A Study on the Impact of Wetting-Drying Cycles on Physical and Mechanical Properties of Expansive Soil Treated with Fly Ash: In Case of GEDA Special Economic Zone

Biniyam Abebayehu Demisse

A Thesis Submitted to the department of Civil Engineering
School of Civil Engineering and Architecture

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

Office of Graduate Studies
Adama Science and Technology University

June 2023

Adama, Ethiopia

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DECLARATION

I declare that this Masters thesis entitled “A Study on the Impact of Wetting-Drying Cycles on Physical and Mechanical Properties of Expansive Soil Treated with Fly Ash: In Case of GEDA Special Economic Zone" is my work and has not been submitted to any university for a similar thesis. The references used in this thesis are duly recognized by proper citations.

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RECOMMENDATION OF ADVISOR

I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "A Study on the Impact of Wetting-Drying Cycles on Physical and Mechanical Properties of Expansive Soil Treated with Fly Ash: In Case of GEDA Special Economic Zone" prepared under my guidance by Biniyam Abebayehu. Therefore, I recommend the submission of revised version of the thesis to the department for further review and evaluation.

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Advisor	Signature	Date

APPROVAL PAGE

I, the advisors of the thesis entitled "A Study on the Impact of Wetting-Drying Cycles on Physical and Mechanical Properties of Expansive Soil Treated with Fly Ash: In Case of GEDA Special Economic Zone" and developed by Biniyam Abebayehu, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Srikanth Vadlamudi(Ph.D.)	_____________	_________________
Advisor	Signature	Date

APPROVAL OF BOARD OF REVIEWERS

We, the undersigned, members of the Board of Examiners of the thesis by Biniyam Abebayehu have read and evaluated the thesis entitled "A Study on the Impact of Wetting-Drying Cycles on Physical and Mechanical Properties of Expansive Soil Treated with Fly Ash: In Case of GEDA Special Economic Zone" and examined the candidate during open defense. This is, therefore, to certify that the thesis accepted for partial fulfillment of the requirement of the degree of Masters of Science in Geotechnical Engineering.

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LIST OF ACRONYMS

AASHTO	American Association of States Highway and Transportation Officials
ASTM	American Society of Testing and Materials
CBR	California Bearing Ratio
ERA	Ethiopian Road Authority
FAAS	Flame Atomic Absorption Spectroscopy
GI	Group Index
Gs	Specific Gravity;
GSEZ	Geda Special Economic Zone
LL	Liquid Limit
MISW	Municipal industrial solid waste
MDD	Maximum Dry Density
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System

LIST OF ABBREVIATIONS

F	Fly ash
gm	gram
gm/cm^3	Gram per Centimeter Cube
kpa	Kilopascal
mm	Millimeter
S/S	Stabilization/Solidification method
Wt	Weight

ABSTRACT

Expansive soils undergo high volume change due to cyclic swelling and shrinkage behavior during the wet and dry seasons. As a result, when using such problematic soils as subgrade materials or foundation bedding, care should take to avoid them entirely or to treat them appropriately. Previous research on treating expansive soil mostly focused on the improvement of soil but not the durability of such improvements. In this paper, the emphasis laid on the relevance of fly ash, which is from municipal industrial solid waste (MISW) incinerator, suggested as a stabilizing agent for expansive soil. On the native expansive soil and when mixed with 10%, 20%, 30%, and 40% fly ash, tests for particle size analysis, Atterberg limits, free-swell, compaction, California bearing ratio (CBR), and unconfined compressive strength were performed according to ASTM. The findings demonstrate that increment of fly ash decreases plasticity index, Free swell, CBR swell, optimum moisture content (OMC) and maximum dry density (MDD). In reverse, CBR %, and UCS are increased with increasing fly ash content. The greater improvements were shown on 60% expansive soil and 40% fly ash, after the determination of better improvement of soil-fly ash mixture specimens allowed to cure for 7, 14, and 28 days; the maximum result found on 28 days, according to UCS testing. Thus, impact of wetting and drying cycles on physical and mechanical properties of expansive soil improved by 40% fly ash after 28-day curing also studied accordingly. For the cyclic wetting-drying tests, which primarily involve cyclic swelling potential and cyclic strength tests, expansive soil samples from both treated and untreated soil are prepared. According to the results of an experiment, fly ash effectively reduces the swelling potential of expansive soil samples and maintained good strength, which prevents them from collapsing when submerged in water. In contrast, untreated soil samples would collapse when it submerged in water. Samples of expansive soil that have been stabilized with 40% fly ash exhibit sufficient durability in terms of swelling capacity and strength, indicating that both the mechanical and physical characteristics of the expansive soil have been successfully improved.

Keywords/terms: Wetting and drying cycles; Expansive soil improvement; Fly ash; Compaction; California bearing ratio (CBR); unconfined compressive strength (UCS).

CHAPTER ONE

INTRODUCTION

1.1 Background

Expansive soils are ones whose volume changes dramatically because of changes in moisture levels. Therefore, it is important to reduce the issues caused by expanding soils and stop structural cracking (Suwantara et al., 2019). Asia, and a few more nations like Ethiopia, South Africa, Tanzania, that are African have expansive soils (Tewelde, 2019). The majority of it is in the country's central region, along the key thoroughfares connecting Addis Abeba with Ambo, Wolliso, Debrebirhan, Gohatsion, and Modjo. Expansive soils are also seen in places like Mekelle and Gambella (Tewelde, 2019). Often troublesome and unsuitable for engineering building, these soil types (Ikeagwuani et al., 2021). As a result, numerous studies have been carried out by researchers all over the world, giving rise to the idea of soil stabilization, which entails enhancing the physical and engineering properties of a soil by blending it with pozzolanic or cementitious additives (chemical stabilization) to increase its strength and decrease its susceptibility to water. Stabilized soil regarded as stable if it can support vehicular (super structural) loads with the permitted amount of distortion. In the past, lime and cement were the two well-known conventional stabilizers utilized for the stabilization of expansive soils all over the world, according to (Ikeagwuani et al., 2021). Nevertheless, geotechnical engineers' attention has increasingly switched to the use of various industrial and agricultural solid wastes (bi-products), such as to enhance efficiency and control wastes that can lead to environmental contamination, employ fly ash, bagasse ash, teff straw ash, wheat straw ash, and rice husk ash, among other materials (James J. and Pandian, 2018).

Ethiopia has broad soils that predominate heavily on its rich, suited for agricultural and forested plains, like many other nations. The stability of the foundation and/or subgrade determines the stability of any structure or pavement. For instance, pavement subgrade made of expansive soil known to have stability issues and has to be improved. Several soil improvement methods, such as soil stabilization, can used to increase the bearing capacity and shear strength of such soil. When the soil is unsuitable for the intended purpose, the process of enhancing the engineering qualities of the soil is known as soil stabilization (Arora, 2003). For many years, additives like bitumen, cement, and lime have been used by road and geotechnical engineers to enhance the local soil's qualities. Laboratory and field testing have shown that the application of such chemical stablizers can improve the stability and strength of poor soils (Singh & Mittal, 2020).

The search for low-cost and waste materials such bagasse ash, fly ash, rice husk ash, and others as stabilizing agents for either full or partial replacement of traditional stabilizers like cement and lime has recently gained increased attention from researchers (Yadu et al., 2011). The utilization of waste material should be promoted for sustainable development in order to preserve natural resources for future generations. In the construction of infrastructures especially highways, fly ash is a common industrial waste product (Singh & Mittal, 2020). Many previous researchers investigate the improvement of problematic soil with this industrial waste product. However, these previous studies mostly not investigate the effect related to seasonal fluctuation. Therefore the current study, concentrated its attention mainly on improving the expansive soil with fly ash and the wetting-drying cycles effect on the physical and mechanical properties of expansive soil treated by fly ash to simulate the impact of seasonal fluctuation.

1.2 Statement of the Problem

Expansive soils are a serious threat to construction of infrastructures built above on it. (Reddy & krishna, 2018). During the wet and dry seasons, expansive soils exhibit exceptionally strong swelling and shrinkage properties, respectively. Expansive soils have a strong tendency to swell and shrink, which causes cracks in the roads, buildings, and railways constructed on them, rendering them unsuitable for foundation construction (Reddy & krishna, 2018).

Expansive soils predominate in the majority of the regions along the Geda special economic zone. These soils have an expanding characteristic when they come into touch with water, making them unsuitable for building infrastructures. Soil stabilization techniques enhance the qualities of expansive soil, and many researches had been used stabilization techniques by using different stabilizer materials. However, most of these studies are ignored the study related to impact of seasonal fluctuation on the improved expansive soil after a certain seasons.

Due to urbanization and growth of population, the establishment of industries and volume of municipal solid wastes increased in rapid rate. From these industries the amount of byproducts and inclinator waste increased from time to time. These situation need to be handled properly by utilizing these wastes in different ways to minimize the deposit of such wastes. For current study, fly ash from municipal solid waste inclinator used as a stabilizer to enhance the expansive soil. Moreover, tolerance to the seasonal variations needs additional research. Therefore the current research is on the “impact of the wetting and drying cycle on the physical and mechanical properties of expansive soil treated with fly ash" is thus necessary to determine the efficacy and durability of such stabilizer following seasonal fluctuation.

1.3 Objectives of the study

1.3.1 General Objective

The general objective of this study is to investigate the impact of wetting and drying cycle on the physical and mechanical Properties of expansive soil treated with fly ash.

1.3.2 Specific Objective

1) To determine the physical and mechanical properties of untreated expansive soil
2) To determine the optimum percentage of fly ash that improves the expansive soil at most.
3) To investigate the effect of fly ash on expansive soil stabilization
4) To investigate the impact of cyclic swelling potential on both untreated and fly ash treated expansive soil.

1.4 Research Questions

- What are the physical and mechanical properties of untreated expansive soil?
- What optimum percentage of Fly ash gives a better improvement of expansive soil?
- What effects have fly ash on expansive soil improvement?
- What is the impact of wetting- drying cycles on treated and untreated expansive soil?

1.5 Significance of the Study

On the current study, the use of industrial municipal solid incinerator waste materials like fly ash as a soil stabilizer to improve the expansive soil's engineering properties studied. In addition, soil-fly ash mix improved specimens, ability to withstand in seasonal fluctuation studied in the current study. These improved soil-fly ash mix specimens' ability to withstand during seasonal variations studied by applying wetting and drying cycles for the same specimen, which simulates the seasonal fluctuation in different seasons. In general, this study made it possible to determine the improvement and durability of expansive soil -fly ash mix.

1.6 Scope and Limitation of the Study

1.6.1 Scope of the Study

The scope of this study focused on the addition of fly ash on expansive soils to enhance its engineering properties and to investigate impact of wetting and drying cycles on physical and mechanical properties expansive soil treated by fly ash. The experimental program carried out for expansive soil and fly ash alone, expansive soil fly ash mix with different percentage, and finally the treated and untreated expansive soil exposed to wetting and drying cycles to check the physical and mechanical properties. Form three test pits at 1.5m depth sample was taken to investigate the index and engineering properties of soil examined by the followed tests:- Sieve analysis, Hydrometer analysis, Moisture content, Atterberg limits, specific gravity, free swell, compaction, CBR and UCS. The stabilization of soil carried out for only one test pit that had the poorest quality, which is low CBR and UCS value.

1.6.2 Limitation of the Study

Due to time, laboratory equipment and budget constraints, only some laboratory tests conducted, like index properties, compaction, CBR and UCS on treated and untreated expansive soil determined in the laboratory. Extra laboratory and field tests on soil and soil-fly ash mix would have to be conduct.

1.7 Structure of the study

The study presented in five chapters, the first of which provides background information, problem statements, general and specific study objectives, study significances, and study scope and limitations. The second chapter includes relevant previous studies on the industrial wastes, the impact of wetting and drying cycles on treated expansive soil, and the chemical stabilization of expansive soils using various pozzolanic additives. The third chapter details the study's materials, study area, detailed work method, and carried out laboratory tests. The results of the laboratory experiments discussed in chapter four along with the impact of the wetting and drying cycles on the physical and mechanical properties of the expansive soil treated with fly ash, as well as index properties, compaction, CBR, and UCS. The conclusion reached based on the outcomes of the laboratory tests presented in the fifth chapter, along with recommendations for further investigation. Lastly, the references cited in the document and tabulated laboratory results and pictures attached in the appendix.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This literature review aims to offer a fundamental framework for a better understanding of expansive soils, as well as its concerns related to civil engineering work, for a better understanding of soil stabilization using the industrial by-product, and for a better understanding of durability with seasonal fluctuation(swell and shrink).

2.2 Swell and Shrink Behaviors

The configuration of the thin crystal lattice layers that clay particles create determines their shape. In shrink-swell clays, water is attracted to and held between the crystalline layers (and on their surfaces) in a firmly bonded "sandwich," according to the molecular structure and arrangement of the clay crystal sheets. Water molecules' electrical dipole structure causes an electrochemical attraction to the microscopic clay sheets. Adsorption is the process by which the water molecules get attached to the clay sheets. Na-smectite clays (montmorillonite) have greater affinity for water and thus, adsorb abundant water molecules between its clay sheets. Theoretically, they can increase in volume by an 800 fold factor, leading to the dispersion of clay platelets by removing repulsive interlayer forces. As a result, they have a significant shrink-swell potential, similar to vermiculite and chlorite, which also show crystalline swelling. Water molecules between the clay sheets in saturated clay shrink-swell soils cause the bulk volume of the soil to increase or decrease with variations in soil water content. This absorption process loosens the inter-clay bonds, resulting in a decrease in soil strength. As there is reduction in soil moisture due to evaporation or by gravitational forces, the water in between the clay sheets is released leading to an overall decrease in the soil volume, thus causing shrinkage. The process also results in voids and desiccation cracks. Shrinkage and swelling typically take place up to a depth of around 3 m in the near-surface, however this might change depending on the climate. The initial water content, internal structure, void ratio, type and quantity of clay minerals and vertical stresses in expansive soils, all affect their shrink and swell potential (Bell & Culshaw, 2001). These minerals, which include smectite (montmorillonite, nontronite), illite, vermiculite, and chlorite control the soil's natural expansiveness. Generally, the shrink-swell potential increases with the soil mineral concentration. However, when other non-swelling minerals like quartz and carbonite are present, it may help in mitigating these effects (Kemp et al. 2005).

As long as the soil's water content is fairly constant, even soils with a high shrink swell potential won't typically pose any problem. This is dependent on the soil mineralogy and water status, variations in water content, and the rigidity and geometry of a building built upon it (Houston et al. 2011). Suction or variations in water content in a soil saturated partially enhance the risk of harm, while clay mineralogy in a soil saturated fully regulates the behavior of swelling and shrinkage.

2.2.1 Processes taking place in swelling soils during drying and wetting

Different processes take place when a swelling soil dries or swells. On drying the soil decreases its volume by shrinkage, and desiccation cracks appear because of internal stresses in the shrunken and dried soil mass. These cracks are created in pre-existing planes of weakness within soil clods. As a result of shrinkage, soil decreases its height by subsidence. On wetting the soil increases its volume by swelling, the cracks are closed, and soil level rises. The process of swelling is mainly caused by the intercalation of water molecules entering to the inter-plane space of smectitic clay minerals (after Low and Margheim 1979, Schafer and Singer 1976, Parker et al. 1982). An schematic visualization of this process is depicted. The expansive characteristics of smectites are affected by the nature of adsorbed ions and molecules. Smectite increases its plane spacing as a result of the loss of adsorbed cations.

2.2.2 Consequences of soil swelling

Soil volumetric changes may cause both unfavorable and favorable effects on human activities. Unfavorable effects are the destruction of buildings, roads and pipelines in uncropped soils, and the leaching of fertilizers and chemicals below the root zone through desiccation cracks (by pass flow). In these soils horizontal cracks break capillary flux of water. On the other hand, swelling clays can be used to seal landfills storing hazardous wastes. This sealing avoids the downward migration of contaminants to groundwater. In cropped soils, the development of a dense pattern of cracks on drying improves water drainage and soil aeration, and decreases surface runoff in sloped areas. Soil cracking is closely related to the recovery of porosity damages by compaction. For a complete revision of these topics, the lecture of a review by A. R. Dexter (1988) recommended. Here in, a conceptual model describing the sequence of paths leading to the development of desiccation cracks provided. Tensile stresses are developed on drying, which to the creation of primary, secondary and tertiary cracks. The cracks are, at the same time, future void spaces, and represent the walls of the future aggregates. This sequence of paths is believed to recover soil porosity in a previously compacted soil layer.

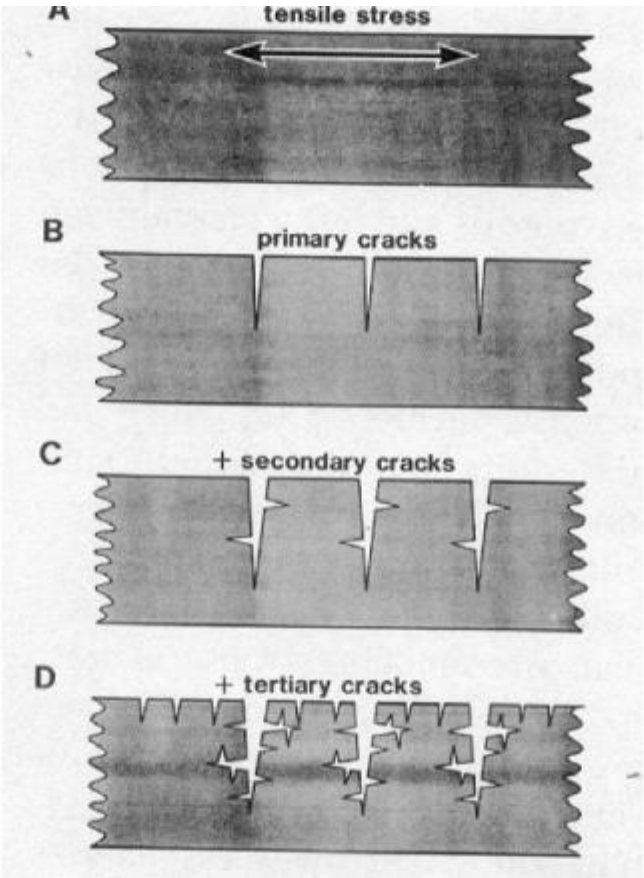


Figure 2. 1 Conceptual model describing the development of primary, secondary and tertiary cracks, resulting from the buildup of tensile stresses on drying (taken from Dexter 1988).

2.2.3 Mechanism behind the Swelling and Shrinkage

Behavior Water gets attached to the surfaces of clay through mechanisms which include:

1. Dipolar electrostatic attraction,

2. Orientation of water molecules to the charged sites,

3. Hydrogen bonding with the oxygen atoms exposed on clay crystals

4. Adsorbed cations associated with the clay (it depends on the type of cations present and clay's cation adsorption capacity).

Swelling pressures develop when a confined body of clay exposed into contact with or is made to sorb water from a solution present externally. The pressure applied to a body of soil, which is exposed for imbibing water, to prevent it from expansion or the the pressure which a soil body exerts on the rigid walls of a container, when it is exposed for imbibing water from a reservoir at atmospheric pressure is called as swelling pressure. It depends upon the difference in osmotic pressures between the external solution and the adsorbed water. The enveloping water thickness in a partially hydrated micelle is less than the potential thickness of the fully expanded diffuse double layer. If available, the truncated double layer will tend to expand to its full potential thickness and dilution by osmotic absorption of additional water.

Each micelle's swarm of positively charged cations repels those of the adjacent micelles as it expands. As a result, the micelles move away from each other causing the whole soil system to swell, which can also lead to closing of larger pores (reducing the system's permeability).

Since there are often twice as many monovalent cations as divalent cations, the osmotic attraction of a clay assemblage for "external" water is typically twice as high with the former. Therefore, swelling is highest in the presence of distilled water as the external solution and monovalent cations like sodium. When calcium occurs as a dominating cation in the exchange complex, there is a great reduction in swelling. The similar effect is caused when trivalent aluminum is present at low Ph. The factors which suppress swelling also includes soil solution with higher salinity. But when a saline soil, having a predominance of sodium salts, gets leached of the excess salts with fresh water, without adding concurrently gypsum or calcium ions, swelling occurs as a result of the higher amounts of sodium ions present in the adsorbed phase.

Therefore, swelling is dependent on:

1. Clay mineral present (its type and quantity),

2. Specific surface area of clays (generally, swelling increases as the specific surface area increases)

3. Soil particles orientation or arrangement,

4.Presence of cementation between the particles (by aluminum or iron oxides, carbonates or humus).

5. Depth of layers of soil (when there is swelling of soils in the surface layers on wetting and subsequent shrinking of soils on drying, the underlying layers of soil get prevented from swelling due to the confinement of the upper layer soil, which is called as the overburden pressure).

When a clay body that has been hydrated is dried, shrinkage takes place instead of swelling. When the process of shrinking begins in the field, the surface soil starts forming various cracks, that disrupts the mass of soil into various fragments, which range small microaggregates to macroaggregates and /or large blocks. For example, as occurs in soils rich in montmorillonite clays like expansive soil. When the soil is exposed to alternating wetting and drying, as occurs in semiarid regions, soils start to heave and then settle, which leads to the formation of wide, deep cracks and slanted sheer planes that extend deep into the soil profile.

2.3 Expansive Soils

A clay soil is one that contains more than 30% clay. A naturally occurring substance with a high degree of flexibility and a propensity for volume changes in reaction to variations in moisture content is referred to as "clay." It is mostly composed of fine-grained minerals. This implies that because of the water they collect during the rainy or wet seasons, they swell and shrink in size during the summer or dry seasons as the water evaporates from them (Sridharan & Prakash, 2016). Due One of the challenging clay soils that many geotechnical engineers experience in the field is expansive soil because of variations in volume proportional to changes in water content (Lucian C., 2008). The expansive clayey soil has acquired a number of names, including gumbo soils, buckshot soils, and black cotton soils (Sridharan & Prakash, 2016). These soils also referred to as "cracking soil" because, during the summer or other dry seasons, they have the propensity to shrink and crack when deprived of water. Civil engineering constructions suffer significant cracking because of such soils' alternating seasonal expansion and contraction. Lightly laden buildings suffer severe cracking because they are unable to resist the swelling pressure brought on by expansive soils. This poses serious issues for foundations, roadways, sidewalks, pipelines, excavations, and commercial, industrial, and agricultural activities. Damage-causing issues include pavement cracking, heaving floors, and diagonal cracks above doors and windows (Magdi & Asim, 2017). Expansive soils include a high concentration of clay minerals, which may use to show the potential for expansion (Lucian C., 2008).

2.4 Formation of Expansive Soils

The varieties of clay created depend on the parent material's composition throughout the early and intermediate phases of weathering. In these stages as opposed to after prolonged, extensive weathering, the parent material's nature is far more crucial time (Kassahun, 2013) .The parent elements that give birth to expansive soil divided into two classes according to (James et al., 2022). The fundamental igneous rocks make up the first group, while the sedimentary rocks in the second group contain montmorillonite, which physically decomposes to generate expansive soil. Igneous rock includes basalts, volcanic glass, and gabbro. The sedimentary rock class consists of limestone, marls high in magnesium, clay stone, and shales (Teklu, 2003).

2.5 Clay Mineralogy and their Characteristics

The mineralogy of soil particles has a significant role in influencing the size, shape, and engineering qualities of the soil. Small, crystalline compounds known as clay minerals developed predominantly from the chemical weathering of certain rock-forming minerals. These are a collection of hydrous aluminum phyllosilicates, also known as layer silicates, together with other metallic ions. The nature and composition of the minerals in soil have such a significant impact on the fine-grained soils' engineering qualities. As a result, expansive soil behavior is primarily influenced by clay mineralogy (Das and Roy, 2014). Kaolinite, montmorillonite, and illite make up the three primary categories of clay minerals. Repeating layers of one silica film and one alumina sheet make up kaolinite. Kaolinite is referred to as a 1:1 clay mineral due to the fact that it is composed of one layer of each of the two fundamental sheets (Figure 2-1). Hydrogen bonds, a type of strong binding that inhibits hydration and expansion, hold this fundamental layer together. They respond to changes in water content by shrinking and expanding less than expected (Lancellotta, 2008).

Kaolinite: Its structural component consists of two sheets of alumina and silica that linked together by hydrogen bonds, which shown in Figure 2.1. It is challenging to separate the layers because of the strong link that holds them together. Kaolinite is hence fairly stable and water cannot get through the layers. Kaolinite has a low degree of expansiveness as a result (Lancellotta, 2008).

Montmorillonite: Among all the clay minerals, it is the one that is most frequently found and is widely recognized for its swelling abilities. An alumina sheet sandwiched between two silica sheets makes up the bulk of the structure. The fundamental montmorillonite units are layered one on top of the other, but the bonds connecting them are relatively weak, making it easy for water to get between the sheets, causing the sheets to split and subsequently swell,which shown in Figure 2.1. Montmorillonite has a very high degree of expansiveness as a result (Lancellotta, 2008).

Illite: Illite has a 2:1 clay mineral because it is made up of a sequence of single aluminum octahedral sheets sandwiched between two silicon tetrahedral sheets. Illite and montmorillonite share a similar fundamental structure which shown in Figure 2.1. Yet, non-exchangeable potassium ions serve as the bonds that hold the fundamental illite units together. As the illite units are stable as a result, they experience substantially less mineral swelling than montmorillonite (Lancellotta, 2008).

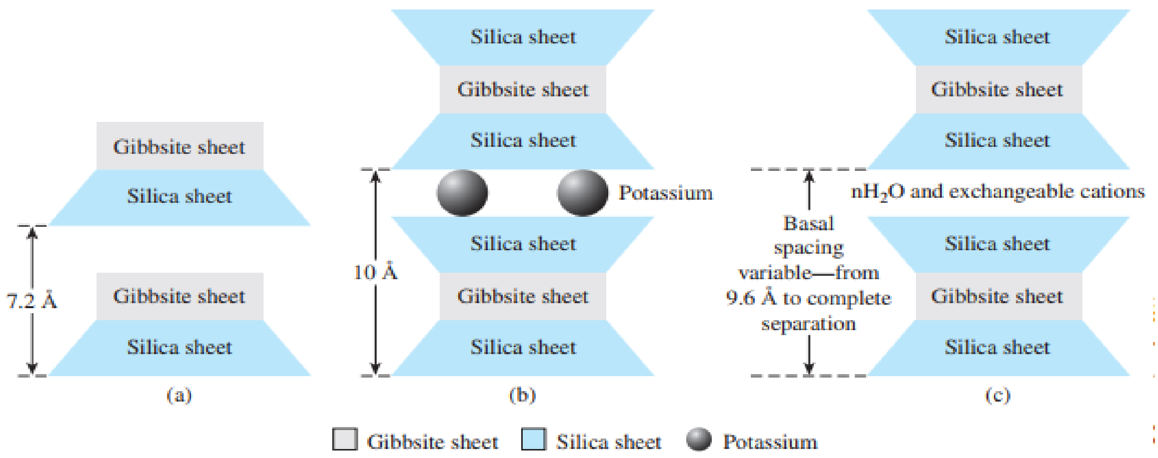


Figure 2. 2 Diagram of the structures of (a) kaolinite; (b) illite; (c) montmorillonite (Lancellotta, 2008)

2.6 Recognition of Expansive Soils

Each geotechnical engineering project must start by identifying and describing the subsurface condition since this information is crucial for choosing the right sample, testing, design, and construction processes. The geotechnical techniques for identifying expansive soils may be roughly categorized into field and laboratory techniques. While laboratory approaches use a variety of specific testes, field techniques often rely on visual identification (Lancellotta, 2008). This topic's main goal is to explore several methods that frequently used to recognize expansive soils.

2.6.1 Field Identification

The objective of the site visit should be to gain enough information about the history of the site. In addition to this, experienced geotechnical engineers are visually identifying potentially expansive soils. Some of the important method that indicates the potential for the expansiveness of a soil as discussed by (Lancellotta, 2008) is: They have mostly the color of black and gray, the appearance of cracking in nearby structures, pattern of polygonal desiccation, or “shrinkage cracks” in the dry season and close down in the rainy season. Moreover 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 (Lancellotta, 2008)

2.6.2 Laboratory Identification

There are a few laboratory techniques available to identify swelling soils. Mineralogical identification and inferential testing techniques (indirect and direct approaches) are the two major categories into which the identification of problematic soil in the laboratory may be generally categorized (Sridharan & Prakash, 2016).

2.6.2.1 Mineralogical Identification Methods

According to (Kassahun, 2013), mineralogical identification can be helpful in evaluating the material but it is inadequate by itself. Thereby further, for better and more accurate findings, it should use in conjunction with other methods. Clay mineralogy is the primary determinant of expansive soil behavior and may found using a variety of methods. X-ray diffraction, chemical analysis, dye adsorption, differential thermal analysis, and scanning electron microscopy are some of the methods that can apply.

The various methods of mineralogical identification are crucial in a research laboratory to investigate the fundamental properties of clay, but they are not appropriate for routine tests because they take a long time, call for expensive test equipment, and require specially trained technicians to interpret the results (Al-Rawas & Mattheus, 2006).

2.6.2.2 Indirect methods

These methods evaluate the swelling potential of expansive soils by using simple soil property tests. Such tests are easy to perform and should be included as routine tests in the investigation of expansive soil. These methods include the index property, potential volume change, and activity methods, which are valuable tools in evaluating the swelling property (Asres E., 2017).

2.6.2.3 Direct methods

This method is the most satisfactory and committed method of determining the swelling pressure and swelling potential of expansive soil and also provides the most valuable data by direct measurement and tests are simple to perform. Direct measurement of expansive soil can be made using a standard one – dimensional consolidometer (Kamil K., 2011). To avoid erroneous conclusions, testing should done on a variety of samples.

2.7 Distribution of Expansive Soils

Expansive soils are present across the world, although they are most common in semi-arid regions of the tropical and temperate zones (Kassahun, 2013). As a result of their swelling during the rainy season and shrinking during the dry season, expansive soils damage engineering structures (Mishra & Rao, 2008). Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Israel, Iran, Mexico, Morocco, Zimbabwe, South Africa, Spain, Turkey, United States of America, and Venezuela are some of the countries (Ehitabezahu, 2011) identifies as having vast terrain (Fikreyesus, 2022). Expansive soils have been created in eastern Africa from the basaltic rocks of the Ethiopian plateau, and they are also widely distributed as alluvial deposits in southern Sudan (Admas, 2020). Expansive soils are prevalent over much of Ethiopia (Alene, 2010). Figure 2.2 shows that Distribution of Expansive soils in Ethiopia (Admas, 2020).

Figure 2. 3 Distribution of Expansive Soils in Ethiopia (Admas, 2020)

2.8 Classification of Expansive Soils

The purpose of an identification and classification system for expansive soils is to characterize qualitatively the potential swelling behavior and to forewarn the engineer in the planning stage about the problems associated with these soils (Elarabi, 2004). It is difficult to find a characterization and classification of expansive soils in national codes of practice; despite the fact, the expansive soils are widely distributed over almost all the geographical locations in the world. The parameters determined for the identification of expansive soil have been combined in different classification schemes. The classification system used is based on an indirect and direct prediction of swell potential as well as combinations to arrive at a final decision (Gawande et al., 2017). The relationship between the liquid limit and plasticity index (plasticity chart) plays a very significant role in classifying fine-grained engineering soils. (Chen, 1988) introduced a single index method for identifying expansive soils using only the plasticity index shown Table 2.1.

Table 2. 1 Classification of expansive soil based on plasticity index (Chen, 1988)

Swelling Potential	Plasticity Index (%)
Low	0-15
Medium	15-35
High	35-55
Very High	55 and above

According to their activities of soil also classified clays, which are developed by (Kassahun, 2013) by combing Atterberg limits and clay content (percent by weight finer than 2µ m) into a single parameter called activity. Skempton classified clays into three classes according to their activities as indicated in Table 2.2.

Activity is defined as: Activity= PI/ Percent of clay < 0.002mm --------------------- (2.1)

Table 2. 2 Classification of expansive soil based on Skempton method (Kassahun, 2013)

Degree of activity	Activity
Low	<0.7
Normal clay	0.75-1.25
Active Clay	>1.25

Generally, there are several classification methods of soils for engineering purposes, but the Unified Soil Classification System (USCS) and the method of the American Association of State Highway and Transportation (AASHTO) have substantial acceptance. Since the USCS and AASHTO systems are widely used, they used to classify soils of the study area. Soils rated CM or CH by the USCS, and A-6 or A-7 by AASHTO can considered potentially expansive soil (Gawande et al., 2017).

2.8.1 Classification of Expansive Soils by USCS and AASHTO system

2.8.1.1 AASHTO Soil Classification System

AASHTO Soil Classification System classifies soils into seven major groups A-1 to A-7 as shown in Table 2.3.

Table 2. 3: AASHTO soil classification chart (Jamal H., 2019)

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		51 min	…				…	…	…	…
	15 max	25 max		35 max	35 max	35 max	35 max				36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	…		…	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of Significant Constituent Materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey Soils	
General Rating as a Subgrade	Excellent to Good							Fair to poor			

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

From Table 2-3, soils in-group A-6 are typically lean clays, and those in group A-7 are typically highly plastic clays (Jamal H., 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 calculated using the following.

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

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 Flexible Pavements Pavement Design Manual, 2013). 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 H., 2019).

2.8.1.2 Unified Soil Classification System

The USCS originally developed by Casagrande and 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 as shown in Table 2.4 and Figure 2.3 (Asres E., 2017).

Table 2. 4: Swelling potential of Soils as per USCS (Asres E., 2017)

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

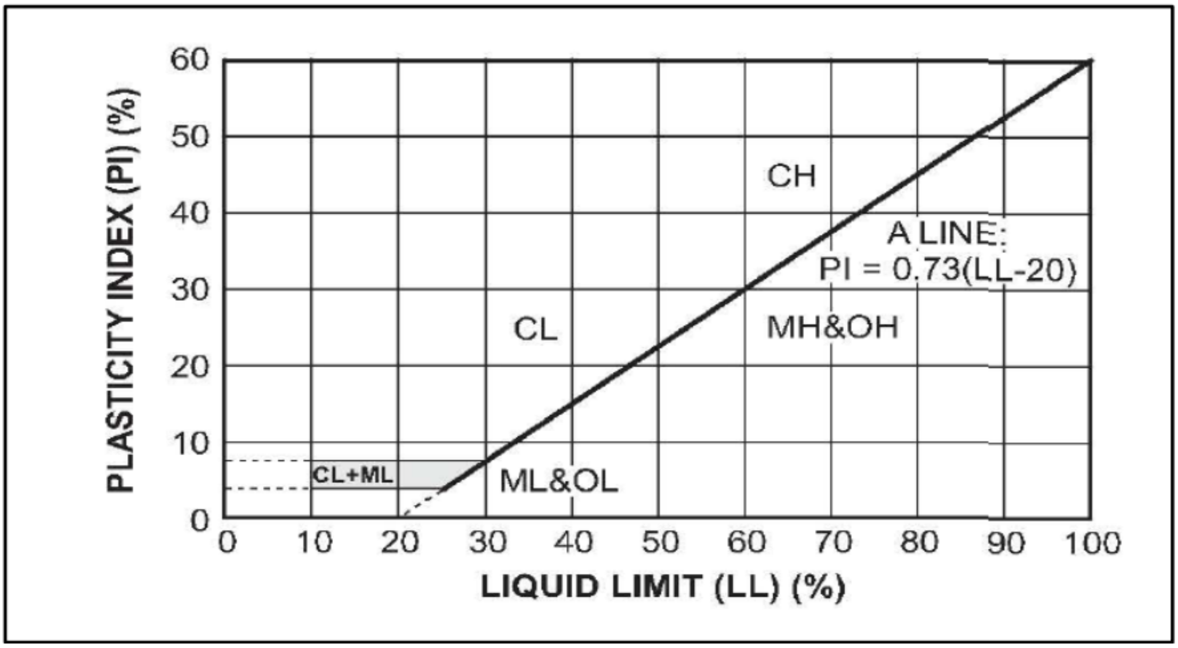


Figure 2. 4: Plasticity Chart

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.

2.9 Problem Associated with Expansive Soil

Expansive soils raise challenges to civil engineers in general and geotechnical engineers in particular (Kassahun, 2013). They cause more damage to infrastructure founded on them, especially light buildings and pavements, than other natural disasters, including earthquakes and flooding, because of their potential to react to changes in moisture regime (Elarabi, 2004). In the world-wide, these problems are well known and reported in many places, especially in the arid and semiarid regions (Elarabi, 2004). Geotechnical engineers did not recognize damages associated with buildings found on expansive soils until the late 1930s. The U.S. Bureau of reclamation made the first recorded observation about soil heaving in 1938 (Kefitawu H., 2021). Since then, several researchers have conducted researches into expansive soils.

2.10 Soil Stabilization

The most often utilized material in civil engineering projects is soil. Nevertheless, the available soil at different places may not be acceptable for the needs of certain building and cause considerable issues for the pavements or structures when erected on it due to its higher compressibility and low bearing capacity. Using stabilizing agents (binder materials) in weak soils to improve their geotechnical characteristics, such as durability, permeability, strength, and compressibility (Makusa, 2012), and make them permanently suitable for construction (Salahudeen & Akiije, 2014) and meet engineering design standards is a very old practice (Gupta J., 2016). Finding a specific, cost-effective approach for stabilizing soil often involves analyzing the properties that characterize that soil, modifying the soil, determining feasibility.

2.10.1 Type of Soil Stabilization

Soil stabilization can be done through a variety of methods (Makusa, 2012).All of these methods fall into two broad groups, namely; mechanical and chemical stabilization (Tefera A. & Leykun, 1999).

2.10.1.1 Mechanical stabilization

Mechanical stabilization is the ancient type of stabilization without any chemical additive. A soil structure 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).

2.10.1.2 Chemical stabilization

The soil considered to chemically stabilizing when organic and inorganic chemical compounds added to it to improve its engineering qualities. To get the desired result, this chemical stabilization technique relies on a chemical interaction between the stabilizer and soil minerals (Asres E., 2017). The process of chemical stabilization involves mixing pozzolanic and cementitious ingredients, such as lime, cement, fly ash, bagasse ash, calcined termite clay powders, agricultural byproducts, and petroleum byproducts, etc., with soils weak in strength to enhance their stability, strength, swelling, permeability, and durability. Chemical stabilization can achieve soil improvement through chemical reactions (Asres E., 2017) listed below.

- **Flocculation and Agglomeration**

As stated by (Asres E., 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 E., 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.

$$Ca^{2+} + 2(OH)^{-} + SiO_2 \text{ (Clay Silica)} \rightarrow CSH \quad \text{---------------} \ (2.2)$$

$$Ca^{2+} + 2(OH)^{-} + Al_2O_3 \text{ (Clay Alumina)} \rightarrow CAH \ \text{---------------} \ (2.3)$$

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.

Some of chemical additives used for soil stabilization are as follows:

2.10.1.2.1 Cement stabilization

Cement stabilization is the most common method of stabilization with additives. Soil cement (mixture of soil and cement) has been used as foundation material in many projects, such as highway pavement, building pad, and slope protection of dams and embankments. Cement stabilization differs from other forms of chemical stabilization in such a way that structural strength is primarily gained from the cementing action rather than from internal friction, cohesion or chemical ion exchange of the materials. Almost all types of soils can be used for cement stabilization except highly organic soils or heavy clay soils (Tefera A. & Leykun, 1999).

2.10.1.2.2 Lime stabilization

Three forms of lime are produced when limestone is broken. These are quick lime (CaO), hydrated lime ($Ca[OH]_2$) and hydrated lime slurry, all of them can be used to treat the soils. Lime has been known to reduce the swelling potential, liquid limit, plasticity index and it improves the workability and compaction of subgrade soils. In addition to being used alone, lime is also used with different admixtures; some of them are lime-fly ash, lime-cement, and lime-bitumen. In general, lime stabilization improves the strength, stiffness and durability of fine-grained soils (Arora, 2003).

2.10.1.2.3 Fly ash stabilization

This method of stabilization is gaining more attention in recent times since it has wide spread availability to be used alone or with other cementitious additives. It is inexpensive and has a long history of use as an engineering material and has successfully employed in geotechnical application. Fly ash can improve the California bearing ratio (CBR) and unconfined compression strength (UCS) while lowering the liquid limit and plasticity index.

2.11 Literature Review on Municipal solid waste to energy by-product Fly ash

Every year, the globe produces more than 20 billion tons of municipal solid waste (MSW) as a result of economic growth and urbanization. In 2050, an estimated 34 billion tons of MSW will be generated (Richardson, 2020). Yet, 33% of them receive harmful treatment, particularly in low- and middle-income nations (Xiao et al., 2020). If MSW is not handled in an environmentally responsible way, it will lead to number of social and environmental issues, including taking up precious urban space, producing hazardous germs, viruses, and other microbes, and contaminating the environment (Yang et al., 2020). What is worse that wind and rain may transport the hazardous compounds to the air and groundwater and even cause a huge impact on the global ecological environment.

2.11.1 Treatment of Incineration Fly Ash

Fly ash is a hazardous substance because it contains heavy metals and soluble salts, thus it must be treated before being disposed of or used. The goal of current treatment methods and technologies is to lessen the danger that fly ash's toxic material poses to the environment. In addition to ensuring ecologically appropriate uses and cost-effective consideration of incinerator fly ash, treatment would also provide an alternative to the currently scarce resources. The fly ash treatment methods that have been developed recently are summarized in this section. The concepts and treatment procedures for incineration ash presented in the section below, Figure 2.4 (Altaf Hussain et al., 2020).

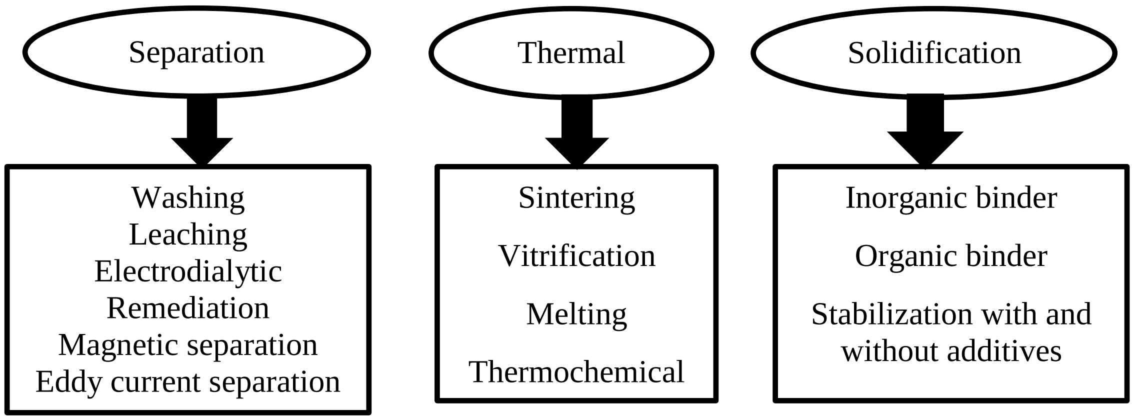


Figure 2. 5 Shows the treatment techniques of incineration ashes (Altaf Hussain et al., 2020)

2.11.1.1 Separation Process

Separation techniques are mostly used to isolate particles from bulk materials that have comparable chemical and physical characteristics (Altaf Hussain et al., 2020). Washing, Leaching, Electrodialytic Remediation, Magnetic separation and Eddy current separation methods categorized in this group.

2.11.1.2 Thermal Treatment

Thermal treatments attempt to stabilize the residue, making it resistant to leaching and producing a residue that may be used. Hence, thermal treatment guarantees those residues are fully detoxified. As opposed to this, alkali chlorides, sulfates, and volatile compounds present during the thermal treatments described in the reference (Mangialardi et al., 1999) to pose a hazard. As a result, these procedures demand a high temperature, which raises the cost but offers the chance to remove contaminants and hazardous elements (Margallo M. et al, 2015). According to the study contribution of (Wang et al., 2010), there is a way to avoid heavy metal volatilization by using additives, temperature, pretreatment, and immobilization in a certain environment. According to the reference (Haiying et al., 2010) ,cooling measures should be taken when thermal treatment is subject to an air control system since this might affect the properties and leaching behavior of residues. Sintering, verification, melting, and thermochemical treatment are the four main thermal treatments.

2.11.1.3 Stabilization/Solidification (S/S)

The methods of stabilization and solidification are frequently used for MSWI wastes. The majority of S/S procedures deal with waste residues whose physical, chemical, and mechanical characteristics encourage lowering the leachability of pollutants from waste (Sabbas et al., 2003). The terms "solidification/stabilization" refer to procedures that use binders or additives to chemically or physically immobilize dangerous materials in trash (Lam et al., 2010). The stabilization process seeks to reduce the toxicity and solubility of pollutants by making them less soluble and poisonous. In contrast, binders like cement typically used for stabilization/solidification to minimize the leachability and immobilize pollutants. By encapsulation, this method lowers the mobility of pollutants in the treated material. In actuality, chemical stabilization followed by waste solidification is the optimum method for solidification/stabilization. Several investigations have demonstrated that cement solidification may remove heavy metals (Altaf Hussain et al., 2020).

2.11.2 Municipal solid waste incinerator fly ash

Large amounts of fly ash and bottom ash are created when MSW exposed for combustion process. Fly ash makes up approximately 2.5% of them, while bottom ash makes up around 7.5%. An estimated 500 million tons of fly ash and more than 1.5 billion tons of bottom ash are generated annually (Diliberto et al., 2020). Addis Abeba is now one of the cities in emerging nations that is urbanizing quickly. Yet, despite the city's growth and the population of the city, the supply of solid waste management services has not raised to the level required. The city produced 851 tonnes of solid waste per day, or 2,297 m^3, toward the end of the 1990s, which was a very low waste generation rate of 0.15 to 0.252 kg per capita/day. The high density ranged from 205 to 370 kg/m^3 throughout the dry and rainy seasons, respectively, with an average of 333 kg/m^3 (Hayal Desta, 2022). One open dumpsite, known as "Reppi" or "Koshe," has been in operation since 1964. The location has been used beyond the scope of its original design and has posed significant risks to public and environmental health. Has 25 hectares of surface area. The property is 13 kilometers from the city center and situated in the Kolfe Keraniyo sub-city in the southwest of the city. More than 9.5 million cubic meters of solid garbage were reportedly disposed of at the landfill up to 2002. (Hayal Desta, 2022) A significant source of revenue for EEPCO will also come from the use of municipal solid waste as feedstock for the Cambridge plant in Rappi (Koshe), with the potential to produce 25 million birr annually. Direct gasification will be used at the Cambridge facility to turn municipal solid waste into electricity. The Cambridge facility's municipal solid waste will be collected, cleaned, and dried before being fed into internal combustion engine or gas turbines to generate power (Fikreyesus, 2022). Fly ash trash makes from 2.5% to 5% of all processed municipal solid garbage. There are typically two types of fly ash, Class F fly ash and Class C fly ash, as specified by ASTM C618.

Class F: This fly ash is pozzolanic and has less than 10% lime in it (CaO). In order for the glassy silica and alumina in Class F fly ash to react and produce cementitious compounds, they need a cementing agent like cement, quicklime, or hydrated lime combined with water. Moreover, a Class F fly ash and a chemical activator like sodium silicate can be used to create a geo-polymer.

Class C: In addition to its pozzolanic properties, class C fly ash has also some self-cementing properties. Class C fly ash hardens and becomes stronger over time when exposed to water. In most cases, Class C fly ash contains more than 20% lime (CaO). Unlike Class F, self-cementing Class C fly ash does not require an activator.

2.12 Previous Work on Soil improvement Using Fly ash

The current study (Younus M., 2010) looks into the possibility of using fly ash, a waste product, in place of sand. The main goal of this study is to use fly ash as much as possible for the mentioned application (landfill). In this study, different criteria for evaluating a material's suitability as a landfill liner have been proposed. These findings indicate that a fly ash-expansive soil mixture with less than or equal to 70% fly ash is suitable for use as a landfill liner. The better result was observed at utilization of 40% fly ash.

In this study (Mohanty et al., 2018) improvement of geotechnical characteristics of expansive soil by mixed with fly ash and lime studied. The ratio of expansive soil to fly ash that produces the best outcomes is determined to be 70% soil and 30% fly ash. The optimal soil-fly ash mixed sample is then given a modest percentage, between 1% and 5%, addition of lime. The combined samples of soil, fly ash, and lime are also put to the test in the laboratory to ascertain their geotechnical characteristics. The soil-fly ash-lime mixed sample sets are submerged in water for 4, 7, 14, 21, 28 and 56 days for soaking CBR. The investigation showed that the expanding soils with 4% lime and 30% fly ash got the highest CBR value.

According to (Sheng-quan et al., 2019) the combination of expansive soil, lime and fly ash that produces the best modifier mixture investigated, the expansive soil's plasticity index lowers by 64.9% when 5% lime is applied, the free swelling ratio drops to around 10%, the unloaded swelling ratio drops to almost 4%, and the stabilized soil no longer demonstrates the expansive feature. Both the unconfined compressive and tensile strengths dramatically rise with the addition of 5% lime. 10% fly ash with 5% lime is the combination that produces the best modifier mixture. According to XRD data, quartz is the primary constituent of the stabilized soil. These are the main reasons behind the increase in strength.

According to experimental findings in this study (Thiha and Swae, 2019), black cotton soil and fly ash mixture with different percentage studies implies that the optimal moisture content learned from the compaction test used to perform the CBR test. After that, CBR experiments are commonly performed using different fly-ash and moisture contents in order to determine the ideal fly-ash and moisture ratio that provides the highest CBR value. According to the test results, the greatest CBR value is produced by 17.8% water content and 16% fly ash. The CBR value of a typical soil sample is around 2.05 (in bad condition), but a fly-ash stabilized mixture has a CBR value of 31.5 (in good condition). Stabilized soil may be utilized as the basis or sub-base in the building of roads.

The clayey soil in this study (Jayanti Munda et al., 2022) was originally treated with five different amounts of fly ash, namely 10, 15, 20, 25, and 30% by total weight of the dry soil, with 20% being the best result. The performance of the stabilized soil was next examined when various concentrations of nano-SiO2 (i.e., 0.5, 1.0, 1.5, 2.0, and 2.5%) were combined with the recommended amount of fly ash. According to the laboratory studies, 20% and 1.5% of fly ash and nano-SiO2 particles respectively were the best dosages. According to the experimental findings, adding nano-SiO2 to soil that has been treated with fly ash dramatically raises its UCS and CBR values.

2.13 Literature Review on wetting and drying cycles

The treated soil samples studied for durability by subjecting them to alternate wetting and drying studies. Wetting and drying studies conducted mainly to simulate the seasonal moisture fluctuations that might occur during different seasons. By putting the modified soil samples to alternate wetting and drying cycle, their durability investigated. Wetting and drying experiments carried out primarily to imitate the seasonal moisture fluctuation that could take place across the seasons. Throughout the durability investigations, the strains under swell and shrinkage conditions and unconfined compressive strengths (UCS) for the soil samples tracked. A wetting-drying cycle consists of submerging in water 48 hours to make it saturate and air drying to the initial moisture content (by the weight of the samples) which determined by weight of sample (Sudhakar and Revanasiddappa, 2004).

2.13.1 Review of Previous Work on Effect of wetting and drying cycles

The collapse behavior of specimens of compacted residual soil from the Bangalore District examined in this research (Sudhakar and Revanasiddappa, 2004) in relation to the effects of alternating wetting and drying cycle. The findings of such a research can be used to predict changes in the way compacted residual soil fills collapse. Reduced water content, a higher void ratio, and perhaps even the development of cementation bonds are all blamed for modifications in the swell/collapse behavior of specimens of compacted residual soil caused by wetting drying processes.

This study (Bao-tian et al., 2015) examines how the engineering parameters of expansive soil enhanced by OTAC-KCl admixtures after a 14-day curing period are affected by wetting and drying cycles. During the wetting and drying experiments, which primarily involve cyclic swelling potential and cyclic strength tests for expansive soil samples from both treated and untreated soil. Additionally, samples of expanded soil modified with 0.3% OTAC + 3% KCl exhibit sufficient durability in terms of swelling capacity, shear strength, and unconfined compressive strength, indicating that the mechanical and physical characteristics of stabilized expansive soil have been effectively enhanced.

According to (Sayem Hossain et al., 2016), due to world-wide distributions of extensively used as construction materials, geotechnical engineers are interested in understanding the mechanical behavior of residual soils which are sometimes referred in the literature as problematic soils. To determine those soil saturated shear strength, several drying-wetting soil specimens are subjected to a series of consolidated drained (CD) triaxial experiments. The impact is more pronounced during the first cycle, diminishes throughout the following 4 cycles. In addition, a mathematical formula that may characterize the saturated shear strength reduction rate of wetting-drying cycle samples is suggested in this article. Studies like this are helpful in understanding potential changes in the behavior of residual soils beneath engineered buildings that experience periodic wetting and drying due to climate fluctuations.

According to this study (Hasriana et al., 2021), the effect of fly ash on expansive soil has been studied. The results show that simply reducing the consistency limit, the fly ash combination changed the way expansive soils expanded and shrink. The addition of 5% to 15% fly ash boosts the soil's compressive strength greatly at the same moisture content. After four cycles, changes in compressive strength and stress-strain modulus were extremely modest or inconsequential. The suggested approach successfully lessens on erosion of expansive soil strength brought on environmental. It therefore makes a substantial contribution to creation of materials that lessen structural damage in expanding soils.

This paper (Wei Qi et al., 2022), report on a phenomenological investigation aimed at exploring the impact of drying–wetting cycles on the shrink–swell behavior of clay loem soil in farmland. The distribution of void ratio and moisture ratio in the four zones significantly changed as a result of the considerable effects of wetting and drying cycles on the soil shrinkage curves. Yet, when number of cycles grew, the influence diminished because of the more stable soil structure. Due to gravity's influence, soil deformation predominated in the beginning of drying, but after entering residual shrinkage, virtually isotropic shrinkage took place.

Table 2. 5 Summary of related works on fly ash as stabilizer

Authors	Title of the study	Fly ash (%)	Major finding
Younus M (2010)	Laboratory investigations to maximize the utility of fly ash for landfill liner	0% - 70% with increasing interval of 10%	Based on these results, it is noted that, with less than or equal to 70 % of fly ash is found appropriate for landfill liner application. Moreover optimum value found on 40% fly ash.
Mohanty et al, (2018)	Stabilization of Expansive Soil	5% to 30% with increase of 5%	The expansive soils mixed with 30% fly ash and 4% lime gained the maximum CBR value.
Sheng-quan Zhou et al, (2019)	Study on Physical-Mechanical Properties and Microstructure of Expansive Soil Stabilized with Fly Ash and Lime	5% to 20% with increase of 5%	After the addition of 10% fly ash and 5% lime, both the UCS and tensile strengths increase significantly..
Thiha and Swae ,(2019)	Study on Stabilization of Expansive Soil with Fly-ash	5% to 30% with increase of 5%	The CBR value of the ordinary soil sample is about 2.05 (poor condition) and that of fly-ash stabilized mixture is 31.5 (good condition).
Jayanti Munda et al, (2022)	Investigation on performance of expansive soil stabilized with fly ash and nano-SiO2	10, 15, 20, 25, and 30%	The experimental results concluded that the use of 1.5% nano-SiO2in the 20% fly ash treated soil enhances the UCS and CBR value significantly.

Table 2. 6 Summary of related works on wetting and drying cycles

Authors	**Title of the study**	**Major finding**
Sudhakar and Revanasiddappa, (2010) (India)	Influence of cyclic wetting drying on collapse behavior of compacted residual soil	Experimental results indicated that the swell/collapse behavior of compacted residual soil specimens from wetting drying effects are attributed to reduction in water content, void ratio and possible growth of cementation bonds.
Bao-tian et al, (2015) (China)	Research on Wetting-Drying Cycles' Effect on the Physical and Mechanical Properties of Expansive Soil Improved by OTAC-KCl	Experimental results show untreated soil specimens will collapse when immersed in water while the treated specimens keep in good conditions on durability of the swelling ability, shear strength, and UCS.
Sayem Hossain et al., (2016) (China)	Effect of Drying-Wetting Cycles on Saturated Shear Strength of Undisturbed Residual Soils	The effect is more significant for the first cycle and decreases with subsequent cycles and finally reaches to a constant state after 4 cycles.
Hasriana et al., (2021) (Indonesia)	Strength Characteristics of Compacted Fly Ash Treated Expansive Soil due to Wetting-Drying Cycles Repetition	According to the findings, the Compressive strength (qu) and stress-strain modulus (Ei) decreased in 1-4 cycles, and after four cycles, changes in compressive strength and stress-strain modulus were very small or insignificant.
Wei Qi et al., (2022) (China)	Experimental Investigation on the Impact of Drying–Wetting Cycles on the Shrink–Swell Behavior of Clay Loam in Farmland	Drying–wetting cycles had a substantial impact on the soil shrinkage curves, causing significant changes in the distribution of void ratio and moisture ratio. However, the impact weakened as the number of cycles increased because the soil structure became more stable.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

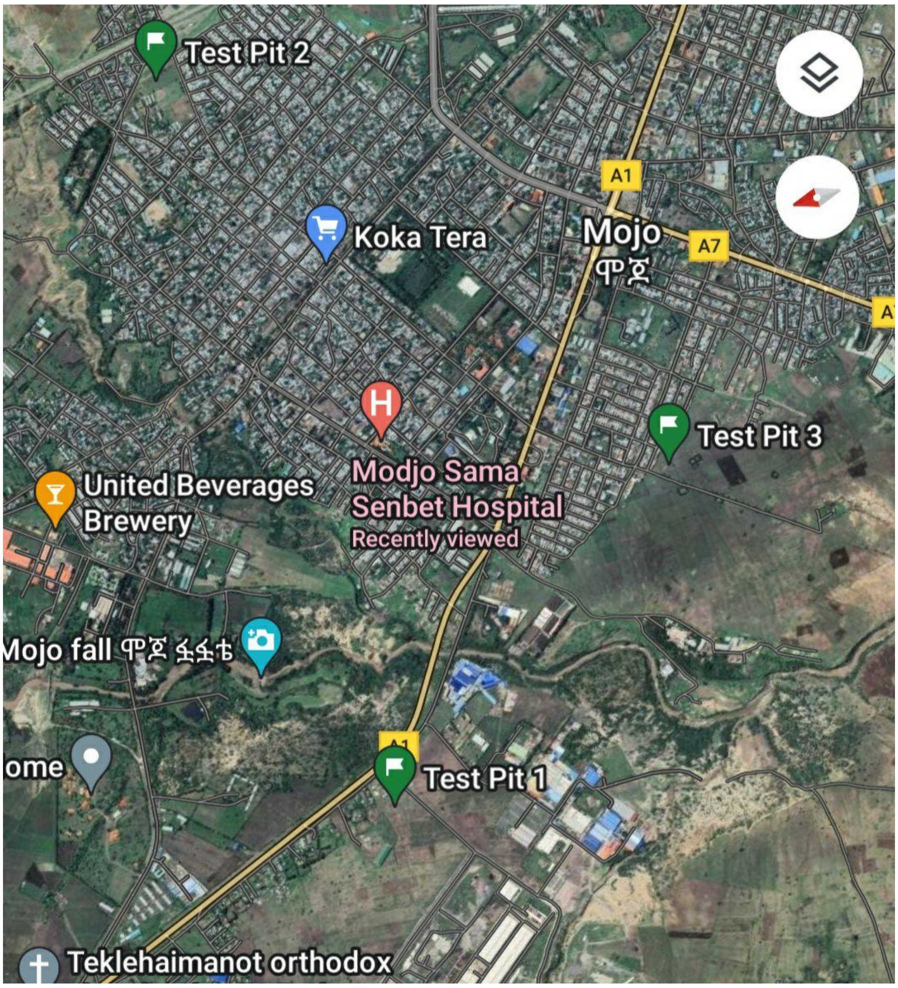
The sample collection and preparation, laboratory experiment techniques, and general method employed in the study all detailed in this section. The experiments carried out mainly in the Gondwana Engineering laboratory and some other laboratories like Adama Science and Technology University and Central Laboratory in Addis Ababa. The sub-sections below go into the specifics of each test that performed on the samples.

3.2 Description of the study area

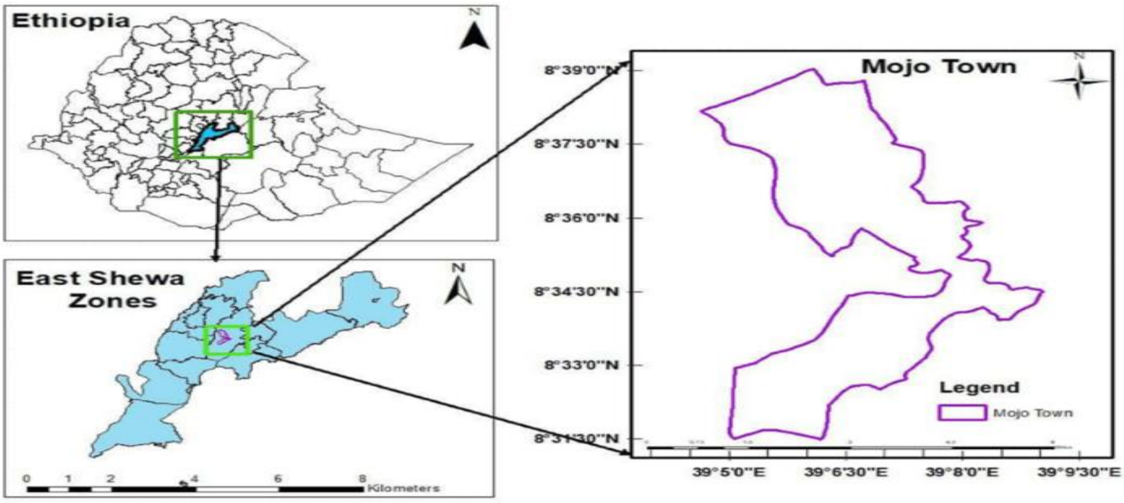
The Gada Special Economic Zone (GSEZ) is a recent development in Ethiopia's overall industrial, commerce, investment, and urban development strategies. The proposed development project spans the astronomical latitudes of 8° 27' 00" to 8° 39' 00" N and the longitudes of 39° 01' 00" to 39° 15' 30" E (which is from Adama to Bishoftu). Gada Special Economic Zone aims to be a pioneer of free trade, export processing, recreational, and smart industrial city by 2063 as the national and regional priority economic strategy to fuel the socio-economic prosperity.

The main Ethiopian rift valley is part of the Gada Special Economic Zone (GSEZ), which is close to the western escarpment. The region is covered in volcanic rock elements of different compositions. The majority of the plain topographic landforms are covered with thick residual clay and silty clay because of the weathering of the rock and subsequent erosion and deposition. Mojo town and a sizable portion of the Gada Special Economic Zone (GSEZ) are affected by this phenomenon.

Mojo is a town in central Ethiopia, named after the nearby Modjo River. Located in the East Shewa Zone of Oromia Region, it has a latitude and longitude of 8°36′N 39°6′E with an elevation between 1788 and 1825 meters above sea level. It is the administrative center of Lome district. The site was at Longitude 39° 06′.22.5″, Latitude 8° 36′03.7′for first pit, Longitude 39° 08′.02.9″, Latitude 8° 36′04.4″ ′ for second pit and Longitude 39° 06′.54.3″, Latitude 8° 35′16.1″ for third pit. Moreover all soils from three pits are black cotton soils with in the 1.5 m depth.



(a)



(b) (Andargie A. et al, 2022)

Figure 3. 1: Map of the Study Area (a) Satellite view of the study area (b) Physical location of study area

3.3 Study Design

This research considered experimental study since it was to measure the index engineering properties of soil and improvement on the soil properties by inclusion different percentage of fly ash (10%, 20%, 30% & 40%) (Younus M., 2010) along with the durability of improved material. Three representative disturbed soil samples at of depth of 1.5 m were collected from different locations of a Mojo town by random sampling technique by using field observation and previous laboratory test results. A laboratory experiment program designed to conduct all of the fundamental laboratory tests to investigate the expansive soil property, the effect of fly ash on properties of expansive soil and wetting drying impact on treated soil.

The disturbed soil samples collected then exposed to air-dried and different laboratory tests conducted according to the American Society for Testing and Materials (ASTM) soil testing procedures.

3.4. Materials

3.4.1. Expansive Soil

The soil was collected from a Mojo town. Three study test pits selected where the problematic soils encountered. The sample soil was extracted at a depth of 1.5 m from each sample pit site, designated as test pit 1, test pit 2 and test pit 3 then after different identification tests were performed in order to select the weaker soil for stabilization work.

Figure 3. 2 Photographic view of expansive sample pit and disturbed soil sample

3.4.2. Fly ash

After producing power and electricity, the Reppie Waste to Energy Plant Technology produces fly ash from incinerated municipal solid waste that burned in Reppie's energy plant categorized as a Class C under ASTM 618 based on its chemical composition as shown in Table 3.2. There are already significant environmental issues with the city's disposal of this ash in the form of particulate matter, which is also going to be a problem in GSEZ in the future. Therefore, this study was done to see utilization impact of this fly ash (class C) from municipal solid waste with different percentage (Younus M., 2010), i.e 10%, 20%, 30% and 40% on expansive soil improvement and durability. Atterbureg limit, Gradation, Hydrometer, specific gravity, standard proctor test and UCS of fly ash were determined in the laboratory.

Fly ash from municipal solid waste that burned in Reppie's energy plant categorized as a Class C under ASTM 618 based on its chemical composition as shown in Table 4.2.

Table 3. 1 Chemical Composition of Fly ash (Simegn et al, 2021)

Constituents	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3	K_2O	Na_2O	MnO	P_2O_5	TiO_2
% Composition	22.54	30.72	14.96	2.82	3.9	8.16	3.62	0.26	2.11	0.04

3.5 Test Programs

Generally, the experimental program was divided in to four parts:

i. In the first part, the properties of the Expansive Soil were identified Atterberg limit, particle size distribution, free swell, specific gravity, standard compaction, CBR and UCS tests conducted.
ii. Based on the test results with different proportion of fly ash (10%, 20%, 30% and 40%) and soil mix, index properties, standard compaction, CBR and UCS tests for better improvement percentage of soil-fly ash was determined.

Soil mix	Percentage (%)	
	Expansive soil	Fly ash
100% Expansive soil and 0% Fly ash	100	0
90% Expansive soil and 10% Fly ash	90	10
80% Expansive soil and 20% Fly ash	80	20
70% Expansive soil and 30% Fly ash	70	30
60% Expansive soil and 40% Fly ash	60	40
0% Expansive soil and 100% Fly ash	0	100

iii. This step involves on the impact of wetting and drying cycles on physical and mechanical properties of treated expansive soil and untreated expansive soil.

iv. Finally, batch-leaching test conducted for MSWI fly ash before and after S/S treatment method and determined the concentration of heavy metals in the leached water by Flame AAS method.

3.6. Methods

A series of laboratory tests including Natural Moisture Content, Particle size analysis, Specific gravity, Atterberg limits, free swell tests, standard proctor compaction, CBR, and UCS test conducted on expansive soil in order to determine weaker soil and soil-fly ash mix in order to determine maximum amount of fly ash for stabilization.

3.6.1 Sample Preparation

After the collection of disturbed soil, moist sample was transported to the laboratory in plastic bags. Sample preparation was done in accordance with the method stated in AASHTO T87- 86. Soil samples were air dried by spreading the material out in trays in the laboratory and leaving it open to the air. Then the soil and fly ash are mixed manually to get uniform mix ratio for each test accordingly to its proposed percentage. The fly ash sample was prepared in different percentage (10%, 20%, 30% and 40%) by dry weight of the soil and then it was mixed in dry state and finally the specified amount of water was added for both compaction and CBR test, no mixing equipment were used since the amount of soil and fly ash used was small.

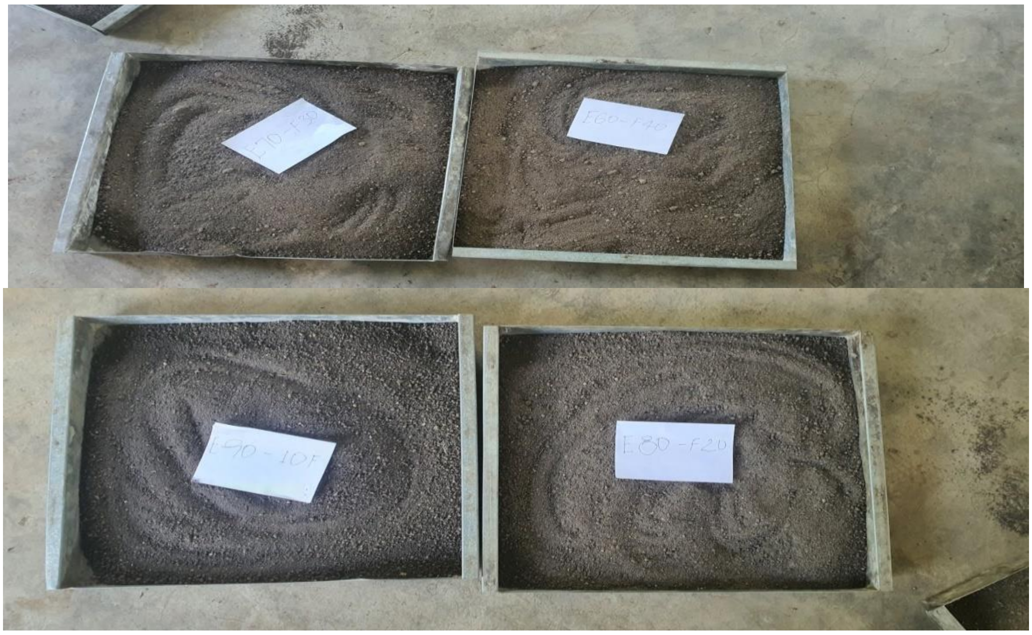


Figure 3. 3 Sample preparation with different percentage of soil-fly ash mix

3.6.2. Natural Moisture Content

The moisture content is an indicator of the amount of water present in soil. The moisture content of the soil samples were determined according to ASTM-D2216, 1998 procedures, while the moisture content (w) was determined in laboratory. The moisture content of a soil mass defined as the ratio of the weight of water (Ww) to the weight of solids (dry weight Ws) of the soil mass.

$$w = \frac{Ww}{Ws} * 100 \text{ -- } (3.1)$$

Drying oven, Balance, Moisture can, and Container Handling Apparatus (gloves, tongs) used to conduct this test.

3.6.3 Specific Gravity

This test method covers the determination of the specific gravity of soil solids that pass through a No. 4 (4.75 mm) sieve ASTM D 854-98. The specific gravity of a soil 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:

$$\text{Mathematically; } G_s = \frac{w_2 - w_1}{(w_4 - w_1) - (w_3 - w_2)} \quad \text{-------------------------------------} \quad (3.2)$$

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, and W_4 = Weight of dry and clean pycnometer + Water, g. To complete this test Pycnometer, Balance, Drying Oven, and Hot plate capable of maintaining a temperature adequate to boil water to remove entrapped air were used.

3.6.4 Particle Size Distribution

Grain size analysis is widely used in engineering classification of soils. It is the quantitative determination of the distribution of particle sizes in soils. The test can be performed in two ways: Mechanical sieving for soils having relatively larger particle generally it used to identify particles with a size greater than 75 μm and Hydrometer analysis performed for soil particle less than 75 μm. The hydrometer test works based on Stokes' low. Grain size analysis was conducted for expansive soils and soil-fly ash mix in order to determine the effect of fly ash on the gradation of the soil according to ASTM-D422.

A hydrometer test carried out with a 40 g/liter sodium hexametaphosphate solution. For samples tested with solution, a 150 milliliter beaker was filled with 50 grams of soil and soil-fly ash mix that had passed sieve number 200 (0.075 mm) with different percentages (0%, 10%, 20%, 30%, 40% and 100%) of fly ash. After thoroughly wetting the soil with 125 milliliters of sodium hexametaphosphate solution (40 g/L) and stirring for 15 minutes, the soil-water slurry transferred to a glass sedimentation cylinder. Distilled water was then added until the volume reached 1000 milliliters, and a hydrometer reading was taken using hydrometer 152H. Ultimately, the outcomes from the two approaches combined to create the overall particle size distribution curve.

To accomplish both mechanical and hydrometer test, Balances, Stirring Apparatus, Hydrometer 152H, Sedimentation Cylinder, Thermometer, Sieves, and Water Bath for maintaining the soil suspension at a constant temperature during the hydrometer analysis used.

3.6.5 Atterberg Limits Test

Following the procedure outlined in ASTM-D4318, this test was conducted to determine the plastic limit (PL), liquid limit (LL), and plasticity index (PI) of the soils and soil-fly ash mix. Both the soil sample and fly ash used for the tests passed through number 40 (425 μm) sieve and fly ash was mixed with a soil according to a specified proportion (0%, 10%, 20%, 30%, 40% and 100%). According to ASTM-D4318, LL is the water content, in percent, of a soil at the arbitrary boundary between the semi-liquid and plastic states. This value was calculated using the Casagrande cup, which involves cutting a groove in the soil sample and raising and lowering the sample cup a predetermined number of times until the groove just about closes. The liquid limit is the soil moisture level at which this occurs. The graph shows the relationship between water content and blow count, and the liquid limit of the soil is defined as the water content that corresponds to the line's intersection with the 25-blow abscissa. The plastic limit found by repeatedly pressing and rolling a soil into a thread with a diameter of 3 mm until the water content was so low that the thread crumbled and could no longer be pressed and re-rolled. The plastic limit is defined as the water content, in percent, of a soil at the boundary between the plastic and semi-solid states. The difference between the liquid limit and the plastic limit is used to calculate the plasticity index (PI) (PI = LL - PL). The LL, PL, and PI variations of native and fly ash mixed soils then investigated. Liquid Limit Device (Casagrande cup), Flat Grooving Tool, Spatula, Ground Glass Plate, Drying Oven, and Moisture Content Can used to conduct these tests in the laboratory.

3.6.6 Free swell tests

A free swell test involves putting a known volume of dry soil that has passed through a No. 40 (425μm) sieve into a graduated cylinder filled with water, and recording the swelling volume of material after the material settles at the bottom of a graduated cylinder, without any surcharge (Tefera A. & Leykun, 1999). The difference between the final volume and the initial volume, expressed as a percentage of the initial volume is a free swell value (Chen, 1988). The free swell of a soil is determined as:

$$\text{Free swell (\%)} = \frac{\text{(final volume) - (initial volume)}}{\text{(Initial volume)}} \quad \text{--------------} \ (3.3)$$

Soils having a free swell value as low as 100% can cause considerable damage to lightly weight structures, while soils having a free swell value below 50% can exhibit appreciable volume change even under very light loadings.

3.6.7 Compaction Test

Compaction is a method for densification of soil by mechanical energy. There are two different proctor tests for compaction: the standard proctor test and the modified proctor test. The standard proctor test in accordance with ASTM D698 was used in this study to compile data. The soil that passed through a No. 4 (4.75 mm) sieve thoroughly mixed with the appropriate amount of water before being compacted in the mold in three layers that were roughly equal in size. Each layer is compacted 25 blows from a 2.54 kg rammer that fell through a height of 304.8 mm. For soil treated with fly ash at its corresponding percentage (10%, 20%, 30%, and 40% by weight of dry soil), compaction was carried out similarly.

3.6.8 California bearing ratio (CBR) test

This test method covers the determination of the California Bearing Ratio of materials from laboratory compacted specimens ASTM D 1883-99. CBR test performed for the natural soil and soil-fly ash mixture based on their mix proportion. The sample preparation for CBR test includes soil alone, soil with varying percentage of fly ash (10%, 20%, 30% and 40%). The test was carried out for samples in water soaked condition for 4 days (96 hours). After 4 days of soaking, the samples taken out from the soaking tank, poured the water off from the top and allowed to drain down ward for 15 minutes.

Calculations

Bearing ratio: the bearing ratio is calculated by dividing the corrected stress values at 2.54mm and 5.08mm to the standard stresses of 6.9MPa and 10.3MPa respectively and multiplying by 100.

$$CBR = \frac{\text{stress}}{\text{standard stress}} * 100 \qquad \text{----------------------------- (3.4)}$$

Figure 3. 4 Photographic view of CBR penetration reading

3.6.9 CBR swell

The CBR swell of the soil or soil-fly ash measured by placing the tripod with the dial indicator on the top marked point of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold was taken just after soaking the sample. At the end of 96 hours (4 days) the final dial reading of the dial indicator was taken and the swell percentage of the initial sample length were given by:

$$\text{CBR swell} = \frac{\text{change in length in mm during soaking}}{116.43\ \text{mm}} * 100 \quad \text{-------- (3.5)}$$

3.6.10 Unconfined compressive strength (UCS) test

UCS test is a special case of unconsolidated undrained triaxial test in which a cylindrical cohesive specimen without any lateral support subjected to axial loading under strain-controlled application. In this study, the UCS carried out according to ASTM D 2166. The sample preparation for UCS test includes soil alone, soil with varying percentage of fly ash (10%, 20%, 30% and 40%) and soil with better improvement observed at fly ash (40%). The test carried out in cylindrical specimen with dimensions of 38mm diameter and 76mm in length. The sample was compacted in a sample tube after mixed with the appropriate amount of moisture. The sample then taken out of the sample tube and checked to have a length to diameter ratio of two and reasonably smooth ends. The sample was then loaded with an axial force and placed in a loading frame. Force applied to the sample and its deformation recorded from the reading of dial gage. The water content of the sample was then calculated and a stress versus strain graph plotted.

Figure 3. 5 Photographic view of UCS specimen preparation

3.7 Wetting and drying cycle procedure

As of finalized the stabilization process and defined the better improvement proportion mix of expansive soil and fly ash, the treated soil samples studied for durability by subjecting them to alternate wetting and drying studies. Wetting and drying studies conducted mainly to simulate the seasonal moisture fluctuations that might occur during different seasons. The strains in swell and shrinkage conditions and unconfined compressive strengths (UCS) monitored for the soil samples during the durability studies. A wetting-drying cycle consists of submerging in water for 48 hours to make saturation and air-dried to the initial moisture content (by the weight of the samples) (Bao-tian et al., 2015). Wetting and drying cycle on cured stabilized soil had applied and finally measurement for physical property (like swelling and shrinking) and mechanical property (like unconfined compressive strength) of the soil specimens monitored. Sample specimens had cured for 7, 14 and 28 days in wet sand after sealed by plastic.

According to (Bao-tian et al., 2015), a wetting-drying cycle entails immersing in water for 48 hours and then air-drying the samples to their initial moisture content (measured by the weight of the samples). By conducting measurement and UCS test studies at the 1st cycle, 2nd cycle, 3rd cycle, and 4th cycle, the effect of additive on the physical and mechanical behavior of treated soils investigated. A control soil tested at the same cycles in contrast (Hasriana et al., 2021).

3.8 Leaching test for municipal solid waste incinerator fly ash

Pollution from industrial solid waste leachate is one of the major environmental issues the world is now dealing with. A problem has a negative impact on society's economy, physical health, and people's daily lives. Current illnesses also connected to the poisoning of land and water resources caused by industrial solid waste leachate.

Fly ash leachate contamination can be a major environmental concern because these wastes are produced in enormous quantities by their respective industrial units, primarily coal-based thermal power plants and iron and steel factories. Poor industrial solid waste management, particularly in developing nations like Ethiopia, has led to serious environmental problems. Because to the leachate's inclusion of different hazardous metals and ions, the uncontrolled dumping of fly ash close to industrial areas in towns and cities has caused a severe environmental issue (Nalawade et al., 2012). Understanding the properties of the rain and surface water that surrounds trash disposal facilities is crucial. The leachate has a significant impact on the nearby water sources (Singh et al., 2014).

The Base Metal Analysis Report revealed the hazardous characteristics of fly ash that were relevant to this phase of study. After the leaching test completed, the concentration of leached heavy metals investigated by using a flame atomic absorption spectrometer after digestion process finalized (Method 3111, 2019). Numerous leaching tests conducted all over the world; for this study, the batch leaching test method used (EPA METHOD 1316, 2012).

3.8.1 Batch leaching tests

Batch leaching tests are straight forward to do since only a solid is combined with the leachant, which can be demineralized water (EPA METHOD 1316, 2012), leachant with a specified chemical make-up (TCLP - acetic acid, $CaCl_2$, or $NaNO_3$) in this investigation, water was chosen as the leachant in this study. The batch tests involve one or more parallel extractions of a substance over the course of 24 hours at pre-set L/S values (EPA 1316: L/S=10, 5, 2, 1 and 0.5 L/kg; EN 12457: L/S=2 and L/S=10) as shown in Table 3.1 and Table 3.2. The suspensions equilibrated for 24 hours by end over end rotation in plastic or glass containers, and then filtered by (0.45 μm) filter membrane. Further assessment by flame atomic absorption spectrometer then conducted to identify toxic elements in filtered water (Method 3111, 2019).

Table 3. 2 : Schedule for leaching test set up (EPA METHOD 1316, 2012)

Test Position	Target L/S	Minimum Dry Mass (g-dry)	Recommended Bottle Size (mL)
T01	10	20	250
T02	5	40	250
T03	2	100	500
T04	1	200	500
T05	0.5	400	1000

Table 3. 3 Parameters as function of maximum particle size (EPA METHOD 1316, 2012)

Particle Size (85% less than) (mm)	US Sieve Size	Minimum Dry Mass (g-dry)	Contact Time (hr)	Recommended Vessel size (mL)
0.3	50	20 ± 0.05	24 ± 2	250
2.0	10	40 ± 0.1	48 ± 2	500
5.0	4	80 ± 0.1	72 ± 2	1000

NOTE: -

- This schedule assumes a target liquid volume of 200 ml.
- This schedule is based on "as tested" solids content of dry weight 20 ± 0.05.

3.9 Flame Atomic Absorption Spectroscopy (FAAS)

Flame atomic absorption is a very common technique for detecting heavy metals and metalloids in environmental samples. It is very reliable and simple to use. With the Flame AA, the sample atomization occurs when a liquid sample drawn into a flame. Atomic absorption spectrometry (AAS) detects elements in either liquid or solid samples through the application of characteristic wavelengths of electromagnetic radiation from a light source after the digestion process finalized (Method 3111, 2019). Individual elements will absorb wavelengths differently, and this absorbance measured and recorded automatically by the help of computer. For this, study the concentrations of heavy metals in the leached water detected by flame atomic absorption spectrometer (Method 3111, 2019).

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Introduction

The analysis part carried out based on the results obtained from the laboratory tests on soil samples and soil - fly ash mix. Results obtained from the experiment program analyzed and interpreted using the statistical description method.

4.2 Properties of Natural soils

The first laboratory tests conducted on soil samples from test Pit 1, Pit 2 and Pit 3 to analyze the behavior of each soil sample and to identify weaker soil.

4.2.1 Natural Moisture Content

The corresponding NMC laboratory results for test pits 1 (TP-1), 2 (TP-2), and 3 (TP-3) are 29.6%, 24.4%, and 26.9%, respectively. The water content of coarse-grained soils, like sand and gravel, can range from a few percent in drier soils to over 20% in saturated soils, according to ASTM D2216. Fine-grained soils like silts and clays have a much wider range of moisture content potential because of the ability of clay minerals to absorb water molecules. The Appendices provide further information on the test results.

4.2.2 Specific Gravity of Soil

The specific gravities of the expansive soil from TP-1, TP-2, and TP-3 are 2.84, 2.73, and 2.78, respectively. The specific gravity of coarse-grained soils, such as sand and gravel, where quartz and feldspar are the predominant minerals, is typically around 2.65, according to ASTM D854. As a result, all soils are fine-grained. However, the soil from test pit one has a higher clay mineralogy than the soil from test pits two and three. The Appendices provide further information on the test results.

4.2.3 The Free Swell Test

On the soils from TP-1, TP-2, and TP-3, the results of the free swell tests are, respectively, 75%, 45%, and 60%. Although soils with free swells over 50% may have swelling issues, those with free swell under 50% are not likely to have expansive properties. Therefore, TP-1 anticipated having a bigger impact on the expansion of structures there. The Appendices provide further information on the test results.

4.2.4 The Atterberg Limit

The corresponding liquid limit and plasticity indices for soils from TP-1, TP-2, and TP-3 are 69.2%, 54.6%, and 62.4%, as well as 33.2%, 22.3%, and 30.5%. The ASTM D4318 specification states that the liquid limit for clays with strong plasticity may reach 100%. The plasticity index for silts is frequently close to zero (a non-plastic soil), but it can reach up to 50% for clays with high plasticity. Because of this, soil from TP-1 is more plastic than soil from TP-2 and TP-3. The Appendices provide further information on the test results.

4.2.5 Grain Size Analysis

The materials finer than 0.075 mm and finer than 0.002 mm have been found to be, respectively, 92.4% and 55% for TP-1, 87.7% and 33.6% for TP-2, and 91.43% and 50% for TP-3, indicating the soils' fine-grained character. This information is shown in the gradation curve (Figure 4.1). The soil from TP-1, contains a higher percentage of clay fractions than the soil from TP-2 and TP-3, which is consistent with the relatively high plasticity and swelling potential.

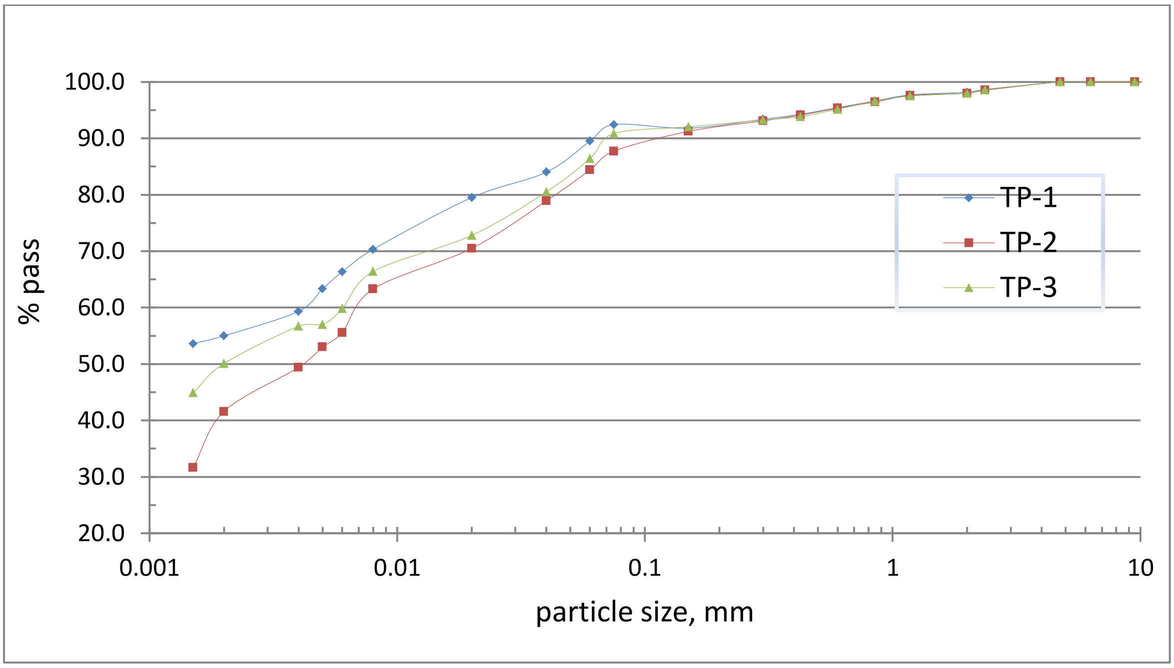


Figure 4. 1 Gradation curve of Expansive soil from TP-1, TP-2 and TP-3.

4.2.6 Soil Classification

The findings indicate that the soil in the study area is very plastic clay. According to the USCS soil classification system, TP-1, TP-3, soil belong to the CH (Inorganic Clays of High Plasticity) and TP-2, soil belongs to the MH (Inorganic Silty of High Plasticity) soil classes. The soil belongs to the A-7 5(42), A-7 5(23), and A-7 5(33) soil classes for TP-1, TP-2, and TP-3, respectively, in accordance with the AASHTO soil classification system. The soils in this classification are typically of poor engineering quality, according to the AASHTO classification scheme. Additional details regarding the test results provided in the appendices.

4.2.7 Compaction Characteristics

The maximum dry densities for TP-1, TP-2, and TP-3 are 1340, 1420, and 1370 kg/m3, respectively, according to the compaction curve. The optimum moisture contents (OMC) of TP-1, TP-2, and TP-3 are 26.9%, 20.1%, and 23.6%, respectively. Results of the tests described in the appendices.

4.2.8 CBR

According to the one point CBR reading and curve for TP-1, TP-2, and TP-3, the soaked CBR value is 2.6%, 6%, and 3.9%, respectively. CBR swell values for TP-1, TP-2 and TP-3 are 9.6%, 2.7% and 6.1% respectively. These findings demonstrate the soil's low bearing capacity behavior under wet conditions and the worst-case scenario discovered on TP-1. The Appendices provide further information on the test findings.

4.2.9 UCS

UCS values of 121.2 kPa, 130.8 kPa, and 124.5 kPa were found for TP-1, TP-2, and TP-3. All test pits' TP-1 soil is the weaker soil because it is weaker than the soil from TP-2 and TP-3. Additional details on the test results provided in the Appendices.

As a result, the soil from test pit one (TP-1) was discovered to be a highly plastic expansive clay with low bearing capacity when it is soaked and high swelling potential, showing that the soil fell below the standard recommendations particularly for highway construction and basic light weight infrastructures. The summaries of natural expansive soil test results shown on Table 4.1.

Table 4. 1 Summary of Natural Soils Properties

Test Type		**Soil Test Results**		
		Pit - 1	Pit - 2	Pit - 3
NMC (%)		29.6	24.4	26.6
Depth of Sampling, (m)		1.5	1.5	1.5
Color		Black	Black	Black
Specific Gravity		2.84	2.73	2.78
Atterberg Limit (%)	Liquid Limit (LL)	69.2	54.6	62.4
	Plastic Limit (PL)	36	32.3	31.9
	Plastic Index (PI)	33.2	22.3	30.5
Free swell, (%)		75	45	60
Percent Passing No. 200 sieve		92.4	87.7	91.4
Percent amount of particle size,(%)	Clay (< 0.002mm) (%)	55	33.6	50
	Silt (0.075-0.002mm)(%)	37.4	54.1	41.4
	Sand (4.75-0.075mm) (%)	7.6	12.3	8.6
	Gravel (> 4.75mm) (%)	0	0	0
Soil Classification	AASHTO Classification	A-7-5 (42)	A-7-5 (23)	A-7-5 (33)
	USCS Classification	CH	MH	CH
Standard Compaction	MDD (g/cm3)	1.34	1.42	1.37
	OMC %	26.9	20.1	23.6
Soaked CBR %		2.6	6	3.9
CBR Swell, (%)		9.6	2.7	6.1
UCS (kPa)		121.2	130.8	124.5

4.3 Property of Fly ash

Fly ash from municipal solid waste that burned in Reppie's energy plant categorized as a Class C under ASTM 618. Some other properties of fly ash are summarize in Table 4.2.

Table 4. 2 Summary of Fly ash Properties

Type of tests		Test Results
Specific Gravity		2.281
Percent amount of particle size,(%)	Gravel (> 4.75mm)	0
	Sand (4.75-0.075mm)	5.7
	Silt (0.075-0.002mm)	58.3
	Clay (< 0.002mm)	36
Atterberg limits, (%)	Liquid Limit (LL)	NP
	Plastic Limit (PL)	NP
	Plasticity Index (PI)	NP
Moisture-Density Relation	MDD, (g/cm^3)	1.22
	OMC, (%)	22.3
UCS (kPa)		47.1

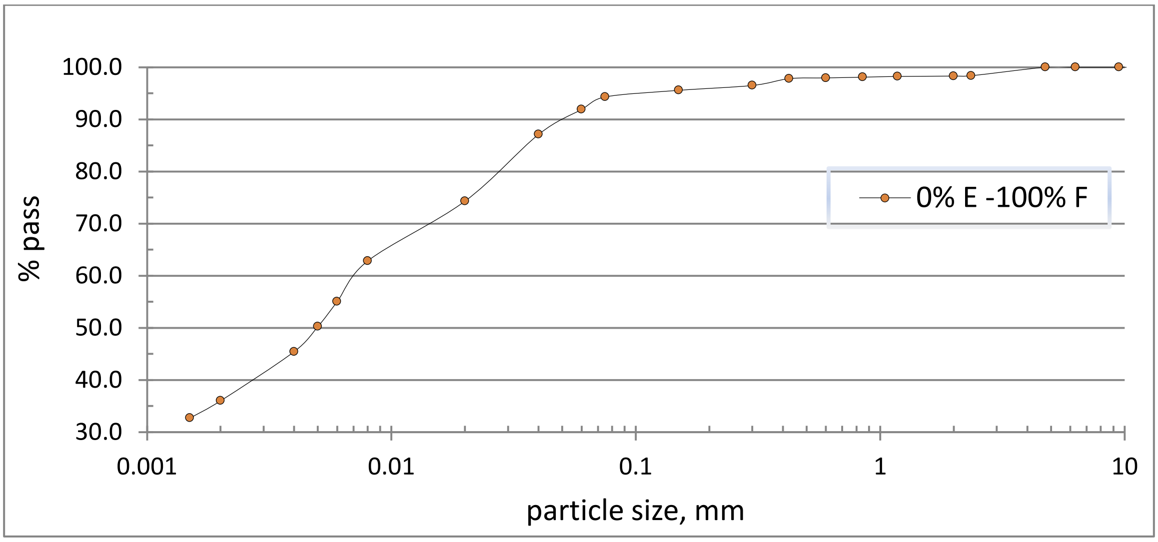


Figure 4. 2 Particle size distribution curve for fly ash

4.3 Result and Discussion on Soil Treated with Fly Ash

The TP-1 soil utilized for additional study on the characteristics of soil-fly ash because it has a lowest UCS value than TP-2 and TP-3.

4.3.1 Effect of fly ash on particle size analysis

The particles finer than 0.075mm are observed to increase with an increase in fly ash content, according to the particle size analysis of different combinations for expansive soil with fly ash. In Table 4.3, a list of particle sizes presented, and Figure 4.3 demonstrates the distribution curves corresponding to these sizes. Table 4.3 demonstrates that the addition of 0% to 40% Fly ash results in a decrease in sand and clay particles and an increase in silt particles. From Figure 4.3 the soil-fly ash particle size distribution curves, it can be seen that as the curves move from right to left, the percentage of particles that are finer than a particular size decreased. This demonstrates that the treated soils' fine particle have become improved by decreasing the clay percentage amount and increasing the silt percentage amount.

Table 4. 3 Particle size distribution of expansive soil with various percentage of fly ash

Expansive soil and Fly ash mix (%)	Particle size (%)			
	Gravel (> 4.75mm)	Sand (4.75-0.075mm)	Silt (0.075 0.002mm)	Clay (< 0.002mm)
100% E -0% F	0	7.6	37.4	55
90% E -10% F	0	7.4	39.9	52.7
80% E -20% F	0	7.2	41.9	50.9
70% E -30% F	0	6.9	43.7	49.4
60% E -40% F	0	6.7	46.2	47.1
0% E -100% F	0	5.7	58.3	36

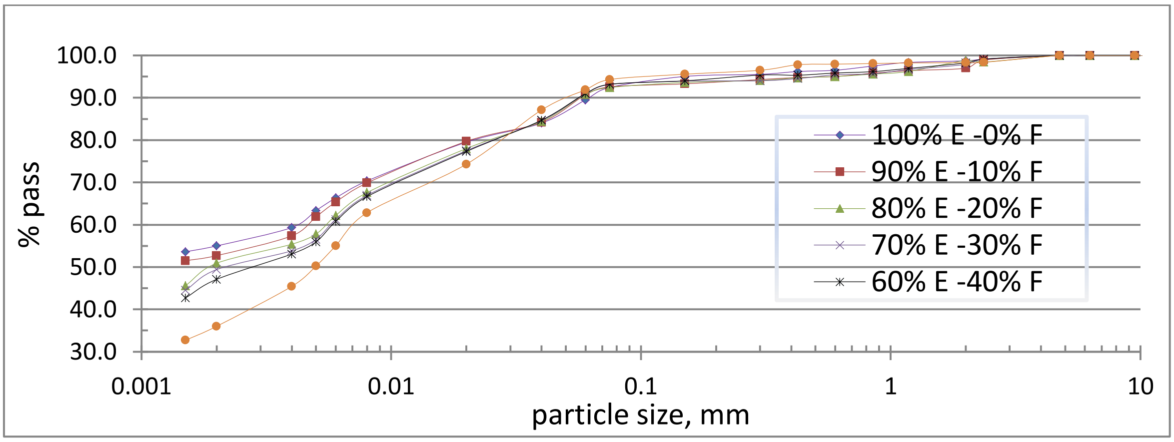


Figure 4. 3 Particle size distribution curve of soil with various percentage of fly ash

4.3.2 Effect of fly ash on specific gravity

Table 4.4 and Figure 4.4 illustrates how fly ash affects the soil's specific gravity.

Table 4. 4 Specific gravity of soil with various percentage of fly ash

Expansive soil and Fly ash mix (%)	Specific gravity (Gs)
100% E - 0% F	2.841
90% E - 10% F	2.785
80% E - 20% F	2.729
70% E - 30% F	2.673
60% E - 40% F	2.617
E 0% - 100% F	2.281

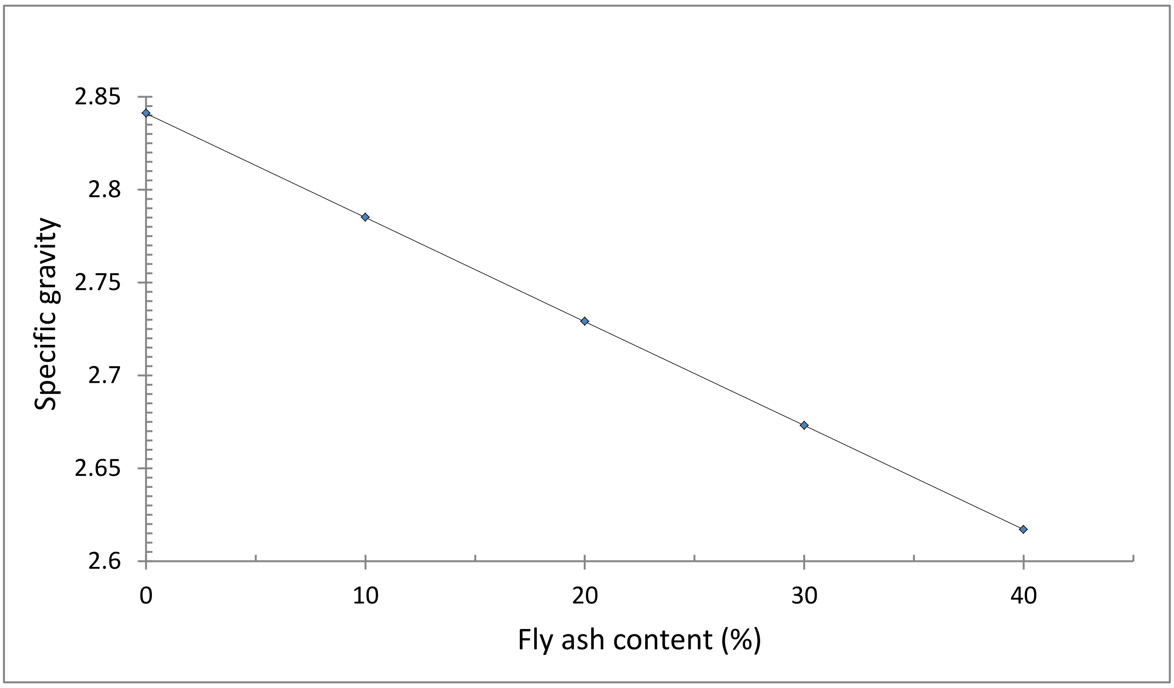


Figure 4. 4 Variation of liquid and plastic limit of soil with fly ash content

With an increase in fly ash concentration from 0% to 40%, the specific gravity of the soil fell from 2.841 to 2.617. For instance, a soil with 40% fly ash component has a specific gravity that is approximately 7.88% lower than a soil alone. Due to the partial replacement of soil with fly ash that has a lower specific gravity than soil (i.e., 2.281), the specific gravity of the soil-fly ash mixture has decreased. The Appendix contains more information on the individual gravity test findings.

4.3.3 Effect of fly ash on Atterberg limit

The effect of fly ash on Atterberg limit of expansive soil summarized in Table 4.5 and shown in Figure 4.5. As shown in figure, liquid limit and plastic limit decreased with the increment in fly ash content. However, the addition of fly ash decreased the plasticity index of soil. The higher reduction in plasticity index was obtained at 40% Fly ash content which is 51.81% lower than that of natural soil.

Table 4. 5 Summary of effect of fly ash on liquid and plastic limit

Expansive soil and Fly ash mix (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Decrease in PI (%)
100% E - 0% F	69.2	36	33.2	0.00
90% E - 10% F	66.3	35.9	30.4	8.43
80% E - 20% F	62.9	35.6	27.3	17.77
70% E - 30% F	57.5	35.3	22.2	33.13
60% E - 40% F	51.1	35.1	16	51.81
E 0% - 100% F	NP	NP	NP	

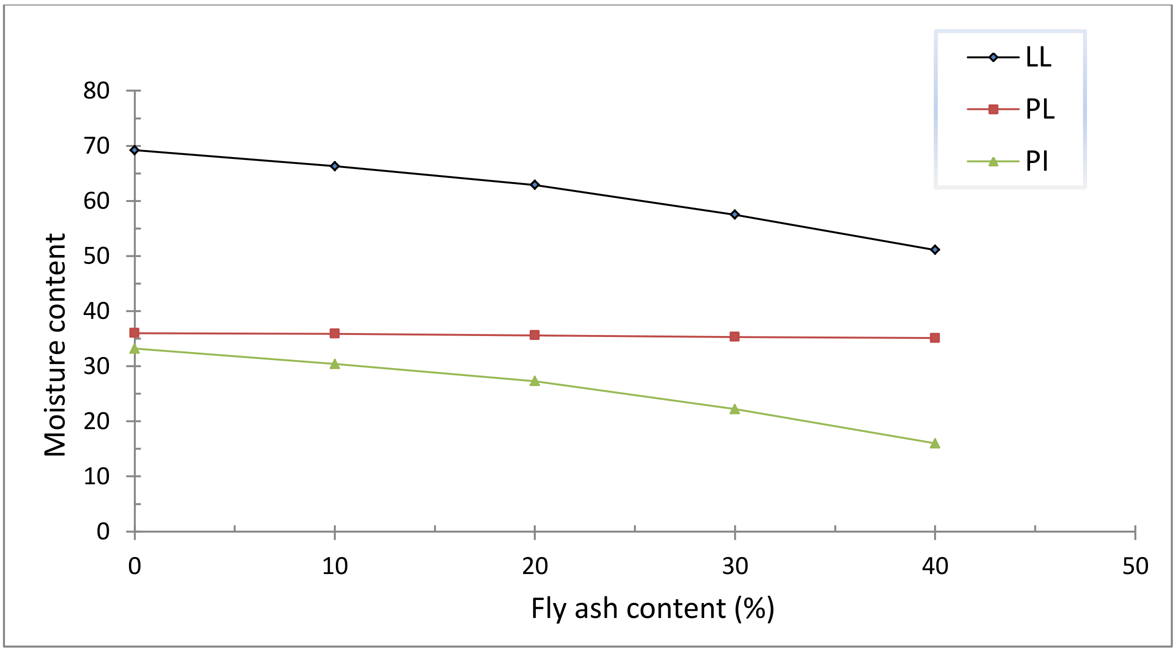


Figure 4. 5 Variation of liquid and plastic limit of soil with fly ash content

The partial replacement of plastic soil particles with non-plastic fly ash materials along with a potential reduction in the clay fraction (which is responsible for plasticity of the soil) are what caused the decline in the plasticity index. The cation exchange and flocculation of clay particles cause the change in plasticity. The foundation of the Casagrande plasticity chart is the plasticity index, which is crucial for classifying fine-grained soils. The greater the plasticity index, the more difficult it will be to use soil as an engineering material. The Appendix contains specifics regarding the results of the Atterberg limit test.

4.3.4 Effect of fly ash on soil classification

The plasticity chart for USCS classification shown in Figure 4.6 and Table 4.6 provides a summary of soil classification. The soil treated with 0%, 10%, 20%, 30%, and 40% of fly ash grouped under the category of A-7-5 with their group indices of 42, 38, 33, 27, and 20, respectively. According to the AASHTO classification system, 100% fly ash classified under the group category that classified under A-4.

According to the USCS classification system, soil treated with 0%, 10% and 20% fly ash falls under the category of CH (Inorganic clays of high plasticity). However, soil containing between 30% and 40% of fly ash falls into the MH (Inorganic Silt with High Plasticity) group while soil containing 100% of fly ash falls into the ML (Inorganic Silt with Low Plasticity) group.

Table 4. 6 Summary of effect of fly ash on soil classification

Expansive soil and Fly ash mix (%)	Liquid limit (%)	Plasticity index (%)	Gravel (> 4.75mm)	Sand (4.75-0.075mm)	Silt (0.075 0.002mm)	Clay (< 0.002 mm)	USCS	AASHTO (%)
100% E - 0% F	69.2	38.8	0	7.6	37.4	55	CH	A-7-5 (42)
90% E - 10% F	66.3	34.1	0	7.4	39.9	52.7	CH	A-7-5 (38)
80% E - 20% F	62.9	29.1	0	7.2	41.9	50.9	CH	A-7-5 (33)
70% E - 30% F	57.5	22.6	0	6.9	43.7	49.4	MH	A-7-5 (27)
60% E - 40% F	51.1	16	0	6.7	46.2	47.1	MH	A-7-5 (20)
E 0% - 100% F	0	0	0	5.7	58.3	36	ML	A-4

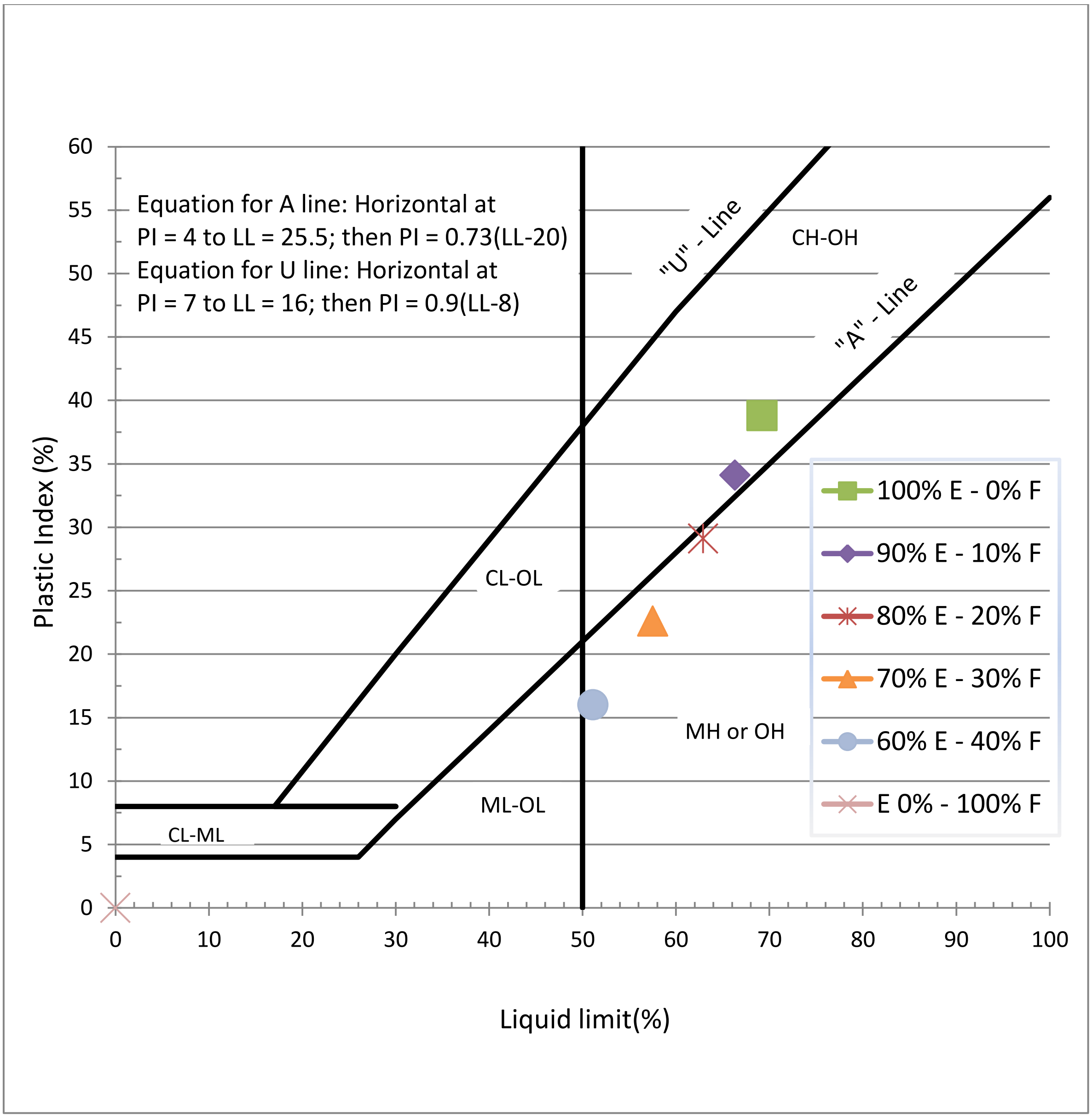


Figure 4. 6 Plasticity chart of soil with various percentage of fly ash

In general, the addition of 0% to 40% fly ash content leads to the classification of soil changed from inorganic clays of high plasticity to inorganic silts.

4.3.5 Effect of Fly ash on free swell

Figure 4.7 illustrates the impact of fly ash on the free expansion of expansive soil. As depicted in the figure, adding fly ash to expansive soil reduces the free swell. The amount of fly ash directly correlates with the reduction in free swell. The soil treated with 40% fly ash, which is 50.67% less than untreated soil produced the greatest reduction in free swell.

Table 4. 7 Effect of fly ash on swelling characteristics of soil

Expansive soil and Fly ash mix (%)	Free swell (%)	Decreased (%)
100% E - 0% F	75	0.00
90% E - 10% F	62	17.33
80% E - 20% F	50	33.33
70% E - 30% F	43	42.67
60% E - 40% F	37	50.67

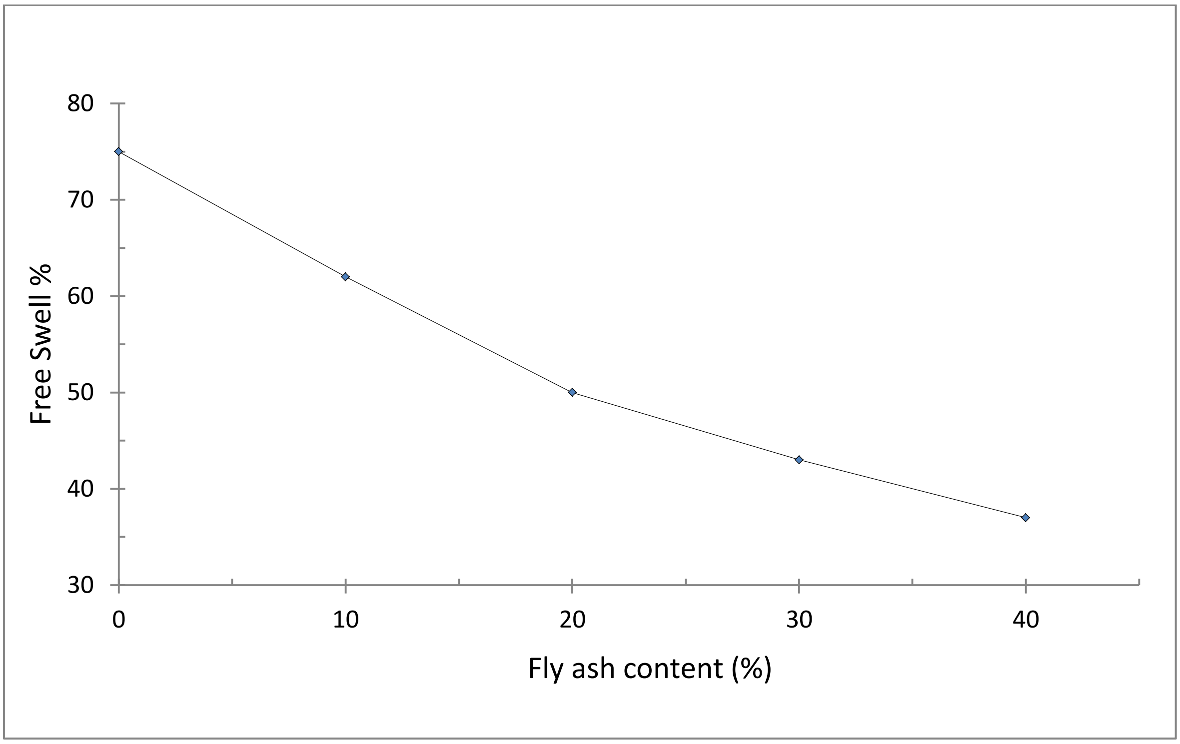


Figure 4. 7 Variation of free swell for different percent of fly ash

4.3.6 Effect of fly ash on compaction characteristics

According to Table 4.8 and Figure 4.8, the maximum dry density of soil samples decreases from 1.34 g/cm3 to 1.30 g/cm3 as fly ash content increases from 0% to 40%. It depicts an addition of 40%; Fly ash decreased the maximum dry density to that of untreated soil by 2.99%. So, the partial replacement of relatively heavy soils with the lighter fly ash and/or the addition of fly ash with a low specific gravity in place of soil with a relatively high specific gravity are the main causes of the decrease in the maximum dry density.

Table 4. 8 Comparison of compaction test results of soil with different fly ash percentage

Expansive soil and Fly ash mix (%)	MDD (g/cm3)	OMC (%)	% decrease in OMC	% decrease in MDD
100% E - 0% F	1.34	26.9	0.00	0.00
90% E - 10% F	1.33	25.8	4.09	0.75
80% E - 20% F	1.32	25.2	6.32	1.49
70% E - 30% F	1.31	24.6	8.55	2.24
60% E - 40% F	1.3	23.8	11.52	2.99
0% E - 100% F	1.22	22.3	17.10	8.96

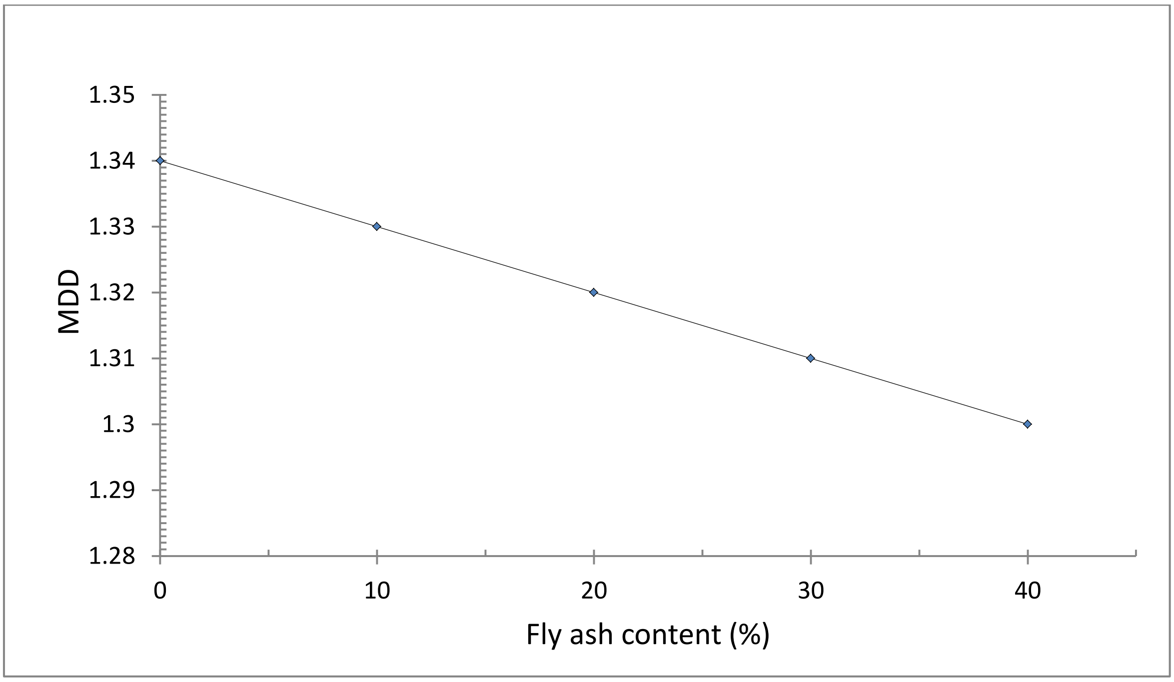


Figure 4. 8 Effect of fly ash on MDD of expansive soil

The optimum moisture content of the soil samples decreased from 26.9% to 23.8% with increases in fly ash content of 0% to 40%, and became 22.3 for 100% fly ash, as it can see in Table 4.9 and Figure 4.9. It demonstrates that adding 40% fly ash to soil causes an 11.2% reduction in the soil's ideal moisture content.

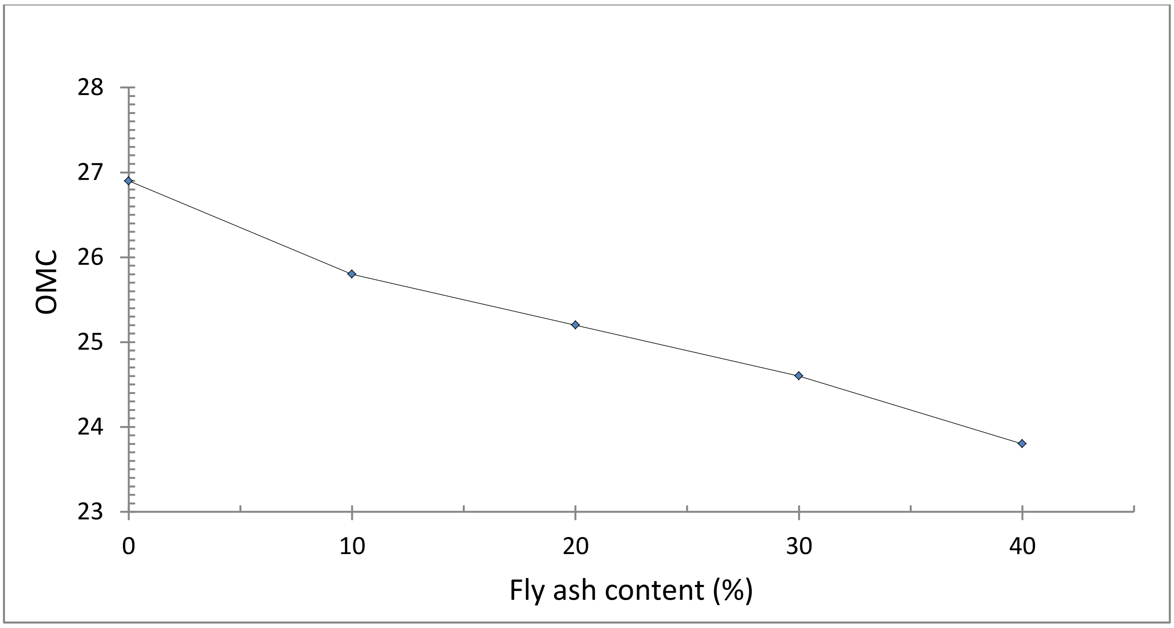


Figure 4. 9 Effect of fly ash on OMC of expansive soil

According to the summary of the compaction curve in Figure 4.10, the compaction curves of treated soil shifted to the left relative to untreated soil as the fly ash content increased. This demonstrates that adding fly ash to soil reduced its maximum dry density and its optimum moisture content. Results of the compaction test detail shown in the appendix.

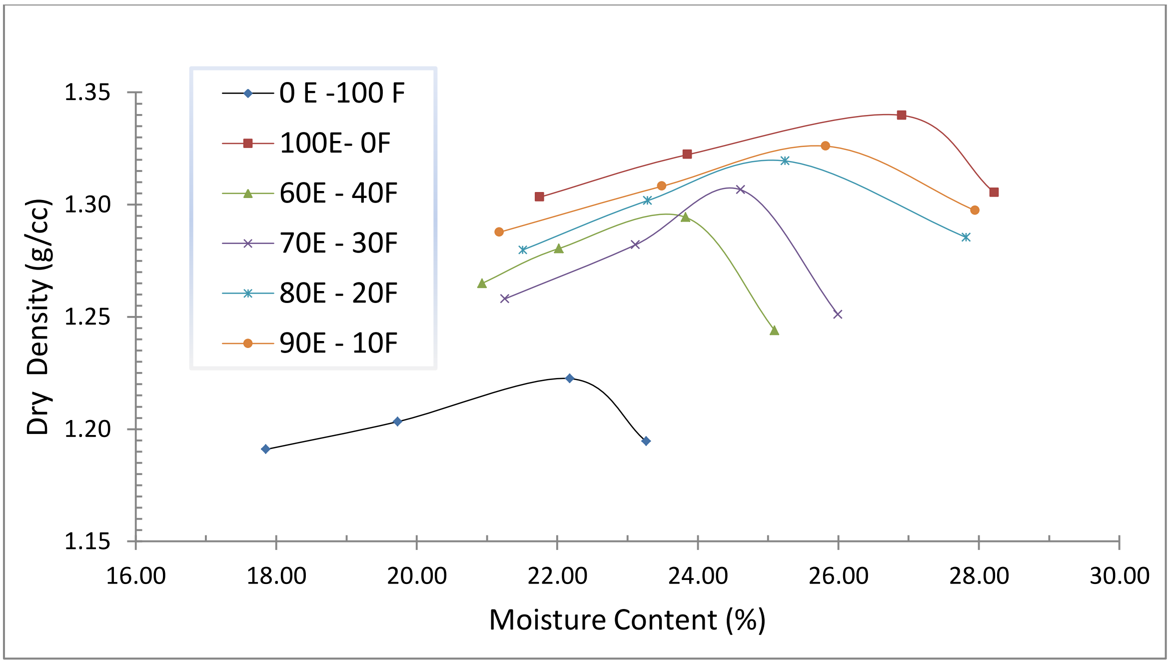


Figure 4. 10 Summary of compaction curves for soil with various percentage of fly ash

4.3.7 Effect of fly ash on CBR and CBR-swell

i) CBR values

The improvement in the CBR test results further demonstrates the potential effectiveness of the fly ash in addition to the aforementioned parameters. The gradual synthesis of cementitious compounds between fly ash and soil, as well as possible pozzolanic reactions between fly ash and soil, are what cause the CBR to rise. In the current study, a one-point CBR test carried out on natural soil mixed with fly ash ranging from 0% to 40% with a 10% interval (ASTM D 1883-99) at a strain rate of 1.27 mm/min. When blended with 40% fly ash, the soil's strength significantly increased in comparison to other proportion soils.

Table 4. 9 CBR test results of soil with different fly ash percentage

Expansive soil and Fly ash mix (%)	CBR (%)	% increase in CBR
100% E - 0% F	2.6	0.00
90% E - 10% F	2.9	11.54
80% E - 20% F	3.4	30.77
70% E - 30% F	3.9	50.00
60% E - 40% F	4.3	65.38

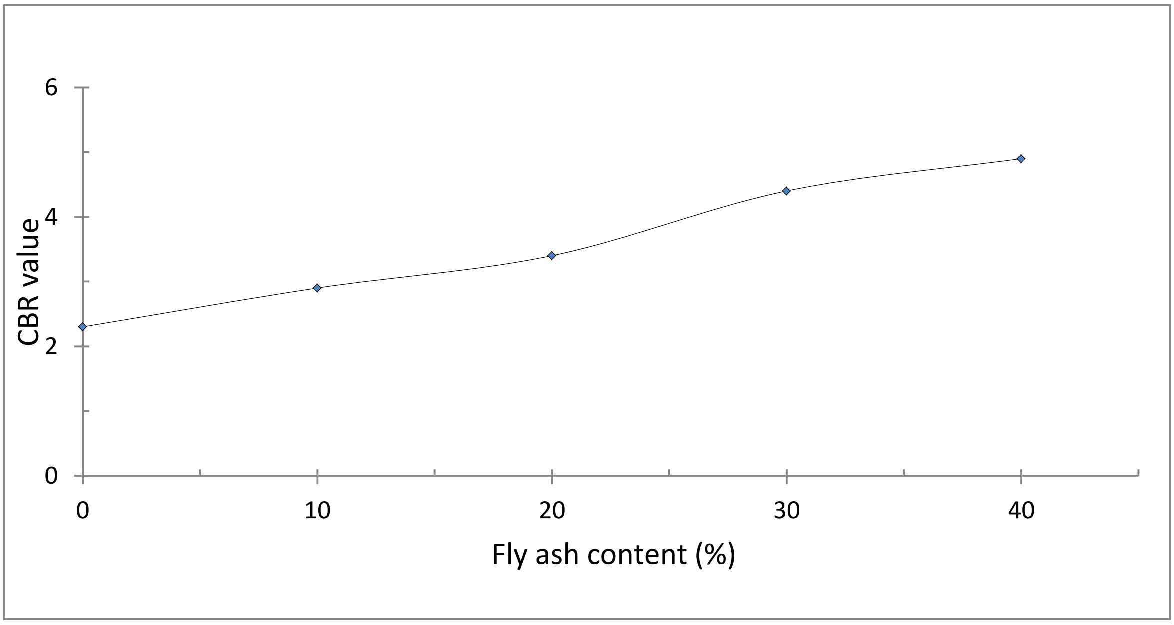


Figure 4. 11 Effect of fly ash on CBR value of expansive soil

The soil-fly ash sample provides a peak CBR value of 4.3% from a value of 2.6% for natural soil when the soil is treated at 40% fly ash content. The maximum CBR value of fly ash-treated soil is 65.38% higher than that of untreated soil. This demonstrates that the fly ash treatment increased the soil's ability to support loads. According to findings from (Thiha and Swae, 2019) study, CBR value increase as the fly ash percentage increase as discussed for current study. Results of the CBR test detail shown in the Appendix.

ii) CBR-swell

Table 4.10 and Figure 4.12 demonstrate the impact of fly ash on the soil mixture's CBR-swell values. The figure clearly shows that the soil mixture's CBR-swell value decreases as fly ash content added.

Table 4. 10 CBR swell results of soil with different fly ash percentage

Expansive soil and Fly ash mix (%)	CBR swell (%)	% decrease in CBR swell
100% E - 0% F	9.6	0.00
90% E - 10% F	7.5	21.88
80% E - 20% F	5.1	46.88
70% E - 30% F	3.9	59.38
60% E - 40% F	3.2	66.67

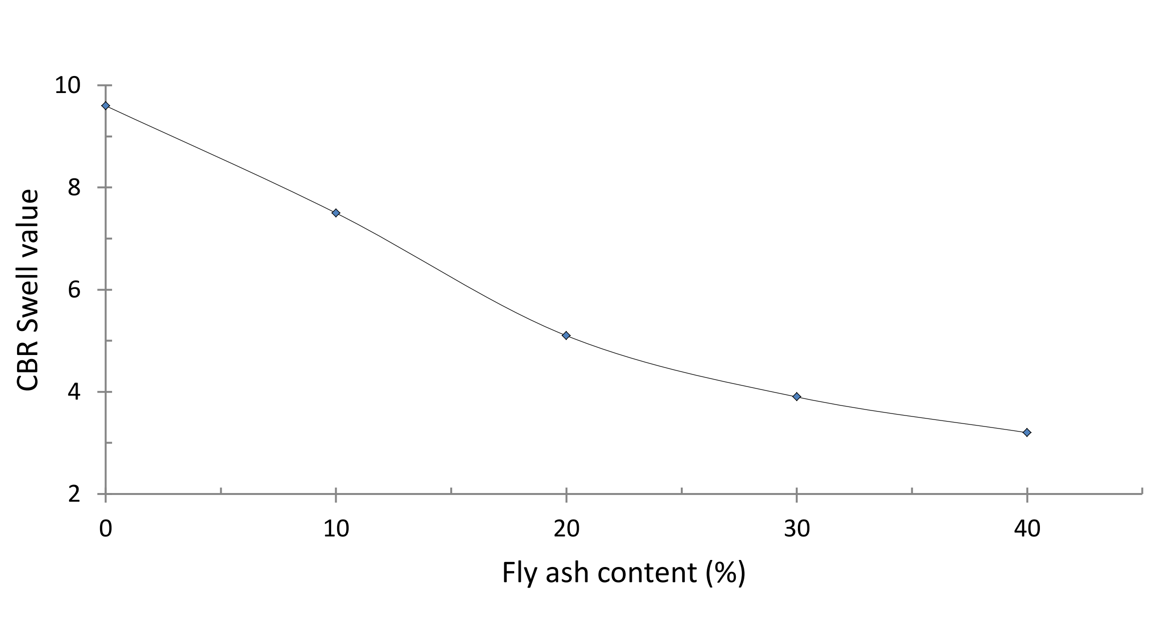


Figure 4. 12 Effect of fly ash on CBR swell of expansive soil

Treating the soil at 40% fly ash content gives CBR-swell value of 3.2%. Thus, the CBR swell value of soil treated with fly ash is 66.67% lower than that of untreated soil and this shows the swelling potential of soil sample decrease with addition of fly ash.

4.3.8 Effect of fly ash on Unconfined Compressive Strength (UCS)

In this test, an axial load without a confining pressure applied to a soil sample. The addition of fly ash increases the UCS value of the soil mixture, as shown in Table 4.11 and Figure 4.13. At 40% fly ash content, the maximum value of UCS was observed to be 257.2 kpa, which is 2.12 times (112.21%) higher than that of untreated soil. In the case of 10% fly ash content, the lower value of UCS is 137.6 kpa, which is 1.14 times (13.53%) greater than that of untreated soil.

Table 4. 11 UCS test results of soil with different fly ash percentage

Expansive soil and Fly ash mix (%)	UCS (kpa)	Cu (kpa)	% increase in UCS value
100% E - 0% F	121.2	60.6	0.00
90% E - 10% F	137.6	68.8	13.53
80% E - 20% F	190.0	95.0	56.77
70% E - 30% F	239.3	119.65	97.44
60% E - 40% F	257.2	128.6	112.21
0% E - 100% F	47.1	23.55	-61.14

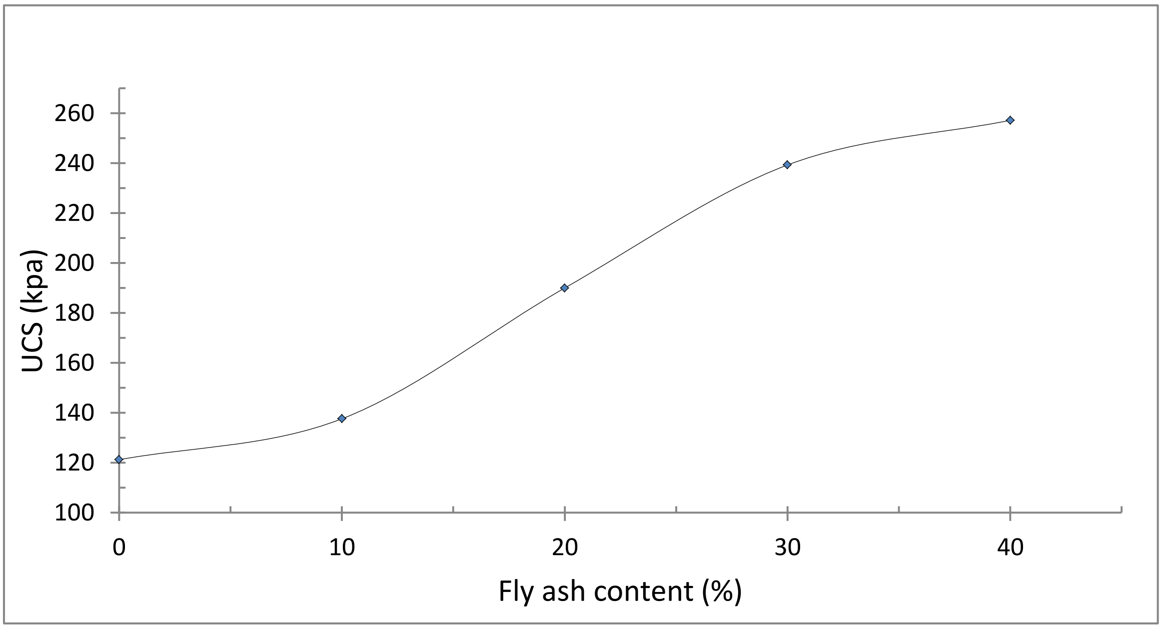


Figure 4. 13 Variation of UCS for different percent of fly ash

The adhesion between the soil sample and the fly ash, which increases resistance to axial applied force, is what causes the UCS to rise. Figure 4.14 displays the stress-strain curves from UCS tests for various fly ash percentages. Results of the UCS test detail shown in the appendix.

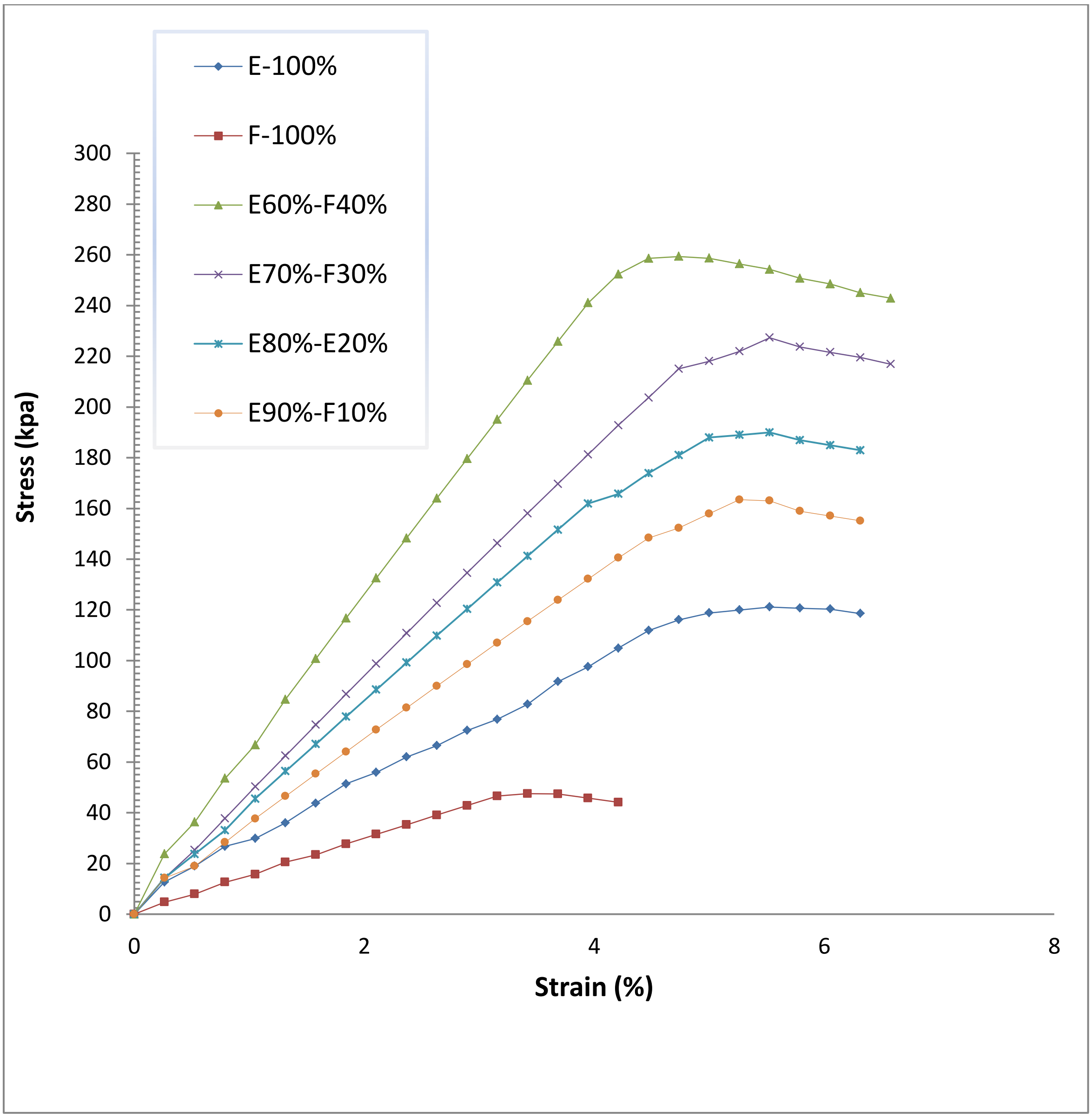


Figure 4. 14 UCS stress-strain curve of soil with different percentage of fly ash

4.3.9 Grand summary of test results for soil treated with fly ash

In here, summary of all test results performed on soil-fly ash presented.

Table 4. 12 Summary of all test results performed on soil with different percent of fly ash

Type of Tests		% of Fly ash Mixed with Expansive Soil					
		0	10	20	30	40	100
Percent amount of particle size, (%)	Gravel (> 4.75mm)	0	0	0	0	0	0
	Sand (4.75-0.075mm)	7.6	7.4	7.2	6.9	6.7	5.7
	Silt (0.075 – 0.002 mm)	37.4	39.9	41.9	43.7	46.2	58.3
	Clay (< 0.002mm)	55	52.7	50.9	49.4	47.1	36
Specific Gravity		2.841	2.785	2.729	2.673	2.617	2.281
Atterberg Limits, (%)	Liquid Limit (LL)	69.2	66.3	62.9	57.5	51.1	NP
	Plastic Limit (PL)	36	35.9	35.6	35.3	35.1	NP
	Plasticity Index (PI)	33.2	32.1	27.3	22.	16	NP
Soil Classification	AASHTO	A-7-5 (42)	A-7-5 (38)	A-7-5 (33)	A-7-5 (27)	A-7-5 (20)	A-4
	USCS	CH	CH	CH	MH	MH	ML
Free swell, (%)		75	62	50	43	37	-
Moisture Density Relation	MDD, (g/cm3)	1.34	1.33	1.32	1.31	1.3	1.22
	OMC, (%)	26.9	25.8	25.2	24.6	23.8	22.3
Soaked CBR, (%)		2.6	2.9	3.4	3.9	4.3	-
CBR Swell, (%)		9.6	7.5	5.1	3.9	3.2	-
Unconfined compression test	UCS, (kPa)	121.2	137.6	190	239.3	257.2	47.1
	Cu, (kPa)	60.6	68.8	95	119.7	128.6	23.55

4.4 Effect of wetting and drying on treated and untreated soil

Prior to beginning the wetting and drying process, the better improvement fly ash content to soil mix must be determined and chosen. Following that, fly ash hardens and gains strength because of pozzolanic reactions that take place over time as a result of curing determined. Therefore, one of a key element influencing the UCS of a fly ash expansive soil mix found the curing period. Both untreated and treated soil specimens are compacted into cylindrical soil samplers with 38 mm diameter and 76 mm height with optimum moisture content. All specimens for treated soils prepared at the optimum moisture content 23.8% and dry density 1.3 g/cm^3. For untreated soil, the specimen prepared optimum moisture content 26.9% and dry density 1.34 g/cm^3.Then each treated sample specimens sealed in plastic bags divided into 3 groups and cured in wet sand for 7 days, 14 days, and 28 days, respectively. As shown below on Table 4.13 and Figure 4.15 the UCS value increase by 51.48% after 28 days curing time.

Table 4. 13 UCS test results of soil with different curing date

60 % Expansive soil and 40 % Fly ash mix Curing period	UCS (kPa)	Cu (kPa)	% increase in UCS value
0 day	257.2	128.6	0.00
7 day	335.5	167.75	30.44
14 day	363.4	181.7	41.29
28 day	389.6	194.8	51.48

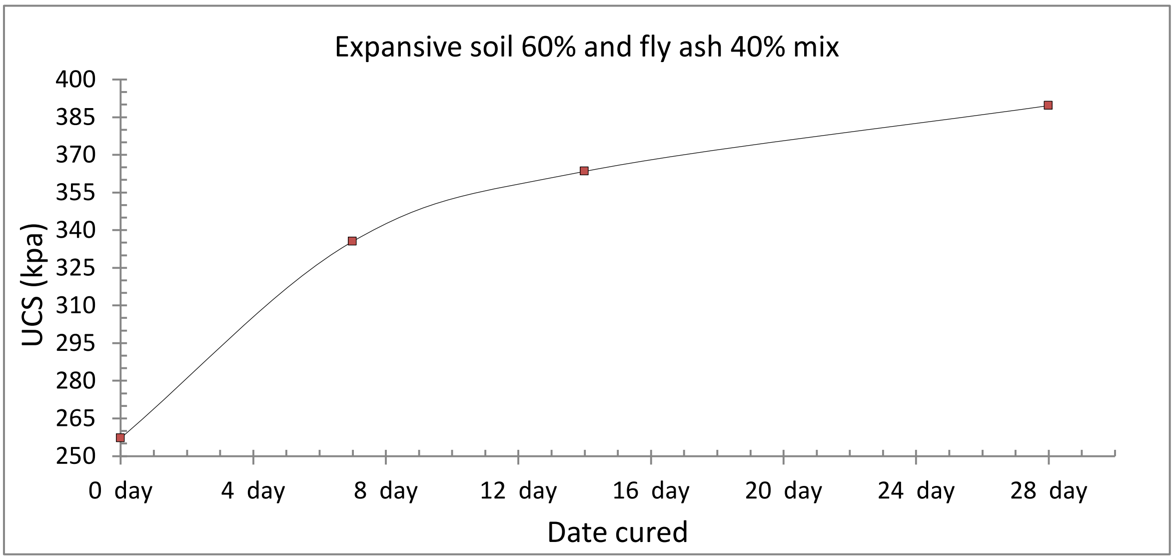


Figure 4. 15 Curing effect for 60% Expansive soil and 40% fly ash on UCS value

The ability to repeat the test on identical samples is generally the wetting-drying cycle test's greatest advantage. Two methods used to examine the stability of stabilized soil during wetting and drying cycles. The first one accounts for swelling stability, and the second one calculates the reduction in unconfined compression strength.

4.4.1 Cyclic Swelling Potential Test

By combining the expansive soil with 40% fly ash, improved specimens are created. Cylindrical ring mold creates soil specimens with a diameter of 50 mm and a height of 20 mm (Bao-tian et al., 2015) with given moisture content and dry density. The molded samples, which sealed in plastic bags, cured for 28 days in wet sand. After curing, the samples go through 4 cycles of wetting and drying. A wetting-drying cycle involves immersing in water until saturation and then air-drying to the samples' initial moisture content of 15.2% (by weight). A vacuum saturation apparatus used to saturate the specimens that are prepared for swelling potential test (Bao-tian et al., 2015). Samples placed inside the desiccator and submerged in water for 48 hours, this phase known as cycle zero. Then, the wet samples removed and allowed to air dry until the initial moisture content is 15.2% or less. A full cycle has so far been finished. The specimens' initial height, designated h_0 at the start of the swell-shrink cycle, is then manually measured with a vernier caliper. The heights of the air-dried samples, designated h_i, and the saturated specimens, designated h_w, are then all measured. The absolute swelling ratio and relative swelling ratio defined quantitatively analyze the regularity and reversibility of swell-shrink characteristics during cyclic wetting-drying process (Bao-tian et al., 2015). The following is the definition of the absolute swelling ratio (S_a): Where h_0 is the starting height and h_w is the height after expansion, S_a is equal to $((h_w-h_o)/ h_o) *100$. The relative swelling ratio (S_r) is defined as follows: S_r is calculated as ((hw- hi)/hi) *100, where hw is height after expansion and hi is height prior to a particular wet-dry cycle.

After four cycles of wetting and drying, the tests are completed, and the results shown in Figures 4.16 and 4.17. The absolute swelling ratio of modified soil is always lower than natural soil, and the improvement of swelling in stabilized specimens is evident, according to Figures 4.16 and 4.17. For both types of soil samples, the absolute swelling ratio reaches equilibrium at the fourth cycle of wetting - drying and rises with more wetting and drying cycles. After 4 cycles, the values are 24% and 4%, whereas the initial values are

20% and 1.5%. In terms of relative swelling ratio, natural soil shrinks as cyclic wetting and drying increases, whereas improved soil grows as cyclic wetting and drying increases. This outcome demonstrates that the soil's expansion is not entirely reversible. According to experiment findings, soil stabilized with 40% fly ash has a relatively low swelling ratio even after four cycles of wetting and drying. When compared to natural soil, the effect of ameliorants on the capacity to swell during wetting-drying cycles is notable.

Table 4. 14 Cyclic Swelling Potential record for Absolute and Relative swelling ratio

Types of soil	Cycles	Initial height (ho) (cm)	height after expansion (hw) (cm)	heights of the air-dried samples (hi) (cm)	Absolute swelling ratio	Relative swelling ratio
Natural soil (100%E - 0%F)	0 cycle	2.00	2.40	2.00	20.00	20.00
	1 cycle	2.00	2.44	2.05	22.00	19.02
	2 cycle	2.00	2.46	2.07	23.00	18.84
	3 cycle	2.00	2.47	2.09	23.50	18.18
	4 cycle	2.00	2.48	2.10	24.00	18.10
Improved soil (60%E-40%F)	0 cycle	2.00	2.03	2.00	1.50	1.50
	1 cycle	2.00	2.06	2.01	3.00	2.49
	2 cycle	2.00	2.07	2.02	3.65	2.62
	3 cycle	2.00	2.08	2.02	3.75	2.72
	4 cycle	2.00	2.08	2.02	4.00	2.97

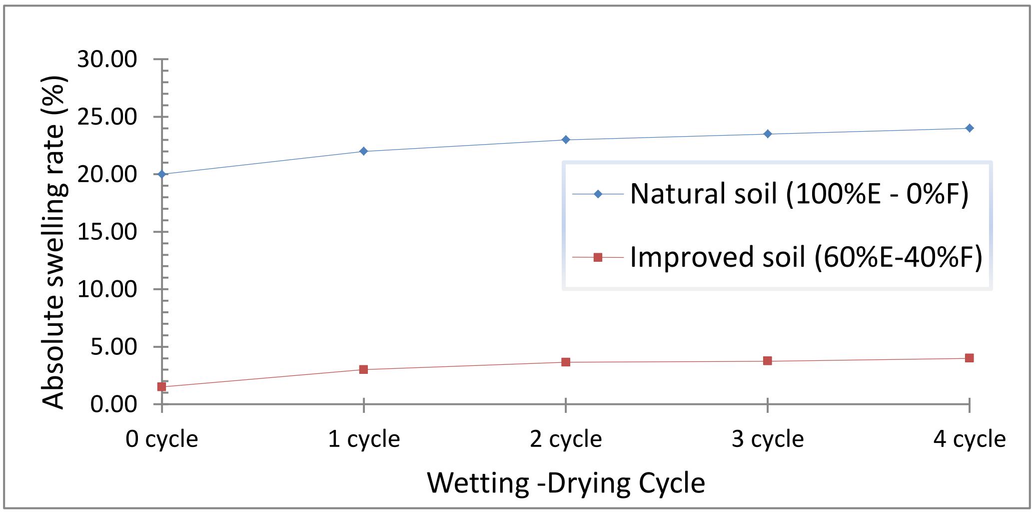


Figure 4. 16 Absolute swelling ratio trends of the natural soil and improved soil.

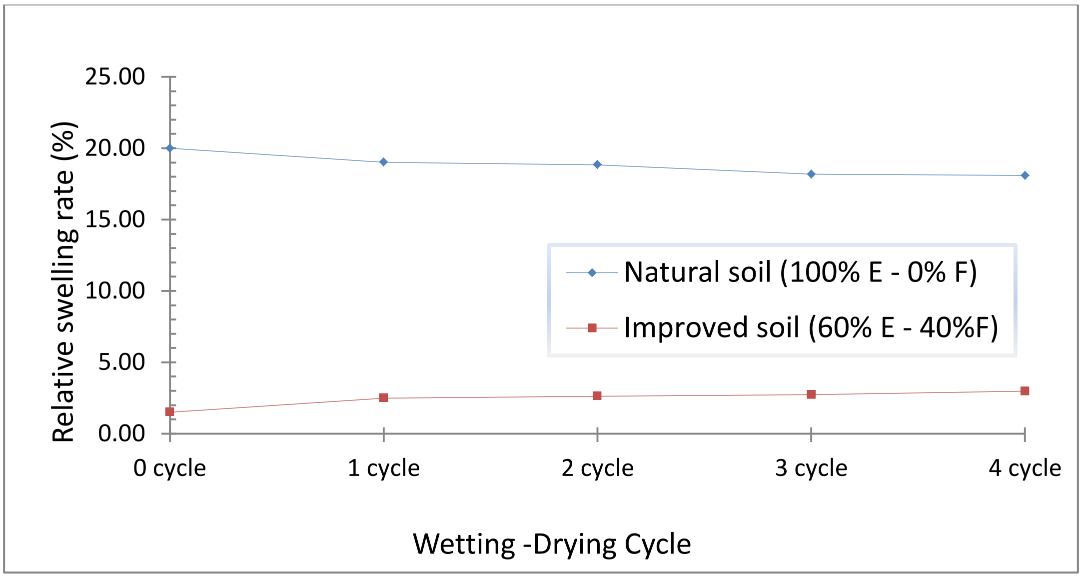


Figure 4. 17 Relative swelling ratio trends of the natural soil and improved soil.

4.4.2 Cyclic Unconfined Compressive Strength Test

Numerous modified specimens created using the 40% of fly ash dosages and starting moisture levels (OMC). Sample sealed in plastic bags cured in wet sand for 28 days curing period. After the curing process finalized, sample spacemen immersed in water for 48 hours in a plastic tanker filled with water. By carrying out UCS studies on the same soils at the 1st cycle, 2nd cycle, 3rd cycle and 4th cycle, it is possible to determine the impact of additive on the strength behavior of treated soils. The same testing cycles also used to test untreated soils. The steps are the same as those described in Section 3.7. The subsequent wetting and drying cycle uses the cylindrical samples and air-dried to their initial water content (NMC). After four wetting-drying cycles, the process comes to the end. Table 4.15 lists the UCS values of untreated and treated samples for various wetting-drying cycle times, and Figures 4.18, 4.19, 4.20, 4.21 and 4.22 display photographs taken during the course of the experiment. According to Table 4.16, which summarizes the results of the UCS tests, an increase in wet-dry cycles causes a gradual weakening in strength, with a strength loss of 23.2% for the specific proportions of fly ash in soil. When compared to natural soil samples, the UCS values of improved soil samples significantly increase, and the increase in durability is apparent. The water stabilities of untreated specimens are quite poor, as shown in Figures 4.18, and they collapse after submerging in water for the first 48 hours. All of the untreated samples fail the water stability test because they disintegrate when submerged in water. However, fly ash-improved soil samples are capable of going through numerous wetting-drying cycles. Because of the admixtures, samples remain in good shape throughout the duration of the test.

Table 4. 15 Results of UCS during cyclic wetting-drying process.

Types of soil	Cycles	Unconfined compressive strength (kPa)	Percentage decrease in UCS value	Conditions of samples immersed in water
Natural soil	0th cycle	121.2	–	Disintegrate before finalized submerged in water for a period of 48 hour.
	0th cycle	–	–	
Improved soil	0th cycle	389.6	0.00	Maintaining good conditions
	1st cycle	354.91	8.90	Maintaining good conditions
	2nd cycle	327.66	15.90	Maintaining good conditions and starting edge peelings
	3rd cycle	313.84	19.45	Maintaining its good condition, with only a few edge peelings
	4th cycle	299.21	23.20	Maintaining its good condition, with only a few cracks on the surface and edge peelings

Figure 4. 18 Natural soil samples disintegrate before 48-hour immersion in water.

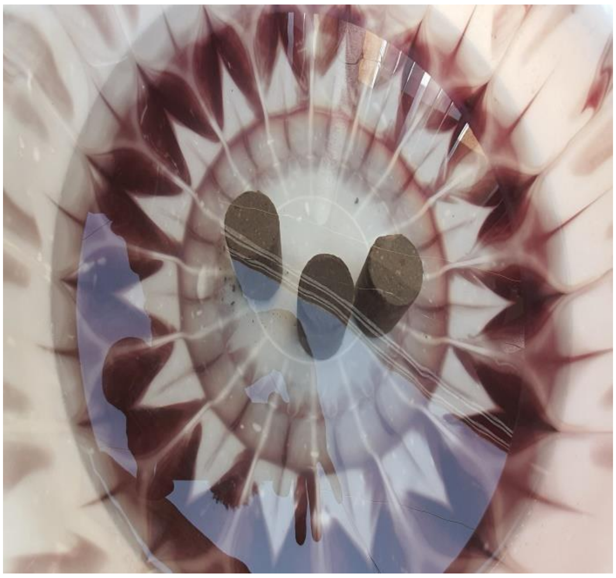


Figure 4. 19 Improved soil samples immersed in water for 48 h at 1st cycle of wetting-drying

Figure 4. 20 Improved soil samples immersed in water for 48 h at 2nd cycle of wetting-drying

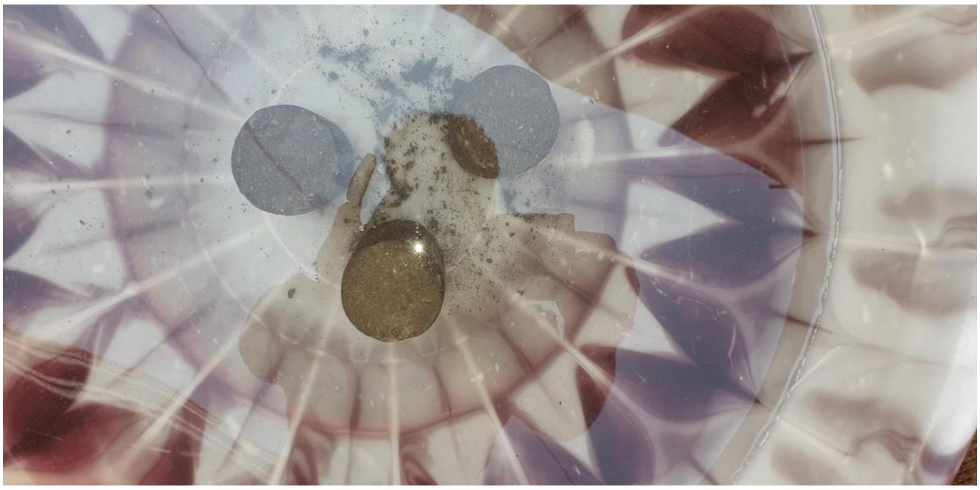


Figure 4. 21 Improved soil samples immersed in water for 48 h at 3rd cycle of wetting-drying

Figure 4. 22 Improved soil samples immersed in water for 48 h at 4th cycle of wetting-drying

4.5 Batch Leaching Test

In this study, water is the liquid (leachant) and fly ash was taken as the solid material, and the liquid to solid ratio selected was ten (L/S=10). The batch-leaching test does not show the rate of leaching; it only used to determine the amount of heavy metals found in lechant after extraction, which were to simulate the effect of leachable fly ash in ground water. (EPA METHOD 1316, 2012). As the fly ash, gradation indicates more than 85% of the particle is less than 0.3 mm. Therefore, the minimum dry mass for testing was taken 20 ± 0.05gm, and contact time 24 ± 2 hr in 250ml cylinder (EPA METHOD 1316, 2012) and then filtered (0.45 µm) filter membrane.

4.5.1 Leached Heavy metal concentration before S/S Treatment Method

As shown below the concentration amount of heavy metals Cu (Copper), Fe (Iron), Ni (Nickel) and Pb (Lead) in leached liquid examined by flame AAS and it is found above the permissible limit of WHO for drinking water as shown in Table 4.16.

Table 4. 16 Concentration of heavy metals, Cu (Copper), Zn (Zinc), Pb (Lead), Ni (Nickel), and Co (Cobalt) (µg/L) in water samples

Heavy metals	Concentration amount (µg/L)	WHO Recommendation (2019) (µg/L)
Cu	2478	2000
Zn	2350	_
Pb	3262	10
Ni	626	70
Co	168	_

4.5.2 Leached Heavy metal concentration after S/S Treatment Method

Due to its serious pollution toxicity, fly ash from municipal solid waste (MSW) must be treated properly before being dumped in a secure waste site. Nowadays, many studies have demonstrated that cement-based stabilization/solidification could reduce the toxicity pollution effectively by encapsulating the heavy metals into cement matrix, which leads to greater capacity and weight as described on section 2.11, sub section 2.11.1.3. Moreover, immobilization mechanism introduced by the interaction between the MSW fly ash and Ordinary Portland Cementmaterials. Before cement-based stabilization/solidification the MSW fly ash prewashed by Acetic acid(liquid to solid ratio 20:1) drastically reduce the toxicity of heavy metals and reduce the consumption of cement (Chengcheng et al., 2018). For the purpose of S/S process 4:6(Cement/Fly ash ratio) (Qiang Tang et al., 2016) used then results are found out as shown below in Table 4.17.

Table 4. 17 Concentration of heavy metals after S/S method, Cu (Copper), Zn (Zinc), Pb (Lead), Ni (Nickel), and Co (Cobalt) (µg/L) in water samples

Heavy metals	Before S/S treatment Concentration amount (µg/L)	After S/S treatment Concentration amount (µg/L)	After S/S treatment with 28 day cured Concentration amount (µg/L)	% Decrease untreated vs treated(but not cured)	% Decrease untreated vs treated (cured)	WHO (2019) Recommendation (µg/L)
Cu	2478	2384	1478	3.79	40.4	2000
Zn	2350	2143	1096	8.81	53.4	_
Pb	2262	428	82	81.08	96.4	10
Ni	626	106	64	83.07	89.8	70
Co	168	104	77	38.10	54.2	_

Copper: Contamination of drinking water with high level of copper may lead to chronic anemia (Gebrekidan and Samuel, 2011). In this study, 2478 µg/L copper concentrations were recorded in water samples before it was treated. However, after treated by S/S method the copper amount in water sample decrease by 40.4% (1478 µg/L), which is within the limit of the guideline set by WHO (2019) for copper level in drinking water 2000 µg/L.

Zinc: is one of the significant trace elements that essential for many organisms' physiological and metabolic processes. The organism, however, may be toxic to higher zinc concentrations (Gebrekidan and Samuel, 2011). Due to its limited mobility from the site of rock weathering or from natural sources, it plays a significant role in protein synthesis and exhibits a relatively low concentration in surface water (Gebrekidan and

Samuel, 2011). In this study, zinc concentrations of 2350 μg/L were found in water samples. However, the zinc content of the water sample decreased by 53.4% (1096 μg/L) following S/S treatment. The WHO (2019) has not established any guidelines for the amount of zinc in drinking water.

Lead: is the most significant of all the heavy metals because it is toxic, very common and harmful even in small amounts (Gebrekidan and Samuel, 2011). Lead enters the human body in many ways. It can be inhaled in dust from lead paints, or waste gases from leaded gasoline. It is found in trace amounts in various foods, notably in fish, which are heavily subjected to industrial pollution. Most of the lead we take is removed from our bodies in urine; however, as exposure to lead is cumulative over time, there is still risk of buildup, particularly in children. Studies on lead are numerous because of its hazardous effects. High concentration of lead in the body can cause death or permanent damage to the central nervous system, the brain, and kidneys (Gebrekidan and Samuel, 2011). In this study, 3262 μg/L lead concentration was found in water sample. This result (1359 μg/L) was more than the admissible limit of lead concentration recommended by WHO (2019) of lead in drinking water (10μg/L). However, after it was treated by S/S method the Lead amount in water sample become 82 μg/L, which is decreased by 96.4%.

Nickel: The maximum permissible level of Ni in drinking water according to WHO (2019) is 70 μg/L. The WHO (2019) maximum admissible limit (70 μg/L) for nickel concentration was exceeded in this study, where nickel concentrations of 626 μg/L were found in water samples. However, after being treated using the S/S method, the amount of nickel in the drinking water was reduced by 89.8% to 64 μg/L.

Cobalt: concentration found 168 μg/L in water sample for this study. However, after it treated by S/S method the Nickel amount in water sample become 77 μg/L, which is decreased by 54.2%. Though the maximum admissible limit, of cobalt is not mentioned by WHO (2019), all the samples analyzed comply the New Zealand (1000 μg/L) and US EPA (100 μg/L) maximum admissible limits of cobalt in drinking water.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study, to determine the effects on strength and durability characteristics of expansive soil, different percentages of fly ash mixed with soil and conducted laboratory tests such as Index properties, compaction, CBR, UCS, and behavior of expansive soils improved by 40% fly ash during wetting-drying cycles has been investigated and the following conclusions are drawn.

- The liquid limit, plasticity index and free swell values decreased from 69.2% to 51.1% by (26.16%), 33.2% to 16% by (51.81%) and 75% to 37% by (50.67%) respectively with increasing the percentage of fly ash from 0% to 40%.
- MDD and OMC decrease from 1.34gm/cc to 1.3gm/cc by (2.99%) and 26.9% to 23.8% by (11.52%) respectively as the percentage of fly ash increased 0% to 40%.
- As the percentage of fly ash increased from 0% to 40%, CBR value increased from 2.6% to 4.3%, which increased by 65.38%. In reverse the CBR swell values decreased from 9.6% to 3.2% which decreased by 66.67% when fly ash percentage increased from 0% to 40%.
- As the percentage of fly ash increased from 0% to 40%, UCS and Cu of the soil increased by 112.21%, which increased from 121.2kPa to 257.2kPa and 60.6kPa to 128.6kPa respectively.
- The untreated soil disintegrated before even finalized the 48 hr wetting process. However, expansive soil improved by 40% fly ash significantly can bear numerous wetting-drying cycles and keep a good condition until the end of the test. This indicates that durability of fly ash in seasonal fluctuation is quite good.
- Comparing expansive soil samples with improved soil samples, lower swelling ability, higher strength and much durability observed in improved soil.
- Cement-based S/S reduced the toxicity of fly ash effectively by encapsulating the heavy metals. In addition, better results recognized for 4:6 cement-fly ash ratio after 28-day curing time, regarding decreased concentration of heavy metals.

5.2. Recommendation

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

- Actual effect and performance of the fly ash in the field should study along with seasonal fluctuation.
- Further study recommended using more than 40% of fly ash; it may give a better improvement.
- Further study recommended on the durability of soil-fly ash mix by considering leaching rate effect of fly ash.
- Further study recommended on treatment of heavy metals by using other treatments methods in addition to S/S method. Moreover, utilize different proportion of cement to fly ash ratio for better outcomes.

References

Geda special economic zone. (2020). Retrieved 2022, from Geda special economic zone web site: https://www.gadasez.gov.et

Admas, (2020). *Potential use of Rice Husk Ash as Woreta Expansive Soil Stabilizer*.Unpublished MSc. Thesis, Addis Ababa University.

Alene, T. (2010). Experimental treatment options for expansive soil on unpaved. *1st AFCAP Practitioners conference,Addis Ababa*.

Al-Rawas, & Mattheus. (2006). Expansive Soils Recent Advances in Characterization and treatment. *Proceedings and Monographs in Engineering, Water and Earth Sciences*,, Netherlands.

Altaf Hussain et al. (2020). Incineration Fly Ash and Its Treatment to Possible Utilization: A Review. *International Journal of Environmental Research and Public Health*.

Andargie et al. (2022). Five year trend of Malaria prevalence in Mojo Town. *Infection and Drug resistance.Volume 15:455-464*.

Aravind Pedarla, (2009). Durability studied on stablization effectiveness of soils contaning different fractions of montmorinolite. *The universty of Texas at Arlingtion*.

Arora, (2003). *Soil Mechanics and Foundation Engineering.* Delhi,A.K.Jain: Standard publisher distributors.

Ashango, & Patra. (2016). Behavior of Expansive Soil Treated with Steel Slag,Rice Husk Ash, and Lime. *J. Mater. Civ. Eng , 28 (7)*.

Asres E. (2017). Stabilization of Expansive Clay Soils Using Quarry Waste. *MSc. Thesis Addis Ababa University*.

Bantayehu, U. (2017). Expansive Soils in Ethiopia: A Review. *International Journal of Scientific Research Engineering & Technology (IJSRET)*, Vol .6 (Issue 2278-0882).

Bao-tian et al. (2015). Research on Wetting-Drying Cycles' Effect on the Physical and Mechanical Properties of Expansive Soil Improved by OTAC-KCl. *Institute of Geotechnical Engineering, Hohai University, Nanjing, Jiangsu 210098, China*.

Chen, (1988). Foundations on expansive soils. *Chen and Associates, Consulting Soil Engineers Denver, Colo., U.S.A.*

Chengcheng et al. (2018). Review on Cement Stabilization/Solidification of Municipal Solid Waste Incineration Fly Ash. *Hindawi Advances in Materials Science and Engineering*.

Das, and Roy. (2014). *Effect of Expansive Soil on Foundation and Its Remedies*. CRC Press, vol. 3, no. 6, pp 92-95.

Diliberto et al. (2020). “A zero-waste process for the management of MSWI fly ashes: production of ordinary Portland cement. *Environmental Technology Journal of Environmental Research*, 1199–1208.

Elarabi, (2004). Factors influencing swelling behavior of expansive soils. *BRRJ., 6*.

Fikreyesus, Y. (2022). *Effect of Bottom ash on stablizing expansive soil*. Thesis work,Adama University.

Gebrekidan, and Samuel. (2011). Concentration of Heavy Metals in Drinking Water from Urban Areas of the Tigray Region, Northern Ethiopia. *ISSN:2220-184X*.

Gupta, J. (2016). Interpretation of Rice Husk Ash on Geotechnical Properties of Cohesive Soil. *irjet(3)*.

Haiying et al. (2010). Thermal characterization of fly ash from one municipal solid waste incinerator (MSWI) in Shanghai. *Process. Saf. Environment*, 269–275.

Hasriana et al. (2021). Strength Characteristics of Compacted Fly Ash Treated Expansive Soil due to Wetting-Drying Cycles Repetition. *Department of Civil Engineering, Politeknik Negeri Ambon, Indonesia*.

Hayal, D. (2022). An overview of solid waste management systems in the city administration of Addis Ababa: past to present. *Resources and Residues - Field Report 01*.

James et al. (2022). Geological Soil Parent Material Influence on Forest Surface Soil Chemical Characteristics in the Inland Northwest. *Collage of Environmental Science and Forestry, State university of New york*.

Jayanti Munda et al. (2022). Investigation on performance of expansive soil stabilized with fly ash and nano-SiO2. *Science Direct*, 1268-1275.

Kamil, K. (2011). Measurement of swelling pressure: direct method versus indirect methods. *Canadian Geotechnical Journal*.

Kassahun, (2013). *Examining Atterberg limits for expansive soils*. Thesis Work Addis Ababa University.

Lam et al. (2010). Use of incineration MSW ash: A review. *Sustainability International Journal of Environmental Research*, 1943–1968.

Lancellotta. (2008). Geotechnical engineering. *CRC Press*.

Lucian C. (2008). *Geotechnical Aspects of Buildings on Expansive Soils in Kibaha*. Doctoral Thesis.

Magdi, & Asim. (2017). Damages of Buildings on Expansive Soils. *J. Mater. Civ. Eng , 32 (5)*, 108-116.

Makusa, G. (2012). soil stabilization methods and materials in engineering practice. *Geotechnical and Geological Engineering*, 152-156.

Mangialardi et al. (1999). P. Optimization of the solidification/stabilization process of MSW fly ash in cementitious matrices. *International Journal of Environmental Research and Public Health*.

Margallo, M. et al, (2015). Environmental Sustainability Assessment of the Management of Municipal Solid Waste Incineration Residues. *A Review of the Current Situation*, 1333- 1353.

Mishra, & Rao. (2008). Analysis of Swelling and Shrinkage Behavior of Compacted Clays. *Geotechnical and Geological Engineering*, 289-298.

Mohanty et al. (2018). Stabilization of Expansive Soil. *Civil Engineering Department, VSSUT, Burla, India*.

Nalawade et al. (2012). Assessment of Groundwater and Surface Water Quality indices for Heavy Metals nearby Area of Parli Thermal Power Plant. *Universal Journal of Environment Research and Technology*.

Qiang Tang et al. (2016). Solidification/Stabilization of Fly Ash from a Municipal Solid Waste Incineration Facility Using Portland Cement. *Hindawi Publishing Corporation*.

Reddy, & krishna. (2018). Lime-Stabilized Black Cotton Soil and Brick Powder Mixture as Subbase Material. *J. Mater. Civ. Eng , 28 (7)*, 1-5.

Richardson, (2020). Urban forests near municipal solid waste incinerators do not show elevated trace metal or rare earth element concentrations across three cities in the northeast USA. *Environmental Science and Pollution Research*, 2-6.

Sabbas et al. (2003). Management of Municipal Solid Waste Incineration Residues. *International Journal of Environmental Research and Public Health*, 61 - 88.

Sayem Hossain et al. (2016). Effect of Drying-Wetting Cycles on Saturated Shear Strength of Undisturbed Residual Soils. *Department of Geological Sciences, Jahangirnagar University, Dhaka, Bangladesh*.

Sheng-quan et al. (2019). Study on Physical-Mechanical Properties and Microstructure of Expansive Soil Stabilized with Fly Ash and Lime. *Advances in Civil Engineering*.

Simegn et al. (2021). Characterization and Optimization of Incinerated Municipal Solid Waste Fly Ash as a Cement Substitute Material in Concrete at Reppie Waste to Energy Plant,Ethiopia. *Advances in Environmental Studies,ISSN: 2642-4231*.

Sridharan, & Prakash. (2016). Classification procedures for expansive soils. *Proc. Instn. Civ. Engrs. Geotech. Engng*, 235–240.

Sudhakar, and Revanasiddappa. (2004). Influence of cyclic wetting drying on collapse behaviour of compacted residual soil. *Department of Civil Engineering, Indian Institute of Science, Bangalore 560 012, India*.

Taddesse, S. (1989). *Investigation into some of the Engineering Properties of Addis Ababa expansive soil*. Addis Ababa, Ethiopia Thesis Work.

Teklu, (2003). *Examining the swelling pressure of Addis Ababa expansive soil*. Thesis Work, Addis Ababa, Ethiopia.

Thiha, and Swae. (2019). Study on Stabilization of Expansive Soil with Fly-ash. *Department of Civil Engineering, Technological University (Monywa), Myanmar*.

Wei Qi et al. (2022). Experimental Investigation on the Impact of Drying–Wetting Cycles on the Shrink–Swell Behavior of Clay Loam in Farmland. *College of Agricultural Science and Engineering, Hohai University, Nanjing 211100, China*.

Xiao et al. (2020). An overview of the municipal solid waste management modes and innovations in Shanghai, China. *Environmental Science and Pollution Research*, 3-6.

Yadu et al. (2011). Comparison of Fly Ash and Rice Husk Ash Stabilized Black Cotton Soil. *International Journal of Earth Sciences and Engineering*, 42-45.

Yang et al. (2020). Risk response for municipal solid waste crisis using ontology-based reasoning. *International Journal of Environmental Research and Public Health*.

Yigzaw, (2021). *Study on behavior of interlocking compressed earth block using municipal solid waste incinerated.* (Issue March).AASTU.

Younus, M. (2010). Laboratory investigations to maximise the utility of fly ash for landfill liner. *Master of technology in Geothecnical Engineering,Guwahati 781039*.

APPENDICES

Appendix- A

Results of Natural Soils

Table A-1 Natural Moisture Content of soil sample 1(pit 1), soil sample 2(pit 2) and soil sample 3(pit 3)

Specimen reference	Pit 1		Pit 2		Pit 3	
Container no	A-5	G-3	E-5	E-1	F-6	M-33
Mass of wet Soil +Container (m2) gm	112	115	124	128	119	123
Mass of dry Soil +Container (m3) gm	90	93	103	106	98	100
Mass of container (m1) gm	18.1	16.2	15.7	17.1	18.5	16
Mass of moisture(m2-m3) gm	22	22	21	22	21	23
Mass of dry soil (m3-m1) gm	71.9	76.8	87.3	88.9	79.5	84
Moisture content (m2-m3)*100/(m3-m1) (%)	30.60	28.65	24.05	24.75	26.42	27.38
Average moisture content (%)	29.6		24.4		26.9	

Table A-2 Specific Gravity of Soil Sample1 (Pit1), Soil Sample2 (Pit2) and Soil Sample 3(pit 3)

Test Method ASTM-D 854						
Determination number	Pit 1		Pit 2		Pit 3	
Bottle no	6	9	12	7	4	8
Wt. of bottle + Water (g) W1	155.7	156.46	157.25	157.75	156.31	157.19
Wt. of bottle + Water+ Soil (g) W2	165.49	166.11	166.7	167.3	165.9	166.8
Wt. of soil (g) Ws	15	15	15	15	15	15
Wt. of bottle + Dry soil (g)	5.21	5.35	5.55	5.45	5.41	5.39
Specific Gravity at 20 °C	2.88	2.80	2.70	2.75	2.77	2.78
Average Specific Gravity	2.841		2.73		2.78	

Table A-3 Free swell of soil sample 1(pit 1), soil sample 2(pit 2) and soil sample 3(pit 3)

Sr No	Material Description	Initial Volume	Final Volume	Free Swell%
1	pit 1	10	17.5	75
2	pit 2	10	14.5	45
3	pit 3	10	16	60

Table A-4 Grain Size Analysis of soil sample 1(pit 1)

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	6.2	1.3	98.7
2	8.5	1.8	98.2
1.18	10.7	2.3	97.7
0.85	15.6	3.4	96.6
0.6	21.2	4.6	95.4
0.425	26.6	5.7	94.3
0.3	30.7	6.6	93.4
0.15	38.0	8.2	91.8
0.075	35.0	7.6	92.4

Table A-5 Hydrometer Analysis of soil sample 1(pit 1)

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L (cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	92.4
0.5	1.0235	0.0041	1.0276	20	9.7	0.01638	0.060	89.55
1	1.0225	0.0041	1.0266	20	11.5	0.01638	0.043	86.30
2	1.0215	0.0041	1.0256	20	12.1	0.01638	0.030	83.06
4	1.0205	0.0041	1.0246	20	12.3	0.01638	0.021	79.81
6	1.0195	0.0041	1.0236	20	12.9	0.01638	0.018	76.57
10	1.0185	0.0041	1.0226	20	13.4	0.01638	0.014	73.32
20	1.0175	0.0041	1.0216	20	13.4	0.01638	0.010	70.08
30	1.0165	0.0041	1.0206	20	13.4	0.01638	0.008	66.84
60	1.0155	0.0041	1.0196	20	13.7	0.01638	0.006	63.59
120	1.0140	0.0041	1.0181	20	13.7	0.01638	0.004	58.72
480	1.0130	0.0041	1.0171	20	13.9	0.01638	0.002	55.48
1440	1.0125	0.0041	1.0166	20	14.4	0.01638	0.001	53.86

Table A-6 Particle size distribution of expansive soil

Expansive soil	Particle size (%)			
	Gravel (> 4.75mm)	Sand (4.75-0.075mm)	Silt (0.075 0.002mm)	Clay (< 0.002mm)
Pit -1	0	7.6	37.4	55
pit -2	0	12.3	46.1	41.6
Pit -3	0	9.2	40.7	50.1

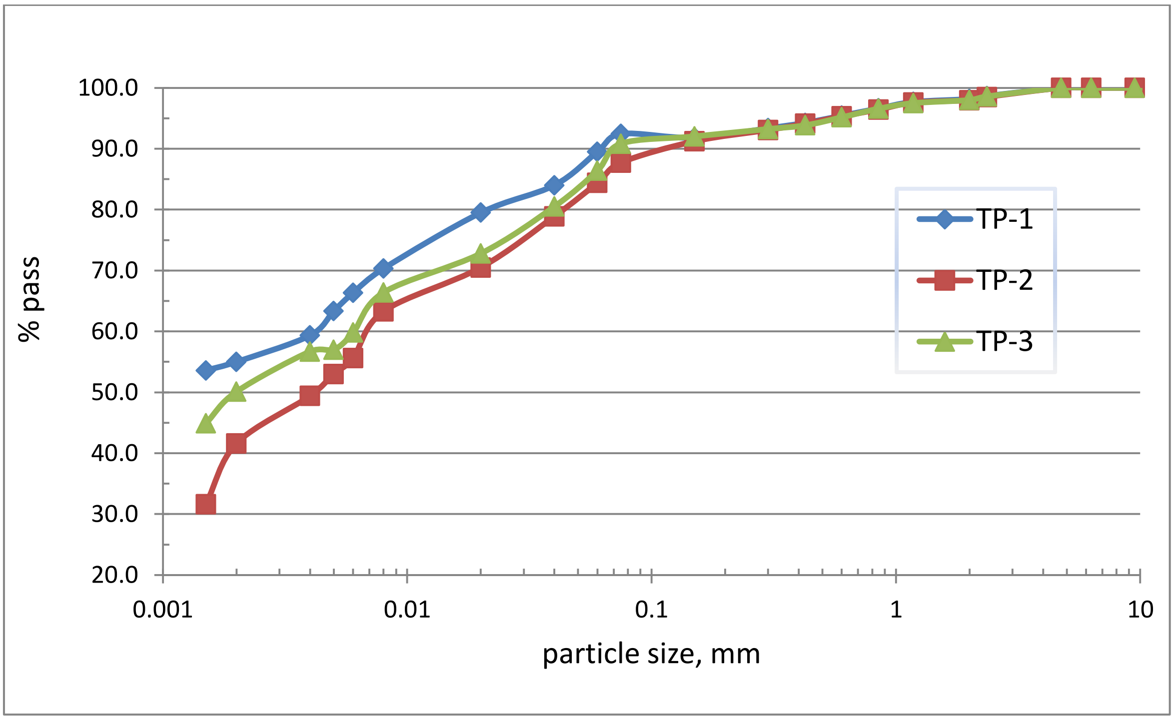


Figure A-1 Gradation curve of Expansive soil from TP-1, TP-2 and TP-3.

Table A-7 Liquid limit and Plastic Limit test of Soil Sample TP-1,TP-2 and TP-3

		Liquid Limit			Plastic Limit	
PIT -1	No. of Blows	32	28	18		
	Wt. of cont. + wet soil (g) = (w1)	43.33	42.53	42.85	21.58	20.32
	Wt. of cont. + dry soil (g.) = (w2)	35.32	34.80	34.94	19.39	18.51
	Wt. of container (g.) = (w3)	23.56	23.65	23.65	12.98	12.87
	Mass of moisture (g.) (w1-w2) = x	8.01	7.73	7.91	2.19	1.81
	Wt. of dry soil (g.) (w2-w3) = y	11.76	11.15	11.29	6.41	5.64
	Moisture Content (%) = (100x/y)	68.11	69.33	70.06	34.17	32.09
	Average	69.2			33.1	
	Plasticity Index(PI) = LL - PL	36				

		Liquid Limit			Plastic Limit	
PIT-2	No. of Blows	34	29	21		
	Wt. of cont. + wet soil (g) = (w1)	41.33	42.03	41.97	20.87	20.71
	Wt. of cont. + dry soil (g.) = (w2)	35.36	36.44	34.91	19.66	19.63
	Wt. of container (g.) = (w3)	24.11	26.23	22.14	14.37	14.65
	Mass of moisture (g.) (w1-w2) = x	5.97	5.59	7.06	1.21	1.08
	Wt. of dry soil (g.) (w2-w3) = y	11.25	10.21	12.77	5.29	4.98
	Moisture Content (%) = (100x/y)	53.07	54.75	55.29	22.87	21.69
	Average	54.4			22.3	
	Plasticity Index(PI) = LL - PL	32				
		Liquid Limit			Plastic Limit	
PIT-3	No. of Blows	33	28	20		
	Wt. of cont. + wet soil (g) = (w1)	42.39	42.56	41.08	20.41	21.74
	Wt. of cont. + dry soil (g.) = (w2)	35.25	35.30	34.30	18.80	19.57
	Wt. of container (g.) = (w3)	23.56	23.65	23.65	13.35	12.69
	Mass of moisture (g.) (w1-w2) = x	7.14	7.26	6.78	1.61	2.17
	Wt. of dry soil (g.) (w2-w3) = y	11.69	11.65	10.65	5.45	6.88
	Moisture Content (%) = (100x/y)	61.08	62.32	63.66	29.54	31.54
	Average	62.4			30.5	
	Plasticity Index(PI) = LL - PL	32				

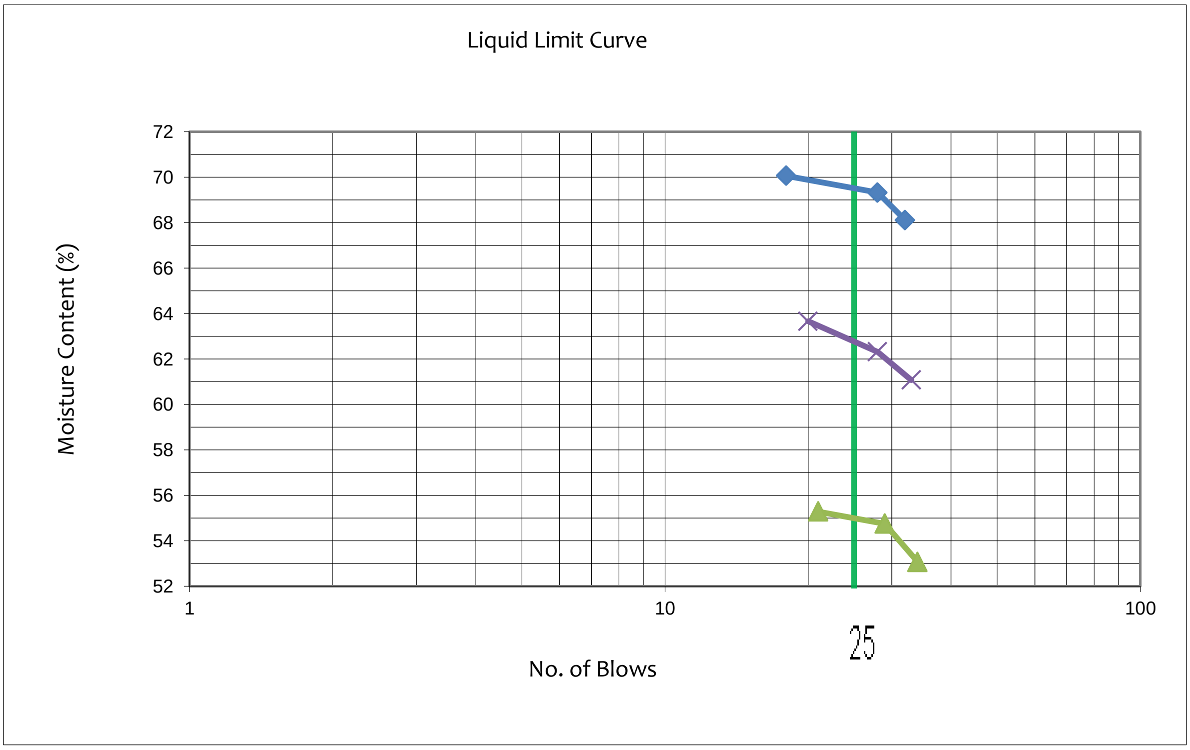


Figure A-2 Liquid Limit determination of soil sample TP1, TP2 and TP3

Table A-8 Compaction property of Soil Sample 1 (pit 1)

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	4673	4719	4777	4764	
Weight of Mould (g)	3175	3175	3175	3175	
Weight of Wet soil (g)	1498	1544	1602	1589	
Volume of Mould (cc)	944	944	944	944	
Wet density (g/cc)	1.59	1.64	1.70	1.68	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	237.2	260.0	220.0	269.0	
Weight of Dry soil + cont. (g)	199.7	215.7	177.5	216.0	
Weight of Container (g)	27.3	30	26.6	34.1	
Weight of water (g)	37.5	44.3	42.5	53	
Weight of Dry soil (g)	172.4	185.7	150.9	181.9	
Moisture content (%)	21.75	23.86	28.16	29.14	
Dry Density (g/cc)	1.30	1.32	1.32	1.30	
	MDD, g/cc	**1.34**	**OMC, %**		**26.9**

Figure A-3 compaction curve of soil sample TP1

Table A-9 CBR test result for expansive soil TP-1

Blow/Layer = 56/5 Plunger area (mm²) = 1935	MDD (g/cm³) = 1.34 OMC (%) = 26.9 CBR (%) = 2.6

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	0.35	11.5	11.15	9.61
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		0.6	0.1		
1.27		1.1	0.2		
1.91		1.3	0.2		
2.54	13.35	1.9	0.3	0.35	**2.6**
3.81		2.3	0.4		
5.08	20	2.7	0.5	0.50	**2.5**
7.62					

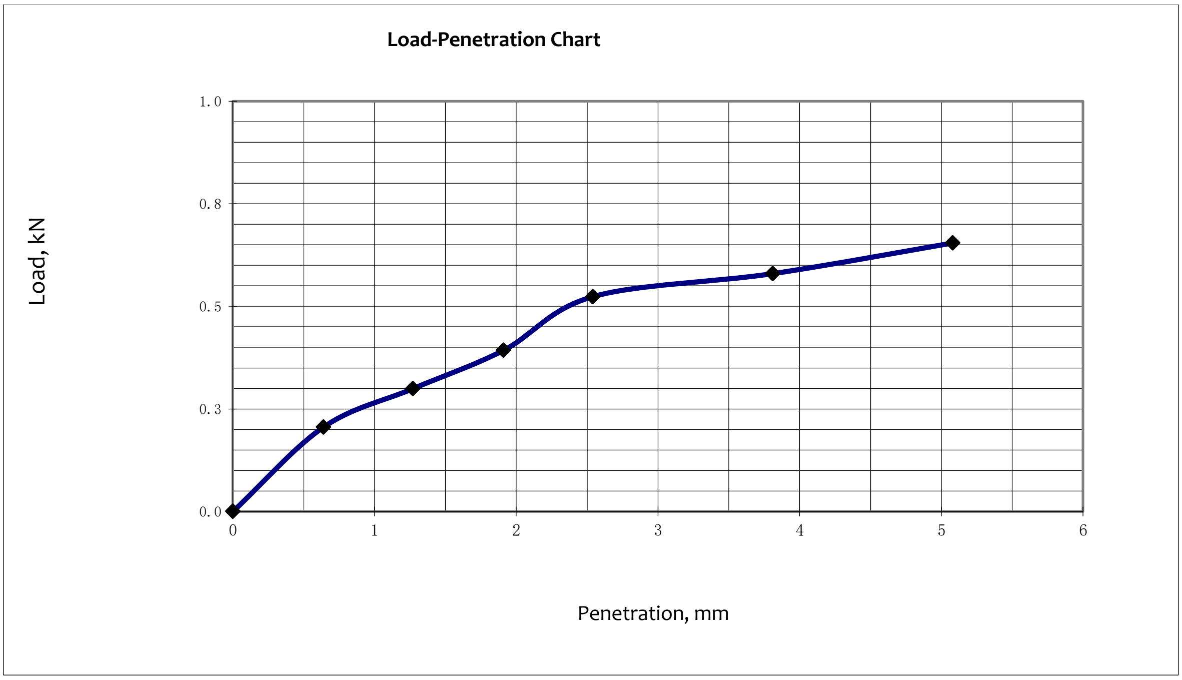


Figure A-4 CBR stress-penetration curve for expansive soil TP-1

Table A-10 CBR test result for expansive soil TP-2

Blow/Layer = 56/5 Plunger area (mm²) = 1935	MDD (g/cm³) = 1.42 OMC (%) = 20.1 CBR (%) = 6

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116 mm		2.2	5.33	3.13	2.70
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		1.5	0.3		
1.27		2.4	0.4		
1.91		3.6	0.7		
2.54	13.35	4.3	0.8	0.80	**6.0**
3.81		4.8	0.9		
5.08	20	5.7	1.1	1.07	**5.3**
7.62					

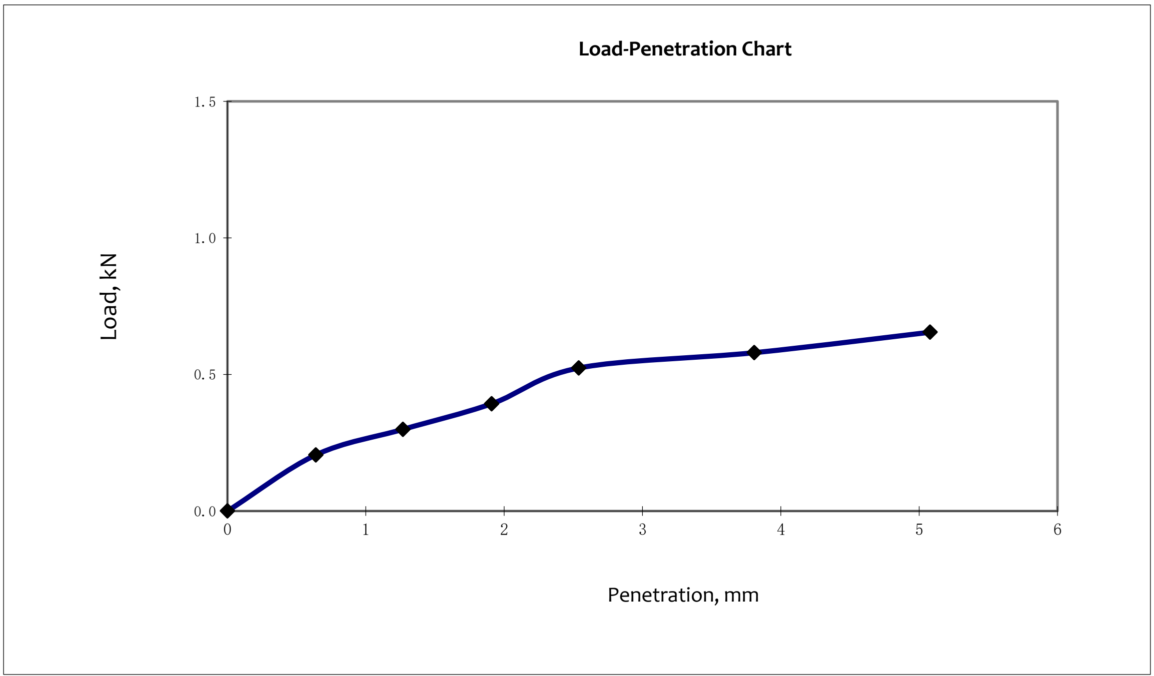


Figure A-5 CBR stress-penetration curve for expansive soil TP-2

Table A-11 CBR test result for expansive soil TP-3

Blow/Layer = 56/5 Plunger area (mm²) = 1935	MDD (g/cm³) = 1.37 OMC (%) = 23.6 CBR (%) = 3.9

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	0.35	7.45	7.10	6.12
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		1.1	0.2		
1.27		1.6	0.3		
1.91		2.1	0.4		
2.54	13.35	2.8	0.5	0.52	**3.9**
3.81		3.1	0.6		
5.08	20	3.5	0.7	0.65	**3.3**
7.62					

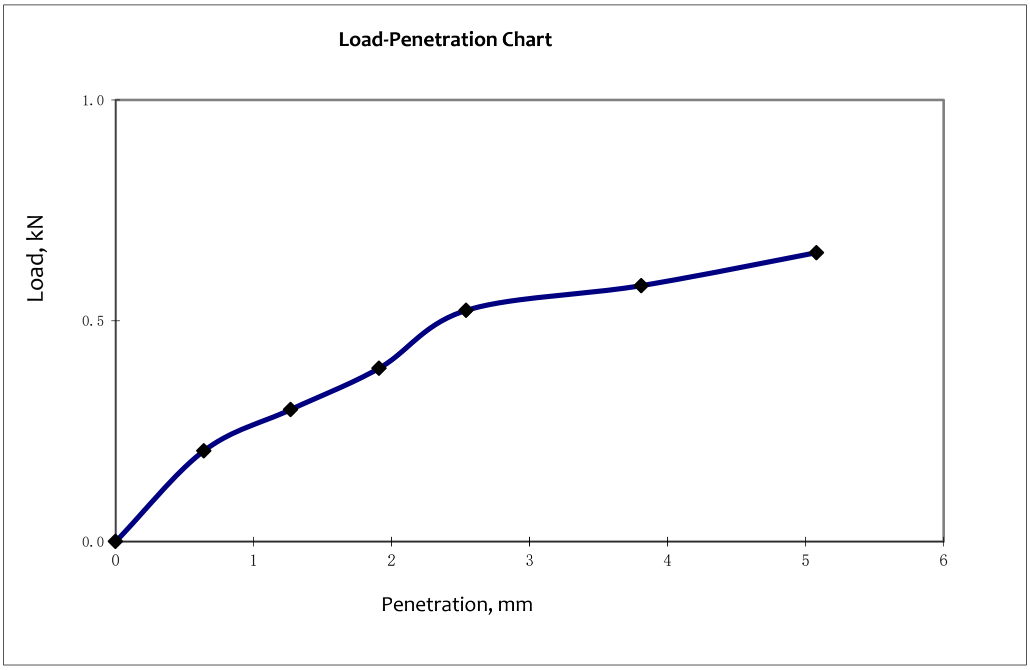


Figure A-6 CBR stress-penetration curve for expansive soil

Table A-12 UCS test result for expansive soil TP-1

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	26.9
Bulk Density (gm/cm^3) =	1.701
Dry Density (gm/cm^3) =	1.340

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	12.664
0.526	18.945
0.789	26.768
1.053	29.838
1.316	36.024
1.579	43.738
1.842	51.411
2.105	55.934
2.368	61.982
2.632	66.451
2.895	72.436
3.158	76.851
3.421	82.774
3.684	91.720
3.947	97.567
4.211	104.902
4.474	111.899
4.737	116.115
5.000	118.801
5.263	119.972
5.526	121.134
5.789	120.675
6.053	120.338
6.316	118.537

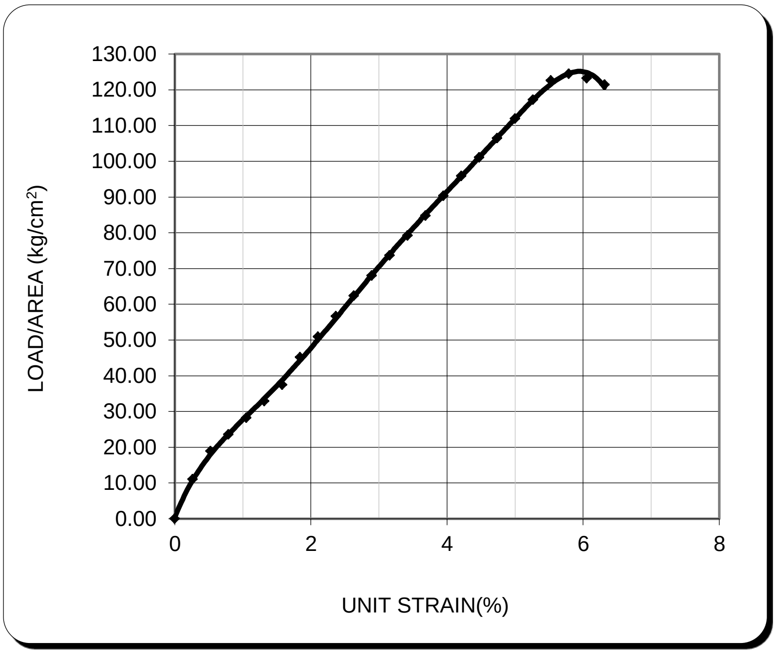


Figure A-7 UCS curve for expansive soil TP-1

Table A-13 UCS test result for expansive soil TP-2

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	20.1
Bulk Density (gm/cm^3) =	1.705
Dry Density (gm/cm^3) =	1.420

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	14.247
0.526	22.103
0.789	25.194
1.053	32.979
1.316	41.819
1.579	49.362
1.842	56.863
2.105	64.324
2.368	71.744
2.632	79.123
2.895	86.461
3.158	93.758
3.421	101.014
3.684	108.230
3.947	115.404
4.211	122.537
4.474	130.801
4.737	129.686
5.000	127.824
5.263	125.971
5.526	124.125

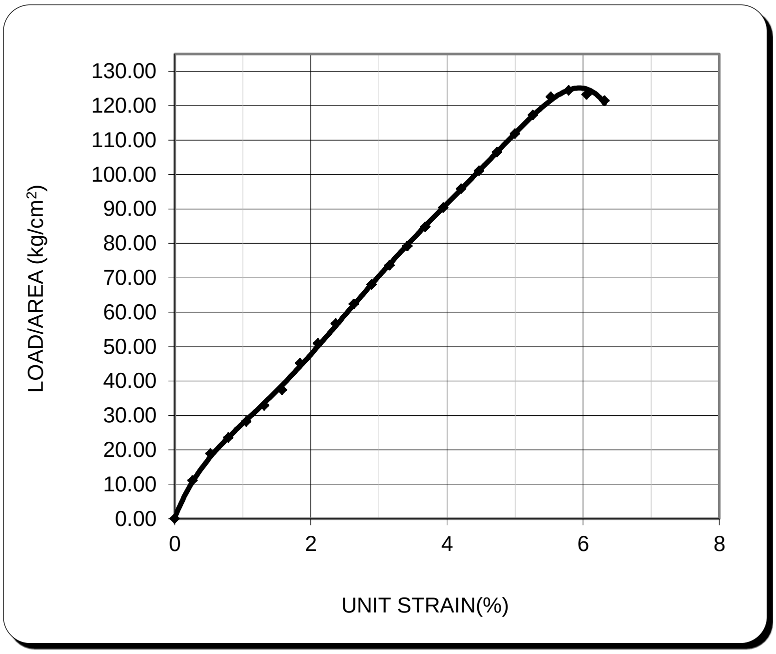


Figure A-8 UCS curve for expansive soil TP-2

Table A-14 UCS test result for expansive soil TP-3

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.6
Bulk Density (gm/cm^3) =	1.694
Dry Density (gm/cm^3) =	1.370

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	11.081
0.526	18.945
0.789	23.619
1.053	28.268
1.316	32.891
1.579	37.490
1.842	45.179
2.105	50.940
2.368	56.669
2.632	62.367
2.895	68.033
3.158	73.667
3.421	79.270
3.684	84.841
3.947	90.381
4.211	95.888
4.474	101.098
4.737	106.528
5.000	111.927
5.263	117.294
5.526	122.630
5.789	124.501
6.053	123.273
6.316	121.464

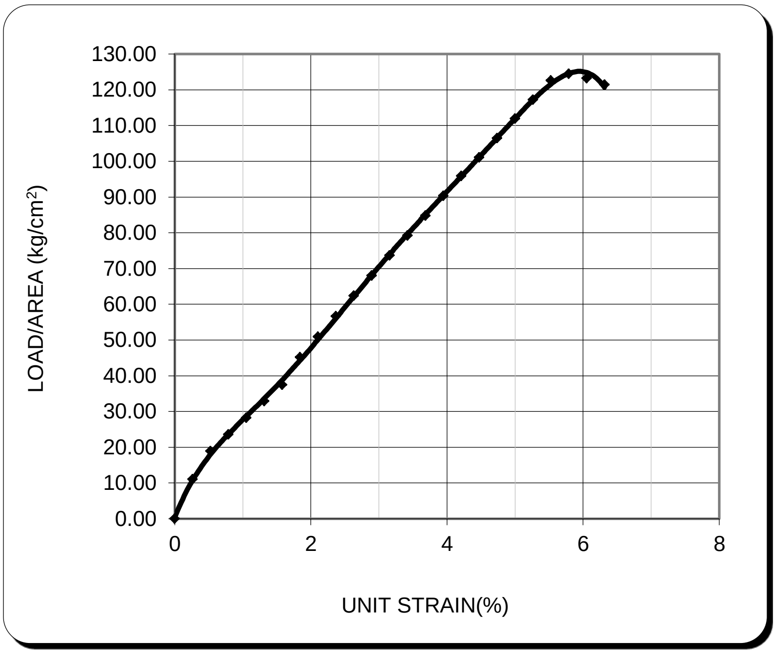


Figure A-9 UCS curve for expansive soil TP-3

Appendix- B

Test Results of Fly ash

Table B-1 Specific Gravity of Fly ash Sample

Test Method ASTM-D 854		
Determination number	Fly ash	
Bottle no	8	3
Wt. of bottle + Water (g) W1	150.7	150.4
Wt. of bottle + Water+ Soil (g) W2	153.9	153.8
Wt. of soil (g) Ws	15	15
Wt. of bottle + Dry soil (g)	11.8	11.6
Specific Gravity at 20 ^{o}C	1.27	1.29
Average Specific Gravity	1.28	

Table B-2 Grain Size Analysis of Fly ash

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	3.5	1.6	98.4
2	3.7	1.7	98.3
1.18	3.8	1.8	98.2
0.85	4.1	1.9	98.1
0.6	4.4	2.1	97.9
0.425	4.7	2.2	97.8
0.3	7.5	3.5	96.5
0.15	9.5	4.4	95.6
0.075	12.2	5.7	94.3

Table B-3 Hydrometer Analysis of Fly ash

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L (cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	94.3
0.5	1.0245	0.0041	1.0286	20	10.7	0.0129224	0.060	91.47
1	1.0220	0.0041	1.0261	20	11.0	0.0129224	0.043	83.48
2	1.0210	0.0041	1.0251	20	11.0	0.0129224	0.030	80.28
4	1.0200	0.0041	1.0241	20	11.0	0.0129224	0.021	77.08
6	1.0190	0.0041	1.0231	20	11.3	0.0129224	0.018	73.88
10	1.0180	0.0041	1.0221	20	11.5	0.0129224	0.014	70.68
20	1.0170	0.0041	1.0211	20	11.8	0.0129224	0.010	67.48
30	1.0160	0.0041	1.0201	20	11.8	0.0129224	0.008	64.29
60	1.0130	0.0041	1.0171	20	12.1	0.0129224	0.006	54.69
120	1.0100	0.0041	1.0141	20	12.3	0.0129224	0.004	45.10
480	1.0070	0.0041	1.0111	20	12.9	0.0129224	0.002	35.50
1440	1.0060	0.0041	1.0101	20	13.9	0.0129224	0.001	32.30

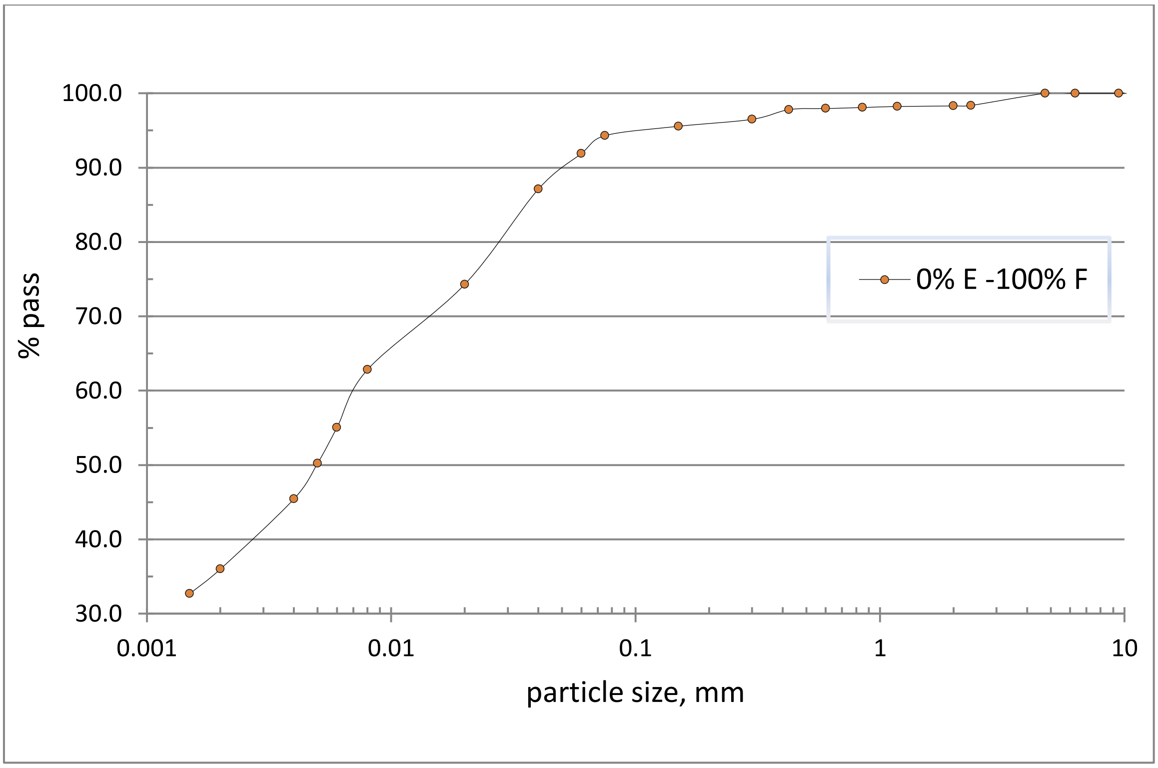


Figure B-1 Gradation curve of Fly ash

Table B-4 Compaction property of Fly ash

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	4500	4535	4585	4565	
Weight of Mould (g)	3175	3175	3175	3175	
Weight of Wet soil (g)	1325	1360	1410	1390	
Volume of Mould (cc)	944	944	944	944	
Wet density (g/cc)	1.40	1.44	1.49	1.47	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	236.9	226.9	210.4	214.0	
Weight of Dry soil + cont. (g)	204.8	193.8	177.2	178.8	
Weight of Container (g)	25.0	26	27.5	27.5	
Weight of water (g)	32.1	33.1	33.2	35.2	
Weight of Dry soil (g)	179.8	167.8	149.7	151.3	
Moisture content (%)	17.85	19.73	22.18	23.27	
Dry Density (g/cc)	1.19	1.20	1.22	1.19	
	MDD, g/cc	**1.22**	**OMC, %**		**22.3**

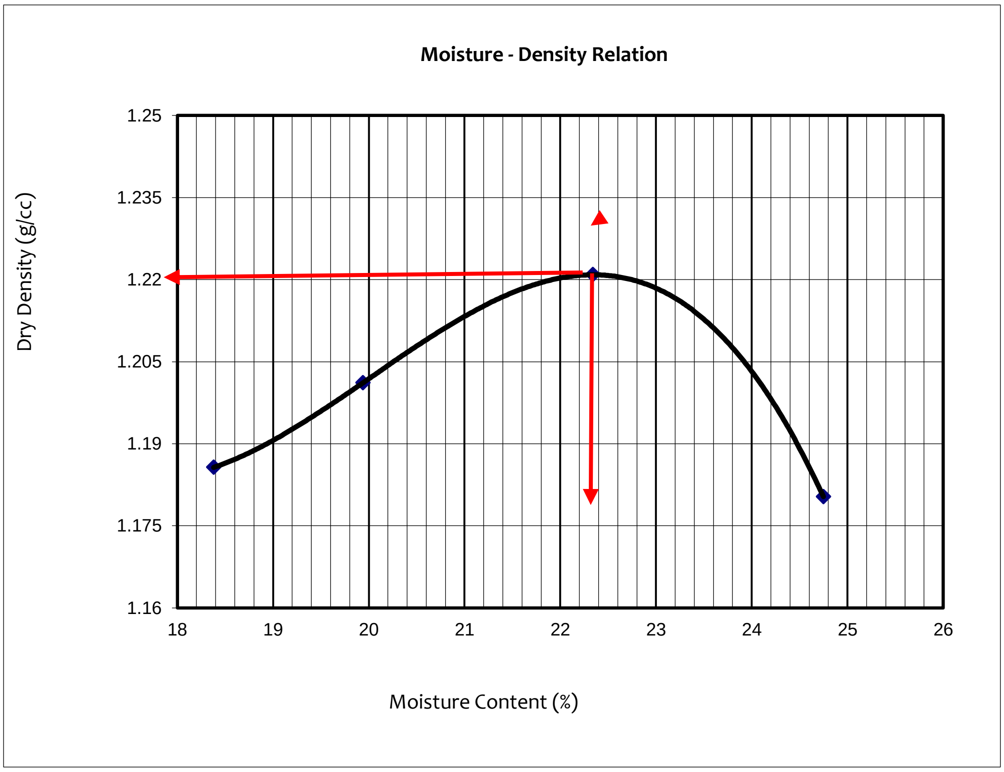


Figure B-2 compaction curve of fly ash

Table B-5 UCS test result for Fly ash

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	22
Bulk Density (gm/cm^3) =	1.492
Dry Density (gm/cm^3) =	1.220

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	3.166
0.526	4.736
0.789	7.873
1.053	9.423
1.316	10.964
1.579	14.059
1.842	15.579
2.105	17.979
2.368	20.107
2.632	22.224
2.895	24.329
3.158	26.422
3.421	28.504
3.684	30.573
3.947	32.631
4.211	34.678
4.474	36.616
4.737	38.633
5.000	40.639
5.263	43.390
5.526	45.413
5.789	47.093
6.053	46.961
6.316	46.830

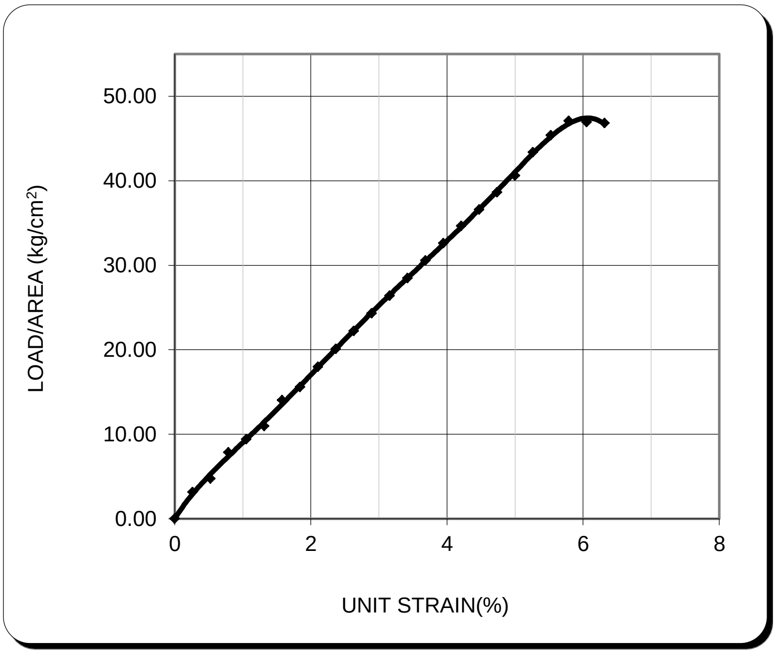


Figure B-3 UCS curve for Fly ash

Appendix- C

Test Results of Expansive soil and Fly ash Mix

Table C-1 Liquid limit and Plastic Limit test of Soil Sample TP-1 and Fly ash (0%, 10%, 20%, 30% and 40%)

		Liquid Limit			Plastic Limit	
100% E - 0% F	No. of Blows	32	28	18		
	Wt. of cont. + wet soil (g) = (w1)	43.33	42.53	42.85	21.58	20.32
	Wt. of cont. + dry soil (g.) = (w2)	35.32	34.80	34.94	19.39	18.51
	Wt. of container (g.) = (w3)	23.56	23.65	23.65	12.98	12.87
	Mass of moisture (g.) (w1-w2) = x	8.01	7.73	7.91	2.19	1.81
	Wt. of dry soil (g.) (w2-w3) = y	11.76	11.15	11.29	6.41	5.64
	Moisture Content (%) = (100x/y)	68.11	69.33	70.06	34.17	32.09
	Average	69.2			33.1	
	Plasticity Index(PI) = LL - PL	36				
		Liquid Limit			Plastic Limit	
90% E - 10% F	No. of Blows	33	28	21		
	Wt. of cont. + wet soil (g) = (w1)	42.63	42.74	41.68	20.87	20.71
	Wt. of cont. + dry soil (g.) = (w2)	35.29	36.17	33.83	19.15	19.11
	Wt. of container (g.) = (w3)	24.11	26.23	22.14	14.37	14.65
	Mass of moisture (g.) (w1-w2) = x	7.34	6.57	7.85	1.72	1.60
	Wt. of dry soil (g.) (w2-w3) = y	11.18	9.94	11.69	4.78	4.46
	Moisture Content (%) = (100x/y)	65.65	66.10	67.15	35.98	35.87
	Average	66.3			35.9	
	Plasticity Index(PI) = LL - PL	30.4				
		Liquid Limit			Plastic Limit	
80% E - 20% F	No. of Blows	33	28	20		
	Wt. of cont. + wet soil (g) = (w1)	42.39	42.56	41.08	20.41	21.74
	Wt. of cont. + dry soil (g.) = (w2)	35.23	35.23	34.27	18.53	19.39
	Wt. of container (g.) = (w3)	23.56	23.65	23.65	13.34	12.69
	Mass of moisture (g.) (w1-w2) = x	7.16	7.33	6.81	1.88	2.35
	Wt. of dry soil (g.) (w2-w3) = y	11.67	11.58	10.62	5.19	6.70
	Moisture Content (%) = (100x/y)	61.35	63.30	64.12	36.22	35.07
	Average	62.9			35.6	
	Plasticity Index(PI) = LL - PL	27.3				
		Liquid Limit			Plastic Limit	
70% E - 30% F	No. of Blows	33	28	20		
	Wt. of cont. + wet soil (g) = (w1)	41.33	42.03	41.97	20.44	20.54
	Wt. of cont. + dry soil (g.) = (w2)	35.10	36.25	34.69	18.60	18.48
	Wt. of container (g.) = (w3)	24.11	26.23	22.14	13.35	12.69
	Mass of moisture (g.) (w1-w2) = x	6.23	5.78	7.28	1.84	2.06
	Wt. of dry soil (g.) (w2-w3) = y	10.99	10.02	12.55	5.25	5.79

	Moisture Content (%) = (100x/y)	56.69	57.68	58.01	35.05	35.58
	Average	57.5			35.3	
	Plasticity Index(PI) = LL - PL	22				
60% E - 40% F	No. of Blows	33	28	20		
	Wt. of cont. + wet soil (g) = (w1)	42.39	42.56	41.08	20.41	21.74
	Wt. of cont. + dry soil (g.) = (w2)	36.06	36.17	35.14	18.59	19.37
	Wt. of container (g.) = (w3)	23.56	23.65	23.65	13.35	12.69
	Mass of moisture (g.) (w1-w2) = x	6.33	6.39	5.94	1.82	2.37
	Wt. of dry soil (g.) (w2-w3) = y	12.50	12.52	11.49	5.24	6.68
	Moisture Content (%) = (100x/y)	50.64	51.04	51.70	34.73	35.48
	Average	51.1			35.1	
	Plasticity Index(PI) = LL - PL	16.0				

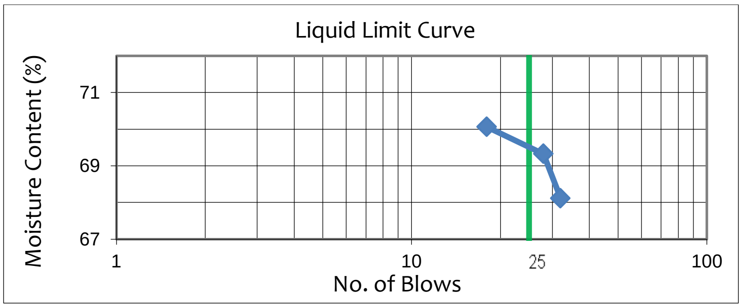


Figure C-1 Liquid Limit determination of soil sample 100% E - 0% F

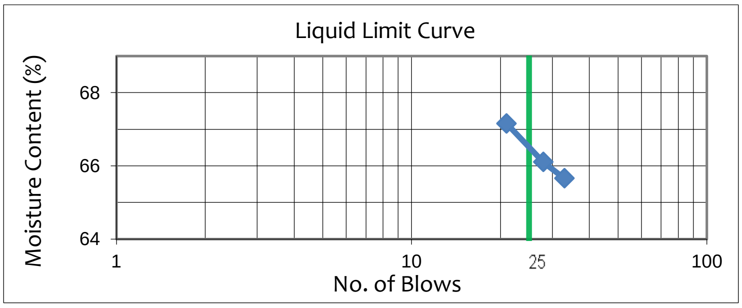


Figure C-2 Liquid Limit determination of soil sample 90% E - 10% F

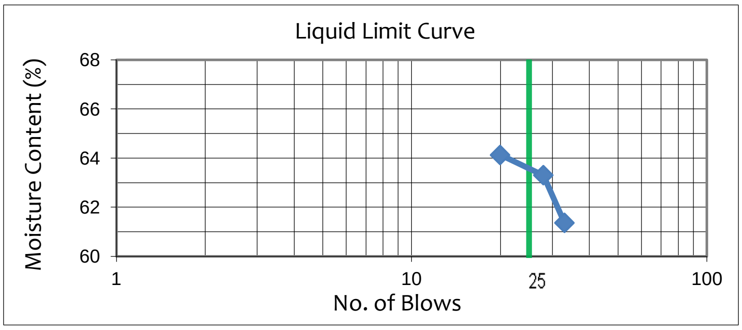


Figure C-3 Liquid Limit determination of soil sample 80% E - 20% F

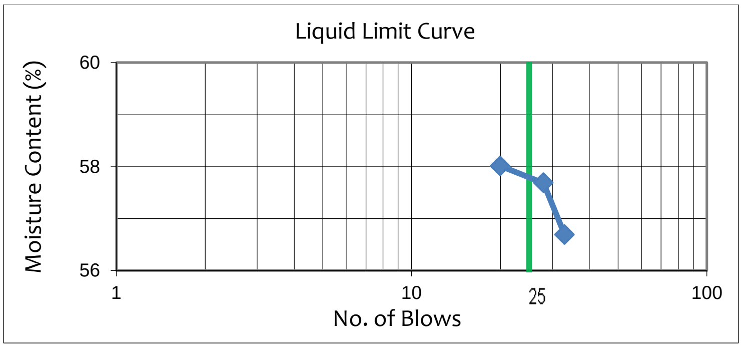


Figure C-4 Liquid Limit determination of soil sample 70% E - 30% F

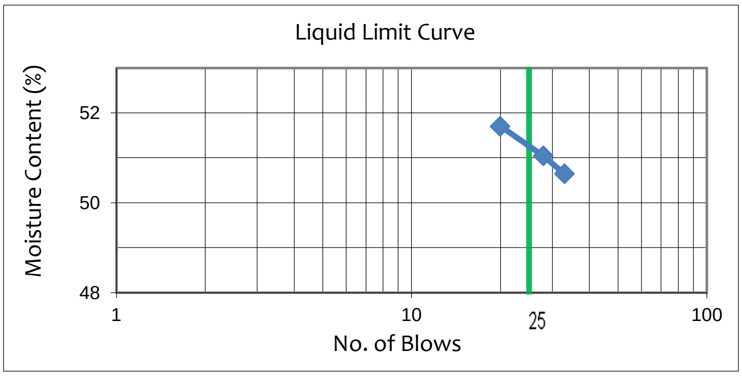


Figure C-5 Liquid Limit determination of soil sample 60% E - 40% F

Table C-2 Specific Gravity of Soil Sample1 (Pit1) and Fly ash (0%, 10%, 20%, 30% & 40%)

Determination number	100% E - 0% F		90% E - 10% F		80% E - 20% F		70% E - 30% F		60% E - 40% F	
Bottle no	6	9	12	7	3	8	2	16	1	11
Wt. of bottle + Water (g) W1	155.7	156.46	157.11	157.65	156.3	157.192	156.16	156.79	156.37	156.15
Wt. of bottle + Water+ Soil (g) W2	165.49	166.11	166.69	167.30	165.8	166.7	165.43	166.29	165.58	165.47
Wt. of soil (g) Ws	15	15	15	15	15	15	15	15	15	15
Wt. of bottle + Dry soil (g)	5.21	5.35	5.42	5.354	5.5	5.492	5.73	5.5	5.79	5.675
Specific Gravity	2.88	2.80	2.768	2.802	2.73	2.73	2.62	2.73	2.59	2.64
Average Specific Gravity	2.841		2.785		2.729		2.673		2.617	

Table C-3 Free Swell of Soil Sample1 (Pit1) and Fly ash (0%, 10%, 20%, 30% & 40%)

Sr No	Material Description	Initial Volume	Final Volume	Free Swell%
1	100% E - 0% F	10	17.5	75
2	90% E - 10% F	10	16.2	62
3	80% E - 20% F	10	15	50
4	70% E - 30% F	10	14.3	43
5	60% E - 40% F	10	13.7	37

Table C-4 Grain Size Analysis of soil sample 1(pit 1) and fly ash mix 100% E-0% F

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
63	0.0	0.0	100.0
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	4.2	0.9	99.1
2	5.9	1.3	98.7
1.18	7.7	1.7	98.3
0.85	11.6	2.5	97.5
0.6	16.2	3.5	96.5
0.425	17.6	3.8	96.2
0.3	20.7	4.5	95.5
0.15	23.0	5.0	95.0
0.075	35.0	7.6	92.4

Table C-5 Hydrometer of soil sample 1(pit 1) and fly ash mix 100% E-0% F

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^0C)	Effective depth Of Hyd. L (cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	92.4
0.5	1.0235	0.0041	1.0276	20	9.7	0.01638	0.072	89.55
1	1.0225	0.0041	1.0266	20	11.5	0.01638	0.056	86.30
2	1.0215	0.0041	1.0256	20	12.1	0.01638	0.040	83.06
4	1.0205	0.0041	1.0246	20	12.3	0.01638	0.029	79.81
6	1.0195	0.0041	1.0236	20	12.9	0.01638	0.024	76.57
10	1.0185	0.0041	1.0226	20	13.4	0.01638	0.019	73.32
20	1.0175	0.0041	1.0216	20	13.4	0.01638	0.013	70.08
30	1.0165	0.0041	1.0206	20	13.4	0.01638	0.011	66.84
60	1.0155	0.0041	1.0196	20	13.7	0.01638	0.008	63.59
120	1.0140	0.0041	1.0181	20	13.7	0.01638	0.006	58.72
480	1.0130	0.0041	1.0171	20	13.9	0.01638	0.003	55.48
1440	1.0125	0.0041	1.0166	20	14.4	0.01638	0.002	53.86

Table C-6 Grain Size Analysis of soil sample 1(pit 1) and fly ash mix 90% E-10% F

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	4.0	1.1	98.9
2	10.9	3.0	97.0
1.18	13.0	3.6	96.4
0.85	15.2	4.2	95.8
0.6	18.0	5.0	95.0
0.425	18.9	5.2	94.8
0.3	20.6	5.7	94.3
0.15	24.3	6.7	93.3
0.075	26.9	7.4	92.6

Table C-7 Hydrometer of soil sample 1(pit 1) and fly ash mix 90% E-10% F

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L(cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	81.45
0.5	1.0240	0.0041	1.0281	20	9.7	0.01638	0.060	91.17
1	1.0230	0.0041	1.0271	20	11.5	0.01638	0.043	87.92
2	1.0220	0.0041	1.0261	20	12.1	0.01638	0.030	84.68
4	1.0210	0.0041	1.0251	20	12.3	0.01638	0.021	81.44
6	1.0200	0.0041	1.0241	20	12.9	0.01638	0.018	78.19
10	1.0190	0.0041	1.0231	20	13.4	0.01638	0.014	74.95
20	1.0170	0.0041	1.0211	20	13.4	0.01638	0.010	68.46
30	1.0150	0.0041	1.0191	20	13.4	0.01638	0.008	61.97
60	1.0137	0.0041	1.0178	20	13.7	0.01638	0.006	57.64
120	1.0122	0.0041	1.0163	20	13.7	0.01638	0.004	52.78
480	1.0120	0.0041	1.0161	20	13.9	0.01638	0.002	52.24
1440	1.0118	0.0041	1.0159	20	14.4	0.01638	0.001	51.70

Table C-8 Grain Size Analysis of soil sample 1(pit 1) and fly ash mix 80% E-20% F

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	5.7	1.5	98.5
2	4.3	1.1	98.9
1.18	13.8	3.6	96.4
0.85	16.1	4.2	95.8
0.6	24.2	6.3	93.7
0.425	28.3	7.4	92.6
0.3	27.6	7.2	92.8
0.15	20.3	5.3	94.7
0.075	27.7	7.2	92.8

Table C-9 Hydrometer of soil sample 1(pit 1) and fly ash mix 80% E-20% F

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L(cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	92.8
0.5	1.0238	0.0041	1.0279	20	9.7	0.01638	0.060	90.52
1	1.0230	0.0041	1.0271	20	11.5	0.01638	0.043	87.92
2	1.0220	0.0041	1.0261	20	12.1	0.01638	0.030	84.68
4	1.0210	0.0041	1.0251	20	12.3	0.01638	0.021	81.44
6	1.0200	0.0041	1.0241	20	12.9	0.01638	0.018	78.19
10	1.0190	0.0041	1.0231	20	13.4	0.01638	0.014	74.95
20	1.0180	0.0041	1.0221	20	13.4	0.01638	0.010	71.70
30	1.0170	0.0041	1.0211	20	13.4	0.01638	0.008	68.46
60	1.0150	0.0041	1.0191	20	13.7	0.01638	0.006	61.97
120	1.0130	0.0041	1.0171	20	13.7	0.01638	0.004	55.48
480	1.0115	0.0041	1.0156	20	13.9	0.01638	0.002	50.61
1440	1.0100	0.0041	1.0141	20	14.4	0.01638	0.001	45.75

Table C-10 Grain Size Analysis f soil sample 1(pit 1) and fly ash mix 70% E-30% F

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	3.1	0.9	99.1
2	7.4	2.1	97.9
1.18	11.6	3.3	96.7
0.85	15.1	4.3	95.7
0.6	15.9	4.6	95.4
0.425	19.0	5.5	94.5
0.3	20.8	6.0	94.0
0.15	21.1	6.1	93.9
0.075	23.9	6.9	93.1

Table C-11 Hydrometer of soil sample 1(pit 1) and fly ash mix 70% E-30% F

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L (cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	93.1
0.5	1.0238	0.0041	1.0279	20	9.7	0.01638	0.060	90.52
1	1.0225	0.0041	1.0266	20	11.5	0.01638	0.043	86.30
2	1.0212	0.0041	1.0253	20	12.1	0.01638	0.030	82.08
4	1.0199	0.0041	1.0240	20	12.3	0.01638	0.021	77.87
6	1.0186	0.0041	1.0227	20	12.9	0.01638	0.018	73.65
10	1.0173	0.0041	1.0214	20	13.4	0.01638	0.014	69.43
20	1.0160	0.0041	1.0201	20	13.4	0.01638	0.010	65.21
30	1.0147	0.0041	1.0188	20	13.4	0.01638	0.008	61.00
60	1.0134	0.0041	1.0175	20	13.7	0.01638	0.006	56.78
120	1.0121	0.0041	1.0162	20	13.7	0.01638	0.004	52.56
480	1.0111	0.0041	1.0152	20	13.9	0.01638	0.002	49.32
1440	1.0095	0.0041	1.0136	20	14.4	0.01638	0.001	44.12

Table C-12 Grain Size Analysis of soil sample 1(pit 1) and fly ash mix 60% E-40% F

ASTM SIEVE (MM)	CUMULATIVE WT. RETAINED (gm)	CUMULATIVE % RETAINED	% PASS
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
25	0.0	0.0	100.0
19	0.0	0.0	100.0
12.5	0.0	0.0	100.0
9.5	0.0	0.0	100.0
6.3	0.0	0.0	100.0
4.75	0.0	0.0	100.0
2.36	2.8	0.8	99.2
2	5.8	1.7	98.3
1.18	10.0	3.0	97.0
0.85	12.5	3.8	96.2
0.6	13.6	4.1	95.9
0.425	17.3	5.2	94.8
0.3	19.2	5.8	94.2
0.15	19.6	5.9	94.1
0.075	22.3	6.7	93.3

Table C-13 Hydrometer of soil sample 1(pit 1) and fly ash mix 60% E-40% F

Elapsed Time T(min)	Actual Hy.reading	Composit correction	(R) Hydro. Reading with com. Correction	Temperature (^{0}C)	Effective depth Of Hyd. L (cm)	Value of K From Table(7-3)	Diameter of soil particle D(mm)	Soil in suspension, P(%of soil finer)
							0.075	93.3
0.5	1.0240	0.0041	1.0281	20	9.7	0.01638	0.060	91.17
1	1.0225	0.0041	1.0266	20	11.5	0.01638	0.043	86.30
2	1.0215	0.0041	1.0256	20	12.1	0.01638	0.030	83.06
4	1.0200	0.0041	1.0241	20	12.3	0.01638	0.021	78.19
6	1.0188	0.0041	1.0229	20	12.9	0.01638	0.018	74.14
10	1.0175	0.0041	1.0216	20	13.4	0.01638	0.014	69.92
20	1.0162	0.0041	1.0203	20	13.4	0.01638	0.010	65.70
30	1.0149	0.0041	1.0190	20	13.4	0.01638	0.008	61.48
60	1.0136	0.0041	1.0177	20	13.7	0.01638	0.006	57.26
120	1.0123	0.0041	1.0164	20	13.7	0.01638	0.004	53.05
480	1.0106	0.0041	1.0147	20	13.9	0.01638	0.002	47.69
1440	1.0097	0.0041	1.0111	20	14.4	0.01638	0.001	42.7

Table C-14 Compaction property of soil sample 1(pit 1) and fly ash mix 100% E-0% F

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	4673	4721	4780	4755	
Weight of Mould (g)	3175	3175	3175	3175	
Weight of Wet soil (g)	1498	1546	1605	1580	
Volume of Mould (cc)	944	944	944	944	
Wet density (g/cc)	1.59	1.64	1.70	1.67	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	237.2	260.0	220.0	269.0	
Weight of Dry soil + cont. (g)	199.7	215.7	179.0	217.3	
Weight of Container (g)	27.3	30	26.6	34.1	
Weight of water (g)	37.5	44.3	41	51.7	
Weight of Dry soil (g)	172.4	185.7	152.4	183.2	
Moisture content (%)	21.75	23.86	26.90	28.22	
Dry Density (g/cc)	1.30	1.32	1.34	1.31	
	MDD, g/cc	**1.34**	**OMC, %**		**26.9**

Figure C-6 compaction curve of soil sample 1(pit 1) and fly ash mix 100% E-0% F

Table C-15 Compaction property of soil sample 1(pit 1) and fly ash mix 90% E-10% F

Trial No.		1	2	3	4	
Weight of Mould + Wet soil (g)		4648	4700	4750	4742	
Weight of Mould (g)		3175	3175	3175	3175	
Weight of Wet soil (g)		1473	1525	1575	1567	
Volume of Mould (cc)		944	944	944	944	
Wet density (g/cc)		1.56	1.62	1.67	1.66	
Moisture Content Determination						
Weight of Wet soil + cont. (g)		260.9	263.6	259.5	282.0	
Weight of Dry soil + cont. (g)		220.2	218.5	212.2	227.5	
Weight of Container (g)		28.0	26.5	29.0	32.5	
Weight of water (g)		40.7	45.1	47.3	54.5	
Weight of Dry soil (g)		192.2	192	183.2	195	
Moisture content (%)		21.18	23.49	25.82	27.95	
Dry Density (g/cc)		1.29	1.31	1.33	1.30	
		MDD, g/cc	**1.33**	**OMC, %**		**25.8**

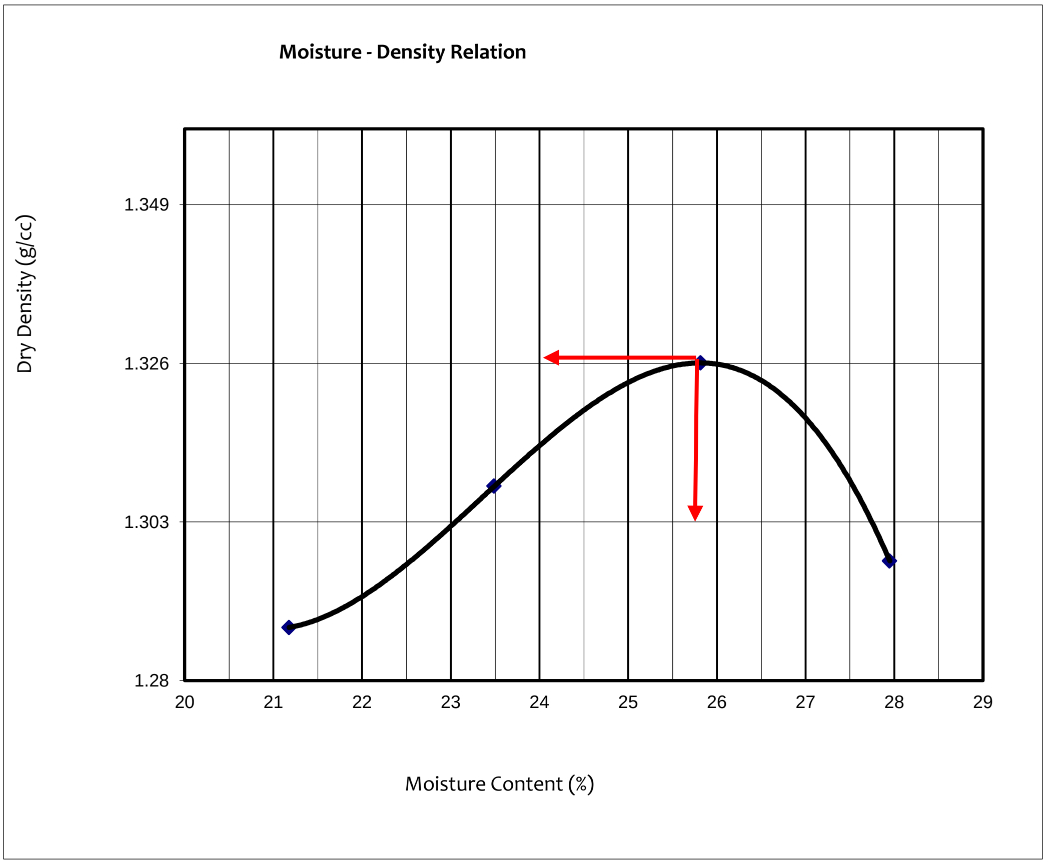


Figure C-7 compaction curve of soil sample 1(pit 1) and fly ash mix 90% E-10% F

Table C-16 Compaction property of soil sample 1(pit 1) and fly ash mix 80% E-20% F

Trial No.		1	2	3	4	
Weight of Mould + Wet soil (g)		4643	4690	4735	4726	
Weight of Mould (g)		3175	3175	3175	3175	
Weight of Wet soil (g)		1468	1515	1560	1551	
Volume of Mould (cc)		944	944	944	944	
Wet density (g/cc)		1.56	1.60	1.65	1.64	
Moisture Content Determination						
Weight of Wet soil + cont. (g)		242.5	232.5	276.0	296.5	
Weight of Dry soil + cont. (g)		204.0	193.5	227.0	238.5	
Weight of Container (g)		25.0	26	32.9	30.0	
Weight of water (g)		38.5	39	49	58	
Weight of Dry soil (g)		179	167.5	194.1	208.5	
Moisture content (%)		21.51	23.28	25.24	27.82	
Dry Density (g/cc)		1.28	1.30	1.32	1.29	
		MDD, g/cc	**1.32**	**OMC, %**		**25.2**

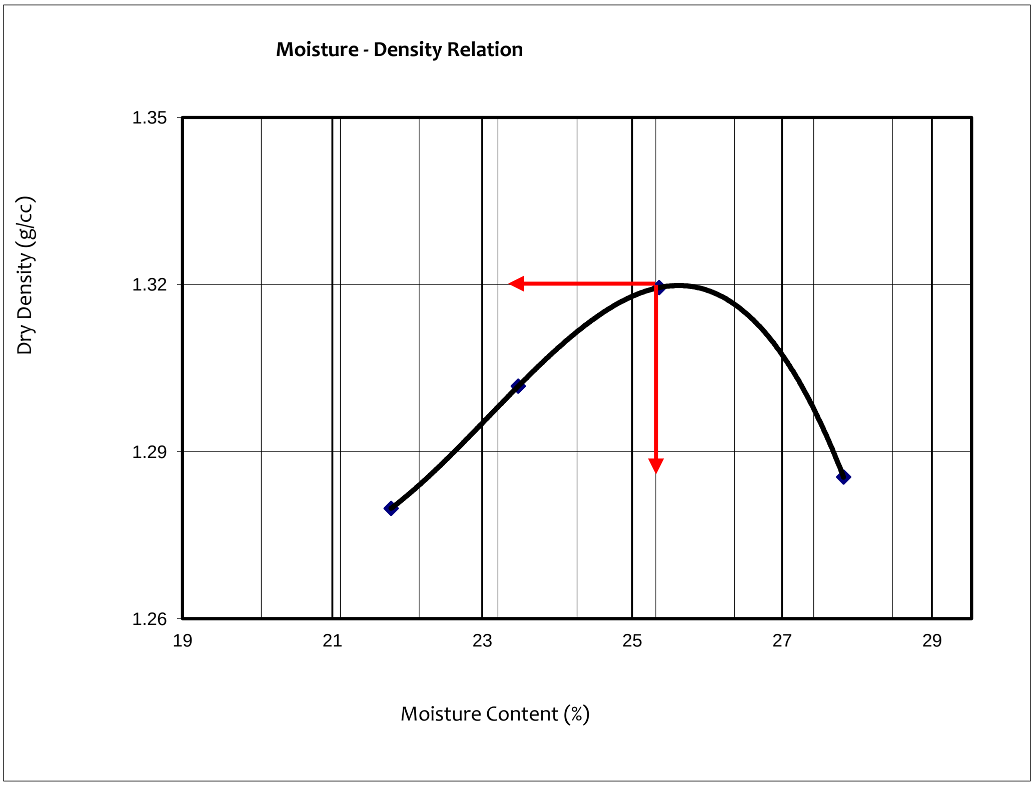


Figure C-8 compaction curve of soil sample 1(pit 1) and fly ash mix 80% E-20% F

Table C-17 Compaction property of soil sample 1(pit 1) and fly ash mix 70% E-30% F

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	4615	4665	4712	4663	
Weight of Mould (g)	3175	3175	3175	3175	
Weight of Wet soil (g)	1440	1490	1537	1488	
Volume of Mould (cc)	944	944	944	944	
Wet density (g/cc)	1.53	1.58	1.63	1.58	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	229.6	307.0	289.6	288.0	
Weight of Dry soil + cont. (g)	194.0	255	241.0	234.5	
Weight of Container (g)	26.5	30	43.5	28.7	
Weight of water (g)	35.6	52	48.6	53.5	
Weight of Dry soil (g)	167.5	225	197.5	205.8	
Moisture content (%)	21.25	23.11	24.61	26.00	
Dry Density (g/cc)	1.26	1.28	1.31	1.25	
	MDD, g/cc	**1.31**	**OMC, %**		**24.6**

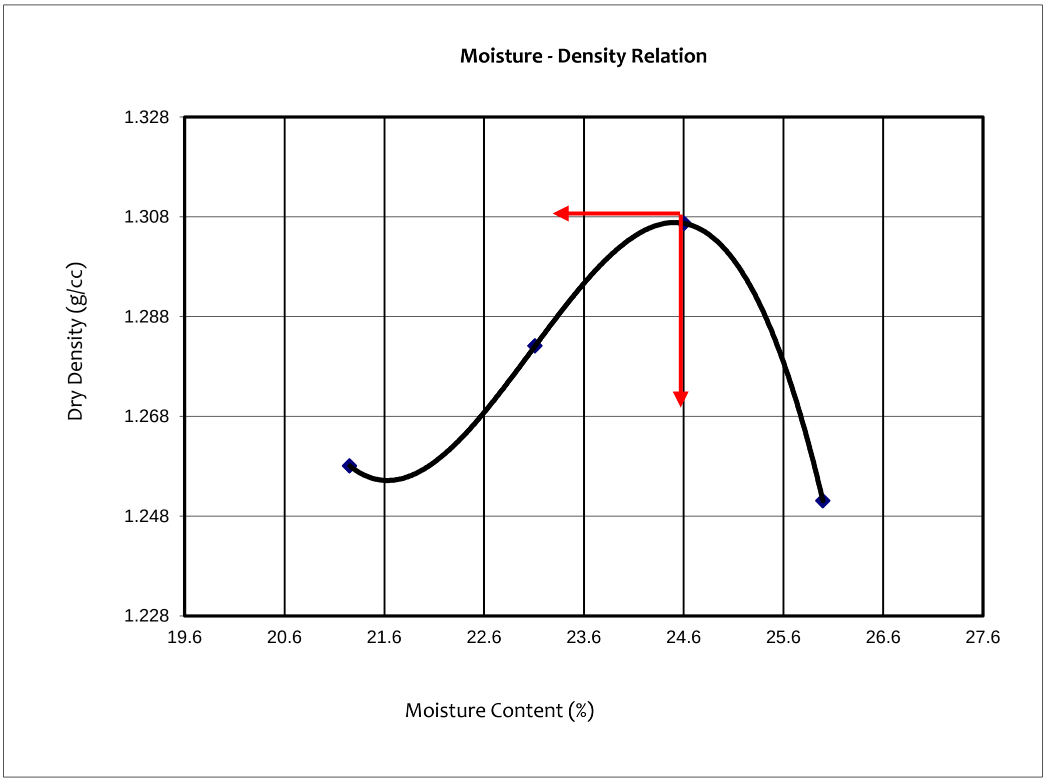


Figure C-9 compaction curve of soil sample 1(pit 1) and fly ash mix 70% E-30% F

Table C-18 Compaction property of soil sample 1(pit 1) and fly ash mix 60% E-40% F

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	4619	4650	4688	4644	
Weight of Mould (g)	3175	3175	3175	3175	
Weight of Wet soil (g)	1444	1475	1513	1469	
Volume of Mould (cc)	944	944	944	944	
Wet density (g/cc)	1.53	1.56	1.60	1.56	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	278.2	228.9	284.2	225.9	
Weight of Dry soil + cont. (g)	234.9	193	234.9	186.0	
Weight of Container (g)	28.0	30	28.0	27.0	
Weight of water (g)	43.3	35.9	49.3	39.9	
Weight of Dry soil (g)	206.9	163	206.9	159	
Moisture content (%)	20.93	22.02	23.83	25.09	
Dry Density (g/cc)	1.26	1.28	1.29	1.24	
	MDD, g/cc	**1.30**	**OMC, %**		**23.8**

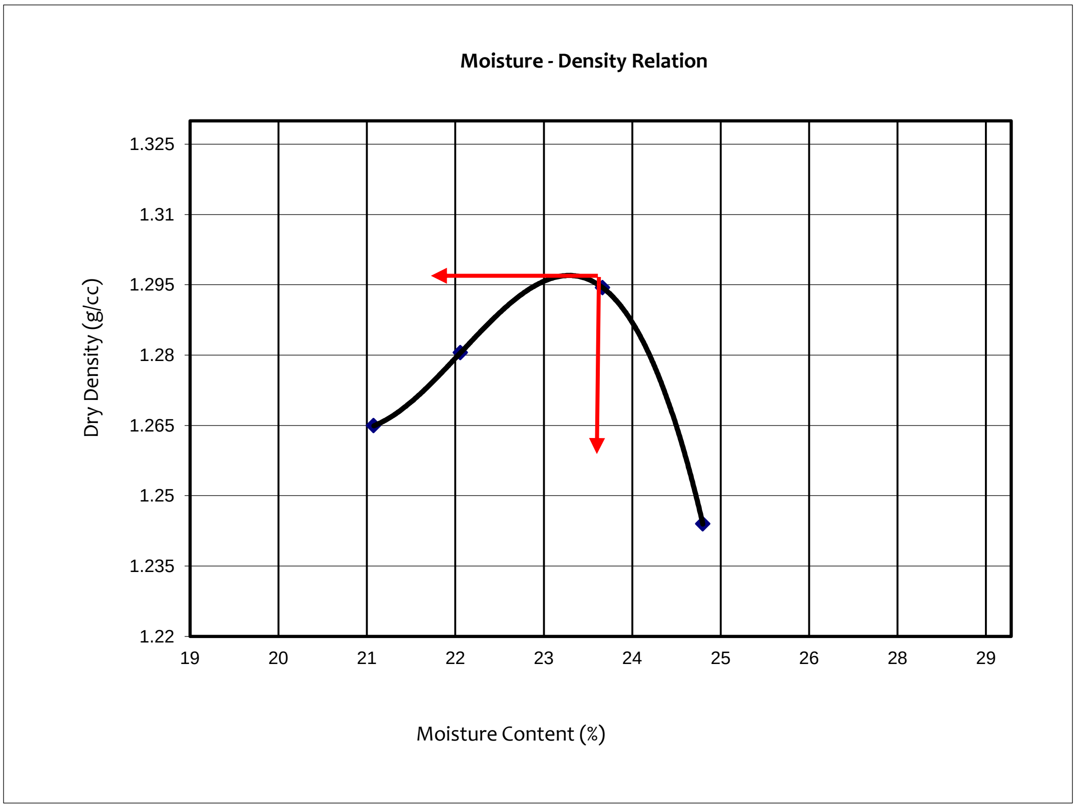


Figure C-10 compaction curve of soil sample 1(pit 1) and fly ash mix 60% E-40% F

Table C-19 CBR Value of soil sample 1(pit 1) and fly ash mix 100% E-0% F

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	0.35	11.5	11.15	9.61
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		0.6	0.1		
1.27		1.1	0.2		
1.91		1.3	0.2		
2.54	13.35	1.9	0.3	0.35	**2.6**
3.81		2.3	0.4		
5.08	20	2.7	0.5	0.50	**2.5**
7.62					

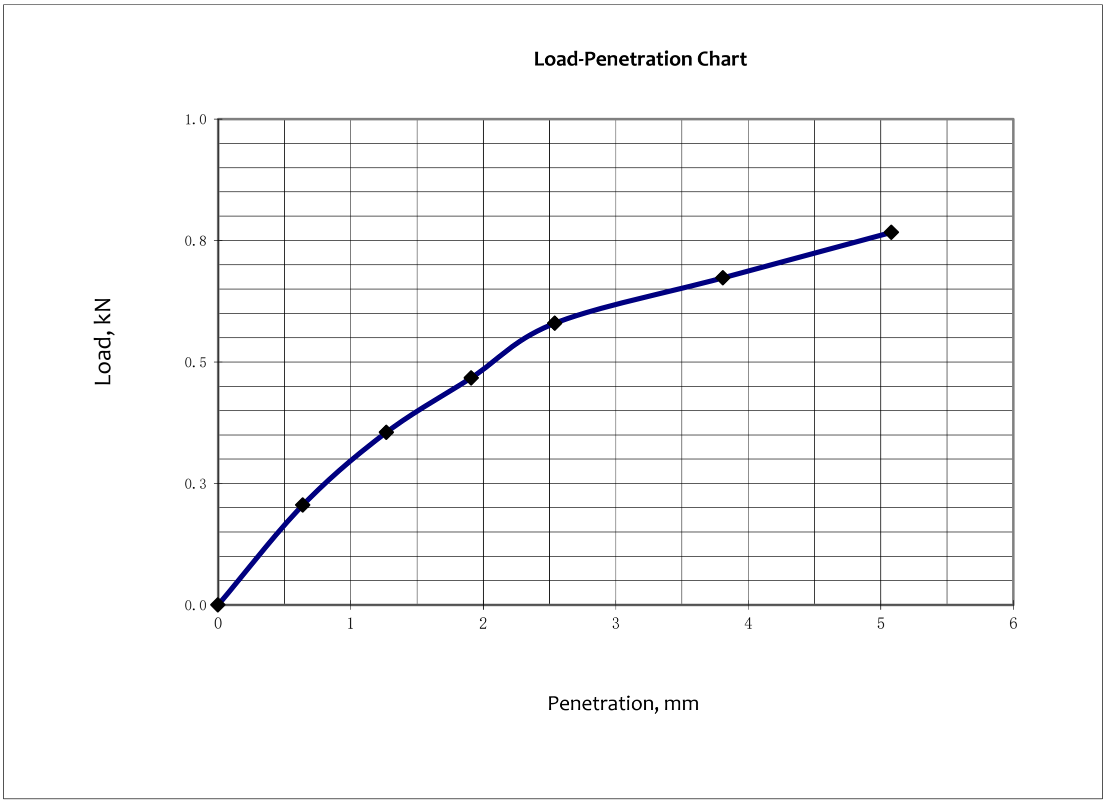


Figure C-11 Load- penetration curve of soil sample 1(pit 1) and fly ash mix 100% E-0% F

Table C-20 CBR Value of soil sample 1(pit 1) and fly ash mix 90% E-10% F

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	0.35	9.06	8.71	7.50
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		0.8	0.1		
1.27		1.2	0.2		
1.91		1.6	0.3		
2.54	13.35	2.1	0.4	0.39	**2.9**
3.81		2.3	0.4		
5.08	20	2.7	0.5	0.50	**2.5**
7.62					

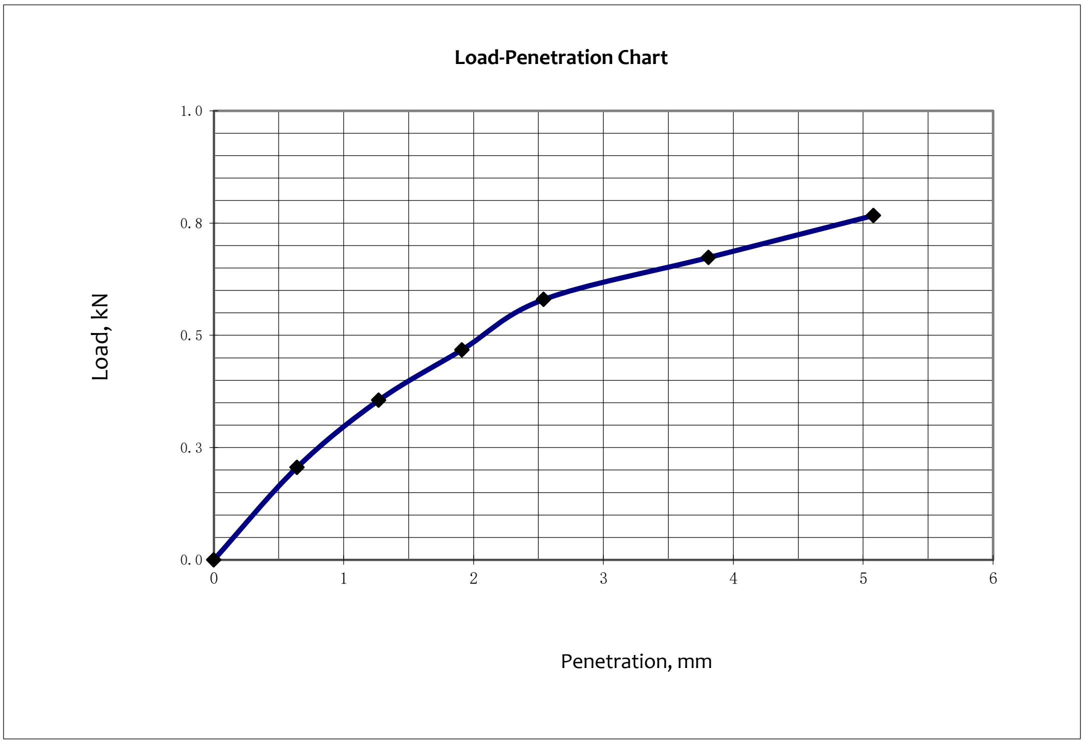


Figure C-12 Load- penetration curve of soil sample 1(pit 1) and fly ash mix 90% E-10% F

Table C-21 CBR Value of soil sample 1(pit 1) and fly ash mix 80% E-20% F

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	1.35	7.27	7.08	5.10
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		0.8	0.1		
1.27		1.5	0.3		
1.91		1.9	0.4		
2.54	13.35	2.4	0.4	0.45	**3.4**
3.81		2.7	0.5		
5.08	20	2.9	0.5	0.54	**2.7**
7.62					

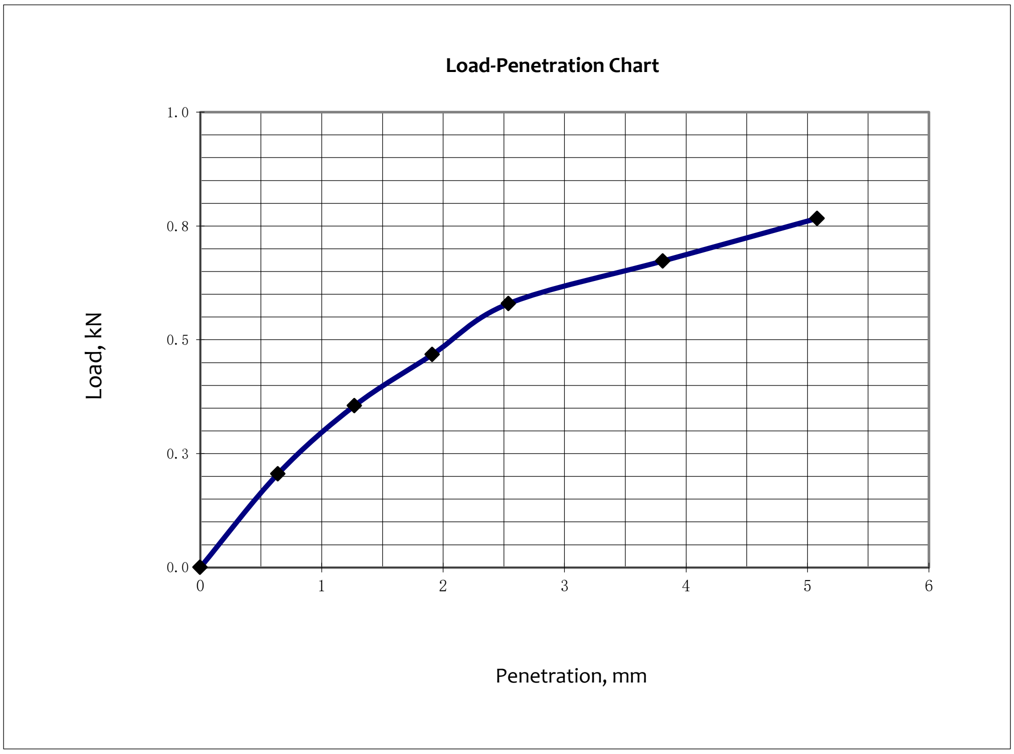


Figure C-13 Load- penetration curve of soil sample 1(pit 1) and fly ash mix 80% E-10% F

Table C-22 CBR Value of soil sample 1(pit 1) and fly ash mix 70% E-30% F

No. of Blows		**56**			
Initial Height of Sample:		Gauge reading		Swell	
		Initial	Final	mm	%
116	mm	1.45	5.97	4.52	3.90
CBR DATA (4 days Soaked)					
Ring factor		**0.187**		**kN/div**	
Penet-ration (mm)	Std load (kN)	Gauge reading	Load	Corrected CBR	
			kN	kN	%
0		0	0.0		
0.64		0.9	0.2		
1.27		1.7	0.3		
1.91		2.2	0.4		
2.54	13.35	2.8	0.5	0.52	**3.9**
3.81		2.9	0.5		
5.08	20	3.2	0.6	0.60	**3.0**
7.62					

Figure C-14 Load- penetration curve of soil sample 1(pit 1) and fly ash mix 70% E-30% F

Table C-23 CBR Value of soil sample 1(pit 1) and fly ash mix 60% E-40% F

<table>
<tr><td colspan="2">No. of Blows</td><td colspan="4">56</td></tr>
<tr><td colspan="2" rowspan="2">Initial Height of Sample:</td><td colspan="2">Gauge reading</td><td colspan="2">Swell</td></tr>
<tr><td>Initial</td><td>Final</td><td>mm</td><td>%</td></tr>
<tr><td>116</td><td>mm</td><td>1.5</td><td>5.21</td><td>3.71</td><td>3.20</td></tr>
<tr><td colspan="4">CBR DATA (4 days Soaked)</td><td></td><td></td></tr>
<tr><td colspan="2">Ring factor</td><td colspan="2">0.187</td><td colspan="2">kN/div</td></tr>
<tr><td rowspan="2">Penet-ration (mm)</td><td rowspan="2">Std load (kN)</td><td rowspan="2">Gauge reading</td><td>Load</td><td colspan="2">Corrected CBR</td></tr>
<tr><td>kN</td><td>kN</td><td>%</td></tr>
<tr><td>0</td><td></td><td>0</td><td>0.0</td><td></td><td></td></tr>
<tr><td>0.64</td><td></td><td>1.1</td><td>0.2</td><td></td><td></td></tr>
<tr><td>1.27</td><td></td><td>1.9</td><td>0.4</td><td></td><td></td></tr>
<tr><td>1.91</td><td></td><td>2.5</td><td>0.5</td><td></td><td></td></tr>
<tr><td>2.54</td><td>13.35</td><td>3.1</td><td>0.6</td><td>0.58</td><td>4.3</td></tr>
<tr><td>3.81</td><td></td><td>3.6</td><td>0.7</td><td></td><td></td></tr>
<tr><td>5.08</td><td>20</td><td>4.1</td><td>0.8</td><td>0.77</td><td>3.8</td></tr>
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</table>

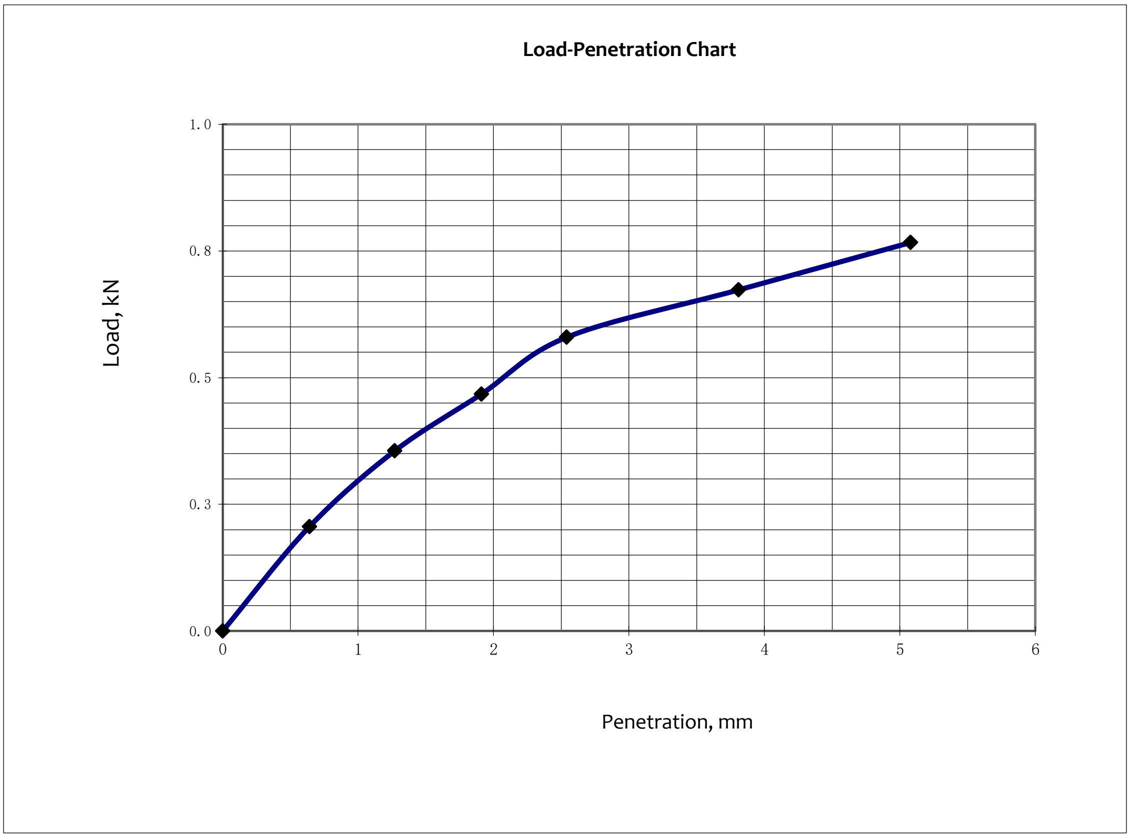

Figure C-15 Load- penetration curve of soil sample 1(pit 1) and fly ash mix 60% E-40% F

Table C-24 UCS test result for soil sample 1(pit 1) and fly ash mix 100% E-0% F

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	27
Bulk Density (gm/cm^3) =	1.701
Dry Density (gm/cm^3) =	1.340

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	12.664
0.526	18.945
0.789	26.768
1.053	29.838
1.316	36.024
1.579	43.738
1.842	51.411
2.105	55.934
2.368	61.982
2.632	66.451
2.895	72.436
3.158	76.851
3.421	82.774
3.684	91.720
3.947	97.567
4.211	104.902
4.474	111.899
4.737	116.115
5.000	118.801
5.263	119.972
5.526	121.134
5.789	120.675
6.053	120.338
6.316	118.537

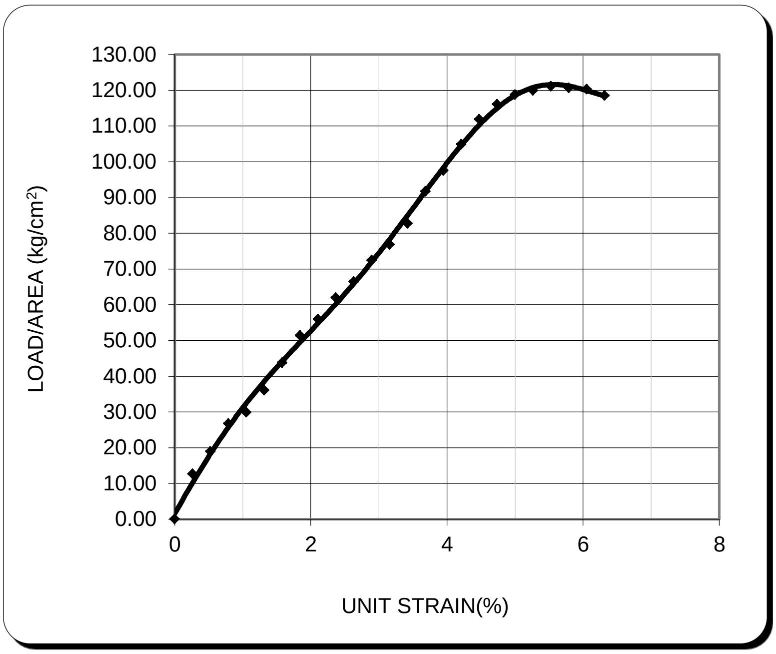

Figure C-16 UCS curve for soil sample 1(pit 1) and fly ash mix 100% E-0% F

Table C-25 UCS test result for soil sample 1(pit 1) and fly ash mix 90% E-10% F

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	25.8
Bulk Density (gm/cm^3) =	1.67
Dry Density (gm/cm^3) =	1.33

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	12.664
0.526	20.524
0.789	29.918
1.053	36.120
1.316	43.855
1.579	51.549
1.842	57.642
2.105	63.703
2.368	73.518
2.632	81.046
2.895	88.533
3.158	95.978
3.421	103.382
3.684	110.743
3.947	118.063
4.211	125.341
4.474	130.045
4.737	134.210
5.000	135.343
5.263	136.468
5.526	137.585
5.789	135.391
6.053	133.546
6.316	131.708
6.579	130.987

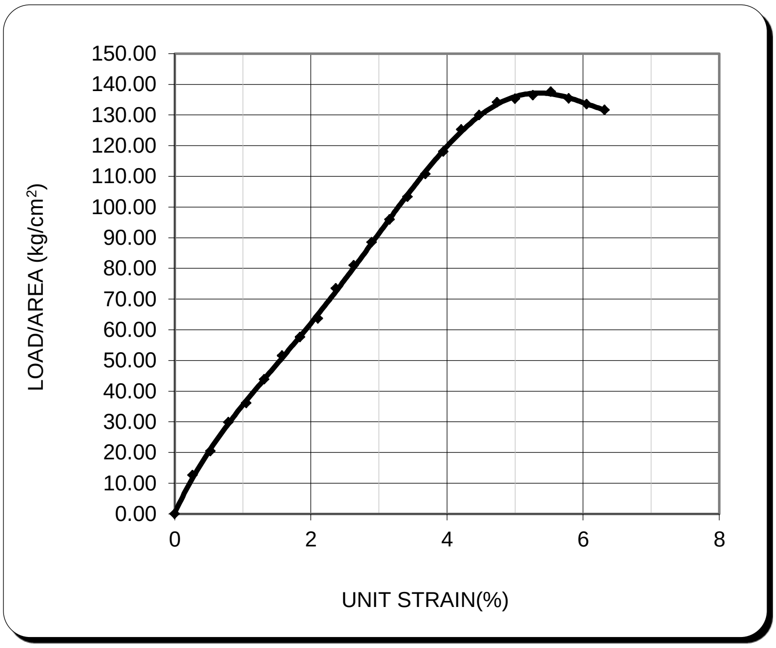

Figure C-17 UCS curve for soil sample 1(pit 1) and fly ash mix 90% E-10% F

Table C-26 UCS test result for soil sample 1(pit 1) and fly ash mix 80% E-20% F

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	25
Bulk Density (gm/cm^3) =	1.652
Dry Density (gm/cm^3) =	1.320

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	14.247
0.526	23.682
0.789	33.067
1.053	45.543
1.316	56.385
1.579	67.065
1.842	77.836
2.105	88.548
2.368	99.201
2.632	109.795
2.895	120.331
3.158	130.808
3.421	141.226
3.684	151.586
3.947	161.886
4.211	165.714
4.474	173.897
4.737	180.958
5.000	187.977
5.263	188.956
5.526	189.927
5.789	186.899
6.053	184.909
6.316	184.391
6.579	183.382

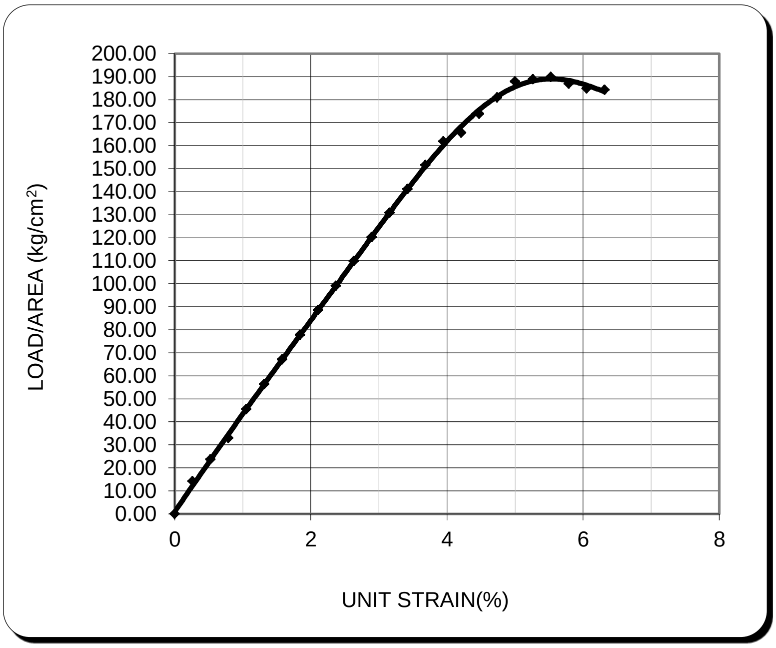

Figure C-18 UCS curve for soil sample 1(pit 1) and fly ash mix 80% E-20% F

Table C-27 UCS test result for soil sample 1(pit 1) and fly ash mix 70% E-30% F

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	25
Bulk Density (gm/cm^3) =	1.632
Dry Density (gm/cm^3) =	1.310

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	15.830
0.526	26.839
0.789	39.365
1.053	51.824
1.316	64.217
1.579	78.104
1.842	89.913
2.105	102.324
2.368	114.667
2.632	126.941
2.895	139.148
3.158	151.287
3.421	163.357
3.684	175.360
3.947	187.295
4.211	199.161
4.474	214.725
4.737	224.689
5.000	234.595
5.263	238.444
5.526	239.278
5.789	235.463
6.053	234.805
6.316	234.148
6.579	232.866

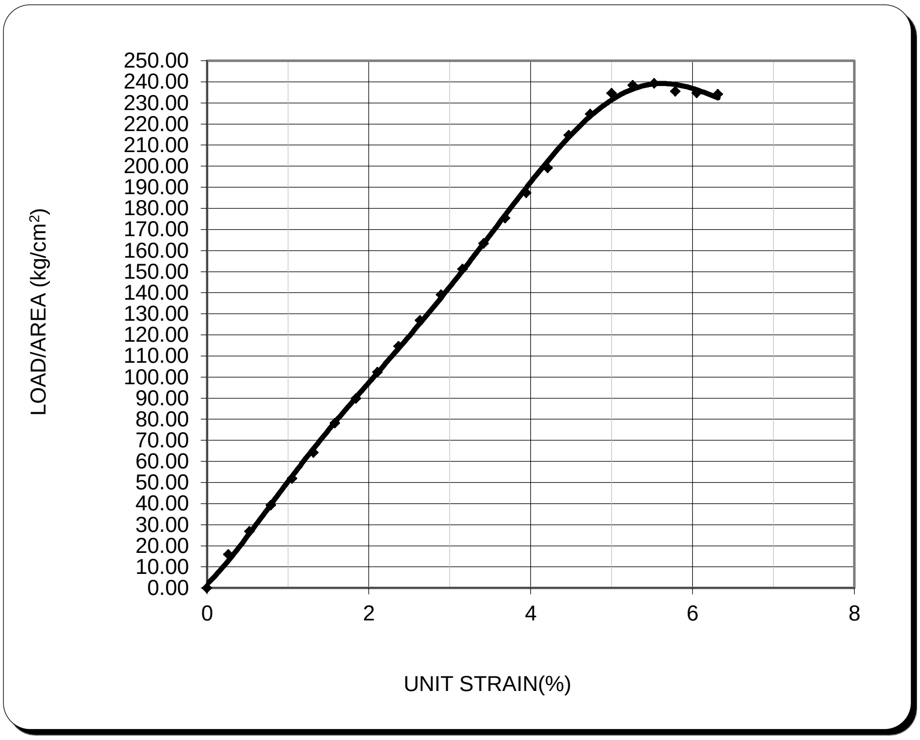

Figure C-19 UCS curve for soil sample 1(pit 1) and fly ash mix 70% E-30% F

Table C-28 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	15.830
0.526	28.418
0.789	39.365
1.053	53.395
1.316	68.915
1.579	80.812
1.842	99.706
2.105	113.422
2.368	130.162
2.632	145.265
2.895	158.743
3.158	172.146
3.421	183.941
3.684	197.198
3.947	208.855
4.211	226.527
4.474	241.944
4.737	250.325
5.000	254.145
5.263	256.440
5.526	257.223
5.789	253.123
6.053	250.948
6.316	250.245
6.579	248.875

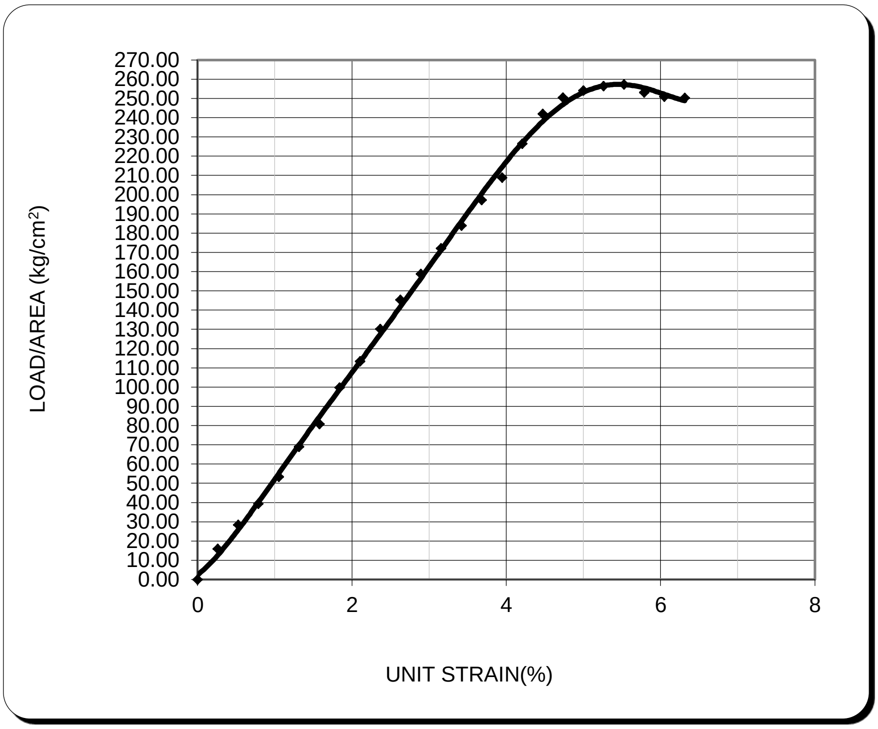

Figure C-20 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F

Appendix- D

Test Results of Wetting – Drying cycle

Table D-1 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 0-day cured time

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	15.830
0.526	28.418
0.789	39.365
1.053	53.395
1.316	68.915
1.579	80.812
1.842	99.706
2.105	113.422
2.368	130.162
2.632	145.265
2.895	158.743
3.158	172.146
3.421	183.941
3.684	197.198
3.947	208.855
4.211	226.527
4.474	241.944
4.737	250.325
5.000	254.145
5.263	256.440
5.526	257.223
5.789	253.123
6.053	250.948
6.316	250.245
6.579	248.875

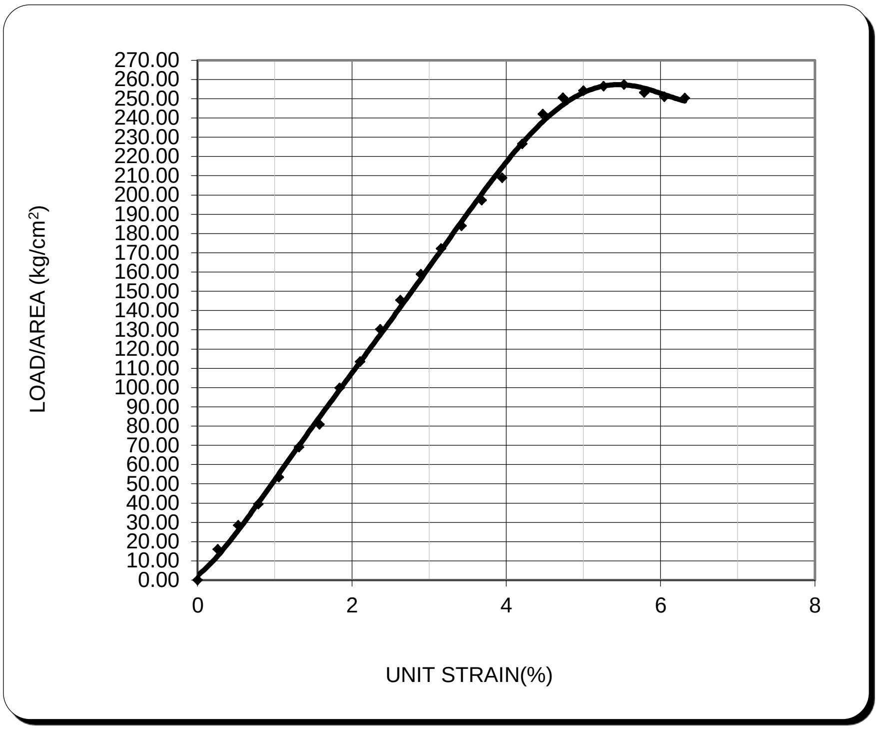

Figure D-1 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 0-day cured time

Table D-2 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 7-day cured time

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	18.996
0.526	31.576
0.789	48.813
1.053	65.958
1.316	81.289
1.579	97.161
1.842	112.948
2.105	128.649
2.368	144.263
2.632	159.792
2.895	175.234
3.158	190.591
3.421	205.861
3.684	221.045
3.947	236.144
4.211	251.156
4.474	265.382
4.737	280.183
5.000	294.898
5.263	309.528
5.526	324.071
5.789	335.535
6.053	333.130
6.316	330.734
6.579	327.467

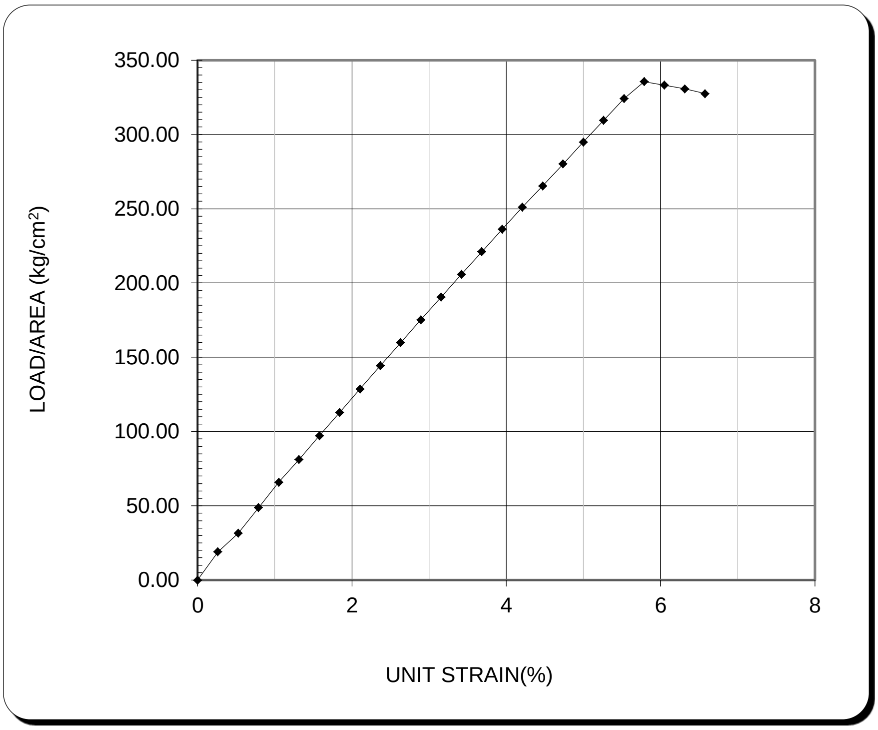

Figure D-2 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 7-day cured time

Table D-3 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 14-day cured time

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	20.579
0.526	41.048
0.789	58.261
1.053	78.522
1.316	97.734
1.579	116.844
1.842	135.849
2.105	154.751
2.368	173.550
2.632	192.245
2.895	210.836
3.158	229.323
3.421	247.708
3.684	265.988
3.947	284.165
4.211	302.238
4.474	319.365
4.737	337.185
5.000	357.908
5.263	363.440
5.526	361.907
5.789	354.666
6.053	352.208
6.316	349.758
6.579	346.388

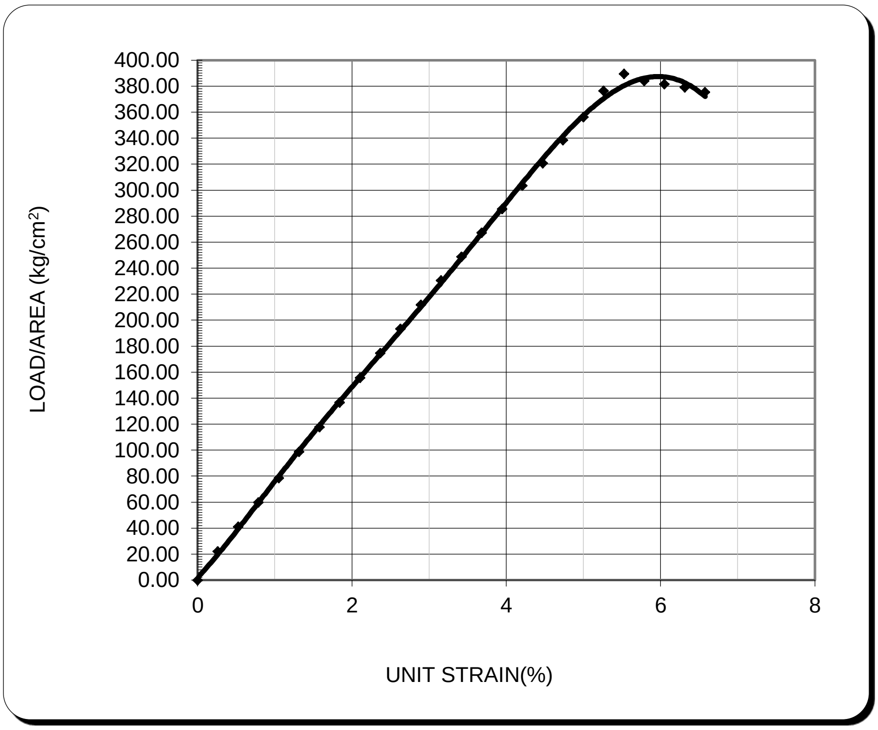

Figure D-3 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 14-day cured time

Table D-4 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 28-day cured time

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	22.161
0.526	41.048
0.789	59.835
1.053	78.522
1.316	98.674
1.579	117.677
1.842	136.725
2.105	155.669
2.368	174.509
2.632	193.245
2.895	211.878
3.158	230.407
3.421	248.832
3.684	267.153
3.947	285.370
4.211	303.484
4.474	320.647
4.737	338.506
5.000	356.261
5.263	376.412
5.526	389.574
5.789	384.099
6.053	381.559
6.316	379.027
6.579	375.496

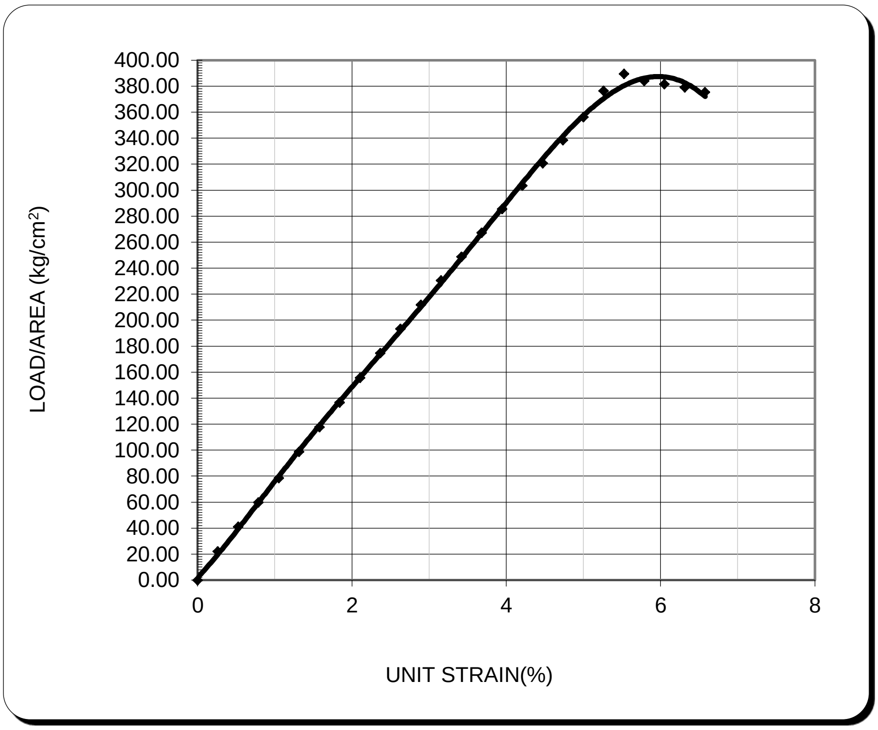

Figure D-4 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 28-day cured time

Table D-5 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 0 cycle

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	22.161
0.526	41.048
0.789	59.835
1.053	78.522
1.316	98.674
1.579	117.677
1.842	136.725
2.105	155.669
2.368	174.509
2.632	193.245
2.895	211.878
3.158	230.407
3.421	248.832
3.684	267.153
3.947	285.370
4.211	303.484
4.474	320.647
4.737	338.506
5.000	356.261
5.263	376.412
5.526	389.574
5.789	384.099
6.053	381.559
6.316	379.027
6.579	375.496

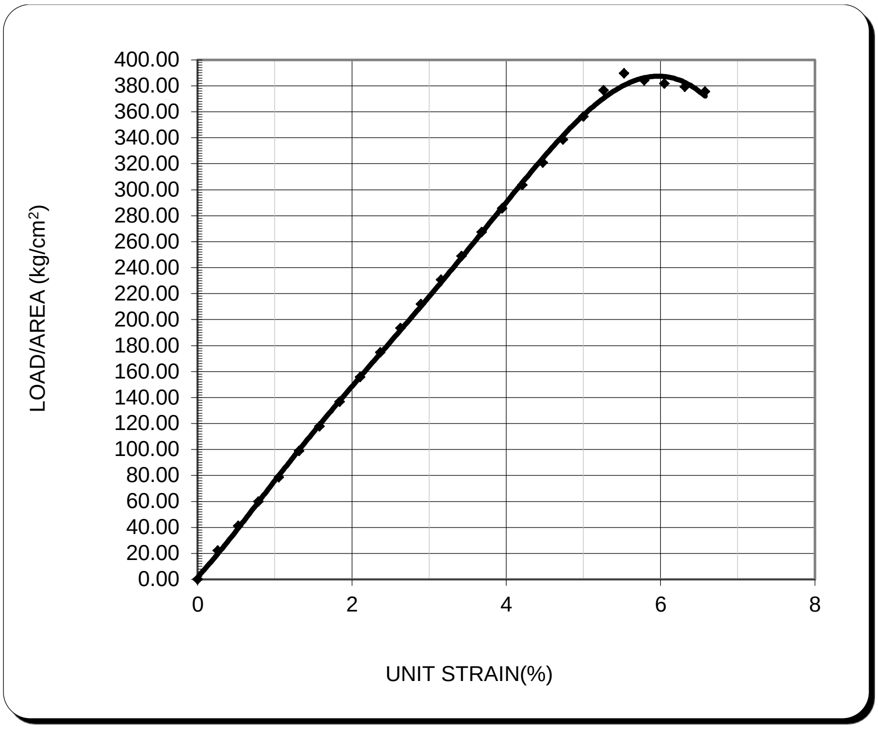

Figure D-5 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 0 cycle

Table D-6 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 1st cycle

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	18.996
0.526	37.891
0.789	51.962
1.053	69.099
1.316	87.710
1.579	104.139
1.842	120.997
2.105	137.764
2.368	154.439
2.632	171.021
2.895	187.512
3.158	203.911
3.421	220.219
3.684	236.434
3.947	252.557
4.211	268.589
4.474	283.780
4.737	299.586
5.000	315.300
5.263	330.923
5.526	346.454
5.789	354.911
6.053	353.676
6.316	351.221
6.579	347.843

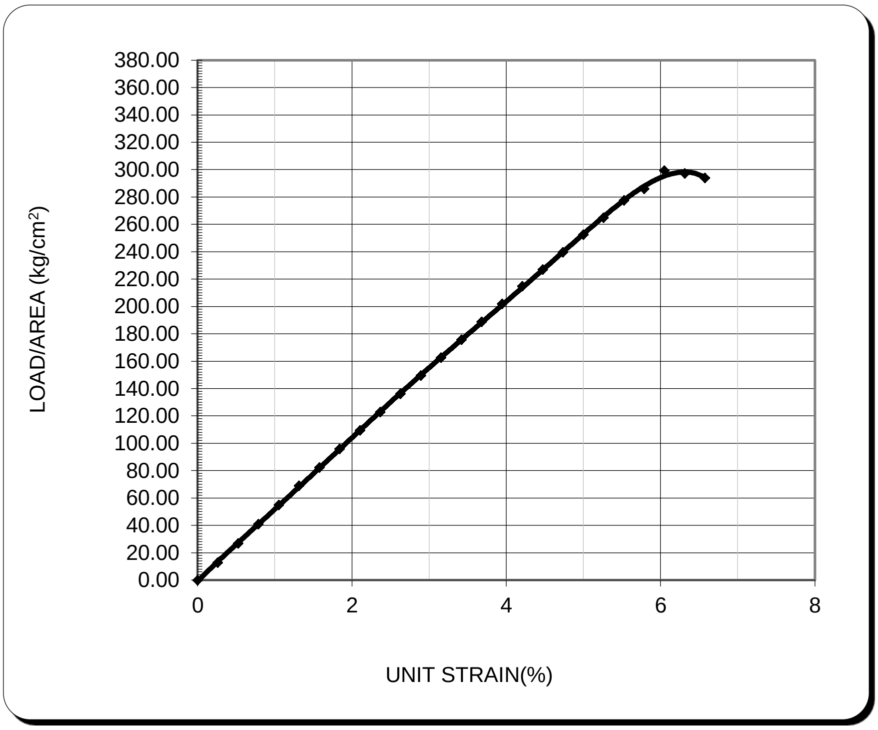

Figure D-6 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 1st cycle

Table D-7 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 2nd cycle

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	17.413
0.526	34.733
0.789	47.238
1.053	64.388
1.316	80.036
1.579	95.599
1.842	111.079
2.105	126.473
2.368	141.784
2.632	157.010
2.895	172.152
3.158	187.209
3.421	202.182
3.684	217.071
3.947	231.875
4.211	246.595
4.474	260.543
4.737	275.056
5.000	289.485
5.263	303.829
5.526	318.090
5.789	327.662
6.053	325.792
6.316	323.416
6.579	320.190

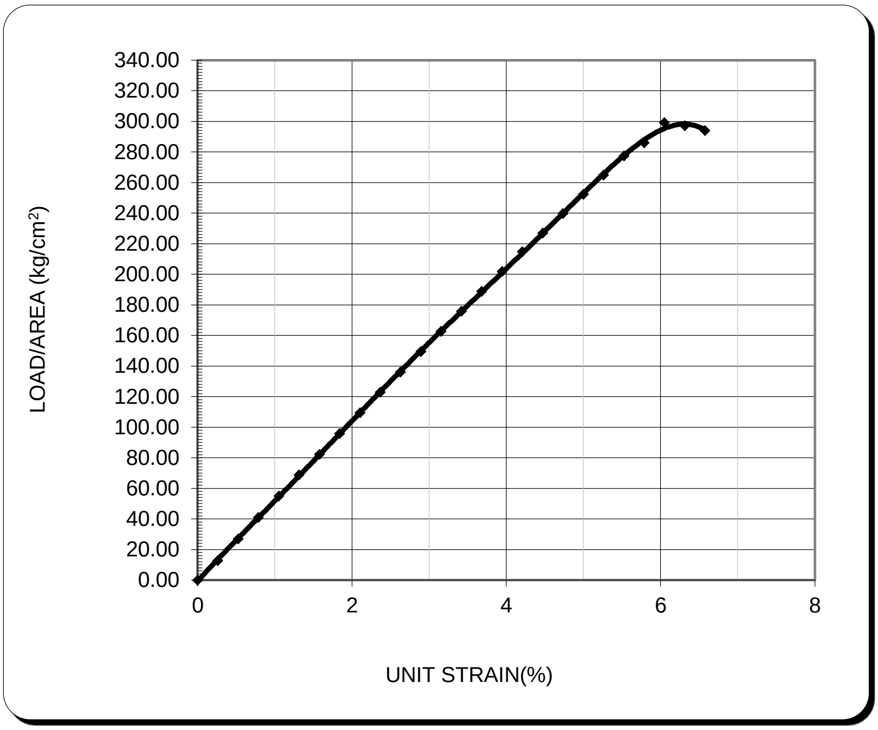

Figure D-7 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 2nd cycle

Table D-8 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 3rd cycle

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	15.830
0.526	29.997
0.789	44.089
1.053	58.891
1.316	73.301
1.579	87.633
1.842	101.887
2.105	116.063
2.368	130.162
2.632	144.183
2.895	158.127
3.158	171.993
3.421	185.781
3.684	199.491
3.947	213.124
4.211	226.679
4.474	239.524
4.737	252.888
5.000	266.175
5.263	283.434
5.526	299.097
5.789	307.574
6.053	313.841
6.316	311.709
6.579	308.547

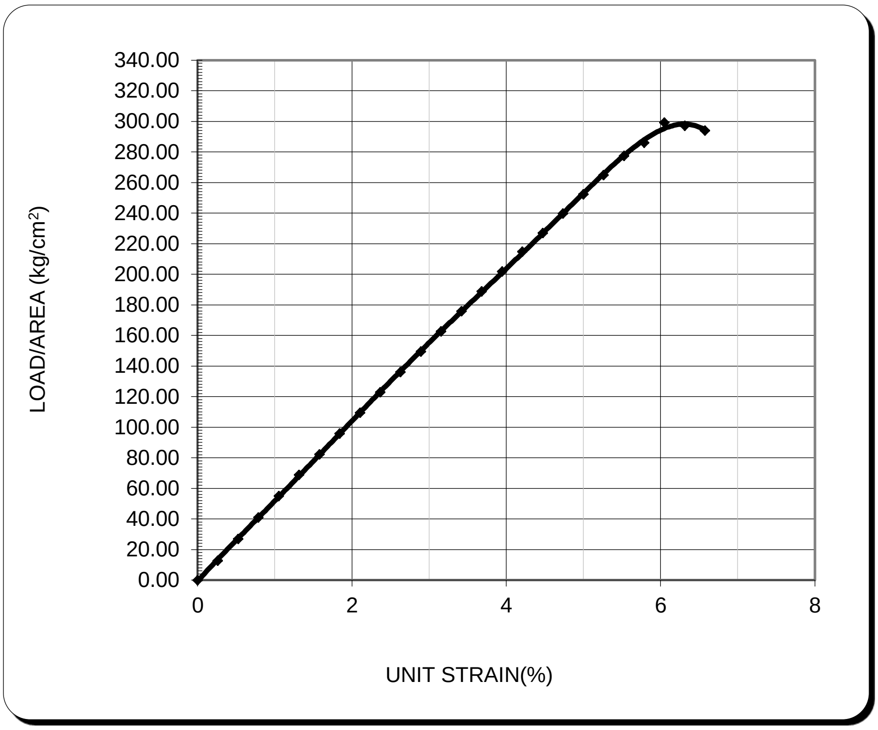

Figure D-8 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 3rd cycle

Table D-9 UCS test result for soil sample 1(pit 1) and fly ash mix 60% E-40% F 4th cycle

Diameter of Sample, (cm) =	3.8
Height of Sample (cm) =	7.6
Moisture Content (%) =	23.8
Bulk Density (gm/cm^3) =	1.609
Dry Density (gm/cm^3) =	1.300

Unit Strain (%)	Load/area (kg/cm^2)
0	0
0.263	12.664
0.526	26.839
0.789	40.940
1.053	54.965
1.316	68.915
1.579	82.270
1.842	95.848
2.105	109.353
2.368	122.783
2.632	136.140
2.895	149.423
3.158	162.631
3.421	175.766
3.684	188.827
3.947	201.813
4.211	214.726
4.474	226.966
4.737	239.697
5.000	252.355
5.263	264.938
5.526	277.448
5.789	286.060
6.053	299.215
6.316	297.075
6.579	293.993

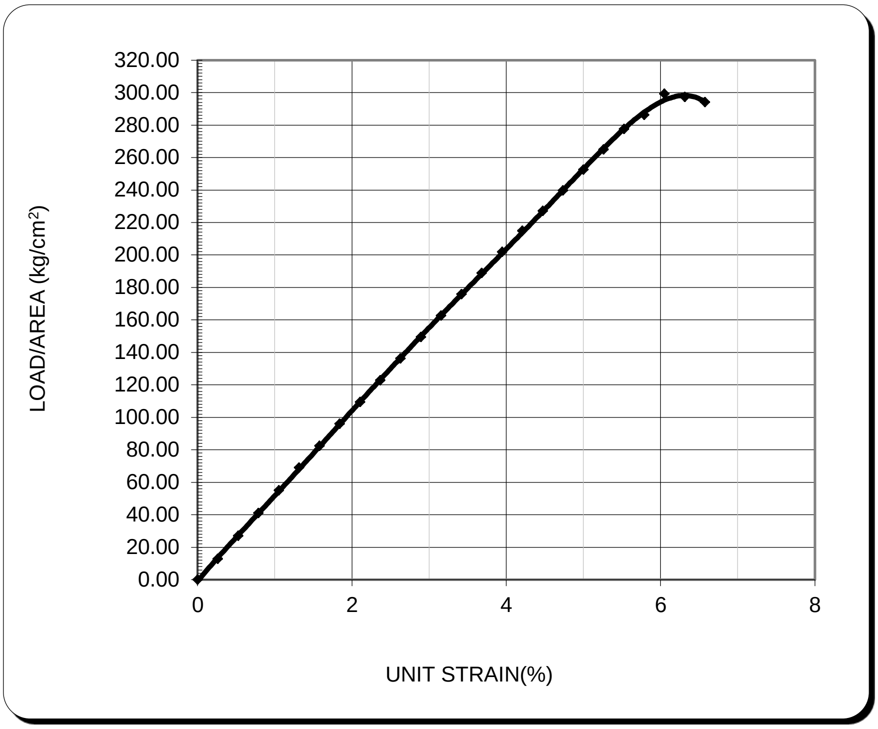

Figure D-9 UCS curve for soil sample 1(pit 1) and fly ash mix 60% E-40% F 4th cycle

Figure D-10 Free Swell test reading

Figure D-11 Material preparation with different percentage mix

Figure D-12 UCS test reading

Figure D-13 Cyclic swelling saturation by using Vacuum pump

Figure D-14 Standard proctor test

Figure D-15 CBR Swell reading

Figure D-16 UCS specimen preparation for curing

Appendix- E

Examination tests for Hazardous Fly ash

Base Metal Analysis test report (Yigzaw, 2021)

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

Issued Date: -25/03/2020

Request No: - CLD/RN/304/20

Report No: - GLD/TR/239/20

Sample Preparation: - 200 Mesh

Number of Sample: - One (1)

Originator: - Tefera Getie Yigzaw,

Sample type: - Soil

Date Submitted: -17/03/2020

Analytical Result. - ppm

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

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

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

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

Analysts

Mulumebet Luelseged

Almaz Eticha

Habtamu Alehegn

Checked By

Dessalew Bitew

Quality Control

Negash Worku

Geological Survey ... Central Geological ...

Flame Atomic Absorption Spectroscopy (FAAS)

Before S/S method treatment

No	Test	Test Results	Unit	Test Method	WHO maximum allowable Concentration (mg/L)
1	Copper(Cu)	2.478	mg/L	Flame AAS	2
2	Zink(Zn)	2.350	mg/L	Flame AAS	_
3	Lead(Pb)	2.262	mg/L	Flame AAS	0.01
4	Nikel(Ni)	0.626	mg/L	Flame AAS	0.07
5	Cobalt(Co)	0.168	mg/L	Flame AAS	_

After S/S method treatment

No	Test	Test Results	Unit	Test Method	WHO maximum allowable Concentration (mg/L)
1	Copper(Cu)	2.384	mg/L	Flame AAS	2
2	Zink(Zn)	2.143	mg/L	Flame AAS	_
3	Lead(Pb)	0.428	mg/L	Flame AAS	0.01
4	Nikel(Ni)	0.106	mg/L	Flame AAS	0.07
5	Cobalt(Co)	0.104	mg/L	Flame AAS	_

After S/S method treatment and 28 days curing

No	Test	Test Results	Unit	Test Method	WHO maximum allowable Concentration (mg/L)
1	Copper(Cu)	1.478	mg/L	Flame AAS	2
2	Zink(Zn)	1.096	mg/L	Flame AAS	_
3	Lead(Pb)	0.082	mg/L	Flame AAS	0.01
4	Nikel(Ni)	0.064	mg/L	Flame AAS	0.07
5	Cobalt(Co)	0.077	mg/L	Flame AAS	_

Figure E-1 Flame Atomic Absorption Spectroscopy (FAAS)