ADDIS ABABA UNIVERSITY

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STABILIZATION OF EXPANSIVE SOIL USING BIOMEDICAL WASTE INCINERATOR ASH

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APPROVAL PAGE

This is to certify that the thesis prepared by **Mr. Asefachew Belete Tseganeh** (GSR/4099/12) entitled: **"Stabilization of Expansive Soil using Biomedical Waste Incinerator Ash''** and submitted as a partial fulfillment for the award of the degree of masters of science in Railway Engineering specialization (Civil Infrastructure) complies with the regulations of the university and meets the accepted standards with respect to the originality content and quality.

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DECLARATION

I, the undersigned, declare that this thesis work entitled: **"Stabilization of Expansive Soil using Biomedical Waste Incinerator Ash''** was prepared by me, with the guidance and supervision of my advisor, Dr.-Ing. Henok Fikre and co-advisor, Mr. Ayele Tesema. The work contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted in whole or in part, for any other degree or professional qualification in any other university.

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ABSTRACT

Expansive soils undergo high volume change due to cyclic swelling and shrinkage behavior during the wet and dry seasons. Thus, such problematic soils should be completely avoided or properly treated when encountered as subgrade materials. In the present study, the biomedical waste incinerator ash (BWIA) and lime combination was proposed to stabilize expansive soil. Particle size analysis, Atterberg limits, free-swell, compaction, and California bearing ratio (CBR) tests were conducted on the natural soil and blended with 3%, 5%, 7%, 9%, and 11% BWIA. The free-swell index kept on decreasing up to 9% BWIA and it became a bit raised when the soil was blended with 11% BWIA, but the reduction is not that much significant. Since the expansiveness of the soil has not decreased tremendously, additional lime was amended and further investigations on the strength of the soil have been performed. In addition, scanning electron microscopy (SEM) tests for representative stabilized samples were also conducted to examine the changes in microfabrics and structural arrangements due to bonding. The addition of BWIA has a promising effect on the strength of the expansive soil. The strength of the expansive soil significantly increased when it was blended with 9% of BWIA amended by 2% and 3% lime. The CBR of the raw expansive soil was 2.3%. When it was treated with 9% BWIA and 3% lime, its CBR was significantly increased to 11.2%. The SEM micrographs showed that the raw expansive soil was dispersed (non-flocculated) fine particles with an open structural arrangement, but it became a homogeneous and flocculated structure when it was treated with 9% BWIA and 3% lime.

Keywords*: Expansive Soil, Subgrade, Soil stabilization, BWIA, CBR*

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ABBREVIATIONS

AASHTO	American Association State of Highway and Transportation Officials
ASTM	American Society for Testing Materials
BWIA	Biomedical Waste Incinerator Ash
CBR	California Bearing Ratio
FAA	Federal Aviation Agency
FSI	Free-Swell Index
FSR	Free-Swell Ratio
GSE	Geological Survey of Ethiopia
IS	Indian Standard
LOI	Loss of Ignition
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plasticity Index
PRA	Public Roads Administration
SEM	Scanning Electron Microscopy
USCS	Unified Soil Classification System

Symbols

%	Percentage
g	Gram
g/cc	Gram per Centimeter Cube
ml	Milliliter
kN	Kilo Newton
µm	Micro Meter
kPa	Kilo Pascal

CHAPTER ONE

1. INTRODUCTION

1.1 Background

Expansive soils are those that change significantly in volume under moisture fluctuation. It is, therefore, necessary to mitigate the problems posed by expansive soils and prevent cracking of structures (Zhang *et al*., 2019, Trivedi *et al*., 2013, Suwantara *et al*., 2019). Expansive soils occur in Ethiopia, South Africa, Tanzania, Asia, and some other African countries (Tewelde, 2019). It is mainly in Ethiopia's central part, on the major roads that link Addis Ababa with Ambo, Addis Ababa-Wolliso, Addis Ababa-Debrebirhan, Addis Ababa-Gohatsion, and Addis Ababa-Modjo. Areas like Mekelle and Gambella are also covered by expansive soils (Tewelde, 2019). Such kinds of soil are generally problematic and are not suitable for engineering construction (Ikeagwuani *et al*., 2021). Because of this, various studies have been conducted by many researchers around the world, leading to the concept of soil stabilization, which entails improving the physical and engineering properties of a soil by blending it with pozzolanic or cementitious additives (chemical stabilization) to improve its strength and reduce its susceptibility to water. If the stabilized soil can withstand vehicular (superstructural) loads within allowable deformation, it is considered stable. As stated by James and Pandian (2018) and Ikeagwuani *et al*. (2021), in ancient times, lime and cement were the two famous traditional stabilizers used for the stabilization of expansive soils all over the world. However, recently, the focus of geotechnical engineers has shifted to the utilization of different industrial and agricultural solid wastes (bi-products) like bagasse ash, teff straw ash, wheat straw ash, and rice husk ash etc. in combination with hydrated lime to gain improved performance and manage wastes that may cause environmental pollution (James and Pandian, 2018, Zorluer, 2020) such as bagasse ash (Liet Chi Dang, 2016, Zorluer, 2020, Tadesse, 2014), rice husk ash (Liu, 2019, Admas, 2020), pumice (Tewelde, 2019), etc. According to their findings, those solid waste materials blended with lime significantly improved the engineering properties of expansive soils. In addition to these waste materials, there is also abundantly available solid waste generated by healthcare facilities which has an adverse impact on human beings and the environment as well; this is called biomedical waste.

The safe disposal of waste generated from agricultural bi-products, healthcare centers, and different industries needs an urgent and cost-effective solution because of the harmful effect of these materials on the environment and on human beings as well. Biomedical waste generated and dumped from healthcare facilities is classified as general and hazardous waste. General healthcare (biomedical) waste is the overall waste that is generated from health-related facilities such as health centers, health stations, hospitals, pharmaceutical factories, and biomedical research centers and institutions (Derso *et al.*, 2018). According to Hodes and Kloos (1988), among the overall healthcare waste generated from those health-related facilities, 85% is non-hazardous (general), and the remaining is considered as hazardous waste.

In Ethiopia, municipal and medical wastes are disposed on open land and burnt with fire in an open firing system. Such kinds of waste disposal systems have an adverse impact on human beings and the environment as well. The numbers of healthcare centers (hospitals, clinics, and health stations) are increasing alarmingly. Thus, the medical waste generated from those health-related facilities that needs safe and proper disposal also increases. As stated by Derso *et al*. (2018), about 420 (39.3%) of the total health-related facilities in Ethiopia have been using an old industrial open-system incinerator to dispose of the generated biomedical waste from those facilities.

Eight high-temperature medical waste incinerators are constructed in Ethiopia to incinerate (burn) biomedical wastes generated from different healthcare facilities. The one built-in Adama city has a burning capacity of 1000 kg per hour. Similar facilities constructed in Bahir Dar, Dessie, Jimma, Nekemte, Dire Dawa, Hawassa, and Mekelle have a burning capacity of 500 kg per hour. Those incineration plants can burn the waste up to 1200 ˚C and also use disinfectants to completely sterilize the toxic elements. According to WHO (2018) report, modern incineration plants having a capacity of burning temperature of 850-1100 ˚C and fitted with special gas cleaning equipment, the incinerated ash has no harmful impact on human beings and the environment. All the plants are complied with the international emission standard as the machines are fitted with special gas cleaning equipment that can handle and release gasses without any harmful substances. The study conducted by (Samuel and Chernetie, 2016), stated that the average daily waste generation of Adama referral hospital was 1.23 kg/bed/day. According to the WHO report (2018), the average annual medical waste generated from healthcare facilities was 93,075 tons per year. Therefore, the incinerated medical waste ash is abundantly available and has no side effects for human beings and the environment because the incineration plants have very high burning capacity (up to 100 kg per hour) and temperature (800-1200 ˚C), and also fitted with special gas cleaning equipment. Since the technology is being adopted very soon, there is no local research conducted by utilizing the incinerated ash for concrete production or as an expansive soil stabilizer. But many researchers have conducted various studies by using medical waste incinerator ash with cement for concrete production. The addition of pharmaceutical waste incinerated ash has a positive effect on the strength of concrete (Talah Aissaa, 2018). The findings obtained by Talah Aissaa (2018), revealed that cement replacement by the incinerated ash showed that there is a positive contribution to the physical and mechanical properties, compressive strength, and durability of concrete. Based on the findings, it is concluded that the ash is suitable for the production of high-strength concrete, and its properties are better than normal concrete. The use of incinerated hospital ash for concrete production was also studied by (Elinwa, 2016).

He has conducted a complete silicate laboratory test, and the obtained results showed that the ash contains a relatively high percentage of CaO (47%), which has cementing value and, thus, can be used for cement manufacture. According to the findings, it is deduced that the hydration characteristics showed the ash to be pozzolanic in nature. An experimental study to assess the suitability of using incinerated hospital ash in cement as a construction material based on the engineering properties of fly ash-cement matrix and the leaching potential of toxic heavy metals from the stabilized mix has been also conducted (Elinwa, 2016). Based on the results on physical and strength properties obtained from the experiment, it is recommended that up to 10% (by weight) of incinerated healthcare waste ash can be incorporated into the mix to produce cement-mortar of optimum quality. The effect of medical waste incinerator ash on the swelling potential, cohesion and angle of internal friction were also studied by (Al Rawas, 2004), but other strength properties like CBR was not addressed by the study. Because the performance of flexible pavement depends on the strength of subgrade soil (CBR) where the self-weight of pavement layers and vehicular load finally rest on it (Rajakumar *et al*., 2021).

Similarly, this study aims at investigating the effect of BWIA on the index properties and strength of expansive soil, and also to investigate its relative effectiveness on the strength (CBR) and microstructural properties with an amendment of hydrated lime.

1.2 Statement of the Problem

Expansive soils are suitable for agriculture but are proved to be a serious threat to construction founded on it (Reddy *et al*., 2018). These soils have a very high swelling and shrinkage property during rainy and dry seasons respectively. Because of this high swelling and shrinkage nature, the structures like roads, buildings, and railways constructed on these soils experience cracks, making them unsuitable for foundation (Reddy *et al*., 2018).

In Ethiopia, especially Addis Ababa and regional cities, the construction of access roads in the road sector increases due to fast growing of these cities and traffic congestions. During construction of those roads, engineers have been faced challenges due to the occurrence of expansive soils at various sections of the alignment. Because, they have to take possible measures at a section of the road alignment where expansive soil is encountered so as to construct an environmentally and structurally sound road in the city. Three possible alternatives shall be taken by the engineers; **(1)** Re-alignment, **(2)** cut the expansive soil up to a desired depth and replace by selected material, **(3)** stabilization of the expansive soil using locally available pozzolanic additives, and **(4)** reinforcing weak subgrade soil by using geofoam and geosynthetic materials. The first, second, and the fourth alternatives are usually unapplicable and uneconomical because it may be totally impossible to re-align the road and selected materials for filling may not be available nearby the cities. So, according to different literatures, the third alternative, stabilization of sections where expansive subgrade soil exists using locally available pozzolanic materials considerably reduce the project cost by enhancing the engineering properties of the soil.

Eight high temperature medical waste incineration centers having a capacity of burning medical wastes up to 1000 kg per hour are constructed in Ethiopia at Adama, Bahir Dar, Jimma, Dessie, Nekemte, Dire Dawa, Mekelle, and Hawassa cities. Now a days, in Ethiopia, it is obvious that the number of healthcare facilities (governmental and non-governmental hospitals, health centers, and health stations) are increasing, so that healthcare (pharmaceutical) wastes generated from these facilities will also increases accordingly. Thus, biomedical (pharmaceutical) waste incinerator ash is abundantly and locally available material in Ethiopia. As cited and clearly presented in the background section, several researches have been conducted by many researchers in the world on the investigation of incinerated medical waste ash as an additive to replace cement in concrete production. But not that much as an expansive soil stabilizer for road subgrade construction. The present study aims at investigating the effect of biomedical waste incinerator ash on the index properties and strength of expansive soil, and also to investigate its relative effectiveness on the strength and microstructural properties with amendment of hydrated lime.

1.3 Objective of the Study

1.3.1 General Objective

This study aims at evaluating the suitability of biomedical waste incinerator ash as an expansive soil stabilizer for road subgrade construction.

1.3.2 Specific Objectives

- ✓ To examine the oxide compositions of biomedical waste incinerator ash by conducting a complete silicate laboratory test.
- ✓ To investigate the effect of biomedical waste incinerator ash (BWIA) on the index properties and strength of expansive soil.
- ✓ To investigate the effect of lime on the strength of the stabilized soil with BWIA.
- ✓ To investigate the effect of BWIA and lime on the microstructural properties of the expansive soil using Scanning Electron Microscopy (SEM).

1.4 Research Questions

This study tries to address the following research questions:

- ✓ What are the major oxide compositions of the BWIA, which may be similar to the lime or cement?
- ✓ What is the effect of biomedical waste incinerator ash on the index properties and strength of expansive soil?
- ✓ What is the effect of lime on the strength of the stabilized soil with BWIA?
- ✓ How is the effect of BWIA amended with lime on the microstructural properties of the highly expansive soil analyzed using SEM?

1.5 Scope of the Study

The scope of this research is to study how “biomedical waste incinerator ash” can be effectively utilized in combination or blended with expansive soils amended with small dosages of lime to enhance and improve the strength, of the soil for use in road subgrade construction.

1.6 Limitations of the Study

Due to time and budget constraints, only an expansive soil sample at one test pit is taken and only its index properties, and strength (CBR) with and without BWIA and lime are determined in the laboratory. Extra laboratory and field tests on subgrade strength and stiffness by taking a greater number of representative samples at various locations would have to be conducted.

1.7 Significance of the Study

- ✓ A shortage of selected fill materials can be resolved.
- ✓ Even though detail economic analysis was not carried out, it can indicate the economic advantage of soil stabilization using BWIA over cutting and replacing techniques by shipping selected materials from a long distance.
- ✓ Minimize waste and environmental pollution by reducing open land for the disposal of medical waste ash residue.

1.8 Structure of the Thesis

The presentation of this research work is organized in five chapters, each consists of sections and sub-sections. The first chapter presents a background of the study in general, statements of the problem, general and specific objectives of the study, study limitations, and significances. The second chapter consists of related previous works conducted on the chemical stabilization of expansive soils using different pozzolanic additives, the quantity of healthcare wastes, and their impact on the environment. The third chapter presents the materials used in the study, study area, detailed work methodology flow chart, and the laboratory tests conducted. Chapter four is about the laboratory experimental results obtained and a discussion about the effect of BWIA on the Atterberg limits. Moisture-density relationships, and amendment of lime on the strength (CBR) of the stabilized expansive soil, and evaluation of its suitability for road subgrade construction as per the ERA (2013) pavement design manual specifications.

The fifth chapter presents the conclusion drawn based on the laboratory test results and recommendations for future investigations. Lastly, the references cited in the document and tabulated laboratory results and charts are attached as an appendix.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Introduction

Various studies on the stabilization of soil have been conducted by many researchers around the world. Among the previously done research regarding expansive soil stabilization, those very related to this study are reviewed and documented here below. Expansive soils are clayey soils, mudstones, or shales that experience high volume changes due to moisture fluctuations (Asres, 2017). Usually, the clay content is relatively high and the clay mineral montmorillonite dominates. Such kinds of soils have high strength when dry, very low strength when wet; wide and deep shrinkage cracks in the dry season; high plasticity, and very poor trafficability when welled. Unless proper attention is given to the properties of problematic soil, the infrastructures founded on it will fail (Uge, 2017). Expansive soils have highly reactive clay minerals. The three most important sets of clay minerals are Montmorillonite, Illite, and Kaolinite. Montmorillonite is a clay mineral that is mostly present in expansive soil. When these minerals are exposed to moisture, water is absorbed between the inter-layering lattice structures and exerts an upward pressure, which is the cause for most damages associated with expansive soil (Asres, 2017).

2.2 Origin and Distribution of Expansive Soils

Expansive soils are found in many parts of the world; hence the problem of expansive soil is wide spread. The origin of such soils is associated with conditions and chemical processes that result in the formation of the native clay minerals having a particular character that expands and contracts when reacted with water. Variations in the conditions and processes may also form other clay minerals, most of which are non-expansive. The composition of the parent material and the degree to which materials are subjected to physical and chemical weathering are the major conditions or processes that can determine the clay mineralogy in a soil mass (Asres, 2017). The major constituents (building-blocks) of expansive soils can be classified into two groups (Yohannes, 1986). The first group includes basic igneous rocks, whereas the second one is associated with the sedimentary rocks that contain the famous clay mineral; montmorillonite. Expansive soils are known to occur in many parts of the world. Some of these countries are Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, U.S.A., U.S.S.R., Venezuela, and Zimbabwe (Chen, 2012). The aerial coverage of expansive soils in Ethiopia is estimated to be

24.7 million acres (Asres, 2017). They are widely spread in the central part of Ethiopia. Also, areas like Mekelle and Gambella are covered by expansive soil (Asres, 2017). The expansive soils found in Ethiopia are derived from both groups described above.

Figure 2 - 1: Distribution of Expansive soil in Ethiopia (Admas, 2020)

2.3 Mineralogy of Expansive Soils

Expansivity of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002mm or less (Solomon, 2011). The grain size alone does not determine clay minerals, so the most important property of fine-grained soils is their mineralogical composition (Chen, 2012). Clay minerals are hydrous aluminum silicates plus other metallic ions, and can form as either primary or secondary minerals (Chen, 2012). Clay minerals can be readily seen, identified and investigated by using a scanning electron microscope (SEM), since they are very small crystals, with a platy-like shape and colloidal size. On the basis of X-ray diffraction (Chen, 2012), it has been found that they are formed of two-dimensional sheets, which are stacked one upon another. There are dozens of clay minerals, depending on the way the basic sheets are stacked together and depending on the cations present in the tetrahedral and octahedral sheets. However, since the objective here is to show the essential features of their microstructure in order to elucidate qualitatively its influence on soil behavior, it is sufficient to describe only a few examples of clay minerals (Lancellotta, 2008).

Kaolinite: is made up of layers of a single tetrahedral and an octahedral sheet (for this reason, kaolinite is also called a 1:1 clay mineral), as represented in Figure 2-2. The basic layer has a thickness of 0.72 nm, and the bonding (hydrogen bonds) between successive layers is of sufficient strength to prevent hydration, so that there is no interlayer swelling and layers can be stacked up to make large crystals. A kaolinite crystal typically contains 70 to 100 layers (Lancellotta, 2008).

Montmorillonite: it has a more complicated structure (Figure 2-2), made up of an octahedral sheet sandwiched between two tetrahedral sheets. Hence, it is commonly known as a 2:1 clay mineral (Lancellotta, 2008). In addition, there is a negative charge deficiency in the octahedral sheet, due to isomorphous substitution of Al^{3+} with Mg^{2+}, so that exchangeable ions (sodium or calcium) lie between the layers, or are attached at the edges of the crystal. Molecular water may also occur between layers (Lancellotta, 2008). As a consequence, the crystals can be rather small (the thickness can be of the order of 1 nm) and clay soils containing montmorillonite are susceptible to swelling, with important engineering implications (Lancellotta, 2008).

Illite: is another example of 2:1 clay mineral(Figure 2-2), similar to montmorillonite, but in this case the layers are strongly bonded by a potassium atom, which fills the hexagonal hole in the tetrahedral sheet, and crystals have a thickness of about 10 to 30 nm (Lancellotta, 2008).

From the above description of microstructural features, it is deduced that the portions of positive and negative charges do not overlap each other, so that a clay particle exhibits a surface negatively charged. In addition, because Al^{3+} can partly substitute Si^{4+} in the tetrahedral sheet and Mg^{2+} can substitute Al^{3+} in the octahedral sheet (this substitution is called isomorphous substitution because it is the same form of the atom substituting the other), a net unit charge deficiency results (Lancellotta, 2008). The letters S and G in the figures (a), (b), and (c) below represent the Silica and Gibbsite sheets, respectively. The chemical compositions of kaolinite are Si, Al, O and H, and that of montmorillonite are Mg, Ca, Al, Si, O, and H, and illite has K, Fe, Mg, Ca, Al, Si, O, and H.

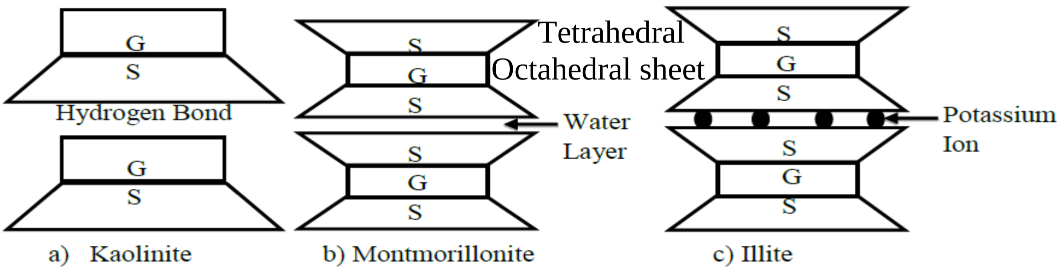


Figure 2 - 2: The structure of the three major clay minerals (Soga, 2005)

2.4 Recognition and Identification of Expansive Soils

The identification and description of the subsurface state is the initial step in any geotechnical engineering project, and it is critical for the selection of appropriate sampling, testing, design, and construction processes. The geotechnical methods of expansive soil identification can be broadly divided into field and laboratory identification methods (Admas, 2020). The various methods used for the identification of expansive soils are presented below.

2.4.1 Field Identification

Site reconnaissance (field visit) is an important stage in every construction project such as buildings, roads, railways, etc. The main purpose of the site visit is to carry out a reconnaissance and detailed survey about how the area looks like and to have sufficient information about the site where the structure is going to be constructed. During this stage, experienced civil (geotechnical engineers) can visually identify the occurrence of potentially expansive soils along the project section so that they can decide on some preconditions for further laboratory investigations. According to Lancellotta (2008), there are various important indicators to identify expansive soils during field investigation that are numerously used by experienced engineers. Some of those indicators are the following:

- ✓ Most expansive soils have black, gray or dark gray color
- ✓ The appearance of cracking in nearby structures,
- ✓ A pattern of polygonal desiccation, or “shrinkage cracks” in the dry season and close down in the rainy season,
- ✓ Shiny appearance on cut surfaces,
- ✓ The wet samples of the soil are soft, cohesive, sticky and it will be relatively difficult to clean the soil from the hands,
- ✓ High dry strength and low wet strength
- ✓ They are very sticky in nature

2.4.2 Laboratory Identification

Soil expansivity can be investigated in the laboratory using three commonly used methods.

A. **Mineralogy Identification**

This method is used for identifying the mineralogy of clay particles such as characteristic crystal dimensions, characteristic reaction to heat treatment, size and shape of clay particles, and charge deficiency and surface activity of clay particles. These properties are a fundamental factor in controlling expansive soil behavior (Asres, 2017). Mineralogical identification can be useful in the evaluation of the material but is not sufficient in itself, which is why for better and more reliable results it should be used in combination (Chen, 1975). The fundamental factor controlling expansive soil behavior is clay mineralogy, and it can be identified using a variety of techniques. The techniques that can be used for clay mineralogy identification are:

- ✓ X-ray diffraction (XRD)
- ✓ Differential thermal analysis
- ✓ Dye absorption
- ✓ Scanning electron microscopy
- ✓ Optical electron microscopy
- ✓ Base exchange capacity, etc.

B. Indirect Methods

Simple soil property tests are among the methods that a practicing engineer can use to identify expansive soil. Such tests are easy and can be performed in soil mechanics laboratory, and they yield an excellent index of expansive properties (Asres, 2017). In this method, measurement of the plasticity, swelling, and shrinkage characteristics of the soil is conducted for identification of soils and provides a wide acceptable means of rating by using liquid limit, plastic limit, shrinkage limit, free swell tests, colloid content tests, etc. (Asres, 2017).

C. Direct Measurement

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement. These methods offer the most useful data by direct measurement, and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions.

2.5 Classification of Expansive Soils

Soil classification is a technique of classifying soils into several groups and subgroups based on their physical, chemical, and engineering properties. Index properties of soils such as grain size and Atterberg limits are used to classify soils for general engineering purposes (Gawande *et al*., 2017). The basic index properties of non-cohesive (coarse-grained) soils are grain size and relative density, whereas consistency and Atterberg's limit are the main index properties of cohesive soils (Gawande *et al*., 2017).

There are various soil classification systems that have been used to classify soils. Among those classification systems, the Casagrande USCS and the American association state of highway and transportation officials' (AASHTO) system are currently in use in civil engineering practice (Gawande *et al*., 2017). Both systems, namely USCS and AASHTO, base their classification of soils for engineering purposes on particle size characteristics, liquid limit (wL) and plasticity index (Ip) of soils (Gawande *et al*., 2017). Soils can be grouped into different subgroups depending on various parameters. As stated in Gawande *et al*. (2017) the subgrouping of coarse-grained soils is done with the help of parameters such as uniformity coefficient (C_u) and coefficient of curvature (C_c) to account for the gradation of soils whereas, the subgrouping of fine-grained soils is entirely based on a plasticity chart (i.e., Ip plotted against wL).

2.5.1 Classification of Expansive Soils by USCS and AASHTO system

- **AASHTO Soil Classification System:**

Table 2 - 1: AASHTO soil classification chart

General Classification	Granular Materials (35 % or less Passing the 0.075 mm Sieve)							Silt-Clay Materials (> 35 % Passing the 0.075 mm Sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve Analysis, % Passing											
2.00 mm (No. 10)	50 max	…	…	…	…	…	…	…	…	…	…
0.425 mm (No. 40)	30 max	50 max	51 min	…	…	…	…	…	…	…	…
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	…		…	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of Significant Constituent Materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey Soils	
General Rating as a Subgrade	Excellent to Good							Fair to por			
Note: Plasticity Index of A-7-5 Subgroup is equal to or less than the LL-30. Plasticity Index of A-7-6 Subgroup is greater than LL-30.											

From Table 2-1, soils in group A-6 are typically lean clays, and those in group A-7 are typically highly plastic clays (Jamal, 2019). In soil groups, one may calculate a group index (GI) to check the quality of a soil in its group to be used as a subgrade material or not. Mathematically, the GI can be calculated using the following formula:

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.1)$$

Where 'F' is the percentage of a soil that passes through a 0.075mm (No. 200) sieve expressed as a whole number, and LL and PI are the liquid limit and plasticity index of a soil, respectively.

As the GI value increases, the quality of material for subgrade becomes very poor. A GI of zero implies that the material has a good quality in its group for subgrade construction. The GI value of soils greater than or equal to 20 are generally poor subgrade materials. According to the ERA manual (2013), there is no upper limit value of GI, but if it is greater than 20, it can be reported as 20. Thus, according to the AASHTO classification system, soils classified in group A-6 and A-7 are lean clays and typically high plastic clays, respectively, i.e., such kinds of soils are potentially expansive (Jamal, 2019).

- **Unified Soil Classification System**

The USCS was originally developed by Casagrande and was later modified by the US Bureau of Reclamation (USBR) and the US Army Corps of Engineers, to enhance its applicability to many more fields. As per the USCS, soils having a group symbol of CL or OH are potentially expansive and have high-volume change under moisture fluctuation (Asres, 2017).

Table 2 - 2: Swelling potential of Soils as per USCS

Category	Soil classification as per USCS
Little or no expansion	GW, GP, GM, SW, SP, SM
Moderately Expansion	GW, SC, ML, MH
High volume change	CL, OL, CH, OH
No rating	Pt

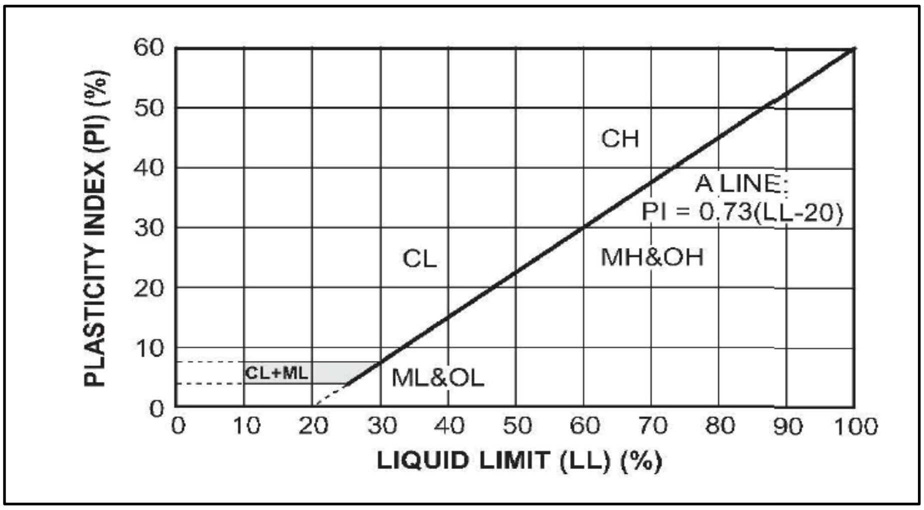


Figure 2 - 3: Plasticity Chart (Asres, 2017)

Where, CH-Inorganic clays of high plasticity (fat clays), CL-Inorganic clays of low plasticity, MH-Inorganic silts of high plasticity, ML-Inorganic silts of low plasticity, OL-Organic silts of low plasticity, OH-Organic clays of low plasticity, Pt-Peat and other highly organic soils, etc., SC-Clayey sands, SM-Silty sands, SW-Well-graded sands, SP-Poorly-graded sands, GC-Clayey Gravels, GM-Silty gravels, GW-Well-graded gravels, and GP-Poorly-graded gravels.

2.5.2 Expansive soil classification based on ERA manual (2013)

According to ERA (2013), expansiveness index (ϵ_{ex}) is correlated with plasticity index, shrinkage limit, and percent of particles finer than No. 40 (0.425 mm) sieve. Based on the expansiveness index from those parameters' soils are classified from low to high expansive. The equation is given by:

$$\epsilon_{ex} = 2.4w_p - 3.9w_s + 32.5 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. Eqn. (2.2)$$

Where, ϵ_{ex}= Expansiveness index

Wp = PI*(% passing 0.425 mm sieve)/100

Ws = SL*(% passing 0.425 mm sieve)/100

PI = Plasticity Index, and SL = Shrinkage limit

Table 2 - 3: Expansive Soil Classification based on ERA manual (2013)

Expansiveness index, ϵ_{ex}	Classification
< 20	Low expansive
20 - 50	Moderately expansive
>50	Highly expansive

2.5.3 Expansive Soil Classification Based on Plasticity Index

Expansive soils can be classified from low expansive to very high expansive based on its plasticity index (Chen, 1975).

Table 2 - 4: Swelling potential of expansive soils based on plasticity

Swelling Potential	Plasticity Index
Low	0 – 15
Medium	10 – 35
High	20 – 55
Very High	35 and above

2.5.4 Swelling potential of Expansive Soils Based on Liquid Limit

Based on the liquid limit, Expansive soils can be classified from low expansive to very high expansive (Dakshanamurthy and Raman, 1973).

Table 2 - 5: Swelling potential of expansive soil based on liquid limit

Swelling Potential	Liquid Limit
Low	$20 < LL \leq 35$
Medium	$35 < LL \leq 50$
High	$50 < LL \leq 70$
Very High	> 70

2.5.5 Potential of Expansiveness Based on Activity (A) of Soil

Activity (A) is the ratio of the plasticity index to the percent by weight of particles having an equivalent diameter smaller than 0.002 mm (ASTM D 4318-98).

$$A = \frac{PI}{F} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots Eqn. (2.3)$$

Where, A = Activity, PI = Plasticity Index, and F = Clay Fraction (percentage finer than 0.002 mm sieve)

Table 2 - 6: Classification of soils based on activity (Arora, 2003)

S. No.	Activity	Soil type
1	< 0.75	Inactive
2	0.75 to 1.25	Normal
3	>1.25	Active

2.5.6 Potential of expansiveness based on free-swell Ratio (FSR)

It is an important soil parameter that helps to identify the potential of soil expansiveness. It's the ratio of the final volume of a soil in distilled water of a graduated cylinder having a 100 ml capacity to the final volume of a soil in kerosene of the same cylinder capacity after 24 hours. according to Prakash *et al*. (2016), FSR is used to classify expansive soils from very high to negligible expansive soils.

Table 2 - 7: Soil classification based on Free-Swell Ratio (Prakash *et al*., 2016)

FSR	Clay property	Expansiveness	Major clay mineral type
1	Non-swelling	Negligible	Kaolinite
1 - 1.5	Mixture of swelling and non-swelling	Low	Kaolinite and montmorillonite
1.5 - 2.0	Swelling	Moderate	Montmorillonite
2.0 - 4.0	Swelling	High	Montmorillonite
> 4.0	Swelling	Very High	Montmorillonite

2.6 Soil Stabilization

2.6.1 Introduction

When soils at a given site fail to meet standards set by agencies, pozzolan additives need to be added in order to further enhance the soil's properties (Hensley, 2010). Properties that are affected by this stabilization include, for example, lower plasticity index (PI), decreased swell potential, increased compressive strength, and durability. It is often more economical to use soil stabilization rather than replacing material or relocating a project, which is not possible in most cases. Most agencies will have designated the soil subgrade to be stabilized and contractors bidding projects are well aware of the stabilization before construction. Therefore, soil stabilization is an economic means of improving soils at the site of construction through the use of stabilizers. Stabilizers are the agents used to enhance the engineering properties of soil, and stabilizers are used on soil when the soil does not possess the desired characteristics for a particular construction. The kind of stabilizer to be used depends on which properties of the soil are to be improved, the nature and type of the soil (Akinmade, 2008). Expansive soils are those that exhibit particularly large volumetric changes, both shrinkage and swell, due to variations in their moisture content. They exhibit poor bearing capacity (similar to some stability problems). Particular care is needed with such expansive soils and, if construction in these soils cannot be avoided, earthworks must be designed to minimize changes due to moisture fluctuation and consequent volume changes. As stated in the ERA (2013), the measures to minimize the effect of expansive soils must be both economic and proportionate to the risk of pavement damage and increased maintenance costs. The problems associated with construction over expansive soils are usually the seasonal changes in these soils rather than their low bearing strength. According to the ERA manual (2013), such kinds of soils are often relatively strong at equilibrium moisture content. Distress occurs as seasonal wetting causes soils at the edge of the pavement to wet and dry out at rates differing from those further under the bituminous surfacing. As per the ERA manual (2013), this mechanism causes differential movements over the roadway cross section and associated crack development, beginning at the shoulder and proceeding towards the carriageway. Earths (soils) are the most widely used materials in civil engineering (Admas, 2020). However, due to higher compressibility and poor bearing capacity, the available soil at various locations may not be suitable for the requirements of particular construction and may create significant problems for the pavements or structures when constructed on it. At this time, contractors have the following four possible alternatives to avoid the problems of the expansive soil encountered on a road project (Admas, 2020):

a) Realigning
b) Cutting and replacing the poor soil by selected granular material
c) Stabilizing the poor soil with a locally available pozzolanic material
d) Reinforcing weak subgrade soil using geofoam and geosynthetic materials as a drainage blanket

Option (a) is usually impossible to implement and uneconomical, whereas option (b) has been the most widely used technique where selected materials are available nearby the project site. However, this may cause several environmental and economic concerns, specifically with regard to its transportation and disposal. In order to prevent this possible scenario, an alternative solution is through option (c), which is the most economical and scientific approach (Admas, 2020). Option (d) is usually uneconomical due to the high cost of the materials.

2.6.2 Types of Soil Stabilization

Soil stabilization can be broadly classified into two types, namely; mechanical and chemical stabilization (Suwantara *et al*., 2019).

2.6.2.1 Mechanical Stabilization

Mechanical stabilization is the ancient type of stabilization without any chemical additive. A soil structure is said to be mechanically stable when it can resist lateral displacement under load (Akinmade, 2008). Mechanical stabilization is one of the oldest types of soil stabilization which involves physical alteration by rearranging the soil mass through static or dynamic compaction. A heavy weight is dropped continually on the ground at regular intervals to ensure a uniformly packed, non-compressible surface, called dynamic compaction (Shimola, 2018). A stable soil can be obtained through controlled grading of the coarse aggregate, fine aggregate, silt, and clay, correctly proportioned and fully compacted. The process of controlled grading is referred to as mechanical stabilization. Therefore, mechanical stabilization involves soil densification through compaction without the addition of chemicals or any materials. Mechanical energy brings about the reduction of voids in the soil, with little or no reduction in water content. It also lowers the compressibility, porosity, and permeability of the soil material and an increase in dry density (Akinmade, 2008).

2.6.2.2 Chemical Stabilization

When organic and inorganic chemical compounds are added to the soil in order to enhance its engineering properties, the soil is said to be chemically stabilized. This method of stabilization depends on the chemical reaction between the stabilizer and soil minerals to achieve the desired effect (Asres, 2017). Calcium chloride is able to absorb and hold moisture in soil bases and surfacing which are stabilized mechanically. Frequent application of calcium chloride plays a vital role in making up for the chemical losses caused by leaching action. The relative humidity of the atmosphere should be > 30% for the salt to be effective (Shimola, 2018).

Chemical stabilization is the blending of cementitious and pozzolanic materials like lime, cement, bagasse ash, calcined termite clay powders, agricultural bi-products, etc., with soils weak in strength to enhance their stability, strength, swelling, permeability, and durability (Tadesse, 2014). Chemical stabilization can achieve soil improvement through three chemical reactions (Asres, 2017).

- **Flocculation and Agglomeration**

As stated by Asres (2017), cation exchange results in a change in the electrical charge around the clay particles. Therefore, this results in an increase in the interparticle attraction, causing flocculation and agglomeration. This leads to the reduction of clay-sized particles and hence the soil surface area, which accounts for the decrease in plasticity and swelling of the soil.

- **Pozzolanic Reactions**

Time-dependent pozzolanic reactions play a major role in the stabilization of the soil (Asres, 2017). They are responsible for the improvement in the various soil properties. Pozzolanic constituents can form compounds like calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) that have the capability to enhance the strength of soil.

$Ca^{2+} + 2(OH)^- + SiO_2$ (Clay Silica) → CSH

$Ca^{2+} + 2(OH)^- + Al_2O_3$ (Clay Alumina) → CAH

The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then sets to form a flocculated structure which increases the soil strength through bonding.

- **Cement and Lime Stabilization**

There is a significant history of the use of chemical additives for soil stabilization to enhance its strength. In ancient times, lime and cement were the two most famous traditional stabilizers used for the stabilization of expansive soils all over the world. Recently, fly ash has been used as an economical alternative to improve subgrade performance (Trivedi *et al*., 2013). There is extensive literature on the utilization of industrial wastes in combination with lime for beneficial effects on lime stabilization (James and Pandian, 2018). Soil cement is defined as the soil after being stabilized with cement (Shimola, 2018). Cementing action takes place by the chemical reactions of cement with the soil during the hydration reaction process, which is a function of curing time. For different types of soil, appropriate amounts of cement should be provided. They are as follows: gravel – 5 to 10%, sand – 7 to 12%, silt – 12 to 15%, and clay – 12 to 20% (Shimola, 2018).

Researchers have been using cement and lime as additives for the stabilization of problematic soils to enhance their engineering properties, depending on the reaction between the soil particles and their properties in the presence of moisture (Majeed *et al*., 2014). The most difficult area for contractors and agencies to complete a project is when unexpected stabilization is required due to wet subgrade, poor design, or weak soils. Soil stabilization is a technique applied to improve the engineering properties of soil, such as shear strength, swelling characteristics, and strength of soils (Harshita, 2018). The addition of lime significantly improves the consistency, density, and strength characteristics of clayey soil (Harshita, 2018). He concluded that there is a significant increase in the dry density of the soil with a reduction in the corresponding values of moisture content. In his study, the addition of lime was made between percentages of 0% and 10%, and an optimum increase in the properties was obtained at a percentage of 8% lime in the soil. He also stated that the UCS and CBR increased from 0.056 MPa to 0.193 MPa in one day with an increase in lime percentage from 0% to 20% and from 7% to 12% with an increase in lime percentage from 0% to 10%. The optimum content of lime to be added for the stabilization of expansive soil is usually between 3% and 7% (Suwantara et al., 2019). In the study by Suwantara *et al*. (2019), the addition of white soil was made between 3% and 5% by the dry weight of the expansive soil sample and 5% H_2SO_4 from the optimum moisture content at 0, 7, 14 and 28 days of aging. Based on the experimental results, the CBR value (both soaked and unsoaked) had increased by 7.4% and 83.25% respectively. In addition to this, the swelling potential and swelling pressure had decreased by 90.71% and 65.71% respectively. They stated that the optimum mixture is the mixture with an addition of 3% white soil and 5% H_2SO_4 at 28 days of aging.

- **Soil Stabilization using Sugarcane Bagasse Ash**

Bagasse ash is an abundant fibrous waste product derived from the sugar-refining industry and is readily available for use without cost or low cost. This material is increasingly identified to pose a risk to the environment, which requires public attention and research on its safe disposal and opportunities for use as a recycled material. Several studies on bagasse ash have been performed to investigate its effect on the properties of expansive soil (Liet Chi Dang, 2016). As per the experimental results, they indicated that bagasse ash admixture caused a significant enhancement in the engineering properties of expansive soil. The study conducted by Tadesse (2014) has also shown the potential effectiveness of bagasse ash and lime on the properties of expansive soil based on Atterberg's limit compaction characteristics and CBR. The laboratory test was conducted by adding 3% lime, 15% bagasse, ash and a combination of the two percentages. Based on the results, it is concluded that the addition of lime and bagasse ash, alone and in combination to the soil has a substantial effect on its physical and mechanical properties. In general, when expansive soil was treated with 3% lime alone, the plasticity index decreased from a natural soil value of 78.1% to 34.6% for the uncured sample and to 33.1% for the 7 days cured sample.

- **Soil stabilization with bitumen**

Sand bitumen stabilization, soil Bitumen stabilization, oiled earth are different types of bitumen available (Shimola, 2018). When a substance is added to soil, it causes both cohesion and reduced water absorption, depending on the soil's characteristics (Shimola, 2018).

- **Soil stabilization by electrical methods**

An expensive soil stabilization method called the electro-osmosis method is performed by electrical stabilization of clayey soils and is very useful for drainage of cohesive soils (Shimola, 2018).

- **Soil stabilization by grouting**

In this method, stabilizers are injected into the soil. The main limitations of this method are that it is not applicable to clayey soils because of their low permeability and that it is expensive (Shimola, 2018). Various grouting techniques are classified, and these methods are suitable for stabilizing zones buried in zones to a certain extent. The various types of grouting techniques are clay grouting, chemical grouting, chrome lignin grouting, polymer grouting, and bituminous grouting (Shimola, 2018).

2.7 Healthcare (Biomedical) Waste

Healthcare (biomedical) waste is the overall waste that is generated from health-related facilities such as health centers, health stations, hospitals, pharmaceutical factories, and biomedical research centers and institutions (Derso *et al*., 2018).

2.7.1 Sources of (Healthcare) Biomedical Waste

The major sources of healthcare (biomedical) waste are (WHO, 2018):

- ✓ Hospitals and other health facilities
- ✓ Laboratories and research centers
- ✓ Mortuary and autopsy centers
- ✓ Animal research and testing laboratories
- ✓ Blood banks and collection services

2.7.2 Types of Biomedical Waste

The waste generated from health facilities are the following:

- **Infectious waste:** waste contaminated with blood and other bodily fluids (e.g., from discarded diagnostic samples), cultures and stocks of infectious agents from laboratory work (e.g., waste from autopsies and infected animals from laboratories), or waste from patients with infections (e.g., swabs, bandages and disposable medical devices) (WHO, 2018);
- **Pathological waste:** human tissues, organs or fluids, body parts and contaminated animal carcasses (WHO, 2018);
- **Sharps waste:** syringes, needles, disposable scalpels and blades, etc. (WHO, 2018);
- **Chemical waste:** for example, solvents and reagents used for laboratory preparations, disinfectants, sterilant and heavy metals contained in medical devices (e.g., mercury in broken thermometers) and batteries (WHO, 2018);

- **Pharmaceutical waste:** expired, unused and contaminated drugs and vaccines (WHO, 2018);
- **Cytotoxic waste:** waste containing substances with genotoxic properties (i.e., highly hazardous substances that are, mutagenic, teratogenic or carcinogenic), such as cytotoxic drugs used in cancer treatment and their metabolites (WHO, 2018);
- **Radioactive waste:** such as products contaminated by radionuclides including radioactive diagnostic material or radiotherapeutic materials; and
- **Non-hazardous or general waste:** waste that does not pose any particular biological, chemical, radioactive or physical hazard (WHO, 2018).

2.7.3 Effect of Biomedical Waste Disposal on the Environment

Treatment and disposal of healthcare waste may pose health risks indirectly through the release of pathogens and toxic pollutants into the environment (WHO, 2018).

- The disposal of untreated health care wastes in landfills can lead to the contamination of drinking, surface, and ground waters if those landfills are not properly constructed (WHO, 2018).
- The treatment of health care wastes with chemical disinfectants can result in the release of chemical substances into the environment if those substances are not handled, stored and disposed in an environmentally sound manner (WHO, 2018).
- Incineration of waste has been widely practiced, but inadequate incineration or the incineration of unsuitable materials results in the release of pollutants into the air and in the generation of ash residue. Incinerated materials containing or treated with chlorine can generate dioxins and furans, which are human carcinogens and have been associated with a range of adverse health effects. Incineration of heavy metals or materials with high metal content (in particular lead, mercury and cadmium) can lead to the spread of toxic metals in the environment (WHO, 2018).
- Only modern incinerators operating at 850-1100 °C and fitted with special gas-cleaning equipment are able to comply with the international emission standards for dioxins and furans (WHO, 2018). All the incineration plants in Ethiopia comply with this scenario.
- Alternatives to incineration such as autoclaving, microwaving, and steam treatment integrated with internal mixing, which minimize the formation and release of chemicals or hazardous emissions, should be given consideration in settings where there are sufficient

resources to operate and maintain such systems and dispose of the treated waste (WHO, 2018).

The eight modern medical waste incineration centers constructed in Ethiopia (Adama, Bahir Dar, Mekelle, Dire Dawa, Jimma, Nekemte, Hawassa, and Dessie) have the capacity to burn pharmaceutical waste up to 1200 °C with a special gas-cleaning equipment. Thus, the plants comply with the international emission standards dioxins and furans (harmful substances released to the air). Therefore, the incinerated ash has no harmful impact on the environment and human beings as well.

CHAPTER THREE

3. MATERIALS AND METHODOLOGY

3.1 Introduction

The sample collection and preparation, laboratory experiment techniques, and general methodology employed in the study are all detailed in this section. The experiments were carried out in the highway and geotechnical engineering laboratory at Addis Ababa Science and Technology University. The sub-sections below go into the specifics of each test that was performed on the samples.

3.2 Study Area

This study was conducted on the expansive soil obtained from the Kality-Meshualekiya road stretch, Akaki-Kality Sub-city, Addis Ababa. Addis Ababa is the capital city of Ethiopia located at an elevation of 2355 m above mean sea level, having metro population of 4,587,857, a total area of 527 km^2, and latitude & longitude of $9^{\circ}0'19.4436''$ N and $38^{\circ}\ 45'\ 48.9996''$ E, respectively. The average annual temperature and precipitation are 15.6 ˚C (60.08 ˚F) and 1874 mm per year, respectively.

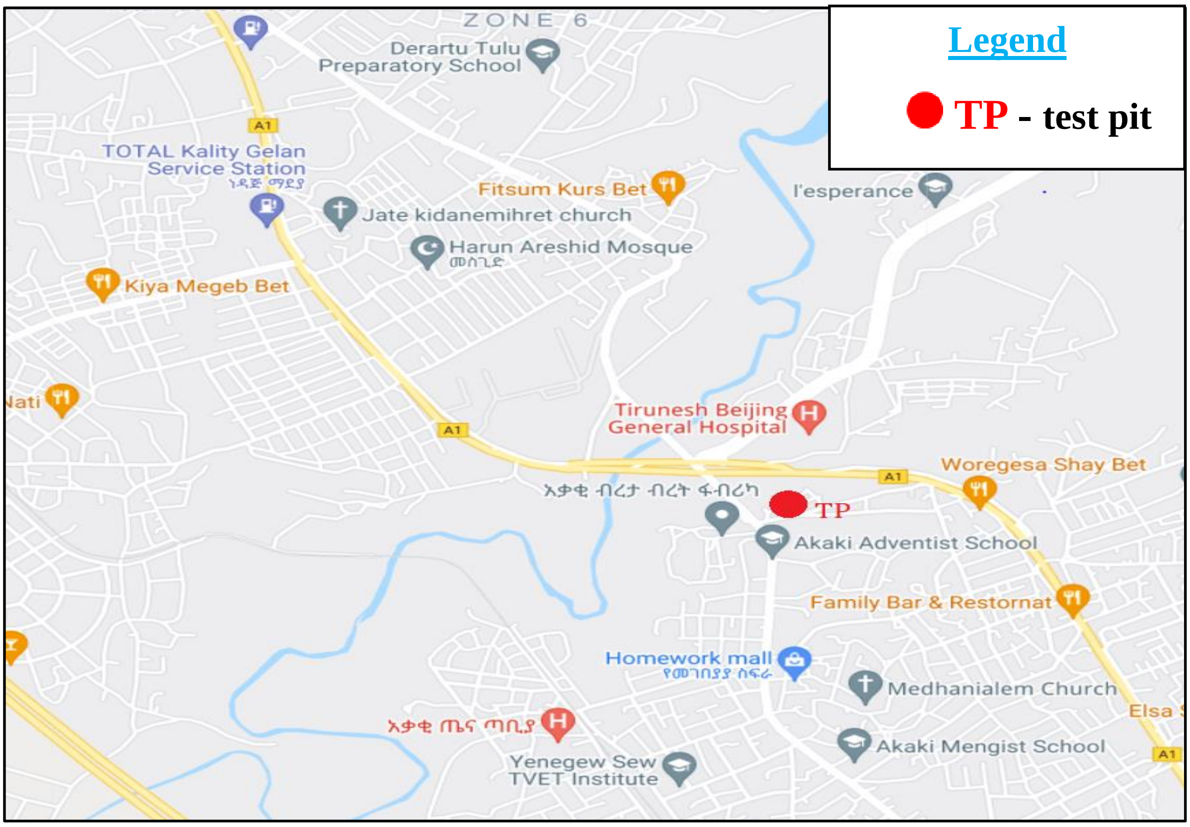


Figure 3 - 1: Map of the Study Area

3.3 Work Methodology Flow Chart

The flow chart shown below represents the overall methodology of the present study.

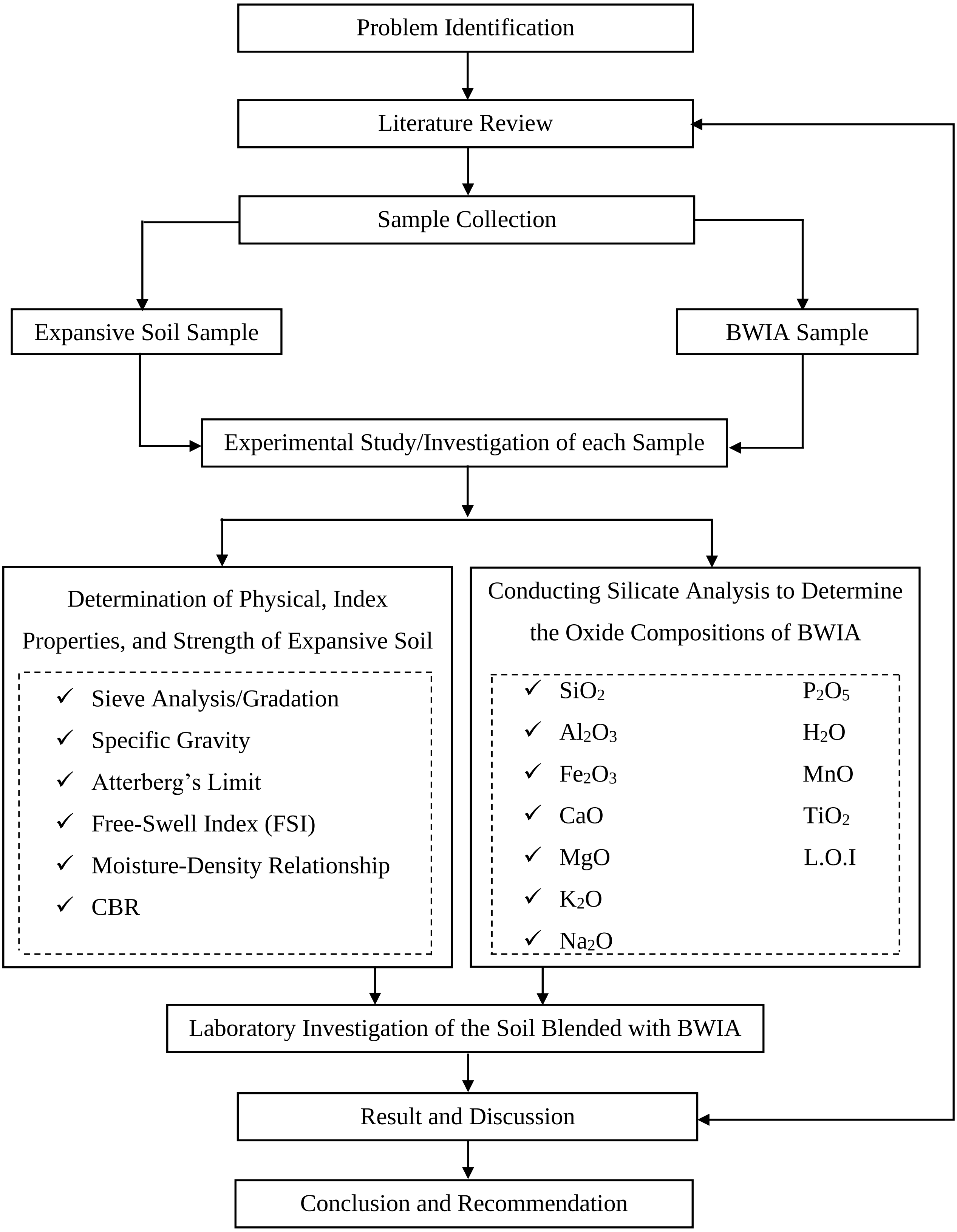


Figure 3 - 2: Work methodology flow chart

3.4 Materials used in the Study

3.4.1 Soil Sample

The expansive soil sample, dark grey in color and highly plastic, for the study was obtained from Kality-Meshualekiya road stretch, Akaki-Kality Sub-city, Addis Ababa, Ethiopia. A pit was excavated using pick and bar beside the road stretch at a depth of 1.5 m below the normal ground surface and the disturbed sample was collected, put into sacks, and transported to the Addis Ababa Science and Technology University highway and geotechnical engineering laboratory for the experimental investigation. A representative soil sample was put into the yellow plastic bag shown in Figure 3-3 (a) and taken into the laboratory to determine its natural moisture content.

(a) (b)

Figure 3 - 3: (a) Soil sample collection, (b) Soil Sample finer than 0.425 mm

3.4.2 Biomedical Waste Incinerator Ash (BWIA)

The biomedical waste incinerator ash to be utilized for this study was obtained from the Adama modern biomedical waste incineration center, which is located in the capital of the Eastern Shewa zone, Oromia Regional State, Ethiopia, around 100 km away from Addis Ababa. The incineration plant has a capacity of burning 1000 kg of medical waste per hour from health centers and governmental and non-governmental hospitals nearby the city and it is fitted with special gas cleaning equipment with burning temperature of 800 -1200 ˚C.

Figure 3 - 4: Adama High Temperature Medical Waste Disposal Incineration Center

(a) (b)

Figure 3 - 5: (a) BWIA Sample preparation, (b) BWIA Sample finer than 150μm Sieve

3.4.3 Hydrated Lime

The hydrated lime utilized for the study was bought from a construction material sales shop at Mercato, Addis Ababa, Ethiopia. It is produced from the Senkelie lime factory, and its chemical compositions were determined at the Ministry of Mining and Energy central laboratory and summarized in Table 4-3.

Figure 3 - 6: Lime sample used in the study

3.5 Laboratory Experimental Methodology

3.5.1 General

Firstly, the collected soil sample was exposed to air, dried, and then pulverized into the required size before conducting the laboratory tests. Thereafter, the laboratory tests such as sieve analysis, Atterberg's limit, specific gravity, moisture-density relationship (compaction), and CBR were conducted on the natural soil as per the American Society for Testing and Materials (ASTM) standards. The detailed properties of the soil are shown in Table 4-1 and the results of each test are also presented and discussed in the "result and discussion" section. The complete silicate laboratory test for the BWIA sample was conducted at the Geological Survey of Ethiopia laboratory, and its mineral (oxide) compositions are listed in Table 4-2 of the next chapter. Then, various percentages (3%, 5%, 7%, 9%, and 11%) BWIA were added by the dry weight of the soil to minimize its expansiveness and enhance its strength. All the above tests were conducted for the natural soil blended with those percentages of BWIA. In addition, the strength of the soil was also further studied by the amendment of 2% and 3% lime with 9% BWIA (the content of BWIA in which the minimum free-swell index was attained).

The clear procedures, standards, and sample preparation for each laboratory test are addressed in each sub-section below. The detailed workflow for the study is shown in Figure 3-2.

3.5.2 Laboratory Tests

The following laboratory tests were conducted on the raw expansive soil, blended with 3%, 5%, 7%, 9%, and 11% of BWIA, and 2% and 3% of hydrated lime were also amended with 9% of BWIA.

3.5.2.1 Particle Size Analysis

This test aims to quantify the distribution of particle (grain) size in soils (ASTM D 422-63). Once the percentage of particle size distribution is quantified, it helps to determine the textural classification of soil such as gravelly, sandy, silty, and clayey, etc. by using USCS and AASHTO classification systems. As a primary classification procedure, if 50% or more by dry weight of the test soil specimen passes the No.200 (75 µm) sieve, the soil is classified as fine-grained soil, whereas, if more than 50% of the test soil specimen is retained on the No.200 (75 µm) sieve, the soil is classified as coarse-grained soil (ASTM D 2487-98). The detailed results of the particle size analysis test are addressed and discussed in the next chapter.

Figure 3 - 7: Particle size analysis test

3.5.2.2 Specific Gravity

This test method covers the determination of the specific gravity of soil solids that pass through a No. 40 (425 µm) sieve (ASTM D 854-98). The specific gravity of a soil is calculated as the weight of a given volume of soil solids at a certain temperature divided by the weight of the same volume of distilled water at the same temperature. It used to calculate the phase relationship of soils, such as void ratio and degree of saturation. It is determined by using the following formula:

$$Mathematically;\ G_s = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots Eqn. (3.1)$$

Where, G_s = Spacific gravity of soil solids

W_1 = Weight of dry and clean pycnometer, g

W_2 = Weight of dry and clean pycnometer + dry soil, g

W_3 = Weight of dry and clean pycnometer + dry soil + water, g

W_4 = Weight of dry and clean pycnometer + Water, g

Figure 3 - 8: Specific gravity test

3.5.2.3 Free-Swell Index (FSI)

The main problem of expansive soil is not bearing capacity, rather it is cyclic swell-shrink behavior due to moisture fluctuation. Thus, it will lead to differential settlement and premature cracks in buildings, unevenness in road pavements and railway track structures. The FSI of the study soil was determined in the laboratory (IS, 1977) by preparing 10 g of the soil sample that passed through the No. 40 (425 µm) sieve and poured into a 100 ml capacity graduated cylinder, one with distilled water (V_f) and the other one with kerosene (V_i) (Admas, 2020). The two samples mixed with both kerosene and water were kept for 24 hours to allow the particles to settle completely. The soil sample mixed with the kerosene has no swelling, thus the volume can be taken from the original volume of the soil (V_i or V_k). But, the soil sample mixed with water has swelling potential and can be taken as the final volume of soil after 24 hours (V_f or V_w). Finally, the FSI of the natural soil and the soil blended with various percentages of BWIA (3%, 5%, 7%, 9%, and 11%) were determined by the formula shown below.

$$FSI\ (\%) = \frac{V_f - V_i}{V_i} * 100 = \frac{V_w - V_k}{V_k} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. Eq.(3.2)$$

The FSI result of the natural soil and the soil blended with the prescribed percentages of BWIA are tabulated in Appendix B, and the variation in free-swell index versus percentages of BWIA is also plotted and clearly discussed in the next chapter.

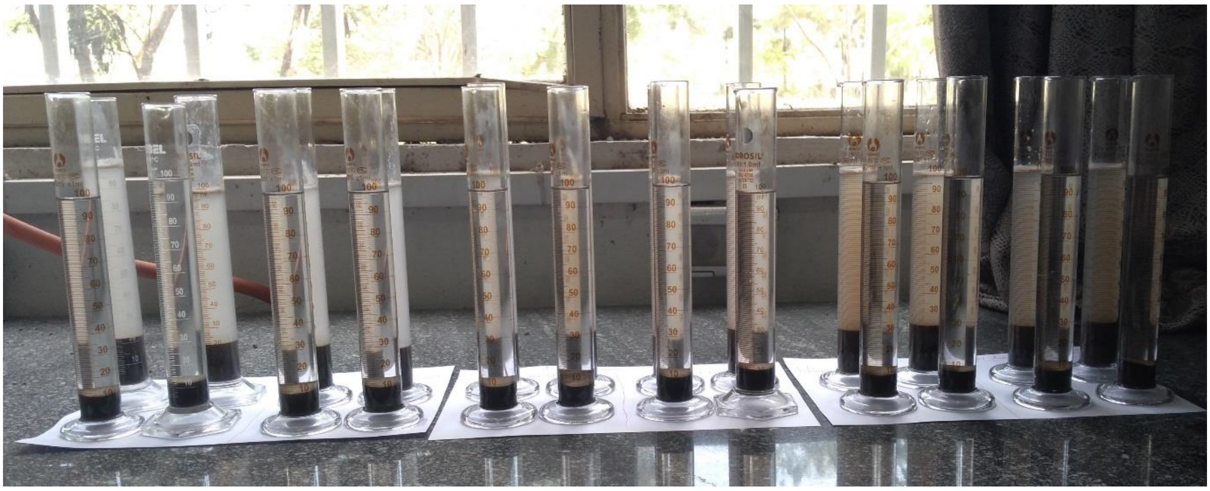


Figure 3 - 9: Free-swell index test

3.5.2.4 Free-Swell Ratio (FSR)

It’s calculated as the ratio of the final volume of a soil in distilled water of a graduated cylinder having a 100 ml capacity to the final volume of a soil in kerosene of the same cylinder capacity after 24 hours. As stated in section 2.5.6, according to Prakash *et al*. (2016), FSR is used to classify soils from very high to negligible expansive soil. The FSR results are tabulated in Appendix B.

3.5.2.5 Atterberg (consistency) Limits

The purpose of this test is basically to determine the Atterberg's limit, (liquid limit, plastic limit, and shrinkage limit) of a soil. All the tests described as Atterberg's limit tests are determined on the portion of a soil sample which is finer than the No.40 (425 µm) sieve (ASTM D 4318-98). The method of testing and sample preparation for Atterberg's limits and definition of some basic terms are discussed below as per the (ASTM D 4318-98) specification.

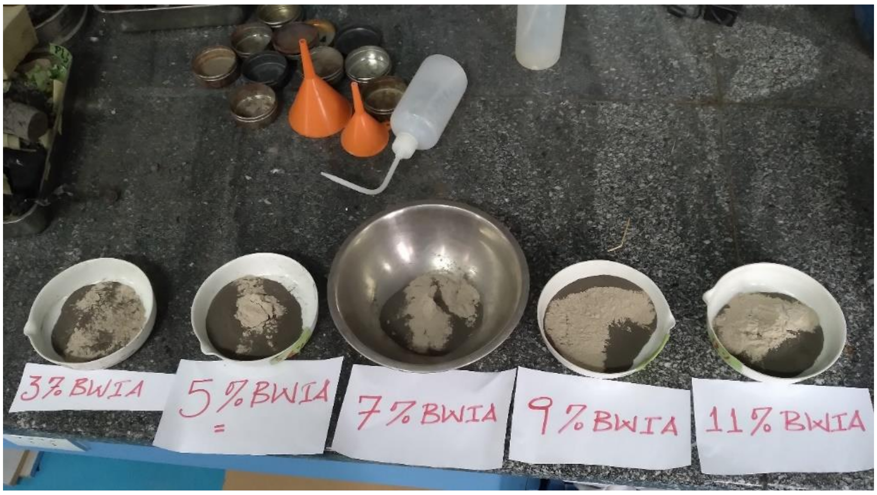


Figure 3 - 10: Sample preparation for the Atterberg limit tests

a) **Liquid Limit (LL)**: The water content of soil, expressed as a percentage, at the arbitrarily defined boundary between semi-liquid and plastic states (ASTM D 4318-98).This test method is used for the classification of fine-grained soils based on USCS and AASHTO classification systems for various engineering purposes. In this study, a liquid limit test was conducted on the natural soil which was air-dried, pulverized, and passed the No. 40 (425 µm) sieve and the soil blended 3%, 5%, 7%, 9%, and 11% of the BWIA using a mechanical apparatus. The detailed results and variation of liquid limit for each sample with the addition of the BWIA are plotted and discussed in the next chapter.

b) **Plastic Limit (PL):** The percentage of water in soil at the interface between the plastic and semi-solid states (ASTM D 4318-98). The effect of BWIA on the plastic limit of the highly expansive soil is plotted and discussed in the next chapter.

c) **Shrinkage Limit (SL):** The smallest water content at which the soil is saturated, or the maximum water content at which a reduction of water content will not cause a decrease in the volume of the soil mass (Arora, 2003). For this study, linear shrinkage of the natural soil and the soil mixed with 3%, 5%, 7%, 9%, and 11% BWIA was conducted. It is expressed as a percentage and numerically expressed as the change in length divided by the initial length when the water content is reduced to the shrinkage limit.

$$Linear\ Shrinkage\ (\%) = \frac{Initial\ length - Final\ length}{Initial\ length} * 100 \ldots \ldots \ldots \ldots \ldots\ Eqn. (3.3)$$

d) **Plasticity Index (PI)**

The range of water content at which soil behaves plastically (ASTM D 4318-98). It's an important parameter to determine the expansiveness index and classify fine-grained soils. Numerically, it is the difference between the liquid limit and the plastic limit. The detailed calculation of the PI for each sample is tabulated in Appendix A, and the variation of PI with the percentages of BWIA is plotted and discussed in the next chapter.

$$PI = LL - PL \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots Eqn. (3.4)$$

Figure 3 - 11: Atterberg limit test

3.5.2.6 Moisture-Density Relationship

This method of test is intended for determining the relation between the moisture content and the density of soils compacted in a mold of a given size with a 2.5-kg (5.5-lb) rammer dropped from a height of 305 mm (ASTM D 698-91). Compaction helps to decrease compressibility, increase shear strength and bearing capacity, and sometimes decrease the permeability of soils for engineering construction. Soil placed as engineering fill (embankments, foundation pads, road bases) is compacted to a dense state to obtain satisfactory engineering properties such as shear strength, compressibility, or permeability (ASTM D 698-91). According to (ASTM D 698-91), there are two types of laboratory compaction tests, standard and modified compaction. The fundamental principle and test procedure are the same except for the number of blows, weight of the hammer, height of fall, and dimension of the mold, etc. In this study, the standard laboratory compaction test was conducted on the natural soil and the soil mixed with 3%, 5%, 7%, 9%, and 11% of BWIA. The dry density versus moisture content curve for all percentages of BWIA is plotted in Figure 4-4 of the next chapter, and the results for each mixture are tabulated and plotted in Appendix C.

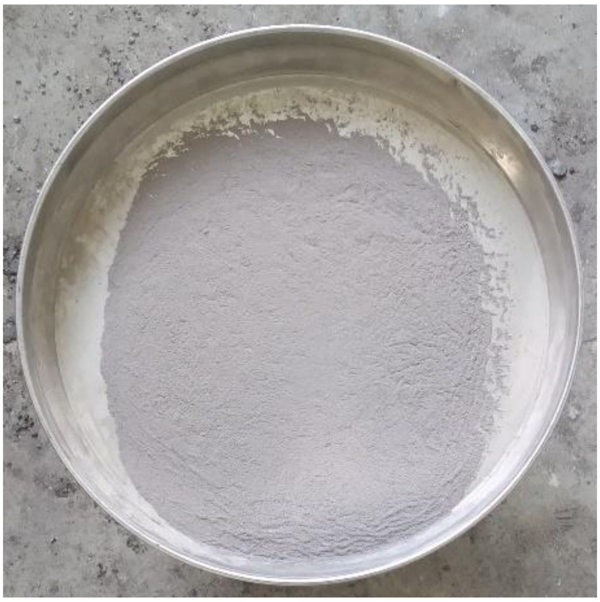

Figure 3 - 12: Compaction test

3.5.2.7 California Bearing Ratio (CBR)

This test method covers the determination of the California Bearing Ratio of pavement subgrade, subbase, and base/course materials from laboratory compacted specimens (ASTM D 1883-99). The design (thickness) of the pavement layer and railway track structure depends on the strength of the subgrade soil. In this study, a three-point soaked CBR test was conducted only for the natural soil, the soil treated with 9% BWIA (content of BWIA in which the minimum free-swell index attained), and the soil + 9% BWIA mixture amended with 2% and 3% lime. The soil sample used for the test was an air-dried soil finer than the No. 4 (4.75 mm) sieve. The detailed results of the test for those samples are tabulated in the appendix part, and the load versus penetration chart for the raw expansive soil is plotted and discussed in the next chapter, whereas, attached in Appendix D for the remaining samples.

3.5.2.8 Scanning Electron Microscopy (SEM)

The microstructural properties such as the configuration of the sample, structural arrangement (bonding), and particle boundary relationship of a soil specimen can be investigated by the SEM imaging device. Clays and clay minerals are more readily observed by scanning electron microscopy (SEM) than by any other usual means (Biruk, 2020). The microstructural properties of the raw expansive soil and the soil + 9% BWIA mixture blended with 3% lime cured for seven days were investigated using SEM at different sizes and magnification rates. The specimens for both mixture is powdered and sieved to get a sample passing 150 µm sieve which can be easily investigated by the SEM machine. A JCM-6000 PLUS Bench Top SEM jeol SEM machine was used to conduct the test. The test was conducted at microbiology laboratory of Adama Science and Technology University, Adama, Ethiopia.



(a)

(b)

Figure 3 - 13: (a) Powdered sample for the SEM test finer than 150 microns, (b) JCM-6000 PLUS Bench Top SEM JEOL machine

CHAPTER FOUR

4. TEST RESULTS AND DISCUSSION

4.1 Materials used for the Study

4.1.1 Characteristics of the Expansive Soil

The soil sample used for the study was obtained from the Kality-Meshualekiya road stretch, Akaki-Kality Sub-city, Addis Ababa, Ethiopia. The soil is dark grey and had a high moisture content in it during excavation. It became shrink and cracks were formed after being exposed to open air for 3 days. This implies that the soil has more fines (clay) content and showing a swell-shrink behavior due to moisture fluctuation. The particles were blended each other because of their stickiness nature while removing from the sack. All these physical properties indicate that the soil sample is highly plastic and expansive. The natural moisture content, liquid limit, plastic limit, and plasticity index of the soil are 46.22%, 103.74%, 45.57%, and 58.18% respectively. The percentage of passing No. 200 (0.075 mm) sieve is 94.22% which is greater than 50% (silt and clay). Thus, according to the AASHTO soil classification system, the soil is classified as A-7-5 (20), and as per the USCS, from the plasticity chart, the soil is classified as CH (inorganic clay of high plasticity) depending on the value of LL and PI, i.e., the soil has a poor quality to be used as subgrade material. Summary of the results of the laboratory investigations performed on the natural soil are summarized in Table 4-1. It is thus clearly visible that the soil is highly expansive and weak.

Table 4 - 1: Properties of the soil sample used in the study

Investigated properties	**Soil sample**
Natural moisture content (W_n), %	46.22
Specific gravity	2.70
% Passing No.200 (0.075 mm) sieve	94.32
Liquid limit, %	103.74
Plastic limit, %	45.57
Plasticity index, %	58.18
Shrinkage limit, %	26

Group index (GI)	20
Soil type as per USCS	CH
Soil type as per AASHTO classification system	A-7-5 (20)
Optimum moisture content, %	33
Maximum dry density, g/cm^3	1.52
Free-swell index (FSI), %	115
Free-swell ratio (FSR), %	2.15
Activity (A)	1.28 active
CBR-Swell, %	9.33
Soaked CBR, %	2.31

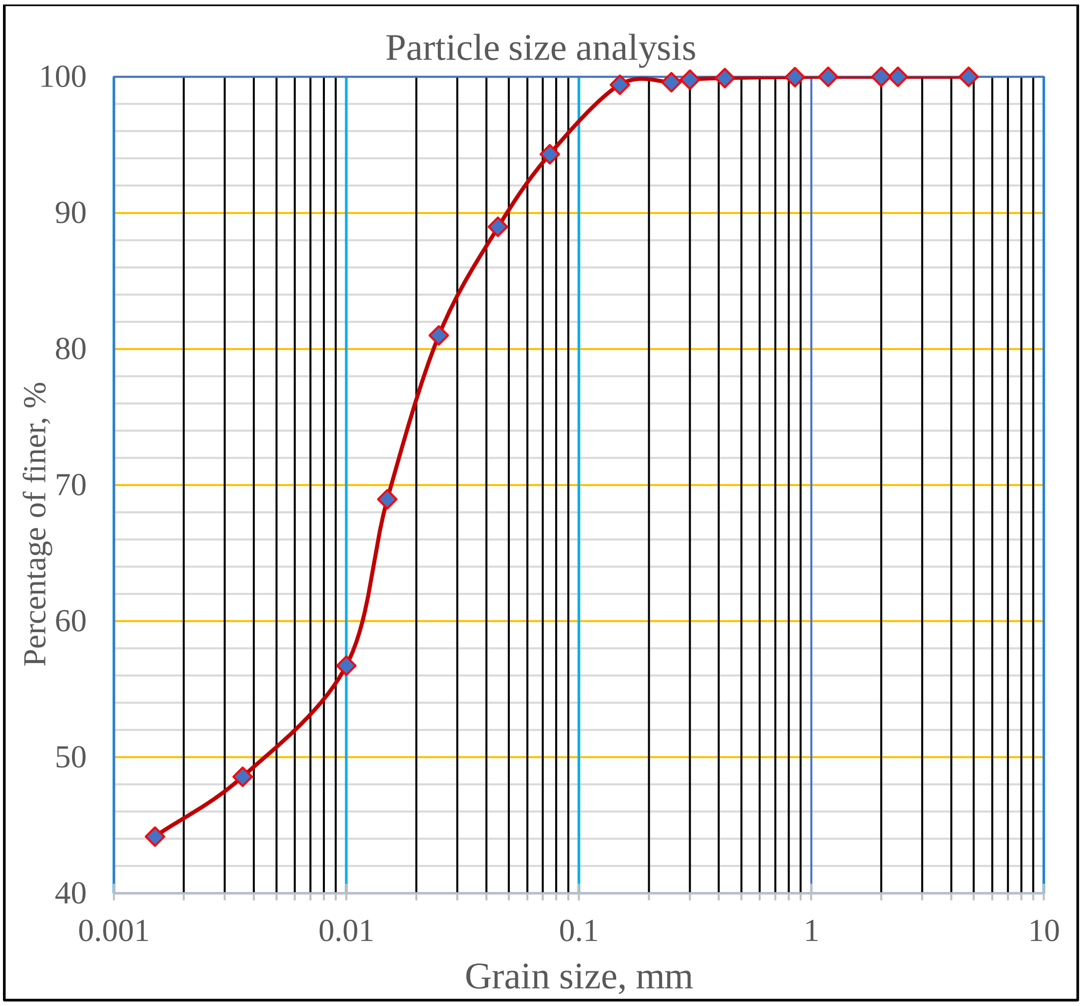


Figure 4 - 1: Particle size distribution curve for the expansive soil

4.1.2 Properties of the BWIA Sample

The BWIA sample used for the study was obtained from the Adama modern medical incineration center which has a capacity of burning 100 kg/hr. It is dark-white in color as shown in Figure 3-5 (b). Its specific gravity is 2.44 with a linear shrinkage limit of zero percent. As shown in Figure 3-11, the initial and final length of the specimen after 24 hours in an oven are the same. The complete silicate analysis was conducted at the Geological Survey of Ethiopia laboratory, and the chemical compositions in percent are tabulated as shown in Table 4-2.

Table 4 - 2: Chemical Compositions of BWIA

S. Code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	L.O. I
BWIA (%)	37.96	14.11	1.12	28.86	4.10	1.94	1.02	0.08	0.75	0.49	2.61	6.93

BWIA is an intermediate material that has a good percentage of both oxides of silicon and calcium. According to the (ASTM C 618-08a) specification of the chemical composition of fly ash, the material's major oxide composition ($SiO_2 + Fe_2O_3 + Al_2O_3$) is equal to 53.19%, which is greater than the minimum requirement of 50%. Thus, BWIA satisfies the criteria of class C fly ash pozzolanic material. So, it can enhance the strength of the soil by forming the compounds of calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). These compounds flocculate the clay particles in a soil mass by creating a bond with the additives.

Table 4 - 3: Standard Chemical Compositions of Fly Ash (ASTM C 618-08a)

Chemical Requirements	**Class of Fly Ash**		
	C	F	N
$SiO_2 + Fe_2O_3 + Al_2O_3$	50	70	70

4.1.3 Properties of the Lime

The hydrated lime obtained from Senkelie lime factory have the following oxide compositions determined at Ministry of Mining and Energy Central laboratory.

Table 4 - 4: Chemical Compositions of the Lime used in the Study

Sample	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	TiO_2	P_2O_5	MnO	SO_3
Lime (%)	6.21	2.18	3.57	59.47	3.91	0.61	0.79	0.33	0.21	0.28	0.58

4.2 Effect of BWIA on the Atterberg Limits

As can be seen from Table 4-5 below, with every 2% increment of BWIA stabilizer, a decrease in liquid limit (not that much significant) and hence plasticity index was observed. The main reason behind the decrease in the liquid limit and plasticity index is due to the percentage replacement of clay particles in the highly expansive soil by the BWIA, which reduces the water affinity of clay particles. Thus, the plasticity index kept on decreasing for all mixtures. Since BWIA has a linear shrinkage of 0%, the addition of 11% BWIA reduced the shrinkage of the highly expansive soil by 50%. The BWIA sample didn't show any shrinkage after it was kept for 24 hours in an oven.

- **Effect of BWIA on liquid limit:** The liquid limit for the natural soil (moisture content versus the number of blows) is plotted as shown below. The moisture content versus the number of blows for the soil blended with all percentages of BWIA is plotted in Appendix A.

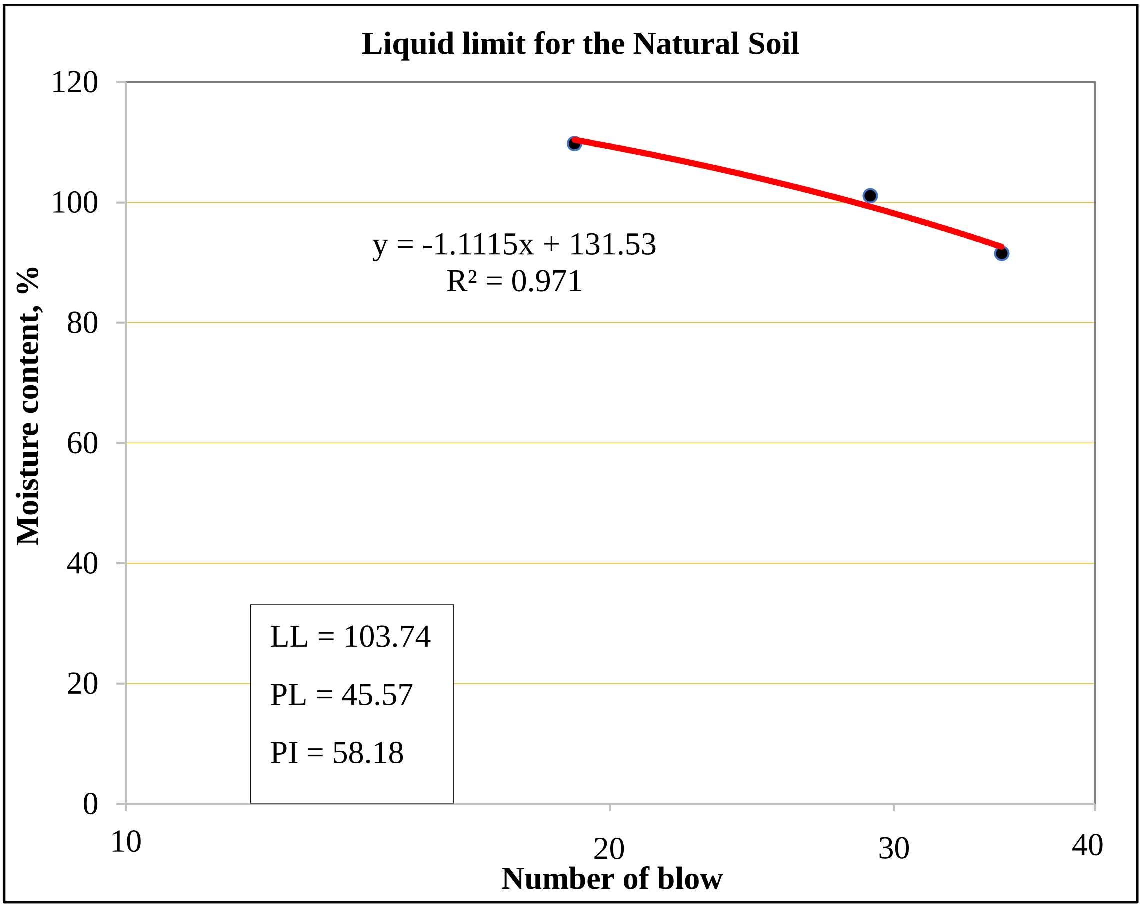


Figure 4 - 2: Effect of BWIA on the liquid limit of the soil

Table 4 - 5: Effect of BWIA on Atterberg limits

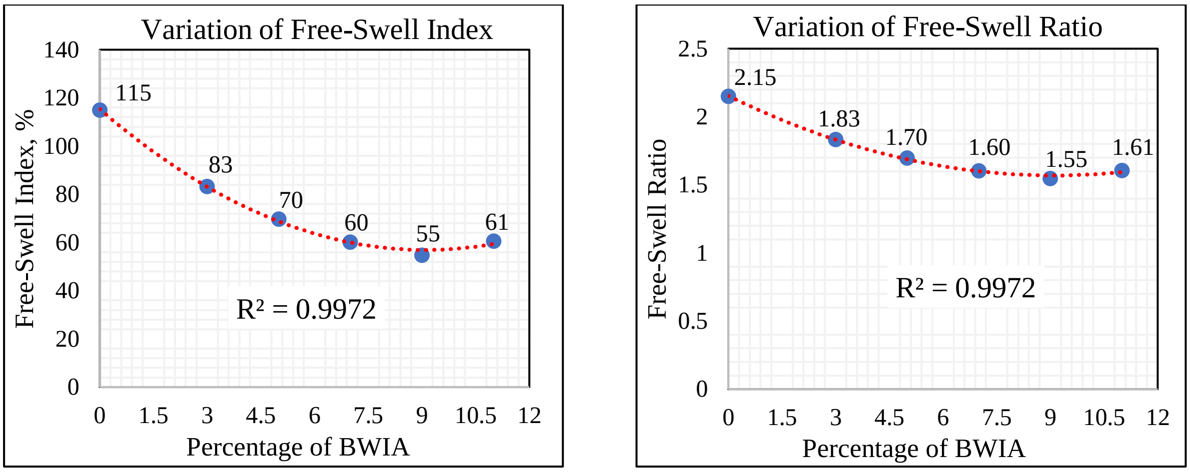
Percentage of BWIA	Variation of liquid limit (LL)	Variation of plastic limit (PL)	Variation of plasticity index (PI)	Variation of Shrinkage limit (SL)
0	103.74	45.57	58.18	26
3	100.11	47.84	52.27	21
5	93.92	48.62	45.30	20
7	90.63	52.06	38.57	18
9	86.6	58.88	27.72	16
11	84.92	60.06	24.86	13

As shown in Table 4-5 above, the liquid limit of the highly expansive soil was 107.34. When blended with 3%, 5%, 7%, 9%, and 11% BWIA, the liquid limit was reduced by 3.5%, 9.46%, 12.63%, 16.52%, and 18.14 %, respectively. The reduction of liquid limit is not significant and expansivity of the soil still remains according to the Dakshanamurthy and Raman (1973).

- **Effect of BWIA on PI and PL:** Unlike the liquid limit, the plastic limit of the soil increased with the addition of BWIA, whereas the plasticity index kept on decreasing for all soil-BWIA mixtures. As can be inferred from the table, the plasticity index decreased by 10.16%, 22.14%, 33.71%, 52.35%, and 57.27%, and the plastic limit increased by 5%, 6.7%, 14.24%, 29.21%, and 31.83% when the soil was blended with 3%, 5%, 7%, 9%, and 11% BWIA. The PI value of the soil treated with 9% and 11% of BWIA became within the permissible range of ERA (2013) flexible pavement design manual specification to be used as subgrade material. The replacement of clay (fine) particles by ash reduces the plasticity of the soil. Because, as the content of fine (clay) particles decreases, the plasticity of soil will also decrease.
- **Effect of BWIA on shrinkage limit:** The results of the linear shrinkage limit of the untreated soil and soil blended with 3%, 5%, 7%, 9%, and 11% BWIA are shown in Table 4-5. The addition of BWIA reduces the shrinkage of the highly expansive soil from 26% of linear shrinkage (natural soil) to 13% when the soil was treated with 11% BWIA. The addition of 11% BWIA reduced the shrinkage of the highly expansive soil by 50%. The reduction is from the property of the BWIA, which has linear shrinkage of 0%. The BWIA sample didn't show any shrinkage after it was kept for 24 hours in an oven as shown in Figure 3-11.

4.3 Effect of BWIA on Swelling Characteristics

The stabilizing potential of the BWIA is further demonstrated by the reduction of the free-swell index test results that are summarized in Figure 4-3. The minimum free-swell index was attained when the soil was blended with 9% BWIA but it became a bit raised when the soil was treated with more than 9% BWIA. The effect of BWIA on the swelling potential of the expansive soil was investigated with respect to the values of free-swell index (IS, 1977) and free-swell ratio (FSR) of the soil. The variation of FSI and FSR with percentages of BWIA are plotted as shown in the figures below.



(a) (b)

Figure 4 - 3: Effect of BWIA on (a) free-swell index and (b) free-swell ratio

As shown in Figure 4-3 (a) and (b), both the free swell-well index and the free-swell ratio kept on decreasing up to 9% BWIA and a little bit raised when the soil was blended with 11% of BWIA. It was also checked for 13% BWIA and again the free-swell index was found to be almost the same as to that of the free-swell index value when the soil was blended with 11% BWIA. Even though the effect is not that much significant, the stabilizing behavior of the BWIA was demonstrated by the reduction of the free-swell index; a 52.17% decrease was obtained when the soil was blended with 9% of BWIA. As can be seen from Figure 4-3 (b), the FSR of the untreated soil was 2.15. This ratio was reduced to 1.55 when the soil was blended with 9% BWIA. As illustrated in Table 2-7, according to Prakash *et al*. (2016), a soil can be classified from negligible to very high expansive soil based on the free-swell ratio value. Thus, the study soil sample is classified as a highly expansive soil having montmorillonite clay mineral because the FSR of the untreated soil (2.15) lies between 2.0 and 4.0.

4.4 Comparison of Free-Swell Index and Atterberg's Limit Test Results

For every 2% increment of BWIA, there was a decrease in liquid limit (but not that much significant) and a slight increment in the plastic limit, hence the plasticity was significantly reduced from the natural soil PI value of 58.18% to 24.86% when it was treated with 11% BWIA. The reduction in liquid limit is not significant and the stabilized soil is classified remains expansive according to Dakshanamurthy and Raman (1973), but the plasticity index (governing parameter which is a function of both the liquid and plastic limit) kept on decreasing. The PI value of the study soil was reduced to 27.72% and 24.86% when it was treated with 9% and 11% BWIA, i.e., according to Chen (1975) method of classification of expansive soils, these values lie between 10-35 and 20-55, and the soil is in the medium and highly expansive range. The average free-swell ratio (FSR) of the natural soil was 2.15 and lies in 2.0-4.0, i.e., the study soil is highly expansive according to (Prakash *et al*., 2016), but when it was treated with 9% BWIA the FSR reduced to 1.55 that lies in 1.5-2.0, i.e., the soil changed from highly to moderately expansive and it is compatible with the method of Chen (1975) expansive soil classification based on plasticity index. Similar studies were conducted by Admas (2020) and Asres (2017) on expansive soil stabilization using rice husk ash and quarry dust at Addis Ababa Institute Technology, respectively. The soil studied by Admas (2020) and Asres (2017) had a liquid limit of 102.6% and 148.82% and a plasticity index of 62.4% and 62.61%, respectively. When they treated it with the optimum content of rice husk ash (20%) amended by 4% lime (Admas, 2020) and 30% quarry dust (Asres, 2017), the LL and PI values were reduced to 78.5% and 23.4%, and 79.64% and 40.46%, respectively. That means, based on the liquid limit and plasticity index, the soil was in a highly and medium-expansive range even when it was treated with the optimum content of rice husk ash amended with 4% hydrated lime. The FSR of the raw soil was 2.3 and lies in 2.0-4.0, i.e., the study soil by (Admas, 2020) was highly expansive, but it became reduced to 1.2 that lies in between 1.0-1.5. Thus, the soil changed into low expansive and this is not compatible with the method of Chen (1975) expansive soil classification based on plasticity index. The same is true for the soil under the study of (Asres, 2017).

4.5 Effect of BWIA on Moisture-Density Relationship

As clearly stated in section 3.4.8, in this study, the standard laboratory compaction test was conducted for the natural soil and the soil blended with 3%, 5%, 7%, 9% and 11% BWIA. The dry density versus moisture content and the variation in OMC and MDD with percentages of BWIA for the natural and stabilized soil are shown in Figure 4-4 below. The detailed results and characteristics curve of all the mixtures are plotted and tabulated in Appendix C.

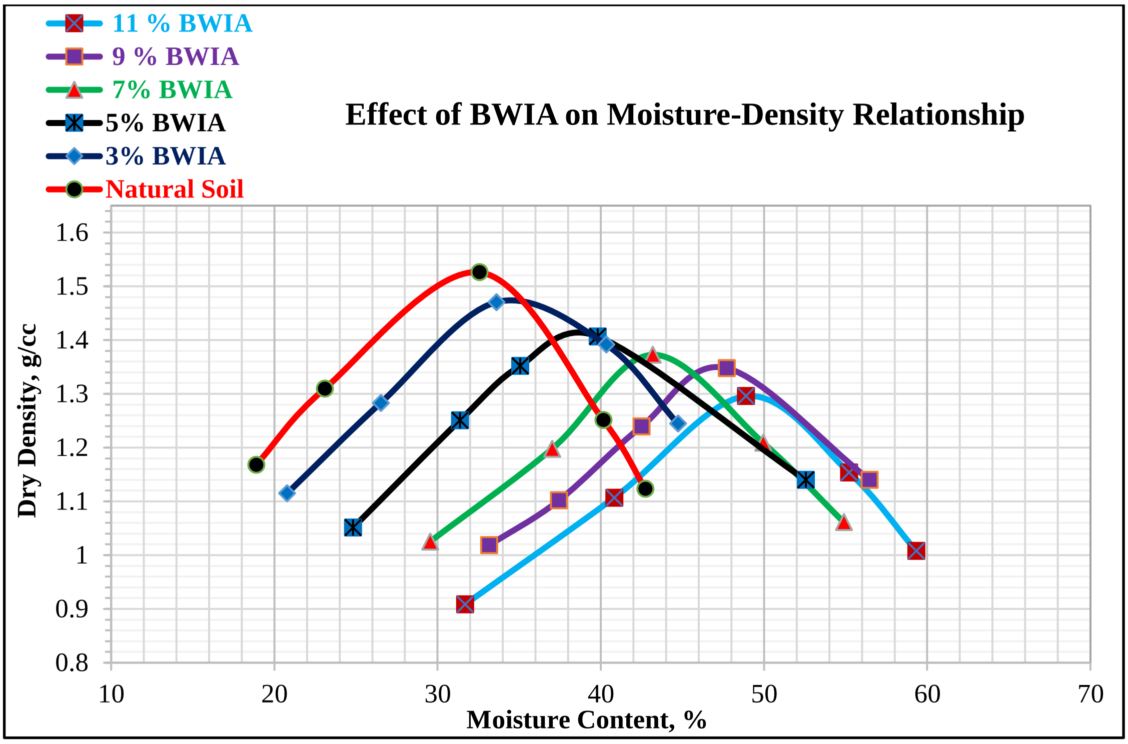
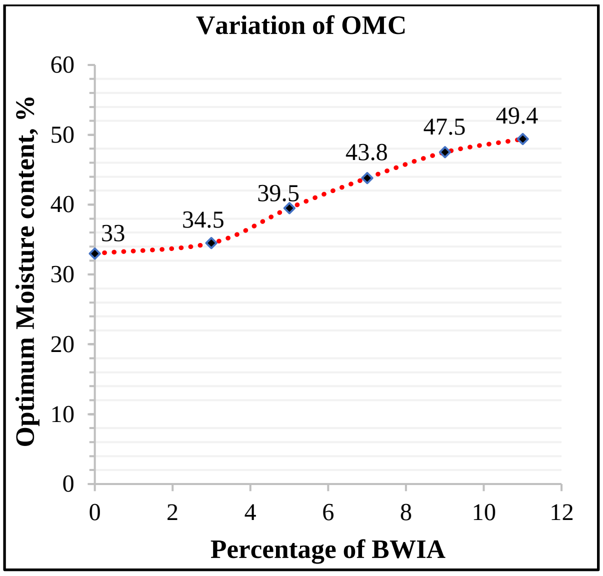


Figure 4 - 4: Effect of BWIA on moisture-density relationship

As can be inferred from Figure 4-4, the addition of BWIA increases the optimum moisture content, whereas the maximum dry density decreases for all percentages. The MDD and OMC of the untreated soil were 1.52 g/cm^3 and 33% respectively. The variations of MDD and OMC for the untreated and stabilized soil with 3%, 5%, 7%, 9%, and 11% BWIA are shown in Figure 4-5.

The parabola moves down to the right, this shows that the soil-BWIA mixture needs more water to get the desired density on the field during compaction. This is because the pozzolanic reaction needs more water as the BWIA has a tendency of coating the soil particle and makes them to dry (moisture content increases, i.e., the curve moves to the right). The BWIA is finer and relatively lighter than the soil so that the percentage replacement of the heavy soil having larger specific gravity (2.7) by the light weight BWIA having specific gravity of BWIA decreases the density (the curve moves down).



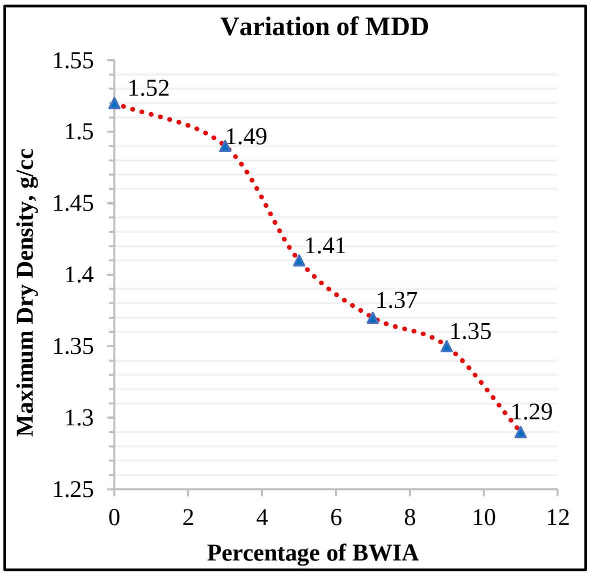


Figure 4 - 5: Variation of OMC and MDD

As shown in Figure 4-5, the optimum moisture content of the soil was increased by 4.55%, 19.67 %, 32.73 %, 43.94 %, and 49.7 %, whereas, the maximum dry density was decreased by 1.97%, 7.24%, 9.87%, 11.18%, and 15.13% when it was blended with 3%, 5%, 7%, 9%, and11% BWIA, respectively. The increase in moisture content is obviously due to the pozzolanic reaction process that the soil-BWIA mixture needs more water to get the desired dry density. It can be concluded that the amount of water content that will be needed to compact the expansive soil blended with BWIA is higher than the water required to compact the untreated soil with the desired density, and the soil-BWIA mixture density will be lower than the density of the unblended soil. This is because the specific gravity of the BWIA (2.44) is lower than the specific gravity of the soil (2.7).

4.6 Effect of BWIA and Lime on California Bearing Ratio

In addition to the above parameters, the potential effectiveness of the BWIA is also demonstrated by the improvement of the CBR test results. Since the expansiveness of the soil has not decreased tremendously, 2% and 3% lime were amended with 9% BWIA to investigate the effect on the strength of the soil. In the present study, a three-point CBR test was conducted at a strain rate of 1.27 mm/min (ASTM D 1883-99) on the natural soil, the soil blended with 9% BWIA, and the soil + 9% BWIA mixture blended with 2% and 3% lime. The strength of the soil improved when it was blended with 9% of BWIA and a significant improvement was also observed when 2% and 3% hydrated lime were amended.

Table 4 - 6: CBR for the natural expansive soil (0% BWIA)

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows				30 Blows				65 Blows			
		Gauge read.	Load	Stress	CBR	Gauge read.	Load	Stress	CBR	Gauge read.	Load	Stress	CBR
			N	N/mm^2	%		N	N/mm^2	%		N	N/mm^2	%
0.00		0.00	0.00	0.00		0.00	0.00	0.00		0.00	0.00	0.00	
0.64		4.00	102.83	0.05		6.00	154.24	0.08		10.50	269.92	0.14	
1.27		5.00	128.54	0.07		7.50	192.80	0.10		12.50	321.34	0.17	
1.91		5.50	141.39	0.07		8.00	205.66	0.11		13.50	347.04	0.18	
2.54	**6.90**	6.00	154.24	0.08	1.2	9.00	231.36	0.12	1.7	14.00	359.90	0.19	2.7
3.81		7.50	192.80	0.10		9.50	244.22	0.13		14.50	372.75	0.20	
5.08	**10.30**	8.50	218.51	0.11	1.1	10.50	269.92	0.14	1.4	16.00	411.31	0.21	2.1
7.62		9.00	231.36	0.12		11.00	282.78	0.15		17.50	449.87	0.23	
10.16		9.50	244.22	0.13		12.50	321.34	0.17		19.00	488.43	0.25	
12.70		11.00	282.78	0.15		13.50	347.04	0.18		19.50	501.29	0.26	

Ring Calibration Factor/div.	25.707
Plunger Area, mm^2	1935.00
Rate of strain, mm/min	1.27
Rammer wt. (kg)	2.50
MDD	1.52

No. of Blows	Dry Density (g/cc)	CBR (%)	Swell (%)
10	1.13	1.2	10.3
30	1.29	1.7	9.14
65	1.52	2.7	8.56
Desired Density, g/cc = 95 % MDD = 0.95*1.52		1.4	

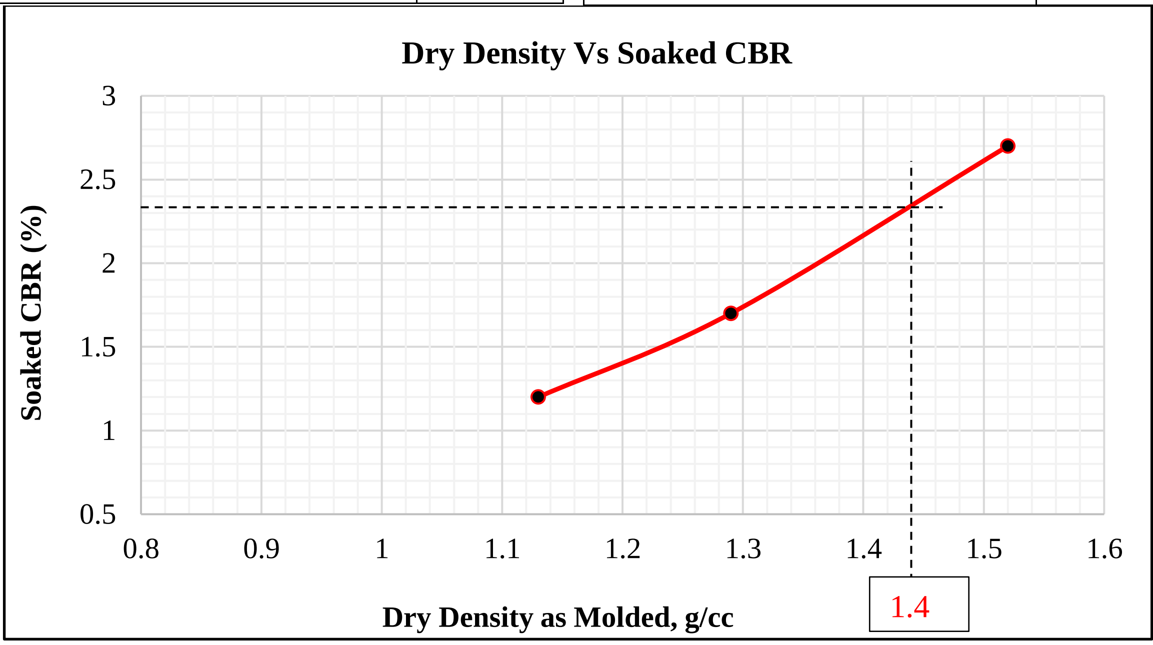


Figure 4 - 6: CBR for the natural expansive soil at 95 % of MDD

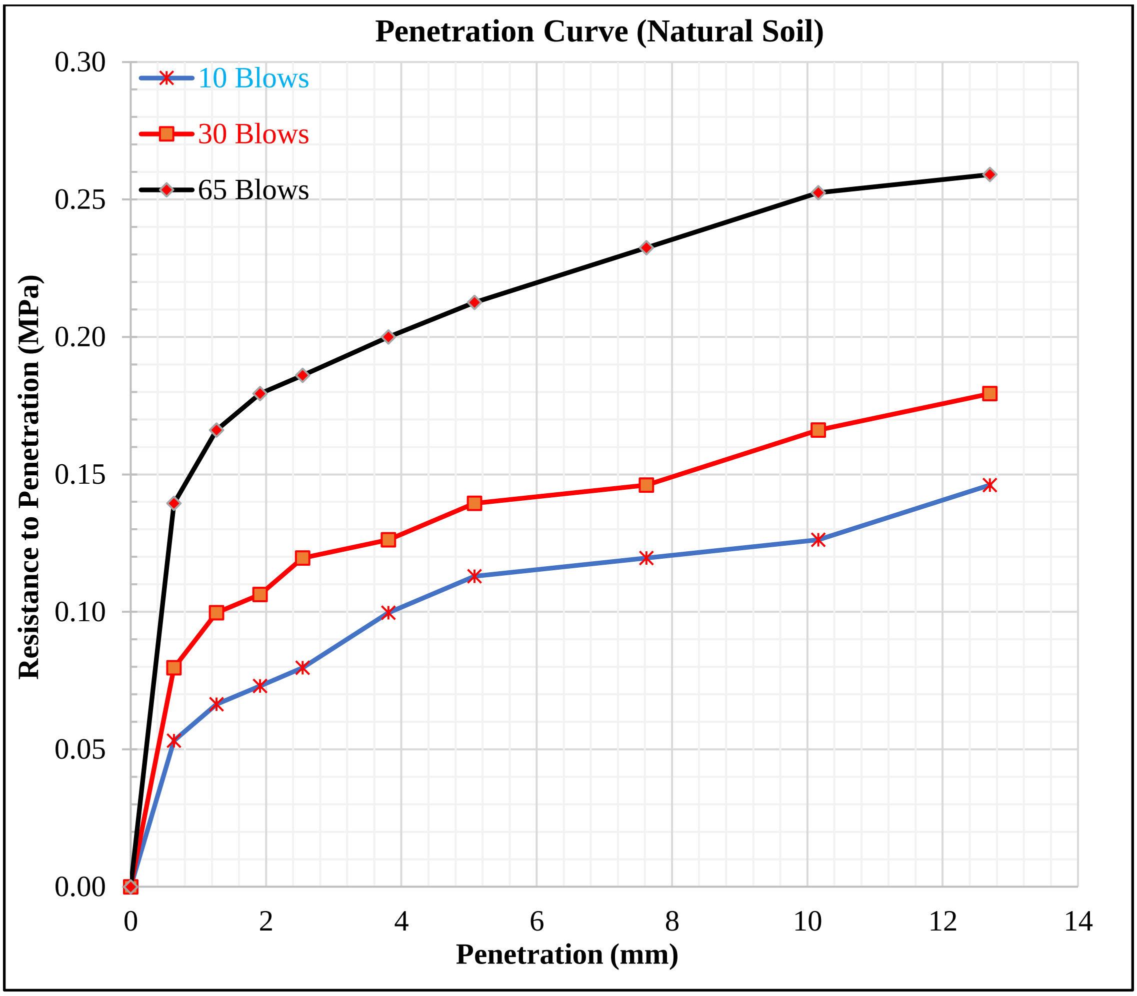


Figure 4 - 7: Penetration Curve (Stress - Penetration) for the Natural Soil

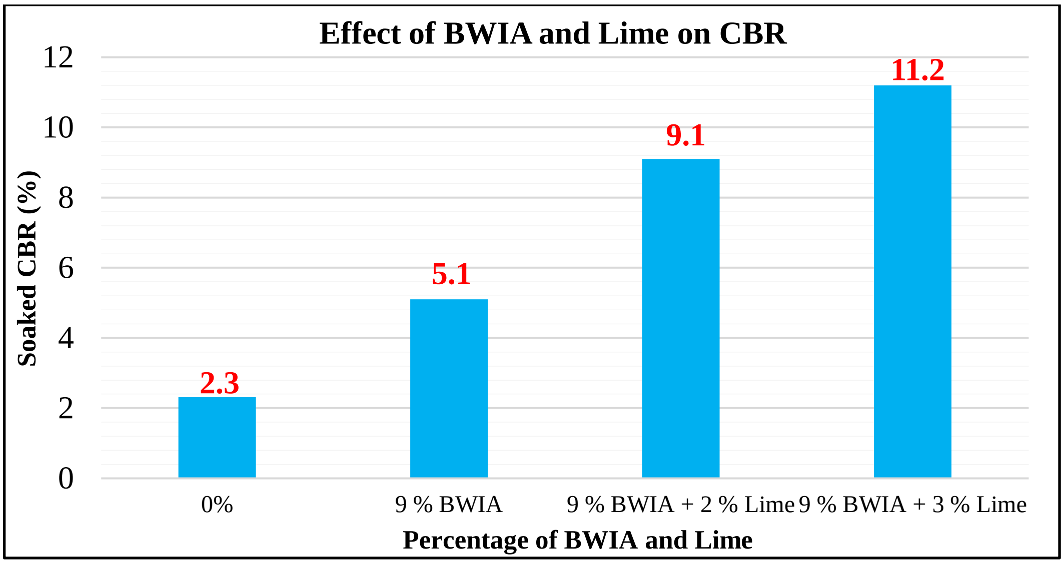


Figure 4 - 8: Effect of BWIA and Lime on CBR

As shown in Figure 4-8 above, the CBR value of the natural soil is very low. According to the Ethiopian Roads Authority (2013) manual standard specification, soils with a CBR value less than 2% are totally poor and special treatment is required and the PI should be less than 30%. The raw expansive soil under the study has a CBR value of 2.3% which is greater than the minimum subgrade CBR specified by the ERA (2013), though it satisfies the requirement, other quality parameters such as PI and GI don't satisfy the standard specification requirements. So, the natural highly expansive soil can not satisfy the requirements by the ERA (2013) standard specification, but it does when treated with 9% BWIA and with the amendment of small percentages of hydrated lime. Its CBR value was increased by 121.74%, 295.65%, and 386.96%, when it was treated with 9% BWIA, 9 % BWIA + 2% lime, and 9% BWIA + 3% lime respectively. As can be inferred from the graph, the addition of lime has significantly increased the CBR of the expansive soil treated with 9% BWIA at both dosages, thus a small content of lime has a significant role in the strength of the expansive soil blended with BWIA. According to the ERA (2013), there are six subgrade strength classes as shown in Table 4-7.

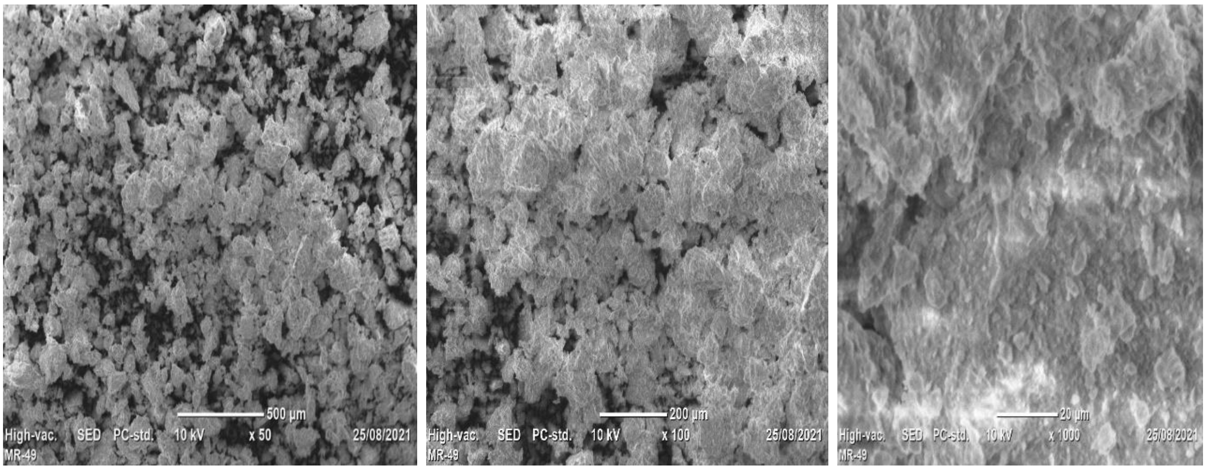
Table 4 - 7: Subgrade strength class for flexible pavement design according to ERA (2013)

Subgrade Strength Class	CBR Range (%)
S1	< 3
S2	3, 4
S3	5, 6, 7
S4	8 to 14
S5	15 to 30
S6	> 30

The raw expansive soil has a CBR value of 2.3% and lies in the subgrade class **S1**. When it was treated with 9% BWIA, 9% BWIA + 2% lime, and 9% BWIA + 3% lime, its CBR was increased to 5.1%, 9.1%, and 11.2%, which lies in the subgrade strength class **S3** and **S4** respectively. In the present study, detailed economic analysis was not carried out, but it is obvious that as the CBR value increases, the thickness of the pavement layer decreases, so that there will be a reduction in the total project cost. The design CBR results and the plot of penetration curves for the remaining specimens are tabulated and drawn in Appendix D.

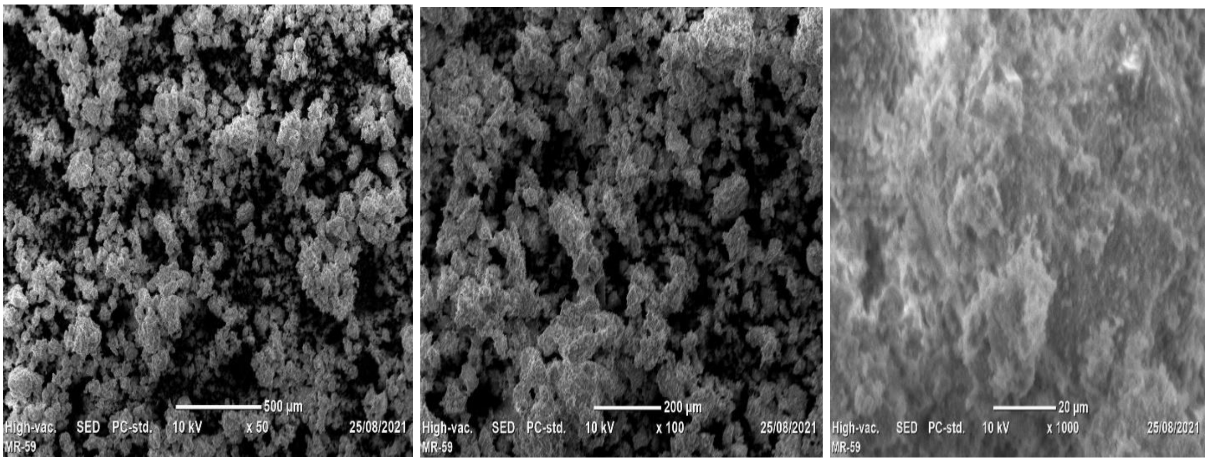
4.7 Scanning Electron Microscopy (SEM) Results

A JCM-6000 PLUS Bench Top SEM JEOL machine was used to conduct the test. The following images for the natural expansive soil (control specimen) and the soil + 9% BWIA blended with 3% lime cured for seven days at different sizes and magnification rates were captured.



(a) 500 µm, x50 (b) 200 µm, x100 (c) 20 µm, x1000

Figure 4 - 9: SEM micrographs of the raw expansive soil at different sizes and magnification rate



(a*) 500 µm, x50 (b*) 200 µm, x100 (c*) 20 µm, x1000

Figure 4 - 10: SEM micrographs of the (soil + 9 % BWIA + 3 % lime) at different sizes and magnification rate

As can be inferred from Figure 4-9 and 4-10, the JCM-6000 PLUS Bench Top SEM JEOL machine gives a clear image of both the raw expansive soil sample and the soil blended with 9% BWIA amended with 3% lime at different sizes and magnification rate. The codes MR-49 and MR-59 at the left bottom of the pictures represent raw expansive soil and the soil blended with BWIA and lime. As can be seen in Figure 4-9 (a, b, and c), the raw expansive soil shows a dispersed and scattered type of fabric and less interlock and bond between particles having an opened (floated) arrangement of a layered structure. But, when it was blended with 9% BWIA and 3% lime, the negatively charged clay surfaces attract the cations in the additives and a more flocculated and aggregated structure is shown in Figure 4-10 (a* and b*), and the particles act as a homogeneous mass with no hole (gap) in between (c*). That means the soil particles and the additives form a more flocculated and bonded structure that can enhance the strength of the soil. This is mainly due to the formation of cementitious compounds. Thus, as shown in Figure 4-10 (c*), the particles act like a homogenous material with no pores. Therefore, the compressibility of the soil has been reduced.

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

To investigate the effect of BWIA on the properties of the expansive soil, laboratory tests such as particle size analysis, Atterberg limits, free-swell index, compaction (moisture-density relationship), and California bearing ratio tests were conducted on the raw expansive soil and the soil blended with 3%, 5%, 7%, 9%, and 11% BWIA. Since the expansiveness of the soil has not decreased tremendously, additional lime was added and further investigations have been performed on the strength of the soil.

Based on the laboratory test results, the following findings are concluded;

BWIA is an intermediate material that has a good percentage of both oxides of silicon and calcium. According to the (ASTM C 618-08a) specification of the chemical composition of fly ash, the material's major oxide composition ($SiO_2 + Fe_2O_3 + Al_2O_3$) is equal to 53.19%, which is greater than the minimum requirement of 50%. Thus, BWIA satisfies the criteria of class C fly ash pozzolanic material.

The present study has focused on the stabilization of expansive soils using industrial waste called biomedical waste incinerator ash (BWIA). The natural soil is classified as CH and A-7-5 (20) as per the USCS and AASHTO classification systems respectively, which is a typical indicator of highly expansive soil that is of poor quality to be used as a road subgrade material. The stabilizing behavior of the BWIA was demonstrated by the reduction of the free-swell index; a 52.17% decrease was obtained when the subgrade was blended with 9% of BWIA but it became a bit raised when the content of BWIA greater than 9%. The addition of 9% BWIA has also doubled the subgrade strength (CBR) of the soil.

Since the expansiveness of the soil has not decreased tremendously, additional lime was amended with 9% BWIA (the content of BWIA at which the minimum free-swell index was attained) and further investigations on the strength (CBR) of the soil have been performed.

A significant improvement in the strength (CBR) were demonstrated when the soil was treated with 9% of BWIA amended by 2% and 3% lime. The CBR was increased from 2.3% (CBR of the raw soil) to 11.2% when 9% BWIA was amended by 3% lime.

According to the ERA (2013) flexible pavement design manual specification, the strength of the natural expansive soil lies in the subgrade strength class **S2**, but when it was treated with 9% BWIA, 9% BWIA + 2% lime, and 9% BWIA + 3% lime, the CBR value significantly increased and the subgrade strength class was raised to **S3** and **S4** respectively. Thus, the BWIA-lime stabilized soil can be used as a road subgrade material and there is a reduction of pavement layer thickness that can minimize the overall project cost.

The morphology and fabric of the control specimen were changed when the soil was blended with BWIA and lime. The raw expansive soil is dispersed (non-flocculated) fine particles with an open arrangement, but it became a homogeneous and flocculated structure when it was treated with 9% BWIA and 3% lime.

5.2 Recommendations

According to the laboratory results obtained in the present study, the following points are recommended and proposed for future investigations;

- The present study is conducted by performing only index and limited strength tests such as gradation, Atterberg limits, free-swell index, compaction, and CBR on a representative soil sample at one test pit. Therefore, extensive laboratory and field tests on the subgrade stiffness, strength, and swelling potential at a greater number of test pits and with more dosage of lime content should be conducted to come up with more promising and realistic results.
- According to Prakash *et al*. (2016), the soil in the present study having a free-swell ratio of 2.15 is found to have a montmorillonite clay mineral in it. This theoretical method of clay mineral identification may not be realistic, so mineralogy identification using X-ray diffraction (XRD) test should be conducted.
- This study is conducted based on the laboratory experimental approach, so the realistic effect and actual performance of the BWIA in the field should also be studied.
- The use of BWIA as a stabilizing agent for expansive road subgrade soils is found to be promising, so road construction agencies and authorities should give awareness to healthcare waste management teams in health facilities to properly collect the pharmaceutical waste and to be incinerated at nearby incineration centers.

- It is recommended that a standard for the BWIA stabilization of expansive soils shall be developed by taking representative samples at different locations of the country where these soils exist and conducting extensive laboratory and field tests with an amendment of famous pozzolanic and cementitious materials because its effect on the index properties of the study soil was not that much significant. The developed standard should give a guideline regarding gradation (particle size distribution), optimal content to be used in accordance with the various properties of the soil and the environment, method of grinding, storage, and field application procedures.
- In this study, only the suitability of BWIA-lime stabilized soil to be used as road subgrade is evaluated as per the ERA (2013) pavement design manual specification that is within the permissible range. The economic advantage in pavement layer thickness reduction when using the BWIA-lime stabilized soil over the cutting and replacing technique where selected material is available around a project site should be analyzed.

Lastly, the findings and results from the present study may not be enough to implement it practically, because the results are based on very limited soil properties and parameters obtained from one test pit only. More samples at different locations with extensive laboratory and field testing would be mandatory before the implementation of the results in the present study. Thus, the findings from the study may be considered as indicative and a reference to conduct further research on expansive subgrade soil stabilization for flexible pavement.

5.3 Proposed Future Research Direction

The present study is conducted to investigate the effect of BWIA on the index properties and amendment of small dosage of lime with the optimum content of BWIA on the strength (CBR) of expansive soil to be used for subgrade construction. So, one can conduct research on expansive soil stabilization using BWIA and other native pozzolanic additives that are abundantly and locally available to use as a foundation material for buildings and/or subgrade material for flexible pavements.

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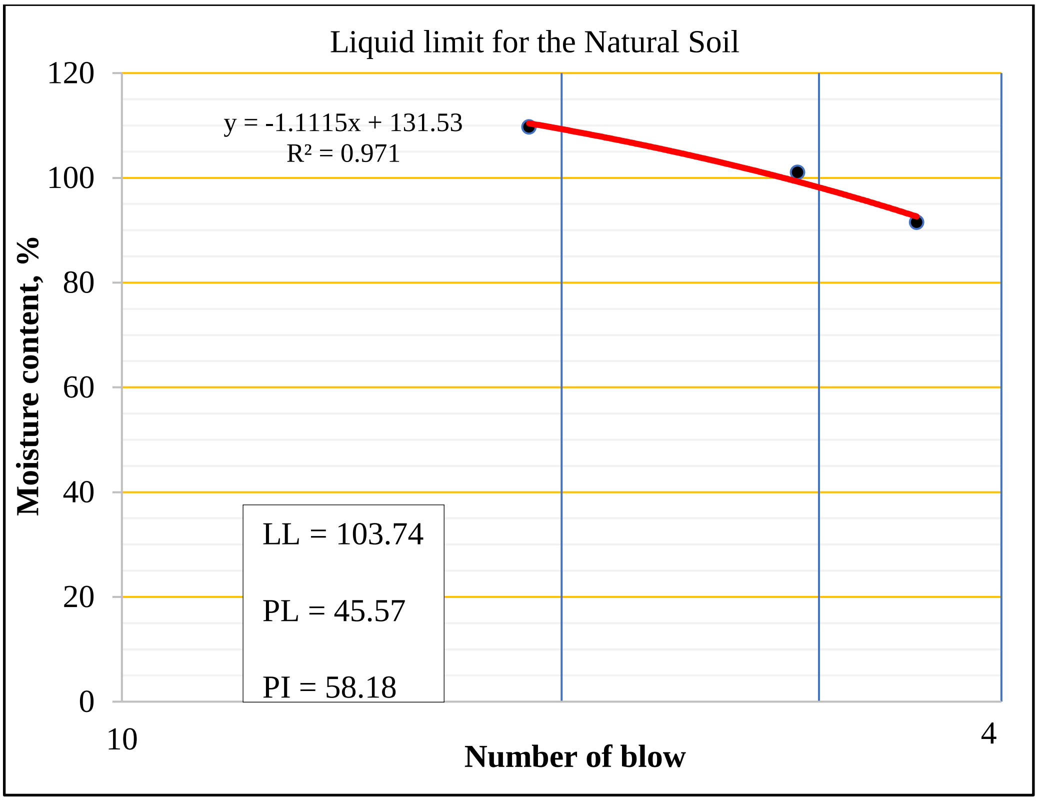
APPENDICES

Appendix A: Atterberg Limit Test Results

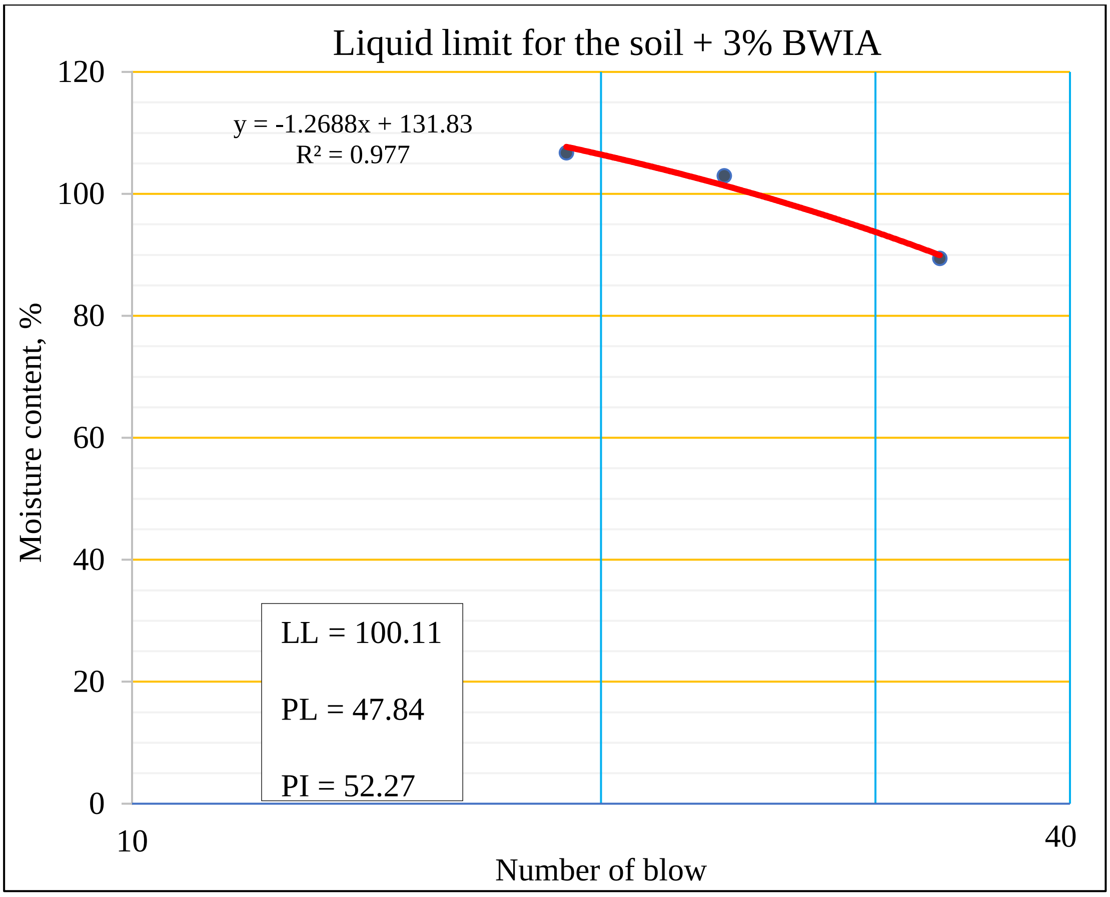
1) Atterberg limit test results for the natural soil without BWIA

Liquid Limit for the Natural Soil			
Trial	**1**	**2**	**3**
Can No.	LL11	LL12	LL13
Can Weight (g)	24.27	24.49	24.13
Can Weight + Moist Soil (g)	40.17	46.98	42.17
Can Weight + Dry Soil (g)	32.57	35.68	32.73
Mass of Water (g)	7.60	11.30	9.44
Mass of Dry Soil (g)	8.30	11.18	8.60
Moisture Content, W (%)	91.52	101.07	109.75
Number of Blow	35	29	19
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL11	PL22	PL3
Can Weight (g)	23.62	23.69	23.67
Can Weight + Moist Soil (g)	25.24	25.44	25.58
Can Weight + Dry Soil (g)	24.71	24.92	24.98
Mass of Water (g)	0.53	0.52	0.60
Mass of Dry Soil (g)	1.09	1.23	1.31
Moisture Content (%)	46.62	44.28	45.80
Plastic Limit (PL)	45.57		
Plasticity Index (PI)	58.18		



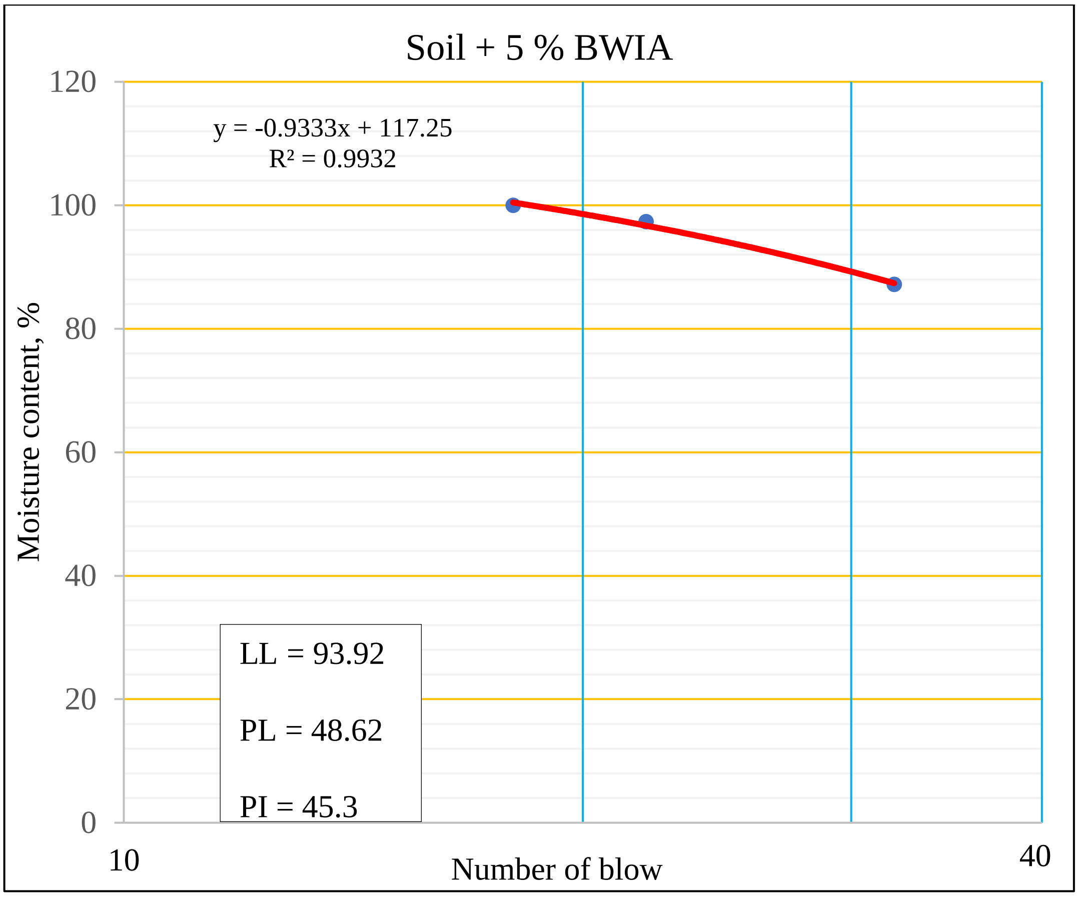
2) Atterberg limit test results for the soil + 3 % BWIA

Liquid Limit (LL)			
Trial	1	2	3
Can No.	LL-31	LL-32	LL-33
Can Weight (g)	23.05	24.04	25.02
Can Weight + Moist Soil (g)	38.94	39.34	38.89
Can Weight + Dry Soil (g)	31.44	31.58	31.73
Mass of Water (g)	7.50	7.76	7.16
Mass of Dry Soil (g)	8.39	7.54	6.71
Moisture Content, W (%)	89.39	102.95	106.71
Number of Blow	33	24	19
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL-31	PL-32	PL-33
Can Weight (g)	13.07	13.17	13.16
Can Weight + Moist Soil (g)	15.28	15.31	15.27
Can Weight + Dry Soil (g)	14.59	14.60	14.58
Mass of Water (g)	0.69	0.71	0.69
Mass of Dry Soil (g)	1.52	1.43	1.43
Moisture Content (%)	45.90	49.51	48.12
Plastic Limit (PL)	47.84		
Plasticity Index (PI)	52.27		



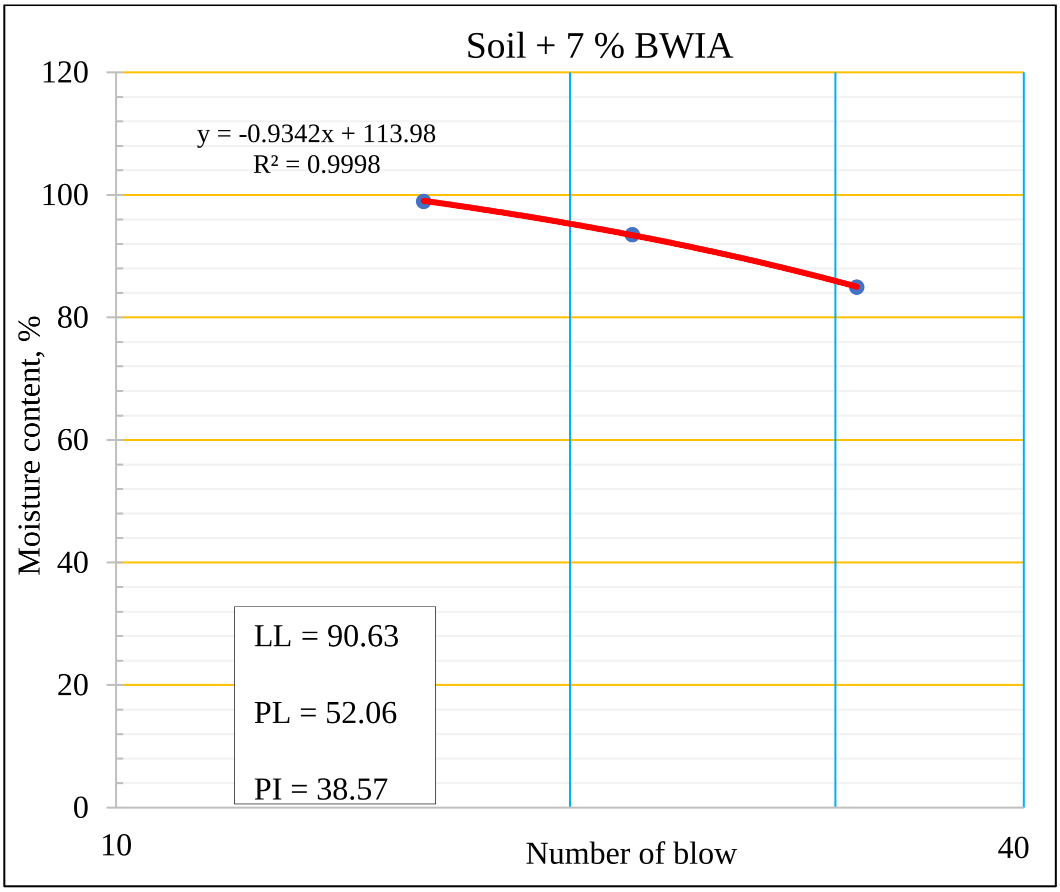
3) Atterberg limit test results for the soil + 5 % BWIA

Liquid Limit (LL)			
Trial	1	2	3
Can No.	LL-51	LL-52	LL-53
Can Weight (g)	24.18	23.79	24.78
Can Weight + Moist Soil (g)	38.74	40.66	40.69
Can Weight + Dry Soil (g)	31.96	32.34	32.74
Mass of Water (g)	6.78	8.32	7.95
Mass of Dry Soil (g)	7.78	8.55	7.95
Moisture Content, W (%)	87.20	97.34	100.00
Number of Blow	32	22	18
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL-51	PL-52	PL-53
Can Weight (g)	13.46	13.35	13.04
Can Weight + Moist Soil (g)	15.98	15.75	15.41
Can Weight + Dry Soil (g)	15.13	14.97	14.64
Mass of Water (g)	0.85	0.77	0.76
Mass of Dry Soil (g)	1.67	1.63	1.61
Moisture Content (%)	50.66	47.65	47.55
Plastic Limit (PL)	48.62		
Plasticity Index (PI)	45.3		



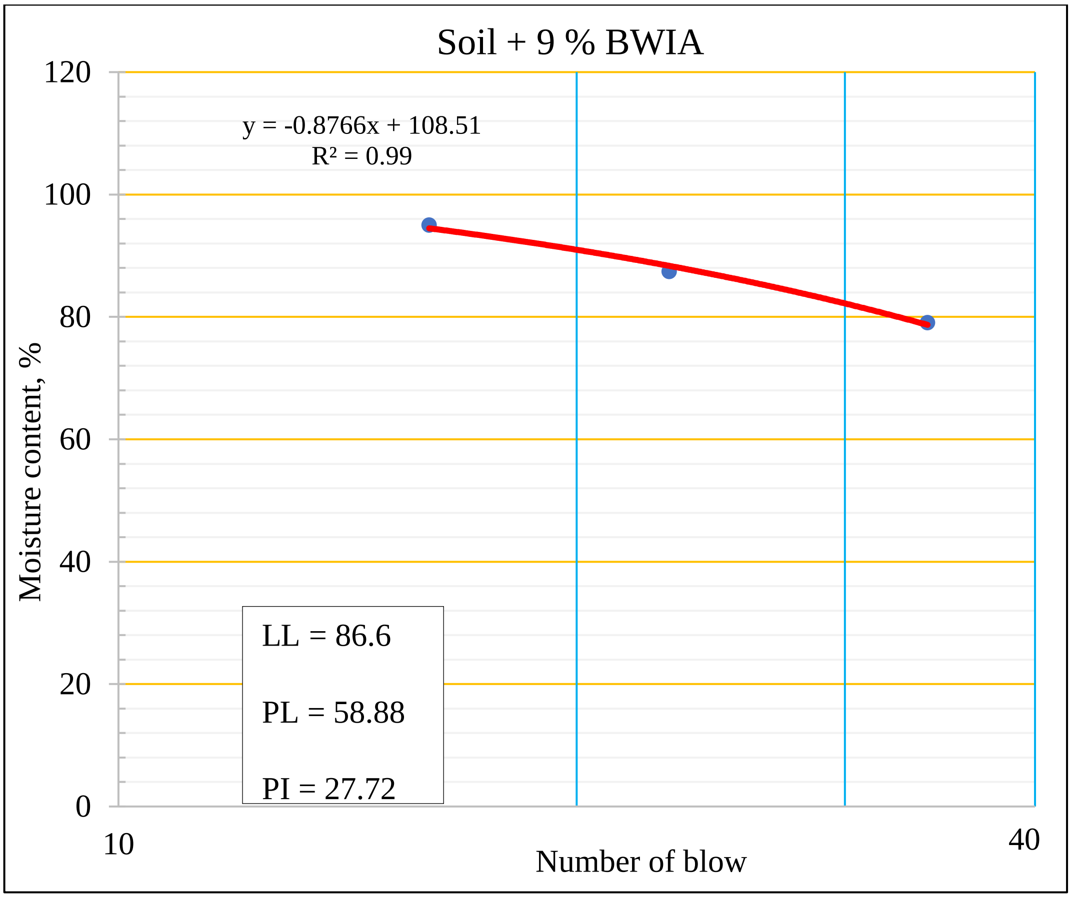
4) Atterberg limit test results for the soil + 7 % BWIA

Liquid Limit (LL)			
Trial	1	2	3
Can No.	LL-71	LL-72	LL-73
Can Weight (g)	23.32	23.24	24.30
Can Weight + Moist Soil (g)	39.45	41.93	41.63
Can Weight + Dry Soil (g)	32.04	32.90	33.01
Mass of Water (g)	7.41	9.03	8.62
Mass of Dry Soil (g)	8.72	9.66	8.71
Moisture Content, W (%)	84.98	93.54	98.97
Number of Blow	31	22	16
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL-71	PL-72	PL-73
Can Weight (g)	13.22	13.17	13.08
Can Weight + Moist Soil (g)	15.08	15.06	15.02
Can Weight + Dry Soil (g)	14.44	14.42	14.35
Mass of Water (g)	0.64	0.64	0.67
Mass of Dry Soil (g)	1.22	1.25	1.27
Moisture Content (%)	52.13	51.28	52.76
Plastic Limit (PL)	52.06		
Plasticity Index (PI)	38.57		



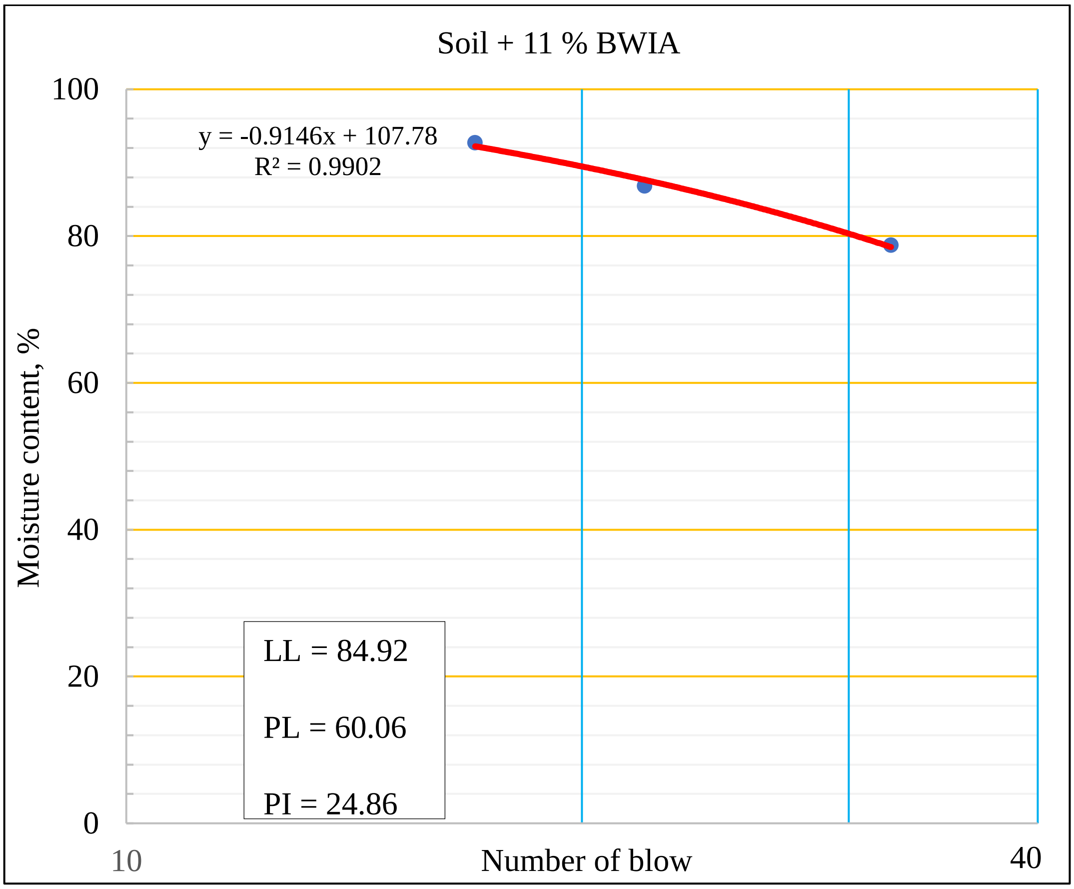
5) Atterberg limit test results for the soil + 9 % BWIA

Liquid Limit (LL)			
Trial	1	2	3
Can No.	LL-91	LL-92	LL-93
Can Weight (g)	36.92	24.14	23.97
Can Weight + Moist Soil (g)	55.28	38.16	42.66
Can Weight + Dry Soil (g)	47.17	31.62	33.55
Mass of Water (g)	8.11	6.54	9.11
Mass of Dry Soil (g)	10.25	7.48	9.58
Moisture Content, W (%)	79.06	87.43	95.04
Number of Blow	34	23	16
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL-91	PL-92	PL-93
Can Weight (g)	13.18	13.43	13.47
Can Weight + Moist Soil (g)	15.42	15.08	15.35
Can Weight + Dry Soil (g)	14.58	14.46	14.67
Mass of Water (g)	0.84	0.62	0.68
Mass of Dry Soil (g)	1.40	1.03	1.20
Moisture Content (%)	59.82	59.19	57.63
Plastic Limit (PL)	58.88		
Plasticity Index (PI)	27.72		



6) **Atterberg limit test results for the soil + 11 % BWIA**

Liquid Limit (LL)			
Trial	1	2	3
Can No.	LL-11,1	LL-11,2	LL-11,3
Can Weight (g)	23.35	23.06	23.17
Can Weight + Moist Soil (g)	42.74	39.78	40.46
Can Weight + Dry Soil (g)	34.20	32.01	32.14
Mass of Water (g)	8.54	7.77	8.32
Mass of Dry Soil (g)	10.85	8.95	8.97
Moisture Content, W (%)	78.77	86.87	92.75
Number of Blow	32	22	17
Plastic Limit (PL)			
Trial	1	2	3
Can No.	PL-11,1	PL-11,2	PL-11,3
Can Weight (g)	11.44	11.46	13.51
Can Weight + Moist Soil (g)	13.40	13.33	15.57
Can Weight + Dry Soil (g)	12.70	12.63	14.76
Mass of Water (g)	0.70	0.70	0.81
Mass of Dry Soil (g)	1.26	1.17	1.25
Moisture Content (%)	59.56	59.83	60.80
Plastic Limit (PL)	60.06		
Plasticity Index (PI)	24.86		



7) Variation of Atterberg limits of the soil blended with BWIA

Variation of Liquid Limit						
Percentage of BWIA	0	3	5	7	9	11
LL, %	103.74	100.11	93.92	90.63	86.6	84.92
Variation of Plastic Limit						
Percentage of BWIA	0	3	5	7	9	11
PL, %	45.57	47.79	48.62	52.06	58.88	60.06
Variation of Plasticity Index						
Percentage of BWIA	0	3	5	7	9	11
PI, %	58.18	52.32	45.3	38.57	27.72	24.86
Variation of Shrinkage Limit						
Percentage of BWIA	0	3	5	7	9	11
SL, %	26	21	20	18	16	13

Appendix B: Results of Free-Swell Index and Free-Swell Ratio

1) Free-Swell Index (FSI) test results

Free-Swell Index Test		
Trial	**Natural Soil (0% BWIA)**	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	23
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	10
2	Volume of soil in distilled water after 24 hours, V_{w2} (ml)	22
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	11
	Average free-Swell Index (%)	115
	Soil + 3% BWIA	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	22
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	12
2	Volume of soil in distilled water after 24 hours, Vw2 (ml)	22
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	12
	Average free-Swell Index (%)	83
	Soil + 5% BWIA	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	20
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	12
2	Volume of soil in distilled water after 24 hours, V_{w2} (ml)	19
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	11
	Average free-Swell Index (%)	70
	Soil + 7% BWIA	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	20
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	13
2	Volume of soil in distilled water after 24 hours, V_{w2} (ml)	20
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	12
	Average free-Swell Index (%)	60
	Soil+9% BWIA	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	20
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	14
2	Volume of soil in distilled water after 24 hours, Vw2 (ml)	20
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	12
	Average free-Swell Index (%)	55
	Soil+ 11% BWIA	
1	Volume of soil in distilled water after 24 hours, V_{w1} (ml)	21
	Volume of soil in kerosene after 24 hours, V_{k1} (ml)	12
2	Volume of soil in distilled water after 24 hours, V_{w2} (ml)	19
	Volume of soil in kerosene after 24 hours, V_{k2} (ml)	13
	Average free-Swell Index (%)	61

Variation of Free-Swell Index						
% of BWIA	0	3	5	7	9	11
FSI, %	115	83	70	60	55	61

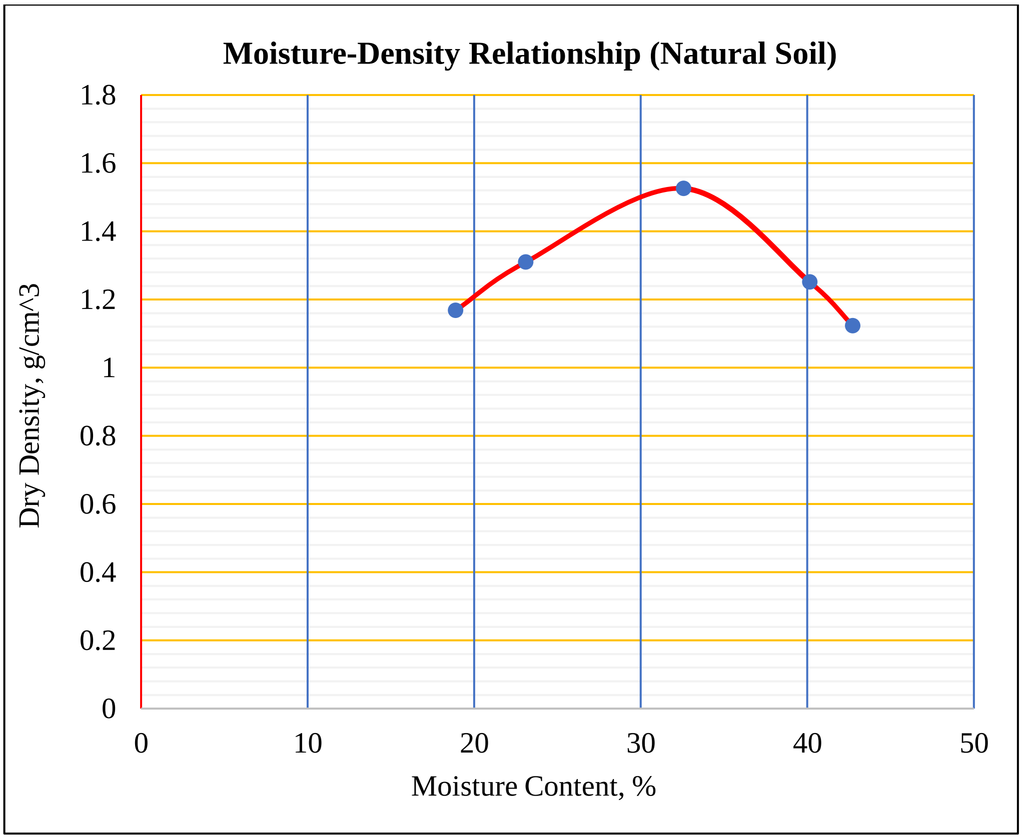
2) Free-Swell Ratio (FSR)

Variation of Free-Swell Ratio						
% of BWIA	0	3	5	7	9	11
FSR	2.15	1.83	1.70	1.60	1.55	1.61

Appendix C: Compaction Test Results

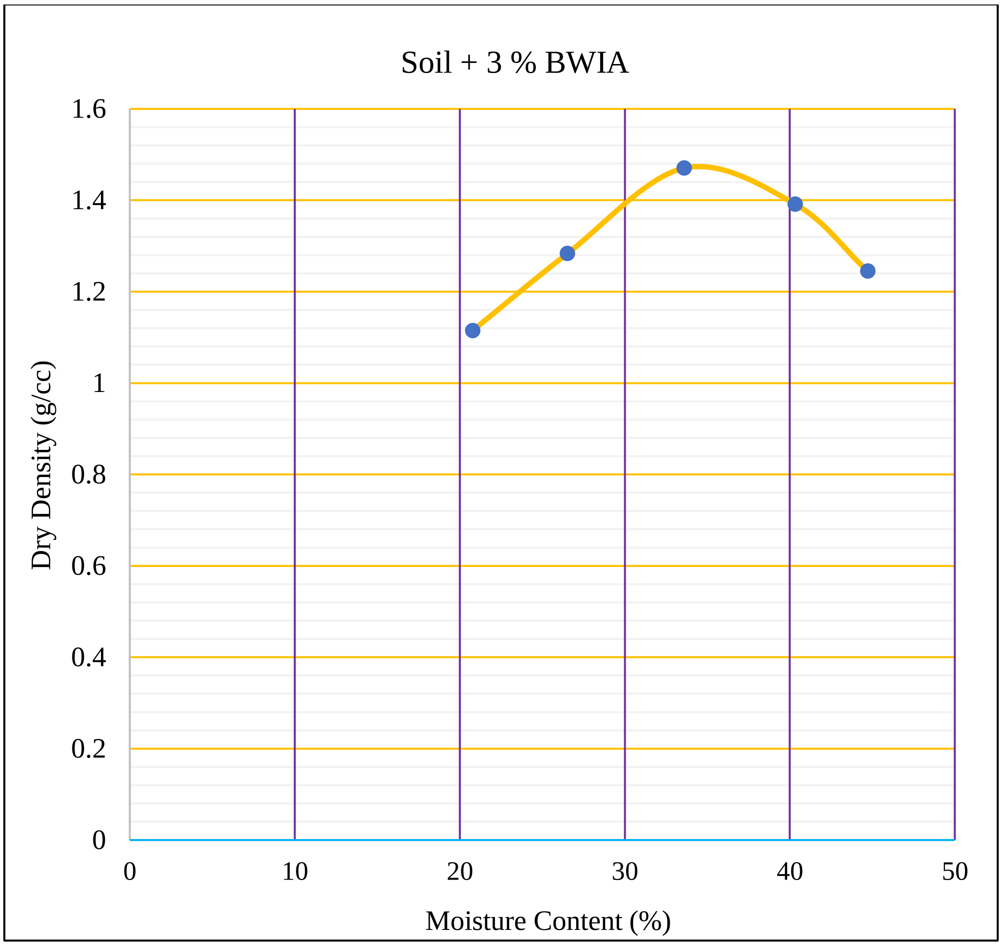
1) Natural Soil (0 % BWIA)

Moisture-Density Relationship for the Natural Soil					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil, g	5448.92	5582.74	5787	5527.61	5406.69
Moisture Can Number	LL-B	LL-D	LL-121	C-3	E-12
Moisture Can Weight, g	23.9	24.1	24	23.7	24
Moisture Can Weight + Moist Soil, g	65.21	66	68.8	68.2	66.4
Moisture Can Weight + Dry Soil, g	58.6486	58.3694	57.7933	55.4519	53.706
Weight of dry soil, g	34.7486	33.0521	33.7933	31.7519	29.706
Weight of Water, g	6.5614	7.6306	11.0067	12.7481	12.694
Moisture Content, %	18.8825	23.0866	32.57066	40.1491	42.7321
Dry Density, g/cc	1.16782	1.30958	1.525953	1.25118	1.12308



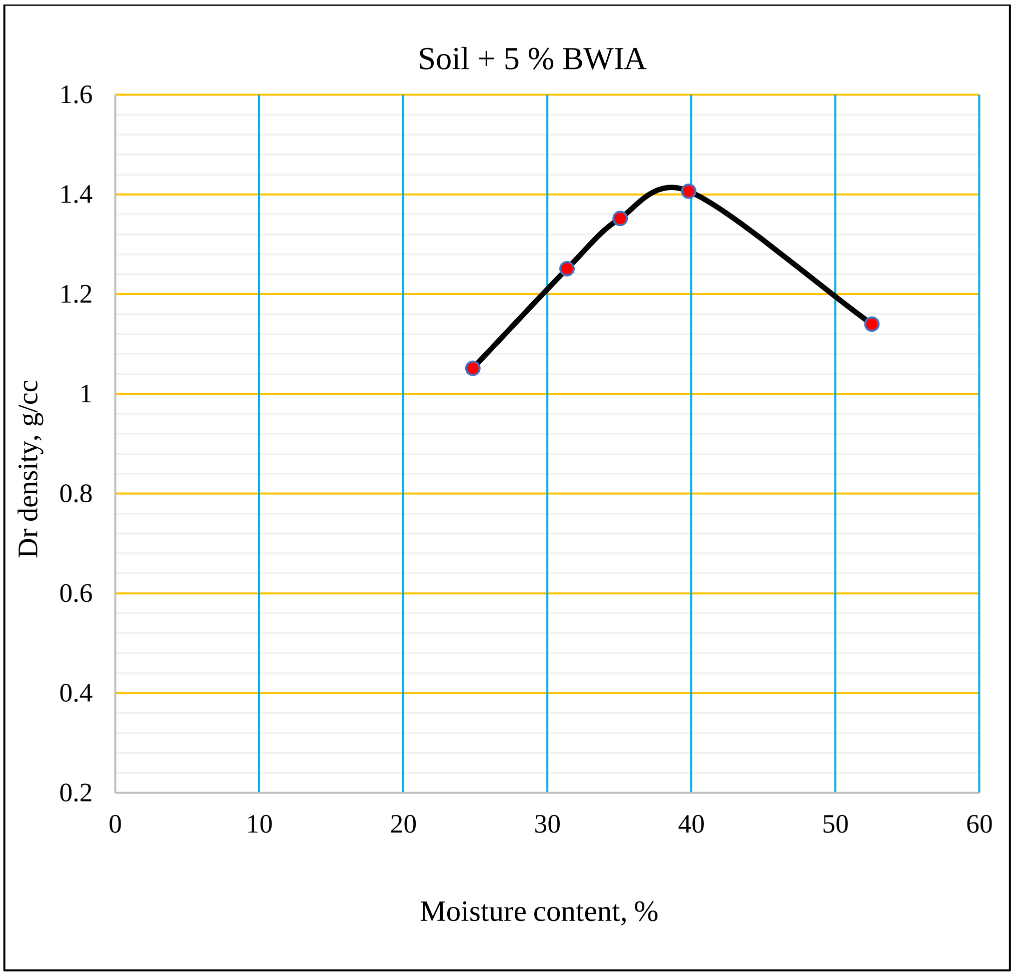
2) Soil + 3 % BWIA

Moisture-Density Relationship for the Soil Blended with 3 % BWIA					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil & BWIA, g	5398.74	5557.68	5734.65	5660.08	5521.37
Moisture Can Number	C-33	A-92	C-11	C-22	E-22
Moisture Can Weight, g	24.1	23	24.3	24.8	25.2
Moisture Can Weight + Moist Soil & BWIA, g	56.54	60.6	58.69	60.64	62.74
Moisture Can Weight + Dry Soil & BWIA, g	50.96	52.72	50.040	50.34	51.14
Weight of dry soil & BWIA, g	26.86	29.72	25.74	25.54	25.94
Weight of Water, g	5.58	7.88	8.65	10.3	11.6
Moisture Content, %	20.7744	26.5141	33.60528	40.3289	44.7186
Dry Density, g/cc	1.11466	1.28303	1.470498	1.3915	1.24457



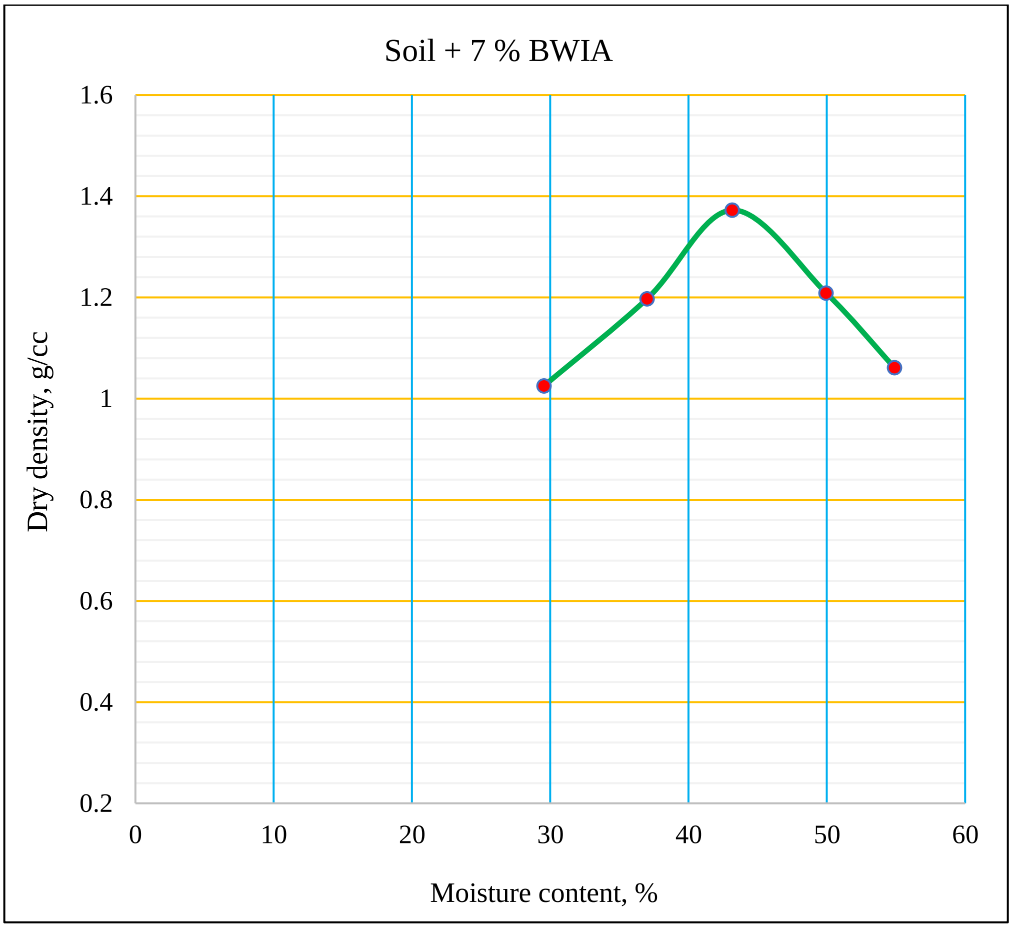
3) Soil + 5 % BWIA

Moisture-Density Relationship for the Soil Blended with 5 % BWIA					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil & BWIA, g	5338.68	5527.14	5622.61	5674.37	5422.47
Moisture Can Number	A-11,1	LL-73	A-93	LL-33	A-52
Moisture Can Weight, g	23	23.2	23	22.7	23.7
Moisture Can Weight + Moist Soil & BWIA, g	57.81	59.051	61.3	60.2	60.81
Moisture Can Weight + Dry Soil & BWIA, g	50.888	50.49	51.36	49.522	48.0269
Weight of dry soil & BWIA, g	27.888	27.29	28.36	26.822	24.3269
Weight of Water, g	6.922	8.561	9.94	10.678	12.7831
Moisture Content, %	24.8207	31.3705	35.04937	39.8106	52.5472
Dry Density, g/cc	1.05104	1.25068	1.351811	1.40664	1.1398



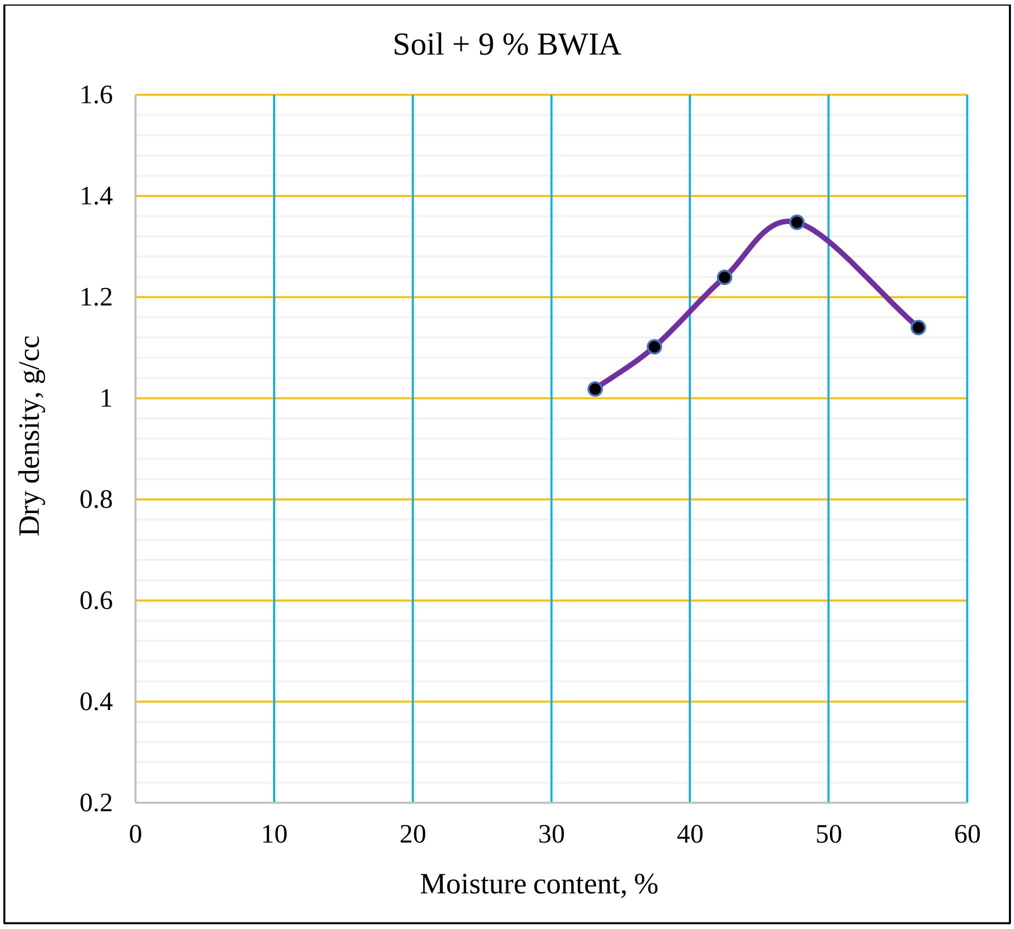
4) Soil + 7 % BWIA

Moisture-Density Relationship for the Soil Blended with 7 % BWIA					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil & BWIA, g	5314.29	5477.13	5642.19	5487.67	5348.27
Moisture Can Number	B-11,2	B-11,3	C-91	LL-71	LL-51
Moisture Can Weight, g	23.2	23.7	24.1	24.7	23.9
Moisture Can Weight + Moist Soil & BWIA, g	62.95	60.94	60.85	55.67	59.3
Moisture Can Weight + Dry Soil & BWIA, g	53.8841	50.88	49.7684	45.3538	46.7548
Weight of dry soil & BWIA, g	30.6841	27.18	25.6684	20.6538	22.8548
Weight of Water, g	9.0659	10.06	11.0816	10.3162	12.5452
Moisture Content, %	29.5459	37.0125	43.17215	49.9482	54.8909
Dry Density, g/cc	1.0252	1.1977	1.372553	1.20887	1.0612



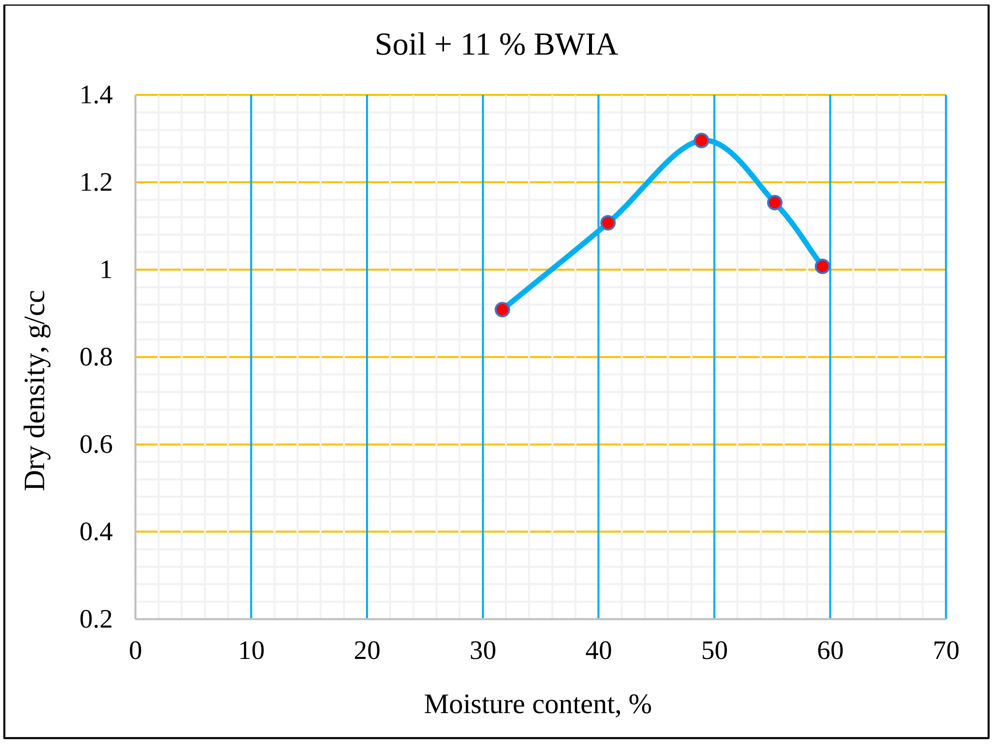
5) Soil + 9 % BWIA

Moisture-Density Relationship for the Soil Blended with 9 % BWIA					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil & BWIA, g	5307.73	5386.61	5516.34	5618.73	5422.58
Moisture Can Number	S-3	P-201	T-55	T-54	T-52
Moisture Can Weight, g	13.2	12.6	12.9	13	23.5
Moisture Can Weight + Moist Soil & BWIA, g	37.22	41.42	42.61	42.64	44.4
Moisture Can Weight + Dry Soil & BWIA, g	31.24	33.57	33.84	33.0674	36.8573
Weight of dry soil & BWIA, g	18.04	20.97	20.64	20.0674	13.3573
Weight of Water, g	5.98	7.85	8.77	9.5726	7.5427
Moisture Content, %	33.1486	37.4344	42.49031	47.7022	56.4687
Dry Density, g/cc	1.01825	1.10181	1.239237	1.3477	1.13992



6) Soil + 11 % BWIA

Moisture-Density Relationship for the Soil Blended with 11 % BWIA					
Trial	1	2	3	4	5
Volume of Mold, g/cc	944	944	944	944	944
Weight of Mold, g	4346.5	4346.5	4346.5	4346.5	4346.5
Weight of Mold + Moist Soil & BWIA, g	5204.17	5391.32	5569.53	5435.37	5297.58
Moisture Can Number	P-101	P-102	PN-4	C-1	C-6
Moisture Can Weight, g	13.4	12.6	13.1	13	12.8
Moisture Can Weight + Moist Soil & BWIA, g	42.11	42.52	43.41	42.14	40.71
Moisture Can Weight + Dry Soil & BWIA, g	35.1869	34.4162	33.4571	31.7742	30.3171
Weight of dry soil & BWIA, g	21.8472	19.8532	20.3571	18.7742	17.5171
Weight of Water, g	6.9231	8.1038	9.9529	10.3658	10.3929
Moisture Content, %	31.6887	40.8186	48.89154	55.213	59.33
Dry Density, g/cc	0.90855	1.1068	1.295583	1.15346	1.0075



7) Variation in OMC and MDD

Variation of OMC and MDD						
Percentage of BWIA	0	3	5	7	9	11
OMC	33	34.5	39.5	43.8	47.5	49.4
MDD	1.52	1.49	1.41	1.37	1.35	1.29

Appendix D: California Bearing Ratio Test Results

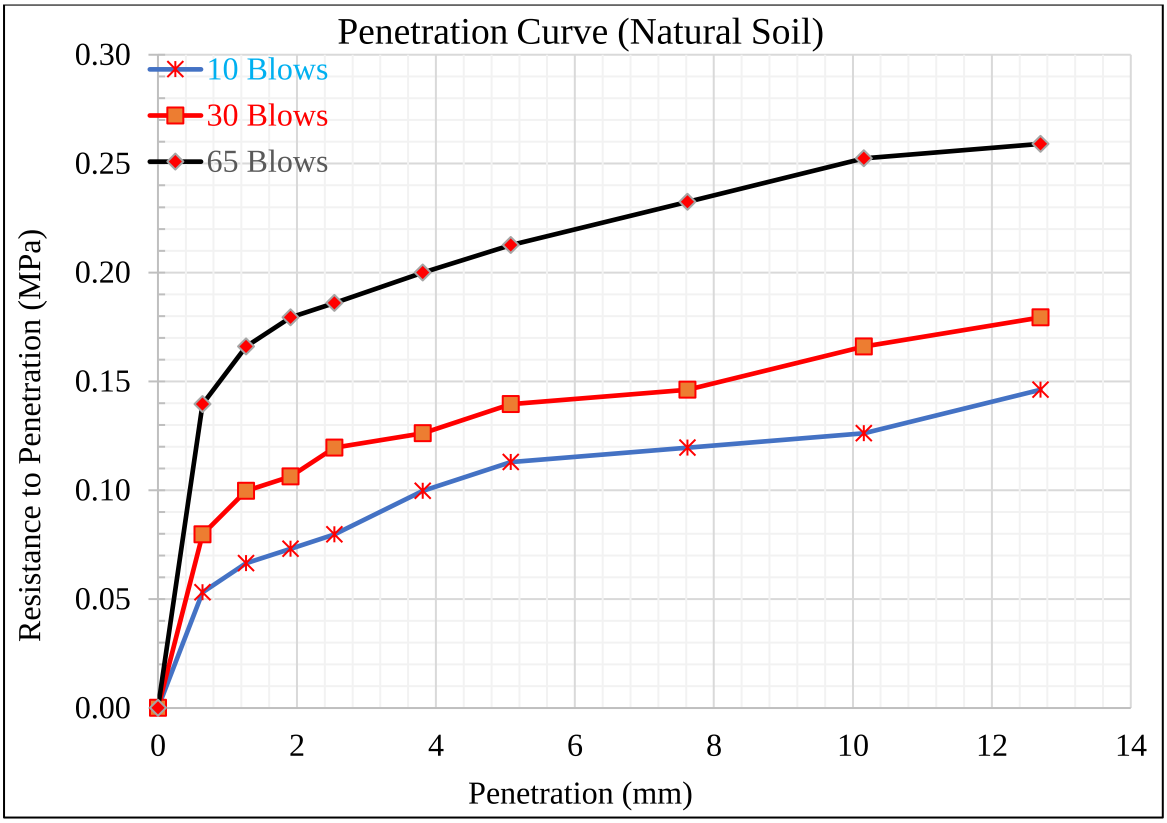
1) CBR for the Natural Soil (0 % BWIA)

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows				30 Blows				65 Blows			
		Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %
0.00		0.00	0.00	0.00		0.00	0.00	0.00		0.00	0.00	0.00	
0.64		4.00	102.83	0.05		6.00	154.24	0.08		10.50	269.92	0.14	
1.27		5.00	128.54	0.07		7.50	192.80	0.10		12.50	321.34	0.17	
1.91		5.50	141.39	0.07		8.00	205.66	0.11		13.50	347.04	0.18	
2.54	**6.90**	6.00	154.24	0.08	1.2	9.00	231.36	0.12	1.7	14.00	359.90	0.19	2.7
3.81		7.50	192.80	0.10		9.50	244.22	0.13		14.50	372.75	0.20	
5.08	**10.30**	8.50	218.51	0.11	1.1	10.50	269.92	0.14	1.4	16.00	411.31	0.21	2.1
7.62		9.00	231.36	0.12		11.00	282.78	0.15		17.50	449.87	0.23	
10.16		9.50	244.22	0.13		12.50	321.34	0.17		19.00	488.43	0.25	
12.70		11.00	282.78	0.15		13.50	347.04	0.18		19.50	501.29	0.26	

Ring Calibration Factor/div.	25.707
Plunger Area, mm^2	1935.00
Rate of strain, mm/min	1.27
Rammer wt. (kg)	2.50
MDD (g/cc)	1.52

No. of Blows	Dry Density (g/cc)	CBR (%)	Swell (%)
10	1.13	1.2	10.3
30	1.29	1.7	9.14
65	1.52	2.7	8.56
Desired Density, g/cc = 95 % MDD 0.95*1.52 = 1.44		1.44	

CBR at 95 % of MDD = 2.31 %



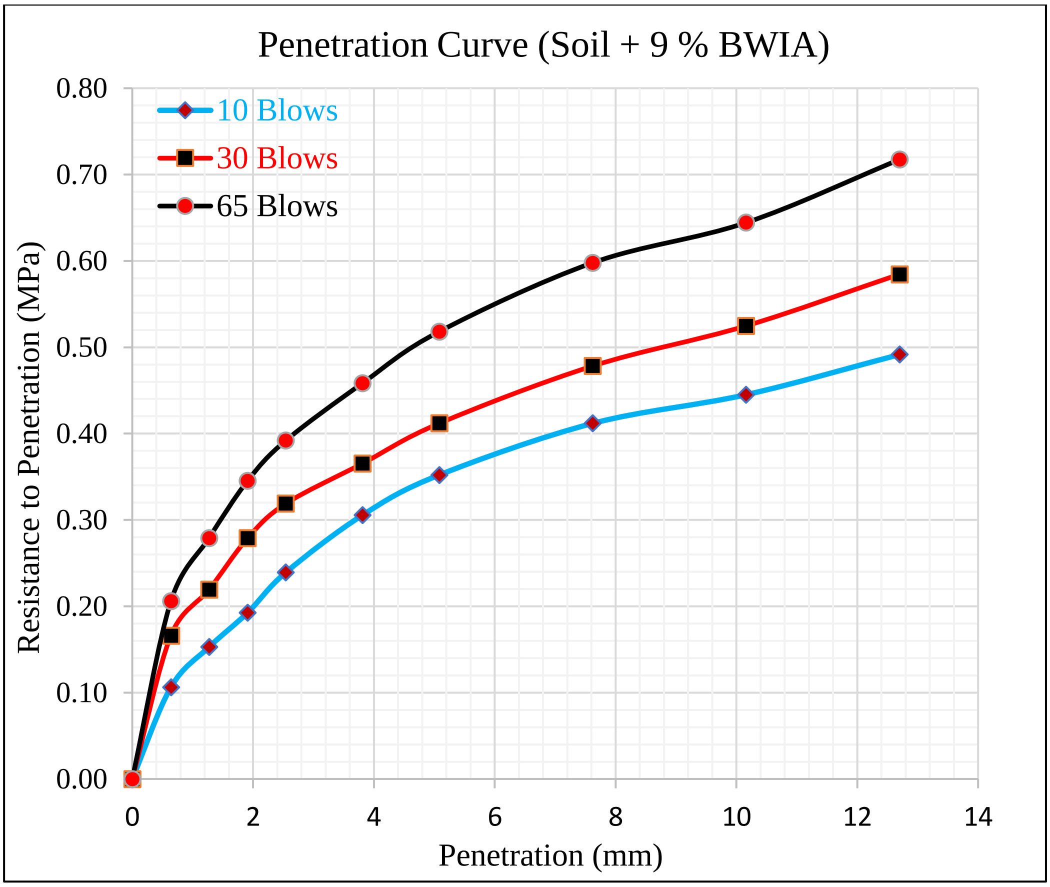
2) CBR for the Soil Blended with 9 % BWIA

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows				30 Blows				65 Blows			
		Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %
0.00		0.00	0.00	0.00		0.00	0.00	0.00		0.00	0.00	0.00	
0.64		8.00	205.66	0.11		12.50	321.34	0.17		15.50	398.46	0.21	
1.27		11.50	295.63	0.15		16.50	424.17	0.22		21.00	539.85	0.28	
1.91		14.50	372.75	0.19		21.00	539.85	0.28		26.00	668.38	0.35	
2.54	**6.90**	18.00	462.73	0.24	3.5	24.00	616.97	0.32	4.6	29.50	758.36	0.39	5.7
3.81		23.00	591.26	0.31		27.50	706.94	0.37		34.50	886.89	0.46	
5.08	**10.30**	26.50	681.24	0.35	3.4	31.00	796.92	0.41	4.0	39.00	1002.57	0.52	5.0
7.62		31.00	796.92	0.41		36.00	925.45	0.48		45.00	1156.82	0.60	
10.16		33.50	861.18	0.45		39.50	1015.43	0.52		48.50	1246.79	0.64	
12.70		37.00	951.16	0.49		44.00	1131.11	0.58		54.00	1388.18	0.72	

Ring Calibration Factor/div.	25.707
Plunger Area, mm^2	1935.00
Rate of strain, mm/min	1.27
Rammer wt. (kg)	2.50
MDD (g/cc)	1.52

No. Oof Blows	Dry Density (g/cc)	CBR (%)	Swell (%)
10	1.13	3.5	8.28
30	1.25	4.6	7.51
65	1.37	5.7	6.82

CBR at 95 % of MDD = 5.1 %



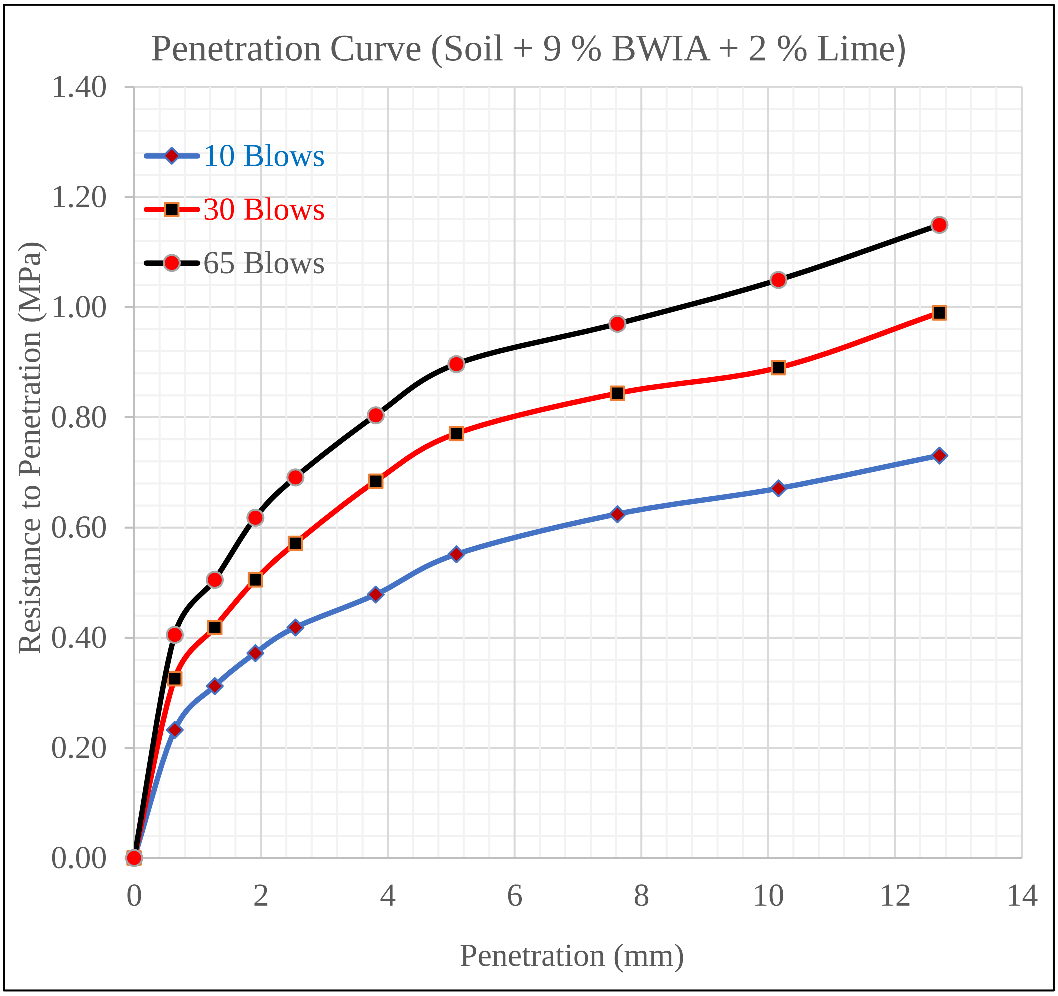
3) CBR for the Soil Blended with 9 % BWIA and 2 % Lime

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows Gauge read.	10 Blows Load N	10 Blows Stress N/mm^2	10 Blows CBR %	30 Blows Gauge read.	30 Blows Load N	30 Blows Stress N/mm^2	30 Blows CBR %	65 Blows Gauge read.	65 Blows Load N	65 Blows Stress N/mm^2	65 Blows CBR %
0.00		0.00	0.00	0.00		0.00	0.00	0.00		0.00	0.00	0.00	
0.64		17.50	449.87	0.23		24.50	629.82	0.33		30.50	784.06	0.41	
1.27		23.50	604.11	0.31		31.50	809.77	0.42		38.00	976.87	0.50	
1.91		28.00	719.80	0.37		38.00	976.87	0.50		46.50	1195.38	0.62	
2.54	**6.90**	31.50	809.77	0.42	6.1	43.00	1105.40	0.57	8.3	52.00	1336.76	0.69	10.0
3.81		36.00	925.45	0.48		51.50	1323.91	0.68		60.50	1555.27	0.80	
5.08	**10.30**	41.50	1066.84	0.55	5.4	58.00	1491.01	0.77	7.5	67.50	1735.22	0.90	8.7
7.62		47.00	1208.23	0.62		63.50	1632.39	0.84		73.00	1876.61	0.97	
10.16		50.50	1298.20	0.67		67.00	1722.37	0.89		79.00	2030.85	1.05	
12.70		55.00	1413.89	0.73		74.50	1915.17	0.99		86.50	2223.66	1.15	

Ring Calibration Factor/div.	25.707
Plunger Area, mm^2	1935.00
Rate of strain, mm/min	1.27
Rammer wt. (kg)	2.50
MDD (g/cc)	1.52

No. Oof Blows	Dr Density (g/cc)	CBR (%)	Swell (%)
10	1.15	6.1	4.73
30	1.35	8.3	3.71
65	1.48	10	3.15

CBR at 95 % of MDD = 9.1 %



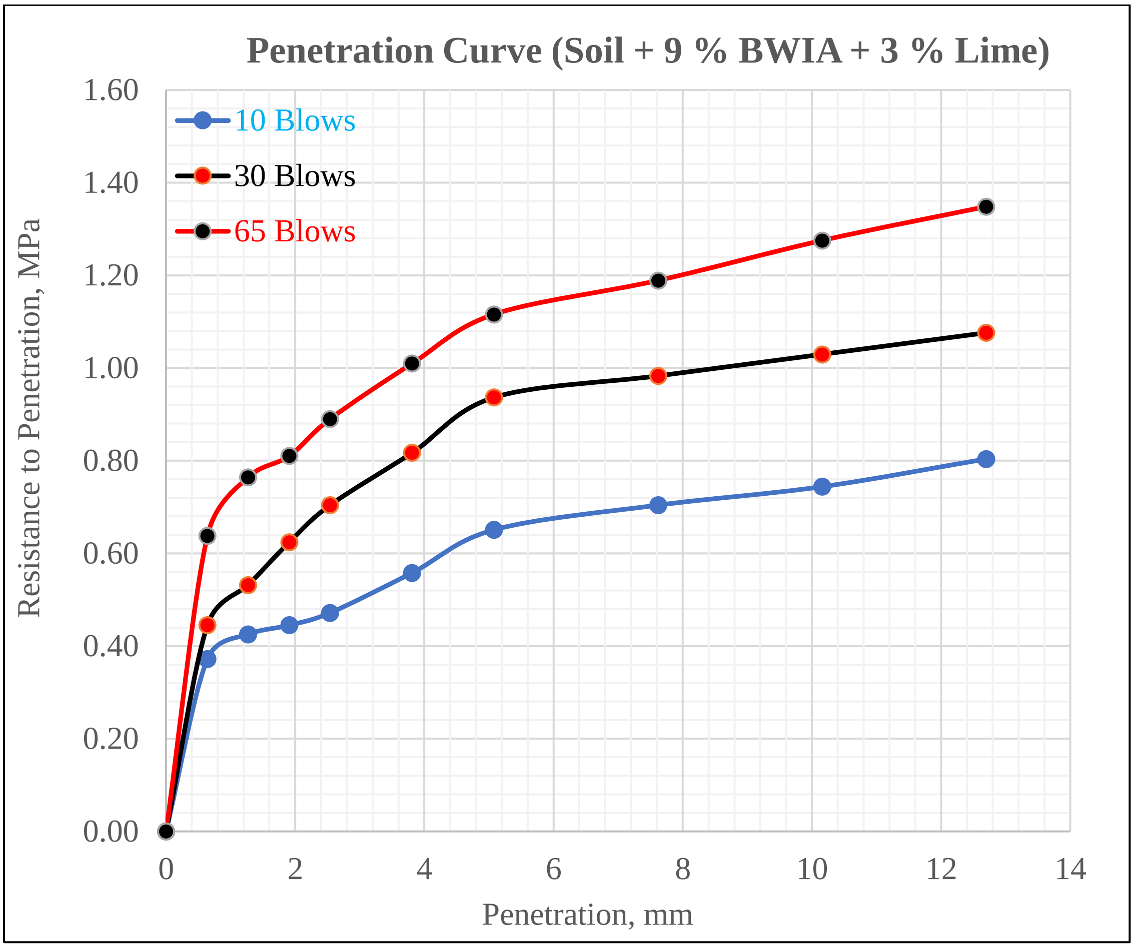
4) CBR for the Soil Blended with 9 % BWIA and 3 % Lime

Penetration (mm)	Stand. Stress (N/mm^2)	10 Blows				30 Blows				65 Blows			
		Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %	Gauge read.	Load N	Stress N/mm^2	CBR %
0.00		0.00	0.00	0.00		0.00	0.00	0.00		0.00	0.00	0.00	
0.64		28.00	719.80	0.37		33.50	861.18	0.45		48.00	1233.94	0.64	
1.27		32.00	822.62	0.43		40.00	1028.28	0.53		57.50	1478.15	0.76	
1.91		33.50	861.18	0.45		47.00	1208.23	0.62		61.00	1568.13	0.81	
2.54	**6.90**	35.50	912.60	0.47	6.8	53.00	1362.47	0.70	10.2	67.00	1722.37	0.89	12.9
3.81		42.00	1079.69	0.56		61.50	1580.98	0.82		76.00	1953.73	1.01	
5.08	**10.30**	49.00	1259.64	0.65	6.3	70.50	1812.34	0.94	9.1	84.00	2159.39	1.12	10.8
7.62		53.00	1362.47	0.70		74.00	1902.32	0.98		89.50	2300.78	1.19	
10.16		56.00	1439.59	0.74		77.50	1992.29	1.03		96.00	2467.87	1.28	
12.70		60.50	1555.27	0.80		81.00	2082.27	1.08		101.50	2609.26	1.35	

Ring Calibration Factor/div.	25.707
Plunger Area, mm^2	1935.00
Rate of strain, mm/min	1.27
Rammer wt. (kg)	2.50
MDD (g/cc)	1.52

No. Oof Blows	Dry Density (g/cc)	CBR (%)	Swell (%)
10	1.2	6.8	2.22
30	1.39	10.2	1.89
65	1.51	12.9	1.69

CBR at 95 % of MDD = 11.2 %



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

Issue Date: -25/08/2021

Customer Name:- Asfachew Belet

Request No:- GLD/RQ/19/21

Report No:- GLD/RN/757 B/21

Sample type : Cement

Sample Preparation: - 200 Mesh

Date Submitted:-14/07/2021

Number of Sample:- One (01)

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

Analytical Method: $LiBO_2$ FUSION, HF attack, GRAVIMETERIC, COLORIMETRIC and AAS

Collector's code	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	TiO_2	H_2O	LOI
BWIA	37.96	14.11	1.12	28.86	4.10	1.94	1.02	0.08	0.75	0.49	2.61	6.93

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

Analysts	Checked By	Approved By	Quality Control
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Nigist Fikadu			
Yirgalem Abraham			

Geological Survey of Ethiopia Central Geological Laboratory