

**EFFECT OF MAIZE COB ASH AND WASTE CERAMIC POWDER
ON GEOTECHNICAL PROPERTIES OF EXPANSIVE SOIL:
IN THE CASE OF DUKEM TOWN**



Aduugna Hunduma Gedefa

A Thesis Submitted to 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

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Adama, Ethiopia

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APPROVAL SHEET

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LIST OF ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials;
ASTM	American Society for Testing and Materials;
CAH	Calcium Aluminate Hydrate;
CBR	Californian bearing Ratio;
Cc	Compression index;
Cr	Recompression index;
CSH	Calcium Silicate Hydrates;
Cu	Undrained shear strength;
Cv	Coefficient of consolidation;
DD	Dry density;
ES	Expansive soil;
FSI	Free Swell Index;
FSR	Free swell ratio;
Gs	Specific Gravity;
IS	Indian Standard;
LL	Liquid Limit;
MC	Moisture content;
MCA	Maize Cob Ash;
MDD	Maximum Dry Density;
NGL	Natural Ground Level;
OMC	Optimum Moisture Content;
PI	Plastic Index;
PL	Plastic Limit;
Qu	Unconfined compressive strength;
TP	Test pit;
USA	United States of America;
USCS	Unified Soil Classification System;
WCP	Waste Ceramic powder;

ABSTRACT

Expansive soil is a term generally applied to any clay soil material that has a potential for shrinking or swelling under changing moisture conditions. Expansive soil poses damages to buildings, roads, and railway which are constructed on it. To come across the risk of these problematic soils, possible geotechnical solutions for the improvement of their properties should be conducted to maintain the safety and stability of any project built over them. Due to the daily increasing costs of cement and lime that were used as improvement techniques, several researchers focused more on the use of potentially cost-effective and locally available waste materials from industrial and agricultural, to enhance the properties of these deficient soils. To improve the properties of expansive soil no research was done using maize cob ash and waste ceramic powder together. Therefore, this research work was aimed to investigate the effects of maize cob ash and waste ceramic powder on the geotechnical properties of expansive soil. The preliminary investigation of the soil to be stabilized indicates that it belongs to A-7-5 class in the AASHTO and CH as USCS soil classification system. Soils under this class are generally of poor engineering use, and truly require to be strengthened. Expansive soil was treated using a similar percentage of 5%, 10%, 15%, and 20% of maize cob ash, and waste ceramic powder separately by dry weight of the soil. The laboratory tests carried out on the natural and stabilized soils include Atterberg limits, free swelling, compaction, unconfined compressive strength, and California Bearing Ratio (CBR) tests. From the analysis of the test result, the plasticity index, and free swell of the soil were decreased while MDD, unconfined compressive strength, and CBR of stabilized soil show increment upon addition of each stabilizer by the same amount from 0% to 20%. The optimum percentage of maize cob ash was found to be 15% MCA+85% and the maximum percentage of waste ceramic powder used was found to be 20%WCP+80%ES based on the strength properties when using the materials separately. In addition to the above, based on the investigated the combined effect of maize cob ash and waste ceramic powder, the optimum amount of mixes was 15%MCA+20%WCP and it is concluded that it is effective to use the materials separately rather than in combination since it was given less strength. MCA is more effective in reducing plasticity and swelling properties of soils, and WCP is more effective to enhance the strength properties of the expansive soil. Generally, this study shows that the stabilized materials were suitable for the improvement of weak soil.

Keywords: Expansive soil, Soil improvement, Maize cob ash, Waste Ceramic powder, Swelling properties, Strength properties.

CHAPTER ONE

1. INTRODUCTION

1.1 Background

Expansive soils are clay soils that change their volume when the water contents of soils change. These soils swell significantly when the moisture content increased and shrinks when the moisture content is reduced. The swelling and shrinkage behavior of expansive soil originates mainly from the most active clay mineral called montmorillonite.

Expansive soils cover many parts of the world as well as Ethiopia. Expansive soils form a major soil group in Ethiopia and occur in the highlands mostly in the western, central, and southwestern parts of the country. The aerial coverage of expansive soils in Ethiopia is estimated to be 23.7 million acres. In most cases the major clay mineral component is montmorillonite and it causes a real financial risk and disruption of life. It is the most problematic soil in which damage buildings, roads, and railways that are constructed on it, due to its swell-shrink characteristics when subjected to change in moisture. Detail knowledge about the expansive soil help to devise the method of improving the nature of the soil for any engineering project constructed over them. The expansive nature of the soil comes from the amount and type of clay particles present in the soil. The engineering design of structures on this type of soil must be recognized with the behavior of the ground. The construction sites for most of the civil engineering works must first be investigated if the soil exhibits swelling characteristics and is classified (Uba, 2016).

The road sectors and building foundations in Ethiopia are suffering from the soil those possessing the dual property of high swelling and shrinking. Many damages occur each year on the constructed road pavements and foundation structure over such soil, creating serious problems including increasing cost of construction and maintenance. The performance of any structure depends on the underlying geotechnical properties of the soil. Soil is a base of any foundation structure, which essentially carries the structural load from beneath and distributes the load effectively. Problematic soils including expansive clay cause major problems in the design, construction, and maintenance of civil structures (Kuleno, 2020).

To come across the stated problem, it is important to increase the strength of the soil which is crucial for successful analysis of the structure to be constructed (Kuleno, 2020).

Soil stabilization is a mechanism in which the natural properties of soil are enhanced by different techniques. Nowadays, the cost of material used for soil stabilization was increased from time to time. The need to bring down the increasing cost of soil stabilizers and the cost of waste disposal has led to intense global research towards the economic utilization of wastes for engineering purposes. The disposal of industrial and agricultural waste products needs urgent and cost-effective solutions because of the debilitating effect of these materials on the environment and the health hazards that these wastes lead. To make weak soils useful and meet geotechnical engineering design requirements, researchers focused more on the use of potentially cost-effective materials that are locally available from industrial and agricultural waste to enhance the properties of deficient soils (Firdawek, 2019).

There is a various method that was used to improve the properties of expansive soil. The most commonly used stabilizers for expansive soils are bitumen, fly ash, lime, and cement, in which the stabilization of expansive soil with lime or cement is more effective (Budhu, 2007). However, the costs of these stabilizers are very high making them economically unattractive as stabilizing agents. Currently, researchers in the field of geotechnical engineering, work more on the search for cheap and locally available waste materials from industry and agriculture such as maize cob ash, waste ceramic dust powder, bagasse ash, steel slag, fly ash, rice husk ash, coconut husk ash as stabilizing agents for replacement of traditional stabilizers like cement and lime (Ashango & Patra, 2016; Gurjap et al . , 2017). Engineering properties of soils are commonly altered when these wastes are blended as admixtures alone or in combination with lime or cement. Moreover, recently, several studies have confirmed the applicability of these wastes as cement replacing material in concrete technology (Firdawek, 2019).

Nowadays, maize cob ash and waste ceramic powder have been investigated in some parts of the world for their pozzolanic properties in such a way that, have been found to enhances the properties of expansive soils. The oxide composition such as SiO_2 , Al_2O_3 , and Fe_2O_3 in both materials is more than 70. This shows that both materials possess a good pozzolanic property, that could help to mobilize the CaOH in the soil for the formation of cementations properties. All the studies conducted were, investigated the effect of MCA and WCP separately. However, in this research, the separate and combined effect, as well as comparative effect, was studied.

1.2 Statement of the problem

In many parts of Ethiopia, expansive soils cause the problem to geotechnical, structural, and hydraulic foundation projects constructed over them. It is due to the presence of a clay mineral montmorillonite in expansive soil that exhibits high swelling and shrinkage when it was subjected to moisture content. A large part of Dukem town is covered by black to gray expansive soils; As a result, many lightweights, residential buildings, and subgrade road projects were affected by these problematic soils. Most of the time the contractors of the many projects in the town remove the in-situ foundation soil to the required depth and replace it with more suitable soil. However, excavation of weak soil, searching, and transporting of better-quality materials brings additional costs and delay the construction time.

Around Dukem town, different waste materials like ceramic dust disposed of factories lead to environmental pollution and are hazardous to the health of human beings. Nowadays an attempt was made to reduce the impact of agricultural and industrial wastes on the environment, then utilizing them for improvement of geotechnical properties of weak soils and also reduce the usage of the high-cost conventional stabilizers like cement and lime. For many years various waste materials were used for treating poor soil. There was limited research done on improving the expansive soil by using maize cob ash and waste ceramic dust separately. However, there was no study conducted on the stabilization of expansive soil by the combination of maize cob ash and waste ceramic dust. Therefore, the research was aimed to investigate the separate and combined effect of maize cob ash and waste ceramic powder on some geotechnical properties of expansive soils.

1.3 Objective of the Study

1.3.1 General Objectives

The general objectives of this study are to investigate the effect of maize cob ash and waste ceramic powder on some geotechnical properties of expansive soils around Dukem town.

1.3.2 Specific Objectives

- To know the engineering and index properties and identify the type soils of Dukem town.
- To assess the effect of maize cob ash and waste ceramic powder on the soil plasticity/consistency and swelling properties.

- To determine the effect of maize cob ash and waste ceramic powder on the compaction characteristic of the soils.
- To evaluate the effect of maize cob ash and waste ceramic powder on the unconfined compressive strength of the soils.
- To investigate the effect of maize cob ash and waste ceramic powder on the California bearing ratio (CBR) of the soils.

1.4 Research question

- What are the index and engineering properties of expansive soil of Dukem area?
- What are the effect of maize cob ash and waste ceramic powder on the plasticity and free swelling characteristics of the soil?
- What are the effect of maize cob ash and waste ceramic powder on the compaction characteristics of the soil?
- What are the effect of maize cob ash and waste ceramic powder on UCS of the soil?
- What are the effect of maize cob ash and waste ceramic powder on the CBR of the soil?
- What are the combined effect of maize cob ash and waste ceramic powder on the strength of the soil?

1.5 Scope of the Study

This study was assisted by reviewing a different kind of literature and performing a series of laboratory experiments. The study was aimed to investigate the effect of improving expansive soils found in Dukem with maize cob ash and waste ceramic powders using various mix proportions with soil samples and conducting various laboratory tests. In addition, the study is also emphasis the investigation of properties of natural expansive soil by collecting the sample from two test pits, then analyzing the effect of applying maize cob ash and waste ceramic powder with a weaker soil sample from the two. The relevant laboratory tests conducted were; natural moisture contents, specific gravity, grain size analysis, consolidation tests to characterize the expansive soils. While, Atterberg limit tests, swelling tests, compaction tests, unconfined compressive strength, and CBR tests were conducted on the soil and material stabilizers, depending on ASTM and ASHTO standard specifications. Generally, these investigation

concern to evaluate the effect of maize cob ash and waste ceramic powder separately on some characteristics of expansive soil, again the evaluation of the combined effect of both stabilizers on some properties of soil were undertaken, finally, a comparative analysis was made by considering the effect each material stabilizer on the properties of expansive soils.

1.6 Significance of the study

The production of traditional and adapted stabilizers, such as cement and lime were requiring high costs and were environmentally hazardous processes. Thus, using those materials only, possess a significant problem for the local environment and its inhabitants. Therefore, the significance of this research work was investigated and introduced maize cob ash and waste ceramic powder as new stabilizers in our country. Using both maize cob, and waste ceramic for stabilization of weak soil is beneficial in that, it was readily available, low costs, and environmentally friendly. Moreover, while utilizing for this purpose, their problem with the environment like pollution and deterioration was reduced. The study also serves as a reference for Geotechnical engineers and other researchers in a related study. Therefore, using waste ceramic powder and natural maize cob wastes for improving geotechnical properties of the soils in the construction of foundations and subgrade was an economical solution for Ethiopia as it is available in large quantities.

1.7 Organization of the thesis

The presentation of this thesis work is organized into five chapters. The first chapter gives a brief description of the thesis background, statement of the problem, objectives, scope, significance, and methodology employed. The second chapter is a literature review that explains the nature and property of expansive soil, the importance of stabilization and its significance in the Ethiopian context and the conceptual background of the additives used for the research, and the important procedure in the stabilization process. The third chapter briefly describes the characterization of materials used for the study, experimental design, and standard testing procedures followed. The fourth chapter reports the test results obtained; analysis of results and discussion of results concerning the theoretical background and concerning findings. Finally, conclusions and recommendations drawn from the study were presented in chapter five.

CHAPTER TWO

2. LITERATURE REVIEW

This chapter presents the origins, distributions, mineralogy, identification, and classification of expansive soil. Furthermore, the foundation consideration for expansive soil, a different method of soil stabilization, and a summary of related work was discussed.

2.1 Review on Expansive soil

Expansive soil refers to soil that possesses a dual characteristic of swelling and shrinking due to changing moisture conditions. Alternate expansion and shrinkage lead to deformation in the soil mass and result in the damage of engineering structures founded on this soil. Thus, expansive soils cause more damage to structures in particularly pavements, and lightweight buildings than any other natural disaster, including earthquakes and floods. The cost needed for the maintenance of already damaged structures is remarkable. Before the construction of engineering structures to reduce the problems associated with expansive soils, so many techniques were applied: removal and replacement with non-expansive fill, ponding or prewetting, the adoption of a better design approach in which, the use of moisture barriers or membranes, and soil stabilization methods were known so far (Nelson & Miller, 1992). Potentially, expansive soils can be found anywhere in the world but it is abundantly confined to semi-arid and arid regions. The climate condition of this region is such that the annual evapotranspiration exceeds the precipitations (Chen, 1988). In fact, in semi-arid zones, the lack of leaching has aided the formation of montmorillonite. Much of the expansive soil problems may not have been recognized in the underdeveloped nations, however, more expansive soil regions were discovered each year as the amount of construction increase (Chen, 1988).

2.1.1 Origin of Expansive soils

The sources of expansive soils are related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, expands. Other conditions and processes may also form clay minerals, most of which are non-expansive. The conditions or processes, which determine the clay mineralogy, include the composition of the parent material and the degree of physical and chemical weathering to which the materials are subjected (Tamrat, 2013).

The parent materials for expansive soils can be classified into two groups. The first group comprises the basic igneous rocks. This group comprises minerals named Feldspar and Pyroxene. The decomposition of these minerals forms an important mineral called smectite which is a group of montmorillonite and other secondary minerals. A small amount of rainfall has hindered the weathering of the active montmorillonite mineral into low active clay types such as illite and kaolinite. The second group comprises sedimentary rocks. The rock comprises smectite (montmorillonite) as a constituent and breaks down physically to form expansive soils. (Chen, 1988).

2.1.2 Mineralogy of Expansive soils

Expansive soils are inorganic clays of medium to high compressibility. clay-minerals can be divided into three general groups based on their crystalline arrangements such as kaolinite, montmorillonite, and illite group.

Kaolinite Minerals: It is built of two-layer minerals having a single tetrahedral sheet joined by a single octahedral sheet to form what is called a 2 to 1 lattice structure. It has a structural unit made up of alumina sheets joined to the silica sheet (Chen, 1988). The bonding combination of hydrogen and van der Waals forces results in considerable strength and stability with little tendency for interlayers to take on water and swell. The bond that exists between layers is tight and hence it is difficult to penetrate between the layers so; kaolinite has a low degree of swelling and least active of the clay minerals.

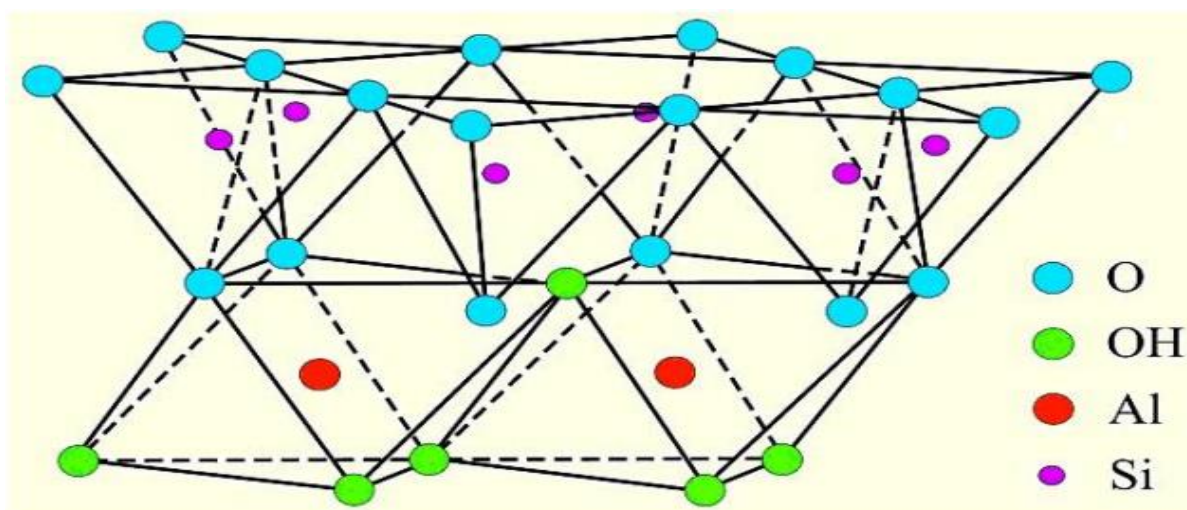


Figure 2. 1:Diagrammatic sketch of kaolinite structure (Musema, 2014).

Illite group: It has a basic structure consisting of a sheet of alumina octahedron between and combined with two sheets of silica tetrahedrons. This produces a 1: 2 minerals with additional difference that some of the silica positions are filled with aluminum atoms and potassium ions are attached between layers to make up the charge deficiency. The bonding results in a less stable condition than for kaolinite, and thus the activity of illite is greater than kaolinite. This clay mineral has a structure similar to montmorillonite, but the layers are more strongly bonded together. Illite units are reasonably stable and so that minerals swell much less than montmorillonite.

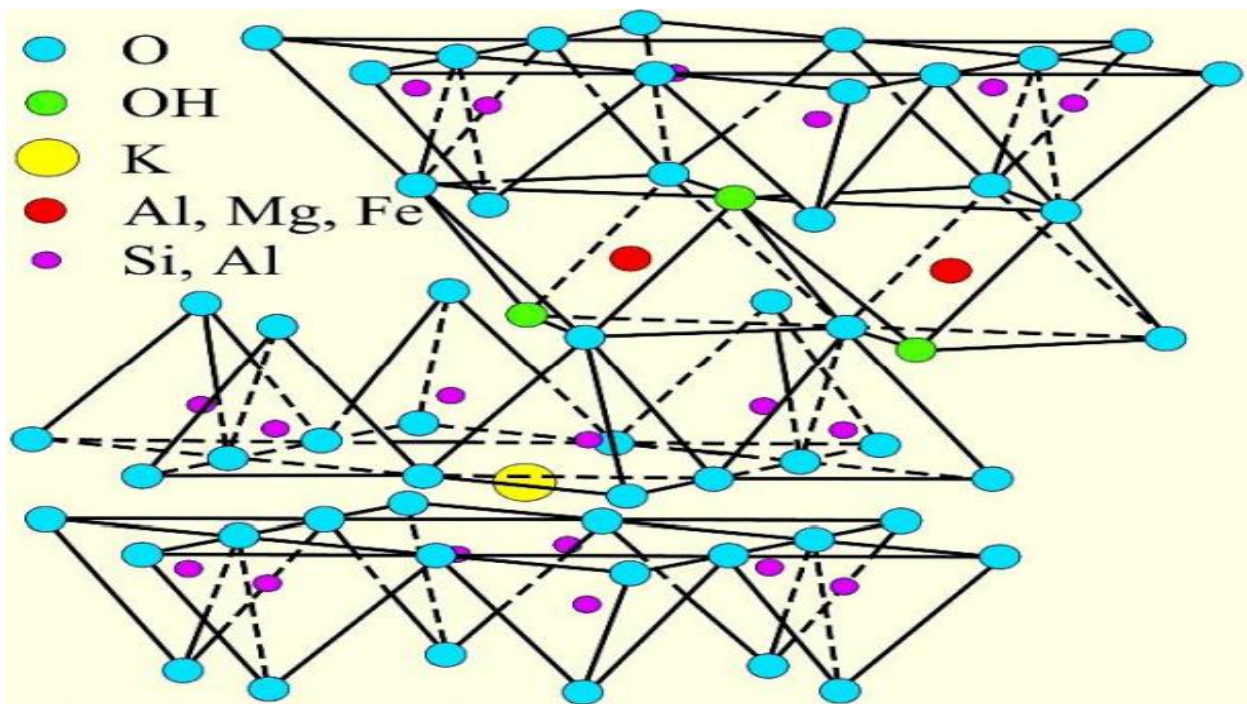


Figure 2. 2:Diagrammatic sketch of illite structure(Onur, 2009).

Montmorillonite group

Montmorillonite is a group of clay minerals that are characterized by weakly bonded layers. It is formed from weathering of volcanic ash under poor drainage conditions or in marine waters. Each layer consists of two silica sheets with an aluminum(gibbsite) sheet in the middle. The basic montmorillonite units are stacked one on top of the other, but the bond between the individual units is relatively weak and water is easily able to penetrate in between the sheets and form separation and hence swelling. Moreover, it possesses the greatest compressibility and lowest shear strength of all clay minerals. The gap between the combined sheets is held by water

molecules and exchangeable cations. Considerable swelling of montmorillonite can occur due to excess water absorbed between the combined sheet (Musema, 2014).

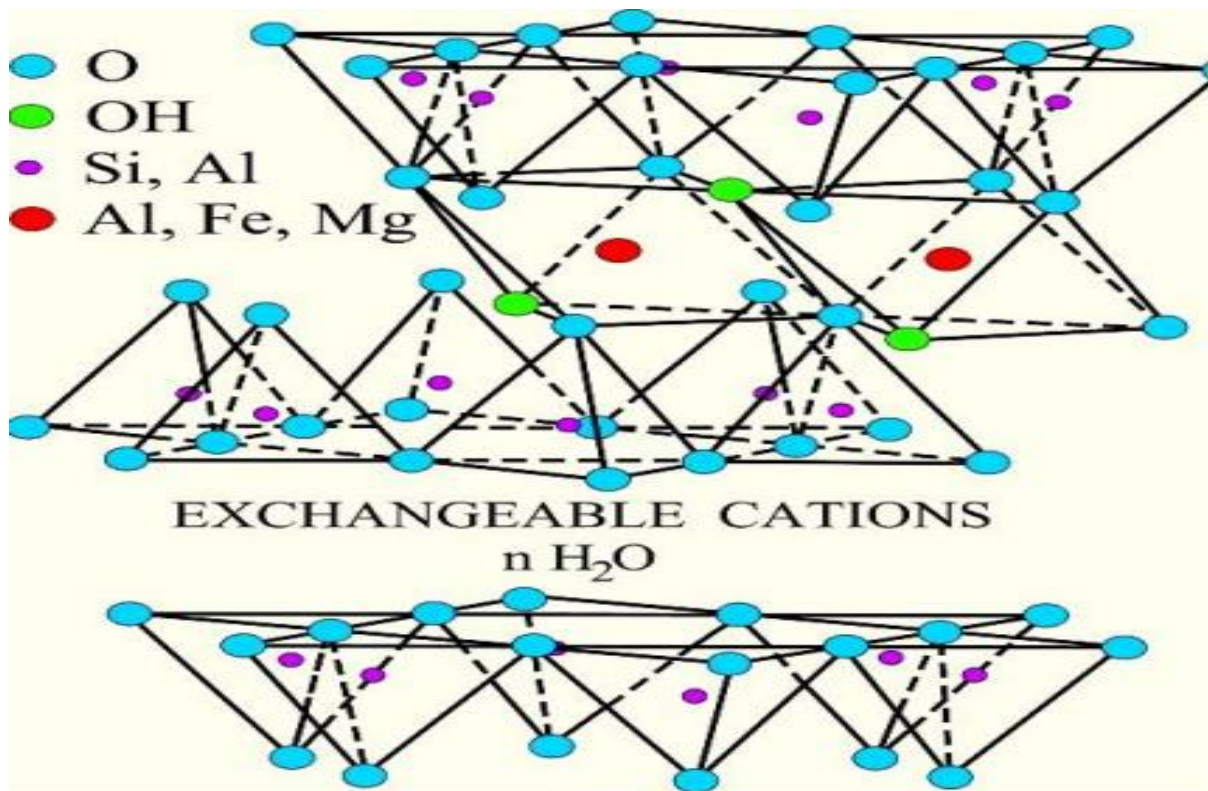


Figure 2. 3:Diagrammatic sketch of montmorillonite structure (Onur, 2009).

2.1.3 Distribution of Expansive soils

Expansive soils are widely spread in the African continent, occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Ghana, Nigeria, etc. In other parts of the world, the case of expansive soils has been widely reported in countries like the USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, and Venezuela (Tilahun, 2004). They are widely spread in the central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis-Gohatsion, Addis-Modjo are covered by expansive soils. Also, areas like Mekele, Gambella, and southwest Ethiopia are covered by expansive soil (Meron & Samuel, 2014).

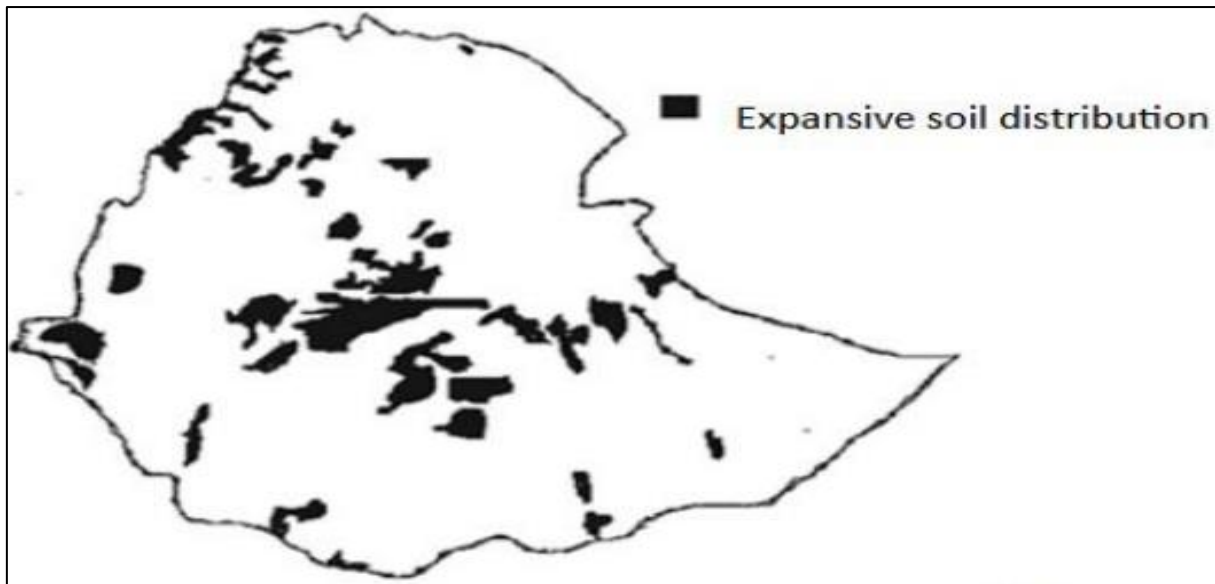


Figure 2. 4: Distribution of Expansive soil in Ethiopia

2.2 Identification and Classification of Expansive Soil

Investigation of expansive soils generally consists of two important parts. The first is knowing and making sure that the soil is expansive and the second is collecting samples and laboratory identification of soil properties. Statistical studies are provided which indicate the assessment of soil expansiveness based on index properties is an overestimation. There are several methods for identifying expansive soils. The method of identification and classification of expansive soils can be divided into three parts as mineralogical identification, indirect methods, and direct methods.

2.2.1 Field Identification

Expansive soil occurrence can be recognized in the field through visual examination. The method is simple and easy to use. Soils that can possess high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages which include important observations as follows (Nelson & Miller, 1992; Chen, 1988).

- ✓ Usually have a color of black or gray.
- ✓ Wide or deep shrinkage cracks.
- ✓ High dry strength and low wet strength.
- ✓ Stickiness and low traffic ability when wet.

- ✓ Cut surfaces have a shiny appearance.
- ✓ The appearance of cracks in nearby structures (Nelson & Miller, 1992).

2.2.2 Laboratory Identification

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

2.2.3 Mineralogical Identification

Clay mineralogy is a major factor controlling expansive soil behavior. Clay minerals can be identified using a variety of mineralogical identification which are important for exploring the basic properties of clays, but it is impractical and uneconomical for practicing engineers. The various methods are important in the research laboratory for exploring the basic properties of expansive soils (Musema, 2014).

The uses of mineralogical identification are to identify the mineralogy of clay soil such as characteristic crystal dimensions, characteristic reaction to heat treatment, size, and shape of clay particles, charge deficiency, and surface activity of clay particles (Nelson & Miller, 1992). These properties are a major factor controlling expansive soil behavior. The methods that can be used are (Chen, 1988).

- X-ray diffraction
- Differential thermal analysis
- Dye adsorption
- Chemical analysis
- Electron microscope resolution (Chen, 1988).

The mineralogical composition of expansive soils has an important bearing on the swelling potentials. There are five techniques that can help in identifying the mineralogy of the expansive soils.

1. X-ray diffraction

The x-ray method is the most widely used technique for the identification of clay minerals. It is used to determine the proportion of the various minerals present in colloidal clay soil by

comparing the ratios of the intensities of diffraction lines from the different minerals with the intensities of lines from the standard materials (Chen, 1988).

- ✓ X-ray diffraction methods are based upon satisfaction of the Bragg law for powdered crystalline materials. The sample for x-ray diffraction analysis can be prepared as random powder samples or as an oriented sample. It states that the angle theta varies when placed in a monochromatic beam with the random orientations that the particles of crushed material will reflect from each of the possible planes of reflection for the materials. For all the reflecting planes, the theta values are determined from measurements of the line positions. Identification of the lines is easy for the high symmetry crystal systems and very complex for the lower symmetry crystals. According to this principle, when the crystal is bombarded with the x-ray of a constant wave length and at certain angles, the intense x-rays are produced as the wave length of scattered x-rays interfere constructively (Merrill, 1959). However, once the pattern has been interpreted and the values and calculations recorded, all future pictures of the same material can be compared against the known standards.

2. Differential thermal Analysis: is also widely used to study the character of clay materials. This technique is well established as a method for the control of materials that undergo characteristics changes on heating. The use of differential thermal analysis technique alone in identifying clay soil is not always accurate instead it is used with x-ray diffraction. But particularly useful in the identification of amorphous material when the XRD (Chen, 1988). DTA was limited primarily due to the sensitivity of the apparatus being used, the thermal curve records in which all the resulted transformations heat is taken up or given off.

3. Dye Adsorption: Mineral can be identified by characteristic colors formed by dyes that are absorbed by the minerals of the clay sample. When an expansive soil sample is pretreated with acid, the color assumed by the absorbed dye depends on the base exchange capacity of the various clay minerals present. The presence of the montmorillonite can be identified if the selected sample contains a mineral that is greater than about 5-10%. It is a relatively simple testing procedure and the speed of dye staining tests compared with X-ray diffraction and differential thermal analysis describe a wider application of the color method (Chen, 1988).

4. Elemental /Chemical Analysis. It is an essential technique to support other methods such as X-ray analysis in identifying clays. chemical analysis can be used to assess the nature of

isomorphism and to show the origin and location of the charge on the lattice in the montmorillonite group of clay minerals. The isomorphism involves three basic variations of the substitution: the substitution for Al for Si in tetrahedral positions in the lattice; the substitution of Fe for Al in the octahedral coordination; and the substitution of Mg for Al in the octahedral positions (Chen, 1988).

5. Electron Microscope Resolution. Microscopic testing of clay minerals offers a direct observation of the material under electron microscope resolution. Under electron microscope resolution two clays may give the same X-ray pattern and the same differential thermal curve but show up distinct geomorphology of clay structure. Therefore, the main objective of the microscopic investigation is to determine the mineralogical composition, texture, and internal structure of clay soil (Chen, 1988).

2.2.4 Indirect Methods

In this method, simple soil property tests can be used for the determination of the swelling potential of expansive soils. Those tests are easy to conduct and should be included as routine tests in the investigation of expansive soils. Such tests may include the following.

i. Atterberg Limits

Measurements of the Atterberg limits of the soil are conducted for identification of all soils and provide a wide acceptable means of rating. Moreover, when combined with other tests, it can be used to classify expansive soils. The Atterberg limit which is useful for engineering purposes is; Liquid limit, plastic limit, and shrinkage limit. It is expressed as the percent of water content. The relation between the swelling potential of clays and the plasticity index is shown in the table below (Chen, 1988).

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

Swelling potential	Plasticity index	Liquid limit
Low	0-15	<29
Medium	10-35	29-40
High	19-55	40-60
Very high	55 and above	>60

ii. Free swell tests

The free swell test may be considered as a measurement of volume change in clay upon saturation and is one of the most commonly used simple tests to estimate the swelling potential of expansive clay. Experiments indicated that a good grade of high swelling commercial bentonite would have a free swell of from 1190 to 1900 percent. Soils having a free swell value as low as 100 percent can cause considerable damage to lightly loaded structures, and soils those having a free swell value below 50 percent seldom exhibit appreciable volume change even under very light loadings. It is believed that this was due to extreme climatic conditions in combination with the expansion properties of the soil.

The free swell percentage can be computed using the Equation below from the relationship between initial and swelled volume (Chen, 1988; Nelson & Miller, 1992; Leikun, 1999).

$$\text{Free swell} = \frac{V_f - V_i}{V_i} \dots\dots\dots (2.1)$$

Where: FS=free swell, (%),

V_i = volume of dry soil, 10cm³, and

V_f = soil volume after swelling, cm³

iii. Free Swell Index

The free swell index is also one of the most commonly used simple tests to estimate the swelling Potential of expansive clay soil. The procedure involves taking two oven-dried soil samples Passing through the 425µm sieve, the 10cc each was placed separately in two 100ml graduated soil Samples. Distilled water was filled with one cylinder and kerosene in the other cylinder up to the 10ml mark. The final volume of soil is computed after 24hours to calculate the free swell index (Al-Rawas, 2005) The free swell index is then calculated using the Equation below.

$$\text{Free swell index (\%)} = \frac{V_w - V_k}{V_k} * 100 \dots\dots\dots (2.2)$$

Where: V_w = final volume in water

V_k = final volume in kerosene

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

Free swell Index (%)	Degree of Expansion
Less than 20	low
20 to 35	Moderate
35 to 50	High
Greater than 50	Very high

ii. Free Swell ratio test

To determine the swell property, Sridharan and Prakash proposed the free swell ratio method of characterizing the swelling of soil. The Free swell ratio is defined as the ratio of sediment volume of 10cc oven-dried soil passing through the 424 μ m sieve in distilled water to that of Kerosene and is computed by Equation below.

$$\text{Free swell ratio (\%)} = \frac{V_w}{V_k} * 100 \dots\dots\dots (2.3)$$

Where: V_w = final volume in water

V_k = final volume in kerosene

The relation between the degree of expansion and the differential free swell ratio is given below.

Table 2. 3: Classification of Soils based on the free swell ratio (Craig, 2003).

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

2.2.5 Direct Methods of Identification of Expansive soils

These identification methods offer the most useful data by direct measurement, and tests are simple to perform and do not require complicated equipment. Testing should be performed on

several samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of a one-dimensional Oedometer/consolidation test.

2.3 Classification of Expansive Soils

2.3.1 Classification Using General Methods

The most widely used general classification systems are, Unified soil classification system (USCS) and the American Association of State High way and Transportation Officials (AASHTO) method depending on index properties (Berhane, 2018).

i. Unified soil classification systems

According to the Unified soil classification system, a correlation is made between swell potential and unified soil classification as follows (Berhane, 2018).

Table 2. 4:Unified soil classification system

Category	Symbol	soil classification in Unified system
Little or no expansion	1	GW, GP, GM, SW, SP, SM
Moderate expansion	2	GW, SC, ML, MH
High volume change	3	CL OL, CH, OH

The above classification system can be summarized as below:

- All sands and gravels, grouped in symbol number 1, exhibit little or no expansion.
- All clayey gravels and sands and all silts, grouped in symbol number 2, exhibit moderate volume changes.
- All clay soil and organic soils, grouped in symbol number 3, exhibit high volume change. In the above classification, soils rated as CH or OH may be considered potentially expansive soil.

ii. AASHTO soil classification system

AASHTO system classifying soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. Evaluation of soils within each group is made using a “group index,” which is a value calculated from an empirical formula. The group

classification, including the group index, should help to determine the relative quality of the soil material. However, for the detailed design of the necessary structures, additional parameters regarding the strength or performance characteristics of the soil under field conditions are usually required. The classification is made by using the test limits and group index values. As shown in figure 2.1 soils that lie on A-6, A-7, and borderline soils A-4, A-6, and A-7 may be considered potentially expansive.

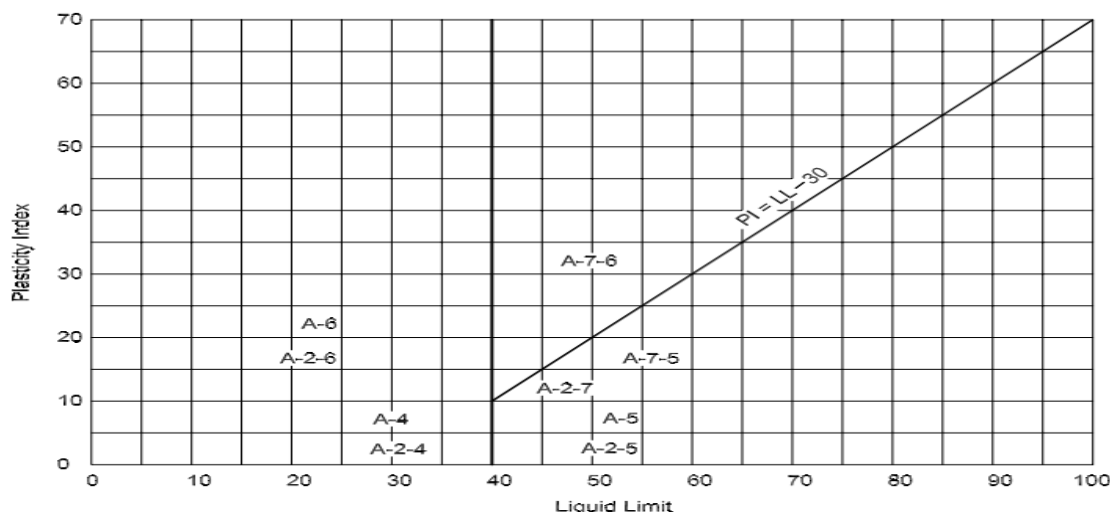


Figure 2. 5:Liquid limit Vs Index chart for AASHTO soil classification method

2.3.2 Classification specific to Expansive soil

The above classification system may give an initial alert that the soil may have expansive character but it does not provide adequate information. A parameter determined from the expansive soil identification tests has been combined in several different classification schemes to give a qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas (Nelson & Miller, 1992).

Hence, it is very important to emphasize that design decision has to be based on predicting testing and analysis, which gives reliable information. An indirect prediction of swell potential based on correlation of index properties, swell, and a combination of them. Some of such classification schemes are discussed below.

2.3.3 Classification Based on Indirect Predictions of swell potential

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

➤ Alemayehu and Mesfin [1999]

Depending on their idea, one may use it to check the accuracy of laboratory test results for expansive soils found in Ethiopia according to the following table.

Table 2. 5: Indicative properties of Expansive soils in Ethiopia.

Clay content smaller than 2 μ m	50-80%
Liquid limit	80-120%
Plastic limit	55-90%
Shrinkage limit	10-16%
Free swell	90-123%

➤ Method of Daksanamurthy and Raman (1973)

The presented method for identifying expansive soils using a single index of only liquid limit.

They suggested four classes of clays according to their liquid limits as shown in Table 2.6 below.

Table 2. 6: Relation between the swelling potential of clays and the liquid (Al-Rawas, 2005).

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

➤ Skempton Method (McKeen, 1976)

This method classifies clays according to their activities which is developed by Skempton (1953) by correlating Atterberg limits and clay content (% percent by weight finer than 2 μ m)

into a single parameter called activity. Thus, Skempton classified clays into three classes according to their activities as indicated in Table 2.7.

Activity is defined as:

$$\text{Activity} = \frac{\text{plasticity index}}{\text{Percent clay}(<0.002\text{mm})} \dots\dots\dots (2.4)$$

Table 2. 7:Degree of colloidal activity (Skempton 's method).

Potential of Expansion	Activity
Low/inactive	$Ac < 0.75$
Medium/Normal	$0.75 < Ac < 1.25$
High/ Active	$Ac > 1.25$

The above method classifies the clay minerals as, montmorillonite very expansive Clay, and defined as active, illite clay as normal and Kaolinitic clay as inactive (Nelson & Miller, 1992; Chen, 1988).

➤ **Chen Method**

In this method, a correlation is made between swell data and percent less than No. 200 sieves, liquid limit, and standard penetration resistance. The classification is as given in table 2.8 (Chen, 1988).

Table 2. 8:Chen method of classification of expansive soil.

<No.200sieve, %	LL, %	Standard penetration	Probable Expansion,	Degree of Expansion
<30	<30	<10	<1	Low
30-60	30-40	10-20	1-5	Medium
60-95	60	20-30	3-10	High
>95	>60	>30	>10	Very high

The classification system developed based on single property alone such as: based on activity (Skempton, 1953), based on index property (Kannan and Brink, 1952), etc. is difficult to use alone as a classification system because it may lead to the wrong conclusion. But, Chen's method of classification of expansive soil, classification based on the bureau of reclamation method, U.S.B.R Classification method, etc. a better indirect classification system developed by combining index property, swell and physical indicators (Ambaye, 2012).

2.4 Foundation consideration for Expansive soils

It is necessary to note that all parts of a building are not equally be affected by the swelling potential of the clay soil. Beneath the center of a building where the soil is protected from sun and rain, the moisture changes are small and the soil movements are least. However, beneath outside walls, the movements are greater as the result of the damage to buildings is greatest due to soil movements (Murthy, 2002). The lightweight structures are most affected by swelling/shrinkage on expansive soils. The most common way in which foundations can be damaged is by uplift force which is caused by expansive soils. Swelling expansive soils cause crack and lift up lightly-loaded, continuous strip footings, and commonly cause distress in floor slabs. The resultant uplift varies in different areas under the building due to the various building loads on different portions of a structural foundation (Foeze, 2016).

Damages experienced by these structures include cracks in the foundation and walls and jammed doors and windows. The design of foundations on expansive soils is much more difficult than ordinary soils. Any mistake in considering the specific requirements of foundation design in expansive soils could result in high construction and repair costs.

❖ Foundation design on the Expansive soil

Three forms of foundations can be considered in expansive soils.

1. Structures that can be kept isolated from the swelling effects of the soils
2. Designing of foundations that shall remain undamaged in case of swelling
3. Elimination of swelling potential of the soil.

Shallow foundations are not recommended to be used on expansive soils for both light and heavy weight building, as the result following methods are adopted (Murthy, 2002).

For lightweight structures, the most economical method is to construct and design the structure that has sufficient rigidity to tolerate the expansion of the soil and for heavy structures, the foundation is designed in the manner to separate the main structure from expansive soil. The recommended foundation alternatives when dealing with potentially expansive soils include: **Strip/continuous footing, pad footings, mat/raft foundation, and drilled pier foundations.**

- Ribbed /mat foundations are the most suitable in moderate expansive soils in which the heave is not excessive. In such conditions, it is used to separate the structure from the expansive soil and extend to a layer of soil with acceptable expansive property, a grade beam on top of a deep foundation that supports the floor and the superstructure.
- Drilled piers are also constructed in the area of high swelling soil and used to counteract uplift forces caused by expansive soils. commonly drilled piers called belled piers when made with an enlarged base, and described as straight-shaft piers, when made without an enlarged base. The method of drilled piers is to provide a relatively inexpensive way of transferring the structural loads down to stable material or to a stable zone where moisture changes are improbable and no direct contact of the soil and the structure with the exception of the soils supporting the piers (Murthy,2002).

If the soil has a low swell potential, standard construction practices may be conducted. However, if the soil possesses a high swell potential, precautions need to be taken, which may include the following (Braja, 2011).

- ✓ Replacing the expansive soil with other materials under the foundation
- ✓ Changing the nature of the expansive soil by compaction control, prewetting, installation of moisture barriers, or chemical stabilization
- ✓ Strengthening the structures to withstand heave, constructing structures that are flexible enough to withstand the differential soil heave without failure, or constructing isolated deep foundations below the depth of the active zone.

One specific method may not be sufficient in all conditions. As the result, combining various techniques and considering local construction experience may be effective. The following are some details concerning the commonly used methods for expansive soils. (Braja, 2011).

Replacement of Expansive Soil

When shallow, moderately expansive soils are present at the surface, it can be removed and replaced by less expansive soils and then compacted properly.

Changing the nature of Expansive soil

(a) Compaction: The heave of swelling clay soils, decreases substantially when the soil is compacted to a lower unit weight on the high side of the optimum moisture content (possibly 3 to 4% above the optimum moisture content). Even under such a situation, a slab-on-ground type of construction should not be considered when the total probable heave is expected to be about 38 mm or more.

(b) Prewetting: It is the technique used for increasing the moisture content of the soil or ponding and hence attaining most of the heave before construction. However, these methods may take a long time because the seepage of water through highly plastic clays is slow.

(c) Installation of moisture barriers: The differential heave of a long term can be reduced by controlling the moisture variation in the soil. This is achieved by constructing vertical moisture barriers about 1.5 m deep around the perimeter of slabs for the slab-on-grade type of construction. These moisture barriers may be built-in trenches filled with gravel, lean concrete, or impervious membranes.

(d) Stabilization of soil: Different stabilization techniques are employed to enhance the properties of swelling clays. Chemical stabilization with the aid of lime and cement has often proved useful. A mix containing about 5% lime is sufficient in most cases. The advantages of lime in stabilizing expansive soils, thereby reducing the shrinking and swelling characteristics.

2.5 Review on soil stabilization

Soil stabilization is the alteration of one or more characteristics of the soil to create an improved soil material possessing the desired engineering properties/ mixing of materials with the soil to improve certain characteristics of the soil. The process may also include the blending of soils to achieve the desired gradation, or the mixing of commercially available additives that may alter the gradation, texture, or plasticity, or act as a binder for cementation of the soil (Guyer, 2011; Mediterian, 2014).

2.5.1 Uses of stabilization

The purposes of soil stabilization may include; increasing the shear strength of an existing ground condition to enhance its load-bearing capacity, achieve a desired improved permeability, and enhancing the durability of the soil (Tesema, 2016).

Commonly, improvement obtained from soil stabilization can be summarized as; (Guyer, 2011) The purposes of soil stabilization may include; increasing the shear strength of an existing ground condition to enhance its load-bearing capacity, achieve a desired improved permeability, and enhancing the durability of the soil (Tesema, 2016).

Quality improvement: the most common modification achieved through stabilization include reduction of plasticity index or swelling potential and increases of durability and strength of the soil. The stabilization process may also be used to provide a working platform for construction operations in wet weather (Guyer, 2011).

Thickness reduction: the strength and stiffness of a soil layer can be improved through the use of a stabilizer to allow a reduction in design thickness of the stabilized material compared with a stabilized or unbound material. The design thickness can be reduced if the strength, stability, and durability requirement of a base or sub-base course can be decreased, suitability is indicated by further analysis and evaluation (Guyer, 2011).

Expansive soils cause several problems and difficulties for civil engineering infrastructure. A different approach is applied to improve the characteristics of expansive soils. Soil stabilization is one of the construction techniques because it improves the engineering properties of soil such as strength and stability. When the soil strength properties cannot be improved by mechanical stabilization, the chemical admixture technique is used to meet the desired strength. Chemical stabilization is the most popular method used to enhance the physical and mechanical properties of problematic soils consisting of soft soil and expansive soil (Niken et al., 2017).

2.5.2 Types of soil stabilization

The two most commonly used methods of stabilizing soils are Mechanical stabilization and chemical stabilization. Mechanical stabilization refers to the process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil (Modifying expansive soil by blending with non-expansive soil). Mechanical stabilization is

best suited for coarse-grained soils and clayey soils are more effective if chemically stabilized (Tesema, 2016)

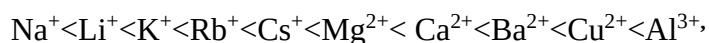
2.5.3 Mechanical soil stabilization

Mechanical stabilization can be defined as a process of enhancing the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The major methods of mechanical stabilization can be classified into compaction, mixing or blending of two or more gradations, applying geo-reinforcement, and mechanical remediation (Makusa, 2012; Guyer, 2011).

2.5.4 Chemical soil stabilization

Soil stabilization using chemical additives is the oldest and most widely used method of soil improvement. Chemical stabilization is the blending of soil with one or a combination of additives of powder, slurry, ash, etc. to meet good engineering properties such as stability, strength, swelling, permeability, and durability. Soil improvement using chemical stabilization can be grouped into three chemical reactions; cation exchange, flocculation, and agglomeration pozzolanic reactions.

i. Cation Exchange: The excess ions of opposite charge that of the surface of the clay, over those of like charge present in the diffuse double layer are called exchangeable ions. These ions can be replaced by a group of different ions having the same total charge, by changing the chemical composition of the equilibrium electrolyte solution. Negatively charged clay surface adsorb cations of specific types and sizes. The ease of replacement or exchange of cations depends on many factors, mainly the valence of the cation. Higher valence cations easily replace lower valence cations. For ions of the same valence, the size of the hydrated ion becomes essential; the larger the ion, the greater the replacement tendency. If other conditions are kept constant, trivalent cations are held more tightly than divalent and divalent cations are held more tightly than monovalent cations. A typically replaceability series is:



The cation exchange includes an immediate reaction of the clay with the stabilizer in few minutes of mixing, resulting in soil with improved texture.

Example of the cation exchange:

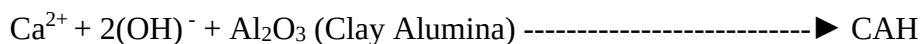
$\text{Ca}^{2+} + \text{Na}^+ \text{-clay} \rightarrow \text{Ca}^{2+} \text{-clay} + (\text{Na}^+)$ The above example shows lower valence cation that is Na^+ is replaced by higher valence cation Ca^{2+} (Assefa, 2016).

ii. Flocculation and Agglomeration

Cation exchange reactions result in the flocculation and agglomeration of the soil particles with a subsequent decrease of the quantity of clay-sized materials and hence the soil surface area, which results in the reduction of plasticity. Due to changes in texture, a significant reduction in the swelling of the soil occurs. Flocculation and agglomeration are the processes that involve rearrangement of the clay particles from face-to-face orientation to more compact edge-face orientation. The fine-grained soil changes to the coarser-grained with much more improved strength as well as workability. As cation exchange, flocculation and agglomeration is also a short-term process, which takes place within few hours of mixing the stabilizer and water with soil materials. (Firdawek, 2019; Emmanuel Emeka, 2015).

iii. Pozzolanic Reactions

Time-dependent pozzolanic reactions play a major role in the stabilization of the soil since it is responsible for the improvement of the various soil properties. The pozzolanic reaction is a long-term process that forms more stable hydrates, and aluminates of calcium after few months of mixing of soil, stabilizer, and water. The PH environment in the system initiates further reaction of the silica and alumina with the clay particles, hence providing extra strength to the stabilized soils. A minimum PH of 12.4 is necessary to maintain the pozzolanic reaction. Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (Emmanuel Emeka, 2015).



The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure which increases the soil strength (Meron & Samuel, 2014).

2.6 Commonly used types of stabilizers/Additives

- **Lime stabilization:** Lime is an inorganic chemical compound that is usually known as quick lime or unslaked lime obtained from a naturally occurring compound called limestone. The lime reacts with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, and increased strength. Strength gain is primarily due to the chemical reactions that occur between the lime and soil particles. When lime is added to soil, it has a beneficial effect on their engineering properties such as reduction in swell potential and plasticity, increasing of strength and stiffness, improving durability and workability of weak soil such as expansive soils (Musema, 2014).
- **Cement stabilization:** Portland cement is composed of calcium silicates and calcium aluminates that hydrate to form cementitious products. Portland cement stabilization, commonly referred to as soil cement, is a mixture of Portland cement, water, and soil compacted to a high density. cement stabilization differs from other forms of chemical stabilization in such a way that structural strength is primarily obtained from the cementing action rather than from internal friction, cohesion, chemical ion exchange, and/or waterproofing of the materials.
- **Fly Ash:** Fly ash is a byproduct of coal-fired electric power generation facilities; it is a chemical additive consisting mainly of silicon and aluminum compounds. It has little cementations properties compared to lime and cement. However, can be mixed with lime or calcium to get the required strength, durability, and stability of the soils (Berhane, 2018).

For many years, waste and recycled materials cause pollution and impact the environment and health of human beings. Nowadays, utilizing those materials as stabilizers is also one part of assessing the risk posed by these materials. Furthermore, using such materials for soil improvement is beneficial due to their availability and cost consideration.

2.7 Stabilization of soils by Agricultural and Industrial wastes materials

The accumulation of agricultural and industrial wastes possessed a problem to the environment. Therefore, these materials have valuable applications in the area of geotechnical engineering to improve/stabilize the soils due to their availability and cheaper costs. This research concerns the potential utilization of maize cob ash and wastes ceramic powder to improve some geotechnical properties of expansive soils. Industrial wastes such as blast furnace slag, fly ash, bagasse ash,

waste ceramic dust/ powder, and silica fume were used as cement replacement materials. The agricultural waste or industrial waste which is used for partial cement replacement is called pozzolanic materials. The agricultural waste products which possess pozzolanic properties as cement are fly ash, rice husk ash, palm oil ash, and sugarcane bagasse ash, and maize cob ash. The sum of SiO_2 , Al_2O_3 , and Fe_2O_3 in the chemical composition of maize cob ash and waste ceramic dust satisfies the pozzolanic requirements of ASTM C618, which is greater than 70% suggesting its good pozzolanic nature (P et al., 2015).

2.8 Properties of maize cob ash and waste ceramic powder

2.8.1 Maize (corn) cob ash

Maize is one of the cereal crops which is grown widely throughout the world as well as in Ethiopia and is the largest crop produced yearly. According to food and agriculture organization (FAO) data, 589 million tons of maize were produced worldwide in the year 2000 (Taffesse et al., 2011). The United States was the largest maize producer having 43% of world production. Africa produced 7% of the world's maize, South Africa has the highest production of 8.04 million tons. Ethiopia was the fifth largest producer of maize in Africa in the year 2020 with 8600 thousand of the ton. Maize body consisting of three natural parts; the chaff and pith covering and the wood ring which carries grains called maize cob. Maize cob is the part of maize on which the grains grow and is often discarded as waste after the grains have been removed, it is the waste product obtained from maize or corn, which is disposed of as useless materials and affects the environment as landfill wastes.



Figure 2. 6:Photographic view of Maize cob from disposed site

Maize cob ash (MCA) is cheaply available locally and is obtained through the burning of waste maize cob in the hole not as to affect the environment. Therefore, the use of maize cob ash as an additive for soil stabilization goes a long way in minimizing the cost of soil stabilization and it is an agricultural waste, which contains a high percentage of silica. Moreover, the sum of SiO_2 , Al_2O_3 , and Fe_2O_3 in the chemical composition of MCA satisfies the requirements of ASTM C618, suggesting its pozzolanic nature.

2.8.2 Related work on stabilization by Maize /Corn cob Ash

Nowadays, due to the increasing costs of traditional soil stabilizing agents and the need for economical use of agricultural and industrial wastes gets valuable acceptance for its workability and applicability. maize cob ash has also application in the production of concrete as a partial replacement for cement and sand. As reviewed from several studies, MCA is more economical and environment-friendly than cement stabilization; MCA stabilization has the potential for the long-term effect that increases the ultimate strength of the soil which results from the high content of pozzolanic materials (SiO_2 , Al_2O_3 , and Fe_2O_3) in the corncob ash. There were several literature/studies on using maize cob ash as soil stabilizers. Some of them are listed below.

P. Madan et al . , (2018) had studied on stabilization of expansive soils by using corncob ash, the researcher was used various percentages of maize cob ash (4%,8%,12%,16%,20%) by the

weight of dry soil and investigated its effect on, OMC, MDD, CBR, free swell index and consistency of expansive soils. From the result of the investigator, the MDD is increased and reached the maximum value at 12% of corn cob content by the dry weight of the soil. The UCS and CBR of the soil were also increased and the maximum value was observed at 12% of maize cob ash. finally, the researcher concluded that using the maize cob ash as an additive decrease swelling and increases the strength of expansive soils.

Sadeeq et al.,(2017) have done an investigation on modification of lateritic soil using corncob ash. The researcher understood the effect of corn cob ash on the Atterberg limits and compaction characteristics. The percentage of corn cob ash used was 0%, 3%, 6%, 9% by dry weight of natural soils. The researchers understand that the introduction of CCA into the soil increases the liquid limit and plasticity index and thereafter decreased. Increased Atterberg limit is due to the addition of CCA which introduced more pozzolanic substance into the specimen and decreased due to the agglomeration and flocculation of the clay particles which is as a result of exchange ions at the surface of the clay particles. The researchers also concluded that stabilization of lateritic soil with CCA showed a general increase in MDD and a decrease in OMC within increased CCA content.

Akinwumi I.I. & Aidomojie O.I, (2015) had studied lateritic soil stabilized with cement-corncob ash (CCA) to ascertain its suitability for use as a pavement layer material. The researchers have performed a series of laboratory tests by varying percentages of corncob from 0 to 12%, were carried out. the researcher understood that, Like the amount of CCA increases, it shows the reduction in plasticity, swell potential, and permeability of the soil but increased its strength. The researcher understood that CCA-stabilization, aside from being more economical and environmentally friendly than stabilizing with cement it also enhances the geotechnical properties of the soil for pavement layer material application.

Osuolale et al., (2019) have studied on strength and workability improvement potential of admixture of corn cob ash and cement for stabilizing lateritic soil, The researcher has investigated, the effect of CCA stabilized with cement on the geotechnical properties of soils such as compaction, unconfined compressive strength, and California bearing ratio, by using (0%,3%,6%,9%,12%) and (0%,2.5%,5%) of CCA and cement content respectively. Finally, the researchers investigate that the MDD and OMC of stabilized soil sample increased with

increasing the content of CCA and Cement except for 12%CCA at OMC was dropped. The peak MDD was obtained at 12% CCA and 5% cement and the OMC increased up to 9% and then decreased. Moreover, the UCS and CBR were increased with increasing content of maize cob ash and cement contents.

Gurjap et al . , (2017) have done an investigation on the effect of cement and corn cob ash on UCS and direct shear test of clayey soil, the researcher investigates that, Corn Cob Ash mixture strength improvement by observing its strength through UCS, Direct shear test of stabilized soils. The researcher investigated adding corn cob ash in varying amounts alongside optimized cement value. From the evaluation, the researcher obtained that, optimum mixture cement 2% and corn cob ash 6% compressive strength enlarged by 310% and 402% equated to virgin soil strength for 7- and 14-days curing spell respectively. Shear strength for optimum mixture increased by 64% equated to virgin soil strength for 14 days curing spell. Consequently, strength parameters were found to show a sufficient improvement in the strength features of the expansive soil.

2.8.3 Waste ceramic powder

The ceramic powder is the principal waste of the ceramic industry which is generated as unwanted dust during the process of dressing, polishing, and other processes. Ceramic dust is obtained waste from ceramic bricks, roof, and floor tiles, and stoneware waste industries. It is estimated that 15 to 30% of wastes are produced from the total raw material used. Disposing this large amount to the environment is harmful, thus utilizing them for improving geotechnical properties of expansive soil is very beneficial due to its availability and cheaper costs than cement/lime. The waste ceramic powder contains a large percent of silica which is the most essential element of cement replacing materials. As result, it has good pozzolanic properties, which can be used to improve the strength and durability of expansive soils. The waste ceramic powder can be obtained either from industries or by grinding the waste ceramic dust which is collected from its disposal site.



Figure 2. 7:Photographic view of Waste ceramic dust from the disposed site

2.8.4 Availability of Waste Ceramic dust powder in Ethiopia

Nowadays, ceramics are manufactured at different places in various parts of Ethiopia for different purposes. Those places are favored due to the availability of raw material in surrounding for production and other consideration. as the result of large production of ceramics, the corresponding dust ceramic powder as residue would take away by those factories. Some industries that produce ceramic for one purpose and others in Ethiopia are East African Zone (DIYUANN CERAMICS PLC), Ethio Ceramic PLC, Tabor Ceramic PLC, Arerity Industry Park, etc. DIYUANN CERAMICS PLC is found in Dukem town, which was established in March 2016 with the planning amount of 218 million USD, invested by China Rongguang Group to respond to the Belt and Road strategy by the Chinese government.

It involves the production of wall and floor tiles, sanitary wares, pottery, and other ceramic products. The factory is over 400,000 m² in the Eastern Industrial Zone of Dukem, around the study area. The production capacity is 60,000 m² per day and waste ceramic disposed of is also 9000m²-18000m²/day. The factory uses the resource material which comes from around Mojo and Bishoftu like Feldspar, Talk, Calcite, Quartz, Muscovite and other material which are necessary as the ingredient for ceramic production (Firdawek, 2019).

2.8.5 Related work on stabilization by Waste Ceramic dust powder

Akshaya, (2012) stabilization of expansive soil using waste ceramic dust: The researcher had investigated the effect of waste ceramic dust on some characteristics of the soils like Atterberg limit, compaction characteristics, unconfined compressive strength, California bearing ratio, shear strength, and swelling pressure of expansive soil was evaluated by adding the ceramic waste from 0-30% within an interval of 5%. From the test results, the researcher understood that the consistency/Atterberg limit was decreased. The compaction property was enhanced, in which the maximum dry density increased from 15 kN/m^3 to 18 kN/m^3 . OMC decreased from 20.4% to 17.6%. The UCS increases from 55 kN/m^2 to 98 kN/m^2 . The researcher also reported that the CBR value increased from 1.6% to 4%. There was a 150% increase in soaked CBR value. The cohesion decreased from 18 kN/m^2 to 13.5 kN/m^2 . Finally, the researcher had understood that the use of ceramic waste up to 30% was economical and enough to improve the strength of sub-grade pavements and the foundations.

Firdawek, (2019) stabilization of expansive subgrade soil using waste ceramic powder; The researcher had studied that, the suitability of waste ceramic powder for stabilization of weak expansive soils. The engineering properties of the subgrade expansive soils was determined from Atterberg limits, free swell, free swell index, free swell ratio, compaction, and California bearing ratio tests when 5%, 10%, 15%, 20%, 25%, 30% WCP was mixed by dry weight the expansive soil. From the test result of the researcher, the plastic index is slightly reduced with increasing in WCP content and optimum moisture content is decreased while MDD is increased. Free swell, free swell index, and free swell ratio of stabilized samples are decreased by increasing WCP. The CBR value was also increased with the addition of WCP.

Rakhil & Devi, (2016) review the effect of waste ceramic dust on the geotechnical properties of expansive soils; Based on the result and investigation of the researcher, the consistency limit goes on decreasing as the percentage of addition of ceramic dust. The MDD and UCS go on increasing and OMC goes on decreasing with an increase in the percentage of addition of ceramic dust. There is a 150% increase in soaked CBR value as compared to untreated soil when 30% ceramic dust was added. The cohesion is decreasing but the angle of internal friction increases as waste ceramic dust content increases. The swelling pressure show reduction with the addition of ceramic dust.

Table 2. 9:Summary of some related work on stabilization by MCA and WCD

Author	Research	Test Conducted	Result
Akshaya, (2012)	Stabilization of Expansive Soil Using Waste Ceramic Dust	Atterberg limit, Compaction test UCS, CBR, shear strength, and swelling pressure	Uses,5%,10%,15%,25% 30%WCD Atterberg limit was decreased MDD increased and OMC decreased Cohesion was decreased The UCS and CBR increased up to 30% Economical and enough to improve the strength of the soil
Firdawek, (2019)	Stabilization of Expansive Subgrade soil using Waste Ceramic Powder	Atterberg limits, Free swells Compaction, and CBR	Uses,5%,10%,15%,20%WCD plastic index was slightly reduced OMC, Free swell, free swell index, and free swell ratio was decreased MDD and CBR increased up to 20%
P.Madan et al. ,(2018)	Stabilization of Expansive Soils by Using Corncob Ash	Atterberg limit Free swell index Compaction CBR tests	Uses (4%,8%,12%,16%,20%) CCA MDD increased and the maximum at 12% of CCA The UCS and CBR increased and the maximum at 12% of CCA Decrease in PI and Swelling with %CCA
Sadeeq et al.,(2017)	Investigation on modification of lateritic soil using corncob ash	Atterberg limit Compaction Characteristics	Uses (0%, 3%, 6%, 9%CCA) Decrease in PI up to 9%CCA Increase in MDD and a decrease in OMC

2.9 Summary of literature review

Stabilization of expansive soil was done by different methods especially using commercial additives such as cement and lime. However, the cost of these materials has been increased from time to time. So, to fill that gap, this study was conducted by using easily available, low cost and environmentally friendly material from agricultural and industrial wastes that were utilized for the treatment of weak soil. The following idea was summarized from the previous researches.

- P. Madan et al . ,(2018) was stabilized expansive soil with CCA in 0%, 4%, 8%, 12 % and 20% amount. The optimum moisture content (OMC) decreased while MDD, UCS, and CBR increased and was reached the maximum at 12% of CCA. Generally, the results show the stabilization of black expansive soil with CCA was reduced plasticity, swell potential, and increased the strength of black cotton soil. However, this study was used only MCA to stabilize the expansive soil.
- Akshaya, (2012) The weak subgrade soil was treated with WCD, in 0%, 5%, 10%, 15 %, and 30%, Atterberg limit, swelling characteristics, OMC, and cohesion was decreased while MDD, UCS, and CBR value of expansive soil were increased with an increasing percentage of WCD. Thus, the studies showed that WCD was economical and enough to improve the strength of the soil. The research was limited to considering the effect of WCD only without the combination of MCA.

In many parts of the world, expansive soil has been investigated and stabilization was researched vastly. Those studies were focused on expansive soil investigation and evaluating the suitability of various chemical additives for weak soil stabilization. Many lightweight and subgrade pavement that was found around dukem town was damaged due to swelling and shrinking effect of expansive soil. Hence, it is necessary to treat this problematic soil by different techniques to construct a safe and stable foundation on it.

However, there was limited research done on the stabilization of weak soil by maize cob ash and waste ceramic powder. Some studies were conducted on the improvement of expansive soil using MCA and WCP separately. So, this study was conducted to assess the effect of MCA and WCP separately as well as in combination to improve the properties of the weak soil. Due to this, the study assured that MCA and WCP have a significant effect to improve the properties of expansive soil.

CHAPTER THREE

3. MATERIALS AND RESEARCH METHODOLOGY

3.1 Introduction

In this chapter, the materials used and the method employed for the study were described with respect to their sources.

3.2 Study Area

Dukem town is one of the recently established towns which is located at about 37km, South East of Addis Ababa along the main road to Adama. The Geographical condition of the town is approximately 08⁰48'N latitude and 38⁰53'E longitude. The town has a temperature of 11.9-25.83°C with an average annual rainfall of 606.13 mm. The town is located in an area of the altitude of 1959m above sea level. The area is found near the western border and experiences the temperate climate of the central Rift valley of Ethiopia. Dukem is located in the main Ethiopian rift valley near the western escarpment. The area is covered by volcanic rock materials with various compositions. Due to weathering of the rock, and corresponding erosion and deposition, thick residual clay and silty clay soil covered most of the plain topographic landforms (Ambaye, 2012).

The town is planned to be one of the largest industrial areas in Ethiopia. In the town, different huge factories such as East Africa Bottling, Elsewedy cable factory, marble factory, etc. have been constructed and many more are planned. According to the master plan of the town, many constructions were carried out in the town. East industry zone is one of the biggest projects to be constructed by Chinese firms. In this industrial zone eighty (80) different types of factories have been included. In addition to this, the other two industry areas are prepared for Ethiopia and foreign investors. Each industry area contains around thirty (30) factories. Moreover, the Addis-Adama new railway and the express highway are passing through the town.

The soil profile of the town can be divided into expansive and non-expansive soils. Approximately 80% of the town is covered by black clay soil, 15% of the area is covered by mountains which mainly consist of cinder and gravel, the remaining 5% is covered by red clay soil, around 50% of the existing structures constructed on black clay soil show appearance of

cracking. Generally, expansive soil covers a wider area of the town. These expansive soils are clay soils that may be black or gray. It has the potential to swell and shrink with the variation of moisture content. This may cause damage to lightweight buildings and road subgrade pavements (Tamrat, 2013).

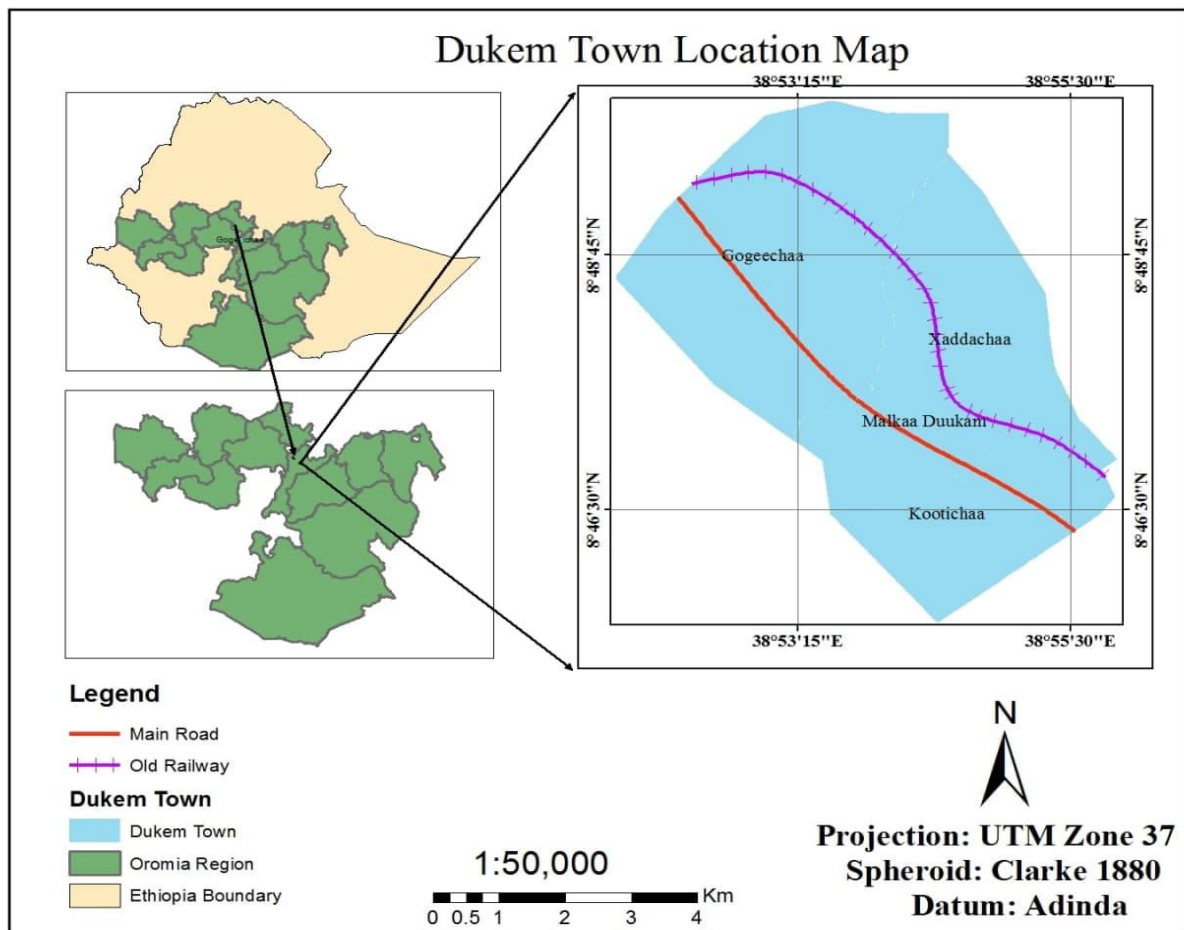


Figure 3. 1:The location map of the study area

3.3 Materials

Three materials were used for the successful accomplishment of these studies. The first one is expansive soil from two test pits, the second is materials involved in the stabilization process like the sample of maize cob ash, Waste ceramic dust powder and explained in detail in the next section. Hence, Soil sample collection, the sample of maize cob ash, and waste ceramic dust powder preparation process, and any further activities were discussed in the following section.

3.3.1 Expansive soils (Black cotton soils)

The soil samples were collected from Dukem town at Tadecha kebele around china's eastern industrial zone along the boundary of Bishoftu and 2km away from Dukem to Addis Ababa Road. The sample was collected below a depth of 1.75m from ground level. The soil is grayish-black in color which is highly plastic clay. Disturbed soil samples were collected from two stations; the two stations are 200 meters apart from each other at a depth 1.75m below NGL to avoid the inclusion of organic matter.



Figure 3. 2:Location of test pits in the study area



(a)



(b)



(c)



(d)

Figure 3. 3: Photo of soil samples from test pit one and two

(a) and (c) Overview of existing crack

(b) and (d) Sample of test pit-1 and test pit-2

3.3.2 Maize cob ash

The maize cob used for this study was obtained from Awash melkasa Agricultural research institute, after harvest season, the maize cobs were sun-dried. The collected samples were then burnt until it was turned to ash by open burning on a metal sheet to prevent the ash from mixing with dust materials. the burnt maize cob grounded after cooling using manual method and the burnt ash was sieved through 425 micrometers. The fine ash was collected and stored in polythene bags to prevent pre-hydration from the atmosphere. The process of maize cob ash preparation is shown in figure 3.4, as below.

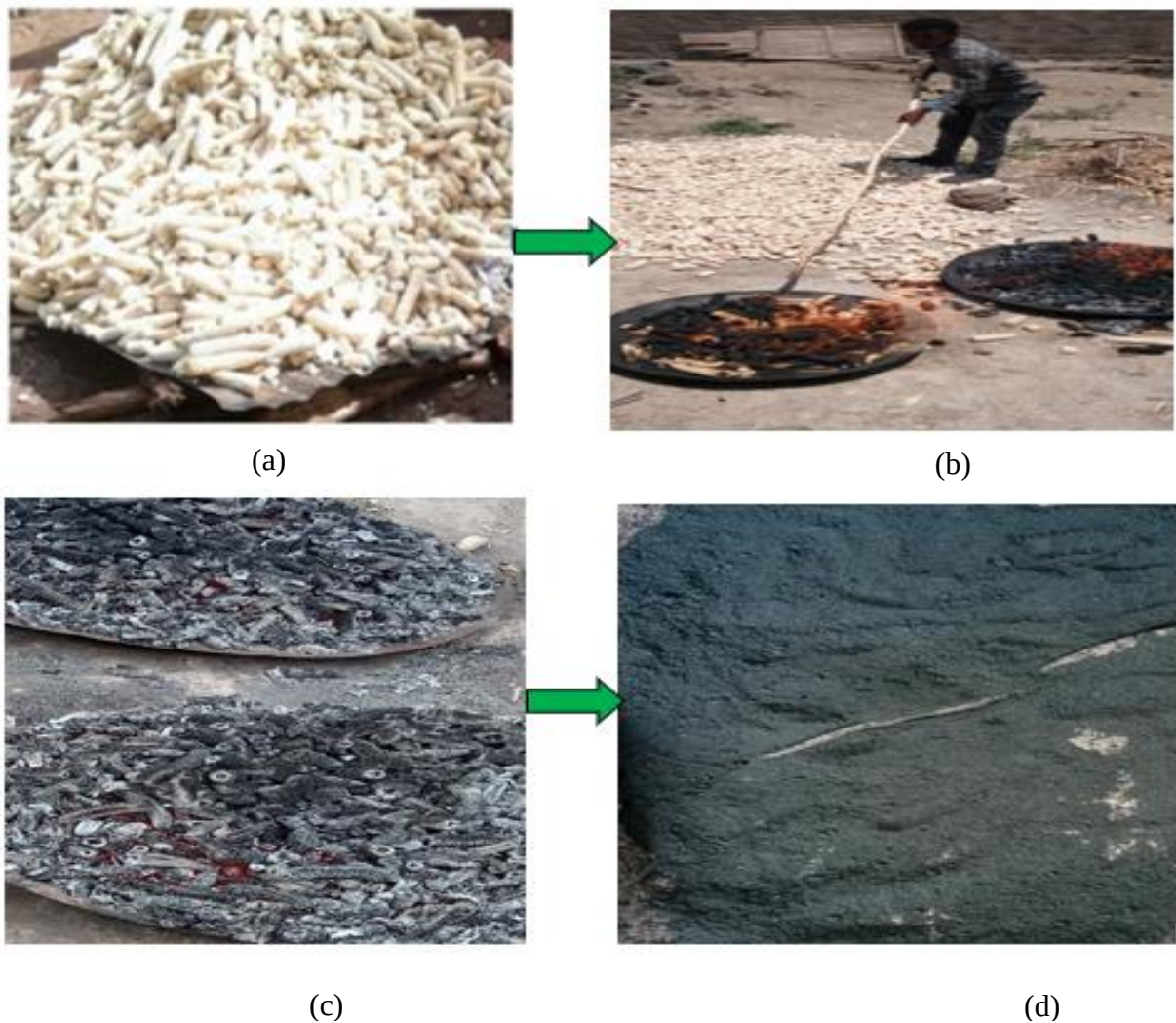


Figure 3. 4: Process of maize cob ash (MCA) sample preparation

3.3.3 Waste ceramic powder

The waste ceramic powder as shown below was obtained from East African Industry (DIYUANN CERAMIC PLC) which is located in Oromia regional state in Dukem town. For this research, 10kg of waste ceramic dust was taken from the wastage accumulated and change to powder using the grinder and then, it was properly collected into sacks and transported to the laboratory.



Figure 3. 5: Photo of waste ceramic dust from the disposed site

3.4 Study design

The research study was performed by applying both experimental and analytical methods. It is dependent on experimental work which is started by collecting samples that are used to accomplish the whole task. The steps in the study comprise of:

- Gathering of information and observation of the study area
- Collecting of expansive soil samples and stabilizers
- Preparing the samples for laboratory tests as per testing standard
- The blending of maize cob ash and waste ceramic powder with the expansive soil separately.

- Analyzing the effect of both materials separately and in combination on some geotechnical properties of the soil by performing different laboratory tests
- Drawing conclusions and recommendations based on their effect on the properties of soil.

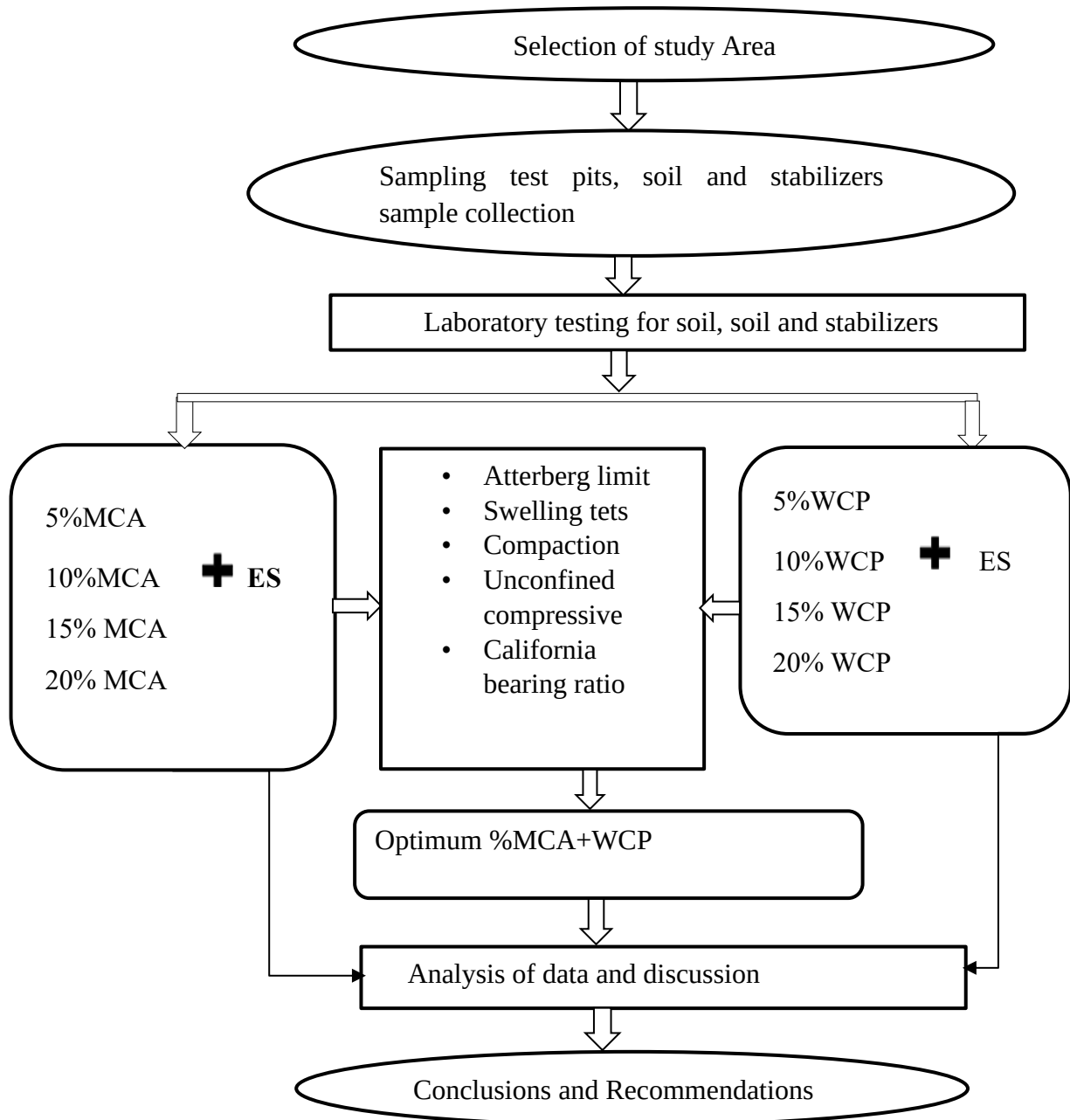


Figure 3. 6: Description flow chart of the study

3.5 Methodology

The present study started by setting a clear framework of the research by reviewing secondary resources from the literature, research bulletins, and previous related studies. Samples of soil that represent expansive soil were collected from Tedacha kebele located 1km away from Dukem –Addis Ababa main road. Primary data was collected for the study by performing a series of laboratory tests. Treatment in the properties of the soil was assessed using Atterberg limit, swell characteristics, compaction, unconfined compressive strength tests, and CBR. In addition, index tests were conducted to characterize both test pit soil samples. Expansive soils were treated in the following mixing ratios; 0%, 5%, 10%, 15%, and 20% of maize cob ash and waste ceramic powder by dry weight of the soils, then separate, combined effect, and comparative analysis were made.

3.6 Sample preparation and Experimental program

The soil samples were first air-dried, properly pulverized and stabilizers were mixed in such a way that the additive is first added to the prepared sample and dry mixed with the soil. It could be done either by air drying of samples or oven drying. Breaking up the soil aggregates by the rubber-covered mallet. Then, sieve analysis is performed to separate the dried soils into two parts. The first group involves preparing uniform samples for Atterberg limits, Specific gravity, free swell, and free swell index, and free swell ratio tests. And the other for compaction, unconfined compressive strength, and CBR values.

Table 3. 1:Experimental program

Samples	Expansive soil (%)	Maize cob ash (%)	Ceramic Powder (%)
Expansive soils	100	0	-
Expansive soils-Maize cob ash	95	5	-
	90	10	-
	85	15	-
	80	20	-
Expansive soil- Waste ceramic powder	95	-	5
	90	-	10
	85	-	15
	80	-	20
Combined effects	Optimum MCA (15%) mixed with 5%-20% of WCP		

3.6.1 Moisture content

The moisture content of the soil is defined as the ratio between the mass of water to the mass of soil solid which was determined quickly after the sample was taken from the site. The samples were excavated from two test pits and collected in a plastic bag to prevent moisture loss during transportation from site to laboratory. The method employed for determining the moisture content was the oven drying as ASTM D 2216 standard method. The measured amount of wet soil was put in a thermostatically oven at $110 \pm 5^{\circ}\text{C}$ and kept for 24 hours and examined for weight loss.

3.6.2 Grain Size Analysis

This test is conducted to determine the percentage of different grain sizes contained within a soil. The wet sieve analysis was performed to determine the distribution of the coarse, larger-sized particles, and the hydrometer method is used to determine the distribution of finer soil particles. For this study, both wet sieve analysis and hydrometer analysis was done according to ASTM D422-63.

3.6.3 Specific Gravity

Specific gravity is the measure of the heaviness of the soil particles and is determined by the method of small pycnometer method using a soil sample passing through a 2mm (No.10) sieve and oven-dried at 110 ± 5 degrees centigrade. Specific gravity is also defined as the ratio of the mass of the unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. The specific gravity of the samples was conducted according to ASTM D 854-83.

3.6.4 Atterberg limit tests

An Atterberg limit test was conducted according to the ASTM D-4318 standard. An Atterberg limits device was used to determine the plasticity index of the soil using the soil passing through a $425 \mu\text{m}$ (No. 40) sieve size. The test includes the determination of the liquid limits, plastic limits, and the plasticity index for the natural soil and the soil- additive mixtures. Then, the samples of soil were subjected to Atterberg limits testing to determine the plasticity of the soils. The plastic limit of soil was determined by using soil passing through a $425 \mu\text{m}$ sieve and rolling 3-mm diameter threads of soil until it began to crack. Generally, for this study, the liquid limit

and plastic limit tests were conducted on the expansive soil and treated soil samples. Then liquid limit and plasticity index was also used to classify soils.

3.6.5 Particle Size Distribution

The soil classification was made according to ASTM D-2487 and AASHTO soil classification standards. The test was conducted for the determination of the particle size distribution for the natural soil. Approximately, 50gm of dry soil sample passing No. 200 sieves is treated with a dispersing agent for 18 hours. Hydrometer analysis is then conducted to determine the amount of silt and clay size particles. The sample is then washed through a series of sieve sizes to determine the percentage of sand-sized particles in the specimens.

3.6.6 Free Swell Index

The test includes the determination of the free swell index of the natural soil and the soil- waste ceramic powder and maize cob ash. It is also one of the most commonly used simple tests to estimate the swelling potential of expansive clay soils. The tests were conducted following IS 2720 (Part 40) 1977 testing procedure. Two samples of the oven-dried soil 10cc each, passing through a 425-micron sieve are taken. The first sample is put in a 100cubic centimeters graduated glass cylinder containing kerosene. The second sample is also collected in a similar cylinder containing distilled water. Both the samples were left undisturbed for 24 hours and then their volumes are noted. Then free swell index is determined using the equation below. The same procedure is also carried out for the treated soil with an increment of both stabilizer materials.

$$\text{Free swell index(\%)} = \frac{V_k - V_w}{V_k} * 100 \dots\dots\dots (3.1)$$

Where: V_w = final volume in water

V_k = final volume in kerosene

3.6.7 Free swell test

The test includes the determination of the free swell for the natural soil and waste ceramic powder as well as the maize cob ash mixture. The test had not been given by AASHTO and ASTM standards till today. The method was suggested by Holtz and Gibbs, (1956) to determine the expansive potential of cohesive soils. The free swell test gives a fair approximation of the

degree of the expansiveness of the soil sample. The procedure consists of pouring very slowly 10 cubic centimeters of that part of the dry soil passing No. 40 sieves into 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately 24 hours until all the completely settles on the bottom of the graduating cylinder. Then, the final volume of the soil is observed.

$$\text{Free swell} = \frac{V_f - V_i}{V_i} \dots\dots\dots (3.2)$$

Where: V_i = initial volume

V_f = final volume

3.6.8 Free Swell Ratio Tests

The free swell ratio is determined following IS: 2720 (Part 40) 1977. For this study, 10gm oven-dried soil with each stabilizer was passing through 425 microns is added to 100ml of distilled water in a jar and another 10gm of the same sample is added to 100ml of kerosene. Then, the free swell ratio is determined by measuring the sediment volume sample taken after 24 hours. It is the ratio of change in volume in water to change in volume in kerosene.

$$\text{Free swell ratio} = \frac{V_w}{V_k} * 100 \dots\dots\dots (3.3)$$

Where V_w = final volume in water,

V_k = final volume in kerosene

3.6.9 One dimensional consolidation test

A consolidation test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. This test is conducted according to the ASTM D2435 standard testing procedure for one-dimensional consolidation properties of soils. From the recorded data, the consolidation curve (pressure-void ratio relationship) can be sketched. This data is useful in determining the compression index, the recompression index, and the coefficient of consolidation. Moreover, one-dimensional consolidation /the oedometer test was conducted to determine, parameters such as compression index (C_c), recompression index (C_r), coefficient of consolidation (C_v), and coefficient of volume change (mv) for test pit one of expansive in the study area.

Compression index

The compression index, C_c , is equal to the slope of the straight portion of the void ratio versus log pressure plot and is given as follows.

$$C_c = \frac{e_1 - e_2}{\log p_2 - \log p_1} \dots \dots \dots (3.4)$$

Coefficient of consolidation: For a given load increment, the coefficient of consolidation C_v can be determined from the laboratory observation of time versus dial reading. There are two graphical methods commonly used for determining it.

Logarithm-of-time-fitting method: The coefficient of consolidation is determined by Casagrande's semi-logarithmic plot method which is given below.

$$C_v = \frac{(0.197) * (H^2 dr)}{t_{50}} \dots \dots \dots (3.5)$$

Square-root-of-time fitting method: The coefficient of consolidation value as determined by Taylor's square root of time fitting method and given by:

$$C_v = \frac{(0.197) * (H^2 dr)}{t_{90}} \dots \dots \dots (3.6)$$

3.6.10 Standard Compaction test

Compaction test includes the determination of the maximum dry density and the optimum moisture content for the natural soil and mixing it with both a stabilizer at varying content of 5% up to 20% in accordance with the ASTM D2937-00 testing standard. The maximum dry density is performed for the natural soils and stabilizers, the mixture of about 2.5kg, by varying the moisture content. The sample is then compacted into 944 cubic centimeters, in three layers of approximately equal mass with each layer receiving 25 blows. One small representative sample is then taken from the top, middle, and bottom of compacted soil for the determination of moisture content. Then, moisture content that is corresponding to maximum dry densities, is taken from the graph of dry density versus moisture content, which gives the optimum moisture content of the sample.

3.6.11 Unconfined compressive strength

Unconfined compressive strength was conducted as per ASTM D-2166 standard. The unconfined compressive strength test of the samples was prepared with optimum moisture and maximum dry density obtained for the respective sample from the standard proctor test. The test was performed with a sample having a dimension of 38mm diameter and 76mm height.

Unconfined compressive strength testing was performed on all extracted specimens using a strain rate of approximately one percent per minute. Then, applied load and deformation were recorded. The Corrections to the cross-sectional area were applied before calculating the compressive stress on the specimen. Each specimen was loaded until maximum stress was obtained. For this study, UCS was conducted for natural and stabilized soil with MCA and WCP according to the above procedure.

3.6.12 California Bearing Ratio (CBR) and CBR swell test

The CBR and CBR- swell tests were conducted as per AASHTO T193-93 for the natural soils and soil, WCP, and MCA mixtures. The CBR is expressed as the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between load and penetration. A 5.0kg of the natural soil and the soil-stabilizer materials mixtures were added at their respective optimum moisture contents in 2124 cubic centimeters mold. The samples are then compacted in five layers with 56 blows from the standard compactor. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the soaked CBR value of the soil, and soil- Stabilizer mixes. The test is assumed as a cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/minute. Then, the maximum value calculated for penetrations at 2.54mm and 5.08mm were recorded as the CBR value. Hence, the CBR value can be determined by the following formula.

$$\text{CBR (\%)} = \left(\frac{x}{y} \right) * 100 \dots\dots\dots (3.7)$$

Where: ‘x’ = material resistance or the unit load on the piston for 2.54 or 5.08 mm of penetration ‘y’=standard unit load (pressure) for well-graded crushed stone.

The standard load for 2.54 mm and 5.08mm penetrations are 6.9Mpa and 10.3Mpa respectively. The CBR swell of the soil and stabilizer were also done by putting the tripod with the dial indicator on the top of the soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the CBR swell of the soil. The initial dial reading of the soil of the dial indicator is taken just before soaking the sample. At the end of 96 hours, the final dial reading of the dial indicator is taken hence the swell percentage of the initial sample length is 116.3mm, Then CBR swell is computed as:

$$\text{CBR swell (\%)} = \frac{\text{change in length during soaking}}{116.3} \dots\dots\dots (3.8)$$

for which 116.3mm is the height of the soil specimen.

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 Introduction

This chapter presents the results of laboratory tests and a discussion pertinent to the results. The relevant engineering properties of the soil are evaluated both for the natural and stabilized state of soil samples with maize cob ash and waste ceramic powder by taking soil samples of test pit one. The laboratory tests include Atterberg limits, specific gravity, free swell, free swell index, free swell ratio, one- dimensional consolidation, moisture density relationship (compaction), unconfined compressive strength, and California bearing ratio (CBR) in which it was conducted as per ASTM and AASTO standard.

4.2 Properties of Material Used in the Study

Materials used in this study were discussed with their properties including expansive soils and material stabilizers.

4.3 The overall characterization of natural Expansive Soils

The laboratory test results conducted for the determination of properties of the natural soil before mixing with maize cob ash and waste ceramic powder are presented in Table 4.1. For this study, TP-1 soil was taken for treatment because it is weaker than TP-2. Generally, for test pit -1, the soil is grayish-black in color, as shown in Figure 4.1, on the particle size distribution curve almost 96.8% of the soil is passing through No. 200 sieves; it exhibits a liquid limit of 96.67, a plastic limit of 36.9%, and plasticity index of 59.8%. from these tests result the soil is highly expansive and highly plastic clay. Accordingly, the soil falls under the A-7-5 soil class based on AASHTO and CH as USCS soil classification system. The results of swelling characteristics also indicate that the soil is highly expansive clay with a free swell of about 130% and a free swell index of 109%. The soil has a maximum dry density of 1.17 g/cm³, optimum moisture content of 33%, The soil also possesses, unconfined compressive strength of 59.3KN/m², soaked CBR and CBR swell of 1.0% and 14.35% respectively.

one-dimensional consolidation test is also conducted on remolded soil samples, according to this test, compression index C_c , of the soil is, 0.526, coefficient of consolidation, C_v , range from

0.05-2.63m²/yr, the coefficient of volume change, Mv, of the area is 0.2046, depending on these data the soil possesses medium to high settlement and compressibility properties.

Table 4. 1:Geotechnical properties of the natural expansive soils of TP-1 and TP-2

Properties	Test pit-1	Test pit-2
Natural moisture content, %	35.2	29.3
Percentage passing No.200 sieve, %	96.8	94
Liquid limit, %	96.67	94
Plastic limit, %	36.9	39.2
Plastic index, %	59.8	55
AASHTO/USCS soil classification	A-7-5/CH	A-7-5/CH
Specific gravity	2.71	2.73
Free swell, %	130	124
Maximum dry density, g/cm ³	1.17	1.21
Optimum moisture content, %	33.13%	41.8
Unconfined compressive strength,	59.3Kpa	64.2Kpa
Soaked CBR Value, %	1.0	1.2
CBR Swell, %	14.35	8.99

- Depending on the above properties determined for both test pits, From the two test pits, pit one (TP-1) have used for taking a soil sample to investigate the effect of maize cob ash and waste ceramic powder because test pit one is weaker and slightly higher expansive than test pit two by depending and comparing of investigated properties. The detailed geotechnical properties of both soil samples were discussed below and given in the Appendix part. Moreover, discussion on stabilization of test pit one with maize cob ash and waste ceramic powder is discussed in the following section.

4.3.1 Particle size distribution for natural soil

The basic use of a soil classification system is to identify the soil type as well as determination of the amount and distribution of the particle sizes in the soil. The distribution of particle sizes larger than 0.075 mm (No. 200 sieves) is determined by wet sieving and particle sizes smaller than 0.075 mm are determined by hydrometer test. Thus, wet sieving was done taking 500g of

natural soil, by washing on sieve size of 75 μ m, then hydrometer test is conducted on 50gm of soil sample passing sieves No.200 and soaked in sodium hex-meta phosphate chemical solution for 24 hours. The tabular laboratory results are presented in the appendix and the particle size distribution curve is shown in Figure 4.1.

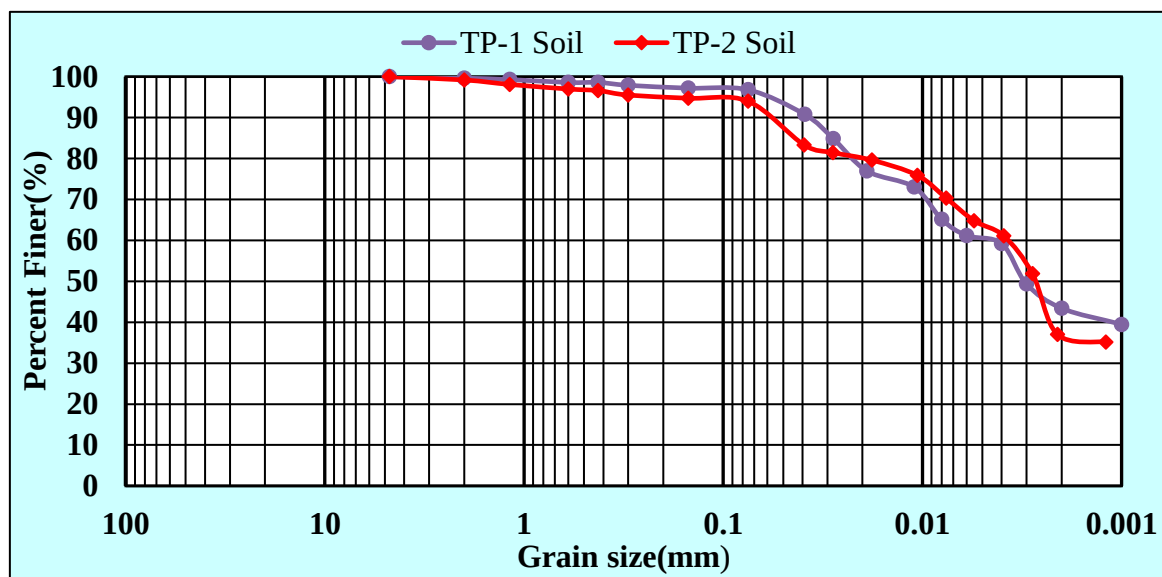


Figure 4. 1:Particle size distribution curve of the Expansive soil for TP1 and TP2

4.3.2 Liquid limit and plastic limit of natural soil

Atterberg limits test was conducted according to ASTM 4318 standard test method. The detailed tabular results of the Atterberg limits tests for both natural soils were shown in the appendix and Table 4.2 as shown below.

Table 4. 2:Summary of Atterberg limit test results of the natural expansive soils

No.	Designation	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)
1	TP1	96.67	36.9	59.8
2	TP2	94	39.2	55

From the Atterberg limit tests for test TP-1, as shown in Table 4.2, the soil sample changed from liquid state to plastic state at an average liquid limit of 96.67%, changed from plastic state to semisolid state at an average plastic limit of 36.9%, and the plastic index of 59.8%. The result obtained from the plastic index indicates that the soil possesses high swelling properties since its plasticity index is greater than 35%, thus the soil needs treatment for the construction of any foundation on it.

4.3.3 AASHTO and USCS Classification of the soils

i) AASTO Soil classification system

AASHTO system classifies soil broadly into granular material and silt-clay material. The granular material is further divided into three groups which are called A-1, A-2, and A-3. The silt-clay material is in turn divided into four groups namely, A-4, A-5, A-6, and A-7. The AASHTO system uses similar techniques as that of USC but the dividing line has an equation of the form $PI = LL - 30$. As it can be observed from the AASHTO soil classification system, the plasticity chart is shown below in Figure 4.2.

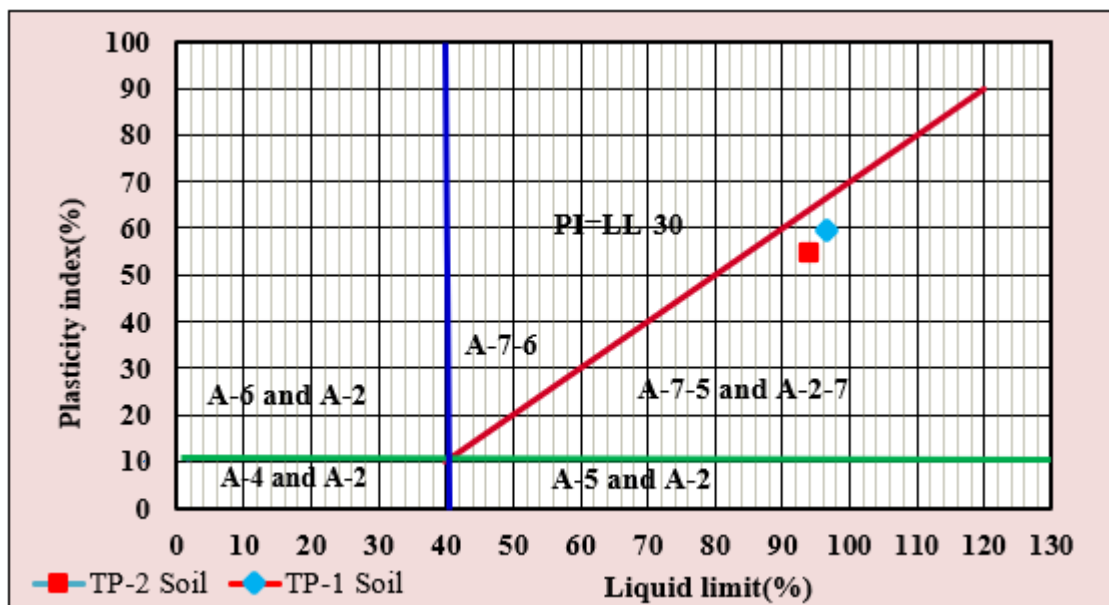


Figure 4. 2: Soil classification according to AASTO system

ii) Unified soil classification

USC is a system for classifying minerals and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, depending on liquid limit and

plasticity index and the system uses similar techniques as that of AASTO but the dividing line has an equation of the form $PI=0.73(LL-20)$. As it can be observed from the AASHTO Soil Classification system the plasticity chart is shown in Figure 4.3 below.

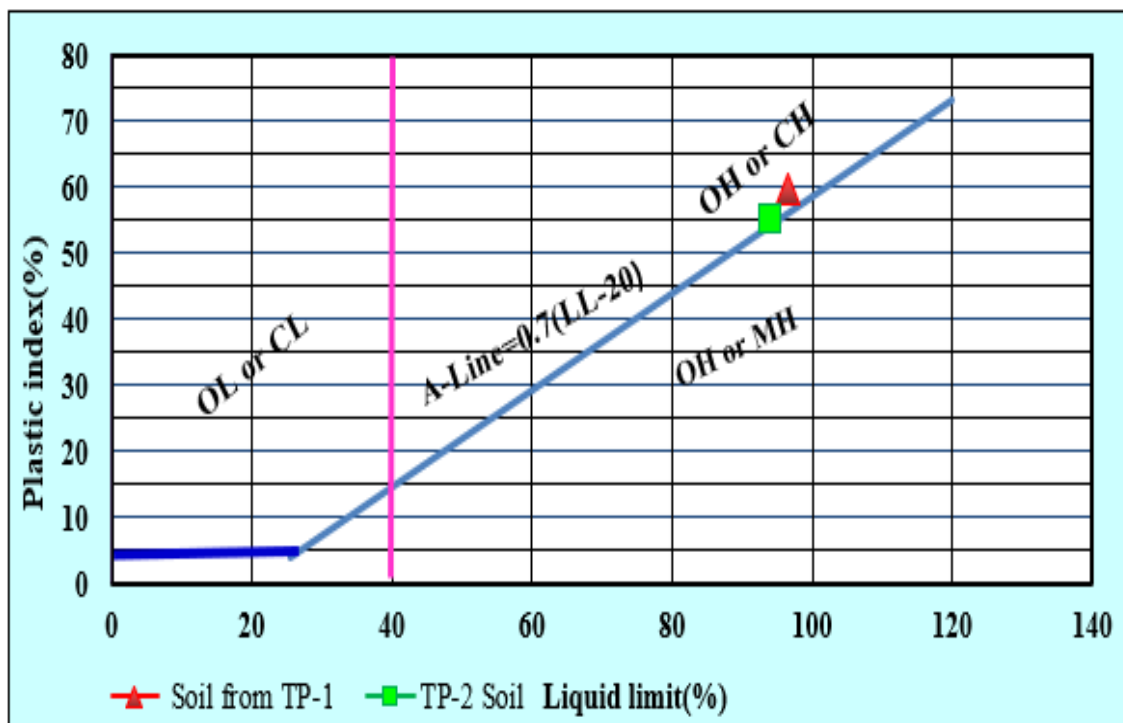


Figure 4. 3:Soils classification according to USCS system

4.3.4 The specific gravity of natural Expansive soils

The Specific gravity of the soil particles and stabilizer were determined by the method of pycnometer using a soil sample passing No. 10 sieves and oven-dried at 105 degrees centigrade. For this study, an average specific gravity of the soil sample is given in the Appendix and Table 4.3 below.

Table 4. 3:Summary of specific gravity of natural soils

Sample No.	TP-1	TP-2
1.	2.7	2.86
2.	2.73	2.6
Average (Gs)	2.71	2.73

4.3.5 Free swelling characteristics for natural soil

Free swelling tests (free swell, free swell index, and free swell ratio) are simple laboratory tests as discussed in chapter two, that are used to determine the expansion and swelling nature of expansive soil as well as after the addition of MCA and WCP stabilizers.

Table 4. 4: Summary of swelling properties for natural soils

Free swelling for natural soil	TP-1	TP-2
Free swell (%)	130	124
Free swell index (%)	109	103.6
Free swell ratio	2.1	2.03

For TP-1, from the laboratory test result, as shown in the table above, the soil under investigation has a free swell index of 109%, which indicates that it is highly expansive and a free swell ratio of 2.1, which also shows that the soil possesses high swelling characteristics. Furthermore, a free swell index of 130% shows, the soil can cause considerable damage to lightly loaded structures. Generally, the soil is very weak for the construction of subgrade and foundation as the result it needs the possible treatment.

4.3.6 Compaction test result for natural soil

Standard proctor compaction tests were performed on the soil to determine the relationship between the moisture content and dry density according to ASTM D-698. The natural expansive soil sample under investigation (TP-1) has an optimum moisture content of 33.1% and a maximum dry density of 1.17gm/cm^3 as shown in figure 4.4. Furthermore, the detailed tabular form is shown in Appendix Part.

Table 4. 5:Results of compaction test for the natural Soils.

Designation	MDD (g/cm^3)	OMC (%)
TP-1	1.17	33.1
TP-2	1.21	41.8

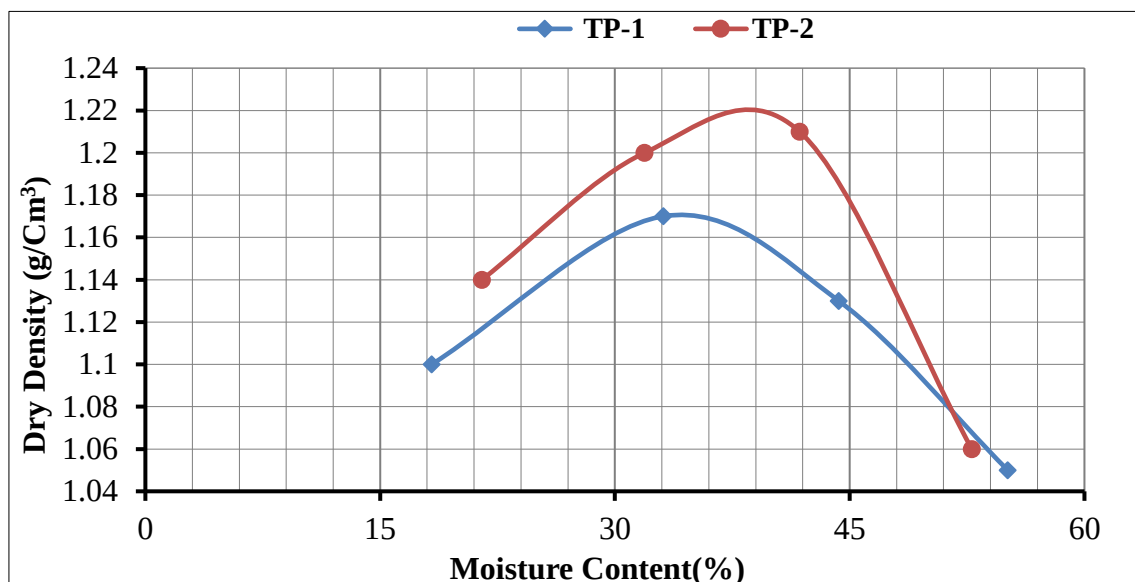


Figure 4. 4:Moisture-density relationship for the natural soils

4.3.7 Unconfined compressive strength test on the natural soils

Unconfined compressive strength (UCS) test is an unconsolidated and undrained load test conducted for determining the unconfined compressive strength of cohesive clay soil samples where the lateral confining pressure is equal to zero during the test. UCS is the compressive stress at which an unconfined cylindrical soil sample fails in a load test and is taken as the maximum load attained per unit area during loading. For UCS test specimens' height-to-diameter ratio is between 2 and 2.5, the undrained shear strength was calculated to be half of the compressive stress at failure. The soil sample under investigation (TP-1) has a maximum unconfined compressive strength of 59.3Kpa and undrained shear strength of 29.7KPa as shown in the table and graph below.

Table 4. 6:Unconfined compression strength test result for natural soils

Designation	UCS(Kpa)	Cu (Kpa)
TP-1	59.3	29.7
TP-2	64.2	32.1

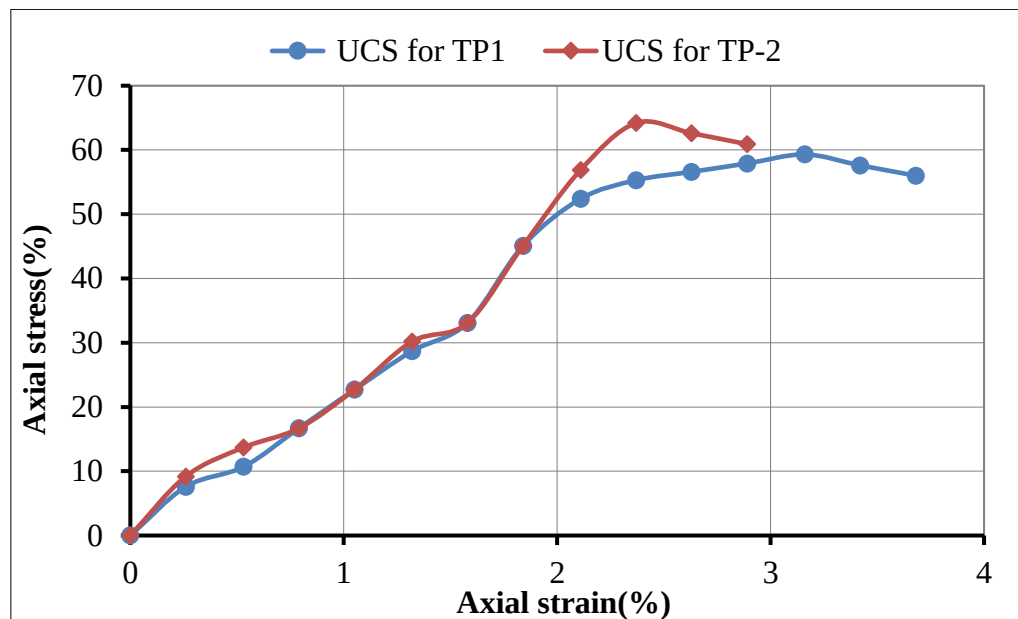


Figure 4. 5:Axial strain Vs Axial stress for natural Soils

4.3.8 California Bearing Ratio (CBR) test result for expansive soils

The CBR tests were conducted according to AASTOT193-93, The CBR test samples were prepared with optimum moisture and maximum dry density with CBR mold for natural soil as well as stabilizer-soil mix.CBR test result for natural soil is shown below and detailed laboratory result was given in the table of Appendix.

Table 4. 7:CBR test result of the expansive natural soils samples

Designation	CBR	CBR Swell
TP-1	1.0%	14.35%
TP-2	1.2%	8.99%

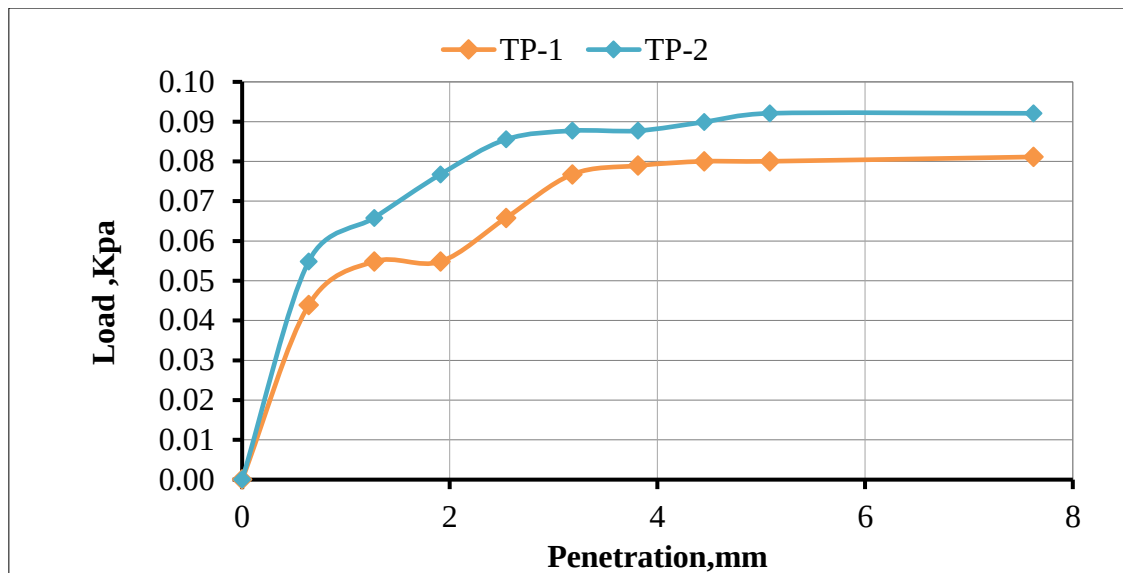


Figure 4. 6:CBR test result for natural soils

4.3.9 One dimensional consolidation test result for natural soil of the Study Area

One- dimensional consolidation test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. This test is conducted according to ASTM D2435 standard testing procedure to know the consolidation properties of soils. Depending on the consolidation test result the study area exhibits coefficients of volume compressibility from (0.060-0.334), the compression index and recompression index of 0.526 and 0.134 respectively, furthermore it has consolidation coefficients from (0.05-2.63), indicating that the soil possesses medium compressibility and consolidation behavior.

Table 4. 8:Summary of consolidation test result for expansive soils of the Study Area

Consolidation test result for Expansive soil of the Area						
No.	Pressure, P(KPa)	Void ratio (e)	Coefficient of volume compressibility (M_v)	Coefficient of consolidation, C_v (m^2/yr)	Compression index, C_c	Recompr. index
0	0	1.589	0.334	-	0.526	0.134
1	160	1.451	0.306	2.63		
2	320	1.331	0.212	0.59		
3	640	1.172	0.103	0.34		
4	1240	1.038	0.060	0.29		
5	2560	0.878		0.05		
6	640	0.960				
7	160	1.041				

4.4 Properties of materials used with soils

4.4.1 Chemical properties of maize cob ash (MCA)

The Chemical composition of MCA as investigated by various researchers from different countries in the world is shown in the table below. As can be seen, the main chemical element of MCA is SiO_2 which is approximately 64% to 68% of the total composition. The next chemical compound is Al_2O_3 which is around 6% to 12% and 3% to 6% Fe_2O_3 of MCA chemical constituents.

Table 4. 9:Chemical composition of MCA from different previous studies.

Constituents	% Composition
SiO ₂	63.5 - 67.85
Al ₂ O ₃	5.8 - 11.21
Fe ₂ O ₃	2.95 - 5.19
CaO	3.5 - 10.24
MgO	1.82 - 2.55
SO ₃	1.1 - 2.53
K ₂ O	4.20 - 8.42
Na ₂ O	0.39 - 0.45

As the test result indicates the combined percent composition of main oxides (SiO₂, Al₂O₃Fe₂O₃) is above the minimum specified by ASTM (C618) which is acceptable as a good pozzolanic property.

4.4.2 The specific gravity of maize cob ash

A specific gravity test was performed on maize cob ash samples that were used for the study and a summary of the test results is given in Table 4.10.

Table 4. 10:Specific gravity of maize cob ash

Specimen number	1	2
Pycnometer bottle number	T20	T15
WP = Mass of empty, clean pycnometer (grams)	38.3	37.8
WPS = Mass of empty pycnometer + dry soil (grams)	48.3	47.8
WB = Mass of pycnometer + dry soil + water (grams)	133.04	135
WA = Mass of pycnometer + water (grams)	139	141.2
Specific Gravity (GS)	2.47	2.63
Average specific gravity (GS)	2.55	

Table 4.10, shows the average specific gravity of maize cob ash was 2.55, which is less than the specific gravity of expansive soil.

4.4.3 Properties of waste ceramic dust powder

The sample of waste ceramic powder was collected from the local industry from Dukem East African (China) Industry was used in the laboratory tests by blending with expansive soil. The main composition element of waste ceramic powder is SiO₂(67.51%) and Al₂O₃ (16.92%) and gives a total of 84.43% which is greater than 70% that implies it satisfies the criteria of pozzolanic.

Table 4. 11:Oxide composition of waste ceramic powder

(Sourced from Ethiopian Geological Survey).

Constituents	% Composition
SiO ₂	67.51
Al ₂ O ₃	16.92
Fe ₂ O ₃	0.75
CaO	3.13
MgO	1.82
K ₂ O	1.31
Na ₂ O	4.8
P ₂ O ₅	0.023
SO ₃	0.01
Cl ₂	0.043
MnO	0.12
TiO ₂	0.04
L.O. I	1.2

4.4.4 The specific gravity of Waste ceramic powder

A specific gravity test was performed on waste ceramic powder samples that were used for the study and a summary of the test results is given in Table 4.12.

Table 4. 12: Specific gravity of waste ceramic powder

Specimen number	1	2
Pycnometer bottle number	T13	T16
WP = Mass of empty, clean pycnometer (gm)	41.6	43
WPS = Mass of empty pycnometer + dry soil (gm)	51.6	5
WB = Mass of pycnometer + dry soil + water (gm)	143.1	145
WA = Mass of pycnometer + water (gm)	136.4	138.5
Specific Gravity (GS)	3	2.86
Average specific gravity (GS)	2.93	

4.5 Laboratory test results of stabilized weak Expansive soil

This section deals with evaluating the effect of maize cob ash and waste ceramic powder on some characteristics of soil by using 5%, 10%, 15% 20% of MCA and WCP separately in similar proportions by dry weight of the soil.

4.6 The laboratory test result of Expansive soil blended with maize cob ash

The effect of maize cob ash on the properties of expansive soil was evaluated by performing the Atterberg limit tests, free swelling tests, standard compaction tests, unconfined compressive strength, CBR tests, and the results for each test was discussed in the following section.

4.6.1 Effect of maize cob ash on the Atterberg limits of treated soil

The effect of maize cob ash on the plasticity index of the soil is shown in Figure 4.7, for expansive soil samples. As shown in Figure 4.7 and Table 4.13, the plasticity index generally decreased with the increment of maize cob ash content. As it has been seen from the graph and Table, the addition of maize cob ash decreases the plasticity index of the expansive soil. The decrease is observed to be more with the increase in the quantity of maize cob ash contents up to 20%. The liquid limit and plastic index of weak expansive soil decrease significantly from a value of 96.67% to 56.37% and 59.8% to 27.4 respectively. Thus, from these test results, one can judge that the behavior of the soil sample was changed from high plasticity soil to low plasticity soil when treated with (5%-20%) maize cob ash content.

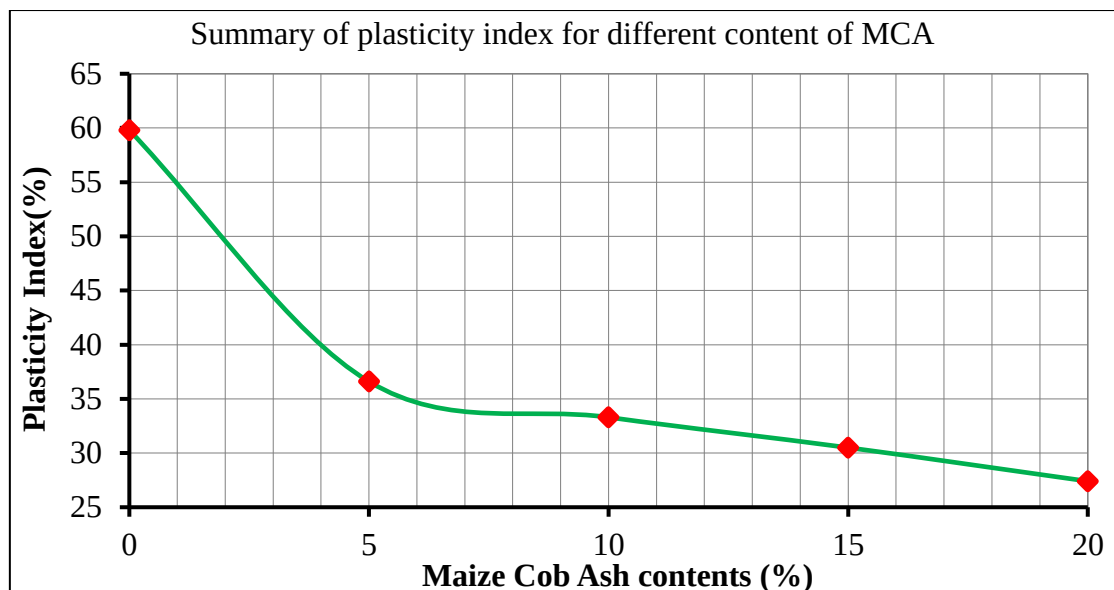


Figure 4. 7: Variation of plasticity index with the addition of different MCA contents

Table 4. 13: Summary of liquid limit and plasticity index for the maize cob ash –soils mixes.

Mix No.	Mix Type	LL (%)	PL (%)	PI (%)	%Decrease PI value
1	100%ES	96.67	36.9	59.8	---
2	95%ES+5%MCA	77.46	41.1	36.6	38.8
3	90%+10%MCA	72.1	38.8	33.3	44.3
4	85%ES+15%MCA	60	29.5	30.5	49
5	80%ES+20%MCA	56.37	28.93	27.4	54.2

In general, the plasticity of the soil is decreased with the addition of maize cob ash contents. These effects are due to the partial replacement of plastic soil particles with maize cob ash which is non -plastic material and due to the reduced surface activities, and flocculation and agglomeration of clay particles caused by cation exchange. In addition, the formation of adsorbed layer and coating of soil by maize cob ash particles (no sufficient time for pozzolanic reaction) is also another cause. Details of the Atterberg limit test results for a different combination of maize cob ash with the natural soil are also shown in appendix C.

4.6.2 Effect of maize cob ash on Swelling characteristics

A. Free Swell

The effect of waste maize cob ash on the free swell of the soil under investigation is shown in Figure 4.8, for the content of maize cob ash increases from 5% to 20%.

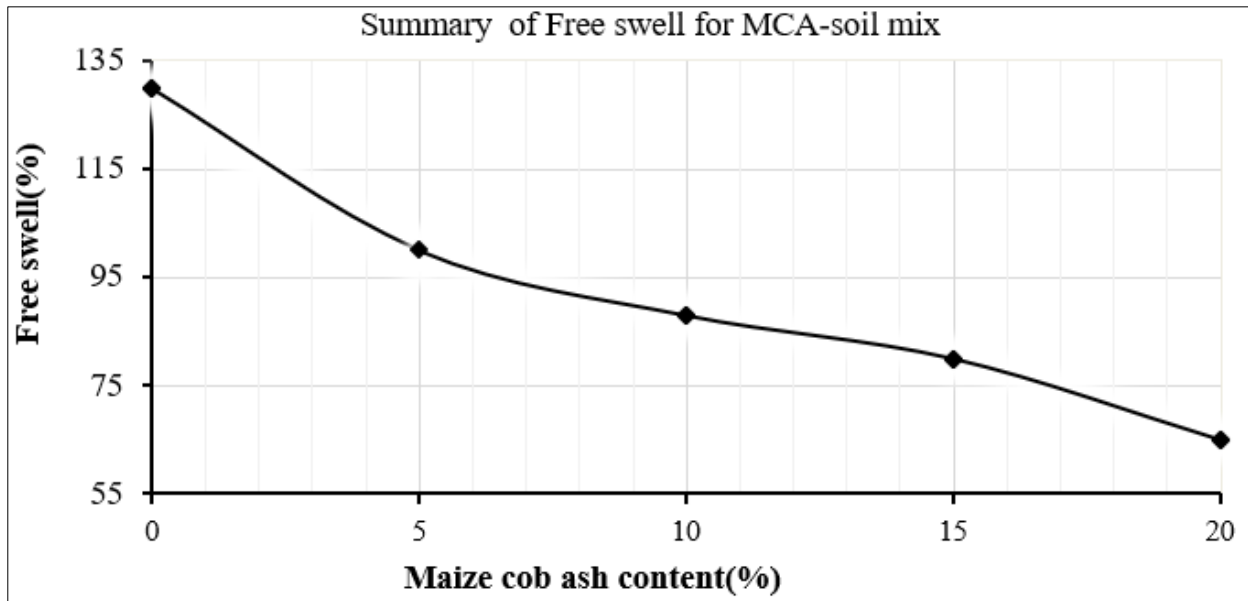


Figure 4. 8: Changes in the free swell with varying percentages of maize cob ash

The reduction in free swell is directly proportional to the quantity of maize cob ash added. The highest reduction in free swell is attained when the expansive soil is treated with 20% of maize cob ash, which is a 65% reduction compared to untreated soil, this shows that the reduction in free swell is directly proportional to the quantity of maize cob ash used.

B. Free swell index

The effects of maize cob ash on the free swell index of the expansive soil are shown in figure 4.9. The free swell index value decreased from 109% to 61.76% with an increased amount of maize cob ash percentages from 0% to 20%.

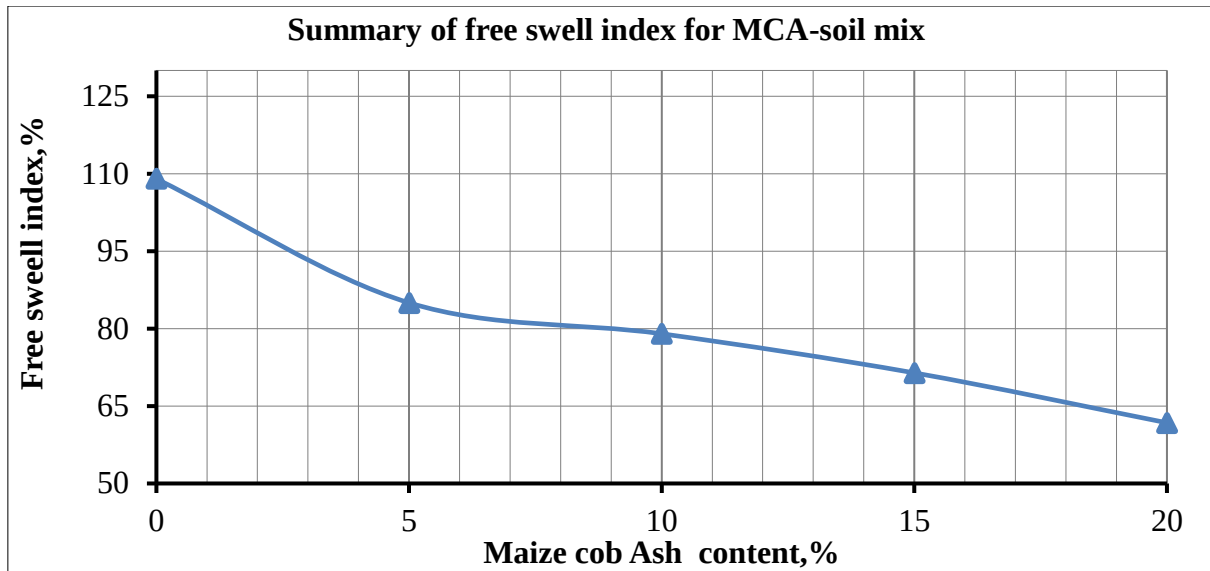


Figure 4. 9: Changes in the free swell index with varying percentages of maize cob ash

From the analysis of test results, the free swell index of expansive soil is 109%. According to swell classification based on the free swell index in chapter two of table 2.2, the soil has a high degree of expansion. As the percentage of maize cob ash increases, the free swell index value decreased up to 61.76% at 20% and makes the soil to be moderate swelling.

B. Free swell ratio tests

As it is shown in Figure 4.10, when maize cob ash is added to the soil sample, the free swell ratio decreases. The free swell ratio decreases from 2.1 to 1.62 as maize cob ash content increased from 0% to 20%. From the analysis of test results the free swell ratio of expansive swell classification based on the free swell ratio in chapter two of table 2.3, the soil is considered as highly swelling soil. According to this classification, the free swell ratio value decreased up to 1.62 at 20% maize cob ash and makes the soil medium/moderate swelling.

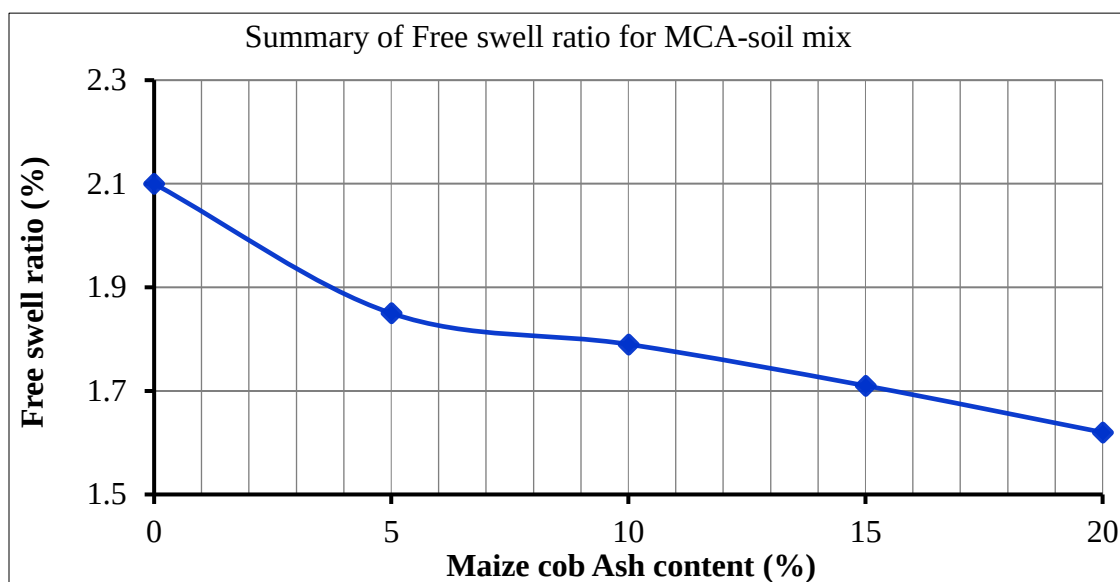


Figure 4. 10: Changes in the free swell ratio with varying percentages of maize cob ash

From the above swelling characteristics (Free swell, Free swell index, and Free swell ratio) the decrease in swelling, as maize cob ash content increased is mainly due to the following reasons.

- ✓ Maize cob ash decreases the swell potential of expansive soils by replacing some of the volumes that are previously held by expansive clay minerals and by cementing the soil particles together, thus preventing the expansion of the soil particles.
- ✓ Generally, as observed above, the soil samples had significantly changed from high swell property to low swell property, this might be due to chemical reaction and cation exchange in the maize cob ash -soil mix.

4.6.3 Effect of maize cob ash on Compaction characteristics of soil

Moisture Density Relationship

The soil was mixed with maize cob ash by the proportion of 5%, 10%, 15%, and 20% of MCA to get moisture content versus dry density graph, Hence, optimum moisture content (OMC) and maximum dry density (MDD) were calculated as the Table in appendix part and the corresponding results were plotted on the graph and given in the table below.

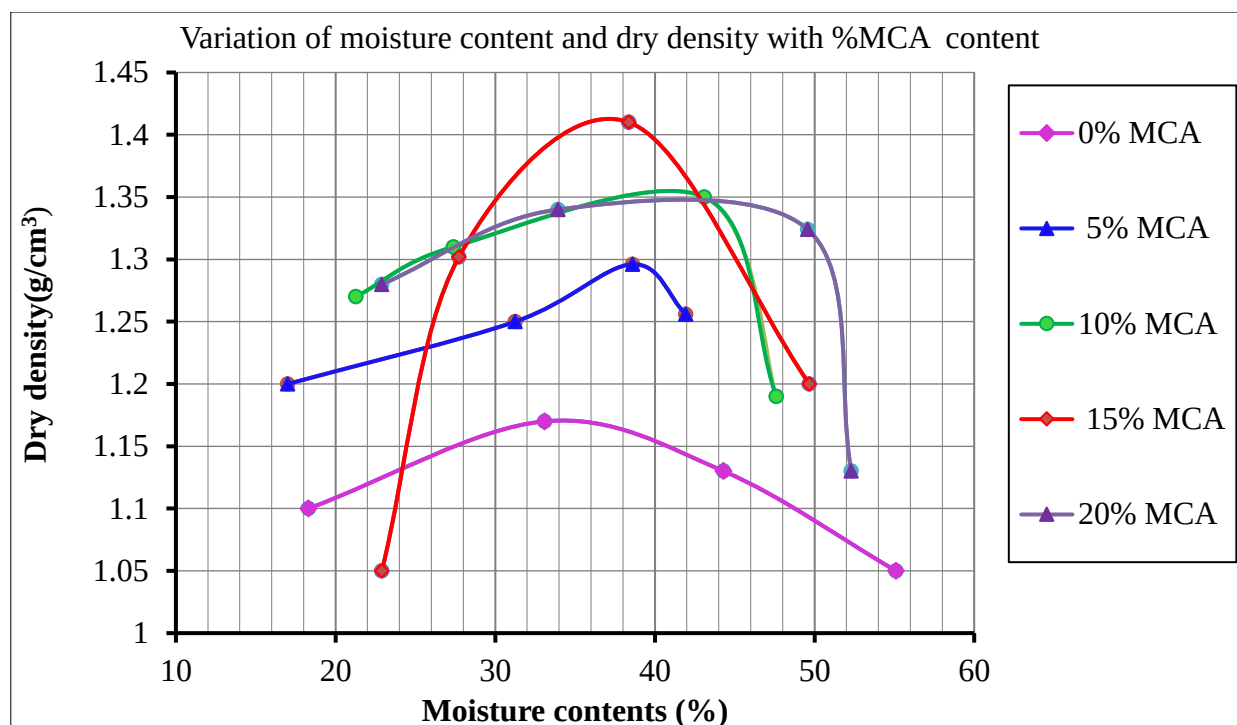


Figure 4. 11:Moisture content versus dry density graph with the variation of MCA Content

Table 4. 14:Summary on the effect of MCA on MDD and OMC of expansive soils

Mix No.	Mix Type	Maximum dry density (g/cm ³)	Optimum moisture content (%)
1	100%ES	1.17	33.1
2	95%ES+5%MCA	1.296	38.6
3	90%ES+10%MCA	1.35	43.1
4	85%ES+15%MCA	1.41	38.4
5	80%ES+20%MCA	1.32	49.7

The effect of maize cob ash on the maximum dry density and optimum moisture content of the expansive soil is shown in graph 4.11 and table 4.14. The result shows that the MDD and OMC for expansive soil were 1.17 g/cm³ and 33.1%, respectively. The maximum dry density increases from 1.17g/cm³ to 1.41 g/m³ for 15%MCA and optimum moisture content increased from 33.1% to 49.7%.MDD of soil gradually increase up to 15% maize cob ash content and was reduced when the amount of maize cob ash was increased further.

- MDD of the soil sample increases gradually up to 15% of maize cob ash then decreases, it may due to the following assumptions.
- ✓ Voids in soil are filled by fine particles of maize cob ash
- ✓ The disintegration of agglomerated particles due to excessive cations which result from the repulsive force
- ✓ It may also be due to the absorption of excess water by maize cob ash, the relatively lower specific gravity of maize cob ash, and the change in particle size compared to clay soil.

4.6.4 Effect of maize cob ash on UCS characteristics of the soil

The unconfined compressive strength (UCS) test is an unconsolidated and undrained load test conducted for determining the unconfined compressive strength of cohesive soil samples where the lateral confining pressure is equal to zero during the test. UCS is the compressive stress at which an unconfined cylindrical soil sample fails in a load test and is taken as the maximum load attained per unit area during loading. For the UCS test specimen's height-to-diameter ratio is between 2 and 2.5, the undrained shear strength was calculated to be half of the compressive stress at failure. The tests were carried out on both un-stabilized and maize cob ash stabilized expansive soil which is mixed at different ratios by the increment of 5% MCA up to 20% MCA content.

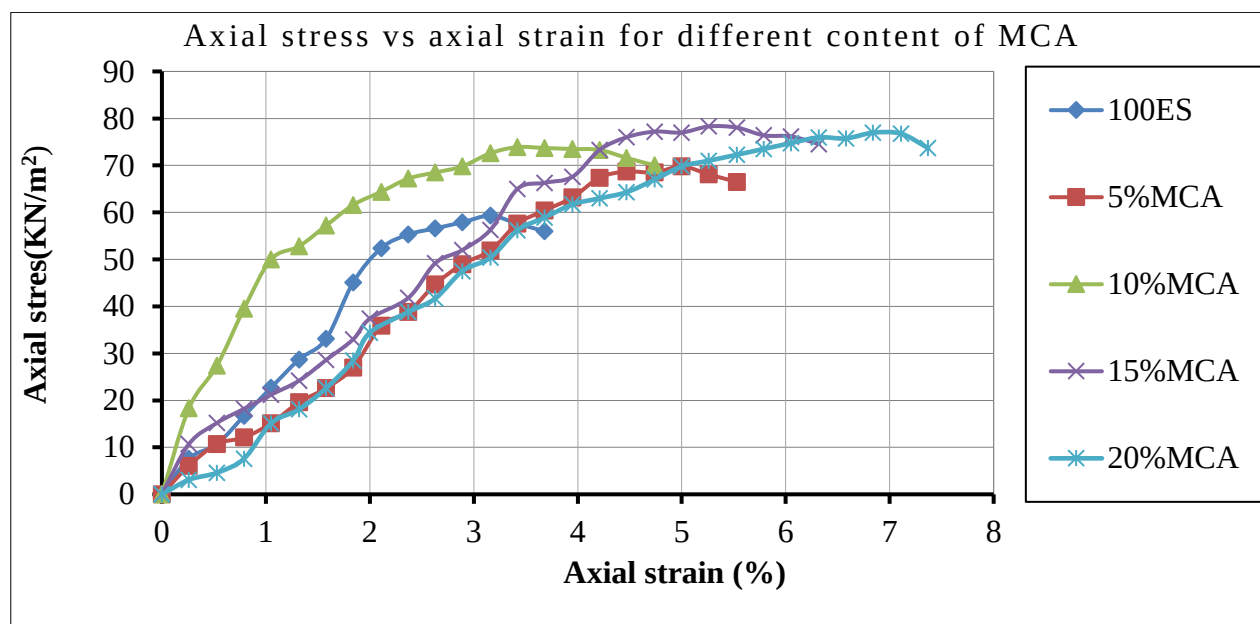


Figure 4. 12: Variation of Axial stress and Axial Strain with different content of MCA

The unconfined compressive strength test result is shown in Figures 4.12. This graph describes the stress-strain behavior of natural soil and maize cob ash-treated soil. Initially, the stress is gradually increasing with the increases of axial strain. After attaining the maximum axial stress, it decreases with the increase of axial strain for all the combinations of maize cob ash content and soil mixes. Furthermore, all the specimen shows shear failure which results in decreasing of unconfined compressive strength.

Table 4. 15:Summary of the effect of MCA on UCS of expansive soils.

Mix No.	Mix type	UCS value (Kpa)
1	100%ES	59.3
2	95%ES+5%MCA	69.8
3	90%+10%MCA	73.9
4	85%ES+15%MCA	78.3
5	80%ES+20%MCA	76.97

From the test result of UCS, as shown in Table 4.15 above, the unconfined compressive strength of expansive Soil generally increases with the increase of MCA content up to 15%, After 15% of MCA stabilized soil cause reduction in UCS. Thus, the UCS value of expansive soil increase from 59.3 Kpa to 78.3Kpa for the combination of soil with 15%MCA, at which the maximum value of strength was attained.

Unconfined compressive strength (UCS) is the most common and adaptable test method of deciding the strength of the stabilized soil. It is the main test recommended for the determination of the required amount of additive to be used in the stabilization of weak soil. The optimum mix was having the highest UCS values for the given proportion and which was found to be 85% Soil and 15% MCA of mixes.

- ❖ UCS of the soil increases gradually up to 15% of maize cob ash added and then decreases due to the following reasons.
 - Since MCA is less dense than soil, as voids are filled by more and more MCA particles the bondage in between soil particle becomes decreases

- Prevention of swelling with the formation of relatively strong bonds between soil and maize cob ash which may impart the strength between the soil and stabilizer.
- During agglomeration the radius of maize cob ash particles that surrounds the soil particle increases resulting in loss of bond strength that is why it reduces after 15% of maize cob ash.

Generally, beyond 15% of MCA, the decrease in UCS value of the expansive soil is since the expansive clay soil decrease in its tendency to bind/cohesion as the percent of maize cob ash material increases cohesion less property increases between the two.

4.6.5 Effect of maize cob ash on CBR and CBR swell of the soils

The effect of maize cob ash on the CBR of expansive soil is shown in figure 4.13. The result of tests indicates that the CBR Values of the soil sample was also increased gradually, up to 15% maize cob ash content, further addition of maize cob ash amount causes the slight decrease of CBR values.

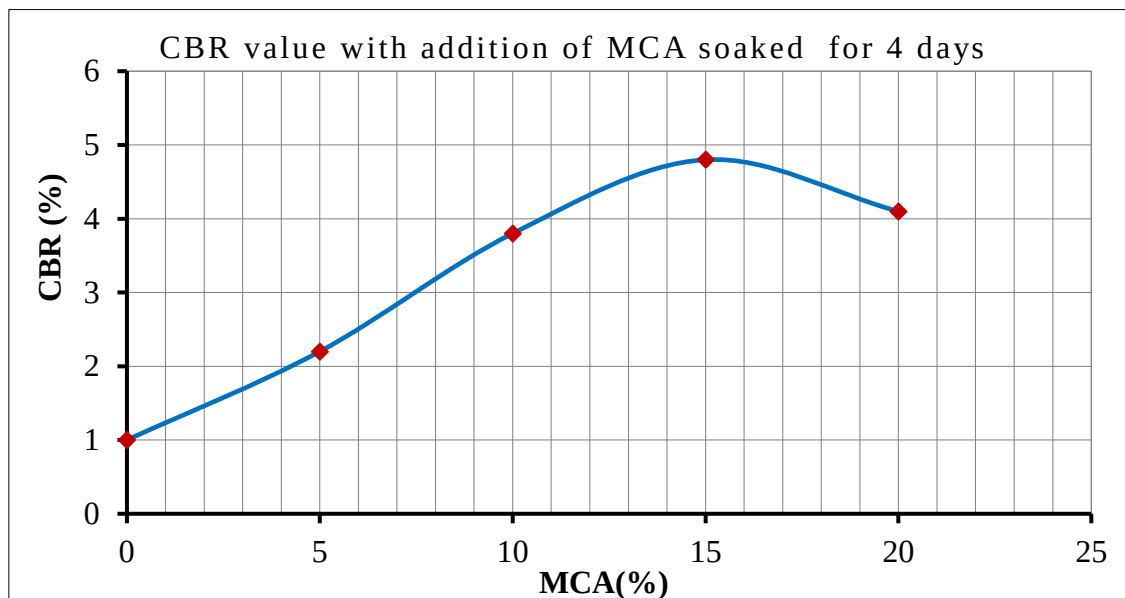


Figure 4. 13: Variation of CBR of the soil with different content of MCA

The decrease in CBR value after 15% MCA content was similar to that of the UCS process. It is due to the excess amount of maize cob ash quantity which was not mobilized in the reaction as the presence of naturally occurring CaOH in the soil is small.

The excess maize cob ash occupies space within the specimen and reduces the clay and silt content in the soil, thus reduces the cohesion in the soil- maize cob ash mixture.

- The CBR swell is also decreased from 14.35 to 5.2 as the maize cob ash percentage increases from 5% to 20%. This reduction in CBR-swell of expansive soil is due to cation exchange between the soil and the stabilizer and flocculation and agglomeration of the soil particles. This is also due to the replacement of some space by maize cob ash that is previously held by expansive clay minerals.

4.7 The laboratory test result of Expansive soil stabilized with waste ceramic powder

The effect of waste ceramic powder on the properties of the expansive soil was also evaluated by conducting, Atterberg limits, free swelling tests, standard compaction tests, unconfined compressive strength test, and CBR test.

4.7.1 Effect of waste ceramic powder on the Atterberg limits of treated soil

The effect of waste ceramic powder on the plasticity index of the soil is shown in Figure 4.14 and Table 4.16. As shown in the figure and table, the addition of waste ceramic powder causes a decrease in the liquid limit and plastic limit of expansive soil, which in turn causes a reduction in the plastic index.

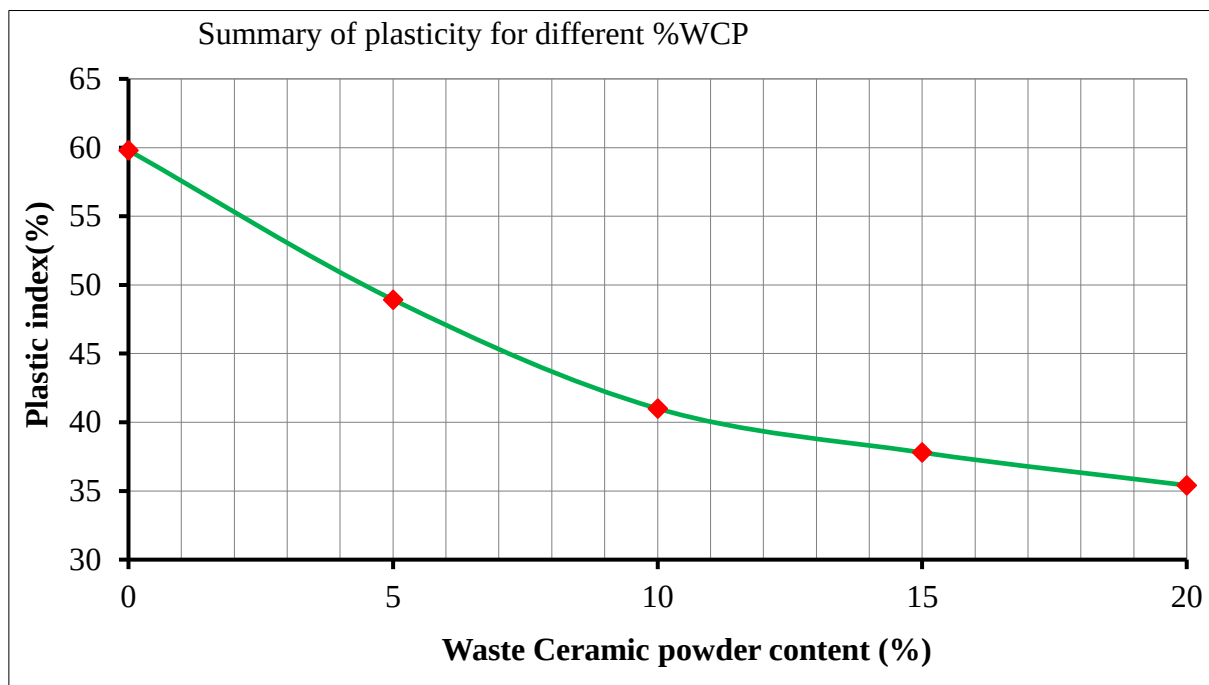


Figure 4. 14: Variation of plasticity index with different contents of WCP.

Table 4. 16:Summary of liquid limit and plasticity index with WCP contents

Mix No.	Mix Type	LL (%)	PL (%)	PI (%)	%Decrease PI value
1	100%ES	96.67	36.9	59.8	-
2	95%ES+5%WCP	81.7	32.77	48.93	18.2
3	90%+10%WCP	79.9	38.9	41	31.43
4	85%ES+15%WCP	73.9	36.1	37.8	36.8
5	80%ES+20%WCP	71.13	35.7	35.41	40.8

The decrease in plasticity index is observed to be more and more with the increase in the quantity of waste ceramic powder up to 20%. In general, this is clearly shown by the fact that the plasticity index of treated soil decreased with increasing additive quantity. The effect of WCP on the plasticity of the soil is similar to that MCA addition, which causes a reduction in the plasticity /consistency of the soil.

Mean that, the effects are due to the replacement of highly plastic soil particles with waste ceramic powder which is non-plastic material, and flocculation and agglomeration of clay particles caused by cation exchange may the reason.

4.7.2 Effect of waste ceramic powder on Swelling characteristics

A. Free swell

The effect of waste ceramic powder on the free swell of the soil under investigation is shown in Figure 4.15, the reduction in free swell is directly proportional to the quantity of waste ceramic powder. The highest reduction in free swell is attained when the expansive soil is treated with 20% of waste ceramic powder, which is a 60% reduction compared to untreated soil, the reduction in free swell is directly proportional to the quantity of waste ceramic powder. Generally, it shows a similar trend with maize cob ash-soil mix.

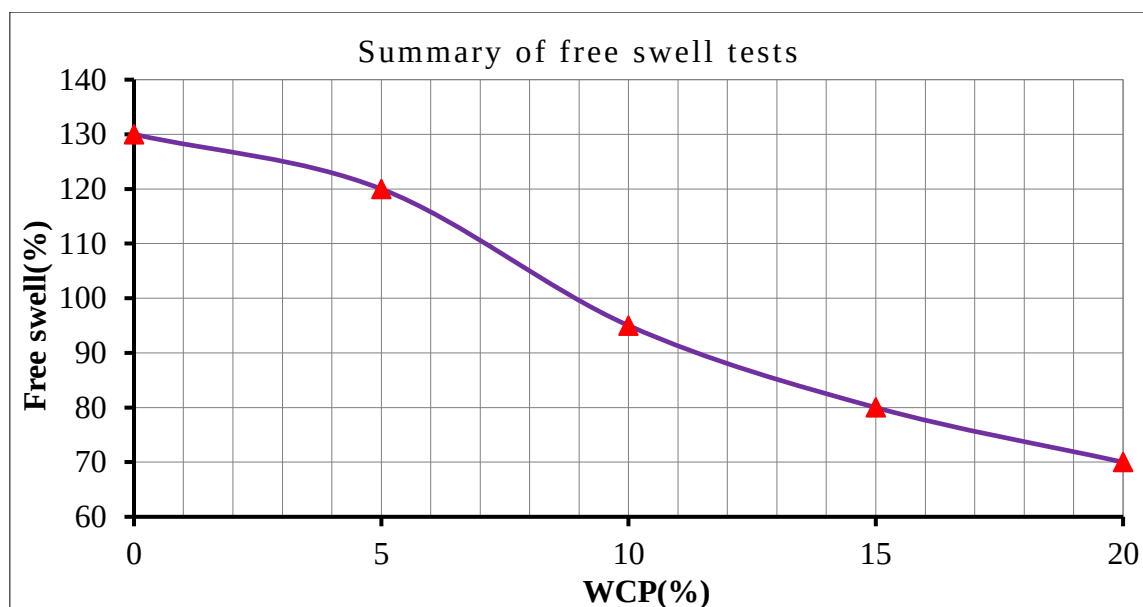


Figure 4. 15:Changes in the free swell with varying percentages of WCP

B. Free swell index

The effects of waste ceramic powder on the free swell index of the expansive soil are shown in figure 4.16 as shown below.

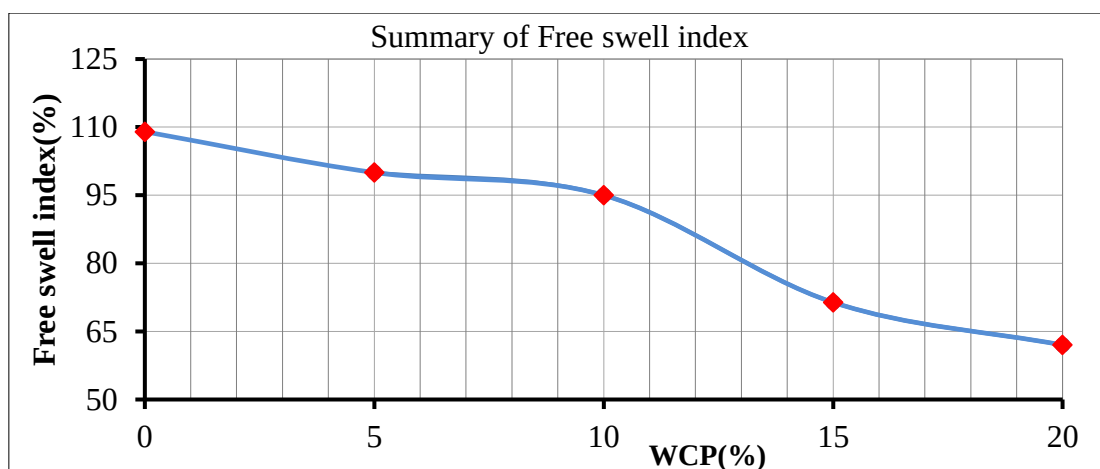


Figure 4. 16:Changes in the free swell index with varying percentages of WCP

The free swell index value of the soil decreased from 109% to 62 % with the increased amount of waste ceramic powder from 0% to 20%. From the analysis of test results, the free swell index of expansive soil is 109%. As the percentage of WCP increases, the free swell index value decreased up to 62 at 20% and makes the soil to be moderate swelling.

C. Free swell ratio tests

As it is shown in Figure 4.17 below when the waste ceramic powder is mixed with the soil the free swell ratio decreases.

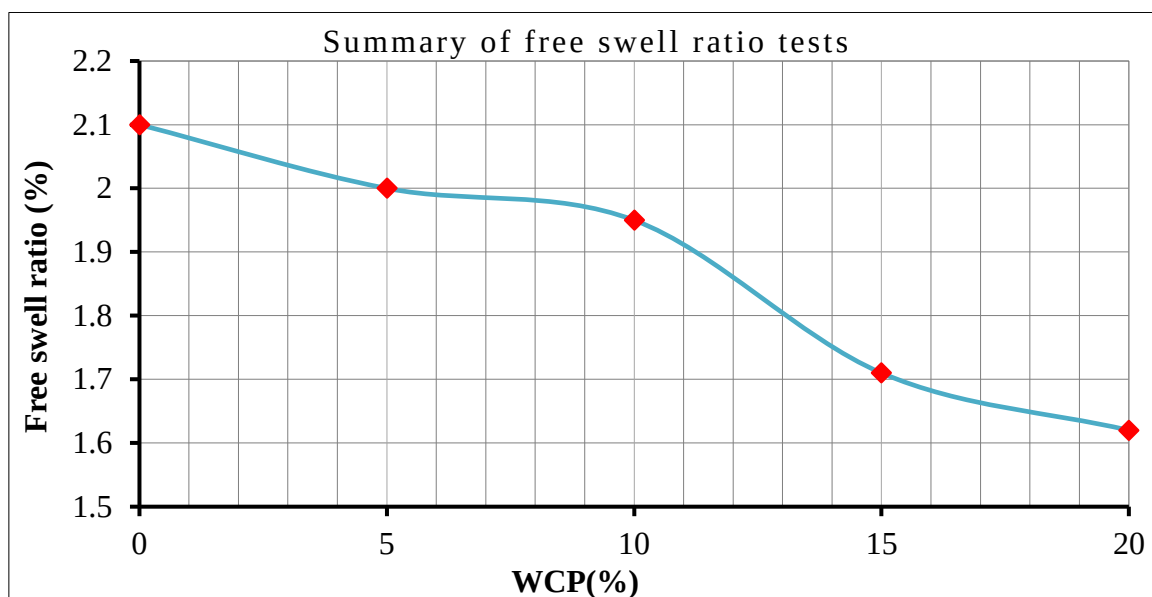


Figure 4. 17: Changes in the free swell ratio with varying percentages of WCP

The free swell ratio decreases from 2.1 to 1.62 as waste ceramic powder content increased from 0% to 20%. From the analysis of test results the free swell ratio of expansive soil swell classification based on the free swell ratio of chapter two in table 2.3, the soil is considered as high swelling soil. According to this classification, the free swell ratio value decreased up to 1.62 at 20% waste ceramic powder and makes the soil medium swelling.

According to the analysis of the above test result, the effect of waste ceramic powder on swelling characteristics of the soils, shows a similar trend as that of maize cob ash stabilized expansive soils.

4.7.3 Effect of waste ceramic powder on compaction characteristics

Maximum dry density and Optimum moisture content

The effect of waste ceramic powder on the dry density and moisture content of the expansive soil is shown in Figure 4.18. The soil was mixed with WCP by ratios of 5%, 10%, 15%, and 20% to get moisture content versus dry density graph, then optimum moisture content (OMC), and maximum dry density (MDD) were determined from the graph.

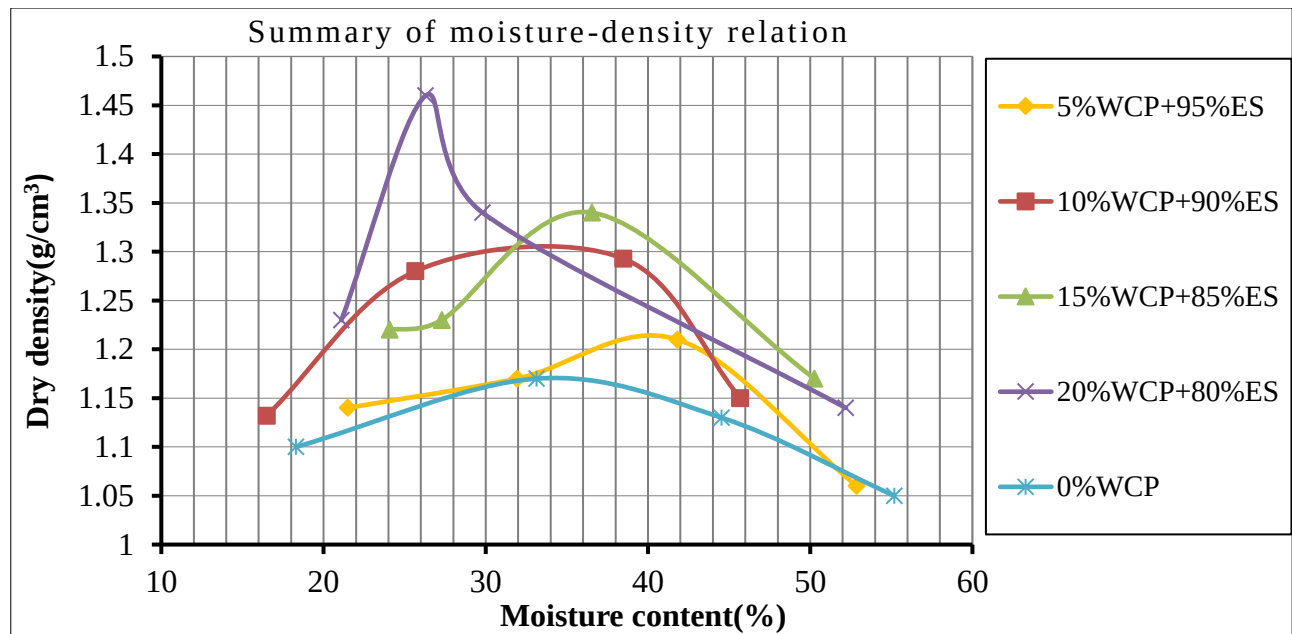


Figure 4. 18: Moisture content versus dry density graph with the variation of WCP

Table 4. 17: Summary on Effect of WCP on MDD and OMC of expansive soils

Mix No.	Mix Type	Maximum dry density(g/cm ³)	Optimum moisture content (%)
1	100%ES	1.17	33.1
2	95%ES+5%WCP	1.21	41.8
3	90%ES+10%WCP	1.29	38.48
4	85%ES+15%WCP	1.34	36.65
5	80%ES+20%WCP	1.46	26.3

MDD goes on increasing irrespective of the increasing percentage addition of waste ceramic powder. The maximum increment of MDD for waste ceramic powder-treated soil is 25% which was obtained for 20% of WCP stabilized clay soil. The result shows that the MDD increases from 1.17g/cm³ to 1.46g/cm³ from virgin/natural soil to 20% waste ceramic powder treated soil. Generally, the maximum dry density of soil increases gradually with an increase of WCP content for stabilized soil samples. OMC goes on decreasing irrespective of the increase in percentage addition of waste ceramic powder. The OMC decrease from 41.3% to 26.3% for 20% waste ceramic-treated soil. The effect of waste ceramic powder on MDD of the soil shows a similar trend like that of maize cob ash up to 15%, but later on, show a different trend /was increased in the case of WCP.

- ❖ The maximum dry density is increased mainly due to;
- ✓ The partial replacement of comparatively heavier waste ceramic powder with the lightweight soil;
- ✓ Comparatively low specific gravity value (2.71) of soil than that of replaced waste ceramic powder (2.93);
- ✓ It may also be attributed to the coating of the soil by the waste ceramic powder which results in large particles with small voids and hence high density;
- ❖ The optimum moisture content show reduction mainly due to:
- ✓ The optimum moisture content of soil decreases with an increase in waste ceramic powder, because the waste ceramic powder is denser than the soil thus, preventing the permeability of water.
- ✓ It is also caused by the decrease in water-absorbing properties of clay particles as the waste ceramic powder content increases in the soil -WCP mix.
- ✓ The decrease in OMC is due to the addition of waste ceramic powder caused by the high absorption of water by waste ceramic powder. This implies that less water is needed to compact the soil with waste ceramic powder mixture.

4.7.4 Effect of waste ceramic powder on UCS characteristics of the soil

Unconfined compressive strength test samples were prepared with optimum moisture content and maximum dry density that was obtained from standard proctor test for respective percentage of waste ceramic powder mixed with weak expansive soil. The results from UCS tests are used to estimate the short-term bearing capacity of fine-grained soils for subgrade and foundation. The variation of axial strain vs axial stress of the soil sample with waste ceramic powder is shown in figure 4.19, moreover, the effect of different content of WCP on the UCS of the soil is illustrated in table 4.18 and appendix D.

From the analysis of the test result, the unconfined compressive strength increased as the content of waste ceramic powder additive increased up to 20%. The result shows that the UCS value increases from 59.3Kpa to 80.4Kpa from virgin soil to 20%WCP-soil mix. Generally, UCS of soil increases gradually with an increase of WCP content for stabilized soil samples.

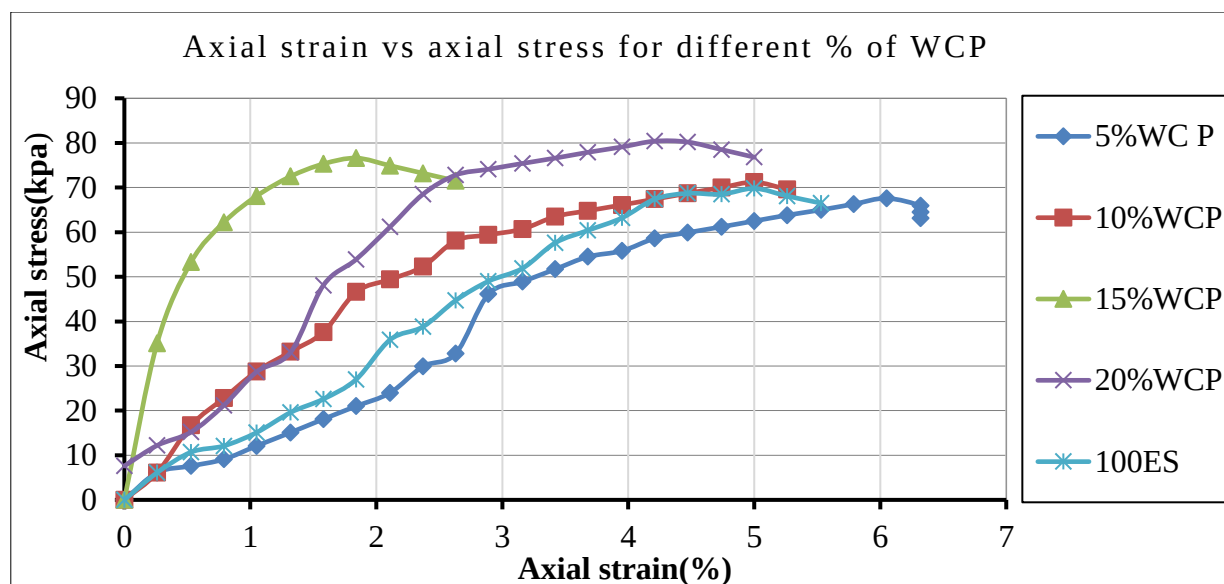


Figure 4. 19:Variation of Axial stress and Axial strain with different content of WCP

Table 4. 18: Summary of the effect of UCS test result for expansive soil mixed with WCP

Mix No	Mix type	UCS value (Kpa)
1	100%ES	59.3
2	95%ES+5%WCP	67.6
3	90%+10%WCP	71.2
4	85%ES+15%WCP	76.6
5	80%ES+20%WCP	80.4

4.7.5 Effect of waste ceramic powder on CBR and CBR swell of the soils

The effect of waste ceramic powder on the CBR values of the soil is shown in figure 4.20. When the expansive soil was treated with 5% to 20% of waste ceramic powder. The CBR value of natural soil was increased from 1% to 4.3%. CBR of the expansive soil increased as waste ceramic powder contents increases. This shows that the load-bearing capacity of the weak soil sample increased significantly with waste ceramic powder contents. This improvement is due to the pozzolanic reactions of waste ceramic powder with the soil. This leads to the agglomeration of clay particles to causes an increase in strength. The result implies that the material can be used as a subgrade per the ERA specification. Detailed test results were given in Appendix D.

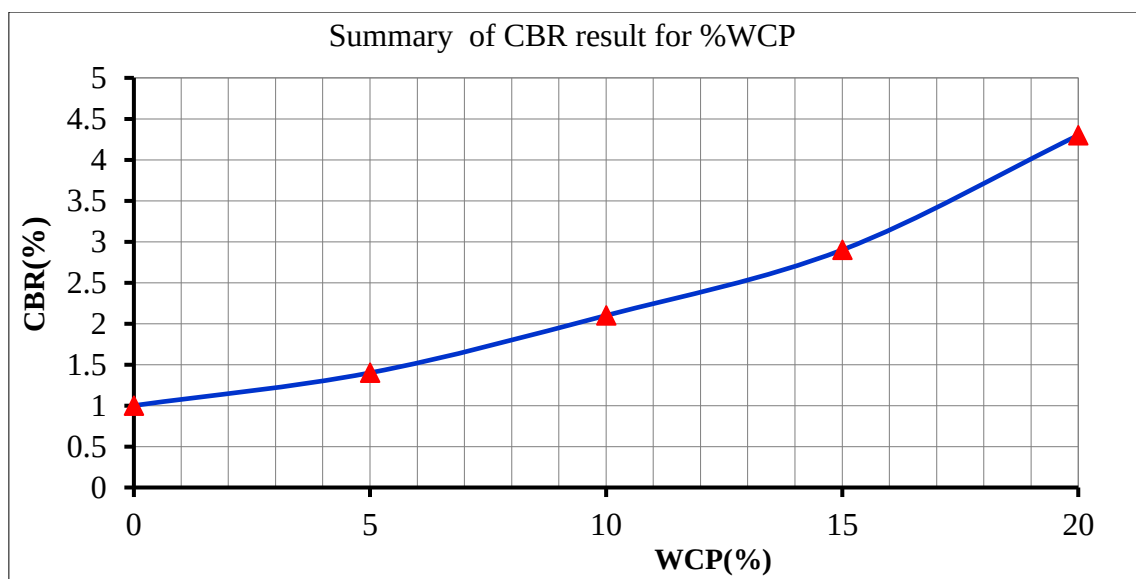


Figure 4. 20: Variation of CBR with different content of WCP

The effect of waste ceramic powder on the CBR swell values of the soil is also shown in figure 4.21. When treating the expansive soil at 20% waste ceramic powder content, it gives a CBR-swell of 4.29 % which was decreased by 10.1% from that of natural soil. Thus, the CBR-swell of the expansive soil declined with increasing waste ceramic powder contents. This is due to; the swelling potential of the specimen decrease with waste ceramic powder addition.

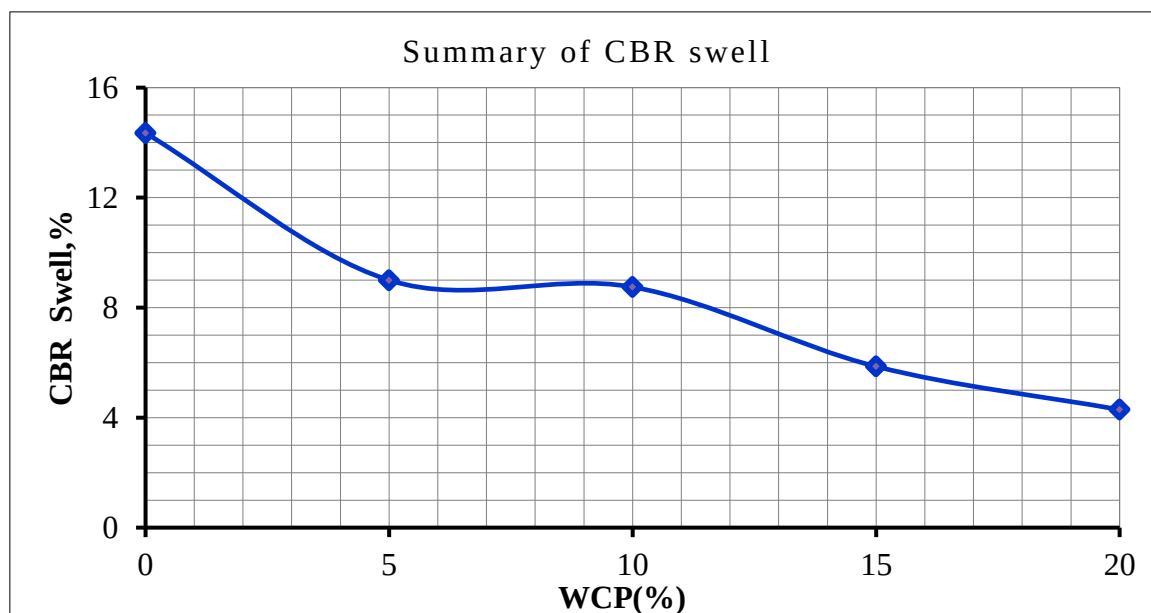


Figure 4. 21: Variation of CBR swell with different content of WCP

4.8 Combined effect of Maize cob ash and Waste Ceramic powder

The combined effect of maize cob ash and waste ceramic on the expansive soil was evaluated for moisture density relationship and unconfined compressive strength characteristic of the soil.

4.8.1 The combined effect of MCA and WCP on the Compaction characteristics

The combined effect of MCA and WCP are evaluated depending on the maximum unconfined compressive strength obtained for the MCA-soil mix. That means the optimum /maximum percentage of maize cob ash was found to be 15% MCA based on the strength (UCS) test result. Hence, the test result of the combined effect of both additives was analyzed by adding a constant 15%MCA on the different content of the waste ceramic powder.

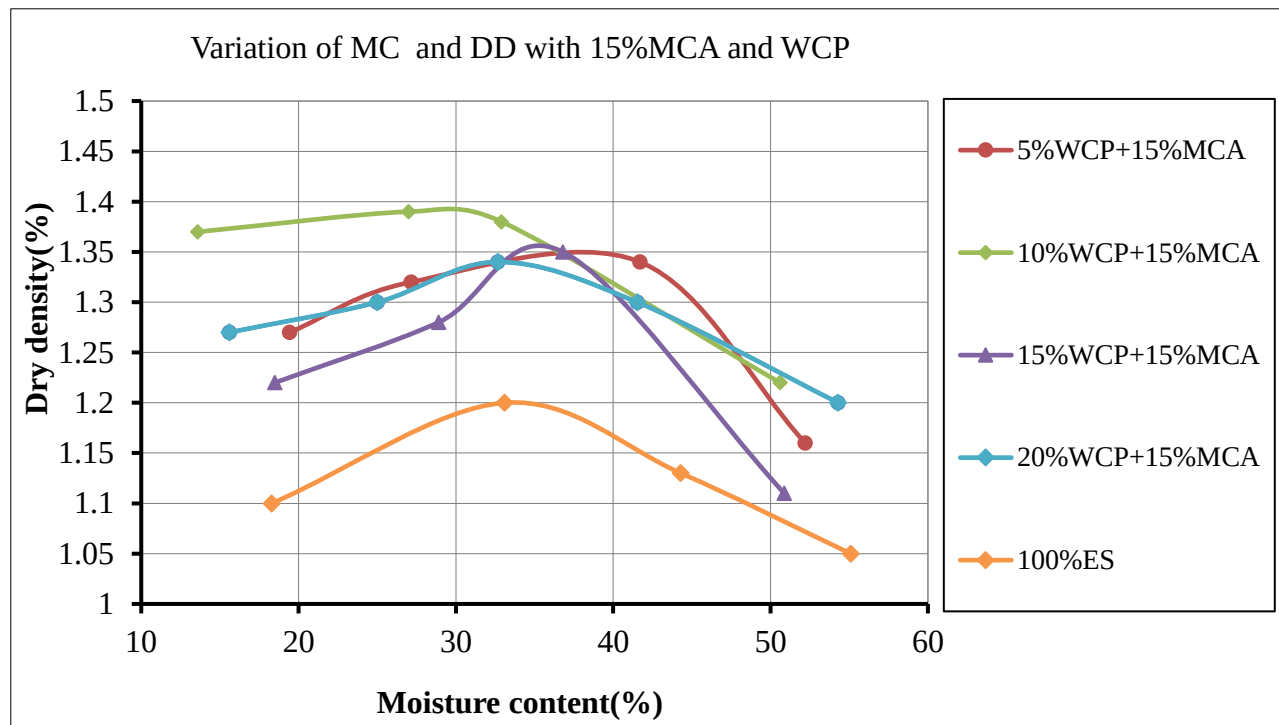


Figure 4. 22:Moisture content versus dry density of combined MCA and WCP with ES

Table 4. 19:Summary of MDD and OMC of the soils when mixed with 15% MCA and WCP.

Mix No.	Mix Type	Maximum dry density (g/cm^3)	Optimum moisture content (%)
1	100%ES	1.17	33.1
2	15%MCA+5%WCP+80%ES	1.34	41.7
3	15%MCA+10%WCP+75%ES	1.39	27
4	15MCA%+15%WCP+70%ES	1.35	36.8
5	15%MCA+20%WCP+65%ES	1.34	32.67

As it can be seen that from the above figure and table, the MDD and OMC show a slight increase in the treatment of weak soil with MCA-WCP additive agents. Hence, the MDD of natural soil increases from 1.170 g/cm^3 to 1.390 g/cm^3 and OMC increases from 33.1% to 41.7% when the two materials were used in combination. Generally, the maximum dry density is increased with the percentage of combined additives giving the largest MDD of 1.39 g/cm^3 for 15%MCA+10%WCP+75%ES. this shows that when both additives are mixed with the soil it gives less MDD than that of treating soil separately. This may due to, the materials are not holding each other properly creating a large void space with the soil particle.

4.8.2 The combined effect of MCA and WCP on the UCS

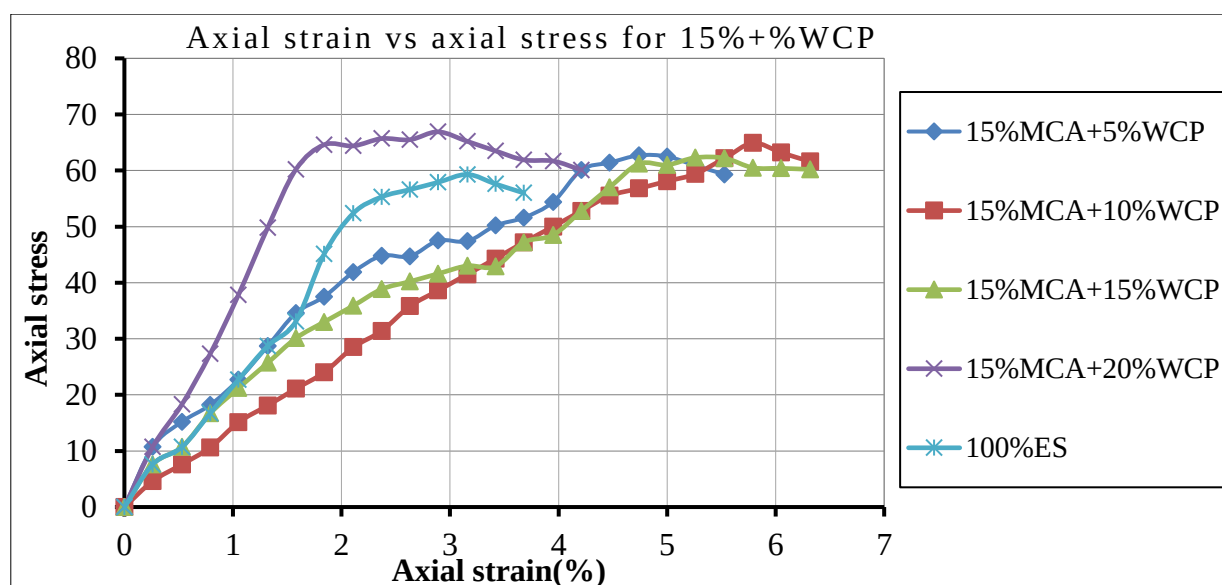


Figure 4. 23:Variation of Axial strain and Axial stress with 15%MCA and WCP Content

Table 4. 20:Summary of average UCS test result for ES mixed with 15%MCA and WCP

Mix No.	Mix type	UCS value (Kpa)
1	100%ES	59.3
2	5%WCP+15%MCA+20%ES	62.7
3	10%WCP+15%MCA+75%ES	64.9
4	15%WCP+15%MCA+70%ES	62.3
5	20%WCP+15%MCA+65%ES	66.9

The unconfined compressive strength of combined material stabilizer with soil has shown on table and figure, From the analysis of the test result, the unconfined compressive strength value was increased as the combined percentage of both materials increased, giving the maximum value of 66.9Kpa for combination 20%WCP+15%MCA with the soil, however, this value is less than when each material mixed separately with the soil. Moreover, this value may have been used as optimum for mixing both materials with the soil. From this, one can understand the greatest strength obtained for the soil when it was treated separately with maize cob ash and waste ceramic powder waster rather than in combination.

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The study has been conducted on the effect of maize cob ash and waste ceramic powder on some geotechnical properties of expansive soil specifically are found around Dukem town. The following conclusions can be drawn from the results of the laboratory tests.

Conclusions on maize cob ash soil stabilization

- The expansive soil used in the investigation was categorized as an A-7-5 soil class based on the AASHTO and CH as the USCS soil classification system.
- The Plasticity index and swelling characteristics of the natural soil samples were reduced as the content of MCA was used from 5% to 20%.
- The MDD, UCS, and CBR of the expansive soil was increased with the addition of maize cob ash content up to 15%MCA and was decreased thereafter.
- Generally, the UCS value of natural soil increases from 59 Kpa to 78.3Kpa giving the maximum values for 15%MCA, and CBR was increased from 1% -4.1% as the maize cob ash percentage increases up to 15%MCA. Thus, 15%MCA+85%WCP is the optimum mixes.

Conclusions on waste ceramic powder soil stabilization

- The plasticity and swelling characteristics of expansive soil were decreased irrespective of the addition of waste ceramic powder content from 5% to 20%.
- The MDD, UCS, and CBR value of the expansive soil increases gradually as WCP content increases from 0%-20%WCP. Thus, MDD increases from 1.17g/cm³ to 1.46g/cm³, and OMC was decreased as waste ceramic powder content increased. The UCS increases from 59.3Kpa to 80.4Kpa, CBR was increased from 1% to 4.3%, as waste ceramic powder increases from 0% to 20%. Hence the highest value of the strength was obtained for 20%WCP+80%ES.

Conclusion on combined and comparison effects of MCA and WCP

- The combined effect of MCA and WCP were evaluated based on the maximum UCS obtained from the MCA-soil mix, which was at 15%MCA, then adding it on the different content of WCP.
- The analysis of the combined effect shows that the MDD was increased with the increasing percentage of both additives giving the largest MDD value of 1.39 g/cm³ for the mixture of 15%MCA+10%WCP+75%ES.
- Unconfined compressive strength of stabilized soil was increased and giving a peak value of 66.9Kpa for the combination of 20%WCP+15%MCA, which have taken as optimum when both materials were used. Thus, the materials yield more strength when was blended with the soil separately. So that, it is effective to use both materials separately rather than using their combination with the soil.

From the comparative analysis of the test results, both materials are effective in improving the geotechnical properties of expansive soil. however, there were slight differences in their effect. Generally, MCA is slightly more effective in reducing plasticity and swelling properties of soils than WCP and WCP is slightly more effective to enhance the strength properties of the expansive soil rather than MCA addition.

5.2 Recommendations

The following recommendation is forwarded based on the finding of this research work.

1. It is obvious that both MCA and WCP are disposed at a different place to the environment. So, it is recommended to make intensive researches to use other waste materials for soil improvement as it is cost-effective and environmentally friendly.
2. The present study is conducted by using each stabilizer separately, thus it will be more effective to use both materials for soil stabilization by mixing with either cement or lime.
3. All laboratory tests were conducted for a soil-stabilizer mix without curing, so it is recommended to evaluate the effect of curing on the properties of the soils for further study.
4. The effect of both materials on natural soil was investigated by performing several laboratory tests. furthermore, to know settlement behavior, the effect of these materials on the consolidation properties should also be conducted for future work.

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APPENDICES

Appendix A: Laboratory test result of the soil sample from test pit one

Table A 1:Natural moisture content for test pit-1

Sample location	Soil of Test pit-1			
Can number	A	B	C	D
Mass the can(g)	19	19	20	19
Mass of can+Mass of moist soil (g)	60	55	56	60
Mass of can +mass of oven dry soil(g)	50	46	46	49
Mass of moisture (Mw)	10	9	10	11
Mass of dry soil (Ms)	31	27	26	30
Water content(w)%	32.3	33.33	38.46	36.6
Average water content(w)%	35.2			

Table A 2:Wet sieve Analysis test results for TP -1

Sieve No.	Sieve opening (mm)	Mass of sieve(g)	Mass of sieve + Re. soils(g)	Mass of Retained soils(g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
No 4	4.75	620	620	0	0	0	100
No 10	2	564	565	1	0.01	0.35	99.65
No 16	1.18	540	541	1	0.02	0.704	99.3
No.30	0.6	509	511	2	0.04	1.41	98.59
No 40	0.425	548	548	0	0.04	1.41	98.59
No 50	0.30	476	478	2	0.06	2.1	97.9
No 100	0.15	458	460	2	0.08	2.82	97.2
No 200	0.075	455	456	1	0.09	3.2	96.8
Pan		252	253	1	2.84	100	----

Table A 3:Hydrometer Analysis test for TP-1

Elapsed Time, min	Actual Hydrometer Rdg (R_h)	Temperature. (C^0)	Effective depth(mm)	Coefficient (K)	Rdg in Dispersant (R_o)	Modified Hydrometer rdg $R_d = R_h - R_o$	Grain size (mm)	Finer (%)	Finer combination
1	51.0	18	7.90	0.013	5.0	46.000	0.039	90.78	87.8
2	48.0	18	8.40	0.013	5.0	43.000	0.028	84.86	82.1
5	44.0	18	9.10	0.013	5.0	39.000	0.019	76.97	74.5
15	42.0	18	9.40	0.013	5.0	37.000	0.011	73.02	70.68
30	38.0	18	10.10	0.013	5.0	33.000	0.008	65.13	63
60	36.0	18	10.40	0.013	5.0	31.000	0.006	61.18	59.2
120	35.0	21	10.60	0.013	5.0	30.000	0.004	59.21	57.3
240	30.0	18	11.40	0.013	5.0	25.000	0.003	49.34	47.76
480	27.0	18	11.90	0.013	5.0	22.000	0.002	43.42	42
1440	25	18	12.2	0.013	5.0	20.000	0.001	39.47	38.2

Table A 4: Specific gravity test for TP-1

Specimen number	1	2
Pycnometer bottle number	T15	T22
WP = Mass of empty, clean pycnometer (grams)	35.9	39.5
WPS = Mass of empty pycnometer + dry soil (grams)	45.9	49.5
WB = Mass of pycnometer + dry soil + water (grams)	137.6	138.22
WA = Mass of pycnometer + water (grams)	143.9	144.56
Specific Gravity (GS)	2.7	2.73
Average (GS)	2.71	

Table A 5: Free Swelling Test Result for TP-1

Swelling characteristic for Natural soil from TP-1		
Description	Water	Kerosene
Initial reading(A)	10	10
Final reading(B)	23	11
Free swell	FS=130	
Free swell index	FSI=109	
Free swell ratio	FSR=2.1	

Table A 6:Atterberg limit test result for TP-1

Determination	Natural soil (TP1)					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	17	28	34			
Wt. of container	20	19	19	18	18	20
wt. of container + wet soil, (g)	40	37	38	30	29	27
Wt. of container+dry soil, (g)	30	28	29	27	26	25
Wt. of water, (g)	10	9	9	3	3	2
Wt. of dry soil, (g)	10	9	10	9	8	5
Moisture content (%)	100	100	90	33.3	37.5	40
Average	96.67			36.9		
Plastic index	59.7					

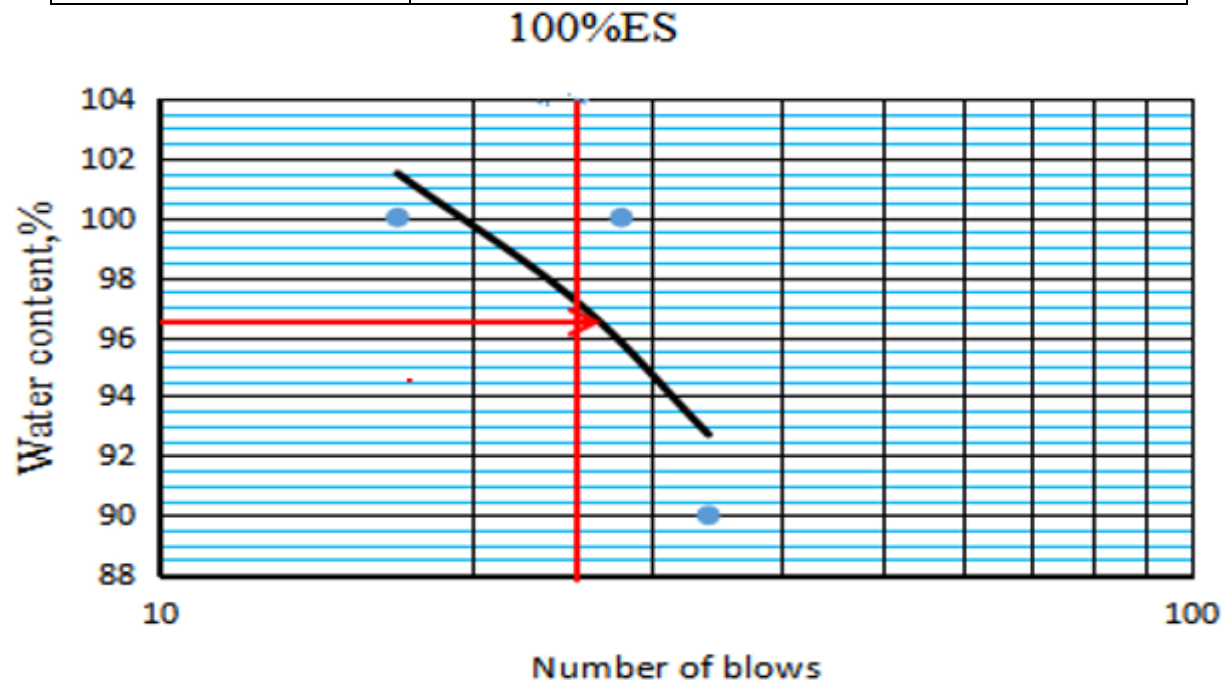


Figure A 1:Liquid limit Curve for test TP-1 soil

Table A 7: Standard Compaction laboratory test result for TP-1

Compaction for Natural soil (100%ES)												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5645			5949			6024			6015		
Wt. of mold(g)	4393			4393			4393			4393		
Wet soil (g)	1252			1556			1631			1622		
Wet density, (g/cm³)	1.252			1.556			1.631			1.622		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt. (g)	20	19	18	18	18	19	19	21	18	19	18	18
Wet soil+can (g)	55	58	60	68	64	68	62	59	67	65	74	84
Dry soil+can(g)	50	52	53	55	52	57	49	47	52	50	53	60
Mass of moisture(g),	5	6	7	13	12	11	13	12	15	15	21	24
Dry soil(g)	30	33	35	37	34	38	30	26	34	31	35	42
Moisture content (%)	16.7	18.2	20	35.1	35.3	28.9	43	46	44	48.4	60	57
Average moisture content (%)	18.3			33.1			44.3			55.1		
Dry density/cm³	1.1			1.17			1.13			1.05		

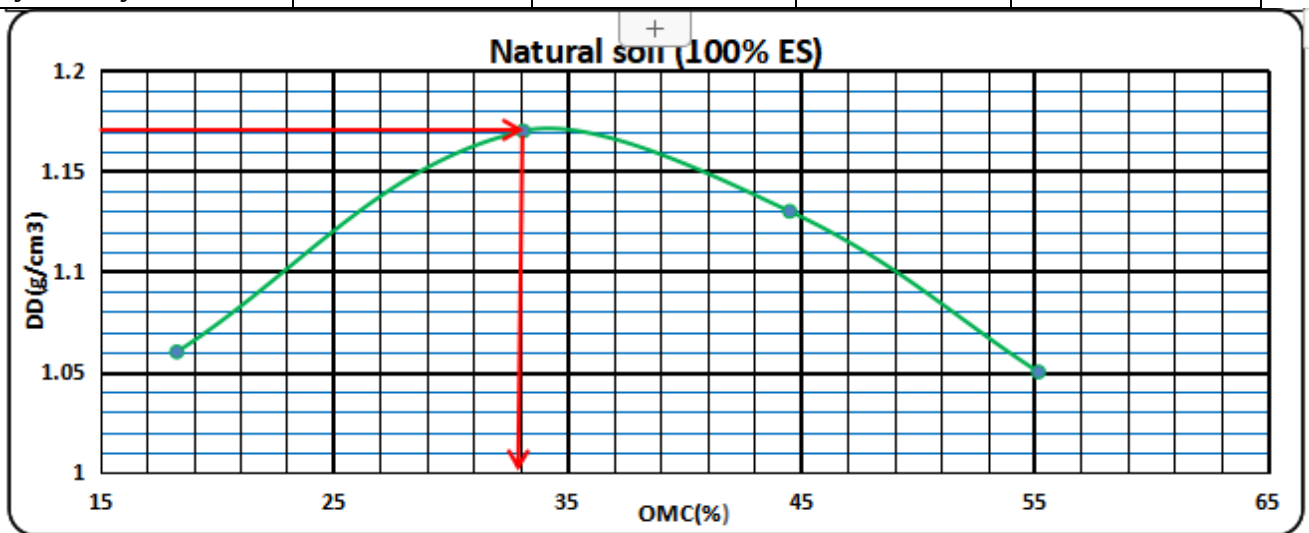


Figure A 2: Compaction curve for TP-1

Table A 8:Unconfined compression strength test for TP-1

Diameter of sample, mm = 38		Cross- Sectional Area, m ² =0.0010			
Length of sample, mm = 76		Ring Calibration Factor, KN/div = 0.00153			
Axial Deformation[mm]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [Kpa]
0.00	0	0.0	0.0000	0.00100	0.0
0.20	0.26	5	0.0077	0.00100	7.6
0.40	0.53	7	0.0107	0.00101	10.7
0.60	0.79	11	0.0168	0.00101	16.7
0.80	1.05	15	0.0230	0.00101	22.7
1.00	1.32	19	0.0291	0.00101	28.7
1.20	1.58	22	0.0337	0.00102	33.1
1.40	1.84	30	0.0459	0.00102	45.1
1.60	2.11	35	0.0536	0.00102	52.4
1.80	2.37	37	0.0566	0.00102	55.3
2.00	2.63	38	0.0581	0.00103	56.6
2.20	2.89	39	0.0597	0.00103	57.9
2.40	3.16	40	0.0612	0.00103	59.3
2.60	3.42	39	0.0597	0.00104	57.6
2.80	3.68	38	0.0581	0.00104	56.0

Figure A 3:Axial Strain Vs Axial Stress for TP-1 soil

Table A 9:CBR Laboratory Test Result for TP-1

CBR Test result for TP-1					
Ring Calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/m ²)	St. Stress (N/mm ²)	CBR (%)
0	0	0.00	0.000		
0.64	2	84.86	0.044		
1.27	2.5	106.08	0.055		
1.91	2.5	106.08	0.055		
2.54	3	127.29	0.066	6.9	1.0
3.18	3.5	148.51	0.077		
3.81	3.6	152.75	0.079		
4.45	3.65	154.87	0.080		
5.08	3.65	154.87	0.080	10.3	0.8
7.62	3.7	156.99	0.081		
Swell Determination					
Reading		Gauge rdg. mm		Swell in %	
		Initial	6.88	14.35	
		Final	23.59		
Swell (%)		14.35%			
CBR (%)		1.0%			

The graph plots Load (MPa) on the y-axis (0.00 to 0.09) against Penetration (mm) on the x-axis (0 to 8). The curve starts at (0,0) and rises steeply, passing through points like (0.64, 0.044), (1.27, 0.055), and (1.91, 0.055). It then continues to rise more gradually, reaching a peak load of 0.080 MPa at 5.08 mm penetration. A specific point is highlighted at 2.54 mm penetration with a load of 0.066 MPa, which is used to determine the CBR value of 1.0%.

Figure A 4:Penetration Vs Load for TP-1 soil

Appendix B: Laboratory test result for natural soil sample from test pit two

Table B 1:Natural Moisture content for TP-2

Sample location	Test pit-2			
Can number	A	B	C	D
Mass of the can(g)	65.2	67.5	78.4	62
Mass of can+Mass of moist soil (g)	315.2	317.5	328.4	312
Mass of can +mass of dry soil (g)	261.2	258.9	269.4	257
Mass of moisture (Mw)	54	58.6	59	55
Mass of dry soil (Ms)	196	191.4	191	195
Water content(w)%	27.5	30.6	30.8	28
Average water content(w)%	29.3			

Table B 2:Wet sieve Analysis test result for TP-2

Sieve No.	Sieve opening (m)	Mass of sieve(g)	Mass of sieve + Re. soils(g)	Mass of Retained soils(g)	Percentage Retained (%)	Cum. Percent. Retained (%)	Perc. Passing (%)
No 4	4.75	620	620	0	0	0	100
No 10	2	564	566	2	0.02	0.752	99.2
No 16	1.18	540	543	3	0.05	1.879	98.1
No.30	0.6	509	512	3	0.08	3.01	96.99
No 40	0.425	548	549	1	0.09	3.4	96.6
No50	0.30	476	479	3	0.12	4.5	95.5
No100	0.15	458	460	2	0.14	5.26	94.74
No 200	0.075	455	454	2	0.16	6.02	94
Pan		252	253	1	2.66	100	----

Table B 3:Hydrometer Analysis for TP-2

Elapsed Time, (min)	Actual Hydrometer Rdg	Temp. C ⁰	Effective depth (mm)	Coefficient. (K)	Rdg In Dispersant (R _o)	Modified Hydrometer Rdg, Rd = R _h -R _o	Grain size (mm)	Finer, %	Finer combination
1	50	18	8.1	0.013 78	5	45	0.039 2	88.62	83.30
2	49	18	8.30	0.013 78	5	44	0.028 1	86.65	81.45
5	48	18	8.40	0.013 78	5	43	0.017 9	84.68	79.60
15	46	18	8.80	0.013 78	5	41	0.010 6	80.74	75.90
30	43	18	9.20	0.013 78	5	38	0.007 6	74.83	70.34
60	40	18	9.70	0.013 78	5	35	0.005 5	68.92	64.79
120	38	21	10.10	0.013 28	5	33	0.003 9	64.99	61.09
240	33	21	10.90	0.013 28	5	28	0.002 8	55.14	51.83
480	25	21	12.20	0.013 28	5	20	0.002 1	39.39	37.02
1440	24	21	12.4	0.013 28	5	19	0.001 2	37.42	35.17

Table B 4: Specific gravity of the soil for TP-2

Specimen number	1	2
Pycnometer bottle number	T1	T16
WP = Mass of empty, clean pycnometer (grams)	36.9	43.7
WPS = Mass of empty pycnometer + dry soil (grams)	46.9	5.7
WB = Mass of pycnometer + dry soil + water (grams)	137	143.5
WA = Mass of pycnometer + water (grams)	143.1	149.3
Specific Gravity (GS)	2.86	2.6
Average Specific gravity	2.73	

Table B 5: Swelling properties of the Soil for TP-2

Swelling Characteristic for Test Pit -2 soil		
Description	water	kerosene
Initial reading (A)	10	10
Final reading (B)	22.4	11
Free swell	FS=124	
Free swell index	FSI=103.6	
Free swell ratio	FSR=2.03	

Table B 6:Atterberg limit Tests result for TP-2

Determination	Natural soil TP2					
	Liquid limit (ASTMD4318)			Plastic limit (ASTMD4318)		
Number of blows	19	29	31			
Wt. of container	61	61	59	20	20	20
Wt. of container+ wet soil, (g)	76	73	70	34	27	31
Wt. of container +dry soil, (g)	68	68	65	30	25	28
Wt. of water, (g)	8	6	5	4	2	3
Wt. of dry soil, (g)	7	7	6	10	5	8
Moisture content (%)	114	85.7	83.33	40	40	37.5
Liquid limit	94			39.2		
Plastic index	55					

100 %ES-TP2

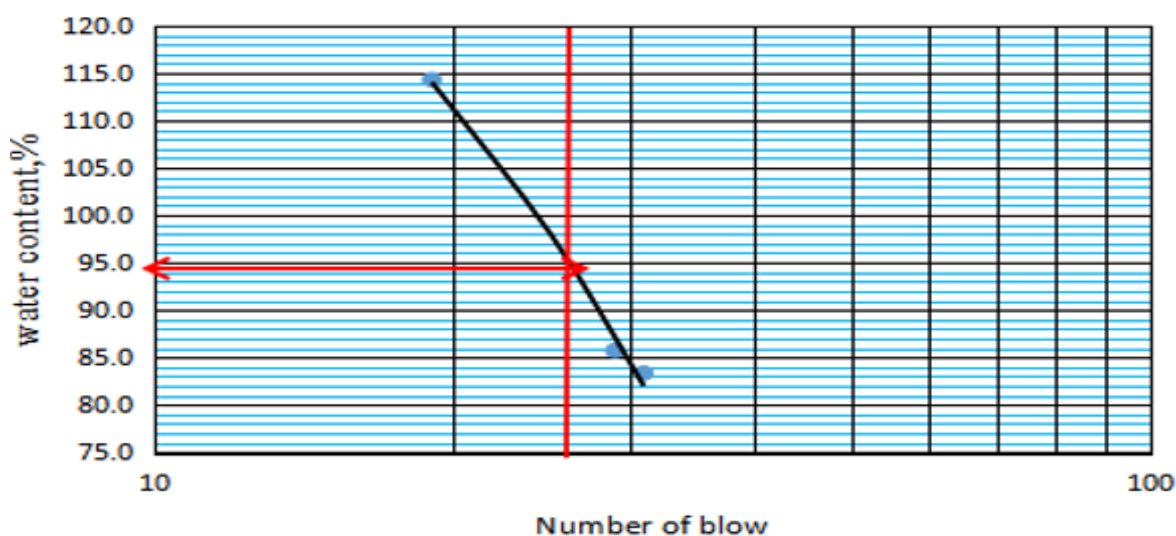


Figure B 1:Liquid limit Curve for test TP-2 soil

Table B 7:Standard Compaction laboratory test result for TP-2

Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5704			5852			6005			5923		
Wt. of mold(g)	4393			4393			4393			4393		
Wt. of Wet soil (g)	1311			1459			1612			1530		
wet density, (g/cm ³)	1.39			1.55			1.71			1.621		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt.(g)	19	20	20	19	20	19	20	19	19	19	19	20
Wet soil+can(g)	61	59	58	57	66	67	77	71	66	96	104	101
Dry soil+can(g)	54	52	51	48	55	55	60	56	52	69	75	73
Wt. of moisture(g),	7	7	7	9	11	12	17	15	14	27	29	28
Dry soil(g)	35	32	31	29	35	36	40	37	33	50	56	53
Moisture content (%)	20	21.8	22.6	31	31.4	33.3	42.5	40.5	42.4	54	51.8	52.8
Avg.moisture content (%)	21.5			31.9			41.8			52.8		
Dry density,	1.14			1.17			1.21			1.06		

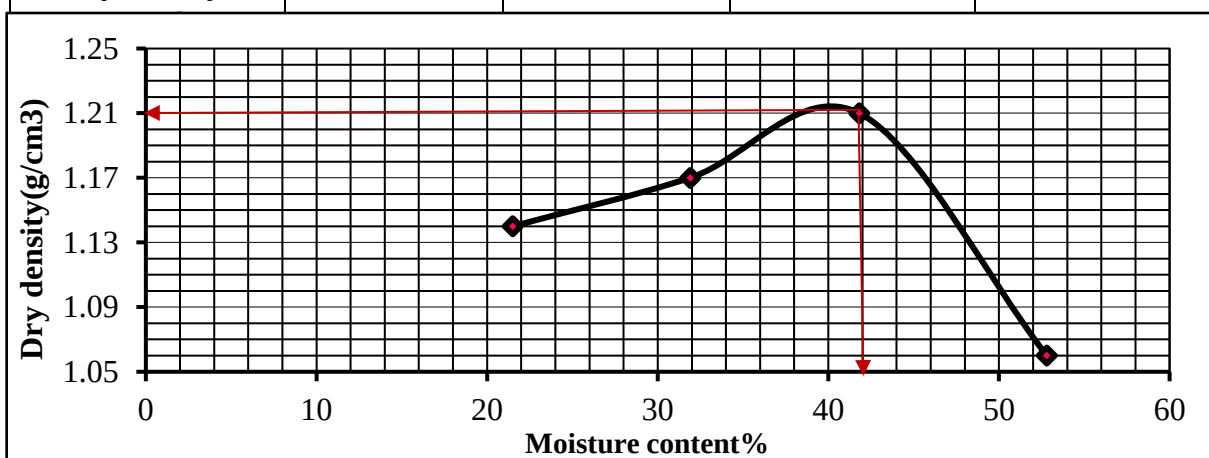


Figure B 2:Compaction Curve for TP-2

Table B 8:Unconfined compressive Strength test result for TP-2

Diameter of sample, mm = 38 Length of sample, mm = 76			Cross- Sectional Area, m ² =0.0010 Ring Calibration Factor/div=0.00153		
Axial Deformation [mm]	Axial strain [%]	Proving Ring reading	Axial Load in [KN]	Correct ed Area [m ²]	Axial stress in [Kpa]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	6	0.0092	0.00100	9.2
0.40	0.0053	9	0.0138	0.00101	13.7
0.60	0.0079	11	0.0168	0.00101	16.7
0.80	0.0105	15	0.0230	0.00101	22.7
1.00	0.0132	20	0.0306	0.00101	30.2
1.20	0.0158	22	0.0337	0.00102	33.1
1.40	0.0184	30	0.0459	0.00102	45.1
1.60	0.0211	38	0.0581	0.00102	56.9
1.80	0.0237	43	0.0658	0.00102	64.2
2.00	0.0263	42	0.0643	0.00103	62.6
2.20	0.0289	41	0.0627	0.00103	60.9

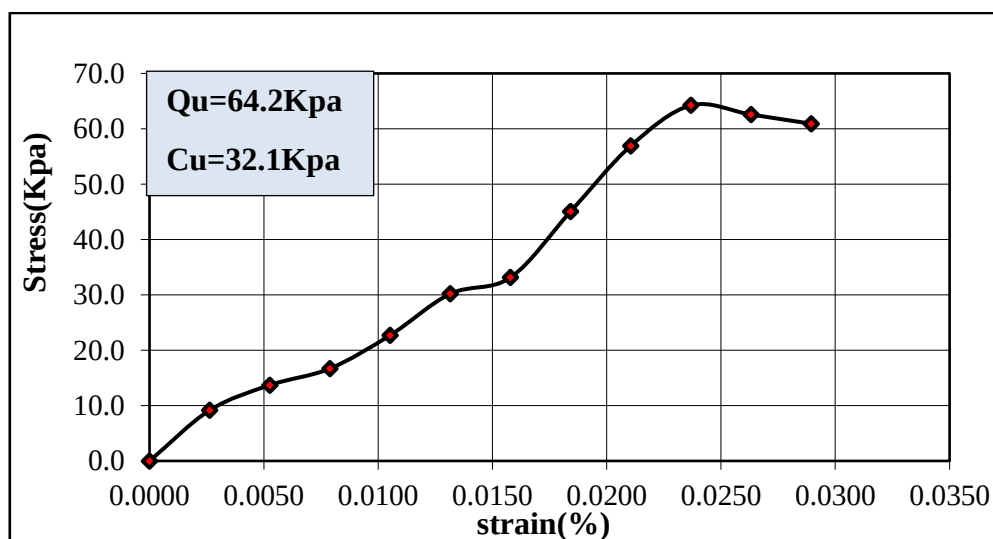


Figure B 3:Strain Vs Stress for TP-2 soil

Table B 9:CBR Test Result for TP-2

CBR Test result for TP-2					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/m ²)	Standard stress(N/mm ²)	CBR (%)
0	0	0	0.000		
0.64	2.5	106.08	0.055		
1.27	3	127.3	0.066		
1.91	3.5	148.51	0.077		
2.54	3.9	165.5	0.086	6.9	1.2
3.18	4	169.7	0.088		
3.81	4	169.72	0.088		
4.45	4.1	173.9	0.090		
5.08	4.2	178.21	0.092	10.3	0.9
7.62	4.2	178.21	0.092		
Swell Determination					
Reading	Gauge rdg. mm			Swell in %	
	Initial		14.53	8.99%	
	Final		25		
Swell	8.99%				
CBR	1.2%				

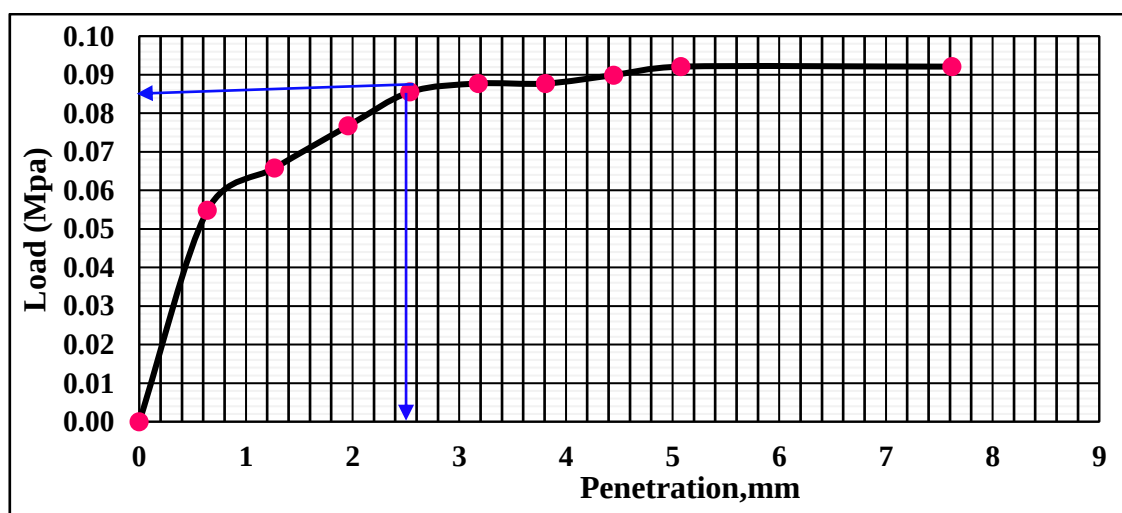


Figure B 4:Penetration Vs Load for TP-2 soil

**Appendix C: Laboratory test result of stabilizing test pit -1 soil sample, using
MCA**

Atterberg limit test Result

Table C 1: Atterberg limit test result of stabilized ES for (5%MCA,95%ES)

Determination	5%MCA+95%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	15	23	33			
Wt. of container	62	61	61	20	20	21
Wt. of container +wet soil, (g)	73	77	73	24	26	28
Wt. of container+ dry soil, (g)	68	70	68	23	24	26
Wt. of water, (g)	5	7	5	1	2	2
Wt. of dry soil, (g)	6	9	7	3	4	5
Moisture content (%)	83.3	77.7	71.4	33.3	50	40
Liquid limit	77.46			41.1		
Plastic index	36.36					

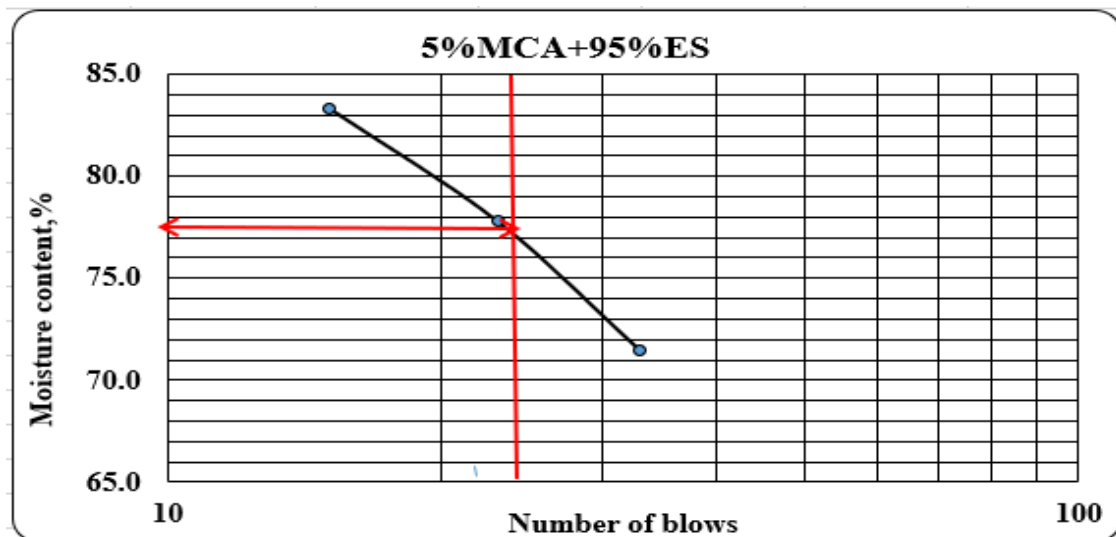


Figure C 1:Liquid limit Curve of stabilized ES for (5%MCA,95%ES)

Table C 2:Atterberg limit test result of stabilized ES with (10%MCA,90%ES)

Determination	10%MCA+90%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	20	28	34			
Wt. of container	20	19	19	19	20	19
Wt. of container wet +soil, (g)	36	40	38	24	25	29
Wt. of container+dry soil, (g)	29	31	30	23	24	25
Wt. of water, (g)	7	9	8	1	1	4
Wt. of dry soil, (g)	9	12	11	4	4	6
Moisture content (%)	77.7	75	72.72	25	25	66.66
Liquid limit	75.1			38.8		
Plastic index	36.3					

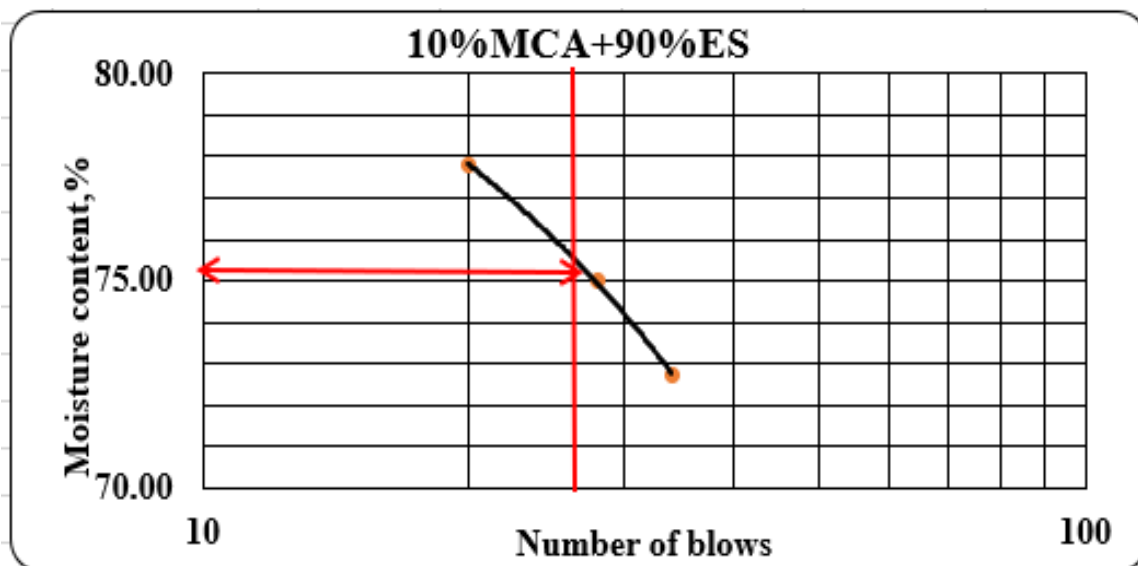


Figure C 2:Liquid limit Curve of stabilized ES for (10%MCA,90%ES)

Table C 3:Atterberg limit test result of stabilized ES for (15%MCA,85%WCP)

Determination	15%MCA+90%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	16	27	32			
Wt. of container	20	20	20	20	19	19
Wt. of container+ wet soil, (g)	40	36	43	35	28	29
Wt. of container+dry soil, (g)	32	30	35	31	26	27
Wt. of water, (g)	8	6	8	4	2	2
Wt. of dry soil, (g)	12	10	15	11	7	8
Moisture content (%)	66.7	60	53.3	36.3	28.6	25
Liquid limit	60			29.5		
Plastic index	30.5					

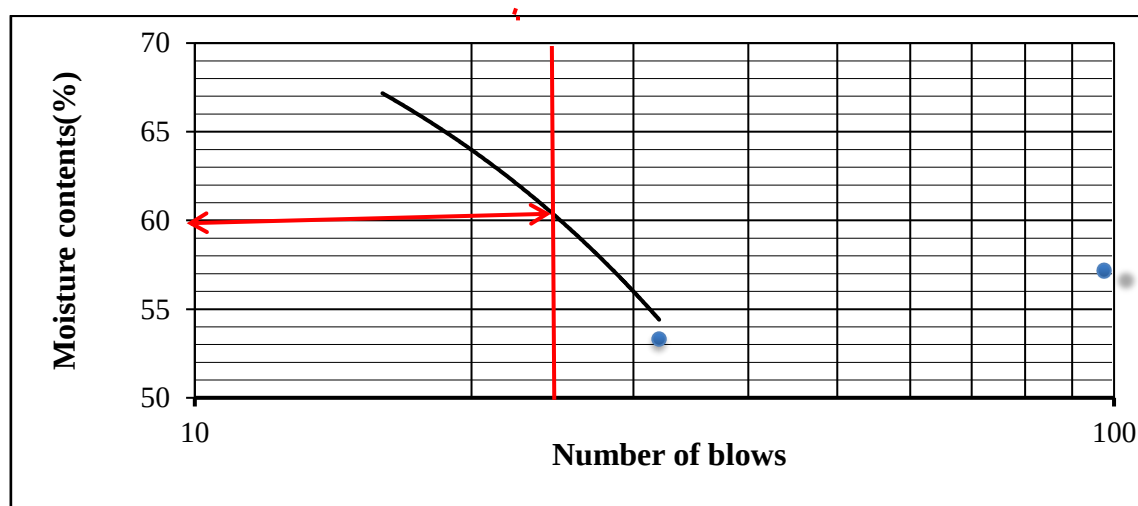


Figure C 3:Liquid limit Curve of stabilized ES for(15%MCA,85%ES)

Table C 4:Atterberg limit test result of stabilized ES for (20%MCA.80%ES)

Determination	20%MCA+90%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	19	28	33			
Wt. of container	61	58.5	61.5	28	19	19
Wt. of container wet soil, (g)	73	70	68	30	27	28
Wt. of container+dry soil, (g)	68	66	66	28	25	26
Wt. of water, (g)	5	4	2	2	2	2
Wt. of dry soil, (g)	7	7.5	4.5	8	6	7
Moisture content (%)	71.4	53.3	44.4	25	33.3	28.6
Liquid limit	56.37			28.93		
Plastic index	27.4					

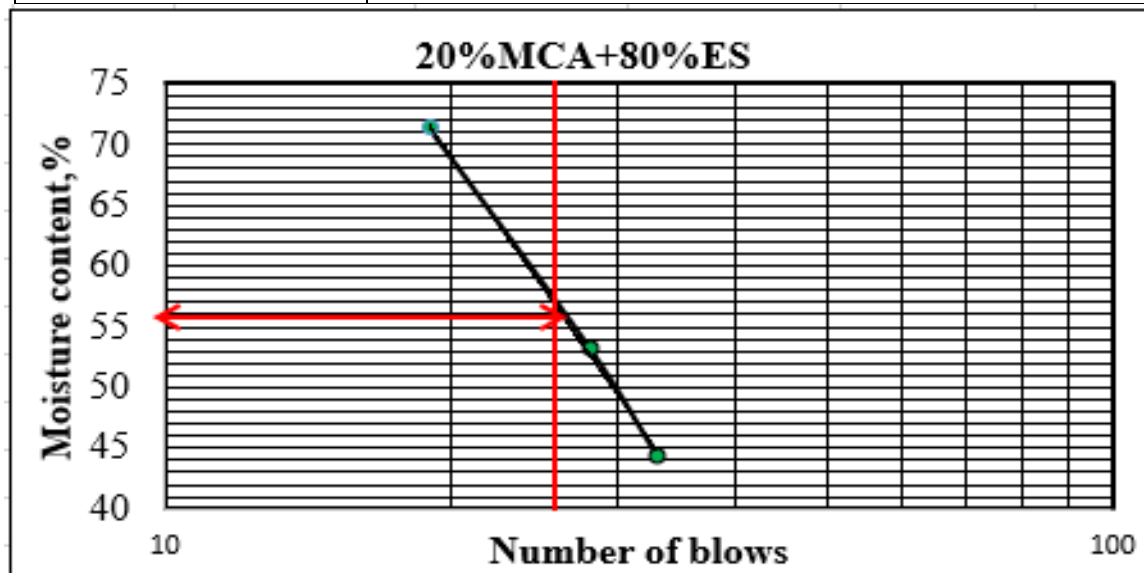


Figure C 4:Liquid limit Curve of stabilized ES for (20%MCA,80%ES)

Table C 5:Effect of maize cob ash on the swelling Characteristics

5% MCA+95%ES			
Water		kerosene	
Initial reading(A)	10	Initial reading	10
Final reading(B)	20	Final reading	10.8
Difference (C=B-A)	10	FSI=85.19	
FS=(C/A)*100	100		
10% MCA+90%ES			
Water		kerosene	
Initial reading(A)	10	Initial reading	10
Final reading(B)	18.8	Final reading	10.5
Difference	8.8	FSI=79.1	
C=B-A			
FS=(C/A)*100	88	FSR=1.79	

15%MCA+85%ES			
Water		Kerosene	
Initial reading(A)	10	Initial reading	10
Final reading(B)	18	Final reading	10.5
Difference(C) C=B-A	8	FSI=71.42	
FS=(C/A)*100	80		
20%MCA+80%ES			
Water		Kerosene	
Initial reading(A)	10	Initial reading	10
Final reading(B)	16.5	Final reading	10.2
Difference C=B-A	6.5	FSI=61.76	
FS=(C/A)*100	65		

Standard Compaction Test Results

Table C 6:Computation of MDD and OMC of stabilized ES for (5%MCA,95%ES)

5%MCA+95%ES															
Trial No.	1			2			3			4			5		
Wt. of Mold +wet soil (g)	5758			5853			5939			6090			6076		
Wt. of mold(g)	4393			4393			4393			4393			4393		
Wt. Wet soil (g)	1365			1460			1546			1697			1683		
Wet density, (g/cm³)	1.45			1.55			1.64			1.80			1.78		
Moisture Content Determination															
Trial No.	1			2			3			4			5		
Can wt.(g)	19	20	1	1	18	1	19	18	19	18	17	18	19	18	18
Wetsoil+can(g)	61	59	5	5	74	5	66	57	60	81	73	85	91	89	95
Dry soil+can(g)	54	52	4	4	59	5	55	50	48	64	58	65	69	69	72
Wt. moisture(g),	7	7	6	6	15	5	11	7	12	17	15	20	22	20	23
Dry soil(g)	35	32	2	2	41	3	36	32	29	46	41	47	50	51	54
Moisture content	20.	21.	2	2	36.6	1	30.6	21.	41.4	37.	36.	42.	44.0	39.2	42.6
Average	21.1			24.5			31.3			38.7			41.9		
Dry density, g/cm³	1.19			1.24			1.25			1.30			1.256		

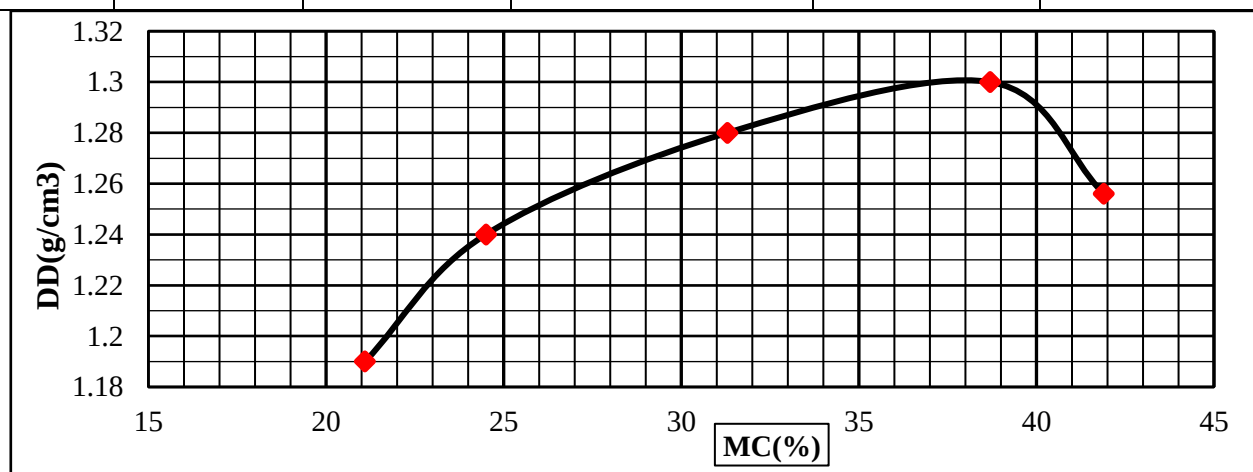


Figure C 5:Compaction curve of stabilized ES for 5%MCA (MDD=1.3g/cm3 and OMC=38.7%)

Table C 7:Computation of MDD and OMC of stabilized ES for (10%MCA,90%ES)

10%MCA+90%ES												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5849			5972			6216			6044		
Wet soil (g)	1456			1579			1823			1651		
Wet density, (g/cm ³)	1.54			1.67			1.93			1.75		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt. (g	19	19	19	18	19	19	20	18	19	19	30	31
Wet soil+can (g	85	66	65	82	94	91	90	92	82	102	138	122
Dry soil+can(g)	73	57	58	67	80	75	65	71	66	76	103	92
Wt. moisture(g),	12	9	7	15	14	16	25	21	16	26	35	30
Dry soil(g)	54	38	39	49	61	56	45	53	47	57	73	61
Moisture	22.	23.6	17.	30.	22.95	28.5	55.5	39.	34.0	45.6	47.9	49.1
Average	21.29			27.38			43.07			47.58		
Dry density/cm ³	1.27			1.31			1.35			1.19		

10%MCA+90%ES

MC(%)	DD(g/cm³)
21.29	1.27
27.38	1.31
43.07	1.35
47.58	1.19

Figure C 6:Compaction curve of stabilized soil for 10%MCA (MDD=1.35g/cm³, OMC=43.1%)

Table C 8:Computation of MDD and OMC of stabilized ES for (15%MCA,85%)

15%MCA+85%ES												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5523			5964			6240			6088		
Wt. of mold(g)	4393			4393			4393			4393		
Wet soil (g)	1130			1571			1847			1695		
Wet density, (g/cm ³)	1.19			1.66			1.96			1.79		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt.(g)	20	20	18	21	19	20	19	18	19	19	20	20
Wt. of Wet soil+can(g)	57	66	58	70	63	70	98	76	70	119	117	124
Dry soil+can(g)	51	60	53	60	53	59	76	61	55	86	85	89
Wt. of moisture(g)	6	6	5	10	10	11	22	15	15	33	32	35
Dry soil(g)	31	40	35	39	34	39	57	43	36	67	65	69
Moisture content (%)	19.4	15.0	14.3	25.6	29.4	28.2	38.6	34.9	41.7	49.3	49.2	50.7
Average moisture content (%)	16.21			27.75			38.37			49.74		
Dry density, g/cm ³	1.0			1.3			1.41			1.2		

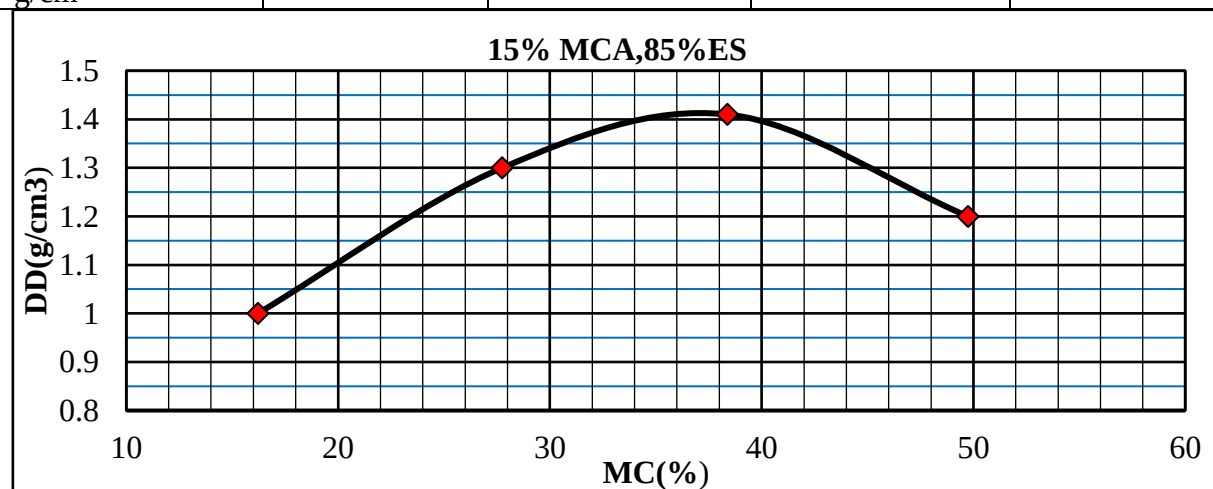


Figure C 7:Compaction curve of stabilized ES for 15%MCA (MDD=1.41g/cm3,OMC=38.37%)

Table C 9:Computation of MDD and OMC of stabilized ES for (20%MCA,80%ES)

20%MCA+80%ES												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5879			6089			6260			5963		
Wt. of mold(g)	4390			4390			4390			4390		
Wet soil (g)	1489			1699			1870			1573		
Wet density, (g/cm ³)	1.577			1.799			1.98			1.67		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt. (g)	20	20	20	18	19	20	18	20	19	19	20	19
Wet soil+can (g)	63	69	78	72	73	70	94	96	95	113	129	120
Dry soil+can(g)	55	60	67	59	59	57	72	73	65	80	90	84
Wt. of moisture(g),	8	9	11	13	14	13	22	23	30	33	39	36
Dry soil(g)	35	40	47	41	40	37	54	53	46	61	70	65
Moisture content (%)	22.86	23.8	24.8	31.7	35	35	40.7	43.3	65.2	54.1	55.7	55.4
Avg.moisture content (%)	22.9			33.94			49.7			55.1		
Dry density, g/cm ³	1.28			1.34			1.324			1.1		

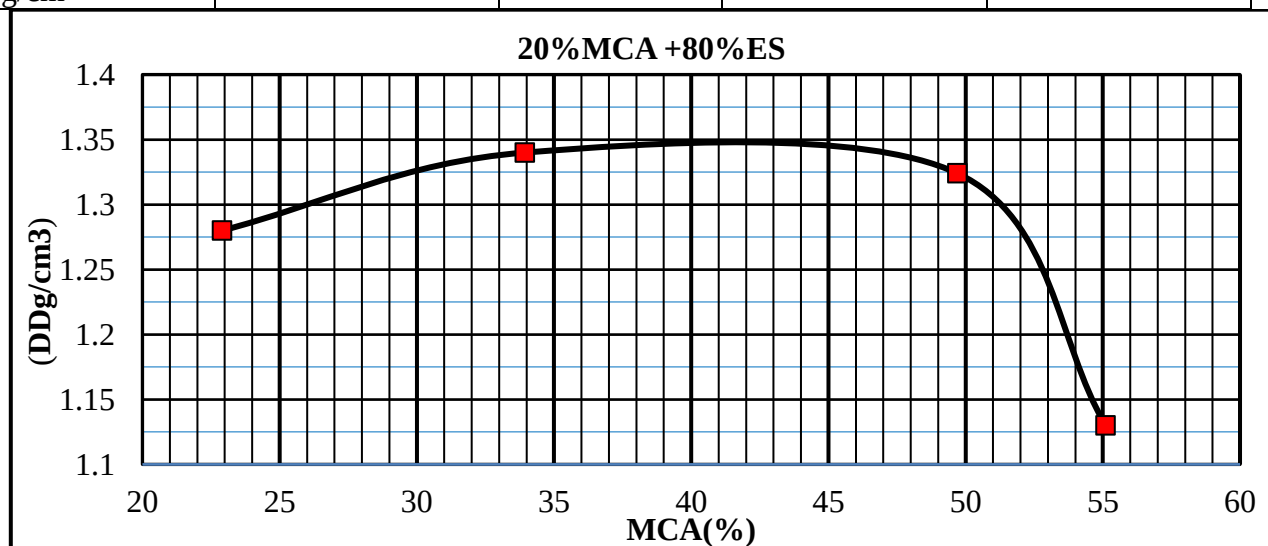


Figure C 8:Compaction curve of stabilized ES for 20%MCA (MDD=1.34g/cm³, OMC=33.94%)

Unconfined compressive strength Test Results

Table C 10:UCS Test result of stabilized ES for (5%MCA,95%ES)

Diameter of sample, mm = 38		Cross- Sectional Area, m ² =		0.0010	
Length of sample, mm = 76		Ring Calibration Factor, KN/div =		0.00153	
Axial Deformation [mm]	Axial strain [%]	Proving Ring reading	Axial Load in [KN]	Corrected Area[m ²]	Axial stress in [Kpa]
0.00	0	0.0	0.0000	0.00100	0.0
0.20	0.26	4	0.0061	0.00100	6.1
0.40	0.53	7	0.0107	0.00101	10.7
0.60	0.79	8	0.0122	0.00101	12.1
0.80	1.05	10	0.0153	0.00101	15.1
1.00	1.32	13	0.0199	0.00101	19.6
1.20	1.58	15	0.0230	0.00102	22.6
1.40	1.84	18	0.0275	0.00102	27.0
1.60	2.11	24	0.0367	0.00102	35.9
1.80	2.37	26	0.0398	0.00102	38.8
2.00	2.63	30	0.0459	0.00103	44.7
2.20	2.89	33	0.0505	0.00103	49.0
2.40	3.16	35	0.0536	0.00103	51.9
2.60	3.42	39	0.0597	0.00104	57.6
2.80	3.68	41	0.0627	0.00104	60.4
3.00	3.95	43	0.0658	0.00104	63.2
3.20	4.21	46	0.0704	0.00104	67.4
3.40	4.47	47	0.0719	0.00105	68.7
3.60	4.74	47	0.0719	0.00105	68.5
3.80	5	48	0.0734	0.00105	69.8
4.00	5.26	47	0.0719	0.00106	68.1
4.20	5.53	46	0.0704	0.00106	66.5

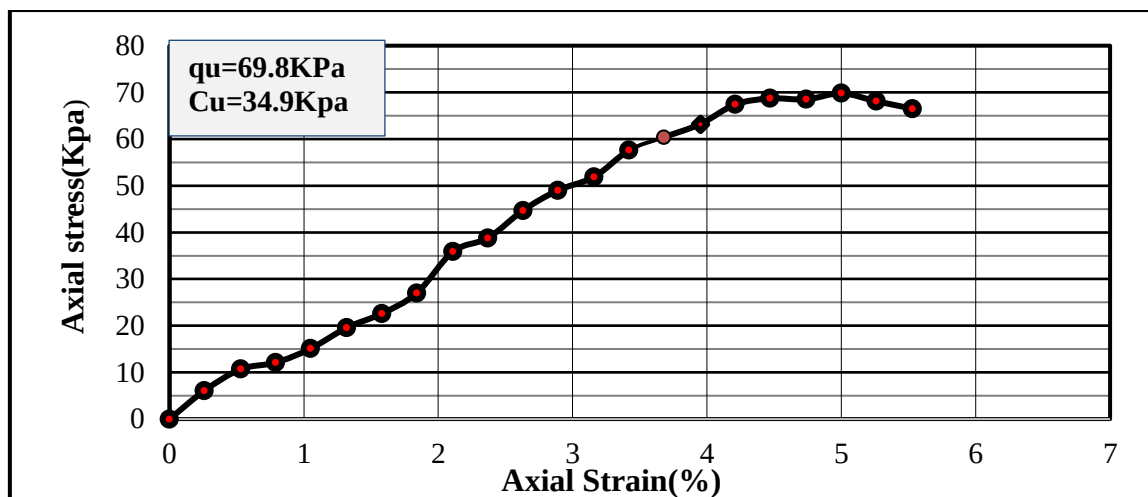


Figure C 9: Axial strain Vs Axial stress for (5%MCA ,95%ES)

Table C 11: UCS Test result of stabilized ES for (10%MCA,90%)

Diameter of sample, mm = 38 Length of sample, mm = 76			Cross- Sectional Area, m ² =0.0010 Ring Calibration Factor/div=0.00153		
Axial Deformation [mm]	Axial Strain [%]	Proving Ring reading	Axial Load [KN]	Corrected Area[m ²]	Axial stress [Kpa]
0.00	0	0.0	0.0000	0.00100	0.0
0.20	0.26	12	0.0184	0.00100	18.3
0.40	0.53	18	0.0275	0.00101	27.4
0.60	0.79	26	0.0398	0.00101	39.5
0.80	1.05	33	0.0505	0.00101	50.0
1.00	1.32	35	0.0536	0.00101	52.8
1.20	1.58	38	0.0581	0.00102	57.2
1.40	1.84	41	0.0627	0.00102	61.6
1.60	2.11	43	0.0658	0.00102	64.4
1.80	2.37	45	0.0689	0.00102	67.2
2.00	2.63	46	0.0704	0.00103	68.5
2.20	2.89	47	0.0719	0.00103	69.8
2.40	3.16	49	0.0750	0.00103	72.6
2.60	3.42	50	0.0765	0.00104	73.9
2.80	3.68	50	0.0765	0.00104	73.7
3.00	3.95	50	0.0765	0.00104	73.5
3.20	4.21	50	0.0765	0.00104	73.3
3.40	4.47	49	0.0750	0.00105	71.6
3.60	4.74	48	0.0734	0.00105	70.0

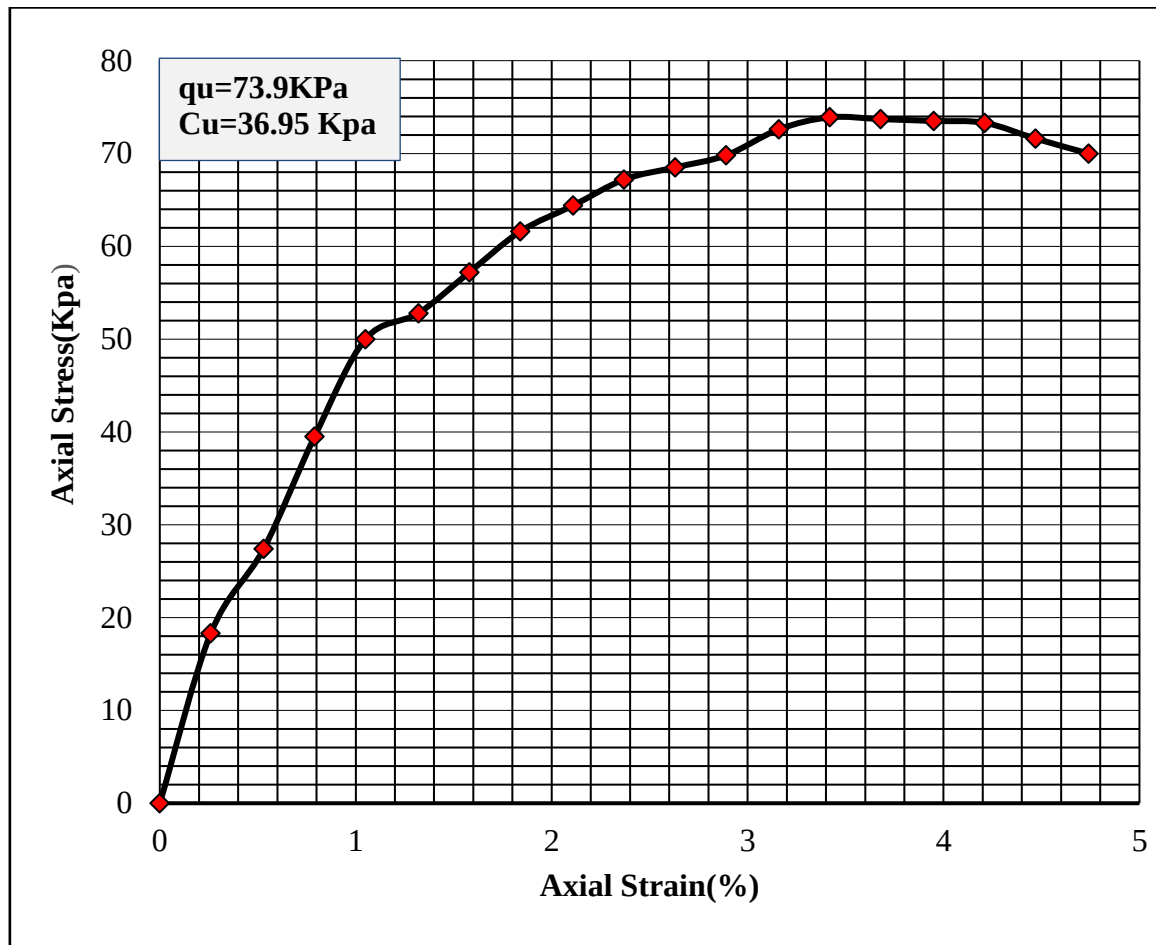


Figure C 10: Axial strain Vs Axial stress for (10%MCA, 95% ES)

Table C 12: UCS Test result of stabilized ES for (15%MCA,85%MCA)

Diameter of sample, mm = 38		Cross- Sectional Area, m ² =		0.0010	
Length of sample, mm = 76		Ring Calibration Factor, KN/div =		0.00153	
Deformation dial in [mm]	Axial strain[mm]	Proving	Axial Load	Corrected Area[m ²]	Axial stress [Kpa]
0.00	0	0.0	0.0000	0.00100	0.0
0.20	0.26	7	0.0107	0.00100	10.7
0.40	0.53	10	0.0153	0.00101	15.2
0.60	0.79	12	0.0184	0.00101	18.2
0.80	1.05	14	0.0214	0.00101	21.2
1.00	1.32	16	0.0245	0.00101	24.2
1.20	1.58	19	0.0291	0.00102	28.6
1.40	1.84	22	0.0337	0.00102	33.0
1.60	2	25	0.0383	0.00102	37.4
1.80	2.37	28	0.0428	0.00102	41.8
2.00	2.63	33	0.0505	0.00103	49.2
2.20	2.89	35	0.0536	0.00103	52.0
2.40	3.16	38	0.0581	0.00103	56.3
2.60	3.42	44	0.0673	0.00104	65.0
2.80	3.68	45	0.0689	0.00104	66.3
3.00	3.95	46	0.0704	0.00104	67.6
3.20	4.21	50	0.0765	0.00104	73.3
3.40	4.47	52	0.0796	0.00105	76.0
3.60	4.74	53	0.0811	0.00105	77.2
3.80	5	53	0.0811	0.00105	77.0
4.00	5.26	54	0.0826	0.00106	78.3
4.20	5.53	54	0.0826	0.00106	78.1
4.40	5.79	53	0.0811	0.00106	76.4
4.60	6.05	53	0.0811	0.00106	76.2
4.80	6.32	52	0.0796	0.00107	74.5

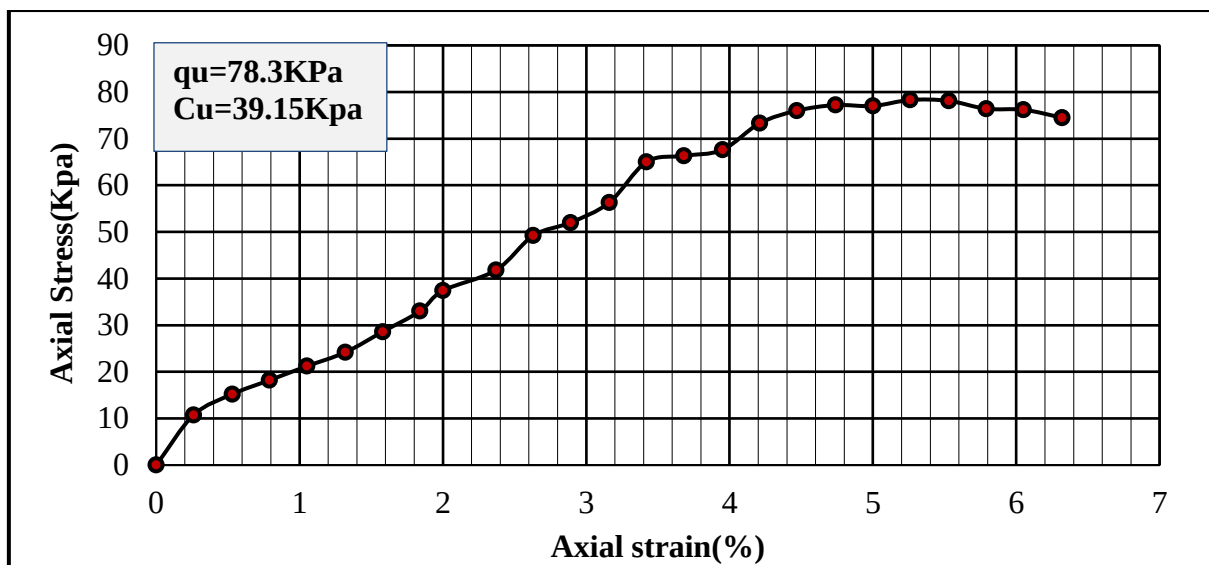


Figure C 11:Axial strain Vs Axial stress for (15%MCA ,85%ES)

Table C 13:UCS Test result of stabilized ES for (20%MCA,80%ES)

Diameter of sample, mm = 38 Length of sample, mm = 76			Cross- Sectional Area, m ² =0.0010 Ring Calibration Factor, kN/div=0.00153		
Axial Deformation [mm]	Axial Strain (%)	Proving Ring reading	Axial Load [KN]	Corrected area[m ²]	Axial stress [Kpa]
0.00	0.00	0.0	0.0000	0.00100	0.0
0.20	0.26	2	0.0031	0.00100	3.1
0.40	0.53	3	0.0046	0.00101	4.6
0.60	0.79	5	0.0077	0.00101	7.6
0.80	1.05	10	0.0153	0.00101	15.1
1.00	1.32	12	0.0184	0.00101	18.1
1.20	1.58	15	0.0230	0.00102	22.6
1.40	1.84	19	0.0291	0.00102	28.5
1.60	2.11	23	0.0352	0.00102	34.4
1.80	2.37	26	0.0398	0.00102	38.8
2.00	2.63	28	0.0428	0.00103	41.7
2.20	2.89	32	0.0490	0.00103	47.5
2.40	3.16	34	0.0520	0.00103	50.4
2.60	3.42	38	0.0581	0.00104	56.2
2.80	3.68	40	0.0612	0.00104	58.9
3.00	3.95	42	0.0643	0.00104	61.7

3.20	4.21	43	0.0658	0.00104	63.0
3.40	4.47	44	0.0673	0.00105	64.3
3.60	4.74	46	0.0704	0.00105	67.0
3.80	5.00	48	0.0734	0.00105	69.76
4.00	5.26	49	0.0750	0.00106	71.0
4.20	5.53	50	0.0765	0.00106	72.3
4.40	5.79	51	0.0780	0.00106	73.5
4.60	6.05	52	0.0796	0.00106	74.75
4.80	6.32	53	0.0811	0.00107	75.97
5.00	6.58	53	0.0811	0.00107	75.76
5.20	6.84	54	0.0826	0.00107	76.97
5.40	7.11	54	0.0826	0.00108	76.8

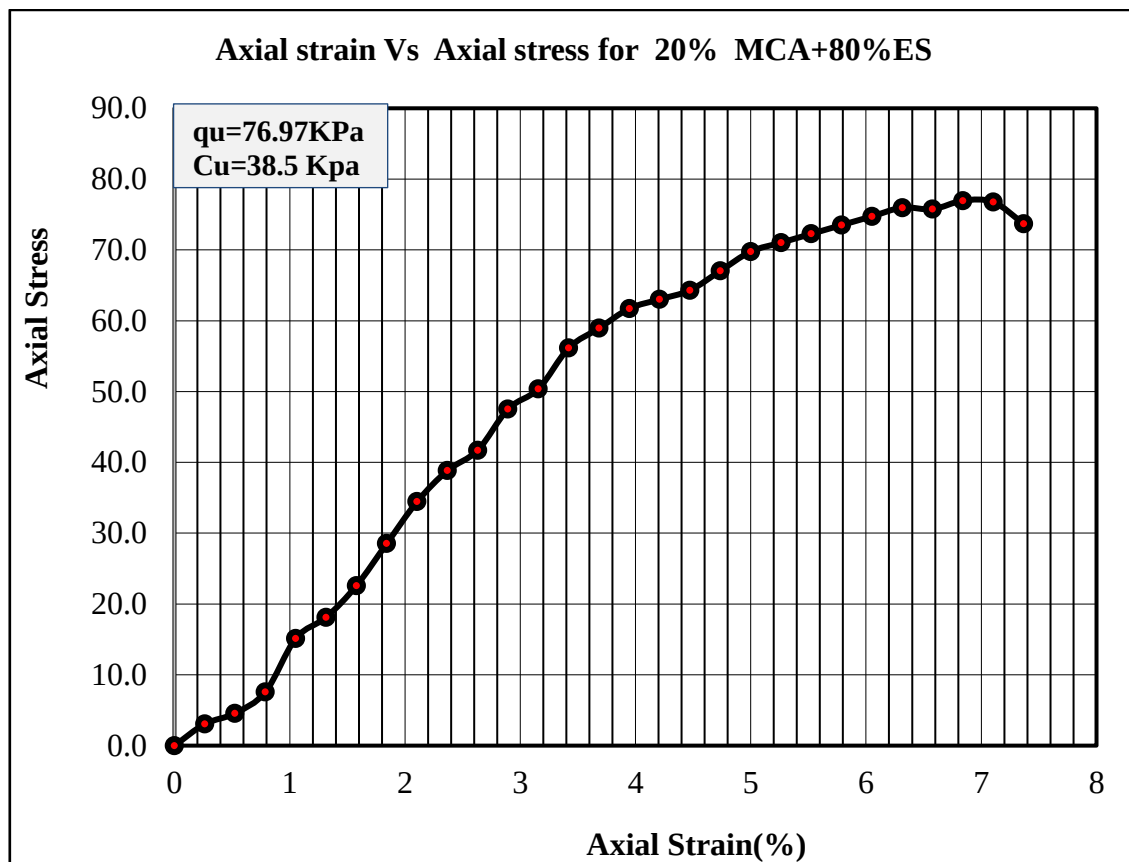


Figure C 12: Axial strain Vs Axial stress for (20%MCA ,80%ES))

CBR tests for different content of MCA

Table C 14:CBR test result of stabilized ES with 5%MCA

5%MCA+95%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/m ²)	Standard stress(N/mm ²)	CBR (%)
0	0	0.00	0.00		
0.64	3	127.29	0.07		
1.27	5	212.15	0.11		
1.96	5	212.15	0.11		
2.54	7	297.01	0.15	6.9	2.2
3.18	7.5	318.23	0.16		
3.81	8	339.44	0.18		
4.45	8	339.44	0.18		
5.08	8	339.44	0.18	10.3	1.7
7.62	8.5	360.66	0.19		
Swell Determination					
Reading	Gauge rdg. mm			Swell in %	
	Initial		10.4	7.31	
	Final		18.9		
Swell	7.31				
CBR	2.2				

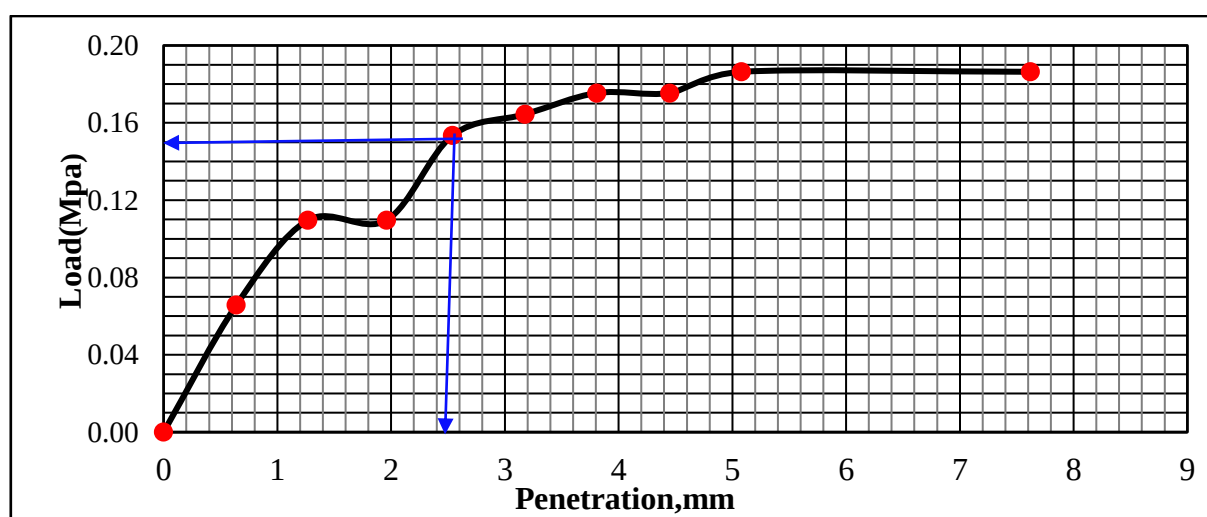


Figure C 13:Stress Vs penetration curve of soaked CBR for (5%MCA,95%ES)

Table C 15:CBR tests result of stabilized ES with 10%MCA

10%MCA+90%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/mm ²)	St. Stress(N/mm ²)	CBR (%)
0	0	0	0.00		
0.64	6	254.58	0.13		
1.27	8	339.44	0.18		
1.91	10	424.3	0.22		
2.54	12	509.16	0.26	6.9	3.8
3.18	14	594.02	0.31		
3.81	16	678.88	0.35		
4.45	16.5	700.1	0.36		
5.08	17	721.3	0.37	10.3	3.6
7.62	19	806.17	0.42		
Swell Determination					
Gauge rdg. mm				Swell in %	
Reading	Initial	12		6.88	
	Final	20			
Swell	6.88				
CBR	3.8				

Penetration (mm)	Load (MPa)
0	0.00
0.64	0.13
1.27	0.18
1.91	0.22
2.54	0.26
3.18	0.31
3.81	0.35
4.45	0.36
5.08	0.37
7.62	0.42

Figure C 14:Stress Vs penetration curve of soaked CBR for (10%MCA,90%ES)

Table C 16:CBR tests result of stabilized ES with 15%MCA

15%MCA+85%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/mm)	Standard stress(N/mm ²)	CBR (%)
0	0	0	0.00		
0.64	8	339.4	0.18		
1.27	10	424.3	0.22		
1.91	14	594.0	0.31		
2.54	15	636.5	0.33	6.9	4.8
3.18	17	721.3	0.37		
3.81	20	848.6	0.44		
4.45	21	891	0.46		
5.08	22	933.5	0.48	10.3	4.7
7.62	21.5	912.2	0.47		
Swell Determination					
Gauge rdg. mm				Swell in %	
Initial Reading			8.3	5.76	
Final reading			15		
Swell		5.76			
CBR		4.8			

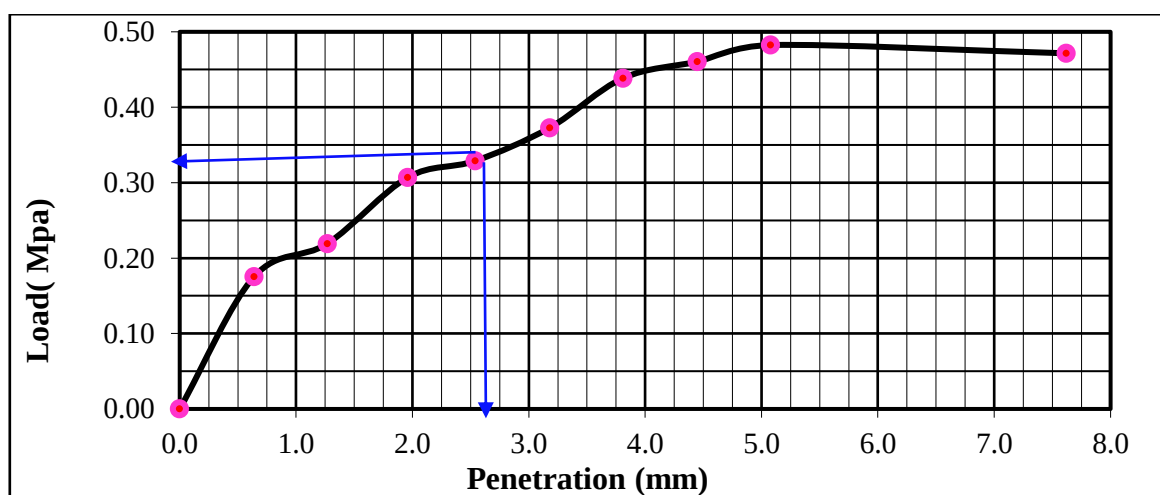


Figure C 15:Stress Vs penetration curve of soaked CBR for 15 % MCA

Table C 17:CBR tests result of stabilized ES with 20%MCA

20%MCA+80%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking					
Pen(mm)	Ring (Div)	Load(N)	Stress (N/mm ²)	Standard stress(N/m ²)	CBR (%)
0	0	0	0.00		
0.64	6.5	275.79	0.14		
1.27	8.5	360.65	0.19		
1.91	12	509.16	0.26		
2.54	13	551.59	0.29	6.9	4.1
3.18	14	594	0.31		
3.81	15	636.45	0.33		
4.45	15.5	657	0.34		
5.08	16	678.88	0.35	10.3	3.4
7.62	16	678.88	0.35		
Swell Determination					
No of blows		Gauge rdg. mm		Swell in %	
56		Initial	11	0.24	
		Final	17		
Swell		5.2			
CBR		4.1			

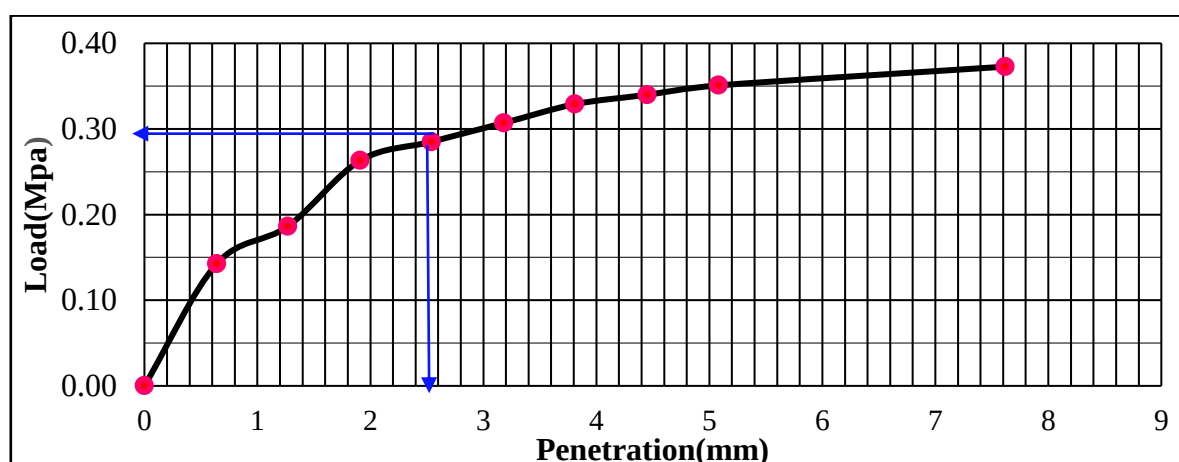


Figure C 16:Stress Vs penetration curve of soaked CBR for 20% MCA

Appendix D: Laboratory test result of stabilizing TP-1 soil using WCP

Atterberg limit Test Result

Table D 1:Atterberg limit Test Result of Stabilized ES for (5%WCP,95%ES)

Determination	5% WCP+95%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	18	28	35			
Wt. of container	18	18	20	19	17	19
wt. Of container wet soil, (g)	40	38	38	23	24	24
Wt. of container+dry soil, (g)	30	29	30	22	22	23
Wt. of water, (g)	10	9	8	1	2	1
Wt. of dry soil, (g)	12	11	10	3	5	4
Moisture content (%)	83.3 3	81.8	80	33.3	40	25
Liquid limit	81.7			32.77		
Plastic index	48.93					

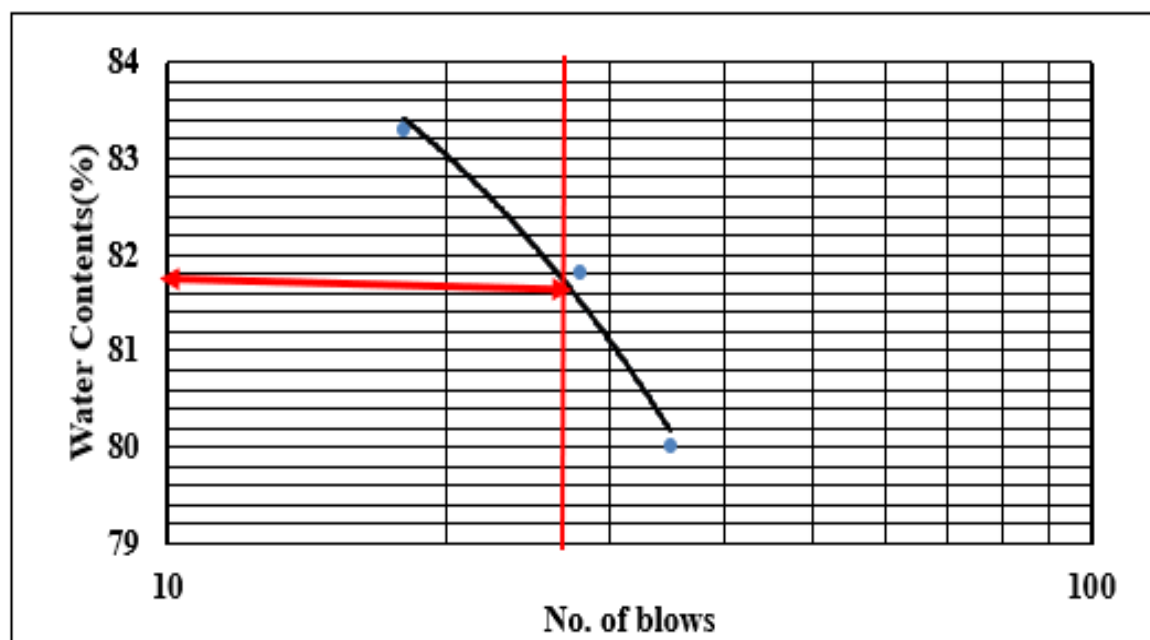


Figure D 1:Liquid limit graph of stabilized ES for(5%WCP,95%ES)

Table D 2:Atterberg limit Test Result of Stabilized ES for (10%WCP,90%ES)

Determination	10%WCP+90%ES					
	liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	17	28	34			
Wt. of container	17	19	19	20	20	21
Wt. of container+wet soil, (g)	37	37	35	23	24	25
Wt. of container+dry soil, (g)	28	29	28	22	23	24
Wt. of water, (g)	9	8	7	1	1	1
Wt. of dry soil, (g)	11	10	9	2	3	3
Moisture (%) content	81.82	80.00	77.78	50.00	33.33	33.33
Liquid limit	79.9			38.9		
Plastic index	41.0					

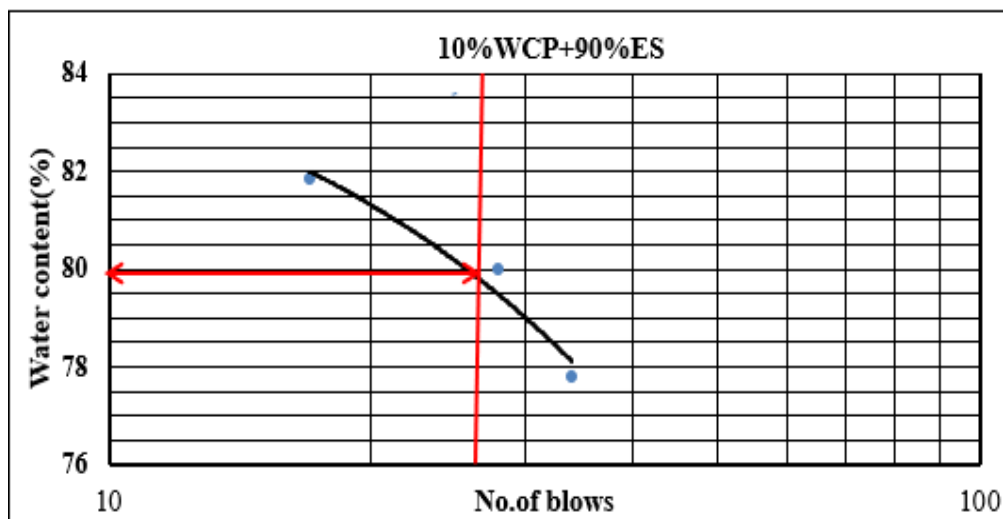


Figure D 2:Liquid limit graph of stabilized ES for(10%WCP,90%ES)

Table D 3:Atterberg limit Test Result of Stabilized ES for (15%WCP,85%ES)

Determination	15%WCP+85%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	19	28	35			
Wt. of container	21	21	22	19	19	19
Wt. of container+wet soil, (g)	41	39	38	23	22	24
Wt. of container+dry soil, (g)	32	31	32	22	21	23
Wt. of water, (g)	9	8	6	1	1	1
Wt. of dry soil, (g)	11	10	10	3	2	4
Moisture content (%)	81.82	80.00	60.00	33.33	50.00	25.00
Liquid limit	73.9			36.1		
Plastic index	37.8					

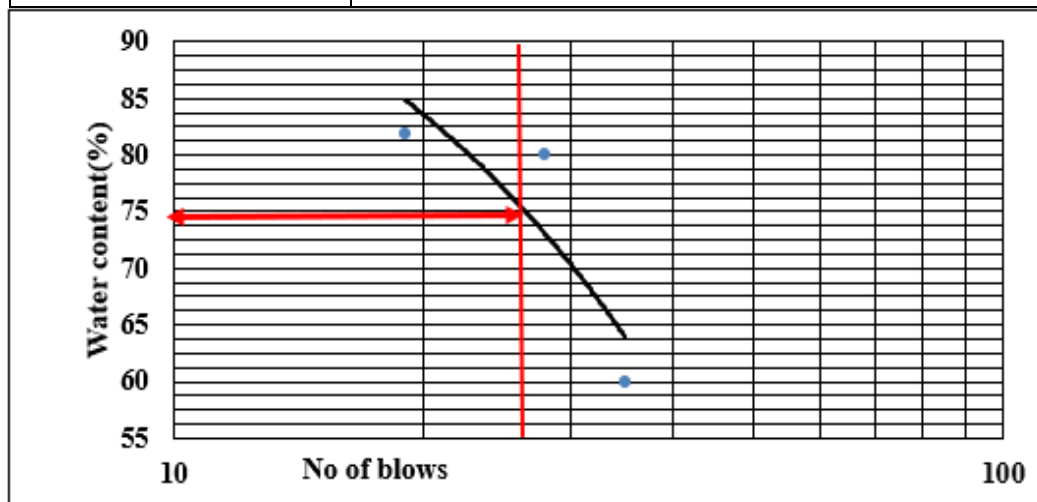


Figure D 3:Liquid limit graph of stabilized ES for (15%WCP,85%ES)

Table D 4:Atterberg limit Test Result of Stabilized ES for (20%WCP,80%)

Determination	20%WCP+80%ES					
	Liquid limit (ASTM4318)			Plastic limit (ASTM4318)		
Number of blows	17	24	33			
Wt. of container	19	19	19	19	19	18
Wt. of container+wet soil, (g)	38	43	41	26	30	34
Wt. of container+dry soil, (g)	30	33	32	25	26	30
Wt. of water, (g)	8	10	9	1	4	4
Wt. of dry soil, (g)	11	14	13	6	7	12
Moisture content (%)	72.73	71.43	69.23	16.67	57.14	33.33
Liquid limit	71.13			35.71		
Plastic index	35.41					

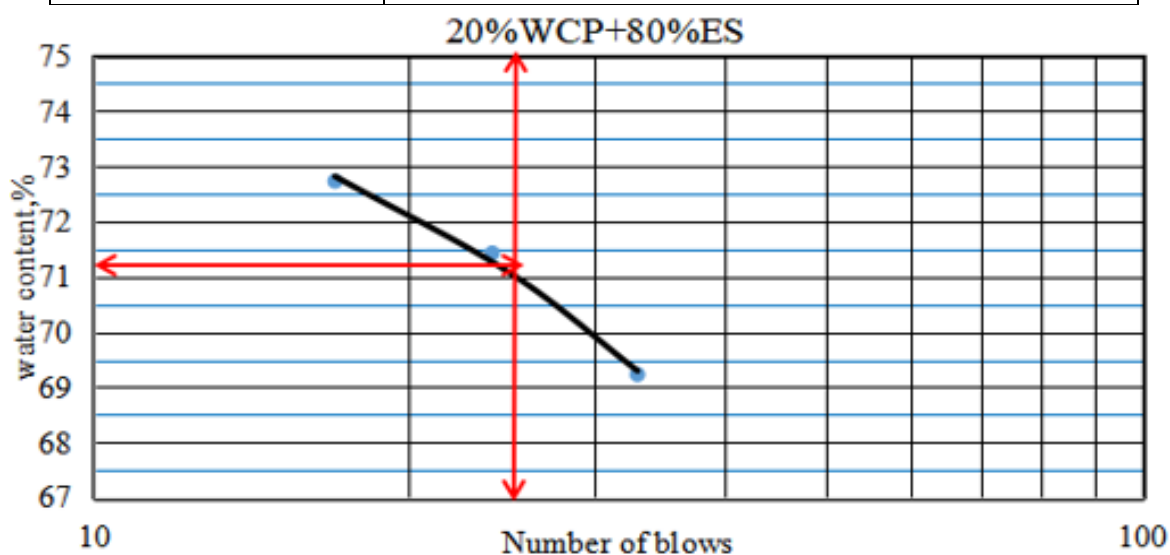


Figure D 4:Liquid limit graph of stabilized ES for (20%WCP,80%)

2. Effect of waste ceramic powder on the swelling characteristics of Expansive soil

Table D 5: Effect of WCP on the Swelling characteristics

5% WCP+95%ES			
Water		kerosene	
Initial reading(A)	10	Initial readin g	10
Final reading(B)	22	Final readin g	11
Difference C=B-A	12	FSI=100	
FS=(C/A)*100	120	FSR=2	
10% WCP+90%ES			
Water		kerosene	
Initial reading(A)	10	Initial readin g	10
Final reading(B)	19.5	Final readin g	10
Difference C=B-A	9.5	FSI=95	
FS=(C/A)*100	95	FSR=1.95	

15 % WCP+85%ES			
Water		kerosene	
Initial reading(A)	10	Initial readin g	10
Final reading(B)	18	Final readin g	10.5
Difference C=B-A	8	FSI=71.43	
FS=(C/A)*100	80	FSR=1.71	
20 % WCP+80%ES			
Water		kerosene	
Initial reading(A)	10	Initial readin g	10
Final reading(B)	17	Final readin g	10.49
Difference C=B-A	7	FSI=62	
FS=(C/A)*100	70	FSR=1.621	

3. Laboratory compaction Tests for different content of WCP

Table D 6: Computation of MDD and OMC of Stabilized ES for (5%WCP,95%ES)

5%WCP+90%ES												
Trial No.	1			2			3			4		
Wt. of Mold	5704			5852			6005			5923		
+wet soil (g)												
Wt. of	4393			4393			4393			4393		
Wet soil (g)	1311			1459			1612			1530		
Wet density,	1.39			1.55			1.71			1.62		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt.(g)	19	2	20	19	20	19	20	19	19	19	19	20
Wet soil+C(g)	61	5	58	57	66	67	77	71	66	96	10	101
		9									4	
Dry soil+C(g)	54	5	51	48	55	55	60	56	52	69	75	73
		2										
M.of moisture	7	7	7	9	11	12	17	15	14	27	29	28
Dry soil(g)	35	3	31	29	35	36	40	37	33	50	56	53
M.content (%)	20	2	22.	31.	31.4	33.3	42.5	40.54	42.	54.	51.	52.83
		1.	58	03	3	3	0		4	00	79	
		8										
Average w(%)	21.49			31.93			41.82			52.87		
Dry density,	1.14			1.17			1.21			1.06		

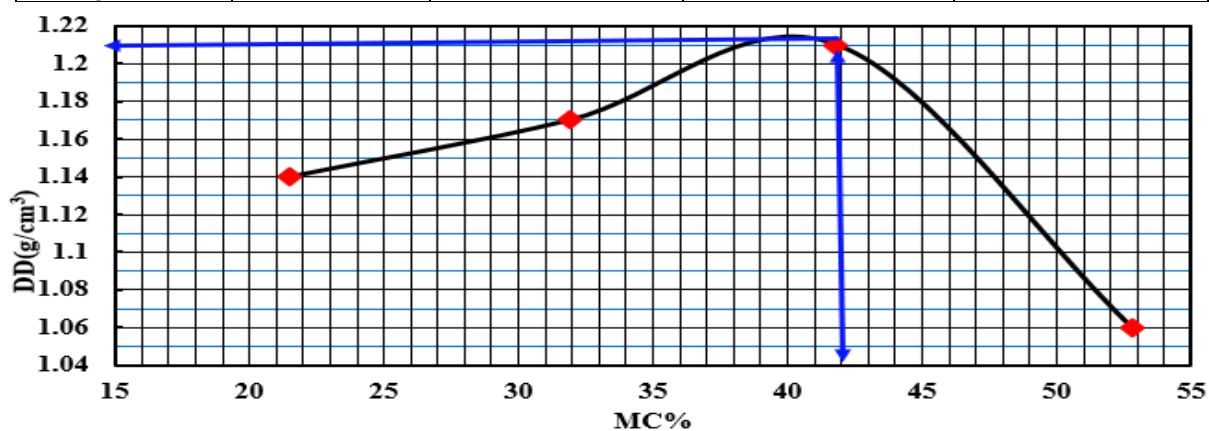


Figure D 5: Compaction curve of stabilized ES for 5%WCP (1.21g/cm³, OMC=41.82%)

Table D 7: Computation of MDD and OMC of stabilized ES for (10%WCP, 90%ES)

10%WCP+90%ES												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5638			5911			6040			5974		
Wt. of mold(g)	4393			4393			4393			4393		
Wt. of wet soil (g)	1245			1518			1690			1581		
Wet density, (g/cm ³)	1.32			1.61			1.79			1.67		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt. (g)	19	18	18	19	19	17	19	19	18	17	19	19
Wet soil+can (g)	44	49	55	53	54	51	88	84	89	91	94	87
Dry soil+can(g)	40	45	50	47	46	44	68	67	69	67	68	69
Wt. of moisture(g),	4	4	5	6	8	7	20	17	20	24	26	18
Dry soil(g)	21	27	32	28	27	27	49	48	51	50	49	50
Moisture content (%)	19.0	14.8	15.6	21.4	29.6	25.9	40.8	35.4	39.2	48.0	53.1	36.0
Average moisture	16.50			25.66			38.48			45.69		
Dry density, g/cm ³	1.132			1.280			1.293			1.150		

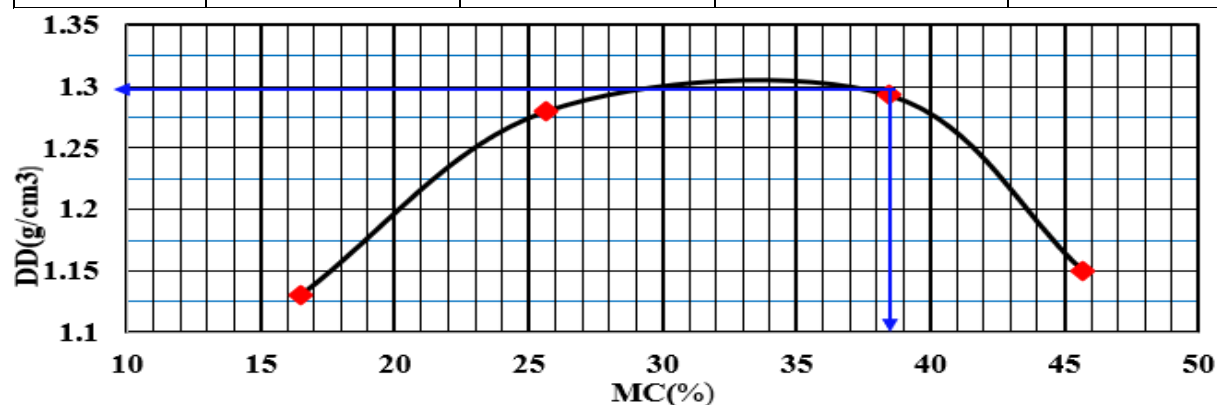


Figure D 6: Compaction curve of stabilized ES for 10%WCP (MDD=1.29g/cm³, OMC=38.48%)

Table D 8:Computation of MDD and OMC of stabilized ES for (15%WCP, 85%MCA)

15%WCP+85%ES											
Trial No.	1			2			3			4	
Wt. of Mold +wet soil (g)	5821			5871			6118			6050	
Wt. of mold(g)	4393			4393			4393			4393	
Wet soil (g)	1428			1478			1725			1657	
Wet density, (g/cm³)	1.51			1.57			1.83			1.76	
Moisture Content Determination											
Trial No.	1			2			3			4	
Can wt. (g	20	19	20	20	19	20	19	20	20	19	20
Wet soil+C(g)	77	69	83	78	80	83	92	88	92	115	118
Dry soil+C(g)	69	61	66	66	66	70	73	70	72	83	85
Wt. of moisture (g),	8	8	17	12	14	13	19	18	20	32	33
Dry soil(g)	49	42	46	46	47	50	54	50	52	64	65
Moisture content (%)	16.3	19	36.95	26.1	29.8	26.0	35.2	36.0	38.5	50.0	50.8
Average MC (%)	24.08			27.29			36.55			50.4	
Dry density, g/cm³	1.22			1.23			1.34			1.17	

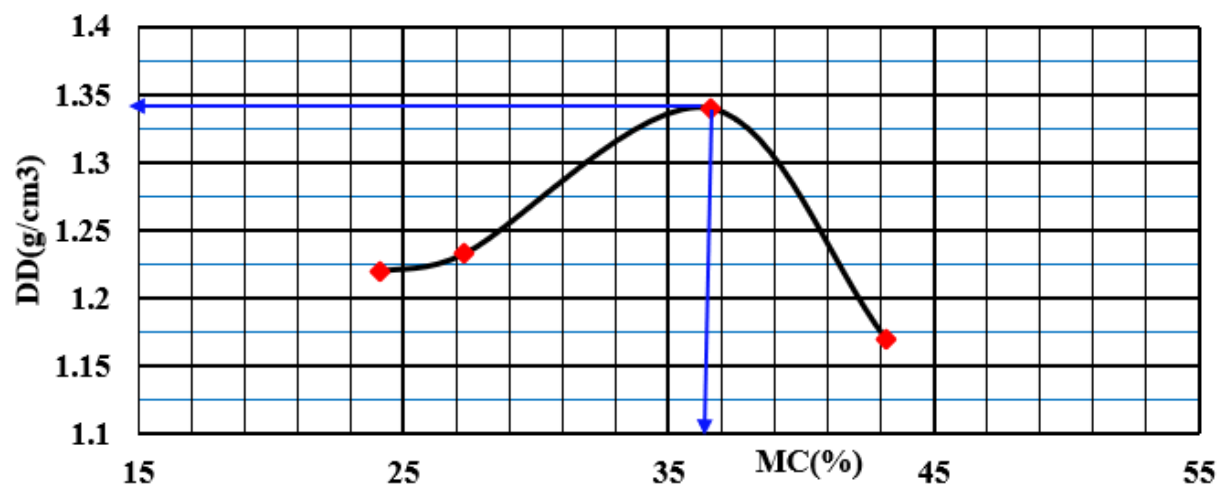


Figure D 7:Compaction curve of stabilized ES for15%WCP (MDD=1.34g/cm3, OMC=36.5%)

Table D 9:Computation of MDD and OMC of stabilized ES for (20%WCP,80%ES)

20% WCP+90%ES												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5762			6133			6035			6030		
Wt. of mold(g)	4393			4393			4393			4393		
Wet soil (g)	1369			1740			1642			1637		
Wet density, (g/cm ³)	1.45			1.84			1.74			1.73		
Moisture Content Determination												
Trial No.	1			2			3			4		
Can wt.(g)	17	18	19	20	20	19	18	20	20	20	19	19
Wet soil+C	59	58	51	79	67	76	82	71	95	94	102	99
Dry soil+C	51	51	46	66	57	65	68	60	76	73	66	76
Wt. of moisture(g),	8	7	5	13	10	11	14	11	19	21	36	23
Dry soil(g)	34	33	27	46	37	46	50	40	56	53	47	57
MC (%)	23.53	21.21	18.52	28	27	24	28.00	27.50	33.93	39.62	76.60	40.35
Avg MC (%)	21.1			26.3			29.8			52.2		
Dry density, g/cm ³	1.23			1.46			1.34			1.14		

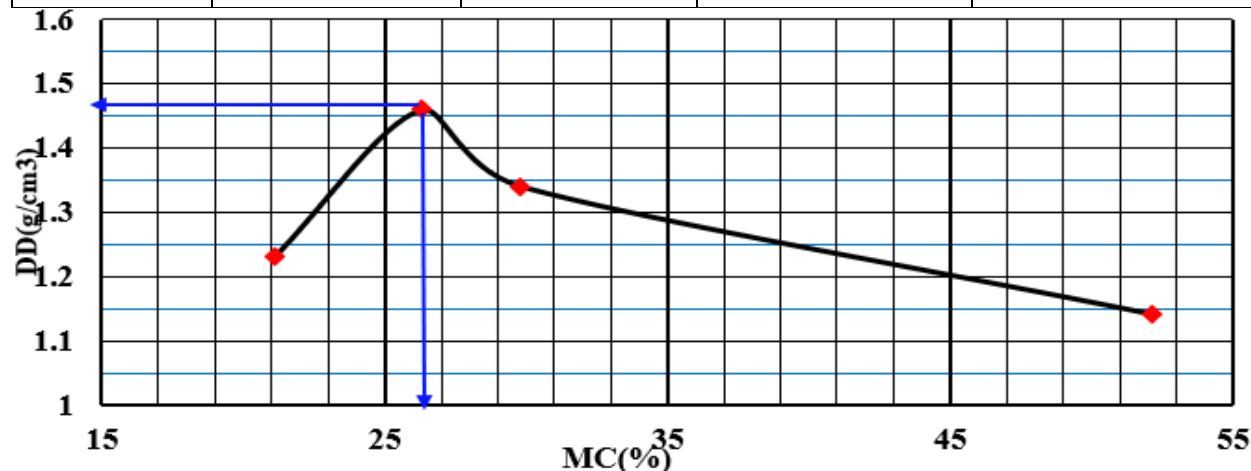


Figure D 8:Compaction curve of stabilized ES for 20%WCP(MDD=1.46g/cm³OMC=26.3%)

4.Unconfined Compressive Strength for different content of WCP

Table D 10:UCS Test result of stabilized ES for (5% WCP,95%ES)

Diameter of sample, mm = 38 Length of sample, mm = 76			Cross- Sectional Area, m ² =0.0010 Ring Calibration Factor, KN/div = 0.00153		
Axial Deformation [mm]	Axial strain [%]	Proving Ring reading [div]	Axial Load in [KN]	Corrected Area[m ²]	Axial stress in [Kpa]
0.00	0.00	0.0	0.0000	0.00100	0.0
0.20	0.26	4	0.0061	0.00100	6.1
0.40	0.53	5	0.0077	0.00101	7.6
0.60	0.79	6	0.0092	0.00101	9.1
0.80	1.05	8	0.0122	0.00101	12.1
1.00	1.32	10	0.0153	0.00101	15.1
1.20	1.58	12	0.0184	0.00102	18.1
1.40	1.84	14	0.0214	0.00102	21.0
1.60	2.11	16	0.0245	0.00102	24.0
1.80	2.37	20	0.0306	0.00102	29.9
2.00	2.63	22	0.0337	0.00103	32.8
2.20	2.89	31	0.0474	0.00103	46.1
2.40	3.16	33	0.0505	0.00103	48.9
2.60	3.42	35	0.0536	0.00104	51.7
2.80	3.68	37	0.0566	0.00104	54.5
3.00	3.95	38	0.0581	0.00104	55.8
3.20	4.21	40	0.0612	0.00104	58.6
3.40	4.47	41	0.0627	0.00105	59.9
3.60	4.74	42	0.0643	0.00105	61.2
3.80	5.00	43	0.0658	0.00105	62.5
4.00	5.26	44	0.0673	0.00106	63.8
4.20	5.53	45	0.0689	0.00106	65.0
4.40	5.79	46	0.0704	0.00106	66.3
4.60	6.05	47	0.0719	0.00106	67.6
4.80	6.32	46	0.0704	0.00107	65.9
5.00	6.58	44	0.0673	0.00107	62.9
5.20	6.84	43	0.0658	0.00107	61.3

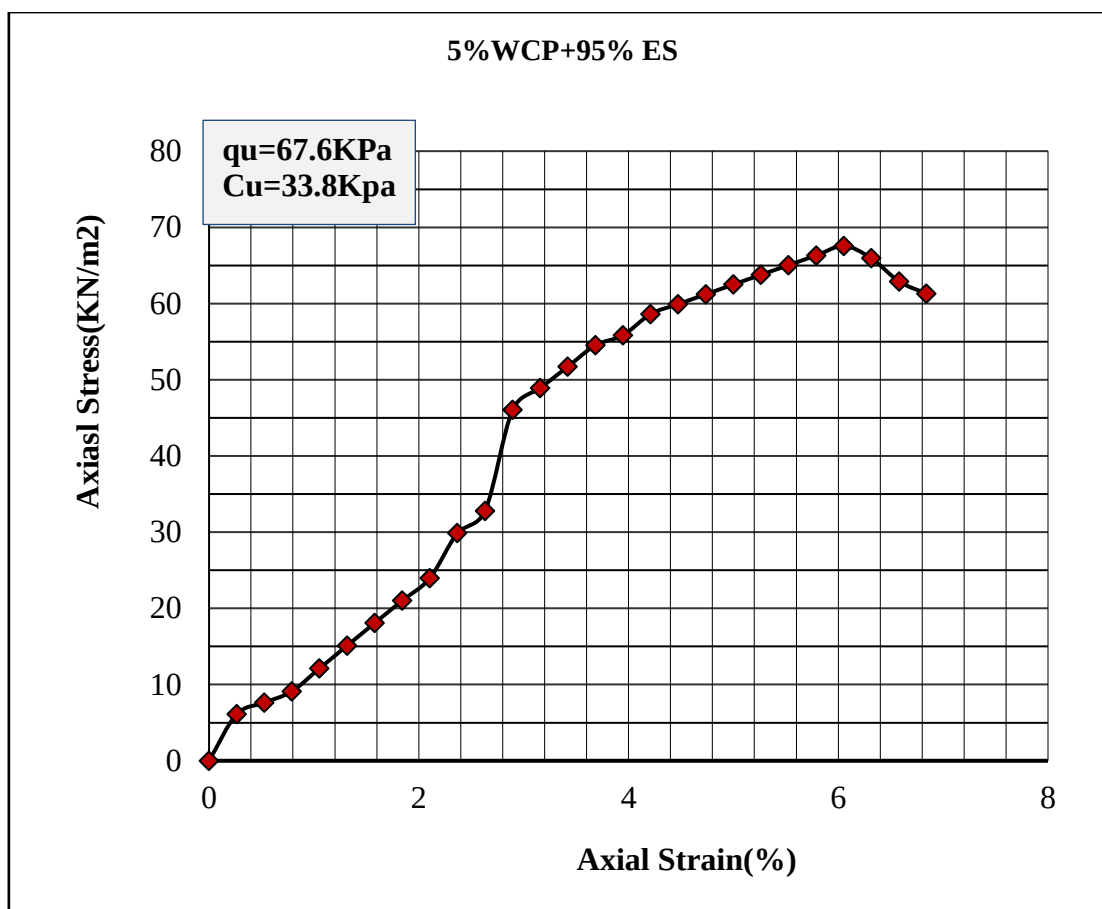


Figure D 9:Axial strain Vs Axial stress for (5%WCP,95%ES)

Table D 11:UCS Test result of stabilized ES for (10%WCP,90%ES)

Diameter of sample, mm = 38		Cross- Sectional Area, m ²		=0.0010	
Length of sample, mm = 76		Ring Calibration Factor, kN/div =		0.00153	
Axial Deformation[m m]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [Kpa]
0.00	0.00	0.0	0.0000	0.00100	0.0
0.20	0.26	4	0.0061	0.00100	6.1
0.40	0.53	11	0.0168	0.00101	16.7
0.60	0.79	15	0.0230	0.00101	22.8
0.80	1.05	19	0.0291	0.00101	28.8
1.00	1.32	22	0.0337	0.00101	33.2
1.20	1.58	25	0.0383	0.00102	37.6
1.40	1.84	31	0.0474	0.00102	46.6

1.60	2.11	33	0.0505	0.00102	49.4
1.80	2.37	35	0.0536	0.00102	52.3
2.00	2.63	39	0.0597	0.00103	58.1
2.20	2.89	40	0.0612	0.00103	59.4
2.40	3.16	41	0.0627	0.00103	60.7
2.60	3.42	43	0.0658	0.00104	63.5
2.80	3.68	44	0.0673	0.00104	64.8
3.00	3.95	45	0.0689	0.00104	66.1
3.20	4.21	46	0.0704	0.00104	67.4
3.40	4.47	47	0.0719	0.00105	68.7
3.60	4.74	48	0.0734	0.00105	70.0
3.80	5.00	49	0.0750	0.00105	71.2
4.00	5.26	48	0.0734	0.00106	69.6
4.20	5.53	47	0.0719	0.00106	67.9
4.40	5.79	46	0.0704	0.00106	64.9
4.60	6.05	45	0.0689	0.00106	61.8

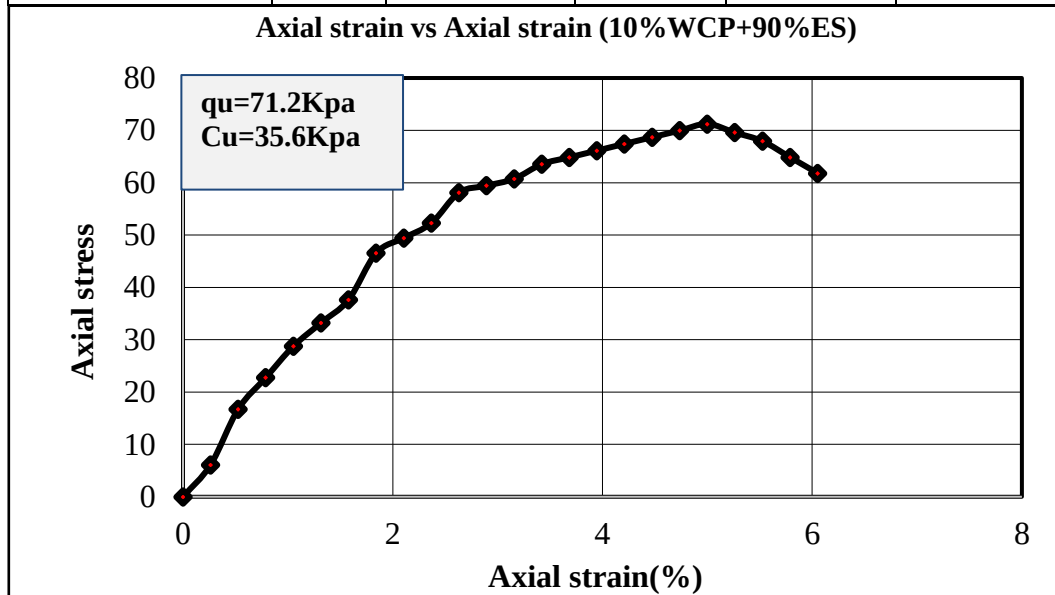


Figure D 10:Axial strain Vs Axial stress for (10%WCP ,90%ES)

Table D 12:UCS Test result for stabilized ES of (15%WCP, 85%ES)

Diameter of sample, mm = 38		Cross- Sectional Area, m ² =0.0010			
Length of sample, mm = 76		Ring Calibration Factor, KN/div = 0.00153			
Axial Deformation [mm]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [Kpa]
0.00	0.000	0.0	0.0000	0.00100	0.0
0.20	0.263	23	0.0352	0.00100	35.1
0.40	0.526	35	0.0536	0.00101	53.3
0.60	0.789	41	0.0627	0.00101	62.2
0.80	1.053	45	0.0689	0.00101	68.1
1.00	1.316	48	0.0734	0.00101	72.5
1.20	1.579	50	0.0765	0.00102	75.3
1.40	1.842	51	0.0780	0.00102	76.6
1.60	2.105	50	0.0765	0.00102	74.9
1.80	2.368	49	0.0750	0.00102	73.2
2.00	2.632	48	0.0734	0.00103	71.5

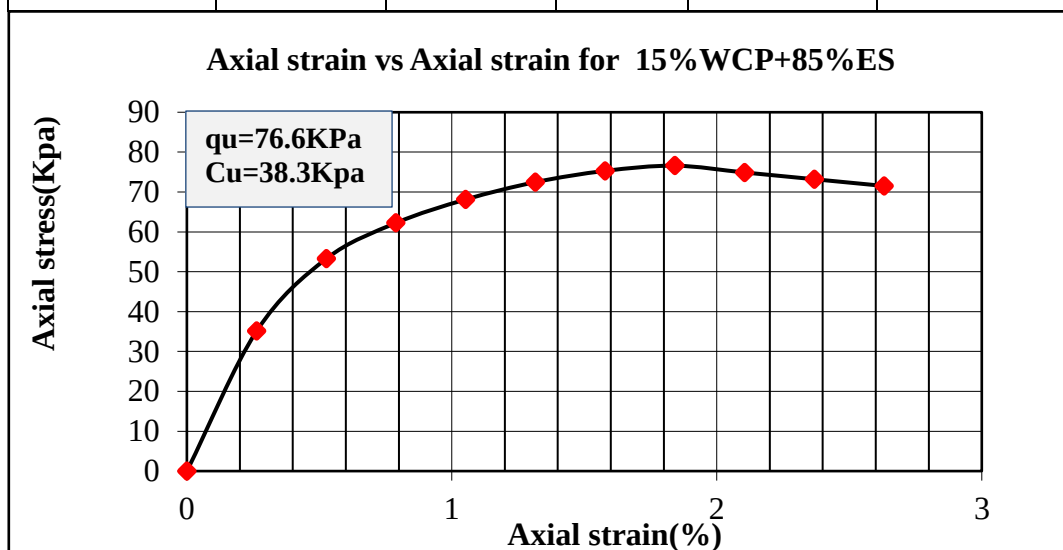


Figure D 11:Axial strain Vs Axial stress for (15%WCP,85%ES)

Table D 13:UCS Test result of stabilized ES for (20%WCP, 80%ES)

20%WCP+80%ES					
Deformation dial in [mm] ΔH	Unit strain $\epsilon = \Delta H/\Delta H_0$	Proving Ring reading	Load in [KN]	Corrected Area[m ²]	stress in [KN/m ²]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	5	0.0077	0.00100	7.6
0.40	0.0053	8	0.0122	0.00101	12.2
0.60	0.0079	10	0.0153	0.00101	15.2
0.80	0.0105	14	0.0214	0.00101	21.2
1.00	0.0132	19	0.0291	0.00101	28.7
1.20	0.0158	22	0.0337	0.00102	33.1
1.40	0.0184	32	0.0490	0.00102	48.1
1.60	0.0211	36	0.0551	0.00102	53.9
1.80	0.0237	41	0.0627	0.00102	61.2
2.00	0.0263	46	0.0704	0.00103	68.5
2.20	0.0289	49	0.0750	0.00103	72.8
2.40	0.0316	50	0.0765	0.00103	74.1
2.60	0.0342	51	0.0780	0.00104	75.4
2.80	0.0368	52	0.0796	0.00104	76.6
3.00	0.0395	53	0.0811	0.00104	77.9
3.20	0.0421	54	0.0826	0.00104	79.1
3.40	0.0447	55	0.0842	0.00105	80.4
3.60	0.0474	55	0.0842	0.00105	80.2
3.80	0.0500	54	0.0826	0.00105	78.5
4.00	0.0526	53	0.0811	0.00106	76.8

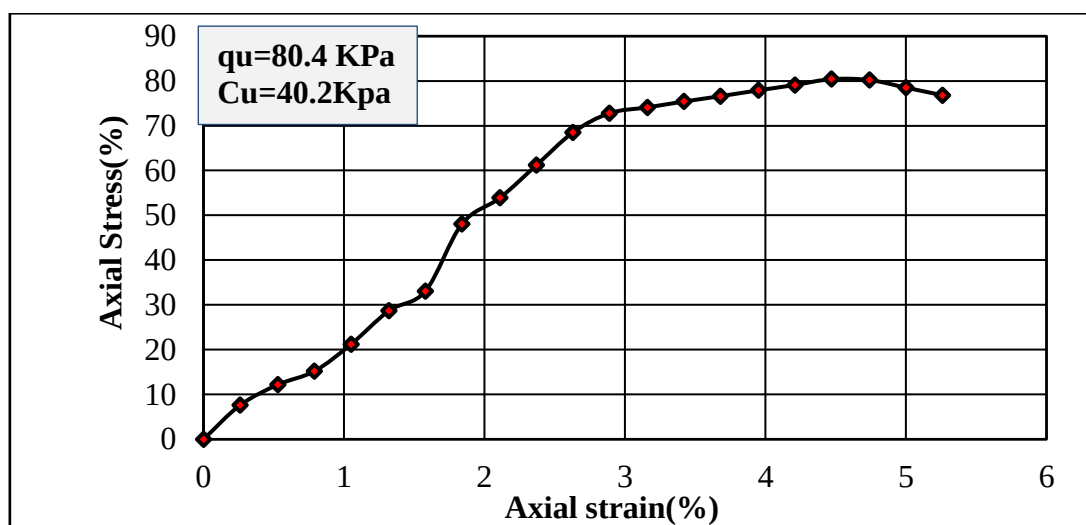


Figure D 12:Axial strain Vs Axial stress for (20%WCP,80%ES)

CBR Tests Results for different content of WCP

Table D 14:CBR Test result of stabilized ES for (5%WCP,95%ES)

5%WCP+95%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet(mm)	Ring (Div)	Load(N)	Stress (N/mm ²)	Standard stress(N/mm ²)	CBR (%)
0	0	0	0.00		
0.64	2.5	106.1	0.05		
1.27	3	127.3	0.07		
1.91	3.8	161.2	0.08		
2.54	4.5	190.9	0.10	6.9	1.4
3.18	4.6	195.2	0.10		
3.81	4.7	195.2	0.10		
4.45	5	212.2	0.11		
5.08	5.5	233.4	0.12	10.3	1.2
7.62	5.5	233.4	0.12		
Swell Determination					
No of blows	Gauge rdg. mm			Swell in %	
Reading	Initial		13	9.5	
	Final		24		
Swell	9.5				
CBR	1.4				

5%WCP+95%ES

Figure D 13:Stress Vs penetration curve of soaked CBR for (5%WCP,95%ES)

Table D 15:CBR Test result of stabilized ES for (10%WCP,90%ES)

10% WCP+90%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet(mm)	Ring (Div)	Load(N)	Stress (N/mm ²)	Standard stress(N/mm ²)	CBR (%)
0	0	0.00	0.00		
0.64	3	127.29	0.07		
1.27	4.5	190.94	0.10		
1.91	5.5	233.37	0.12		
2.54	6.5	275.80	0.14	6.9	2.1
3.18	6.8	288.52	0.15		
3.81	7	297	0.15		
4.45	7	297	0.15		
5.08	7.5	318.2	0.16	10.3	1.6
7.62	8	339.4	0.18		
Swell Determination					
No of blows		Gauge rdg. mm		Swell in %	
56	Initial		15	8.75	
	Final		25.19		
Swell		8.75			
CBR		2.1			

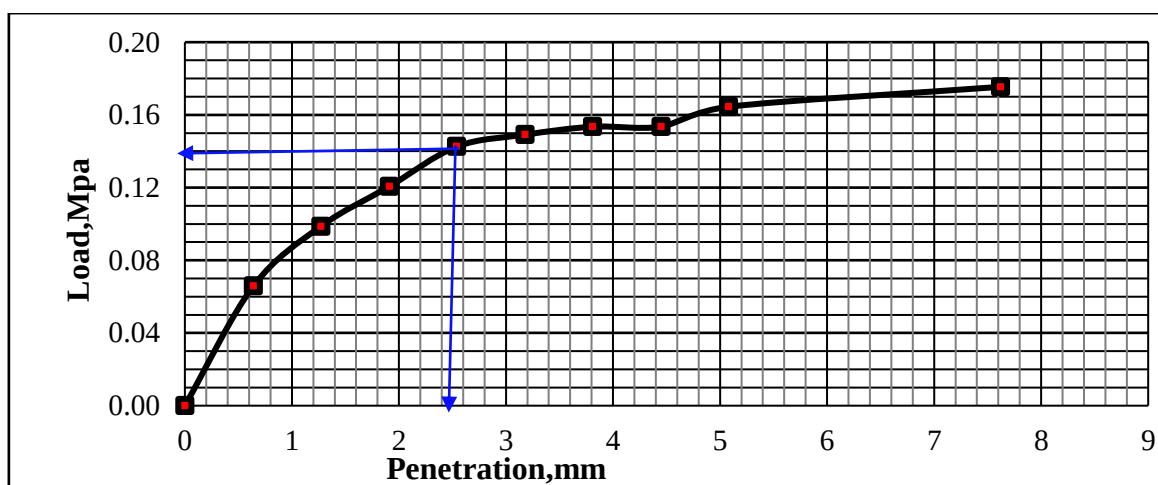


Figure D 14:Stress Vs penetration curve of soaked CBR for (10%WCP,90%ES)

Table D 16:CBR Test result of stabilized ES for (15%WCP,85%ES)

15%WCP+85%ES					
Ring calibration fact.			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking					
Penet(mm)	Ring (Div)	Load(N)	Stress(N/mm)	Standard stress(N/mm ²)	CBR (%)
0	0	0	0.00		
0.64	5	212.15	0.11		
1.27	7	297.01	0.15		
1.91	8.5	360.655	0.19		
2.54	9	381.87	0.20	6.9	2.9
3.18	10	424.3	0.22		
3.81	11	466.73	0.24		
4.45	12	509.2	0.26		
5.08	13	551.59	0.29	10.3	2.8
7.62	13.5	572.8	0.3		
Swell Determination					
No of blows		Gauge rdg. mm		Swell in %	
56		Initial	16	5.86	
		Final	22.82		
Swell		5.86			
CBR		2.9			

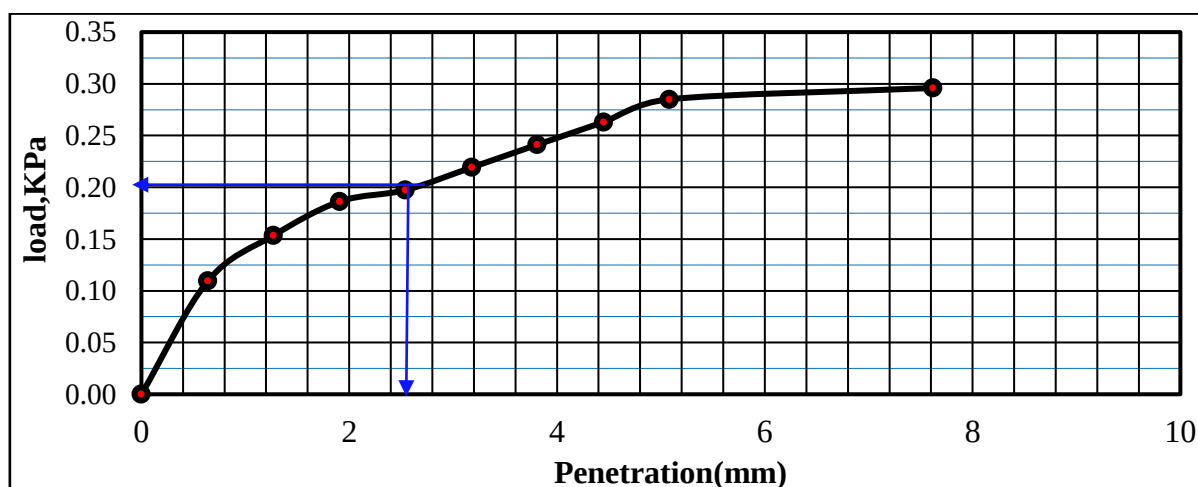


Figure D 15:Stress Vs penetration curve of soaked CBR for (15%WCP,85%ES)

Table D 17:CBR Test result of stabilized ES for (20%WCP,80%ES)

20% WCP+80%ES					
Ring calibration factor			42.43N/div		
Plunger Area			1935mm ²		
Rammer wt.			4.5kg		
Blows/layer			56/5		
CBR Penetration determination/soaking after 4 days					
Penet. (mm)	Ring (Div)	Load(N)	Stress (N/mm ²)	Standard stress(N/mm ²)	CBR (%)
0	0	0.0	0.000		
0.64	7	297.0	0.153		
1.27	9	381.9	0.197		
1.91	13	551.6	0.285		
2.54	13.5	572.8	0.296	6.9	4.3
3.18	16	678.9	0.351		
3.81	19	806.2	0.417		
4.45	19.5	827.4	0.428		
5.08	19.5	827.4	0.428	10.3	4.2
7.62	19.5	827.4	0.428		
Swell Determination					
Reading	Gauge rdg. mm			Swell in %	
	Initial		18	4.29	
	Final		23		
Swell (%)	4.29				
CBR (%)	4.3				

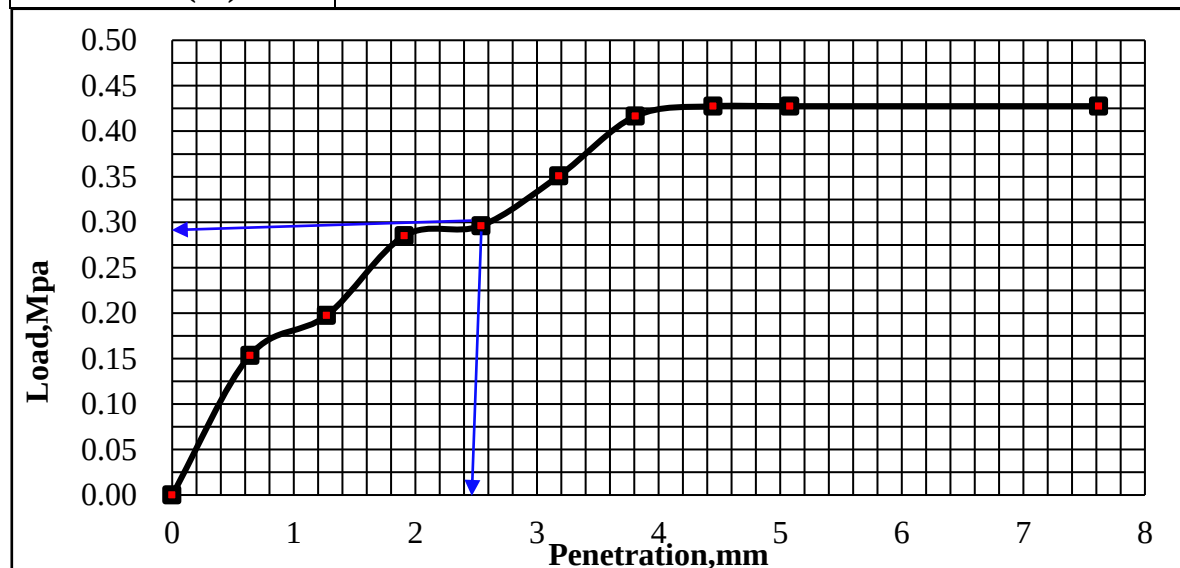


Figure D 16:Stress Vs penetration curve of soaked CBR for (20%WCP,80%ES)

Appendix E: Laboratory test result of combined effect of MCA and WCP on ES

1.Compaction Laboratory test result for Combination of both Materials

Table E 1:Compaction test result of stabilized ES for (15%MCA ,5%WCP)

15%MCA+5%WCP												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5825			5977			6180			6063		
Wt. of mold(g)	4390			4390			4390			4390		
Wet soil (g)	1435			1587			1790			1673		
Wet density, (g/cm ³)	1.52			1.68			1.896			1.77		
Moisture content determination												
Trial No.	1			2			3			4		
Can wt.(g)	19	18	18	29	28	29	30	30	29	19	18	18
Wet soil+can (g)	61	88	57	117	12	10	134	12	127	10	10	99
Dry soil+can(g)	53	78	51	97	10	87	104	94	100	77	76	72
Wt. of moisture(g),	8	10	6	20	20	15	30	30	27	32	30	27
Dry soil(g)	34	60	33	68	76	58	74	64	71	58	58	54
Moisture content (%)	23.5	16.6	18.2	29.4	26.3	25.8	40.5	46.8	38	55.7	51.7	50
Avg. moisture W (%)	19.43			27.16			41.7			52.2		
Dry density, g/cm ³	1.277			1.32			1.34			1.163		

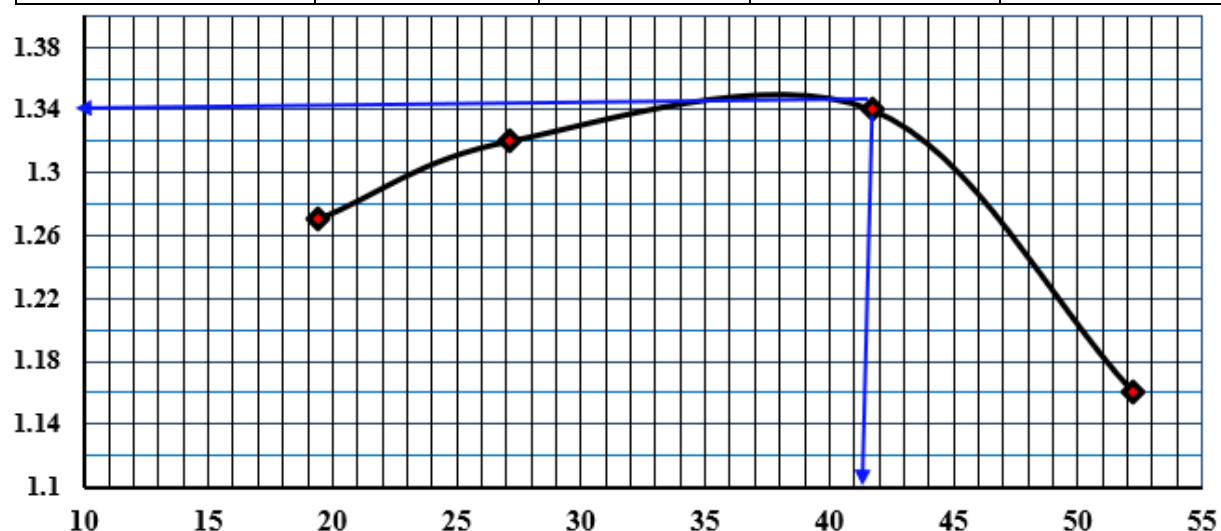


Figure E 1:Compaction Curve of stabilized ES for 15%MCA and 5%WCP

Table E 2:Compaction test result of stabilized ES for (15%MCA ,10%WCP)

15%MCA+10%WCP												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5860			6050			6116			6021		
Wt. of mold(g)	4390			4390			4390			4390		
Wet soil (g)	1470			1660			1726			1631		
Wet density, (g/cm ³)	1.56			1.76			1.83			1.73		
Moisture content determination												
Trial No.	1			2			3			4		
Can wt. (g)	18	18	18	18	19	19	19	18	18	18	19	18
Wet soil+can(g)	64	82	66	75	89	69	98	81	93	113	102	105
Dry soil+can(g)	60	74	59	63	73	59	83	68	72	82	74	77
Wt. of moisture(g),	4	8	7	12	16	10	15	13	21	31	28	28
Dry soil(g)	42	56	41	45	54	40	44	50	54	64	55	59
Moisture content (%)	9.52	14.3	19.6	26.6	29.6	25	34	26	38.8	48.5	50.9	47.5
Avg.moisture content (%)	13.64			27			32.9			48.96		
Dry density, g/cm ³	1.37			1.39			1.38			1.22		

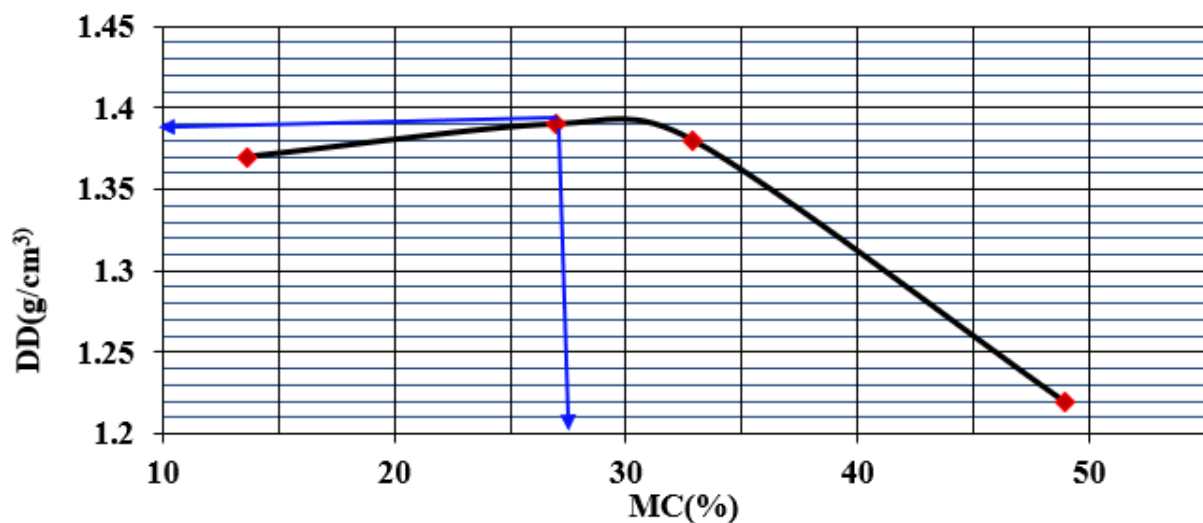


Figure E 2:Compaction Curve of stabilized ES for 15%MCA and 10%WCP

Table E 3:Compaction test result of stabilized ES for (15%MCA,15%WCP)

15%MCA+15%WCP												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5750			5940			6132			5971		
Wt. of mold(g)	4390			4390			4390			4390		
Wet soil (g)	1360			1550			1742			1581		
Wet density, (g/cm ³)	1.44			1.642			1.845			1.67		
Moisture content determination												
Trial No.	1			2			3			4		
Can wt.(g)	20	19	19	29	29	30	19	19	18	18	18	18
Wet soil+can(g)	93	84	86	121	106	130	77	96	98	125	122	135
Dry soil+can(g)	82	74	75	100	88	109	62	74	77	89	87	95
Wt. of moisture(g),	11	10	11	21	18	21	15	22	21	36	35	40
Dry soil(g)	62	55	56	71	59	79	43	55	59	71	69	77
Moisture content (%)	17.7	18.2	19.6	29.6	30.5	26.6	34.8	40	35.6	50	50.7	51.9
Avg. moisture content (%)	18.5			28.9			36.8			50.87		
Dry density, g/cm ³	1.22			1.28			1.35			1.11		

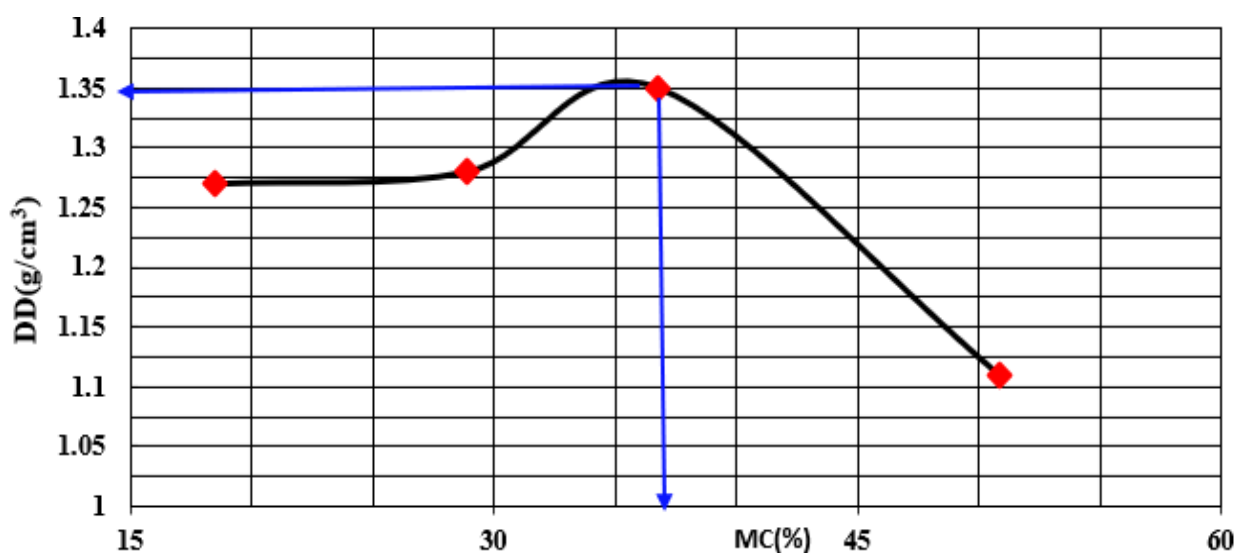


Figure E 3:Compaction Curve of stabilized ES for 15%MCA and 15%WCP

Table E 4:Compaction test result of stabilized ES for (15%MCA,20%WCP)

15%MCA+20%WCP												
Trial No.	1			2			3			4		
Wt. of Mold +wet soil (g)	5780			5960			6059			6128		
Wt. of mold(g)	4390			4390			4390			4390		
Wet soil (g)	1390			1570			1669			1738		
Wet density, (g/cm ³)	1.47			1.66			1.77			1.84		
Moisture content determination												
Trial No.	1			2			3			4		
Can wt. (g)	19	20	20	18	20	20	19	18	18	20	19	17
Wet soil+can(g)	69	68	51	69	57	62	78	79	85	110	94	107
Dry soil+can(g)	62	62	51	59	50	53	64	63	69	81	70	77
Wt. moisture(g),	7	6	5	10	7	9	14	16	16	29	16	30
Dry soil(g)	43	42	31	41	30	33	45	45	51	61	59	60
Moisture content (%)	16.3	14.3	16.1	24.4	23.3	27.3	31.1	35.5	31.4	47.5	27.1	50
Avg. moisture content (%)	16.67			25			32.67			41.53		
Dry density, g/cm ³	1.27			1.3			1.34			1.3		

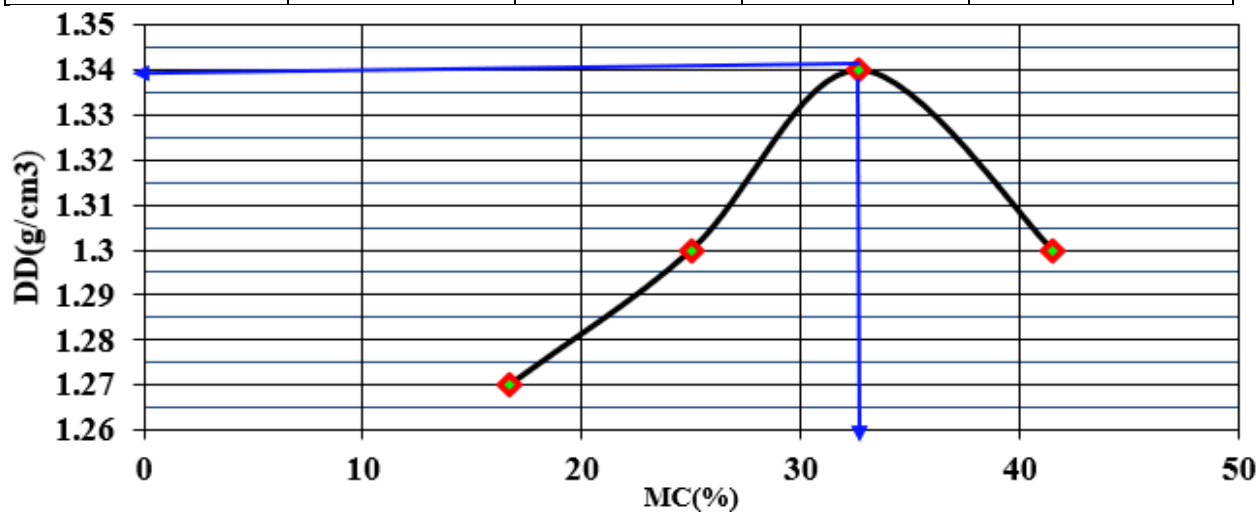


Figure E 4:Compaction Curve of stabilized ES for 15%MCA and 20%WCP

2. Unconfined Compressive strength for combination of MCA and WCP on the soil

Table E 5:UCS test result of stabilized ES for (15%MCA,5%WCP)

Diameter of sample, mm = 38 Length of sample, mm = 76			Cross- Sectional Area, m ² =0.0010 Ring Calibration Factor, kN/div = 0.00153		
Axial Deformation[m]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [Kpa]
0.00	0.0000	0.0000	0.0	0.0000	0.0
0.20	0.0026	0.2632	7	0.0107	10.7
0.40	0.0053	0.5263	10	0.0153	15.2
0.60	0.0079	0.7895	12	0.0184	18.2
0.80	0.0105	1.0526	15	0.0230	22.7
1.00	0.0132	1.3158	19	0.0291	28.7
1.20	0.0158	1.5789	23	0.0352	34.6
1.40	0.0184	1.8421	25	0.0383	37.5
1.60	0.0211	2.1053	28	0.0428	41.9
1.80	0.0237	2.3684	30	0.0459	44.8
2.00	0.0263	2.6316	30	0.0459	44.7
2.20	0.0289	2.8947	32	0.0490	47.5
2.40	0.0316	3.1579	32	0.0490	47.4
2.60	0.0342	3.4211	34	0.0520	50.2
2.80	0.0368	3.6842	35	0.0536	51.6
3.00	0.0395	3.9474	37	0.0566	54.4
3.20	0.0421	4.2105	41	0.0627	60.1
3.40	0.0447	4.4737	42	0.0643	61.4
3.60	0.0474	4.7368	43	0.0658	62.7
3.80	0.0500	5.0000	43	0.0658	62.5
4.00	0.0526	5.2632	42	0.0643	60.9
4.20	0.0553	5.5263	41	0.0627	59.3

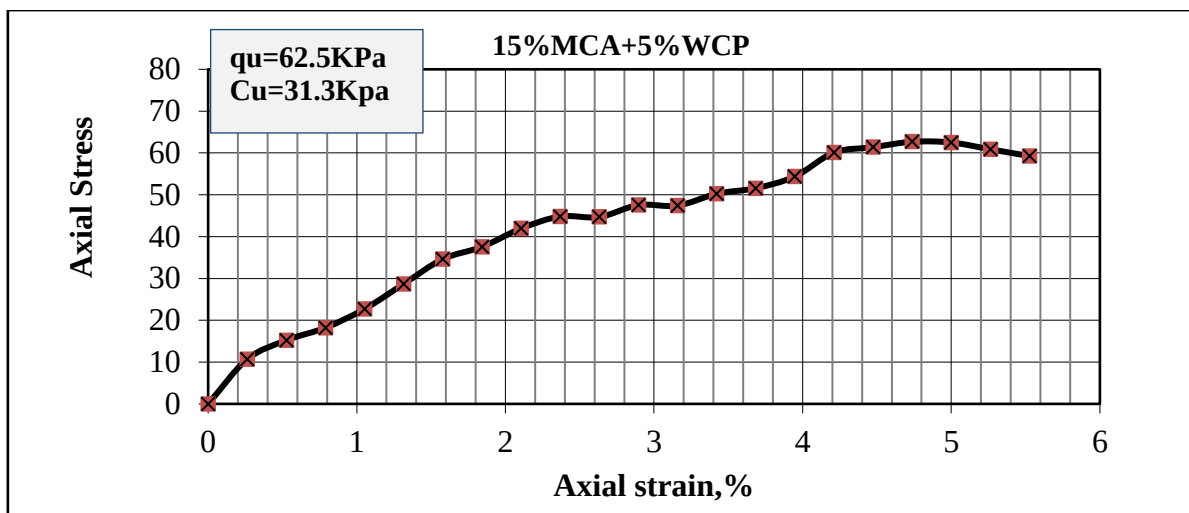


Figure E 5: Axial strain Vs Axial stress for (15%MCA, 5%WCP)

Table E 6: UCS test result of stabilized ES for (15%MCA, 10%WCP)

15%MCA+10%WCP					
Axial Deformation[mm]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [Kpa]
0.00	0.0000	0.0	0.0000	0.00100	0.0
0.20	0.0026	3	0.0046	0.00100	4.6
0.40	0.0053	5	0.0077	0.00101	7.6
0.60	0.0079	7	0.0107	0.00101	10.6
0.80	0.0105	10	0.0153	0.00101	15.1
1.00	0.0132	12	0.0184	0.00101	18.1
1.20	0.0158	14	0.0214	0.00102	21.1
1.40	0.0184	16	0.0245	0.00102	24.0
1.60	0.0211	19	0.0291	0.00102	28.5
1.80	0.0237	21	0.0321	0.00102	31.4
2.00	0.0263	24	0.0367	0.00103	35.8
2.20	0.0289	26	0.0398	0.00103	38.6

2.40	0.0316	28	0.0428	0.00103	41.5
2.60	0.0342	30	0.0459	0.00104	44.3
2.80	0.0368	32	0.0490	0.00104	47.2
3.00	0.0395	34	0.0520	0.00104	50.0
3.20	0.0421	36	0.0551	0.00104	52.8
3.40	0.0447	38	0.0581	0.00105	55.5
3.60	0.0474	39	0.0597	0.00105	56.8
3.80	0.0500	40	0.0612	0.00105	58.1
4.00	0.0526	41	0.0627	0.00106	59.4
4.20	0.0553	43	0.0658	0.00106	62.2
4.40	0.0579	45	0.0689	0.00106	64.9
4.60	0.0605	44	0.0673	0.00106	63.2
4.80	0.0632	43	0.0658	0.00107	61.6

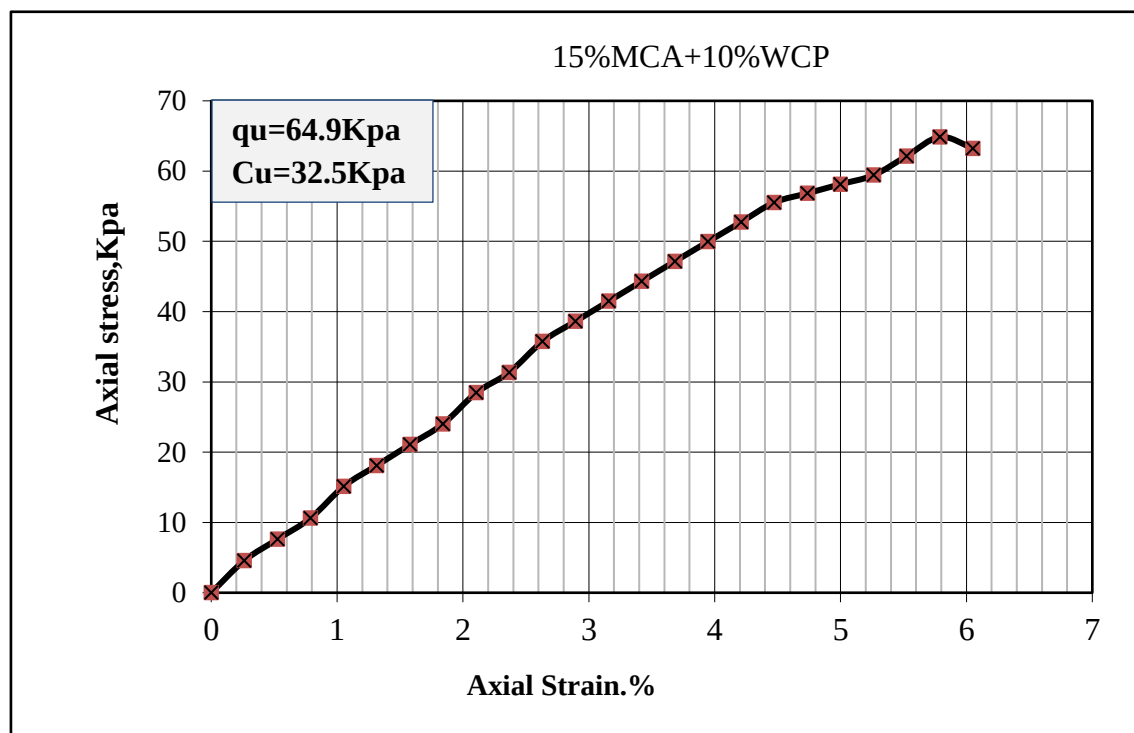


Figure E 6: Axial strain Vs Axial stress for (15%MCA ,10%WCP)

Table E 7:UCS test result of stabilized ES for (15%MCA,15%WCP)

Diameter of sample, mm = 38			Cross- Sectional Area, m ² =0.0010		
Length of sample, mm = 76			Ring Calibration Factor, KN/div = 0.00153		
Axial Deformation[mm]	Axial strain [%]	Proving ring reading[div]	Axial load [KN]	Corrected Area[m2]	Axial Stress [Kpa]
0.00	0	0	0.0	0.0000	0.0
0.20	0.0026	0.26	5	0.0077	7.6
0.40	0.0053	0.53	7	0.0107	10.7
0.60	0.0079	0.79	11	0.0168	16.7
0.80	0.0105	1.05	14	0.0214	21.2
1.00	0.0132	1.32	17	0.0260	25.7
1.20	0.0158	1.58	20	0.0306	30.1
1.40	0.0184	1.84	22	0.0337	33.0
1.60	0.0211	2.11	24	0.0367	35.9
1.80	0.0237	2.37	26	0.0398	38.8
2.00	0.0263	2.63	27	0.0413	40.2
2.20	0.0289	2.89	28	0.0428	41.6
2.40	0.0316	3.16	29	0.0444	43.0
2.60	0.0342	3.42	29	0.0444	42.9
2.80	0.0368	3.68	32	0.0490	47.2
3.00	0.0395	3.95	33	0.0505	48.5
3.20	0.0421	4.21	36	0.0551	52.8
3.40	0.0447	4.47	39	0.0597	57.0
3.60	0.0474	4.74	42	0.0643	61.2
3.80	0.0500	5.00	42	0.0643	61.0
4.00	0.0526	5.26	43	0.0658	62.3
4.20	0.0553	5.53	43	0.0658	62.2
4.40	0.0579	5.79	42	0.0643	60.5
4.60	0.0605	6.05	42	0.0643	60.4
4.80	0.0632	6.32	42	0.0643	60.2

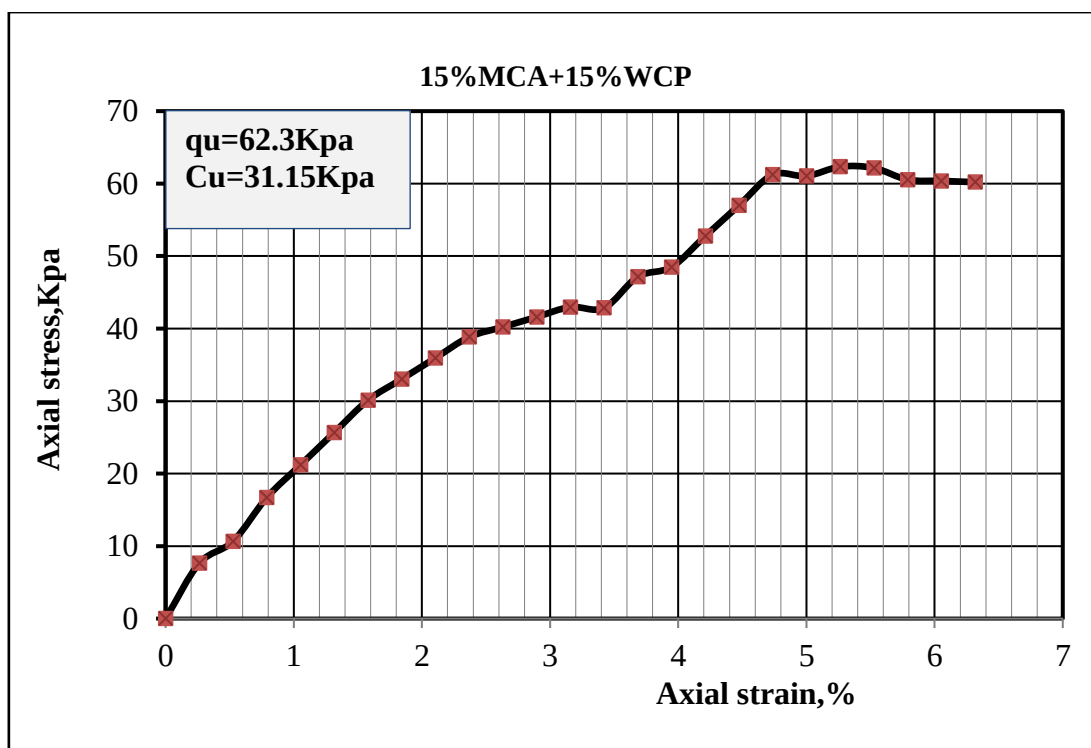


Figure E 7: Axial strain Vs Axial stress for (15%MCA, 15%WCP)

Table E 8: UCS test result of stabilized ES for (15%MCA, 20%WCP)

Axial Deformation [mm]	Axial strain [%]	Proving ring reading [div]	Axial load [KN]	Corrected Area[m ²]	Axial Stress [KN/m ²]
0.00	0.000	0.0	0.0	0.0000	0.0
0.20	0.003	0.263	7	0.0107	10.7
0.40	0.005	0.526	12	0.0184	18.3
0.60	0.008	0.789	18	0.0275	27.3
0.80	0.011	1.053	25	0.0383	37.8
1.00	0.013	1.316	33	0.0505	49.8
1.20	0.016	1.579	40	0.0612	60.2
1.40	0.018	1.842	43	0.0658	64.6
1.60	0.021	2.105	43	0.0658	64.4
1.80	0.024	2.368	44	0.0673	65.7
2.00	0.026	2.632	44	0.0673	65.5
2.20	0.029	2.895	45	0.0689	66.9
2.40	0.032	3.158	44	0.0673	65.2
2.60	0.034	3.421	43	0.0658	63.5
2.80	0.037	3.684	42	0.0643	61.9
3.00	0.039	3.947	42	0.0643	61.7
3.20	0.042	4.211	41	0.0627	60.1

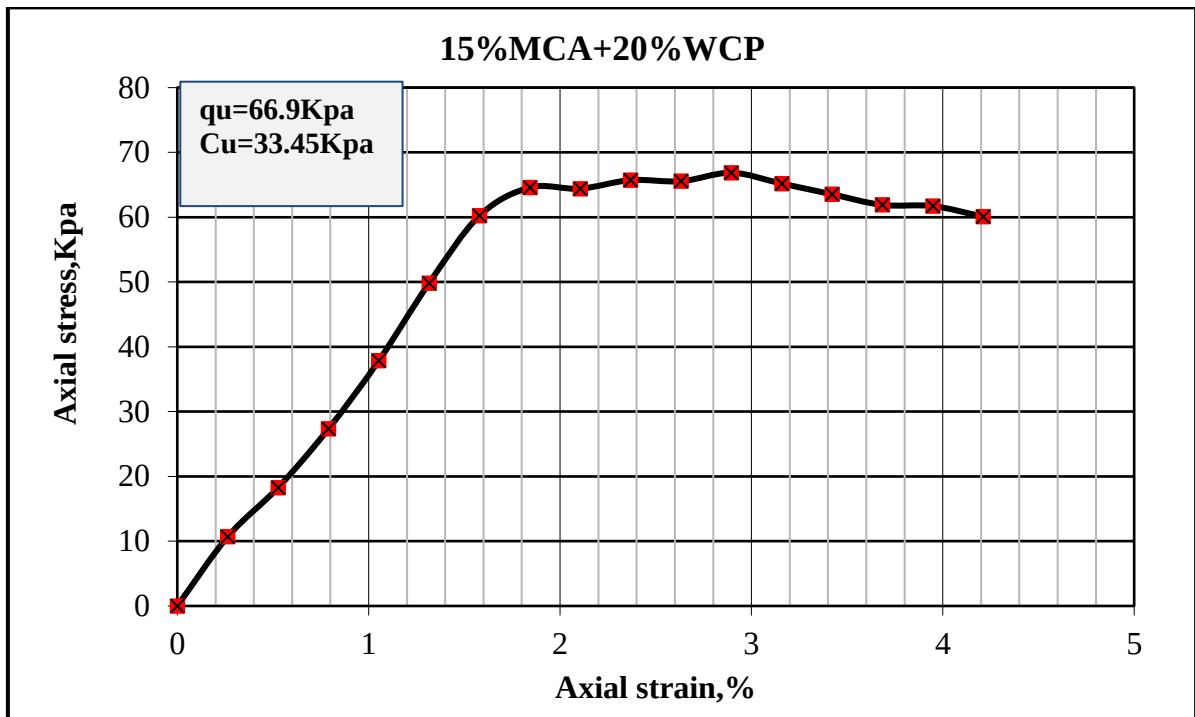


Figure E 8:Axial strain Vs Axial stress for (15%MCA ,20%WCP)

Appendix F : Consolidation property of the soil in the study area

Data before the consolidation test for test pit-1

Sample type	Remolded	Volume of mold(V).....	944cm ³
Density (γ)	1.17g/cm ³	Mass of the Ring (MR).....	111.5g
Natural moisture content (w).....	35.2%	Mass of the ring +soil (MS+R)	244.7g
Specific gravity of the soil.....	2.71		
Intial moisture content (Mo).....	44%		

Table F-1 Data before of the consolidation for test pit-1

Sample Thickness (H_o) (mm)	20.00
Sample diameter(D)(mm)	75
Area (A) (cm ²)	44.18
Volume (V) (cm ³)	88.37
Bulk Density (r_b) (g/cm ³)	1.507
Dry Density (r_d) (g/cm ³)	1.047
Intial void ratio: $e_o = (G_s / \gamma_d) - 1$	1.589
Height of solid: $H_s = H_o / (1 + e_o)$	7.725
Intial Saturation (S_o) = $(M_o * G_s) / e_o$	75.04

Data at the end of the consolidation test:

Mass of ring + wet soil +can (MS+R+C).....283.90g

Mass of ring + oven dried soil+ can (MS+R+C).....253.10g

Mass of ring + can (MS+C).....160.6g

Moisture content at the end of testing (Mf).....33.3 %

Mass of soil (Ms) = 123.3g

Table F-2 Data at the end of the consolidation test:

Overall settlement (mm)	5.232
Volume change (cm ³)	23.12
Final Volume (cm ³)	65.25
Final Bulk Density (gm/cm ³)	1.890
Final Dry Density (gm/cm ³)	1.418
Final saturation (S _f)(%)	0.912

Table F-3 Dial gauge reading for each incremental loading

Time(min)	$\sqrt{\text{time}}$	Cumulative Compression, (mm)				
		160[kpa]	[320kpa]	[640kpa]	[1280kpa]	[2560kpa]
0	0.00	1.000	2.068	2.996	4.220	5.256
0.25	0.50	1.400	2.318	3.270	4.480	5.566
0.5	0.71	1.464	2.348	3.296	4.494	5.576
1	1.00	1.502	2.380	3.320	4.512	5.592
2	1.41	1.560	2.416	3.358	4.536	5.616
4	2.00	1.624	2.456	3.396	4.570	5.648
8	2.83	1.690	2.504	3.450	4.616	5.694
15	3.87	1.744	2.552	3.512	4.674	5.748
30	5.48	1.810	2.618	3.602	4.760	5.838
60	7.75	1.880	2.698	3.724	4.828	5.954
120	10.95	1.930	2.786	3.862	4.954	6.118
240	15.49	1.976	2.866	3.914	5.028	6.288
480	21.91	1.996	2.928	4.130	5.160	6.328
1440	37.95	2.068	2.996	4.220	5.256	6.496
Net Cumulative compression		1.068	1.996	3.220	4.256	5.496

Table F-4 Cumulative dials gauge reading at the end of each consecutive unloading

Time(min.)	$\sqrt{\text{time}}$	Cumulative compression	
		640[kpa]	160[kpa]
0	0	6.496	5.860
1440	37.95	5.860	5.232
Net cumulative compression		4.860	4.232

Table F-5 Determination of void ratio, Coefficient of Compressibility and Consolidation

Voids ratio					Coefficients of Compressibility (M_v)			Coefficients of consolidation (C_v)		
No	Pressure (Kpa)	Cum.compressi on (ΔH) mm	Consolidated height $H = H_0 - \Delta H$	Void ratio $e = (H - H_s) / H_s$	Incremental		$M_v = (\Delta H / H) * (1000 / \Delta p) \text{ m}^2 / \text{MN}$	t_{90} (mm)	$H = 0.5 (H_1 + H_2)$	$(0.111H)^2 / (t_{90}) \text{ m}^2 / \text{yr}$
					Height(ΔH) (mm)	Pressure change, ΔP (Kpa)				
0	0	0.000	20.000	1.589						
					1.068	160	0.334			
1	160	1.068	18.932	1.451				16	19.46	2.63
					0.928	160	0.306			
2	320	1.996	18.004	1.331				64	18.47	0.59
					1.224	320	0.212			
3	640	3.220	16.780	1.172				100	17.39	0.34
					1.036	600	0.103			
4	1240	4.256	15.744	1.038				100	16.26	0.29
					1.240	1320	0.060			
5	2560	5.496	14.504	0.878				484	15.12	0.05
6	640	4.860	15.140	0.960						
	160	4.232	15.768	1.041						

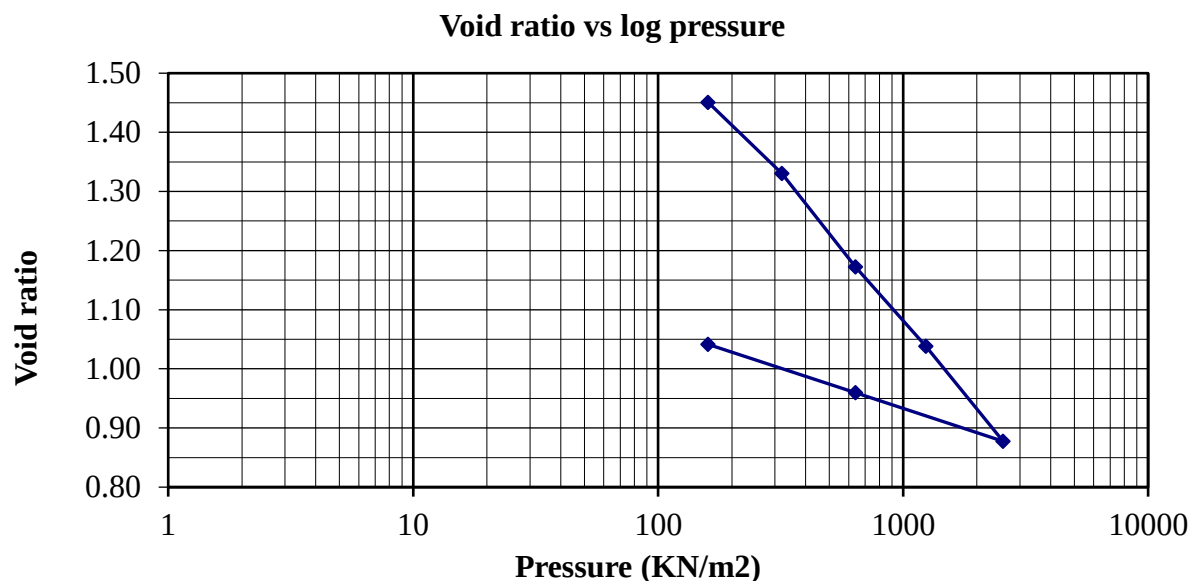


Figure F-1. Pressure vs Void ratio

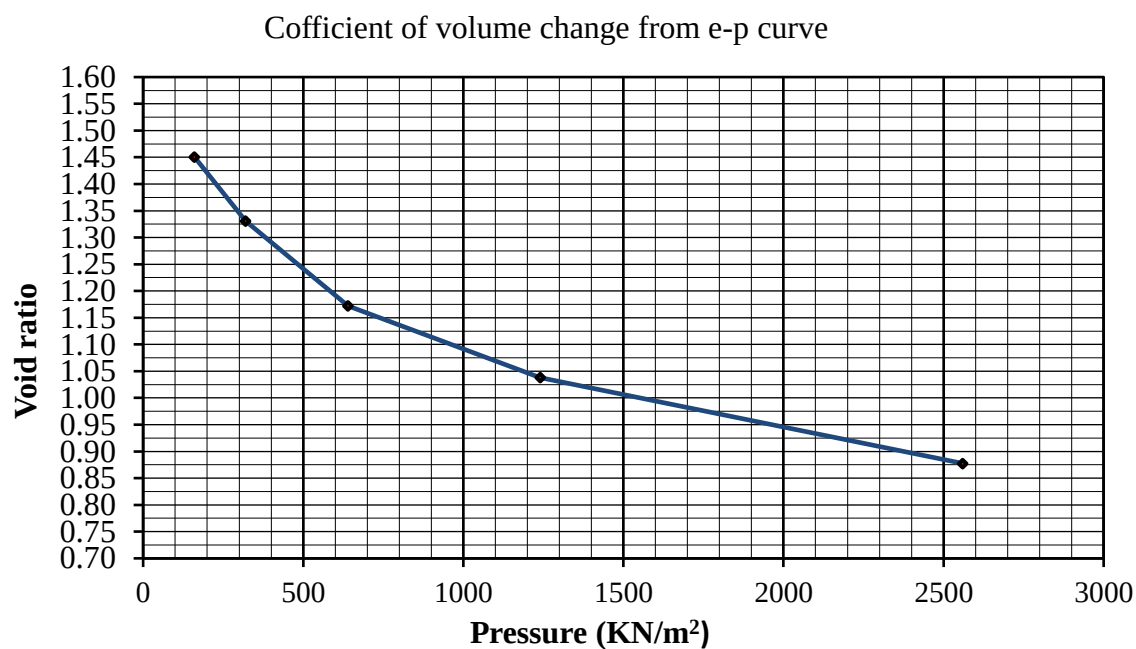


Figure F-2 Coefficient of volume change from e-p curve

Table F-6 Determination of M_v from e-p curve

$e_1 = 1.451$	$P_1(\text{KN/m}^2) = 160$
$e_2 = 1.331$	$P_2(\text{KN/m}^2) = 320$
$e_3 = 1.172$	$P_3(\text{KN/m}^2) = 640$
$e_4 = 1.038$	$P_4(\text{KN/m}^2) = 1240$
$a_{v1} = (e_1 - e_2) / (P_1 - P_2) = 0.000751$ -coefficient of volume change	
$M_{v1} = a_{v1} / (1 + e_1) = 0.000306 \text{ m}^2/\text{KN}$	
$a_{v2} = (e_3 - e_4) / (P_3 - P_4) = 0.0002235$	
$M_{v2} = a_{v2} / (1 + e_3) = 0.0001029 \text{ m}^2/\text{KN}$	
Avg. M_v =	$(M_{v1} + M_{v2}) / 2 = \mathbf{0.0002046 = 0.2046 \text{ m}^2/\text{KN}}$

Compression index from (C_c) from e-p curve

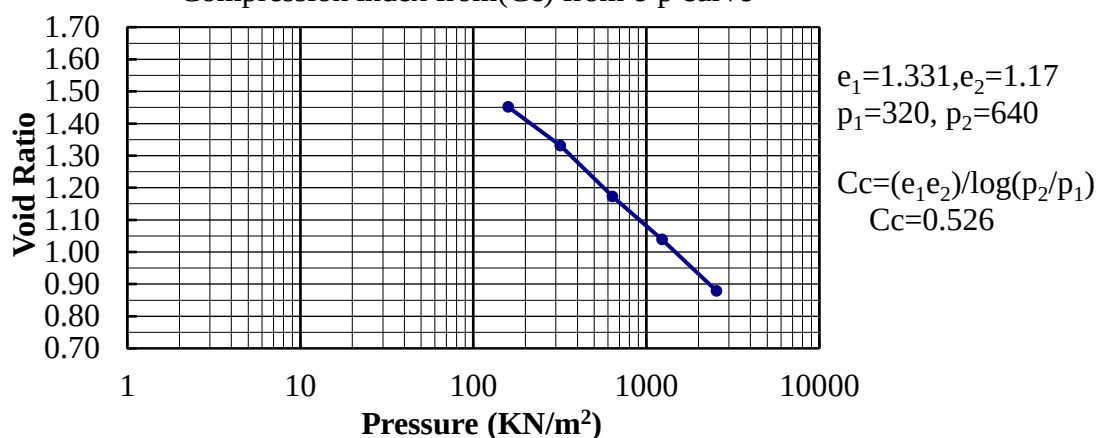


Figure F-3 Compression index from (C_c) from e-p curve

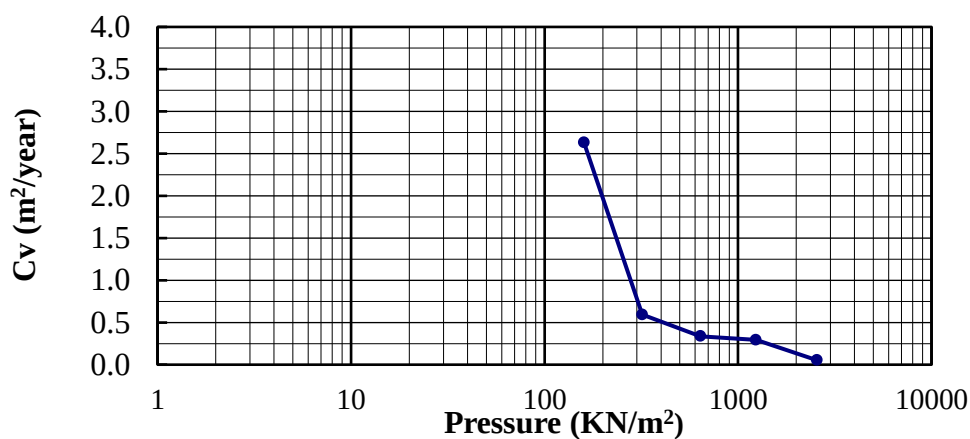


Figure F-4 Coefficient of consolidation vs log pressure

Appendix G : Photos taken during the study

1. During sample taking



2. During sieve and Hydrometer Analysis



3. During Atterberg limit tests



4. During Compaction tests



5. During Unconfined Compressive strength tests



6. During Soaking CBR tests

