Method ☐ Dry Method					
Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % (WR/TW*100)	Passing % ((Tw-εwr)/Tw)*100	Spec's Requirements
3"	75.00				Graval
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				
No.4	4.75	1.0000	0.20	99.80	Course Sand
No.8	2.36	2.0000	0.40	99.40	
No.10	2.00	0.0000	0.00	99.40	
No.16	1.18	2.0000	0.40	99.00	Medium Sand
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	3.0000	0.60	98.40	
No.40	0.425	0.0000	0.00	98.40	
No.50	0.300	4.0000	0.80	97.60	Fine Sand
No.100	0.150	4.0000	0.80	96.80	
No.200	0.075	2.0000	0.40	96.40	
Pan		482.0000	96.40	0	
Total Weight (g)		500.0000			
Loss (g)					

Appendix C: Particle Size Distribution

Table C. 2 particle size distribution at BH2

PARTICLE SIZE ANALYSIS					
Source: <u>Dukem Town</u>		Wgt. Before Washing (g): <u>500</u>			
Sample No.: <u>BH 2 at 1.5m</u>		Wgt. After Washing (g): <u>484</u>			
Test No.: <u>01</u>		Material Type: _____			
		Date: _____			
SIEVE ANALYSIS					
☒ Wet Method		☐ Dry Method			
Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % (WR/TW*100)	Passing % ((Tw-εwr)/Tw)*100	Spec's Requirements
3"	75.00				
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				
No.4	4.75	1.0000	0.20	99.80	
No.8	2.36	2.0000	0.40	99.40	
No.10	2.00	1.0000	0.20	99.20	
No.16	1.18	1.0000	0.20	99.00	
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	0.0000	0.00	99.00	
No.40	0.425	0.0000	0.00	99.00	
No.50	0.300	2.0000	0.40	98.60	
No.100	0.150	4.0000	0.80	97.80	
No.200	0.075	5.0000	1.00	96.80	
Pan		484.0000	96.80	0.00	
Total Weight (g)		500.0000			
Loss (g)		0	100.0		

PARTICLE SIZE ANALYSIS

Source : Dukem Town

Wgt. Before Washing (g): 500

Sample No. : BH 3 at 1.50m

Wgt. After Washing (g): 466

Test No.: 01

Material Type:

Date:

SIEVE ANALYSIS



Wet Method



Dry Method

Sieve ASTM	Sieve Size (mm)	Weight of grains Retained (g)	Retained % (WR/TW*100)	Passing % ((Tw-εwr)/Tw)*100	Spec's Requirements
3"	75.00				
2"	50.00				
1-1/2"	37.50				
1"	25.00				
3/4"	19.00				
1/2"	12.50				
3/8"	9.50				
-	5.00				
No.4	4.75	0.0000	0.00	100.00	
No.8	2.36	1.0000	0.20	99.80	
No.10	2.00	1.0000	0.20	99.60	
No.16	1.18	3.0000	0.60	99.00	
No.18	1.00	0.0000	0.00	99.00	
No.30	0.600	4.0000	0.80	98.20	
No.40	0.425	0.0000	0.00	98.20	
No.50	0.300	6.0000	1.20	97.00	
No.100	0.150	10.0000	2.00	95.00	
No.200	0.075	9.0000	1.80	93.20	
Pan		466.0000	93.20	0	
Total Weight (g)		500.0000			
Loss (g)		0	100.0		

Appendix D: Soil Classification

Table D. 1 AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282)

General classification	Granular materials (35 percent or less of total sample passing No,200						Silt –clay materials (more than 35 percent of total samples passing No,200)				
Group classification	A-1		A3	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	50max	51 min								
No.200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	35 max	35 max	35 max	35 max
Characteristics of fraction passing No.40											
liquid limit											
plastic index	6 max		N.P	40max 40 min 10 max 10max		40 max 41 min 11 min 11 max		40 max 41 min 10 max 10 max		40 max 41 min 11 min 11 min	
Usual types of significant constituent	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							

Appendix E: Specific Gravity Determination

Table E. 1 Specific Gravity Determination

		Sample No	
S/N	Particulars	T1	T2
1	Weight of Density bottle W_1 (g)	48	47.9
2	Weight of Density bottle with dry soil W_2 (g)	67	65.7
3	Weight of Density bottle with dry soil and water W_3 (g)	157.892	157.324
4	Weight of Density bottle full of water W_4 (g)	146.13	146.13
5	Weight of dry soil (W_2-W_1) (g)	19	17.8
6	Weight of an equal volume of water(W_2-W_1)-(W_3-W_4)	7.238	6.606
7	Specific gravity of the soil $G= (5/6)$	2.63	2.69

Appendix F: Moisture Content Determination

Table F. 1 Natural Moisture Content Determination at BH1

Moisture -Density Relation of Soil AASHTO T180 Methode													
Tested by :- Sime Wadajo							Date Sampled						
Station :- Dukem Town (BH-1)							Date Tested						
Depth:- 1.50M							Sample of						
Material Type :- Black Cotton													
mass of Compacted soil +mold (g)	mass of mold+baseplate (g)	Mass of Compacted Soil (g)	Volume of mold (cm3)	Wet unit weight (kn/m3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)
5093	3729	1364	1000	1.364	1	66.29	63	3.29	18.63	44.37	7.4149	24.27	1.10
					2	71.71	66	5.71	19.36	46.64	12.243		
					3	84.59	62	22.59	19.5	42.50	53.153		
5102	3729	1373	1000	1.373	1	66.93	66.38	0.55	19.18	47.20	1.1653	24.21	1.11
					2	79.06	62	17.06	19.49	42.51	40.132		
					3	77.27	63.5	13.77	19.56	43.94	31.34		
5119	3729	1390	1000	1.39	1	71.09	60	11.09	18.93	41.07	27.00	24.83	1.11
					2	66.71	60.5	6.21	18.87	41.63	14.917		
					3	75.84	62	13.84	19.52	42.48	32.58		
5148	3729	1419	1000	1.419	1	57.37	49.42	7.95	19.77	29.65	26.81	25.97	1.13
					2	50.56	44.29	6.27	19.73	24.56	25.529		
					3	65.13	55.82	9.31	19.42	36.40	25.577		
5310	3729	1581	1000	1.581	1	53.53	45.24	8.29	19.49	25.75	32.194	31.11	1.21
					2	53.83	45.85	7.98	19.45	26.40	30.227		
					3	71.75	59.41	12.34	19.48	39.93	30.904		
5412	3729	1683	1000	1.683	1	56.12	47.03	9.09	19.65	27.38	33.199	35.55	1.24
					2	60.24	50.09	10.15	19.39	30.70	33.06		
					3	83.41	65	18.41	19.41	45.59	40.382		
5444	3729	1715	1000	1.715	1	61.48	50.16	11.32	19.48	30.68	36.897	39.73	1.23
					2	62.26	50.8	11.46	19.52	31.28	36.637		
					3	71.23	55	16.23	19.46	35.54	45.667		
5429	3729	1700	1000	1.700	1	72.06	56.4	15.66	19.61	36.79	42.566	42.60	1.19
					2	72.48	56.53	15.95	19.22	37.31	42.750		
					3	58.76	47.08	11.68	19.59	27.49	42.488		

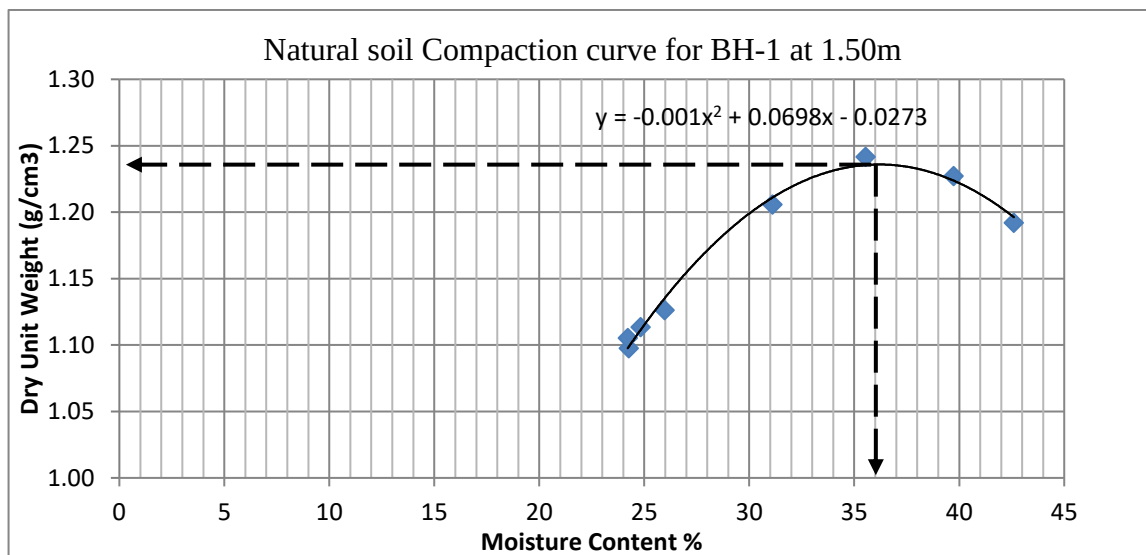


Figure F. 1 Natural Moisture Content Determination at BH1

Table F. 2 Natural Moisture Content Determination at BH2

Moisture -Density Relation of Soil AASHTO T180 Methode													
Tested by :- Sime Wadajo							Date Sampled						
Station :- Dukem Town (BH-2)							Date Tested						
Depth:- 1.50M							Sample of						
Material Type :- Black Cotton													
mass of Compacted soil +mold (g)	mass of mold+baseplate (g)	Mass of Compacted Soil (g)	Volume of mold (cm3)	Wet unit weight (kn/m3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)
5076	3755	1321	997.46	1.3244	A	57	51	6	20	31	19.35	17.35	1.13
					B	57	52	5	20	32	15.63		
					C	67	60	7	19	41	17.07		
5164	3755	1409	997.46	1.4126	D	65	57	8	18	39	20.51	18.50	1.19
					E	61	54	7	19	35	20.00		
					F	63	57	6	17	40	15.00		
5281	3755	1526	997.46	1.5299	G	53	46	7	18	28	25.00	21.15	1.26
					H	37	37	0	20	17	0.00		
					I	72	57	15	18	39	38.46		
5467	3755	1712	997.46	1.7164	J	55	49	6	19	30	20.00	25.07	1.37
					K	75	61	14	18	43	32.56		
					L	83	71	12	18	53	22.64		
5526	3755	1771	997.46	1.7755	M	62	55	7	18	37	18.92	27.50	1.39
					N	81	69	12	19	50	24.00		
					O	85	66	19	18	48	39.58		
5418	3755	1663	997.46	1.6672	p	91	69	22	18	51	43.14	40.36	1.19
					Q	80	60	20	17	43	46.51		
					R	64	53	11	18	35	31.43		

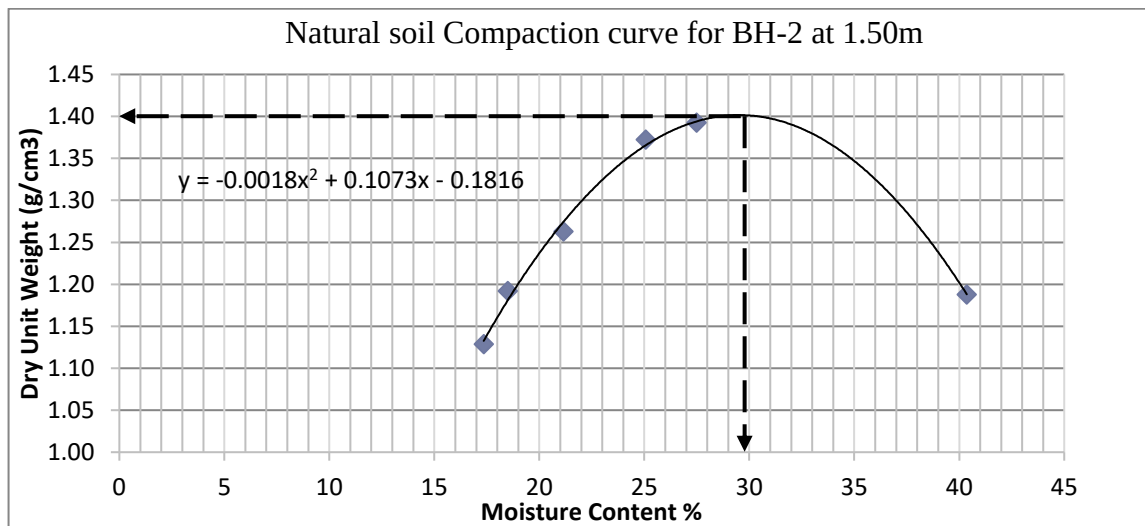


Figure F. 2 Natural Moisture Content Determination at BH2

Table F. 3 Natural Moisture Content Determination at BH3

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo						Date Sampled								
Station :- Dukem Town (BH-3)						Date Tested								
Depth:- 1.50M						Sample of								
Material Type :- Black Cotton														
mass of Compacted soil +mold (g)	mass of mold+basep late (g)	Mass of Compacte d Soil (g)	Volume of mold (cm3)	Wet unit weight (kn/m3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)	
5081	3755	1326	997.46	1.3294	A	57	52	5	20	32	15.625	14.01	1.17	
					B	57	53	4	20	33	12.121			
					C	67	61	6	19	42	14.286			
5172	3755	1417	997.46	1.4206	D	65	59	6	18	41	14.634	15.43	1.23	
					E	61	55	6	19	36	16.667			
					F	63	57	6	17	40	15			
5276	3755	1521	997.46	1.5249	G	53	47	6	18	29	20.69	18.01	1.29	
					H	37	35	2	20	15	13.333			
					I	72	63	9	18	45	20			
5460	3755	1705	997.46	1.7093	J	55	49	6	19	30	20	25.07	1.37	
					K	75	61	14	18	43	32.558			
					L	83	71	12	18	53	22.642			
5521	3755	1766	997.46	1.7705	M	62	54	8	18	36	22.222	28.60	1.38	
					N	81	69	12	19	50	24			
					O	85	66	19	18	48	39.583			
5411	3755	1656	997.46	1.6602	P	91	69	22	18	51	43.137	40.36	1.18	
					Q	80	60	20	17	43	46.512			
					R	64	53	11	18	35	31.429			

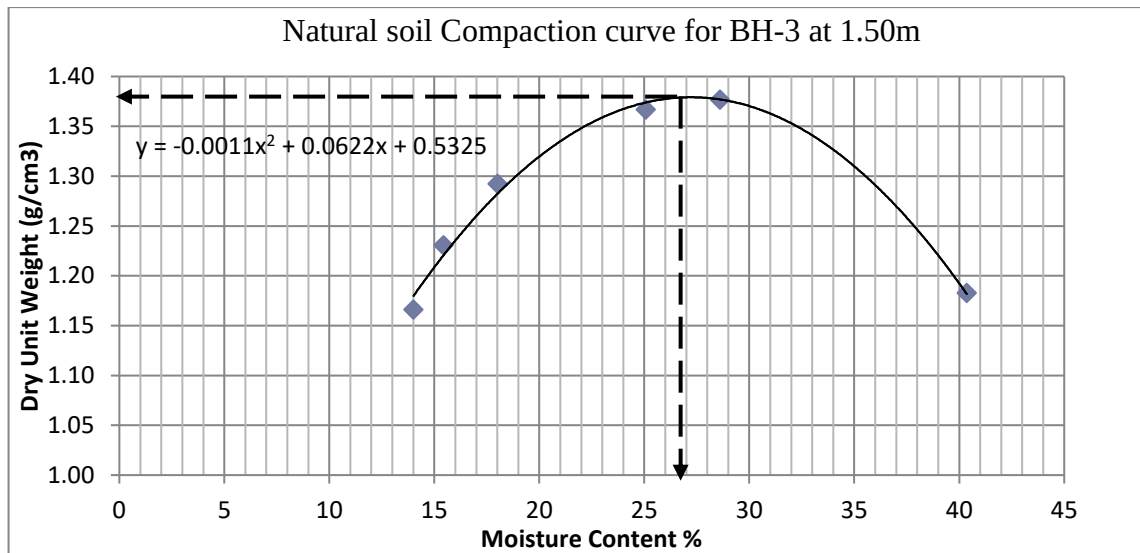


Figure F. 3 Natural Moisture Content Determination at BH3

Table F. 4 Moisture -Density Relation at soil+4% fly ash

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo							Date Sampled							
Station :- Dukem Town							Date Tested							
Depth:- BH-1 1.50M							Sample of							
Material Type :- Black Cotton														
Added 4% Fly Ash														
Trial No	mass of Compacte d soil +mold (g)	mass of mold+base plate (g)	Mass of Compacte d Soil (g)	Volume of mold (cm3)	Wet unit weight (g/cm3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)
1	5100	3756	1344	997.46	1.347422	Fly-1	43.28	38.6	4.68	19.1	19.5	24	17.87	1.14
	0	0		997.46		Fly-2	53.83	49.2	4.63	19.52	29.68	15.59973		
	0	0		997.46		Fly-3	70.25	64	6.25	19.36	44.64	14.0009		
2	5300	3756	1544	997.46	1.547932	Fly-4	36.55	33.5	3.05	19.24	14.26	21.3885	21.73	1.27
	0	0		997.46		Fly-5	46.42	42.86	3.56	19.54	23.32	15.26587		
	0	0		997.46		Fly-6	39.72	35.23	4.49	19.5	15.73	28.54418		
3	5397	3756	1641	997.46	1.645179	Fly-7	38.13	35.3	2.83	19.31	15.99	17.69856	23.58	1.33
	0	0		997.46		Fly-8	54.05	49.28	4.77	19.78	29.5	16.16949		
	0	0		997.46		Fly-9	42.25	36.15	6.1	19.61	16.54	36.88029		
4	5511	3756	1755	997.46	1.759469	Fly-10	45.33	40.25	5.08	19.65	20.6	24.66019	30.02	1.35
	0	0		997.46		Fly-11	51.55	43.64	7.91	19.49	24.15	32.75362		
	0	0		997.46		Fly-12	65.87	54.39	11.48	19.23	35.16	32.65074		
5	5410	3756	1654	997.46	1.658212	Fly-13	44.49	37.77	6.72	19.69	18.08	37.16814	33.68	1.24
	0	0		997.46		Fly-14	46.21	38.67	7.54	19.32	19.35	38.96641		
	0	0		997.46		Fly-15	51.62	45.27	6.35	19.76	25.51	24.8922		
6	5300	3756	1544	997.46	1.547932	Fly-16	48.47	39.62	8.85	19.33	20.29	43.61755	36.35	1.14
	0	0		997.46		Fly-17	54.2	44.51	9.69	19.64	24.87	38.96261		
	0	0		997.46		Fly-18	49.84	43.5	6.34	19.56	23.94	26.48287		

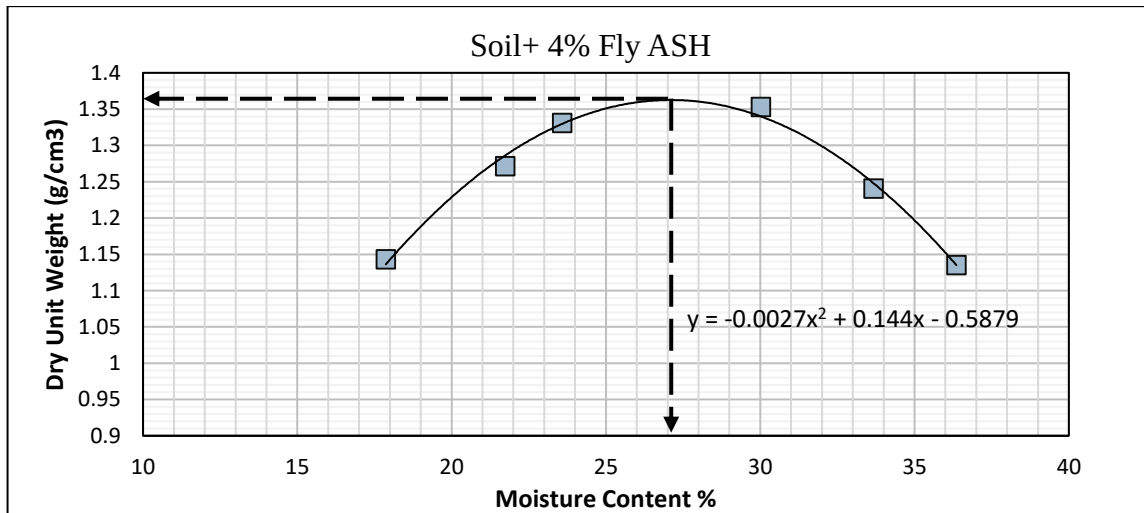


Figure F. 4 Moisture -Density Relation at soil+4% fly ash

Table F. 5 Moisture -Density Relation at soil+8% fly ash

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo							Date Sampled							
Station :- Dukem Town							Date Tested							
Depth:- BH-1 1.50M							Sample of							
Material Type :- Black Cotton														
Added 8% fly ash														
Trial No	mass of Compact ed soil +mold (g)	mass of mold+bas eplate (g)	Mass of Compact ed Soil (g)	Volume of mold (cm3)	Wet unit weight (kn/m3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (kn/m3)
1	5300	3756	1544	997.46	1.547932	Fly-1	42.8	40.1	2.7	18.75	21.35	12.64637	17.00	1.32
				997.46		Fly-2	48.8	46.3	2.5	18.54	27.76	9.005764		
				997.46		Fly-3	36.99	33	3.99	19.4	13.6	29.33824		
2	5350	3756	1594	997.46	1.598059	Fly-4	34.99	33	1.99	19.3	13.7	14.52555	18.08	1.35
				997.46		Fly-5	42.89	39	3.89	19.5	19.5	19.94872		
				997.46		Fly-6	45.86	41.5	4.36	19.43	22.07	19.75532		
3	5430	3756	1674	997.46	1.678263	Fly-7	43.89	39.6	4.29	18.86	20.74	20.68467	21.37	1.38
				997.46		Fly-8	45.6	40.9	4.7	19.38	21.52	21.84015		
				997.46		Fly-9	35.94	33.02	2.92	19.49	13.53	21.58167		
4	5500	3756	1744	997.46	1.748441	Fly-10	39.97	36.98	2.99	19.27	17.71	16.88312	25.20	1.40
				997.46		Fly-11	45.98	39.98	6	19.33	20.65	29.05569		
				997.46		Fly-12	40.97	35.98	4.99	19.16	16.82	29.66706		
5	5498	3756	1742	997.46	1.746436	Fly-13	51.1	43.36	7.74	19.45	23.91	32.37139	33.52	1.31
				997.46		Fly-14	53.46	44.82	8.64	19.65	25.17	34.32658		
				997.46		Fly-15	46.51	39.64	6.87	19.36	20.28	33.87574		
6	5398	3756	1642	997.46	1.646181	Fly-16	42.14	35.7	6.44	19.54	16.16	39.85149	40.19	1.17
				997.46		Fly-17	46.09	38.48	7.61	19.44	19.04	39.96849		
				997.46		Fly-18	68.95	53.91	15.04	17	36.91	40.74776		

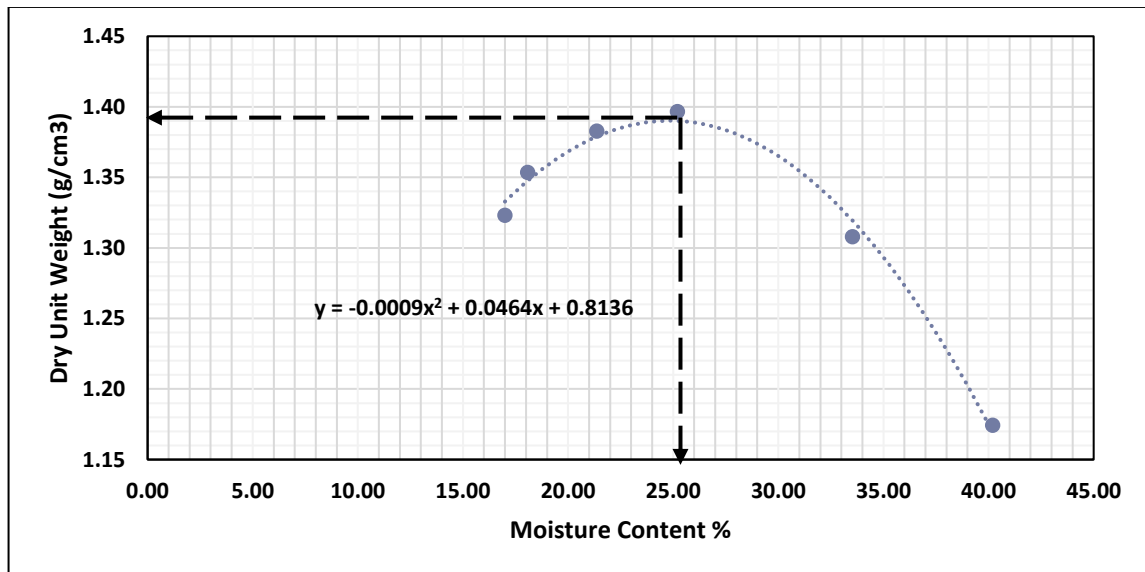


Figure F. 5 Moisture -Density Relation at soil+8% fly ash

Table F. 6 Moisture -Density Relation at soil+12% fly ash

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo								Date Sampled						
Station :- Dukem Town								Date Tested						
Depth:- BH-1 1.50M								Sample of						
Material Type :- Black Cotton														
Added 12% fly ash														
Trial No	mass of Compact ed soil +mold (g)	mass of mold+bas eplate (g)	Mass of Compact ed Soil (g)	Volume of mold (cm3)	Wet unit weight (g/cm3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)
1	5057	3756	1301	997.46	1.304313	F ₁	51.93	48.5	3.43	19.32	29.18	11.75463	16.59	1.12
						F ₂	43.4	40	3.4	19.44	20.56	16.53696		
						F ₃	40.74	37	3.74	19.58	17.42	21.46958		
2	5350	3756	1594	997.46	1.598059	F ₄	43.87	40	3.87	19.21	20.79	18.61472	21.19	1.32
						F ₅	39.99	36	3.99	19.73	16.27	24.52366		
						F ₆	46.95	42.3	4.65	19.55	22.75	20.43956		
3	5500	3756	1744	997.46	1.748441	F ₇	44.23	38.5	5.73	19.69	18.81	30.46252	26.81	1.38
						F ₈	45.43	39.84	5.59	19.47	20.37	27.44232		
						F ₉	56.85	50	6.85	19.6	30.4	22.53289		
4	5550	3756	1794	997.46	1.798568	F ₁₀	51.14	43.99	7.15	19	24.99	28.61144	27.61	1.41
						F ₁₁	64.15	53.999	10.151	19	34.999	29.00369		
						F ₁₂	54.05	46.99	7.06	19	27.99	25.22329		
5	5495	3756	1739	997.46	1.743428	F ₁₃	53.34	43	10.34	19.22	23.78	43.48192	40.02	1.25
						F ₁₄	67	53.55	13.45	18.5	35.05	38.37375		
						F ₁₅	60.81	49.4	11.41	19.54	29.86	38.21165		
6	5227	3756	1471	997.46	1.474746	F ₁₆	64.29	50.03	14.26	19.3	30.73	46.40417	46.43	1.01
						F ₁₇	75.03	57.24	17.79	19.39	37.85	47.00132		
						F ₁₈	64.47	50.28	14.19	19.36	30.92	45.89263		

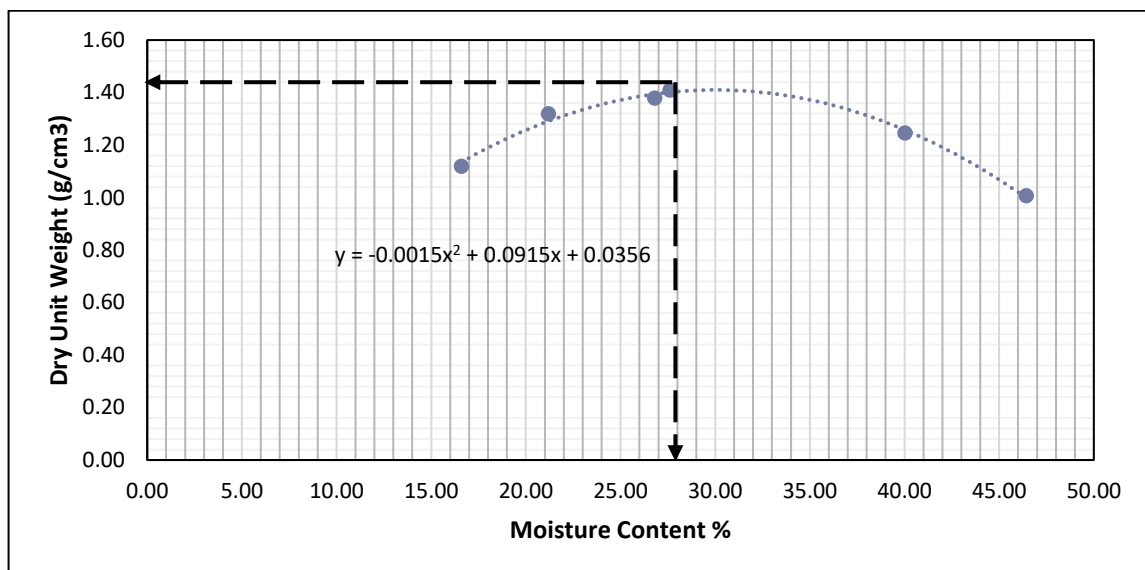


Figure F. 6 Moisture -Density Relation at soil+12% fly ash

Table F. 7 Moisture -Density Relation at soil+16% fly ash

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo							Date Sampled							
Station :- Dukem Town							Date Tested							
Depth:- BH-1 1.50M							Sample of							
Material Type :- Black Cotton														
Added 16% fly ash														
Trial No	mass of Compact ed soil +mold (g)	mass of mold+bas eplate (g)	Mass of Compact ed Soil (g)	Volume of mold (cm3)	Wet unit weight (g/cm3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (kn/m3)
1	5018	3756	1262	997.46	1.265214	F ₁	46.26	43.10	3.16	30.47	12.63	25.01979	18.640	1.066
				997.46		F ₂	46.35	44.15	2.20	29.91	14.24	15.44944		
				997.46		F ₃	59.24	55.30	3.94	29.80	25.50	15.45098		
2	5250	3756	1494	997.46	1.497804	F ₄	44.85	42.20	2.65	30.06	12.14	21.82867	20.836	1.240
				997.46		F ₅	55.11	51.00	4.11	30.33	20.67	19.88389		
				997.46		F ₆	41.84	39.80	2.04	29.99	9.81	20.79511		
3	5300	3756	1544	997.46	1.547932	F ₇	46.86	43.99	2.87	28.98	15.01	19.12059	21.254	1.277
				997.46		F ₈	63.21	57.40	5.81	29.63	27.77	20.92186		
				997.46		F ₉	60.04	54.30	5.74	30.10	24.20	23.71901		
4	5569	3756	1813	997.46	1.817617	F ₁₀	47.15	43.96	3.19	30.06	13.90	22.94964	25.270	1.451
				997.46		F ₁₁	55	49.99	5.01	29.98	20.01	25.03748		
				997.46		F ₁₂	58.1	51.99	6.11	30.03	21.96	27.82332		
5	5780	3756	2024	997.46	2.029154	F ₁₃	55.2	48.95	6.25	30.72	18.23	34.28415	35.824	1.494
				997.46		F ₁₄	61.24	53.13	8.11	29.94	23.19	34.97197		
				997.46		F ₁₅	57.65	49.98	7.67	29.91	20.07	38.21624		
6	5457	3756	1701	997.46	1.705332	F ₁₆	58.15	49.53	8.62	30.26	19.27	44.73275	45.459	1.172
				997.46		F ₁₇	65.22	54.16	11.06	30.10	24.06	45.96841		
				997.46		F ₁₈	64.00	53.70	10.30	31.15	22.55	45.67627		

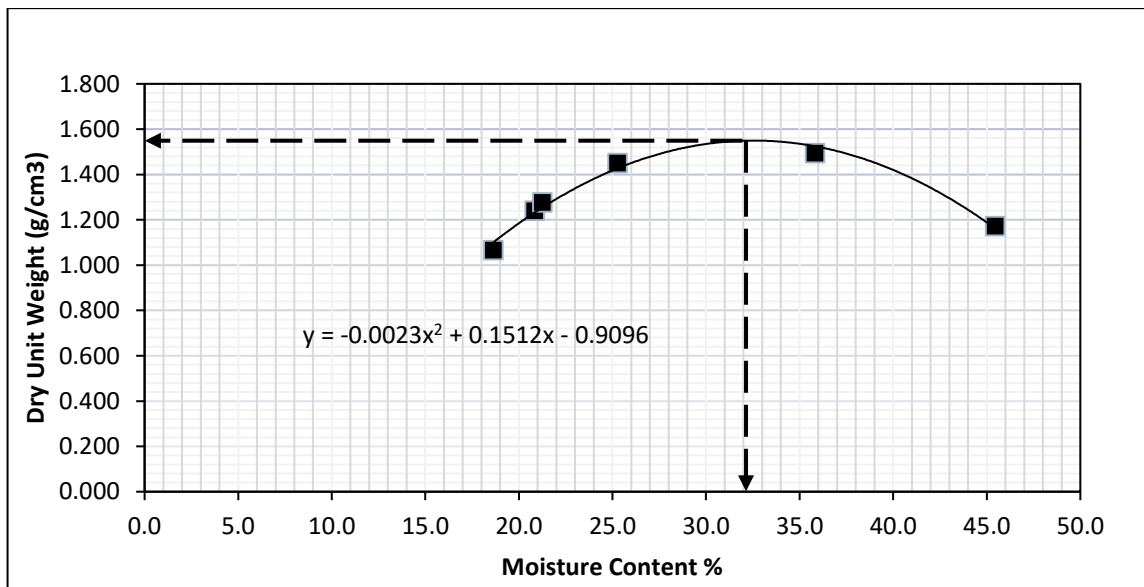


Figure F. 7 Moisture -Density Relation at soil+16% fly ash

Table F. 8 Moisture -Density Relation at soil+20% fly ash

Moisture -Density Relation of Soil AASHTO T180 Methode														
Tested by :- Sime Wadajo							Date Sampled							
Station :- Dukem Town							Date Tested							
Depth:- BH-1 1.50M							Sample of							
Material Type :- Black Cotton														
Added 20% fly ash														
Trial No	mass of Compact ed soil +mold (g)	mass of mold+bas eplate (g)	Mass of Compact ed Soil (g)	Volume of mold (cm3)	Wet unit weight (g/cm3)	Can No	Mass of wet Soil +can (g)	Mass of dry soil + can (g)	mass of water (g)	mass of can (g)	Mass of Dry Soil (g)	Moisture content (%)	Moisture content avrage (%)	Dry unit weight (g/cm3)
1	5012	3755	1257	997.46	1.260201	Fly A	50	45	5	19.4	25.6	19.53125	12.75	1.12
						Fly B	48	46	2	19.34	26.66	7.501875		
						Fly C	49	46	3	19.25	26.75	11.21495		
2	5295	3755	1540	997.46	1.543922	Fly-1	41.44	37	4.44	19.54	17.46	25.42955	17.41	1.32
						Fly-2	49.32	46.05	3.27	19.47	26.58	12.30248		
						Fly-3	53.28	49	4.28	19.46	29.54	14.48883		
3	5398	3755	1643	997.46	1.647184	Fly-4	47.53	43.6	3.93	19.56	24.04	16.34775	19.41	1.38
						Fly-5	46.78	41.5	5.28	19.44	22.06	23.93472		
						Fly-6	46.04	42	4.04	19.49	22.51	17.94758		
4	5480	3755	1725	997.46	1.729393	Fly-7	45.74	39.89	5.85	19.68	20.21	28.94607	21.17	1.43
						Fly-8	44.59	40.85	3.74	19.42	21.43	17.45217		
						Fly-9	38.69	35.85	2.84	19.25	16.6	17.10843		
5	5802	3755	2047	997.46	2.052213	Fly-10	41.79	37.5	4.29	19.73	17.77	24.14181	29.92	1.5796
						Fly-11	43.84	38	5.84	19.57	18.43	31.68747		
						Fly-12	57.68	48	9.68	19.48	28.52	33.94109		
6	5660	3755	1905	997.46	1.909851	Fly-13	62.3	48	14.3	19.60	28.4	50.35211	36.02	1.40
						Fly-14	56.52	48	8.52	19.41	28.59	29.80063		
						Fly-15	49.74	43	6.74	18.85	24.15	27.9089		
7	5475	3755	1720	997.46	1.72438	Fly-16	58.96	47.04	11.92	19.13	27.91	42.70871	41.84	1.22
						Fly-17	66.61	52.72	13.89	19.35	33.37	41.62421		
						Fly-18	81.46	63.37	18.09	19.44	43.93	41.17915		
8	5320	3756	1564	997.46	1.567983	Fly-19	54.07	44	10.07	19.18	24.82	40.57212	42.56	1.10
						Fly-20	60.18	47.07	13.11	19.34	27.73	47.27732		
						Fly-21	64.98	52	12.98	19.42	32.58	39.84039		

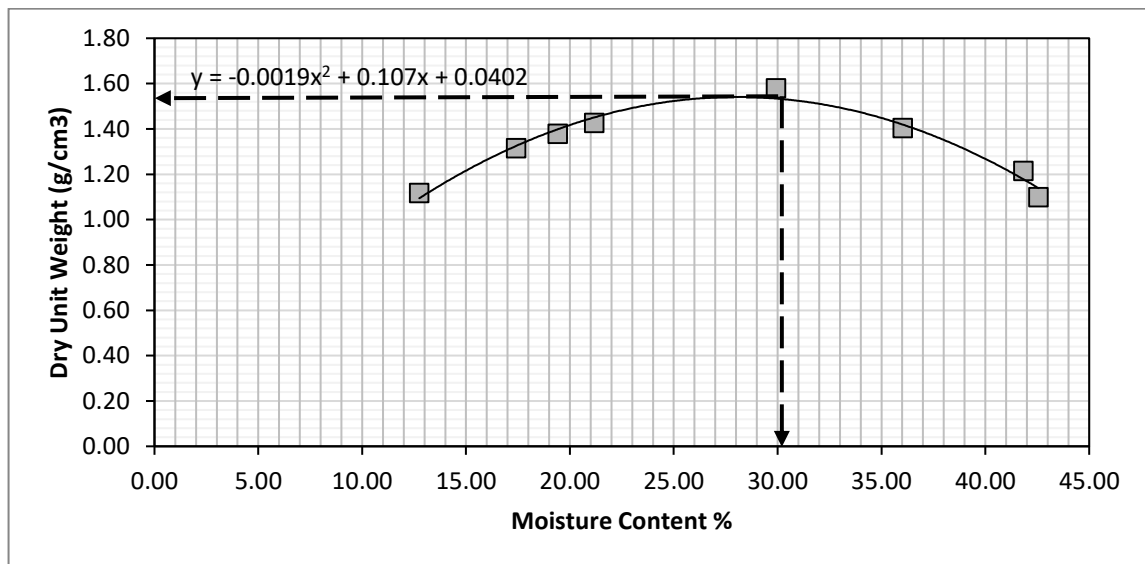


Figure F. 8 Moisture -Density Relation at soil+20% fly ash

Appendix G: Properties of Black cotton soil.

Table G. 1 Properties of Black cotton soil.

S.N.	Properties	Values obtained
1	Specific Gravity	2.69
2	Co-efficient of Curvature	1.09
3	Uniformity Co-efficient	11.79
4	Liquid Limit	55%
5	Plastic Limit	28.10%
6	CBR a) SOAKED b)UNSOAKED	1.28% 2.38%
7	MDD	1.451g/cm ³
8	OMC	27.2%
9	Unconfined Compressive Strength (qu)	1.618kg/cm ²

Appendix H: Unconfined compressive strength (UCS) test data for uncured and 7 days cured condition

For all UCS test samples the following data are similar

Initial Diameter (mm)	38	Initial Volume (cm ³)	Initial Volume (cm ³)	86.2
Initial Height Ho, (mm)	76	P.R. Factor (KN/div)	0.00153	0.00153
Initial Area Ao (m ²)	0.001			

Table H. 1 UCS test result for BH1(uncured)

UCS test result for BH1(uncured)						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \epsilon)$	stress in [kN/m ²]
0.00	0.0000	0	0	0	0.00100	0
0.20	0.0026	0.2632	12	0.01836	0.00100	18.3
0.40	0.0053	0.5263	18	0.02754	0.00101	27.4
0.60	0.0079	0.7895	23	0.03519	0.00101	34.9
0.80	0.0105	1.0526	27	0.04131	0.00101	40.9
1.00	0.0132	1.3158	32	0.04896	0.00101	48.3
1.20	0.0158	1.5789	38	0.05814	0.00102	57.2
1.40	0.0184	1.8421	43	0.06579	0.00102	64.6
1.60	0.0211	2.1053	48	0.07344	0.00102	71.9
1.80	0.0237	2.3684	51	0.07803	0.00102	76.2
2.00	0.0263	2.6316	54	0.08262	0.00103	80.4
2.20	0.0289	2.8947	58	0.08874	0.00103	86.2
2.40	0.0316	3.1579	62	0.09486	0.00103	91.9
2.60	0.0342	3.4211	66	0.10098	0.00104	97.5
2.80	0.0368	3.6842	67	0.10251	0.00104	98.7
3.00	0.0395	3.9474	66	0.10098	0.00104	97.0
3.20	0.0421	4.2105	65	0.09945	0.00104	95.3

Table H. 2 UCS test result for BH2(uncured) and 7 days cured

UCS test result for BH2 (uncured)						
Axial deformation (mm)	Unit strain $\varepsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \varepsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.2631579	8	0.00000	0.001002639	0.0
0.4	0.005263158	0.5263158	13	0.01224	0.001005291	12.2
0.6	0.007894737	0.7894737	19	0.01989	0.001007958	19.7
0.8	0.010526316	1.0526316	25	0.02907	0.001010638	28.8
1	0.013157895	1.3157895	30	0.03825	0.001013333	37.7
1.2	0.015789474	1.5789474	42	0.04590	0.001016043	45.2
1.4	0.018421053	1.8421053	48	0.06426	0.001018767	63.1
1.6	0.021052632	2.1052632	53	0.07344	0.001021505	71.9
1.8	0.023684211	2.3684211	59	0.08109	0.001024259	79.2
2	0.026315789	2.6315789	62	0.09027	0.001027027	87.9
2.2	0.028947368	2.8947368	65	0.09486	0.00102981	92.1
2.4	0.031578947	3.1578947	67	0.09945	0.001032609	96.3
2.6	0.034210526	3.4210526	68	0.10251	0.001035422	99.0
2.8	0.036842105	3.6842105	69	0.10404	0.001038251	100.2
UCS test for BH2-7 days cured						
Axial deformation (mm)	Unit strain $\varepsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \varepsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	8	0.01224	0.001002639	12.21
0.4	0.005263158	0.526316	14	0.02142	0.001005291	21.31
0.6	0.007894737	0.789474	20	0.0306	0.001007958	30.36
0.8	0.010526316	1.052632	26	0.03978	0.001010638	39.36
1	0.013157895	1.315789	31	0.04743	0.001013333	46.81
1.2	0.015789474	1.578947	38	0.05814	0.001016043	57.22
1.4	0.018421053	1.842105	43	0.06579	0.001018767	64.58
1.6	0.021052632	2.105263	49	0.07497	0.001021505	73.39
1.8	0.023684211	2.368421	54	0.08262	0.001024259	80.66
2	0.026315789	2.631579	59	0.09027	0.001027027	87.89
2.2	0.028947368	2.894737	63	0.09639	0.00102981	93.60
2.4	0.031578947	3.157895	66	0.10098	0.001032609	97.79
2.6	0.034210526	3.421053	68	0.10404	0.001035422	100.48
2.8	0.036842105	3.684211	69	0.10557	0.001038251	101.68
3	0.039473684	3.947368	70	0.1071	0.001041096	102.87
3.2	0.042105263	4.210526	69	0.10557	0.001043956	101.12

Table H. 3 UCS test result for BH3(uncured) and test result soil + 4% FA(uncured)

UCS test result for BH 3 (uncured)						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	7	0.01071	0.001002639	10.68
0.4	0.005263158	0.526316	15	0.02295	0.001005291	22.83
0.6	0.007894737	0.789474	23	0.03519	0.001007958	34.91
0.8	0.010526316	1.052632	29	0.04437	0.001010638	43.90
1	0.013157895	1.315789	35	0.05355	0.001013333	52.85
1.2	0.015789474	1.578947	40	0.0612	0.001016043	60.23
1.4	0.018421053	1.842105	46	0.07038	0.001018767	69.08
1.6	0.021052632	2.105263	51	0.07803	0.001021505	76.39
1.8	0.023684211	2.368421	55	0.08415	0.001024259	82.16
2	0.026315789	2.631579	59	0.09027	0.001027027	87.89
2.2	0.028947368	2.894737	62	0.09486	0.00102981	92.11
2.4	0.031578947	3.157895	65	0.09945	0.001032609	96.31
2.6	0.034210526	3.421053	68	0.10404	0.001035422	100.48
2.8	0.036842105	3.684211	70	0.1071	0.001038251	103.15
3	0.039473684	3.947368	66	0.10098	0.001041096	96.99
UCS test result for soil + 4% FA (uncured)						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	10	0.0153	0.00100	15.26
0.4	0.005263158	0.526316	18	0.02754	0.00101	27.40
0.6	0.007894737	0.789474	27	0.04131	0.00101	40.98
0.8	0.010526316	1.052632	36	0.05508	0.00101	54.50
1	0.013157895	1.315789	42	0.06426	0.00101	63.41
1.2	0.015789474	1.578947	48	0.07344	0.00102	72.28
1.4	0.018421053	1.842105	53	0.08109	0.00102	79.60
1.6	0.021052632	2.105263	58	0.08874	0.00102	86.87
1.8	0.023684211	2.368421	61	0.09333	0.00102	91.12
2	0.026315789	2.631579	62	0.09486	0.00103	92.36
2.2	0.028947368	2.894737	63	0.09639	0.00103	93.60
2.4	0.031578947	3.157895	67	0.10251	0.00103	99.27
2.6	0.034210526	3.421053	64	0.09792	0.00104	94.57
2.8	0.036842105	3.684211	63	0.09639	0.00104	92.84
3	0.039473684	3.947368	62	0.09486	0.00104	91.12

Table H. 4 UCS test result for for soil +4% FA 7days cured and soil +8% FA uncured

UCS test result for soil + 4 % FA 7 days "cured *						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_o$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_o / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	10	0.0153	0.00100	15.26
0.4	0.005263158	0.526316	16	0.02448	0.00101	24.35
0.6	0.007894737	0.789474	22	0.03366	0.00101	33.39
0.8	0.010526316	1.052632	27	0.04131	0.00101	40.88
1	0.013157895	1.315789	31	0.04743	0.00101	46.81
1.2	0.015789474	1.578947	38	0.05814	0.00102	57.22
1.4	0.018421053	1.842105	46	0.07038	0.00102	69.08
1.6	0.021052632	2.105263	50	0.0765	0.00102	74.89
1.8	0.023684211	2.368421	53	0.08109	0.00102	79.17
2	0.026315789	2.631579	57	0.08721	0.00103	84.92
2.2	0.028947368	2.894737	61	0.09333	0.00103	90.63
2.4	0.031578947	3.157895	63	0.09639	0.00103	93.35
2.6	0.034210526	3.421053	67	0.10251	0.00104	99.00
2.8	0.036842105	3.684211	71	0.10863	0.00104	104.63
3	0.039473684	3.947368	74	0.11322	0.00104	108.75
3.2	0.042105263	4.210526	73	0.11169	0.00104	106.99
UCS test result for soil + 8 % FA uncured						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_o$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_o / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	14	0.02142	0.00100	21.36
0.4	0.005263158	0.526316	22	0.03366	0.00101	33.48
0.6	0.007894737	0.789474	31	0.04743	0.00101	47.06
0.8	0.010526316	1.052632	38	0.05814	0.00101	57.53
1	0.013157895	1.315789	42	0.06426	0.00101	63.41
1.2	0.015789474	1.578947	48	0.07344	0.00102	72.28
1.4	0.018421053	1.842105	53	0.08109	0.00102	79.60
1.6	0.021052632	2.105263	57	0.08721	0.00102	85.37
1.8	0.023684211	2.368421	61	0.09333	0.00102	91.12
2	0.026315789	2.631579	65	0.09945	0.00103	96.83
2.2	0.028947368	2.894737	68	0.10404	0.00103	101.03
2.4	0.031578947	3.157895	70	0.1071	0.00103	103.72
2.6	0.034210526	3.421053	71	0.10863	0.00104	104.91
2.8	0.036842105	3.684211	74	0.11322	0.00104	109.05
3	0.039473684	3.947368	73	0.11169	0.00104	107.28

Table H. 5 UCS test result for for soil +8% FA 7days cured and soil +12% FA uncured

UCS data for soil + 8% FA 7 days cured						
Axial deformation (mm)	Unit strain $\epsilon=\Delta H/\Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area A = $A_0/1-\epsilon$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	13	0.01989	0.00100	19.84
0.4	0.005263158	0.526316	23	0.03519	0.00101	35.00
0.6	0.007894737	0.789474	34	0.05202	0.00101	51.61
0.8	0.010526316	1.052632	41	0.06273	0.00101	62.07
1	0.013157895	1.315789	46	0.07038	0.00101	69.45
1.2	0.015789474	1.578947	53	0.08109	0.00102	79.81
1.4	0.018421053	1.842105	57	0.08721	0.00102	85.60
1.6	0.021052632	2.105263	61	0.09333	0.00102	91.37
1.8	0.023684211	2.368421	65	0.09945	0.00102	97.09
2	0.026315789	2.631579	71	0.10863	0.00103	105.77
2.2	0.028947368	2.894737	75	0.11475	0.00103	111.43
2.4	0.031578947	3.157895	78	0.11934	0.00103	115.57
2.6	0.034210526	3.421053	80	0.1224	0.00104	118.21
2.8	0.036842105	3.684211	82	0.12546	0.00104	120.84
3	0.039473684	3.947368	86	0.13158	0.00104	126.39
3.2	0.042105263	4.210526	84	0.12852	0.00104	123.11
UCS data for soil + 12 % FA uncured						
Axial deformation (mm)	Unit strain $\epsilon=\Delta H/\Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area A = $A_0/1-\epsilon$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	12	0.01836	0.001	18.36
0.4	0.005263158	0.526316	20	0.0306	0.00101	30.30
0.6	0.007894737	0.789474	28	0.04284	0.00101	42.42
0.8	0.010526316	1.052632	35	0.05355	0.00101	53.02
1	0.013157895	1.315789	42	0.06426	0.00101	63.62
1.2	0.015789474	1.578947	50	0.0765	0.00102	75.00
1.4	0.018421053	1.842105	55	0.08415	0.00102	82.50
1.6	0.021052632	2.105263	59	0.09027	0.00102	88.50
1.8	0.023684211	2.368421	64	0.09792	0.00102	96.00
2	0.026315789	2.631579	67	0.10251	0.00103	99.52
2.2	0.028947368	2.894737	71	0.10863	0.00103	105.47
2.4	0.031578947	3.157895	73	0.11169	0.00103	108.44
2.6	0.034210526	3.421053	74	0.11322	0.00104	108.87
2.8	0.036842105	3.684211	76	0.11628	0.00104	111.81
3	0.039473684	3.947368	78	0.11934	0.00104	114.75
3.2	0.042105263	4.210526	77	0.11781	0.00104	113.28

Table H. 6 UCS test result for soil +12% FA 7days cured and soil +16% FA uncured

UCS data for soil + 12% FA 7 days cured						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	12	0.01836	0.00100	18.31
0.4	0.005263158	0.526316	22	0.03366	0.00101	33.48
0.6	0.007894737	0.789474	31	0.04743	0.00101	47.06
0.8	0.010526316	1.052632	39	0.05967	0.00101	59.04
1	0.013157895	1.315789	48	0.07344	0.00101	72.47
1.2	0.015789474	1.578947	55	0.08415	0.00102	82.82
1.4	0.018421053	1.842105	61	0.09333	0.00102	91.61
1.6	0.021052632	2.105263	68	0.10404	0.00102	101.85
1.8	0.023684211	2.368421	72	0.11016	0.00102	107.55
2	0.026315789	2.631579	75	0.11475	0.00103	111.73
2.2	0.028947368	2.894737	78	0.11934	0.00103	115.89
2.4	0.031578947	3.157895	82	0.12546	0.00103	121.50
2.6	0.034210526	3.421053	84	0.12852	0.00104	124.12
2.8	0.036842105	3.684211	86	0.13158	0.00104	126.73
3	0.039473684	3.947368	87	0.13311	0.00104	127.86
3.2	0.042105263	4.210526	89	0.13617	0.00104	130.44
3.4	0.044736842	4.473684	88	0.13464	0.00105	128.62
UCS test result for 16% FA uncured						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0 / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	15	0.02295	0.00100	22.89
0.4	0.005263158	0.526316	23	0.03519	0.00101	35.00
0.6	0.007894737	0.789474	32	0.04896	0.00101	48.57
0.8	0.010526316	1.052632	37	0.05661	0.00101	56.01
1	0.013157895	1.315789	39	0.05967	0.00101	58.88
1.2	0.015789474	1.578947	46	0.07038	0.00102	69.27
1.4	0.018421053	1.842105	53	0.08109	0.00102	79.60
1.6	0.021052632	2.105263	62	0.09486	0.00102	92.86
1.8	0.023684211	2.368421	70	0.1071	0.00102	104.56
2	0.026315789	2.631579	78	0.11934	0.00103	116.20
2.2	0.028947368	2.894737	85	0.13005	0.00103	126.29
2.4	0.031578947	3.157895	90	0.1377	0.00103	133.35
2.6	0.034210526	3.421053	93	0.14229	0.00104	137.42
2.8	0.036842105	3.684211	95	0.14535	0.00104	140.00
3	0.039473684	3.947368	96	0.14688	0.00104	141.08
3.2	0.042105263	4.210526	94	0.14382	0.00104	137.76

Table H. 7 UCS test result for for soil +16% FA and soil +20% FA 7days cured

UCS test result for soil + 16 % FA 7 days cured						
Axial deformation (mm)	Unit strain $\epsilon=\Delta H/\Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0/1-\epsilon$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	14	0.02142	0.00100	21.36
0.4	0.005263158	0.526316	22	0.03366	0.00101	33.48
0.6	0.007894737	0.789474	31	0.04743	0.00101	47.06
0.8	0.010526316	1.052632	38	0.05814	0.00101	57.53
1	0.013157895	1.315789	45	0.06885	0.00101	67.94
1.2	0.015789474	1.578947	52	0.07956	0.00102	78.30
1.4	0.018421053	1.842105	61	0.09333	0.00102	91.61
1.6	0.021052632	2.105263	69	0.10557	0.00102	103.35
1.8	0.023684211	2.368421	77	0.11781	0.00102	115.02
2	0.026315789	2.631579	84	0.12852	0.00103	125.14
2.2	0.028947368	2.894737	89	0.13617	0.00103	132.23
2.4	0.031578947	3.157895	92	0.14076	0.00103	136.31
2.6	0.034210526	3.421053	94	0.14382	0.00104	138.90
2.8	0.036842105	3.684211	95	0.14535	0.00104	140.00
3	0.039473684	3.947368	93	0.14229	0.00104	136.67
UCS data for soil + 20% FA 7 days cured						
Axial deformation (mm)	Unit strain $\epsilon=\Delta H/\Delta H_0$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_0/1-\epsilon$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	14	0.02142	0.00100	21.36
0.4	0.005263158	0.526316	22	0.03366	0.00101	33.48
0.6	0.007894737	0.789474	31	0.04743	0.00101	47.06
0.8	0.010526316	1.052632	38	0.05814	0.00101	57.53
1	0.013157895	1.315789	45	0.06885	0.00101	67.94
1.2	0.015789474	1.578947	52	0.07956	0.00102	78.30
1.4	0.018421053	1.842105	61	0.09333	0.00102	91.61
1.6	0.021052632	2.105263	69	0.10557	0.00102	103.35
1.8	0.023684211	2.368421	77	0.11781	0.00102	115.02
2	0.026315789	2.631579	84	0.12852	0.00103	125.14
2.2	0.028947368	2.894737	89	0.13617	0.00103	132.23
2.4	0.031578947	3.157895	92	0.14076	0.00103	136.31
2.6	0.034210526	3.421053	94	0.14382	0.00104	138.90
2.8	0.036842105	3.684211	97	0.14841	0.00104	142.94
3	0.039473684	3.947368	93	0.14229	0.00104	136.67
3.2	0.042105263	4.210526	90	0.1377	0.00104	131.90

Table H. 8 UCS test result for soil + 20 % FA uncured

UCS data for soil + 20 % FA uncured						
Axial deformation (mm)	Unit strain $\epsilon = \Delta H / \Delta H_o$	axial strain (%)	Proving Ring reading 0	Load in [kN]	Cross - sectional Area $A = A_o / (1 - \epsilon)$	stress in [kN/m ²]
0	0	0	0	0	0.001	0
0.2	0.002631579	0.263158	12	0.01836	0.00100	18.31
0.4	0.005263158	0.526316	20	0.0306	0.00101	30.44
0.6	0.007894737	0.789474	28	0.04284	0.00101	42.50
0.8	0.010526316	1.052632	35	0.05355	0.00101	52.99
1	0.013157895	1.315789	42	0.06426	0.00101	63.41
1.2	0.015789474	1.578947	53	0.08109	0.00102	79.81
1.4	0.018421053	1.842105	58	0.08874	0.00102	87.11
1.6	0.021052632	2.105263	62	0.09486	0.00102	92.86
1.8	0.023684211	2.368421	67	0.10251	0.00102	100.08
2	0.026315789	2.631579	70	0.1071	0.00103	104.28
2.2	0.028947368	2.894737	73	0.11169	0.00103	108.46
2.4	0.031578947	3.157895	76	0.11628	0.00103	112.61
2.6	0.034210526	3.421053	81	0.12393	0.00104	119.69
2.8	0.036842105	3.684211	88	0.13464	0.00104	129.68
3	0.039473684	3.947368	96	0.14688	0.00104	141.08
3.2	0.042105263	4.210526	77	0.11781	0.00104	112.85

Appendix I: California Bearing Ratio Test Data

Table I. 1 California Bearing Ratio Test Data at 0% fly ash

Representative station	Soil +0% fly ash			source	BH				
depth	0-150cm			Lab No.					
visual description	Black cotton			purpose	Stabilization				
sampling date	10/8/2015			test date	14/08/2015				
<u>CALIFORNIA BEARING RATIO</u>									
<u>DENSITY DETERMINATION</u>									
SOAKING CONDITION	10 Blows		30 Blows		65 Blows				
	Before	After	Before	After	Before	After			
MOLD NUMBER	34		31		33				
WEIGHT OF SOIL + MOLD, g	9248	10028	9453	9750	9865	10442			
WEIGHT OF MOLD, g	6274		6117		6116				
WEIGHT OF SOIL , g	2974	3754	3336	3633	3749	4326			
VOLUME OF MOLD, g	2198		2198		2198				
WET DENSITY OF SOIL, g/cm ³	1.35	1.71	1.52	1.65	1.71	1.97			
DRY DENSITY OF SOIL, g/cm ³	1.13	1.34	1.28	1.34	1.44	1.57			
<u>MOISTURE DETERMINATION</u>									
SOAKING CONDITION	10 Blows		30 Blows		65 Blows				
	Before	After	Before	After	Before	After			
CONTAINER NUMBER	3	8	7	5	1	6			
WET SOIL + CONTAINER, g	184.07	183.11	185.18	163.62	192.59	157.69			
DRY SOIL + CONTAINER, g	159.89	151.71	161.42	138.59	169	132.97			
WEIGHT OF CONTAINER, g	35.15	35.40	35.38	31.44	38.41	34.68			
WEIGHT OF WATER , g	24.18	31.4	23.76	25.03	23.59	24.72			
WEIGHT OF DRY SOIL, g	124.74	116.31	126.04	107.15	130.59	98.29			
MOISTURE CONTENT, %	19.4	27.0	18.9	23.4	18.1	25.2			
<u>PENETRATION BOTTOM TEST</u>									
PENETRATION DATE					RING FACTOR	0.125672			
PENETRATION (mm)	10 Blows			30 Blows		65 Blows			
	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00	0.0	0.00		0.0	0.00		0.0	0.00	
0.64	0.0	0.00		0.5	0.06		0.5	0.06	
1.27	0.0	0.00		0.50	0.06		1.0	0.13	
1.91	0.5	0.06		1.00	0.13		1.00	0.13	
2.54	0.5	0.06	0.46	1.0	0.13	0.93	1.5	0.19	1.39
3.18	0.5	0.06		1.0	0.13		1.5	0.19	
3.81	0.5	0.06		1.5	0.19		2.0	0.25	
4.45	1.0	0.13		1.5	0.19		2.0	0.25	
5.08	1.0	0.13	0.62	2.0	0.25	1.24	2.0	0.25	1.24
7.62	1.0	0.13		2.0	0.25		2.5	0.31	
MDD (gm/cc)	1.34				SWELL				
95% MDD	1.27				Number of blows	10	30	65.0	
CBR at 95 %	1.14				Initial reading (mm)	0.00	0.00	0.00	
SWELL at 95 %	9.45				Final reading (mm)	13.15	12.05	11.01	
					Height of specimen (mm)	118	118	118	
					Percent Swell (%)	11.14	10.21	9.33	
					Average	10.23			

CBR - CURVE

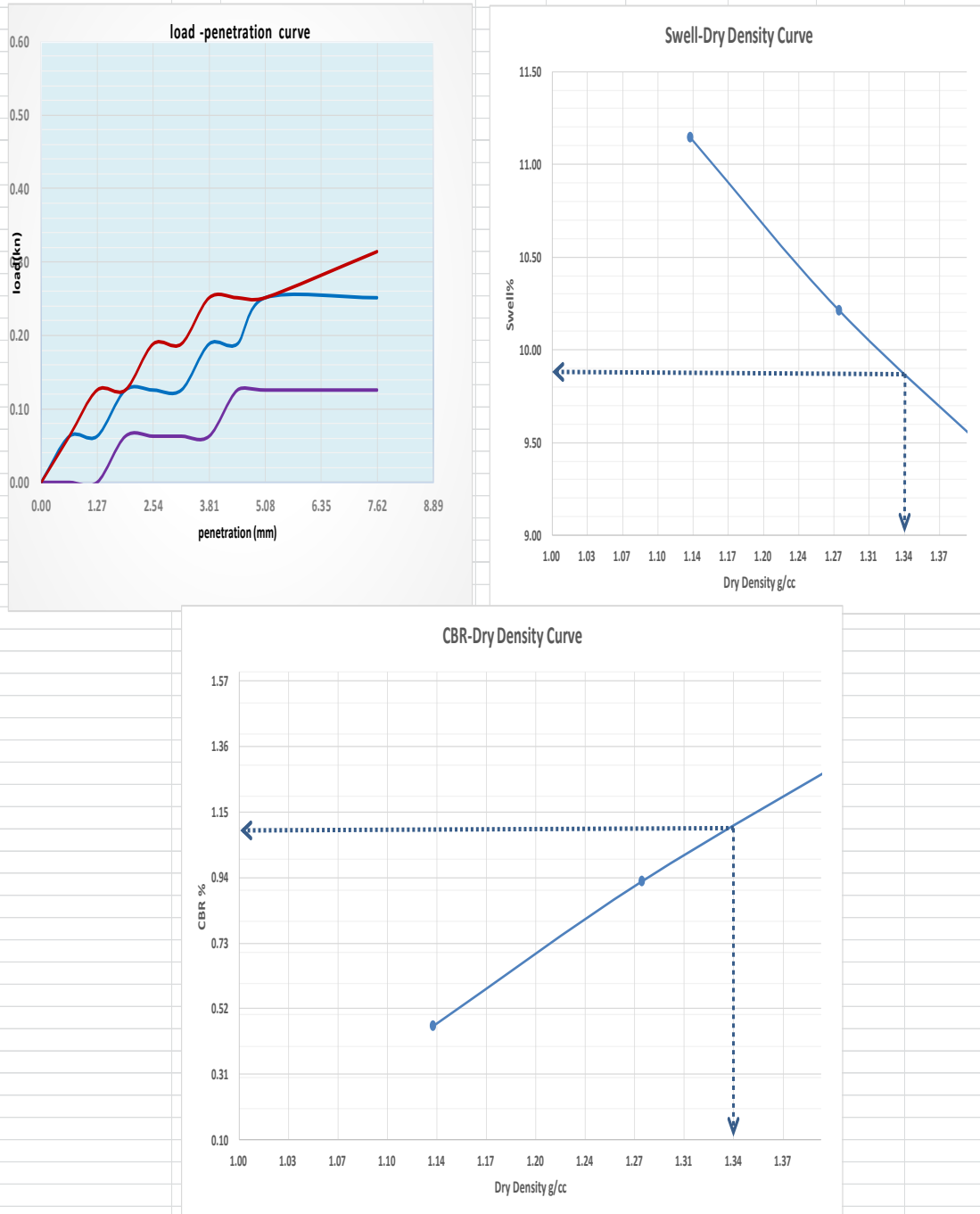


Figure I. 1 Graphical illustration of California Bearing Ratio Test Data at 0% fly ash

Table I. 2 California Bearing Ratio Test Data at 4% fly ash

Representative station	Soil +4% fly ash			source		BH				
depth	0-150cm			Lab No.						
visual description	Black cotton			purpose		Stabilization				
sampling date	10/8/2015			test date		14/08/2015				
CALIFORNIA BEARING RATIO										
DENSITY DETERMINATION										
SOAKING CONDITION				10 Blows		30 Blows		65 Blows		
				Before	After	Before	After	Before	After	
MOLD NUMBER				15		14		16		
WEIGHT OF SOIL + MOLD, g				11063	12032	11251	12015	11840	12511	
WEIGHT OF MOLD, g				8232		8129		8217		
WEIGHT OF SOIL , g				2831	3800	3122	3886	3623	4294	
VOLUME OF MOLD, g				2177		2177		2177		
WET DENSITY OF SOIL, g/cm³				1.30	1.75	1.43	1.79	1.66	1.97	
DRY DENSITY OF SOIL, g/cm³				1.09	1.25	1.21	1.34	1.41	1.50	
MOISTURE DETERMINATION										
SOAKING CONDITION				10 Blows		30 Blows		65 Blows		
				Before	After	Before	After	Before	After	
CONTAINER NUMBER				70	12	89	1	55	2	
WET SOIL + CONTAINER, g				256.25	161.69	256.86	199.77	256.18	183.22	
DRY SOIL + CONTAINER, g				219.06	125.8	221.46	159.5	221.83	147.59	
WEIGHT OF CONTAINER, g				31.25	34.37	32.65	38.42	33.0	33.44	
WEIGHT OF WATER , g				37.19	35.89	35.4	40.27	34.35	35.63	
WEIGHT OF DRY SOIL, g				187.81	91.43	188.81	121.08	188.83	114.15	
MOISTURE CONTENT, %				19.8	39.3	18.7	33.3	18.2	31.2	
PENETRATION BOTTOM TEST										
PENETRATION DATE							RING FACTOR		0.125672	
PENETRATION (mm)		10 Blows			30 Blows			65 Blows		
		Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00		0.0	0.00		0.0	0.00		0.0	0.00	
0.64		0.5	0.06		1.0	0.13		1.0	0.13	
1.27		0.5	0.06		1.5	0.19		2.0	0.25	
1.91		1.0	0.13		1.5	0.19		2.0	0.25	
2.54		1.0	0.13	0.93	1.8	0.23	1.67	2.9	0.36	
3.18		1.5	0.19		2.5	0.31		3.5	0.44	
3.81		1.5	0.19		3.0	0.38		4.0	0.50	
4.45		2.0	0.25		3.0	0.38		4.0	0.50	
5.08		2.0	0.25	1.24	3.0	0.38	1.87	4.5	0.57	
7.62		2.5	0.31		3.5	0.44		5.5	0.69	
MDD (gm/cc)		1.37			SWELL					
95% MDD		1.30			Number of blows		10	30	65.0	
CBR at 95 %		2.50			Initial reading (mm)		0.00	0.00	0.00	
SWELL at 95 %		9.80			Final reading (mm)		13.27	12.90	11.45	
					Height of specimen (mm)		120	120	120	
					Percent Swell (%)		11.06	10.75	9.54	
					Average		10.45			

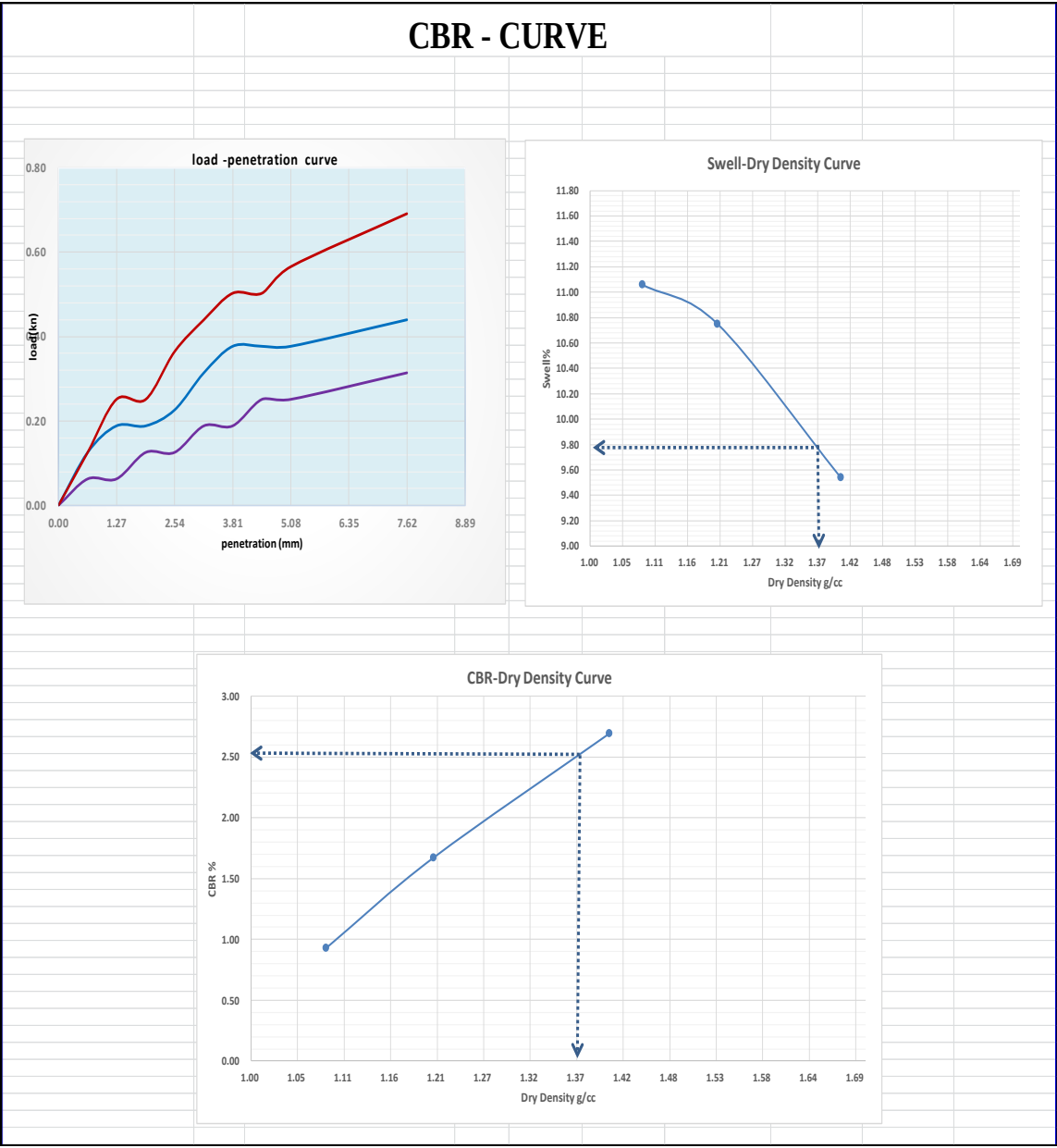


Figure I. 2 Graphical illustration of California Bearing Ratio Test Data at 4% fly ash

Table I. 3 California Bearing Ratio Test Data at 8% fly ash

Representative station	Soil +8% fly ash			source		BH			
depth	0-150cm			Lab No.					
visual description	Black cotton			purpose		Stabilization			
sampling date	10/8/2015			test date		14/08/2015			
<u>CALIFORNIA BEARING RATIO</u>									
<u>DENSITY DETERMINATION</u>									
SOAKING CONDITION				10 Blows		30 Blows		65 Blows	
				Before	After	Before	After	Before	After
MOLD NUMBER				6		3		8	
WEIGHT OF SOIL + MOLD, g				8772	9258	9169	9136	9410	9720
WEIGHT OF MOLD, g				5408		5400		5392	
WEIGHT OF SOIL , g				3364	3850	3769	3736	4018	4328
VOLUME OF MOLD, g				2177		2177		2177	
WET DENSITY OF SOIL, g/cm ³				1.55	1.77	1.73	1.72	1.85	1.99
DRY DENSITY OF SOIL, g/cm ³				1.26	1.28	1.42	1.30	1.49	1.51
<u>MOISTURE DETERMINATION</u>									
SOAKING CONDITION				10 Blows		30 Blows		65 Blows	
				Before	After	Before	After	Before	After
CONTAINER NUMBER				17	18	18	40	65	60
WET SOIL + CONTAINER, g				202.14	176.39	194.77	186.76	219.68	208.67
DRY SOIL + CONTAINER, g				169.69	135.23	164.27	148.52	184.12	165.57
WEIGHT OF CONTAINER, g				26.36	27.50	27.51	27.23	32.54	28.3
WEIGHT OF WATER , g				32.45	41.16	30.5	38.24	35.56	43.1
WEIGHT OF DRY SOIL, g				143.33	107.73	136.76	121.29	151.58	137.27
MOISTURE CONTENT, %				22.6	38.2	22.3	31.5	23.5	31.4
<u>PENETRATION BOTTOM TEST</u>									
PENETRATION DATE							RING FACTOR	0.125672	
PENETRATION (mm)	10 Blows			30 Blows			65 Blows		
	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00	0.0	0.00		0.0	0.00		0.0	0.00	
0.64	1.0	0.13		1.5	0.19		1.5	0.19	
1.27	1.5	0.19		2.50	0.31		2.50	0.31	
1.91	2.0	0.25		3.50	0.44		3.50	0.44	
2.54	2.5	0.31	2.32	4.5	0.57	4.18	5.0	0.63	4.64
3.18	3.0	0.38		5.0	0.63		6.0	0.75	
3.81	4.5	0.57		6.0	0.75		7.0	0.88	
4.45	5.0	0.63		7.0	0.88		8.0	1.01	
5.08	6.0	0.75	3.73	8.0	1.01	4.97	9.0	1.13	5.60
7.62	7.5	0.94		9.5	1.19		10.0	1.26	

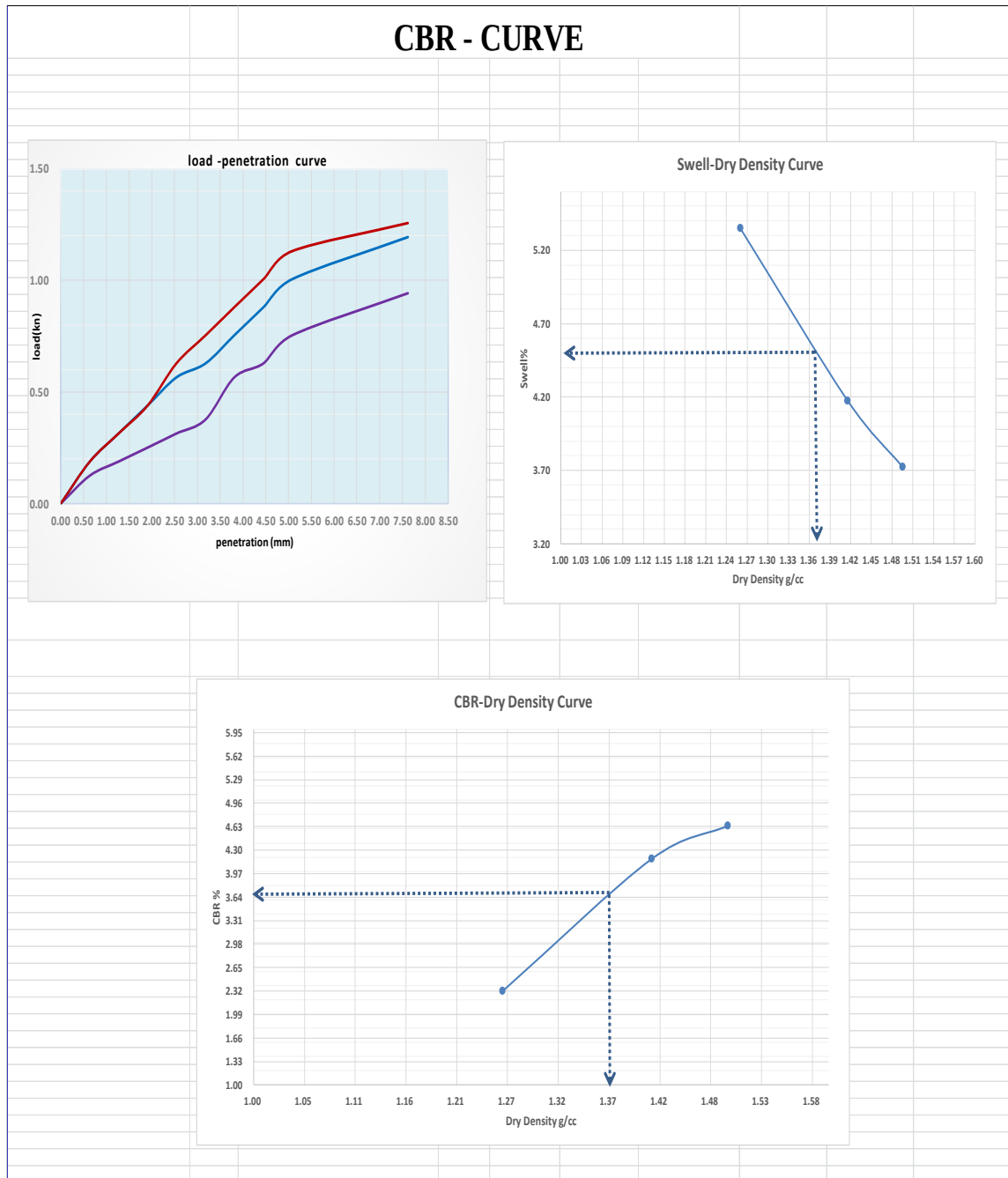


Figure I. 3 Graphical illustration of California Bearing Ratio Test Data at 8% fly ash

Table I. 4 California Bearing Ratio Test Data at 12% fly ash

Representative station	Soil +12% fly ash			source		BH				
depth	0-150cm			Lab No.						
visual description	Black cotton			purpose		Stabilization				
sampling date	10/8/2015			test date		14/08/2015				
<u>CALIFORNIA BEARING RATIO</u>										
<u>DENSITY DETERMINATION</u>										
SOAKING CONDITION				10 Blows		30 Blows		65 Blows		
				Before	After	Before	After	Before	After	
MOLD NUMBER				7		8		3		
WEIGHT OF SOIL + MOLD, g				7017	9130	8701	8790	10068	9140	
WEIGHT OF MOLD, g				5335	5335	5445	5445	5328	5328	
WEIGHT OF SOIL , g				1682	3795	3256	3345	4740	3812	
VOLUME OF MOLD, g				2177		2177		2177		
WET DENSITY OF SOIL, g/cm ³				0.77	1.74	1.50	1.54	2.18	1.75	
DRY DENSITY OF SOIL, g/cm ³				0.69	1.43	1.34	1.31	1.94	1.51	
<u>MOISTURE DETERMINATION</u>										
SOAKING CONDITION				10 Blows		30 Blows		65 Blows		
				Before	After	Before	After	Before	After	
CONTAINER NUMBER				60	16	19	17	16	20	
WET SOIL + CONTAINER, g				257.6	232.38	244	174.6	185.9	195.16	
DRY SOIL + CONTAINER, g				232.36	195.63	220.83	152.51	168	172.03	
WEIGHT OF CONTAINER, g				28.34	25.86	26.2	26.38	24	24.03	
WEIGHT OF WATER , g				25.24	36.75	23.17	22.09	17.9	23.13	
WEIGHT OF DRY SOIL, g				204.02	169.77	194.63	126.13	144	148	
MOISTURE CONTENT, %				12.4	21.6	11.9	17.5	12.4	15.6	
<u>PENETRATION BOTTOM TEST</u>										
<u>PENETRATION DATE</u>						<u>RING FACTOR</u>		0.125672		
PENETRATION (mm)		10 Blows			30 Blows			65 Blows		
		Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00		0	0.00		0.0	0.00		0.0	0.00	
0.64		1	0.13		2.0	0.25		4.0	0.50	
1.27		2	0.25		4.0	0.50		6.0	0.75	
1.91		3	0.38		5.0	0.63		7.0	0.88	
2.54		4	0.50	3.71	6.0	0.75	5.57	8.5	1.07	
3.18		4.5	0.57		7.0	0.88		9.0	1.13	
3.81		5	0.63		8.0	1.01		9.5	1.19	
4.45		6	0.75		9.0	1.13		10.0	1.26	
5.08		6.5	0.82	4.04	9.5	1.19	5.91	11.0	1.38	
7.62		7.5	0.94		11	1.38		13.0	1.63	
MDD (gm/cc)		1.42			SWELL					
95% MDD		1.35			Number of blows		10	30	65.0	
CBR at 95 %		6.00			Initial reading (mm)		0.00	0.00	0.00	
SWELL at 95 %		1.70			Final reading (mm)		1.35	2.35	2.50	
					Height of specimen (mm)		160	180	145	
					Percent Swell (%)		0.84	1.31	1.72	
					Average		1.29			

CBR - CURVE

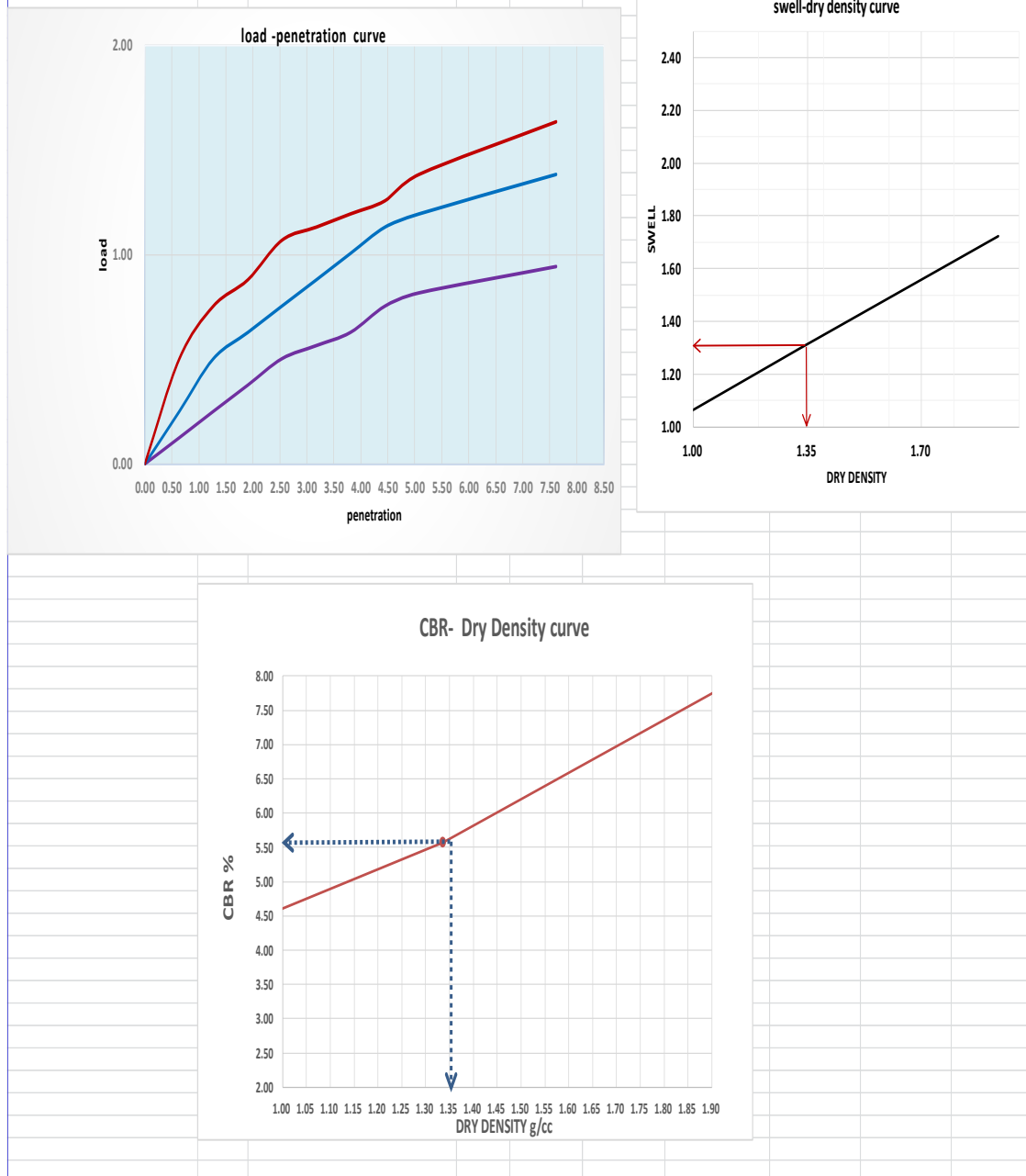


Figure I. 4 Graphical illustration of California Bearing Ratio Test Data at 12% fly ash

Table I. 5 California Bearing Ratio Test Data at 16% fly ash

Representative station	Soil +16% fly ash			source	BH				
depth	0-150cm			Lab No.					
visual description	Black cotton			purpose	Stabilization				
sampling date	10/8/2015			test date	14/08/2015				
<u>CALIFORNIA BEARING RATIO</u>									
<u>DENSITY DETERMINATION</u>									
SOAKING CONDITION	10 Blows		30 Blows		65 Blows				
	Before	After	Before	After	Before	After			
MOLD NUMBER	33		32		30				
WEIGHT OF SOIL + MOLD, g	9400	9810	10523	10108	10869	10127			
WEIGHT OF MOLD, g	6118		6189		6091				
WEIGHT OF SOIL , g	3282	3692	4334	3919	4778	4036			
VOLUME OF MOLD, g	2198		2198		2198				
WET DENSITY OF SOIL, g/cm ³	1.49	1.68	1.97	1.78	2.17	1.84			
DRY DENSITY OF SOIL, g/cm ³	1.18	1.21	1.56	1.34	1.73	1.39			
<u>MOISTURE DETERMINATION</u>									
SOAKING CONDITION	10 Blows		30 Blows		65 Blows				
	Before	After	Before	After	Before	After			
CONTAINER NUMBER	82	40	77	17	70	18			
WET SOIL + CONTAINER, g	185.23	195.89	200.86	213.15	196.73	197.12			
DRY SOIL + CONTAINER, g	153.43	150.78	165.80	166.86	162.72	155.84			
WEIGHT OF CONTAINER, g	32.05	35.67	33.19	26.37	31.23	27.51			
WEIGHT OF WATER , g	31.8	45.11	35.06	46.29	34.01	41.28			
WEIGHT OF DRY SOIL, g	121.38	115.11	132.61	140.49	131.49	128.33			
MOISTURE CONTENT, %	26.2	39.2	26.4	32.9	25.9	32.2			
<u>PENETRATION BOTTOM TEST</u>									
<u>PENETRATION DATE</u>					<u>RING FACTOR</u>	0.125672			
PENETRATION (mm)	10 Blows			30 Blows		65 Blows			
	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00	0.0	0.00		0.0	0.00		0.0	0.00	
0.64	1.0	0.13		1.5	0.19		2.0	0.25	
1.27	2.0	0.25		2.5	0.31		4.0	0.50	
1.91	3.0	0.38		5.0	0.63		6.0	0.75	
2.54	5.0	0.63	4.64	8.0	1.01	7.43	9.0	1.13	8.35
3.18	7.0	0.88		10.0	1.26		11.0	1.38	
3.81	9.0	1.13		11.0	1.38		12.0	1.51	
4.45	10.0	1.26		11.5	1.45		13.0	1.63	
5.08	11.0	1.38	6.84	12.0	1.51	7.46	14.0	1.76	8.71
7.62	12.0	1.51		14.0	1.76		16.0	2.01	
MDD (gm/cc)	1.57				SWELL				
95% MDD	1.49				Number of blows	10	30	65.0	
CBR at 95 %	6.90				Initial reading (mm)	0.00	0.00	0.00	
SWELL at 95 %	11.76				Final reading (mm)	14.85	14.32	13.82	
					Height of specimen (mm)	118	118	118	
					Percent Swell (%)	12.58	12.14	11.71	
					Average	12.14			

CBR - CURVE

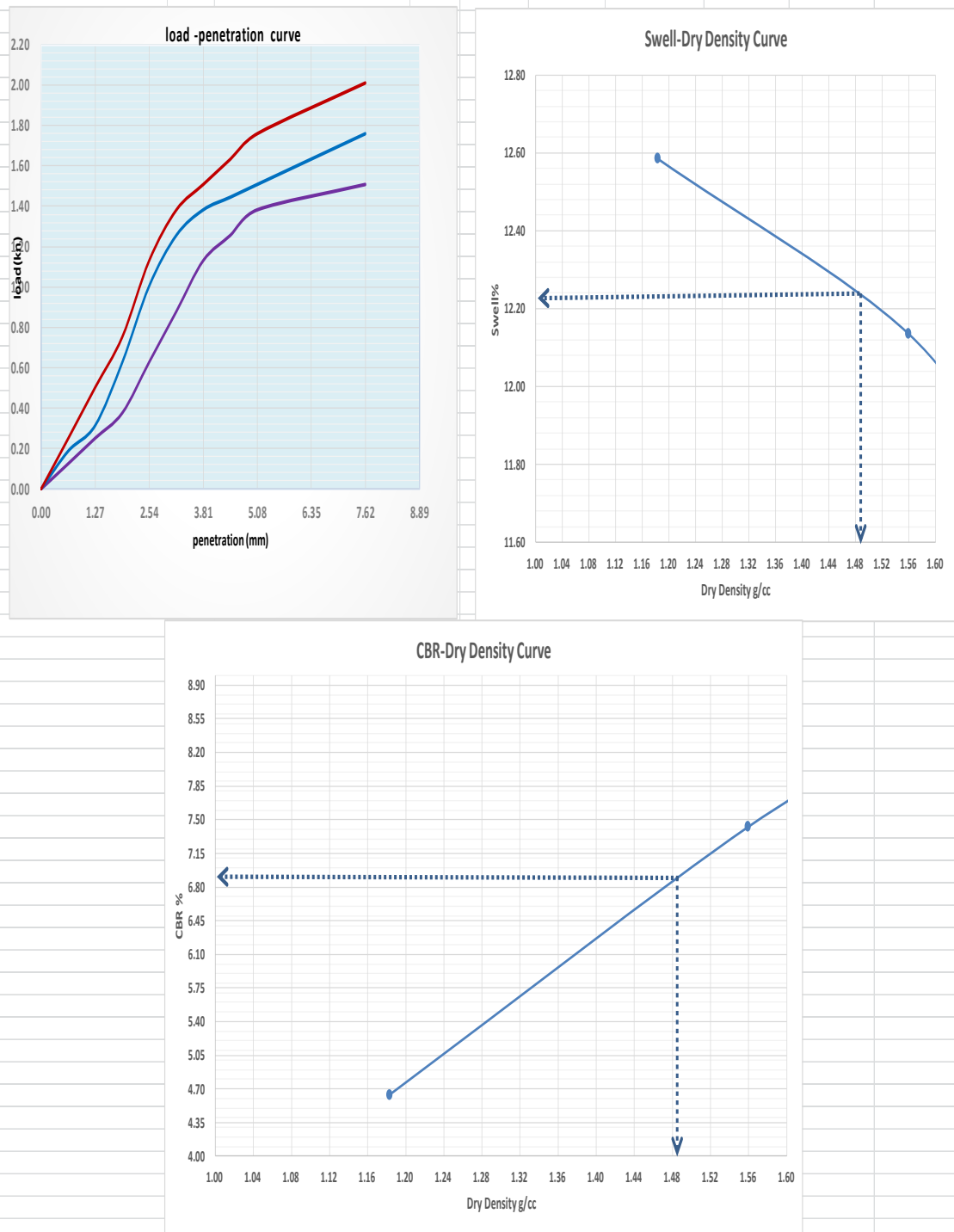


Figure I. 5 Graphical illustration of California Bearing Ratio Test Data at 16% fly ash

Table I. 6 California Bearing Ratio Test Data at 20% fly ash

Representative	Soil +20% fly ash			source		BH			
depth	0-150cm			Lab No.		-			
visual description	Black cotton			purpose		Stabilization			
sampling date	10/8/2015			test date		14/08/2015			
CALIFORNIA BEARING RATIO									
DENSITY DETERMINATION									
SOAKING CONDITION				10 Blows		30 Blows		65 Blows	
				Before	After	Before	After	Before	After
MOLD NUMBER				33		32		30	
WEIGHT OF SOIL + MOLD, g				9400	9910	9823	10548	10869	10127
WEIGHT OF MOLD, g				6118		6189		6091	
WEIGHT OF SOIL , g				3282	3792	3634	4359	4778	4036
VOLUME OF MOLD, g				2198		2198		2198	
WET DENSITY OF SOIL, g/cm ³				1.49	1.73	1.65	1.98	2.17	1.84
DRY DENSITY OF SOIL, g/cm ³				1.18	1.24	1.31	1.49	1.73	1.39
MOISTURE DETERMINATION									
SOAKING CONDITION				10 Blows		30 Blows		65 Blows	
				Before	After	Before	After	Before	After
CONTAINER NUMBER				82	40	77	17	70	18
WET SOIL + CONTAINER, g				185.23	195.89	200.86	213.15	196.73	197.12
DRY SOIL + CONTAINER, g				153.43	150.78	165.80	166.86	162.72	155.84
WEIGHT OF CONTAINER, g				32.05	35.67	33.19	26.37	31.23	27.51
WEIGHT OF WATER , g				31.8	45.11	35.06	46.29	34.01	41.28
WEIGHT OF DRY SOIL, g				121.38	115.11	132.61	140.49	131.49	128.33
MOISTURE CONTENT, %				26.2	39.2	26.4	32.9	25.9	32.2
PENETRATION BOTTOM TEST									
PENETRATION DATE				RING FACTOR				0.125672	
PENETRATION (mm)	10 Blows			30 Blows			65 Blows		
	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)	Dial reading	Load (KN)	C.B.R(%)
0.00	0.0	0.00		0.0	0.00		0.0	0.00	
0.64	1.0	0.13		1.5	0.19		2.0	0.25	
1.27	2.0	0.25		2.5	0.31		4.0	0.50	
1.91	3.0	0.38		5.0	0.63		6.0	0.75	
2.54	5.0	0.63	4.64	8.0	1.01	7.43	9.0	1.13	8.35
3.18	7.0	0.88		10.0	1.26		11.0	1.38	
3.81	9.0	1.13		11.0	1.38		12.0	1.51	
4.45	10.0	1.26		11.5	1.45		13.0	1.63	
5.08	11.0	1.38	6.84	12.0	1.51	7.46	14.0	1.76	8.71
7.62	12.0	1.51		14.0	1.76		16.0	2.01	
MDD (gm/cc)	1.58					SWELL			
95% MDD	1.50					Number of blows	10	30	65.0
CBR at 95 %	8.05					Initial reading (mm)	0.00	0.00	0.00
SWELL at 95 %	11.76					Final reading (mm)	14.85	14.32	13.82
						Height of specimen (mm)	118	118	118
						Percent Swell (%)	12.58	12.14	11.71
						Average	12.14		

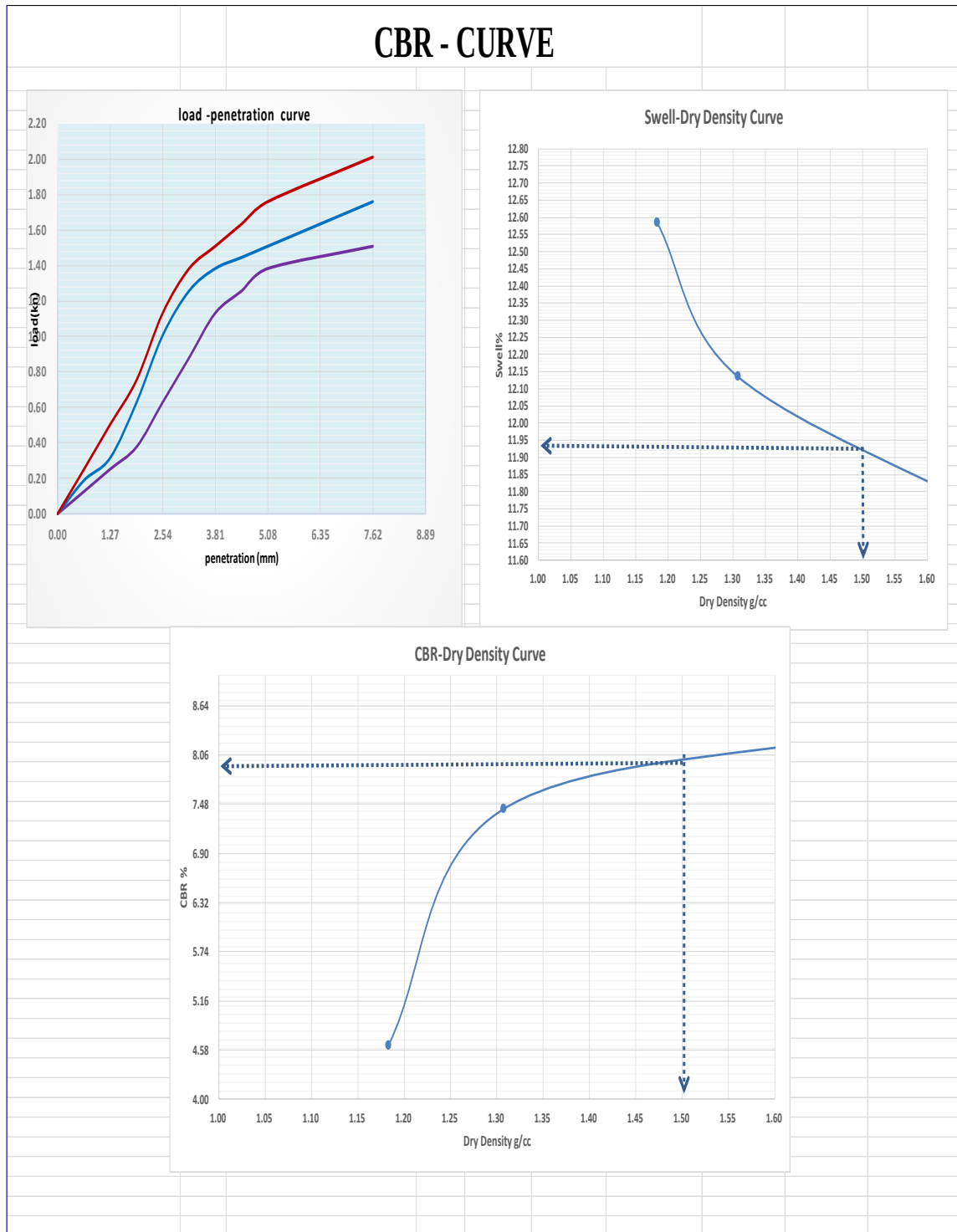


Figure I. 6 Graphical illustration of California Bearing Ratio Test Data at 20% fly ash