Investigating the Effect of Eggshell Ash and Crushed Stone Powder on Geotechnical Properties of Expansive Soils: In Case of Dukem Town

Tadesse Endale Teferra

A Thesis Submitted to the Department of Civil Engineering,
School of Civil Engineering and Architecture

Office of Graduate Studies
Adama Science and Technology University

February, 2023
Adama, Ethiopia

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Advisor: Eleyas Assefa (Ph.D.)

A Thesis Submitted to the Department of Civil Engineering,
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Presented in Partial Fulfillment of the Requirement for the Degree of
Master's in Civil Engineering
(Specialization in Geotechnical Engineering)

Office of Graduate Studies
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DECLARATION

I hereby declare that this Master thesis entitled **"Investigating the effect of eggshell ash and crushed stone powder on geotechnical properties of expansive soils: In the case of Dukem town"** is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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RECOMMENDATION

I/we, the advisor(s) of this thesis, hereby certify that I/we have read the revised version of the thesis entitled "**Investigating the effect of eggshell ash and crushed stone powder on geotechnical properties of expansive soils: In case of Dukem town**" prepared under my/our guidance by Tadesse Endale submitted in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering. Therefore, I/we recommend the submission of revised version of the thesis to the department following the applicable procedures.

Eleyas Assefa (Ph.D.)	________________	24/02/2023
Major Advisor/Supervisor	Signature	Date
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APPROVAL PAGE

I/we, the advisors of the thesis entitled **“Investigating the effect of eggshell ash and crushed stone powder on geotechnical properties of expansive soils: In case of Dukem town**” and developed by Tadesse Endale, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

Eleyas Assefa (Ph.D.)		24/02/2023
Major Advisor/Supervisor	Signature	Date
________________	________________	_________
Co-advisor	Signature	Date

APPROVAL OF BOARD OF REVIEWERS

We, the undersigned, members of the Board of Examiners of the thesis by Tadesse Endale have read and evaluated the thesis entitled **"Investigating the effect of eggshell ash and crushed stone powder on geotechnical properties of expansive soils: In case of Dukem town**" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Civil Engineering.

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ABBREVIATIONS

ASTU	Adama Science and Technology University
AASTU	Addis Ababa Science and Technology University
AASHTO	American Association of State, Highway & Transport Official
ASTM	American Society for Testing Material
BS	British Standard
C-S-H	Calcium Silicate Hydrate Compound
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CSA	Central Statistical Agency
CSP	Crushed Stone Powder
ENMA	Ethiopian National Metrological Agency
ESA	Eggshell Ash
ESP	Eggshell Powder
FAO	World Food and Agriculture Organization
FSI	Free Swell Index
FSR	Free Swell Ratio
FTIR	Fourier Transform Infrared Spectroscopy
GI	Group Index
JCPDS	Joint Committee on Powder Diffraction Standards
LS	Linear Shrinkage
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
QD	Quarry Dust
SEM	Scanning Electron Microscope
SI	Shrinkage Index
SL	Shrinkage Limit

SOM	Soil Organic Matter
TP	Test Pit
T-O-T	Tetrahedron-Octahedron-Tetrahedron Bond
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
XRD	X-ray Diffraction
ZAV	Zero Air Void

ACRONYMS

A	Activity of clay minerals
ϕ	Angle of Internal Friction
C_V	Coefficient of Consolidation
C_C	Coefficient of Curvature
K	Coefficient of Permeability
Cu	Coefficient of Uniformity
C	Clay
Cc	Compressibility Index
G	Gravel
e_0	Initial Void Ratio
NP	Non-plastic Materials
O	Organic Soil
S	Sand
M	Silt
C_r	Swelling index
Su	Undrained Shear Strength
e	Void Ratio
V_K	Volume of Soil in Kerosene
V_W	Volume of Soil in Water

ABSTRACT

Expansive soils are those having the ability to undergo excess swelling-shrinkage behavior due to available smectite mineral in soil mass and which causes distress on overlaid structures as moisture content varies. This soil type covers more than 40% of land area in Ethiopia, and currently it becomes an alarming issue throughout the overall world. So, to overcome such problems this study was tried to apply ground improvement by using different stabilizing agents like eggshell ash (ESA) and crushed stone powder (CSP) which are selected based on their cost-effectiveness, availability, pollutant reducibility and environmental compatibility. For this study expansive soil samples from three different test pits were collected to be categorized and identified as the weakest based on their plasticity property, gradation, swelling property and compressive strength and further the weakest soil, selected additives and soil-additive mixtures were characterized for their mineralogical compositions using Fourier transform infrared spectroscopy (FTIR), scanning electron microscope (SEM) and X-ray diffraction (XRD) analysis. This paper also presents the results of some index and engineering properties including Atterberg limits, compaction, free swelling, and unconfined compressive strength of expansive soil mixed with the ESA at 2%, 4%, 6%, and 8% by dry weight of soil. The result shows that the decrease in liquid limit (84.38% to78.23%), plasticity index (47.16% to 32.65%), maximum dry density (1.32g/cm^3 to 1.29g/cm^3), free swelling index (95% to 36.36%), free swell ratio (1.95 to 1.36), and linear shrinkage (21.96% to 18.87%) with the addition of ESA up to 8%. Whereas, increase in optimum moisture content (35.78% to 36.36%), plastic limit (37.22% to 45.58%), and UCS from 94.74kPa to 149.17kPa (uncured) and 177.68kpa (cured) were observed for addition of ESA to soil sample. And also 6% ESA was combined with CSP at 5%, 10%, 15% and 20% by dry weight of soil to investigate their effect on properties of natural soil. Again, the result shows better improvement of soil properties by reducing plasticity property, shrinkage capacity, optimum moisture content, swelling characteristics, and degree of compressibility and increasing rate of consolidation, maximum dry density, and compressive strength of natural soil. Results also shows that curing have positive impact on improvement of expansive soil related to strength. Based on the overall test results, the optimum ratio of ESA and CSP for the improvement of expansive soil in this study was proposed as 6% and 15% respectively.

Keywords: Crushed stone powder, Eggshell ash, Expansive soil, Fourier transform infrared spectroscopy (FTIR), Scanning electron microscope (SEM), X-ray diffraction (XRD)

CHAPTER ONE
INTRODUCTION

1.1. Background

Expansive soils are found in the areas with arid and semi-arid climatic regions and formed due to poor drainage and hot climate condition. They are mostly available with colors varies from gray to deep black and rarely reddish or yellowish, as thickness varies from 30 cm to 15 m (Argaw & Patra, 2014). This type of soil is one of the most problematic soils in the world and even in Ethiopia more than 40% of land area is covered with such soils *(*Yitagesu *et al*., 2009; Aga, 2021). Specifically, the large area of Dukem town is covered with such expansive soils Tamrat, (2013). Expansive soils have a property of volume change caused swelling and shrinking due to seasonal moisture content variation which cause ground vibrations, which might cause structural damage (Canakci *et al*., 2015).

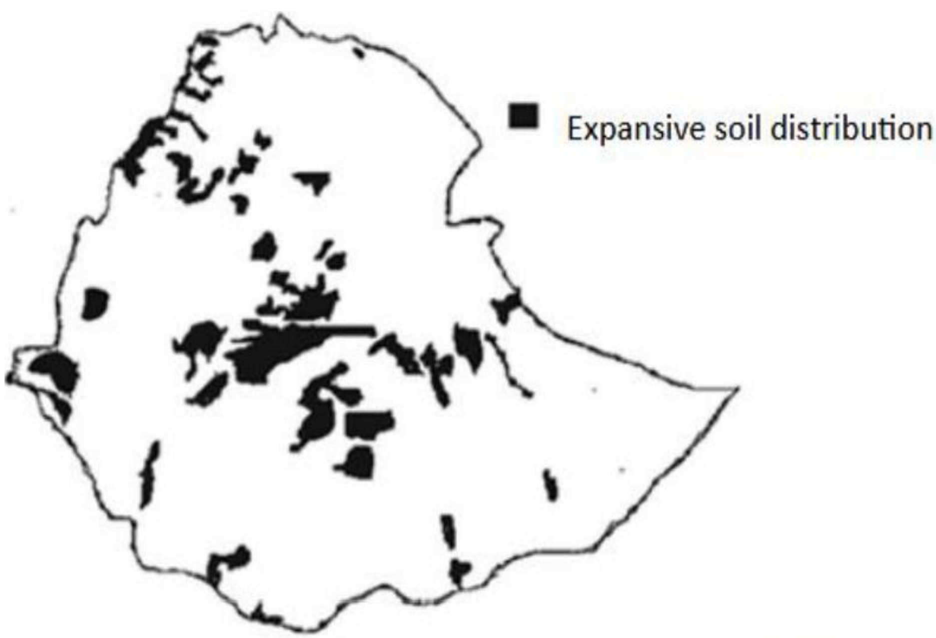


Figure 1-1: Distribution of expansive soil in Ethiopia (Aga, 2021)

Expansive soils can be problematic in engineering applications, depending mostly on their interactions with water. They are mostly contain smectite or montmorillonite minerals joined with weak Vander wall's bond which have greater affinity to absorb more water and expand, causes volume change of mass. Such changes in soil volume can cause very significant damage to lightweight structures such as cracks on the walls of the single-story buildings, and distortion on pavements and pipelines constructed on them (Canakci *et al*., 2015). The annual damage to civil engineering structure due to expansive soil are estimated to be in billions of dollars throughout the world (Puppala & Cerato, 2009; Atahu *et al*., 2019) , which is more than damages caused by natural disasters like earthquakes, floods, tornados, and hurricanes (Calik & Sadoglu, 2014; Atahu *et al*., 2019).

Different strategies have been formulated to overcome the damages arising from the characteristics of expansive soils. Removing of such poor soil and replacing it by an appropriate material or improving it by using chemical or mechanical stabilization are the most commonly used solutions. In practice, traditional stabilizers, like lime and cement, are widely used. But recently, there is a tendency to replace conventional stabilizers with industrial and agricultural bi-products due to the limited sources and increase in the cost of traditional stabilizers (Atahu *et al*., 2019). Numerous researchers investigated the application of several waste materials like rice husk ash, coffee husk ash, bagasse ash, rock powder and eggshell powder as a stabilizing agent to enhance the properties of expansive soils and to reduce the construction cost (Eberemu & Sada, 2013; Atahu *et al*., 2019; Surjandari *et al*., 2017; Blayi *et al*., 2021; Bapiraju & Prasad, 2019). From their finding result obtained, they concluded that incorporating of such additives to expansive soil was improved engineering properties of samples by increasing strength and decreasing plasticity index & compressive behavior of poor soils.

In developing country like Ethiopia there is large expansion scale of construction industry which causes production of huge mass of aggregate production. In line to this, there is also production of large mass of stone powder produced dumped on nearby open land in crusher sites while aggregate production process. And also, according to statistical data from Ethiopian CSA, there were around 18.4 thousand tons of eggshell waste produced annually throughout the country. Both waste material produced in large volumes are available in least or no cost and have negative impact on environment if they are not well managed. Therefore, this research is aimed to examine effect of ESA and CSP on geotechnical properties of expansive soil collected from early selected study area. Soil samples from Dukem town were collected and investigated for their various engineering and strength properties. The properties of original bare soil such as natural moisture content, grain size distribution, specific gravity, Atterberg limit, linear shrinkage, free swell, compaction, and UCS were determined and as well as strength properties of reinforced soils such as Atterberg limit, free well, compaction, UCS and one dimensional consolidation at different percentages of ESA (2%, 4%, 6% and 8%) and CSP (5%, 10%, 15% and 20%) were investigated and compared. Finally, the proportion that gives better result were proposed as optimum dosage of admixtures.

1.2. Statement of the Problem

In most areas of our country, expansive soil causes cracks and defects on structural and non-structural members of building, highway, railway, dams, and underground structures constructed over it, which causes extra expense of maintenance and even loss of life. It is due to the presence of a montmorillonite type clay mineral in expansive soil that exhibits potential swelling and shrinkage when they are subjected to moisture content variation. As per Tamrat, (2013) large part of Dukem town is covered with such expansive soils. However, so many light weights, residential building and industrial buildings were constructed on this problematic soils and they are failed before their design life. Therefore, the strength improvement of such type of soil with locally available materials is required. Hence, from daily usage of egg in domestic, fast-food joints and poultry areas, many thousand tons of eggshell was obtained as waste product throughout overall part of the country which has significant effect on environmental impact if it is not well managed and properly controlled. And also, on different local crusher sites there is stone powder produced with large volume while aggregate production process which is available in least cost and this powder is mostly cart away to nearby sites & reduces availability of open land. Finally, current cost of conventional admixtures was also forced the researcher to find optional way of getting and using low-cost materials.

1.3. Objectives of the Study

1.3.1. General objective

The main objective of this study is to investigate the effect of eggshell ash and crushed stone powder on geotechnical properties of expansive soil.

1.3.2. Specific objectives

- To determine engineering properties and major available mineral components in expansive soil and admixtures.
- To determine effect of eggshell ash and crushed stone powder on consistency limit and swelling index of expansive soil.

- To investigate effect of eggshell ash and crushed stone powder on compaction parameters of expansive soil.
- To assess effect of eggshell ash and crushed stone powder on compressive strength and compressibility parameters of expansive soil.

1.4. Research Question

- What are engineering properties and major available mineral components in expansive soil and admixtures?
- What is the effects of incorporating eggshell ash and crushed stone powder on consistency and compaction characteristics of expansive soils?
- What is the effects of eggshell ash and crushed stone powder on strength and compressibility parameters of expansive soils?
- What is the optimum amounts of eggshell ash and crushed stone powder needed to improve expansive soils?

1.5. Scope of the Study

This research was compiled through by reviewing different literatures related to the area and performing a series of laboratory experiments on materials used for the study. It was aimed to investigate the improvement in geotechnical properties of expansive soils collected from Dukem town with addition of eggshell ash and crushed stone powder at different mix proportions. The laboratory tests conducted were mineralogical characterization (FTIR, SEM and XRD) tests, specific gravity test, grain size analysis, free swell index test, standard compaction tests, Atterberg limit tests, linear shrinkage test, unconfined compressive strength test and one dimensional consolidation test. The standard method used for laboratory test was most commonly under control of ASTM standard, except British standard was used for linear shrinkage test and Indian standard was used for free swell index test. Finally, the result achieved through study will be analyzed, presented, discussed and concluded in a well-defined manner.

1.6. Significance of the Study

Construction on expansive soil for geotechnical application causes major problems due to its poor strength characteristics in addition to continuous variation in volume change. This cyclic volume change initiate damage to the lightweight structures resting on it. So, to overcome such problems in respective study area and mitigate the impact of expansive soils on various construction projects, this study maintains better understanding to use locally available domestic & industrial wastes like ESA and CSP for ground improvement. Nowadays, an effort is also made to reduce the impact of such wastes on the environment, then utilize them for the improvement of geotechnical properties of weak soils and reduce the usage of high cost conventional stabilizers like cement and lime. And also the study is going to be used as a source of information and reference for the future works in Ethiopia by introducing and exploring previous work done abroad on this area.

1.7. Structure of the Thesis

The thesis is divided into five chapters. The first chapter starts with introduction, and describes background, purpose, scope, limitation and significance of the study. The second chapter provides a brief literature review on expansive soil, soil stabilization methods, and completed related research works on expansive soil improvement and microstructural studies. The third chapter addresses the method and the materials used for the investigation, as well as the laboratory testing protocols performed. The fourth chapter includes scientific analysis and discussion of the test findings, as well as a comparison of the test results to other similar test outcomes. The fifth chapter summarizes the overall findings and recommendations that may be drawn from the current study effort.

CHAPTER TWO

LITERATURE REVIEW

2.1. Origin of Expansive Soil

The destructive process in the formation of soil from rock may be either through physical or chemical weathering. Physical process of soil formation is simply changing of rock particle size which is most commonly undergoes due to action of wind, water or glacier erosion and also cyclic action of freezing and thawing which creates cracks in rock mass. The resulting soil particles have the same mineralogical composition as that of the parent rock, but their particle size is too finer. But chemical processes result in changes in the mineral form of the parent rock due to the action of water, especially if it contains traces or drops of acid or alkali, oxygen and carbon dioxide. Chemical weathering results in the formation of groups of crystalline particles of colloidal size less than 0.002 mm, known as clay minerals. For example, the overall feldspar weathering process can be simplified as the alteration of sodium feldspar to common clay minerals, such as kaolinite *(*Yuan *et al.*, 2019).

$$2NaAlSi_3O_8 + 2H^+ + 9H_2O \rightarrow Al_2Si_2O_5(OH)_4(\text{kaolinite}) + 4H_2SiO_{4(aq)} + 2Na^+ \qquad (1)$$

And also potassium feldspar altered to kaolinite due to action of acidic rain as shown on chemical equation below.

$$2KAlSi_3O_8 + 2CO_2 + 11H_2O \rightarrow Al_2Si_2O_5(OH)_4 + 4H_4SiO_{4(aq)} + 2K^+ + 2HCO_3{}^- \qquad (2)$$

2.2. Properties of Expansive Soil

Expansive soil is clay such as montmorillonite or bentonite that is prone to expansion or shrinkage, directly related to seasonal variation in water content. Expansive soils swell when they exposed to large amounts of water and shrink when the water evaporates or when they get dry. This continuous cycle of wetting and drying of soil keeps the soil in perpetual motion, causing structures built on such soil to sink or rise unevenly, often requiring extra foundation repair. Expansive soils are composed primarily of minerals in incredibly fine particles with little or no organic material and are thus incredibly viscous, proving difficult to drain. Many soils that exhibit swelling and shrinking behavior contain expansive clay minerals, such as smectite, that absorb water, the more of this clay soil contains the higher its swell potential and the more water it can absorb. As a result, these materials swell, and thus increase in volume, when they get wet and shrink when they dry. The more water they absorb, the more their volume increases (Jones *et al*., 2012).

2.3. Clay Mineral Basic Unit

The properties that define the composition of clay minerals are derived from chemical compound present in clay minerals, symmetrical arrangement of atoms and ions and the forces that bind them together. The clay minerals are mainly known as the complex silicates of various cations such as aluminum, magnesium and iron. On the basis of the arrangement of these ions, basic crystalline units of the clay minerals are of two types (Neeraj & Mohan, 2021). These are:

2.3.1. Silicon-oxygen tetrahedron

It consists of a single central silicon atom surrounded by four oxygen atoms, with which the central atom bonds. The geometric figure drawn for this arrangement has four sides, with each side being an equilateral triangle. To visualize this, imagine a three-dimensional ball and stick model in which three oxygen atoms are holding up their central silicon atom, much like the three legs of a stool, with the fourth oxygen atom sticking straight up above the central atom. Then tetrahedron units were joined together in different configuration to form the tetrahedral silica sheet.

2.3.2. Aluminum or magnesium octahedron

Consists of a single aluminum or magnesium atom surrounded by six hydroxyl ions and combined to form gibbsite or brucite octahedron sheet, respectively. That means, if aluminum is the dominating atom in each connected unit it becomes gibbsite sheet unless it becomes brucite sheet if magnesium is the dominating atom.

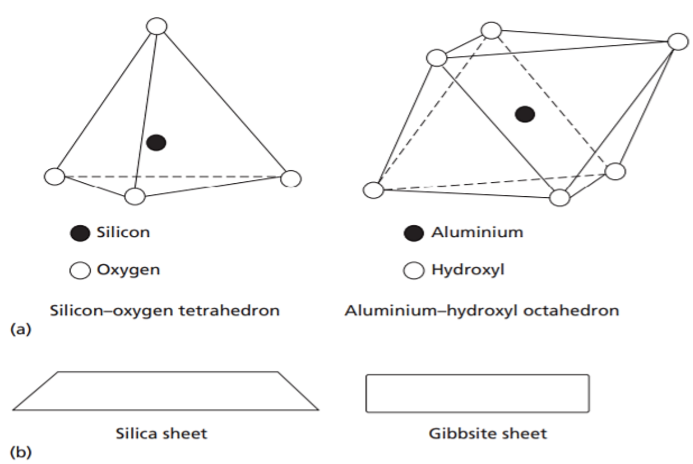


Figure 2-1: Clay minerals basic unit (Craig & Knappett, 2012)

2.4. Common Structures of Clay Mineral Groups

2.4.1. Kaolinite group

This group of clay minerals consist of single silica tetrahedral sheet & alumina octahedral sheet and its structure is often referred to as the 1:1 lattice type with typical basal spacing of about 7.2Å (Mitchell & Soga, 2005). They are derived from the weathering of either sodium or potassium feldspar under acidic water action and contain no exchangeable cations (Yuan *et al*., 2019).

The hydrogen bonding between the tetrahedral and octahedral layers is significant. This bonding holds the 1:1 layer tightly together leaving little or no interlayer space for absorption of water or cations, thus shrink & swell phenomenon is virtually non-existence (Dixon, 1989). There is high degree of regularity in the stacking of well crystallized kaolinite units, with crystal diameter ranging from 0.5-4μm and with specific surface area of 10-20 m²/g (Chen, 1975). The theoretical formula of kaolinite is given as $Al_4Si_4O_{10}(OH)_8$. It is the least active clay mineral with the lowest plasticity and lowest capacity for adsorbing cations among three clay mineral groups.

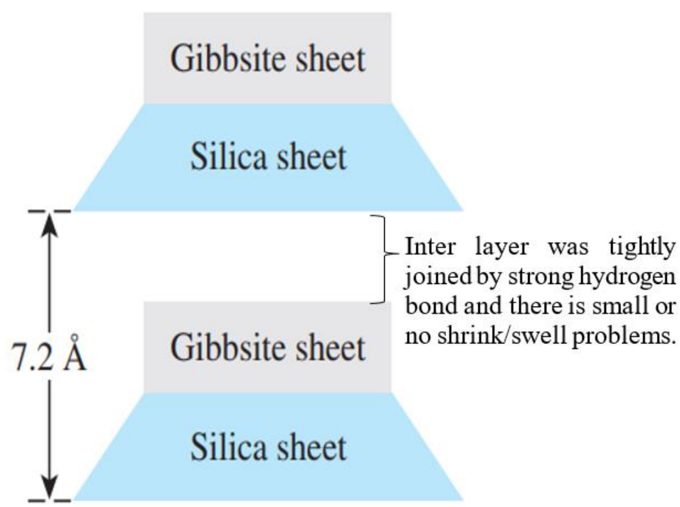


Figure 2-2: General structural arrangements of kaolinite clay mineral (Das & Sobhan, 2014)

2.4.2. Illite group

The structure of illite was formed of two silica tetrahedral sheets with a central sandwich of aluminum or magnesium octahedral sheet. The illite group was also arranged with a 2:1 lattice type, which is similar to the montmorillonite group in structural arrangements. They are derived from weathering of silicates including mica and alkali feldspar under alkaline conditions (Mitchell & Soga, 2005).

When they compared to montmorillonite minerals, illite minerals have low plasticity and swell potential due to their larger particle diameter of 0.5-10μm and lower specific surface area of 65-180 m^2/g (Chen, 1975). Poorly hydrated potassium cations occupy the area between T-O-T sequences of layers, which is accountable for the lack of swelling. Illite is structurally similar to Muscovite, but contains somewhat more silicon, magnesium, iron & water and significantly less tetrahedral aluminum and interlayer potassium. They have a common formula of $[(K, H_3O)(Al, Mg, Fe)_2(Si, Al)_4O_{10}[(OH)_2, (nH_2O)]]$ and contain an inner layer potassium ion (K^+) that is non-exchangeable. The shrink & swell potential of illite minerals is slightly higher than that of kaolinite, but much lower than that of smectite group minerals.

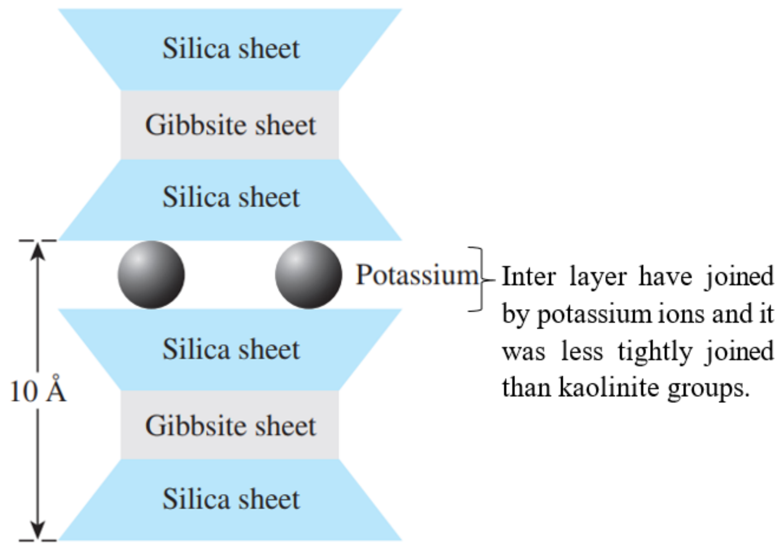


Figure 2-3: General structural arrangements of illite clay minerals (Das & Sobhan, 2014)

2.4.3. Montmorillonite or smectite group

The general term for all expanding 2:1 phyllosilicate minerals are called smectite group, which consists of two silica tetrahedral sheets connected by aluminum or magnesium octahedral sheets at the center. Individual clay particles are plate-shaped with an average diameter of roughly 1μm and a thickness of 9.6Å detected under magnification of about 25,000 times using a scanning electron microscope with a typical basal spacing of 9.6-17.1Å or more (Mitchell & Soga, 2005). Montmorillonite minerals show extensive inner layer expansion on hydration. The particle diameter ranges from 0.05-10 μm with large active specific surface area of about 50-840 m^2/g exposed, allowing enormous range of guest molecules like water to introduce because inter sheet bonding is mainly due to weak van der Waal's forces (Chen, 1975). Chemically, it was a hydrated sodium calcium aluminum magnesium silicate hydroxide, with a chemical formula $(Na, Ca)_3(Al, Mg)_2(Si_4O_{10})(OH)_2.nH_2O$.

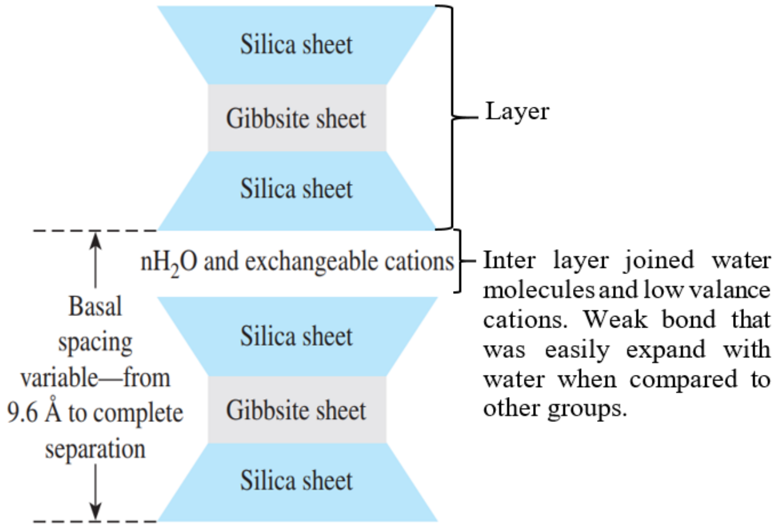


Figure 2-4: General structural arrangements of smectite clay mineral (Das & Sobhan, 2014)

2.5. Methods of Identifying Expansive Soils

Investigation of expansive soils on earth crust was generally done through two important methods. The first method is through visual identification and recognize the material as expansive soil and the second method is through sampling, characterizing, testing and measuring of material properties to be used as the basis for design.

2.5.1. Field identification

Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages (Nelson & Miller, 1975). Important identities to be observed while investigation was done includes:

- ✓ High dry strength and low wet strength.
- ✓ Sticking property and low traffic ability when it gets wet.
- ✓ Usually have a black or gray color.
- ✓ Wide or deep shrinkage cracks appears on the surface.
- ✓ Cut surfaces have a shiny appearance.
- ✓ Appearance of cracks on nearby structures.

These types of soil are most commonly found in arid and semi-arid areas of the globe because there is a large variation in temperature and rainfall.

2.5.2. Laboratory identification

This method of identification can be done through mineralogical identification and inferential methods.

A) Mineralogical identification: Clay mineralogy is a fundamental governing factor that control property of expansive soil (Nelson & Miller, 1975). Clay minerals can be identified using a variety of techniques, the more common of which are:

- ✓ X-ray diffraction (XRD)
- ✓ Differential thermal analysis (DTA)
- ✓ Dye adsorption
- ✓ Chemical analysis
- ✓ Electron microscope resolution (EMR)

Chen, (1975) argued that these techniques should be used in combination with each other to get better and consistent outcomes. However, these procedures are only used in research laboratories due to the need for complex and specialized apparatus, which is expensive, as well as professional analysis skill was required for the generated result data.

B) Inferential Testing Methods: These methods were tried to link some of the index properties of fine-grained soils with the soil clay mineralogical composition to estimate their swell potential. They can be classified as indirect methods and direct methods.

1) Indirect methods: These method make use of soil index properties such as liquid limit, shrinkage limit, percent clay size composition of soils and also some of the indices such as plasticity index, shrinkage index and the like to estimate the swell potential of soils (Asuri & Keshavamurthy, 2016).

a) Liquid limit: This upper bound of plasticity limit is determined in the laboratory by the conventional Casagrande method or by the fall cone method. Liquid limit of a soil is regarded as the water holding capacity of the soil, which in turn has been taken as a measure of soil swell potential. Many classification schemes are available in different geotechnical engineering literatures to recognize the degree of soil swell potential based on the liquid limit of fine-grained soils, as shown in table 2-1.

Table 2-1: Expansive soil classification based on LL (Asuri & Keshavamurthy, 2016)

Swell potential	Liquid limit (%)		
	Chen (1975)	Snethan *et al*.(1977)	IS:1498(1970)
Low	<30	<50	20-35
Medium (marginal)	30-40	50-60	35-50
High	40-60	>60	50-70
Very high	>60	-	70-90

b) Plasticity Index: It is the difference between the liquid limit and plastic limit of fine-grained soils. Higher the plasticity index, more plastic the soil is and have higher soil swell potential. Different schemes available to recognize the soil expansiveness based on the plasticity index of the soils are given in table 2-2.

Table 2-2: Expansive soil classification based on PI (Asuri & Keshavamurthy, 2016)

Swell potential	Plasticity Index (%)		
	Holtz and Gibbs (1956)	Chen (1975)	IS:1498(1970)
Low	<18	0-15	<12
Medium	15-28	10-35	12-23
High	25-41	20-55	23-32
Very high	>35	>35	>32

c) Shrinkage limit: It represents the lower bound water content for any volume reduction of a soil mass and regarded as a measure of volume stability of the in situ soils (Asuri & Keshavamurthy, 2016). The table given below present the schemes available to recognize swelling potential of fine-grained soils based on shrinkage limit.

Table 2-3: Expansive soil classification based on SL (Emmanuel *et al*., 2021)

Swell Potential Holtz and Gibbs (1956)	Shrinkage Limit (%)	Volume change Altmeyer (1956)	Shrinkage Limit (%)
Low	>15	Non-critical	>12
Medium	10-16	Marginal	10-12
High	7-12	Critical	<10
Very high	<11		

d) Shrinkage Index: is the numerical difference between shrinkage limit and plastic limit for remolded soils (IS 2809, 1972). The classification of the swell potential of fine-grained soils based on their shrinkage index is given in the table below.

Table 2-4: Expansive soil classification based on SI (IS:1498, 2007)

Swell Potential	Shrinkage Index (%)
Low	<15
Medium	15-30
High	30-60
Very high	>60

e) Free swell index: Free swell index is also one of the most commonly used simple test method to estimate the swelling potential of expansive clay (Rao, 2006). In this method, two oven-dried samples weighing 10g each and passing through 0.425 mm sieve were taken. One sample is put in a 100ml graduated glass cylinder containing kerosene which is nonpolar liquid and the other sample is put in a similar cylinder containing distilled water. Both the samples are gently stirred and left undisturbed for 24 hours, and then their volumes are noted. Then free swell index is expressed as:

$$\text{Free swell index}(\%) = \left(\frac{V_W - V_k}{V_k}\right) * 100 \tag{3}$$

Where: V_w and V_k are final soil volume soaked in water and kerosene, respectively.

Table 2-5: Relationship between FSI and degree of expansiveness (Rao, 2006)

Degree of expansiveness	Free swell index (%)
Low	<20
Moderate	20-35
High	35-50
Very High	>50

f) Free swell ratio: To determine the swell property Prakash & Sridharan, (2004), proposed the free swell ratio method of characterizing the soil swelling. Free swell ratio is defined as the ratio of sediments volume of 10 cm^3 oven dried soil passing through 0.425 mm sieve in distilled water to that of kerosene.

$$\text{Free swell ratio} = \frac{V_w}{V_k} \tag{4}$$

Where: V_w and V_k are final soil volume soaked in water and kerosene, respectively.

Table 2-6: Classification of soil based on FSR (Prakash & Sridharan, 2004)

FSR	Soil expansiveness	Clay type	Dominant mineral type
<1.0	Negligible	Non-swelling	Kaolinite
1.0-1.5	Low	Mixture of swelling & non-swelling	Kaolinite and montmorillonite
1.5-2.0	Moderate	Swelling	Montmorillonite

2.0-4.0	High	Swelling	Montmorillonite
>4.0	Very High	Swelling	Montmorillonite

g) Cation exchange capacity (CEC): is the quantity of exchangeable cations required to balance the negative charge on the surface of the clay particles and expressed in milli equivalents per 100 grams of dry clay, and it is related to clay mineralogy. Clays having high CEC values have a high surface activity and have a greater water holding capacity. In general, swell potential increases as the CEC increases (Nelson & Miller, 1975).

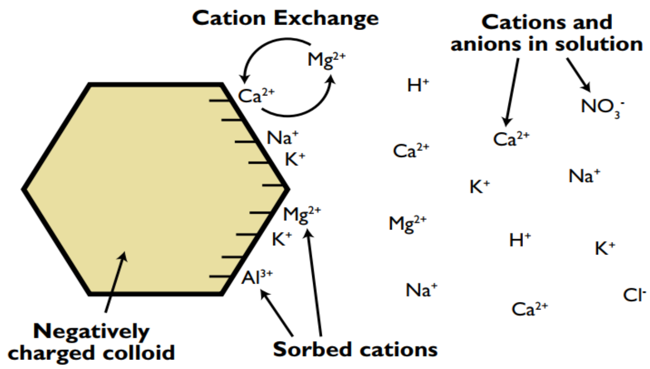


Figure 2-5: Simplified representation of cation exchange capacity (McCauley *et al*., 2005)

Table 2-7: Typical CEC values of basic clay minerals (Nelson & Miller, 1975)

Clay mineral	CEC (meq/100gm)
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

h) Activity of clays: As per Venkatramaiah, (2006) revealed, indirect method of obtaining information on the type and effect of clay mineral in a soil is to relate plasticity index to the amount of clay size particles. It is known that for a given amount of clay mineral, the plasticity resulting in a soil will vary for the different types of clays. Activity can be determined from the results of the standard laboratory tests such as the hydrometer test, liquid limit and plastic limit test. Activity (A) is defined as the ratio of plasticity index to the percentage of clay sizes, as shown in equation below.

$$\text{Activity(A)} = \frac{\text{Plasticity index}}{\text{\% Clay fraction}} \tag{5}$$

Clays containing kaolinite will have relatively low activity and those containing montmorillonite will have high activity. Based on this classification:

- ✓ Montmorillonite clay (expansive clay) is defined as active.

- ✓ Illite clay as normal and,
- ✓ Kaolinite clay as inactive.

Table 2-8: Degree of colloidal activity (Venkatramaiah, 2006)

Degree of activity	Activity
Inactive clay	<0.75
Normal clay	0.75-1.25
Active clay	>1.25

2) Direct methods: The most satisfactory and convenient method of determining the swelling potential and swelling pressure of an expansive clay is by direct measurement. Direct measurement of expansive soils can be achieved by the use of the conventional one dimensional consolidation (Chen, 1975). These methods offer the most useful data by direct measurement; and tests are simple to perform and do not require complicated equipment.

2.6. Soil Classification System

Soils differ according to their origins, compositions, locations, geological histories, and a variety of other characteristics. Even if two soils were recovered from close boring holes on the same building site, they may be substantially different in their geotechnical properties. However, it is more convenient and comfortable for engineers when soils are classified into groups with comparable engineering properties. Without doing laboratory or field-testing, engineers may comprehend the approximate engineering properties of such categorized soils. Soil classification is a procedure that assists engineers during the early design stage of geotechnical engineering problems.

There are many soil classification systems that are currently used in many countries. Those methods mostly uses consistency limits and soil gradation information for the classification of soils. Commonly, there are two classification systems that are widely used in geotechnical engineering aspect. These are:

A. Unified soil classification system (USCS) and,
B. American association of state highway and transportation officials (AASHTO) classification system.

2.6.1. Unified Soil Classification System

This system was firstly developed by Arthur Casagrande for wartime airfield construction in 1940 and the system was modified and adopted for regular use by army corps of engineers and then by the bureau of reclamation in 1952 as the unified soil classification system which currently incorporated on ASTM standard manual. According to this method, soils are divided into two sub major categories.

1) Coarse grained soil that are gravelly and sandy in nature with less than 50% of soil sample passing through the sieve size of 0.075 mm, i.e. F_{200}<50%. Naming of categorization starts with prefix G for gravelly soils and S for sandy soils.
2) Fine-grained soils when 50% or more of soil sample passing through the sieve size 0.075 mm, i.e. F_{200} > 50%. Naming of group symbol starts with M for inorganic silts, C for inorganic clays, O for organic silt and clays, and Pt for peats and muck for highly organic soils.

Again, coarse grained materials subdivided into gravel and sand based on amount of soil retained on sieve having opening size of 4.75 mm. When more than 50% soil retained on sieve No.4 the soil is gravel and when more than 50% of soil pass sieve No.4 material falls under sandy soil. Similarly, fine-grained soils also subdivided into silt and clay based on their location on the plasticity chart shown on figure 2-6.

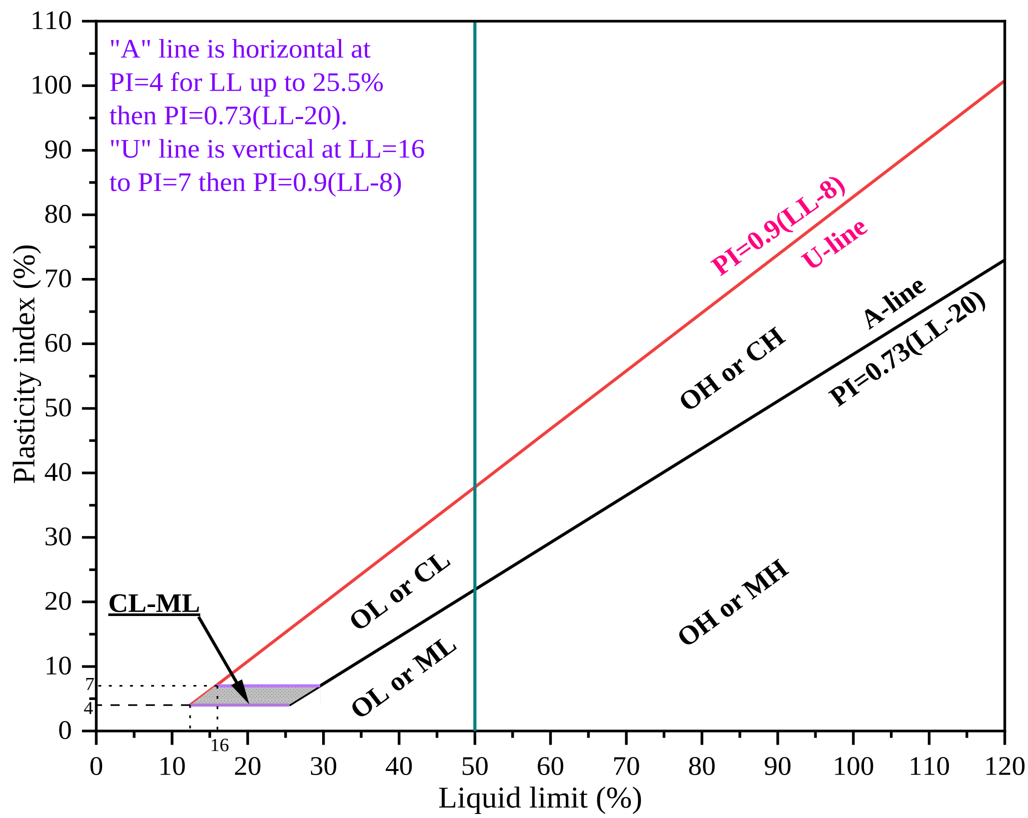


Figure 2-6: Plasticity chart for fine-grained soils classification by USCS (ASTM D 2487, 2000)

Table 2-9: Soil classification based on particle size range by USCS

Soil type		Particle Size (mm)
Clay		$\leq$0.002
Silt		0.002-0.075
Sand	Fine	0.075-0.425
	Medium	0.425-2.0
	Coarse	2.0-4.75
Gravel		4.75-75

There are also some additional qualitative suffixes added on group symbol based on amount of Coefficient of uniformity (C_u) and coefficient of curvature (C_C) obtained from gradation curve for coarse grained soils and suffixes added on group symbol of fine-grained soil based on amount of plastic index and liquid limit amount as summarized in table 2-10.

Table 2-10: Unified soil classification system chart (ASTM D 2487, 2000)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines [E]	$C_U \geq 4$ and $1 \leq C_C \leq 3$ [C]	GW	Well-graded gravel [D]
			$C_U < 4$ and/or $1 > C_C > 3$ [C]	GP	Poorly graded gravel [D]
		Gravel with Fines More than 12% fines [E]	Fines classify as ML or MH	GM	Silty gravel [D,F,G]
			Fines classify as CL or CH	Urule; 1>GC	Clayey gravel [D,F,G]
	Sands 50% or more of coarse Fraction passes No. 4 sieve	Clean sands Less than 5% fines [I]	CU > 6 and 1 $\leq$ CC $\leq$ 3 [C]	SW	Well-graded sand [H]
			$C_U < 6$ and/or $1 > C_C > 3$ [C]	SP	Poorly graded sand [H]
		Sands with Fines More than 12% fines [I]	Fines classify as CL or CH	SM	Silty sand [F,G,H]
			Fines classify as CL or CH	SC	Clayey sand [F,G,H]
FINE-GRAINE SOILS 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic	$\frac{LL(\text{oven dried})}{LL(\text{dried})} < 0.75$	OL	Organic clay [K,L,M,N]
				OL	Organic silt [K,L,M,O]
	Silt and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic silt [K,L,M]
		Organic	$\frac{LL(\text{oven dried})}{LL(\text{dried})} < 0.75$	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
HIGHLY ORGANIC SOILS	Primarily organic matter , dark in color , and organic odor			PT	Peat

A Based on the material passing the 3-in. (75 mm) sieve. **B** If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name. **C** $C_U = D_{60}/D_{10}$ & $CC = (D_{30})^2 / D_{10} \times D_{60}$. **D** If soil contains $\geq$ 15% sand, add "with sand" to group name. **E** Gravels with 5 to 12% fines require dual symbols: GW-GM: well-graded gravel with silt, GW-GC: well graded gravel with clay, GP-GM: poorly graded gravel with silt, GP-GC: poorly graded gravel with clay. [F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM. [G] If fines are organic, add "with organic fines" to group name. [H] If soil contains $\geq$ 15% gravel, add "with gravel" to group name. [I] Sand with 5 to 12% fines require dual symbols: SW-SM: well-graded sand with silt, SW-SC: well graded sand with clay, SP-SM: poorly graded sand with silt, SP-SC: poorly graded sand with clay. [J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay. [K] If soil contains 15 to 29% plus No.200, add "with sand" or "with gravel," whichever is predominant. [L] If soil contains $\geq$ 30% plus No.200, predominantly sand, add "sand" to group name. [M] If soil contains $\geq$ 30% plus No.200, predominantly gravel, and add "gravelly" to group name. [N] PI $\geq$ 4 and plots on or above "A" line. [O] PI < 4 and plots below "A" line. [P] PI plots on or above "A" line. [Q] PI plots below "A" line.

2.6.2. AASHTO classification system

This soil classification system is used to determine the suitability of soils for earthworks, embankments, and road bed materials in highway construction. According to this classification system, granular soils are soils in which 35% or less are finer than the No. 200 sieve. Silty clay soils are soils in which more than 35% are finer than the No. 200 sieve (Muni Budhu, 2010). The classification method uses liquid limit, plastic limit and information obtained from grain size distribution curve to classify soils into seven major groups, A-1 through A-7. The first three groups, A-1 through A-3, are granular (coarse grained) soils, while the last four groups, A-4 through A-7, are silty clay (fine-grained) soils. Figure 2-7 and table 2-11 were used for soil classification system. The procedure of classification uses an elimination process of column, from the upper left corner toward downward and right. If the condition on the row is not satisfied, the entire column is eliminated, and it is never referred back. After the last row check for PI, one or possibly more than one column may survive this elimination process. If more than one column survived, the first column from the left is selected as a group or subgroup.

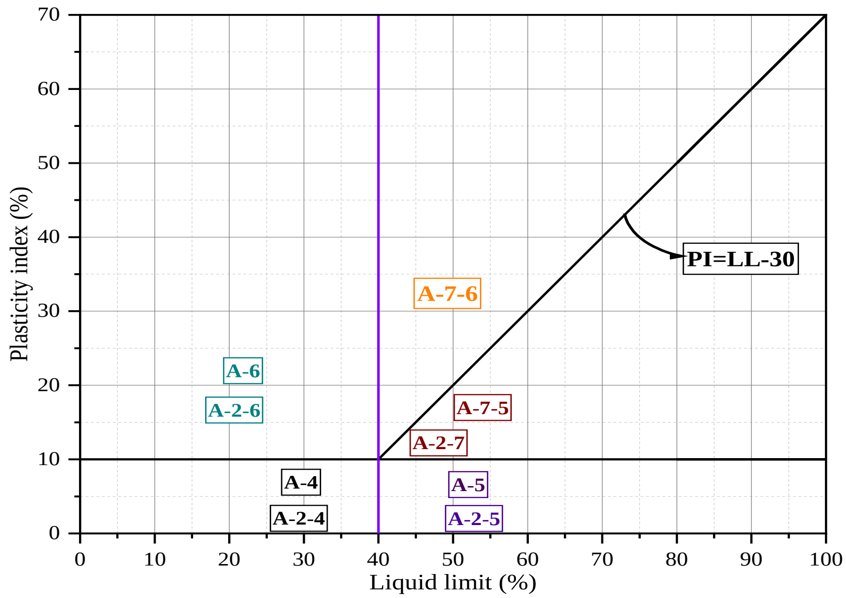


Figure 2-7: Liquid limit and plasticity index ranges for fine-grained materials (AASHTO:M 145-91, 2008)

Table 2-11: Classification of soils and soil-aggregate mixtures (AASHTO:M 145-91, 2008)

General Classification	**Granular Materials (35 Percent or Less Passing 75μm)**							**Silt-Clay Materials (More Than 35 Percent Passing 75μm)**			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, percent passing											
2.00mm(No.10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425mm(No.40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
75μm(No.200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425mm (No.40)											
Liquid limit		-	-	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plastic index		6 max	NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[a]
Usual type of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to Good							Fair to Poor			

[a] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30 and, Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A–3 over A–2.

Group index (GI) value is appended additionally in parentheses to the main group to provide a measure of quality of a soil as highway subgrade material and calculated through the following formula.

$$\mathrm{GI} = (\mathrm{F} - 35)[0.2 + 0.005(\mathrm{LL} - 40)] + 0.01(\mathrm{F} - 15)(\mathrm{PI} - 10) \qquad (6)$$

Where: F is percentage passing 75 µm (No. 200) sieve, expressed as a whole number, LL is liquid limit, and PI is plasticity index.

When the calculated group index is negative, the group index shall be reported as zero and should be reported to the nearest whole number. And also when calculated GI is more than 20, it is assumed to be taken as 20. Soil having less GI has good strength and quality, but soil having higher group index has less quality and not appropriate to use as construction material.

2.7. Theoretical Concept on Consolidation of Fine Grained Soils

Structures are built on soil mass and transfer loads to the subsoil through their foundations. The effect of the loads is felt by the soil, normally up to a depth of about four times the width of the foundation. The soil within this depth gets compressed due to the imposed stresses. The compression of the soil mass leads to the decrease in the volume of the mass, which results in the settlement of the structure. If the settlement is not kept to a tolerable limit, the desire use of the structure may be impaired and the design life of the structure may be reduced. It is therefore important to have a mean of predicting the amount of soil compression or consolidation.

Soil consolidation refers to the process by which the volume of a partially or fully saturated soil decreases due to an applied stress. When a load is applied to a low permeability soil, it is initially carried by the water that exists in the porous saturated soil and result in a rapid increase of pore water pressure. This excess pore water pressure is dissipated as water drains away from the void space and the pressure is transferred to the soil skeleton, which is gradually compressed, resulting in settlements. The consolidation procedure lasts until the excess pore water pressure is dissipated, most of the time up to 24 hours. The total compression of soil under load is composed of three components which includes elastic settlement, primary consolidation settlement, and secondary compression. Elastic settlement occurs due to elastic deformation of soil mass without changing in the moisture content, and primary consolidation is happened because of expulsion of the water that occupies the void spaces. But secondary consolidation occurred due to the plastic adjustment of soil fabrics under a constant effective stress (Muni Budhu, 2010).

The rate of consolidation of soil mass depends on the permeability of the soil and length of drainage path, H_{dr} in the soil. A soil with a lower hydraulic conductivity will take longer time to drain the initial excess pore water, and settlement will proceed at a slower rate. And also, consolidation process with single drainage will take four times longer to reach a particular settlement than for soil consolidated in double drainage path (Muni Budhu, 2010).

Clayey soils undergo consolidation settlement not only under the action of external loads or surcharge loads but also occurred under its own weight or weight of soils that exist above the clay which is geostatic loads. Clayey soils should also undergo settlement when the earth was dewatered (e.g., groundwater pumping) because the effective stress on the clay increases.

2.7.1. Compressibility characteristics

The standard one dimensional consolidation test is conducted on a saturated specimen using an Odometer. The soil specimen is placed inside a metal ring with two porous stones, one at the top of the specimen and another at the bottom. The load is applied to the specimen through a lever arm, and deformation is measured through an attached dial gauge on the consolidation machine. The specimen is kept soaked in water bath during the test. Each load is usually kept for 24 hours. After that, the load is usually doubled and the compression measurement recording is continued. At the end of the test, the dry mass of the test specimen is determined, which is required to determination of volume of solid soils (Fikadu, 2015). Then compressibility characteristics of a soil such as compression index (Cc), recompression index (Cr), coefficient of consolidation (C_v) and coefficient of volume compressibility (m_v) are determined from the one dimensional consolidation test result.

Plots of void ratio (e) after consolidation against effective stress (σ') for a saturated soil were drawn to show an initial compression of loading followed by unloading and recompression. The e-log σ' relationship for a normally consolidated soil is nearly linear, and is called the one dimensional compression line. During compression along this line, permanent or irreversible changes in soil structure continuously take place, and the soil does not revert to the original structure during expansion. If a soil is over consolidated, its state will be represented by a point on the expansion or recompression parts of the e-log σ′ plot. The changes in soil structure along this line are almost fully recoverable, as shown on figure 2-8.

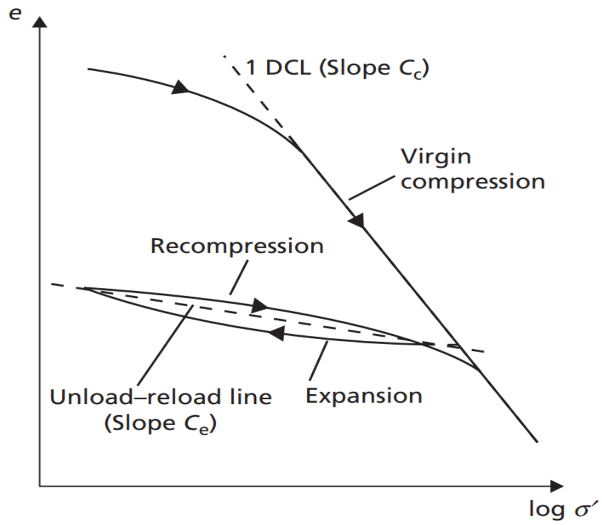


Figure 2-8: Void ratio-stress relationship on logarithmic scale (Craig & Knappett, 2012)

The compressibility of the soil can be quantified by one of the following coefficients. These coefficients are compression index (C_C) and coefficient of volume compressibility (m_v).

A) Compression index (C_C): is the slope of the linear portion of void ratio (e) versus logarithm of effective pressure (log σ′) relationship, is extensively used for settlement determination (Sridharan & Gurtug, 2005). It is also negative slope of consolidation curve and dimensionless parameter of consolidation.

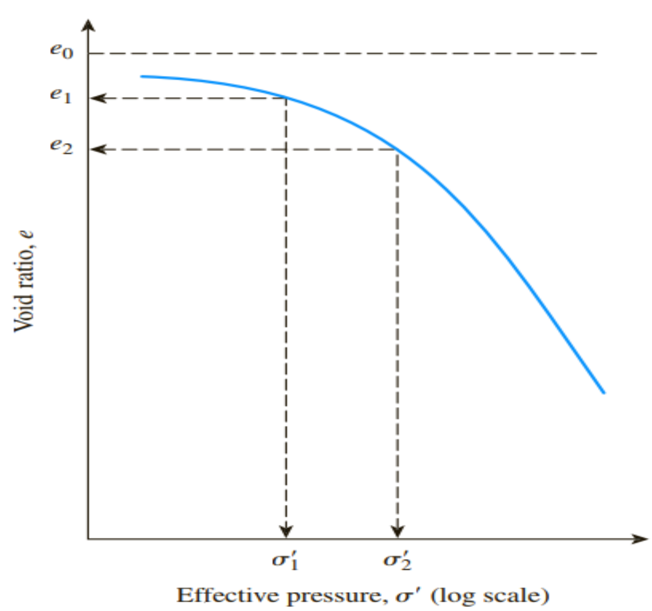


Figure 2-9: Typical plot of e versus log δ' (Das & Sobhan, 2014)

For any two points on the linear portion of the plot shown on figure 2-9, compressibility index becomes calculated by the following equation.

$$\mathrm{Cc} = \frac{e_1 - e_2}{\log(\frac{\sigma_{2'}}{\sigma_{1'}})} \quad (7)$$

Where Cc stands for coefficient of compressibility, e_1 and e_2 are void ratios of soil specimen before and after loading respectively and σ_2' and σ_1' are axial stress that the specimen withstand that are after and before load is doubled respectively.

B) Coefficient of volume compressibility (m_v): defined as the volume change per unit volume per unit increase in effective stress that means ratio of volumetric strain to applied stress (Craig & Knappett, 2012). The units of m_V are the inverse of stiffness (MPa^{-1}). This parameter was calculated from e-σ curve and used to assess degree of compressibility and hydraulic conductivity of soil specimen. The volume change may be expressed in terms of either void ratio or specimen thickness. If, for an increase in effective stress from $\sigma_1{}'$ to $\sigma_2{}'$, the void ratio decreases from e_1 to e_2, then m_V is calculated as:

$$M_v = \frac{(e_1 - e_2)}{(1 + e_0)(\sigma_2' - \sigma_1')} \quad (8)$$

Where m_V is coefficient of volume compressibility of soil specimen at two different effective axial stresses $\sigma_1{}'$ and $\sigma_2{}'$ with respective void ratios e_1 and e_2, and e_0 is initial void ratio.

2.7.2. Pre-consolidation pressure determination

Pre consolidation pressure is the maximum effective vertical overburden stress that a particular soil sample has sustained in the past. It is important in geotechnical engineering, particularly for finding the expected settlement of foundations and embankments. The parameter is determined from e-log σ' curve of loading condition shown on figure 2-10 after conducting consolidation test in the laboratory through some procedures include:

- Draw the tangent to the curve at D through line DH and bisect the angle between the tangent DH and the horizontal line DF through line DG;
- Produce back the straight line part BC, which is the tangent to the end of consolidation curve;
- The abscissa of the point of intersection of the bisector line DG and BC produced gives the approximate value of the pre consolidation pressure.

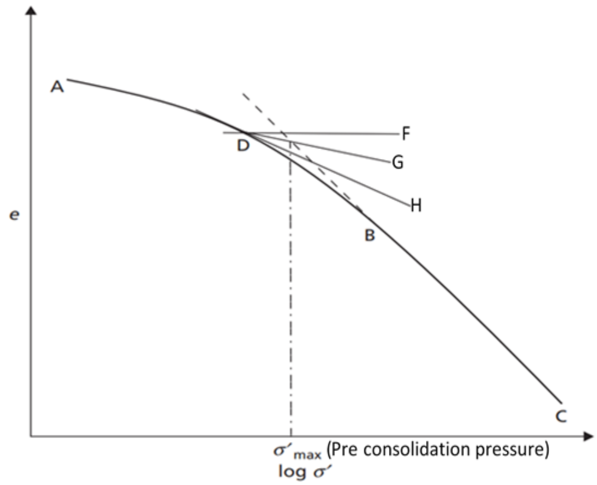


Figure 2-10: Determination of pre consolidation pressure (Craig & Knappett, 2012)

2.7.3. Coefficient of consolidation determination methods

The coefficient of consolidation relates to how long it will take for an amount of consolidation to take place, which is mainly depending up on length of drainage path and permeability of soil mass. It is used to calculate hydraulic conductivity of material. From the test result, it is possible to get deformation at required time interval under action of every axial stresses. And then a plot of deformation versus log time or square root of time was drawn to determine coefficient of consolidation as per Casagrande or Taylor method respectively.

A) Logarithm of time (Casagrande) method: The experimental curve is obtained by plotting the dial gauge readings in the odometer test against the time in minutes, plotted on a logarithmic axis. In the experimental curve, the point corresponding to starting of primary consolidation (U_V=0) can be determined by using the fact that the initial part of the curve represents an approximately parabolic relationship between compression and time (Craig & Knappett, 2012). The final part of the experimental curve is linear but not horizontal, and the point of primary consolidation ending (a_{100}) corresponding to U_V =100% is taken as the intersection of the two linear parts of the curve. Then two points on the curve are selected, which are A and B for which the values of time at point B is four times that of A and the vertical distance between them, ζ, is measured. An equal distance of ζ above point A fixes the dial gauge reading (a_s) corresponding to starting of primary consolidation.

After that, average of deformation gap between a_s and a_{100} will give a dial reading at 50% consolidation (a_{50}) with respective time (t_{50}) to determine coefficient of consolidation by the following equation.

$$C_v = \frac{(T_v * H_{dr}{}^2)}{t_{50}} \qquad (9)$$

Where T_V is time factor taken as 0.197 for 50% consolidation, is taken as half of average thickness of specimen if drainage is two-way and full average thickness of specimen for one way drainage.

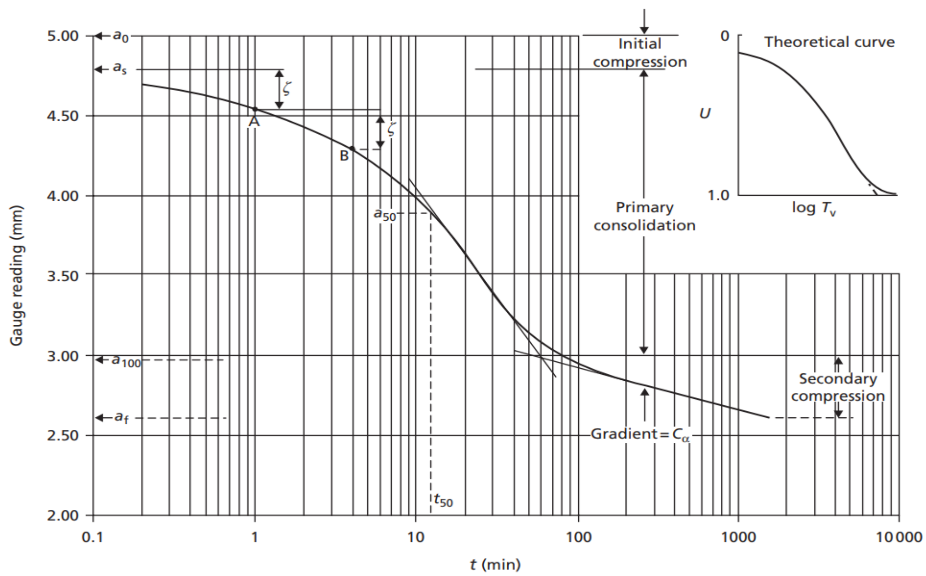


Figure 2-11: Deformation versus time curve on logarithmic horizontal scale (Craig & Knappett, 2012)

B) The square root of time (Taylor's) method: From the result recorded, the dial gauge readings being plotted against the square root of time in minutes as shown on figure 2-12. Then draw a line AB through the early portion of the curve. Finally, draw a line AC based on distance of OC is equal to 1.15 times OB and then abscissa of point D where line AC cross the consolidation curve is equal to square root of t_{90}.

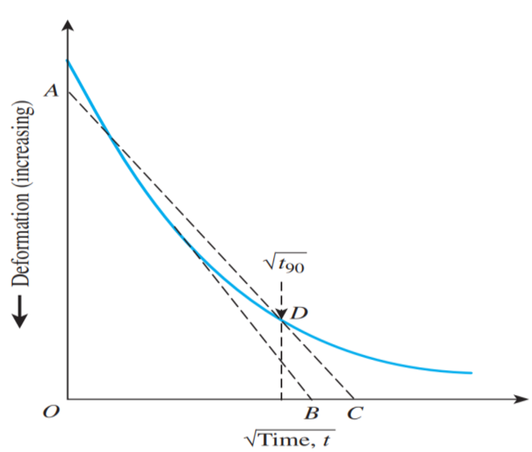


Figure 2-12: Deformation versus square root of time curve (Craig & Knappett, 2012)

For 90% consolidation time factor, T_V is equal to 0.848 and C_v is calculated by the following formula:

$$C_v = \frac{(0.848 * H_{dr}{}^2)}{t_{90}} \tag{10}$$

Where: H_{dr} is determined in the same way as in logarithm of time method.

2.8. Methods of Improving Expansive Soil

Soil stabilization is defined as chemical or mechanical treatment designed to increase and maintain the stability of a soil mass or otherwise to improve its engineering properties (ASTM D653-97, 2014). There are two most commonly methods used for soil stabilization. These are:

2.8.1. Mechanical stabilization

It is a process which includes physical manipulation of earth materials, which most commonly refers to controlled densification either by placement and compaction of soils as designed engineered fills or in situ methods of improvement for deeper applications. Many engineering properties and behaviors of swelling soils can be improved by controlled densification of soils, called compaction methods. Other in situ methods of improvement may involve adding material to the ground, as is the case for strengthening and reinforcing the ground with non-structural members (Nicholson, 2015). The main objective of mechanical stabilization techniques of expansive soil is the reduction of the expansion potential and swelling stress without modifying the original soil chemistry (Fondjo *et al.*, 2021).

A) Compaction: is the process in which stress applied to a soil mass to cause densification as air is displaced from the pores between the soil grains. The main purpose of compaction is to obtain soil that can satisfy three fundamental requirements includes minimize settlement amount, reduce permeability and pore water pressure, increase shear strength and bearing capacity (Fondjo *et al.*, 2021).

B) Soil replacement: is most usually applied in mechanical soil stabilization procedures, in which expansive soil in active zone will be replaced with non-expansive and impermeable material to prevent moisture movement to the surface and differential settlement caused due to moisture variation. Because of the higher replacement expenses of some undesirable soils like swelling soils, the cost-effective improvement procedure may include an efficient stabilization technique (Fondjo *et al.*, 2021).

C) Blending with various soils: includes blending of local soil with another soil of various degrees to get a targeted engineering properties of the last mixture. It can be performed at the worksite or at different locations and then the material mixture is delivered to the place of work, spread, and compacted with an adequate density to attain the required degree of compaction (Fondjo *et al.*, 2021).

D) Pre wetting: Pre wetting of expansive soils has been used with varying levels of success to reduce the expansion potential. Installation of vertical sand drains is done on a grid pattern to reduce the time needed for water penetration and heave by reducing the length of the flow path. In previous decades, it has been applied fairly commonly, but it is being used less frequently in recent years. The concept behind this treatment method assumes that increasing the water content in the expansive foundation soils will cause heave to occur prior to construction, thereby reducing post construction expansion potential. Results of the pre wetting method are unreliable, and its use is not encouraged (Nelson *et al.*, 2015).

E) Wetting and drying cycles: Research works conducted on this area reported that heaving soils exposed to successive drying and wetting exhibits a considerable reduction in expansion potential. Wetting and drying cycles are additionally utilized to investigate the durability of chemical additives utilized in soil stabilization to comprehend the time dependent performance of such substances under field conditions by changing drying and wetting cycle on stabilized soil material (Fondjo *et al.*, 2021).

F) Soil reinforcement: Soil reinforcement technique is the utilization of synthesized or natural meshes to enhance the properties of soils, which can be achieved by adding materials with higher tensile strength such as fiber to enhance the shear resistance of soil material (Fondjo *et al.*, 2021).

2.8.2. Chemical stabilization

Chemical stabilization is the commonly used technique for expansive soil treatment. The purpose of the chemical stabilization of soils is to enhance their stability by increasing grain size of soil particles, decreasing plasticity index, decreasing swelling & shrinking potential, and cementation. Soil stabilization is performed by adding a specific chemical compound amount to the expansive soil (Fondjo *et al.*, 2021).

A) Cement: is a chemical admixture used in soil stabilization to improve engineering properties and mechanical characteristics of soil, like degree of compaction. Generally, cement stabilization is ideally suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles (Musema, 2014). The addition of small quantities of cement, 1% to 2% by weight, to fine-grained soils was used to reduce the liquid limit, plasticity index, and water absorption tendency. The effect of the cement is to bring individual soil particles into aggregations, thus artificially adjusting the grading of the soil.

B) Lime: is a chemical additive that has been used in ground improvement for centuries and have ability to react with medium to fine-grained soils. The main advantage of lime in soil stabilization is to reduce plasticity, swell characteristics and increase strength and workability of soil (Musema, 2014).

C) Kiln dust: is a fine powder, by-product of the Portland cement manufacturing process available in the least cost and mostly used admixture recently (Muhunthan & Sariosseiri, 2008).
D) Fly ash: is chemical additive having mainly silicon and aluminum and mixed with lime or calcium to get better quality (Musema, 2014).

E) Bitumen: is a chemical additive that occurs naturally or as a by-product of petroleum distillation. It is the black pitch used to make asphalt. Asphalt cement, cutback asphalt, tar and asphalt emulsions are all used to achieve bituminous soil stabilization (Musema, 2014).

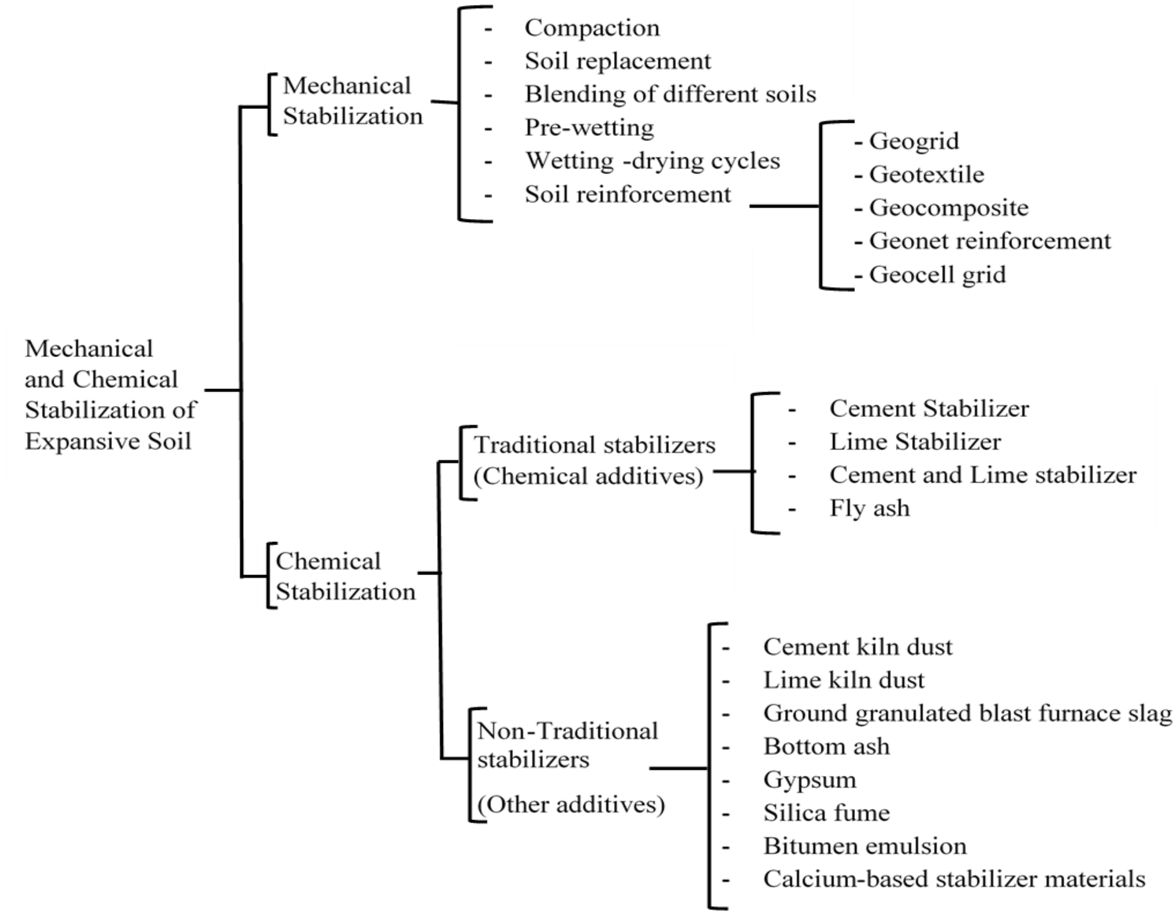


Figure 2-13: Methods of stabilizing expansive soils (Fondjo *et al*., 2021)

2.9. Factors Affecting Choice of Improvement Method

When someone faces difficulties or challenging geotechnical problem regarding soil quality, they must consider a number of variable parameters in determining the type of solution that will best achieve the desired results. The type of solutions that engineers are decided to take as per Nicholson, (2015) may be affected by the following factors.

- Soil type
- Area, depth, and location of land where treatment is required
- Required soil properties
- Availability of materials
- Availability of skills, local experience, and local preferences
- Environmental concerns and,
- Cost of admixtures.

2.9.1. Soil type

This is one of the most important parameters that was control what approach will be applicable and ensures certain ground improvement methods are applicable to only certain soil types and grain sizes. An updated figure was presented by Nicholson, (2015) to graphically represent various ground improvement methods suitable for ranges of soil grain sizes.

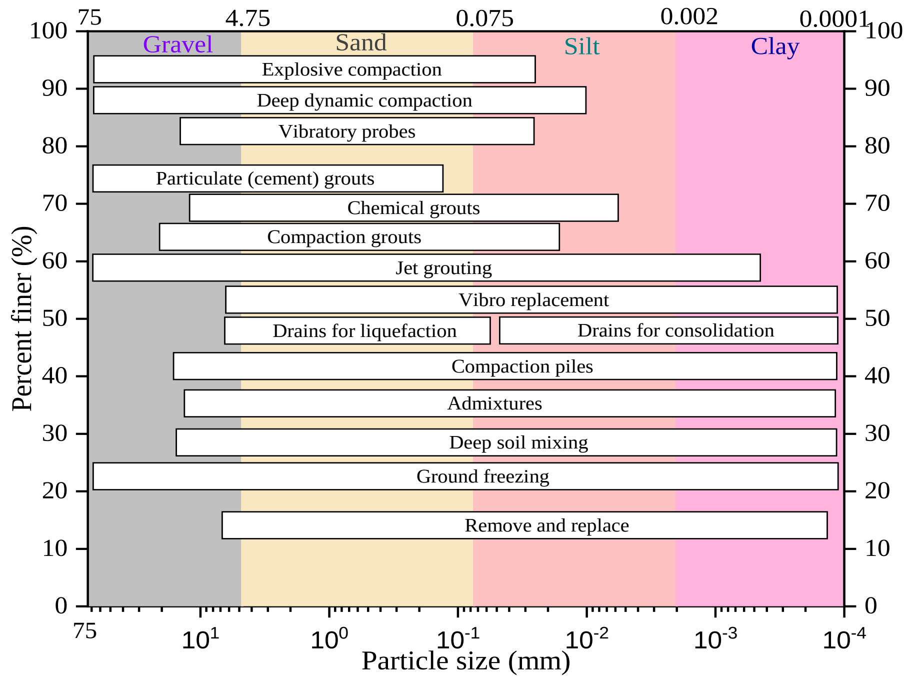


Figure 2-14: Soil improvement methods applicable for different ranges of soil sizes

2.9.2. Area, depth, and location of treatment required

As per Nicholson, (2015), many ground improvement methods have depth limitations that render them unsuitable for application to deeper soil horizons. Depending on the areal extent of the project, economic and equipment capabilities may also play an important role in the decision as to which method is best suited for the project. Location may play a significant role in the choice of method, particularly if there are adjacent structures, concerns of noise and vibrations, or if temperature and availability of water is a factor.

2.9.3. Required soil properties

Nicholson, (2015), reveals that, different methods are used to achieve different engineering properties, and certain methods will provide various levels of improvement and uniformity to improved sites.

2.9.4. Availability of materials

Depending on location of the project and materials required for each feasible ground improvement approach, some materials may not be readily available or cost and logistics of transportation may rule out certain methods (Nicholson, 2015).

2.9.5. Availability of skills, local experience, and local preferences

While the engineers may possess the knowledge and understanding of a preferred method, some localities and project owners may resist trying something that is unfamiliar and locally unproven. This is primarily a social issue, but should not be underestimated or dismissed, especially in more remote and less developed locations.

2.9.6. Environmental concerns

With a better understanding and greater awareness regarding the effects of additives on the natural environment, more attention has been placed on methods that assure less environmental impact. This concern has greatly changed the way that construction projects are undertaken and had a significant effect on methods, equipment, and particularly materials used for ground improvement.

2.9.7. Cost of admixtures

When all else has been considered, the final decision on choice of improvement method will often come down to the ultimate cost of a proposed method, or cost will be the deciding factor in choosing between two or more otherwise suitable methods. Included in this category may be time constraints, in that a more costly method may be chosen if it results in a faster completion, allowing earlier use of the completed project.

2.10. Improvement of Expansive Soil Using Domestic and Industrial Wastes

For many years, unrecycled waste materials cause pollution and impact on the environment and health of human beings. Nowadays, utilizing those materials as stabilizers is one part of assessing the risk posed by such materials. Furthermore, using those materials for soil improvement will be beneficial due to their ease of availability and cost-effectiveness.

2.10.1. Annual production capacity eggs in Ethiopia

According to Ethiopian CSA, (2020), data on poultry population are collected as part of the livestock survey, and the total poultry population at country level is estimated to be about 57million. From these, around 34.36% of total population are laying hens and produces about

369 million eggs annually. Even though as per (Ayalew, 2022), weight of single eggshell is approximated to 4.98gm that means there was production of eggshell waste of 1838 tons per year throughout the country which is large amount of waste needed to be managed and used for the purposes like ground improvement. Again statistical data of world Food and Agriculture Organization of the United Nations also shows annual egg production of Ethiopia from domesticated birds in 2020 is approximated to 49,800 tons as shown in figure 2-15.

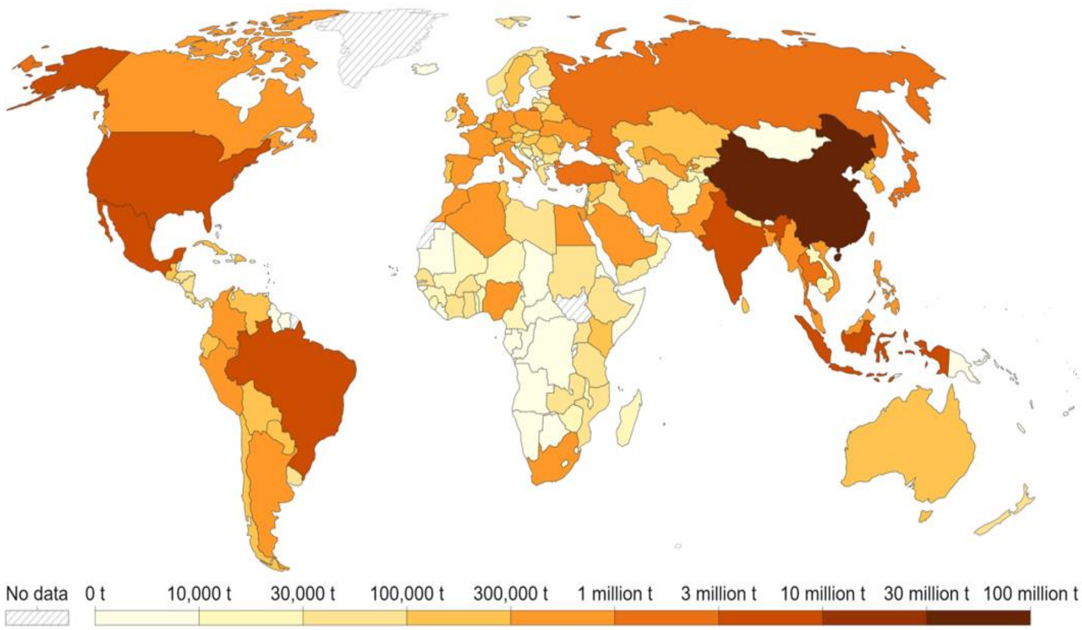


Figure 2-15: World annual egg production map of 2020 (World Food and Agriculture Organization, 2022)

2.10.2. Properties of eggshell ash

An eggshell is the outer covering of a hard shelled egg, and of some forms of eggs with soft outer coats of yolk. Eggshells are domestic waste materials generated from chick hatcheries, bakeries, and fast food restaurants which can disorder the environment and consequently constituting environmental pollution problems which would require proper handling. In the ever-increasing efforts to convert waste to wealth, the efficacy of converting eggshell to beneficial use becomes an idea worth embracing. It is scientifically known that the eggshell is mainly composed of calcium compounds which includes 93.70% calcium carbonate, 4.20% organic matter, 1.30% magnesium carbonate, and 0.8% calcium phosphate (Okonkwo *et al.*, 2012). Since the dominating compound in eggshell is calcium carbonate, during incineration to ash the calcium carbonate will decompose into calcium oxide and carbon dioxide.

$$CaCO_3(\text{egg shell powder}) \xrightarrow{\text{heated at } 650\pm25°C \text{ for 3 hours}} CaO + CO_2 \tag{11}$$

Eggshell ash (ESA) has not been fully explored despite sporadic studies exist highlighting its utilization in civil engineering applications as a stabilizing material, and it could be a good replacement for industrial lime since its chemical composition is similar to that of quick lime. The eggshell waste was washed to remove impurities and allowed to dry before grinding. Then dried eggshell was milled to have powder passed sieve size of 2 mm, which is going to calcine in a muffle furnace at a heating temperature of 650 ± 25 ˚C for 3 hours. The specific gravity of eggshell ash is 2.546.

Table 2-12: Chemical composition of the eggshell ash (Tchuente *et al*.,2019)

Composition	Proportion
Calcium oxide (CaO)	96.36%
Magnesium oxide (MgO)	0.96%
Silicon dioxide (SiO_2)	0.37%
Alumina trioxide (Al_2O_3)	0.24%
Ferric trioxide (Fe_2O_3)	0.11%
Sulfur trioxide (SO_3)	0.33%
Phosphorus pentoxide (P_2O_5)	0.49%

2.10.3. Properties of crushed stone powder

Crushed stone dust is fine material obtained as solid wastes during the crushing of stones to obtain aggregates. They exhibit high shear strength, which is highly beneficial for their use as a geotechnical material. It has good permeability and variation in water content does not seriously affect its strength properties. They are also proved to be a promising substitute of sand and can be used to improve the engineering properties of soils. It was sieved through a sieve size 0.425 mm and then stored for further investigations. Its specific gravity is 2.776.

Table 2-13: Chemical composition of crushed stone powder (Blayi et al., 2021)

Composition	Proportion	Composition	Proportion
Calcium oxide (CaO)	3.84%	Titanium dioxide (TiO_2)	0.40%
Magnesium oxide (MgO)	0.81%	Potassium oxide (K_2O)	6.13%
Silicon dioxide (SiO_2)	70.02%	Sodium oxide (Na_2O)	3.65%
Alumina trioxide (Al_2O_3)	12.43%	Manganese oxide(MnO)	0.06%
Ferric trioxide (Fe_2O_3)	2.82%	Phosphorus pentoxide (P_2O_5)	0.10%

2.10.4. Previous studies related to stabilization of soil using eggshell ash

Now days, due to increasing costs of traditional soil stabilizing agents and the needs for economical use of agricultural, domestic and industrial wastes gets valuable acceptance for its workability and applicability. Eggshell ash has almost the same chemical content with quick lime and available with the least cost than such traditional stabilizer. And also, as reviewed from several studies, using eggshell ash as admixture is more economical and environment-friendly than lime stabilization. Several researches using eggshell as soil stabilizer are listed below.

Anoop *et al.,* (2017) studied the properties of soil stabilized by using lime and eggshell powder. Tests were conducted to assess the potential of eggshell powder in replacing lime, which can make the overall stabilization process economical and eco-friendly. Results obtained show that all the treated mixes gave much better strength than untreated soil. Eggshell powder was introduced in quantities of 0.5%, 1%, 1.5% and 2% of dry weight of soil. Tests were conducted replacing up to 50% of the lime used for stabilization and observed that 25% replacement of lime by eggshell powder gave better strength properties and can be adopted for practical purposes.

Muthu & Tamilarasan, (2014) had studied effect of eggshell powder on index and engineering properties of soil mixed with dosage of 0.5% to 5% by weight at 0.5% increment interval. Initially disturbed soil for study purpose was collected from specified study area and characterized for its bare properties before mixing with additives. After that effect eggshell powder on its Atterberg limit, compaction, and UCS (for immediate and three days delay) was investigated. Based on the investigation result, material added had significant effect on natural soil property by reducing liquid limit from 45% to 36%, increasing plastic limit, OMC and MDD from 26.6% to 31.8%, 14% to 16%, and 1.66g/cm^3 to 1.76g/cm^3 respectively. And also stabilizer added increase compressive strength of soil from 100.694kPa to 250kPa for immediate condition and 150kPa to 300kPa for three days curing condition which shows delayed compaction effect leads to increase in unconfined compressive strength of soil when compared to the without delay in compaction. Finally, 3% of ESP was proposed as the optimum dosage of material gives better strength results.

Amu *et al.,* (2005) studied the effect of eggshell powder on the stabilizing potential of lime on an expansive clay soil. They conducted a series of tests to determine both the optimum quantity of lime and lime-eggshell powder combination. The optimal quantity of lime was gradually replaced with a suitable amount of eggshell powder. Results of the maximum dry density, California bearing ratio, Unconfined compression test and undrained tri-axial shear strength test all indicated that lime stabilization at 7% is better than the combination of 4% ESP + 3% lime.

Okonkwo *et al.,* (2012) had a study aimed at determining the effects of eggshell ash on strength properties of cement stabilized lateritic soil. All proportions of cement and eggshell ash contents were measured in percentage by weight of the dry soil. The compaction test, California bearing ratio, unconfined compressive strength and durability test were carried out on the soil-cement-eggshell ash mixture. Increase in eggshell ash content was increased optimum moisture content but reduce the maximum dry density of the soil cement eggshell ash mixtures. Also, the increase in eggshell ash content considerably increases the strength properties of soil-cement-eggshell ash mixture up to 35%.

2.10.5. Previous studies related to stabilization of soil using crushed stone powder

Blayi et al., (2021) studied results of some index and engineering properties of expansive soil mixed with the rock powder, including Atterberg limit, compaction, free swelling, compressive strength, CBR, direct shear, and permeability at different percentages (0%, 8%, 16%, 24%, 32% and 40% by dry weight of soil). Results obtained reveals that the index properties like liquid limit, plastic limit, and linear shrinkage, optimum moisture content, free swelling, cohesion, and coefficient of permeability decrease with the addition of rock powder up to 40%. Whereas, the maximum dry density, UCS, and CBR increase up to 24% of rock powder thereafter reversed. Furthermore, the internal friction angle was increased by 95.32%, while the sub-base thickness layer for a roadway could be reduced by 40% with the addition of 40% rock powder to the soil sample. Based on the test results, they concluded that the optimum ratio of rock powder for the improvement of expansive soil is about 24%.

Agarwal, (2015) carried out tests such as compaction, specific gravity and CBR in the laboratory on expansive clays mixed with different proportions (10%, 20%, 30%, 40%, and 50%) of stone dust by dry weight of soil. Then from the test results, addition of 30% stone dust to black cotton soil decreases its OMC, increases MDD, specific gravity and CBR value increased nearly by 50% which is found to be optimum.

Venkateswarlu *et al.*, (2015) studied the behavior of expansive soil treated with quarry dust mixed with a proportion of 5%, 10%, and 15% to check its effect on bare soil's plasticity property, compaction characteristics, CBR, and strength parameter. Finally, they concluded that as all liquid limit, plastic limit, OMC, and cohesion property of soil decreased through the addition of quarry dust by up to 15%. But MDD and CBR under soaked condition was increased for mix proportion up to 10%. Generally results were found that up to the addition of 10% of stone dust there is an increase in strength parameters, but beyond that it is not effective.

Jemal *et al*., (2019) studied utilization of crushed stone dust added at of 5% to 50% through increment of 5% by weight as ground improvement for soil used as subgrade in highway construction. Throughout the study they conduct tests includes Atterberg limit, free swell index, compaction, CBR, and CBR swell test to analyze the effect of inert material on soil sample. Based on these, they were identified that liquid limit, plastic limit, MDD, and OMC decreases from 80.08% to 31.75%, 35.27% to 17.32%, 1.323g/cm^3 to 1.735g/cm^3, and 30.91% to 18.16% respectively with addition of 50% stone dust. And also free swell index was decreased from 90% to 11% with addition of 20% stone dust. Additionally, they investigate that CBR was increased from 1.817% to 5.583% with incorporation of 35% of crushed stone dust and then decreased to 4.167% when admixture amount further increases up to 50%.CBR swell also decreased from 3.181% to 1.195% with addition of 50% stabilizers. Finally, authors concluded that incorporating this material while highway construction will save 5% of total cost per 1 km length of road segment.

2.10.6. Previous studies related to stabilization of soil using both ESA and CSP

Birundha *et al.*, (2017) studied the combined effect of eggshell powder and quarry dust on clayey soil. Quarry dust has been introduced in proportion of 10%, 20%, 30%, and 40% and eggshell powder has been added in proportion of 20% (constant). Then laboratory tests are conducted to determine their effect on geotechnical properties of clayey soil. By adding eggshell powder and quarry dust on soil, its engineering property was improved. Investigation result of this study reflects that the combination of quarry dust and eggshell powder is more effective than the addition of quarry dust or eggshell powder alone for the improvement of the properties of clay soil.

Singh & Arora, (2019) study the strength improvement of silty soil having low plasticity by using eggshell powder and quarry dust through a mix proportion of 2% to 10% by weight with 2% increment interval for both materials. During the finding, property of natural soil collected from the study area was determined before the strength improvement inquiry. Then soil was mixed with predefined eggshell powder amounts to get the optimum dosage which was going to be mixed with quarry dust and determine the combined effect of both additives on both compaction and UCS (cured and uncured condition) of natural soil. Based on this with addition of eggshell powder, maximum dry density of natural soil decreased from 2.007g/cm^3 to 1.928g/cm^3 and OMC increased from 12.43% to 15.12% due to less specific gravity and high water absorption property of admixture. UCS value of soil-eggshell powder mix increased significantly with increase in percentage eggshell powder due to same chemical composition as lime and optimum percentage of eggshell powder was observed to be 6%. Again an increase in maximum dry density (1.976g/cm^3 to 2.278g/cm^3) and decrease in optimum moisture content (12.02% to 8.34%) was observed with increase in dosage of quarry dust with optimum eggshell powder content. Compressive strength of soil mixed with optimum eggshell powder and varying quarry dust was increased by a significant amount due to quarry dust have high shear strength and cementing property. Curing of the sample also reflected a positive effect on the strength characteristics of soil.

Bapiraju & Prasad, (2019) studied the effect of both eggshell powder and quarry dust with the optimum proportion of eggshell powder on properties of black cotton soil. And from result obtained, the liquid limit reduced steadily up to 4% and then the rate of decrease is falling. But plastic limit value increases with the addition of eggshell powder. The effect of eggshell powder on compaction curves is to increase the maximum dry density up to a certain percentage and then follow a recession. The rock dust with a combination of optimum eggshell powder is proving to be more effective with an increase in dry density and a decrease in optimum moisture content values. Rock dust with a greater specific weight is contributing to higher densities. Also, the optimum eggshell powder content was participating in CEC, satisfying the negative charge of the interacting soil may facilitate the easy movement of the particles in the densification process resulting in higher density. The expansion ratio decreases from a value of 7 to 1.17 which is a significant decrease in expansion potential. CBR value increases steadily with the addition of quarry dust in combination with optimum (4%) eggshell powder.

Paul *et al.*, (2014) investigated the effect of eggshell powder and quarry dust on improvement of clayey soil. Through this study effect of mixing virgin soil with eggshell powder of dosage (1%, 3%, 5%, 10%, 15%, 20%, 25%, and 30%) alone and combination of 20% eggshell powder with (10%, 20%, and 30%) of quarry on properties of natural soil were studied. So from result obtained mixing eggshell powder on weak soil was reduced liquid limit, plastic limit, plastic index, maximum dry density, cohesion, and coefficient of compressibility of soil. But it was to increase optimum moisture content, angle of internal friction, shear strength, permeability, and coefficient of consolidation of natural soils. Again combination of optimum (20%) eggshell powder with varied quarry dust gives the same property changing scenario even the best result was obtained except MDD was increased and OMC was decreased due considering the replacement of clay with material having higher specific gravity and less affinity to water.

2.10.7. Summary of related works and general research gaps identified

Table 2-14: Summary of previous related literatures

Authors	Additive	Dosage	Soil type	Major finding
Anoop *et al.*, (2017)	ESP and L	4% L with 12.5%, 25%, 37.5% & 50% ESP replacement	Soft clayey	25% replacement of lime by eggshell powder gave better strength properties.
Muthu & Tamilarasan, (2014)	ESP	(0.5% - 5%) of ESP at 0.5% interval	Brown clay	LL decrease, PL, OMC, MDD & UCS increase and 3% ESP is optimum amount.
Amu *et al*., (2005)	ESP and L	(1% - 6%) at 1% interval for both ESP and L	Dark brown clay	Lime stabilization at 7% gives better result of UCS and shear strength parameters than combination of 4% ESP + 3% lime.
Okonkwo *et al*., (2012)	C and ESA	(6% & 8%) C and (0% to 10%)ESA at 2% intervals	clayey sand	Increase in ESA dosage was increase the strength & OMC and decrease MDD of the material matrix up to about 35% ESA.
Blayi *et al*., (2021)	RP	(0% - 40%) of RP at 8% interval	High plasticity silt	LL, PL, LS, OMC, swelling index, cohesion and permeability of sample were reduced for RP up to 40% and increase in MDD, UCS, CBR and ϕ of soil was obtained for RP up to 24%. 24% of RP is proposed as optimum ratio.
Agarwal, (2015)	SD	(0% - 50%) of SD at 10% interval	Silty soil	Decreases OMC and increases MDD, specific gravity, increase

				CBR by 50% with adding 30% of SD which is optimum.
Venkateswarlu *et al*., (2015)	QD	(5%, 10% & 15%) of QD	High plasticity clay	Found that up to addition of 15% of QD there was decrease in LL, PL, OMC & cohesion and increase in CBR and MDD for addition of 10% QD which is optimum.
Jemal *et al*., (2019)	CSD	(5% - 50%) of CSD at 5% interval	Highly plastic clay	Incorporating stone dust to soil was reduced its plasticity, OMC, swelling index, CBR swell and increase its MDD & CBR & save 5% of total cost while highway construction.
Birundha *et al*., (2017)	ESP & QD	20% ESP and (10%, 20%, 30% &40%) of QD	Clayey soil	Combination of QD and ESP is more effective than the addition of QD or ESP alone for the improvement of the properties of clay soil.
Singh & Arora, (2019)	ESP & QD	(2% -10%) of both ESP & QD at 2% interval	Low plasticity silt	Silt soil mixed with combined ESP and QD have high strength and compressibility characteristics
Bapiraju & Prasad, (2019)	ESP & RD	(0.5% - 5%) of ESP at 0.5% interval and (5% - 30%) of RD at 5% interval	Black cotton soil	Using ESP alone reduces LL and OMC and increases PL and MDD. But the combination of both ESP and RD increases MDD and CBR and reduces OMC and expansion ratio.
Paul *et al*., (2014)	ESP & QD	(1%, 3%, 5%, 10%, 15%, 20%, 25% & 30%) of ESP and (10%, 20%, and 30%) of QD	Clayey soil	Mixing ESP on clay soil was reduced LL, PL, PI, MDD, cohesion and Cc of soil. But it was increase OMC, ϕ, shear strength, K and C_V of natural soil. Further addition of QD was improved soil better.

From related reviewed literatures, the following conclusion and gaps were identified. Expansive soils have been a serious issue that reported all over the world, and major areas of Ethiopia are also covered by these soils. The overall geotechnical properties of expansive soil can be improved by the addition of different industrial and domestic wastes like eggshell ash and stone powders. The previous investigations conducted on these areas are mainly focused on only single admixture effect to improve the properties of expansive soils. That means limited studies have been done on the combination of both additives for improvement of expansive soils. The study on the combined effect of ESA and CSP on the geotechnical properties of expansive soil in Ethiopia is limited in the literature. And also, the properties and mineral composition of materials used in the study was not well-defined and less or no characterization was available. Investigation of additives effect on natural soil was done only by considering the effect on Atterberg limit, compaction, UCS, and swelling property value but the effect of the ESA on the consolidation properties was not studied. Therefore, it is necessary to conduct an experimental study to investigate the effects of eggshell ash and stone powder on the strength and consolidation properties of expansive soils to mitigate the problems that raised from expansive soils.

CHAPTER THREE

MATERIALS AND METHOD

3.1. Description of Study Area

The study was conducted in Dukem town which is one of the recently established towns located at about 37 km, South East of Addis Ababa along the main road to Adama. The geographical location of the town is found approximately in between 8°45'25"N to 8°50'30"N latitude and 38°51'55"E to 38°56'5"E longitude. The area has a temperature ranges from 7℃ to 26℃ with total average annual rainfall of 898 mm and altitude depth of 1950 m above mean sea level. The area is found near the western border and experiences the temperate climate of the central rift valley of Ethiopia. And also it is located in the main Ethiopian rift valley near the western escarpment which is covered by volcanic rock materials with various compositions. Due to weathering of these rocks and corresponding erosion and deposition, thick residual clay and silty clay soil covered most of the plain topographic landforms.

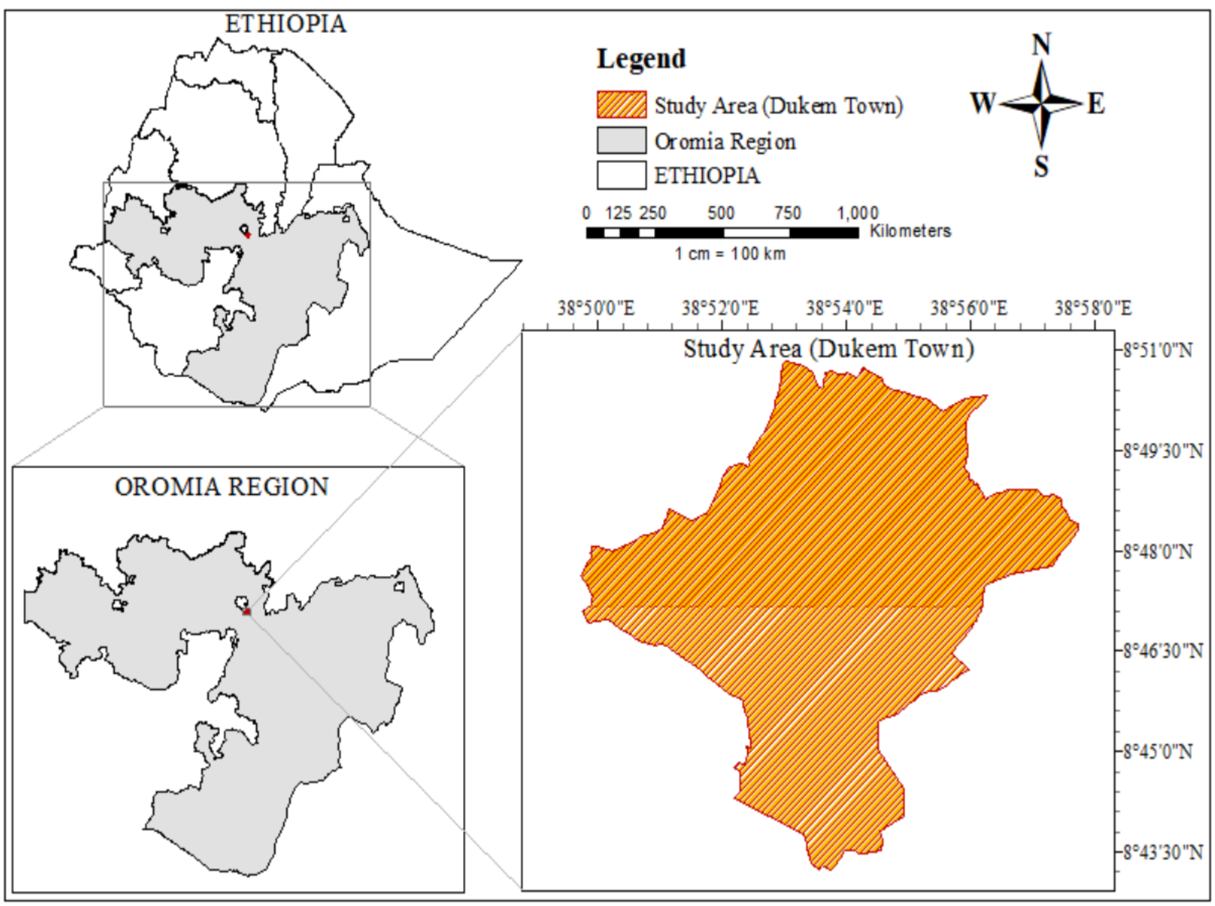


Figure 3-1: Map of study area

3.2. Weather and climate of study area

According to Berhanu *et al.*, (2014), climate in Ethiopia is geographically quite diverse which most commonly follows varied topography and categorized in to five different sub zones based on altitude, rainfall and temperature variation. It ranges from the high cold area named as "wurch" to the highly hot climatic condition area known as "Berha" having different physical measurement characteristics such as altitude, mean annual precipitation and temperature as shown in table 3-1.

Table 3-1: Climate zones of Ethiopia and their physical characteristics (Berhanu *et al.*, 2014)

Zones	Altitude (m)	Mean annual rainfall (mm)	Mean annual temperature (°C)
Wurich (cold to moist)	>3200	900-2200	Below 11.5
Dega (cool to humid)	2300-3200	900-1200	11.5-17.5
Weynadega (cool sub-humid)	1500-2300	800-1200	17.5-20
Kola (warm semi arid)	500-1500	200-800	20-27.5
Berha (hot arid)	< 500	Below 200	Above 27.5

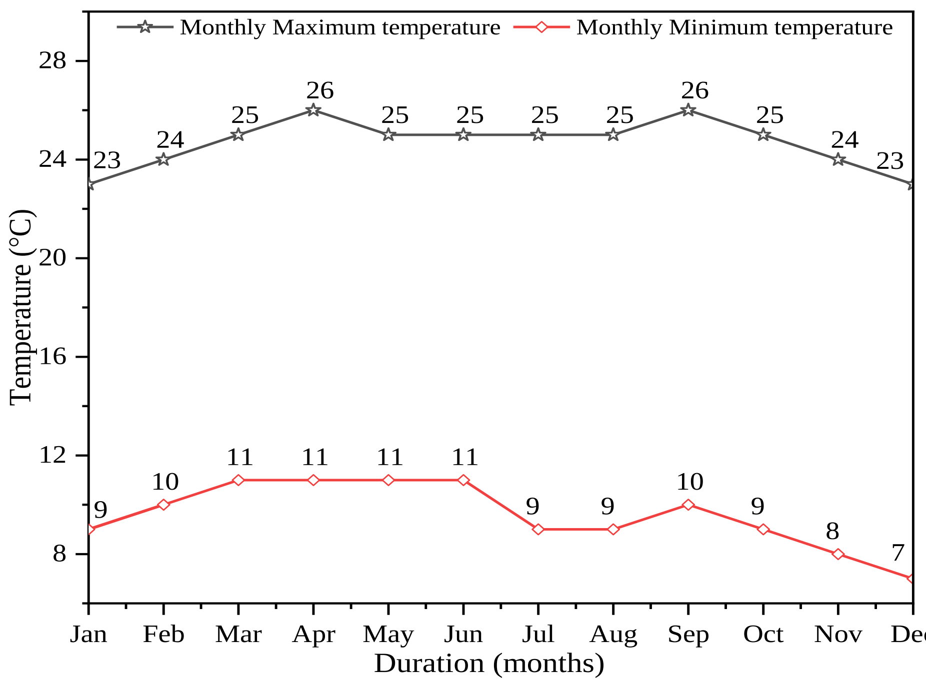


Figure 3-2: Monthly mean minimum and maximum temperature of Dukem town (Source: ENMA)

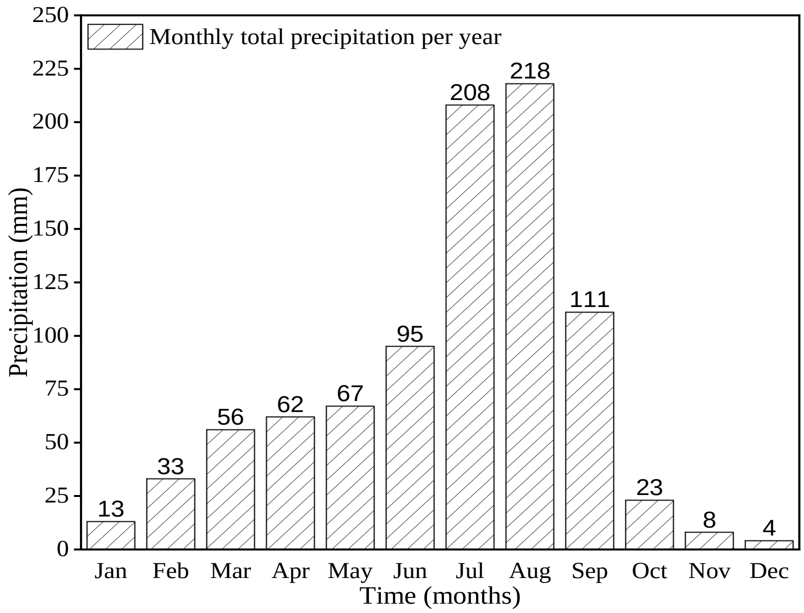


Figure 3-3: Monthly mean rainfall of Dukem town (Source: ENMA)

For the case, the study area is located on altitude of 1950 m which is between 1500 m & 2300 m and categorized under Weynadega (cool sub-humid) climate zone. Based on ENMA data, mean monthly temperature of the study area is around 17.8 °C, which is within range (17.5 mm-20 mm) of specified climate zone. And also maximum monthly mean rainfall is recorded in the month of August with rainfall of 218 mm and the month having the least mean rainfall of 4 mm is December. Mean annual rainfall of the study area is 898 mm, which is in the range of 800 mm-1200 mm.

3.3. Materials

For the accomplishment of this research work, there were three materials used for investigation which includes natural soil, eggshell ash and crushed stone powder.

3.3.1. Natural soil

Disturbed expansive soil samples were collected from Dukem town through digging of test pits to depth of 2 m from the natural ground surface. Then soil samples were packed in to plastic bags having specified identity of borehole number and transported to geotechnical laboratory for further investigation. And also, soil samples for natural moisture content determination from each of the test pits was collected and covered with plastic bags to maintain its moisture. As per Tamrat, (2013) studied properties of soils in this area and around 80% of Dukem town soil is characterized and classified as fat clay (CH) having black color and around 50% of the existing structures constructed on black clay soil show the appearance of cracking. Depending up on this previous site characterization, borehole was selected to take samples from three sampling sites having geographical locations shown on table 3-2. Laboratory tests such as Atterberg limits, linear shrinkage, differential free swell test, particle size distribution analysis, standard compaction, specific gravity, and compressive strength tests were performed on dry disturbed soil samples to determine the properties of soil in the study area and to select the weakest soil based on its strength and swelling capacity for further stabilization work.

The soil sample collected was air dried and sieved with sieve size of 4.75 mm as required for laboratory test. Then if undisturbed sample will be needed in the case, disturbed soil having known weight was remolded at minimum of 95% of maximum dry density with optimum moisture content by using appropriate molds under control of certain standard.

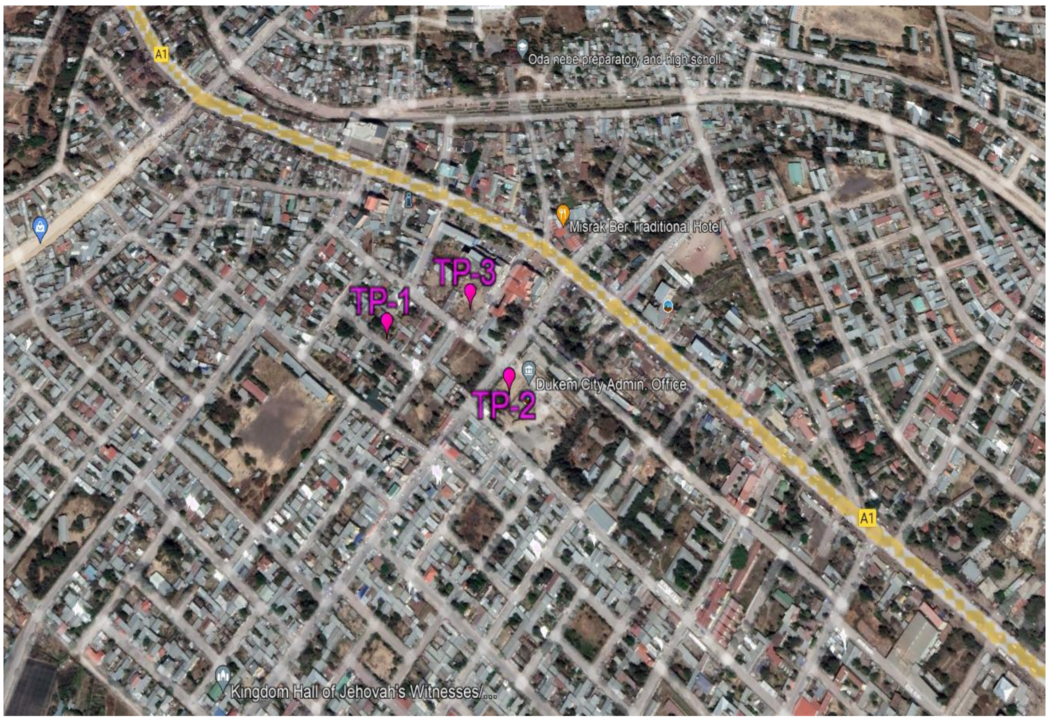


Figure 3-4: Geographical location of the test pits

Table 3-2: Geographical locations of sampling sites

Sampling site	Latitude (Northing)	Longitude (Easting)
TP-1	8°47'29.64"	38°54'9.42"
TP-2	8°47'27.78"	38°54'14.04"
TP-3	8°47'30.06"	38°54'12.48"

3.3.2. Eggshell ash

Eggshell used for this study was obtained and collected from daily usage of eggs in domestic, fast food joints and different local poultry. Then, the collected shell was cleaned for its impurities and milled to make it ready for burning operation. Burning operation called calcination process was done under controlled temperature of 650 $\pm$ 25 °C for three hours by using a muffle furnace. Finally, the ash produced after calcination was again milled and sieved to be used in material passing 150 µm for the study.

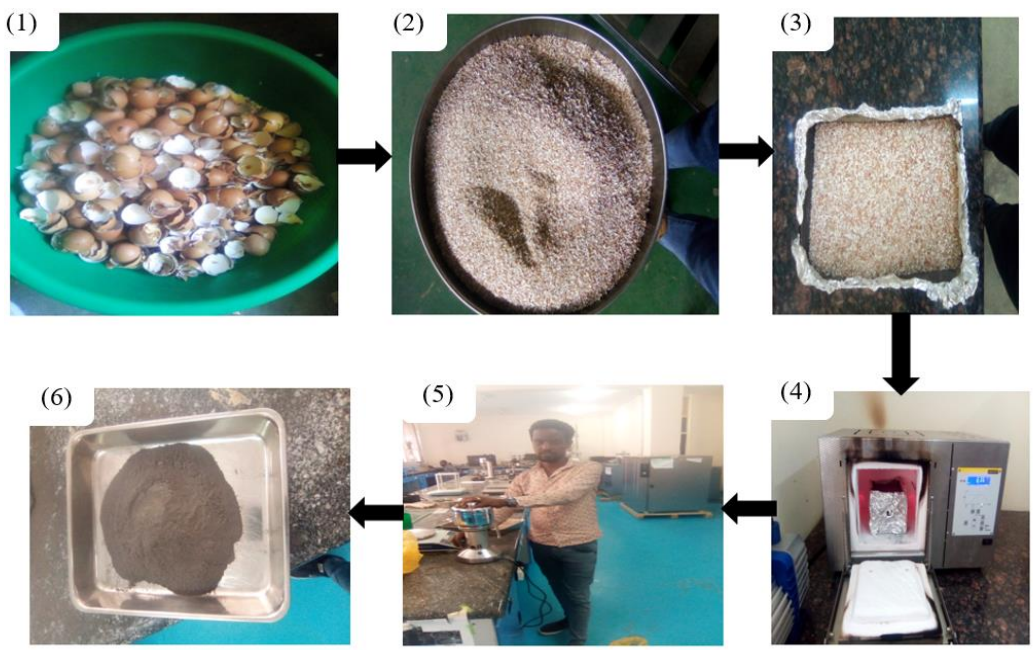


Figure 3-5: ESA preparation process (1) raw eggshell (2) ESP (3) prepared ESP for burning (4) Calcination process (5) milling calcined ESA (6) sample used for the study

3.3.3. Crushed stone powder

Crushed stone powder, was collected from local crusher sites found near Adama town which was described by geographical location of 8°32'49.8"N latitude and 39°18'12.54"E longitude and sample collected was prepared for tests as per standard required. Based on that, the collected sample was dried and sieved to use particles finer than 425 µm sieve.

Figure 3-6: Images of (a) test pit and (b) materials used in the study

3.4. Sample Preparation and Mix Design

Soil samples collected from all three test pits were dried separately and the larger particles of soil mass were crushed to the maximum particle size of 4.75 mm and prepared for the laboratory tests. Sample drying was done either by oven dry machine or open air drying system. Eggshell ash was prepared through processes includes collecting, removing its impurities, drying & changing into particle finer through 2 mm sieve, calcination or burning using muffle furnace at 650 $\pm$ 25 °C for 3 hours, milling & sieving through 150 μm sieve. Then ESA passes through 150 microns was mixed with soil at varying percentages of 2%, 4%, 6% and 8% and conduct a series of laboratory tests to decide the optimum amount of ESA needed. Again stone powder from a local crusher site was collected, air-dried and sieved through 425 μm sieve to be mixed with soil and optimum amount of ESA at varying dosages of 5%, 10%, 15% and 20%.

Table 3-3: Mix design and material proportion

Sample	Natural soil (%)	ESA (%)	CSP (%)
Soil	100	0	0
Expansive soil + ESA	98	2	0
	96	4	0
	94	6	0
	92	8	0
Expansive soil + CSP	89	Optimum amount of ESA was used throughout this stage	5
	84		10
	79		15
	74		20

3.5. General Flow Chart of the Study

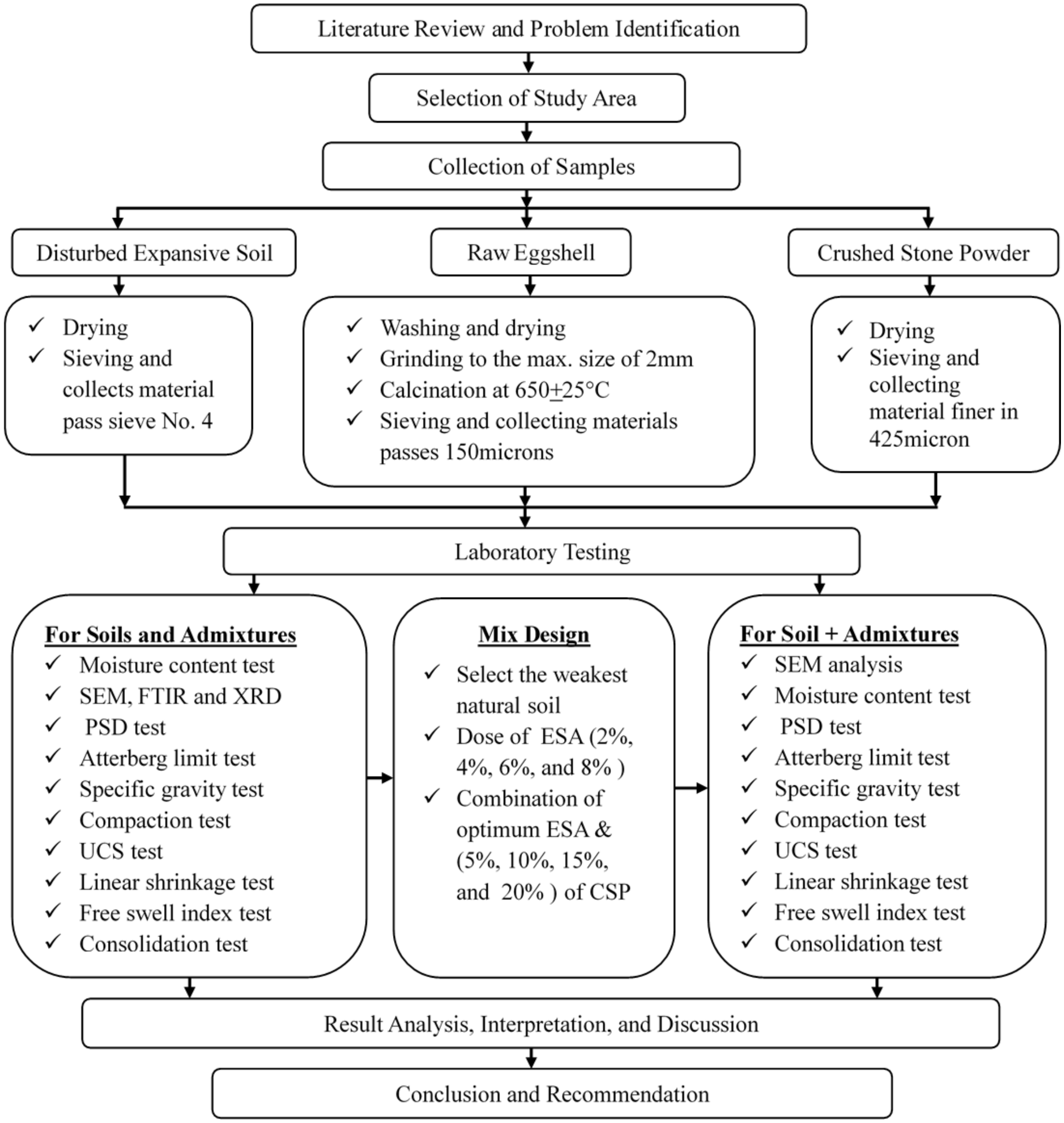


Figure 3-7: Summarized methodology chart of the study

3.6. Laboratory Test

After all materials required for the study were prepared, different laboratory experiments performed to investigate effect of both additives on geotechnical properties of natural soil. Laboratory tests conducted for this study were listed with their standard test method of operation in table 3-4.

Table 3-4: Laboratory tests conducted on materials

Test conducted	Standard code used
Natural moisture content test	ASTM D 2216
Specific gravity test	ASTM D 854
Grain size analysis test	ASTM D 6913
Free swell index test	IS 2720-40
Atterberg limit test	ASTM D 4318
Linear shrinkage test	BS 1377-2
Standard compaction test	ASTM D 698
UCS test	ASTM D 2166
Consolidation test	ASTM D 2435

3.6.1. Natural moisture content

The natural moisture content will give an idea of the state of soil in the field, and also it shows the depth of groundwater table. The natural water content also called the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage. Tools used for this test were oven dry machine, digital balance having accuracy of 0.01g, moisture cans, gloves and spatula. This test was conducted under control of ASTM D 2216 standard test methods for laboratory determination of water content of the soil.

3.6.2. Specific gravity test

The specific gravity of a material is the ratio of the mass of a unit volume of soil solids at a specific temperature to the mass of an equal volume of gas free distilled water at the same temperature. This test method is applied to the soil fraction that passes through sieve size of 4.75 mm and in case it was used to determine phase relationship of solids such as the degree of saturation, void ratio and other soil properties. However, the test is used to determine the specific gravity of soil by using the density of water and usually reported at 20 °C temperature i.e. if testing temperature is different from room temperature (20 °C), result obtained is going to be corrected for temperature effect. To determine specific gravity of a single material, at least two trial tests were needed to take the average of the two result and report it as specific gravity. For the case of this study, about 10gm of dry soil sample finer than sieve No.4 was used to determine specific gravity of each material and calculated by the following formulas.

$$G_T = \frac{M_s}{M_s + (M_a - M_b)} * \rho_{w(T)} \tag{12}$$

Where G_T is specific gravity of soil at testing temperature T(°C), M_s is mass of dry soil in gram, M_b is mass of pycnometer containing soil and water, M_a is mass of pycnometer filled with water at temperature T(°C) which is calculated from:

$$M_a = \frac{\rho_{w(T)}}{\rho_{w(T')}} * \left(M_a' - M_p\right) + M_p \qquad (13)$$

Where $\rho_{w(T)}$ is density of water at temperature T(℃), $\rho_{w(T')}$ is density of water at temperature T'(℃), M_a' is weight of pycnometer filled with water in grams, M_p is weight of empty pycnometer in grams.

Finally, specific gravity calculated at temperature T(℃) was corrected and reported for temperature effect by multiplying it with some factor k as shown in equation 15.

$$G_{20} = k * G_T \qquad (14)$$

3.6.3. Grain size analysis test

Grain size analysis is a typical laboratory test conducted in the soil mechanics field, which used to determine the particle size distribution of soils. The analysis is conducted via two techniques which are through sieve analysis which is capable of determining the particle size ranging from 0.075 mm to 100 mm, whereas distribution of particles smaller than 0.075 mm can be determined using the hydrometer analysis method.

i) Sieve analysis: is a method that is used to determining the grain size distribution of soils that are greater than 0.075 mm in diameter. It is usually performed for sand and gravel, but cannot be used as the method for determining the grain size distribution of finer soil. The sieves used in this method are made of woven wires with square openings. Data obtained from sieve analysis may also be useful in developing relationships concerning the porosity and packing in the soil mass. Information obtained from the particle size analysis, uniformity coefficient (Cu), coefficient of curvature (Cc), and effective size (D_{10}), are also used to classify the soil and determine whether the soil is suitable for construction purpose. And also this test result gives clue about soil water movement if permeability test is not available. The standard test method used for this test was ASTM D 6913.

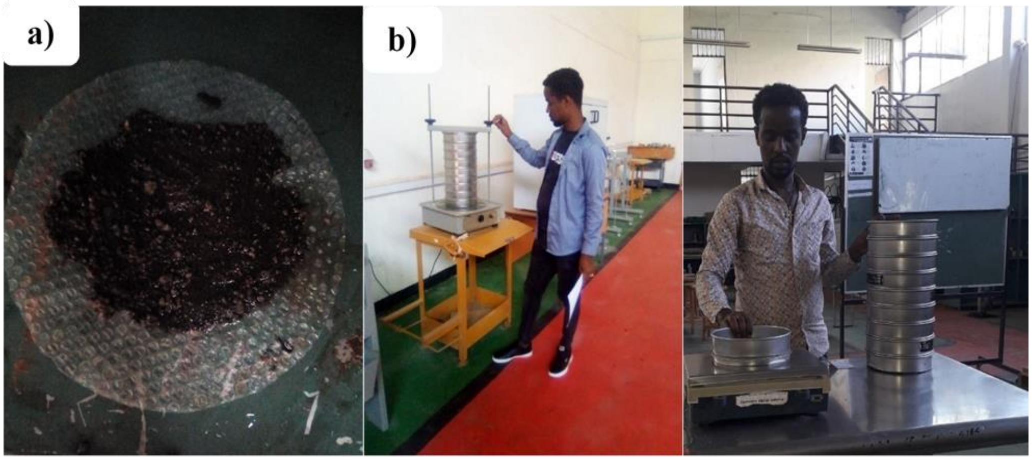


Figure 3-8: Gradation test (a) wet sample retained on sieve No.200 after washing (b) shaking sieve stack containing sample (b) measuring mass of material retained on each sieve

ii) Hydrometer analysis: The hydrometer analysis of soil is based on Stokes law to calculate the size of soil particles from the speed at which they settle out of suspension in the water. For this study around 50g of dry soil passing 75 µm sieve was taken and mixed with distilled water and dispersing agent like sodium hexa-metaphosphate, $(NaPO3)6$ to form a gently mixed slurry. Then the prepared slurry was backfilled to a cylinder having 1000ml mark capacity and 152H hydrometer was immersed into the cylinder to record a reading as per ASTM D 7928 standard method. Then from recorded data percent finer and respective particle diameter was determined which shows the grain size distribution for soils finer than No.200 sieve. However, when this result was combined with a sieve analysis, it can offer a complete gradation profile called particle size distribution curve of soils and addresses some interesting information such as C_u and C_C which are obtained by the following equations.

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10} * D_{60}} \tag{15}$$

Where: D_{10}, D_{30}, and D_{60} are the particle size that corresponds to the percent finer of 10%, 30%, and 60%, respectively.

3.6.4. Atterberg limit

Atterberg limit tests are an investigation conducted on fine-grained soils to determine the amount of water required to attain different behavioral states. This test is done to determine the liquid limit, plastic limit, and the plasticity index of the soil and soil-additives mixture which have significant role in soil classification system. The mass of dry sample passing through sieve

No.40 (425 μm) was used to perform the tests. The tests were done by following ASTM D 4318 standard method.

Liquid limit (LL): is arbitrarily defined as the water content, in percent, at which a paste of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. To conduct the test first, the soil or soil-admixtures sample was exactly mixed with water by repeatedly stirring, kneading, and chopping with the spatula and placed it in the Casagrande cup. The cup containing the sample was lifted and dropped at a rate of 2 drops/sec until the two sides of the sample come in contact at the bottom of the groove, along with a distance of about 13 mm. The number of blows required to close the grove to this distance was recorded, and its moisture content was determined. The relationship between the water content and the number blow is plotted on xy plane and the water content corresponds to 25 blows is called liquid limit of the materials.

Plastic limit (PL): is the water content at which soil change its state from a plastic to a semi-solid state. This test involves repeatedly rolling a soil or soil-additive mixtures paste sample on a smooth glass plate into a 3 mm thread until it reaches a point where it crumbles. Then, water content of rolled thread was determined and recorded as a plastic limit.

Plasticity index (PI): is the difference between the liquid limit and the plastic limit of soil and described as the range of moisture at which a material behaves in a plastic state, which used in classifying fine-grained soils.

3.6.5. Linear shrinkage test

Linear shrinkage is the percentage decrease in length of a wet soil sample filled in to the standard mold when it gets fully dried, starting with a moisture content of the sample from liquid limit. The test was conducted based on BS 1377-2 standard method. To perform the test, an oven dried sample of about 150g passing through a sieve size of 425 μm is needed. Then a sample having known mass was mixed with water to attain its liquid limit. Finally, mud prepared was filled and compacted into standard mold which have greased its inner faces and put it inside an oven dry machine to remove available moisture in the soil sample and determine the compressed length of dry sample. Then linear shrinkage was calculated by the following equation, shown for every sample separately.

$$\text{Linear shrinkage(LS)} = \frac{(L_O - L_D)}{L_O} * 100\% \qquad (16)$$

Where: L_O is original length of wet sample and, L_D is oven dried length of dry sample.

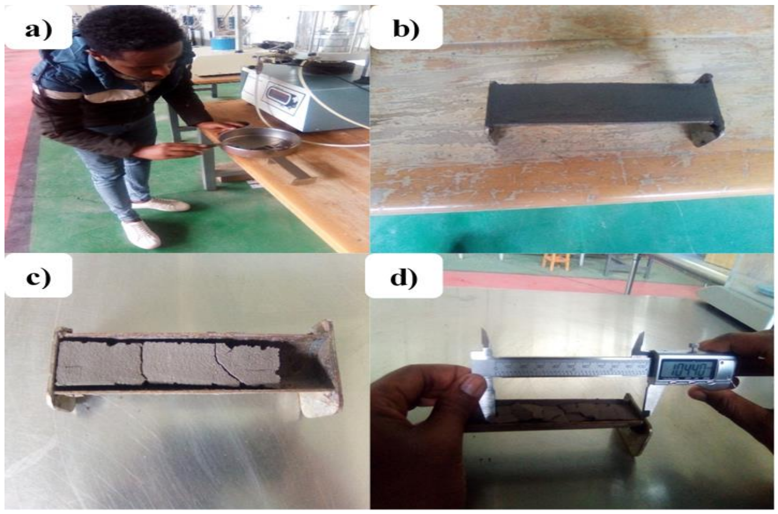


Figure 3-9: Linear shrinkage determination (a) filling wet sample in to the mold (b) sample prepared for drying (c) oven dried sample (d) measuring length of dry sample

3.6.6. Free swell index test

This is the simplest test conducted in the study to determine the swelling properties of the natural soil, additives alone, soil-ESA mixture and soil-optimum ESA-CSP mixture. The method was suggested by Holtz and Gibbs (1956) to measure the swelling potential of cohesive soils. Free swell index test was conducted following IS 2720-40, (1977) testing procedure. First two oven-dried soil or soil-additives mixtures samples passing through 425 µm sieve were taken and 10gm of each sample were placed separately in two 100ml graduated cylinder. The first cylinder is filled with kerosene and the other is filled with distilled water to the mark of 100ml and gently shaking or stirring the mixtures with a steel rod to remove entrapped air voids. Then allow the suspension to settle and attain the state of its equilibrium for a duration of not less than 24 hours. After that, final volumes of materials soaked were recorded and then FSI & FSR was calculated by using the following equations.

$$\text{FSI} = \frac{(\text{Vw} - \text{Vk})}{\text{Vk}} * 100\% \quad \& \quad \text{FSR} = \frac{\text{Vw}}{\text{Vk}} \qquad (17)$$

Where: V_w and V_k are volumes of soil specimen read from the graduated cylinder containing distilled water and kerosene, respectively.

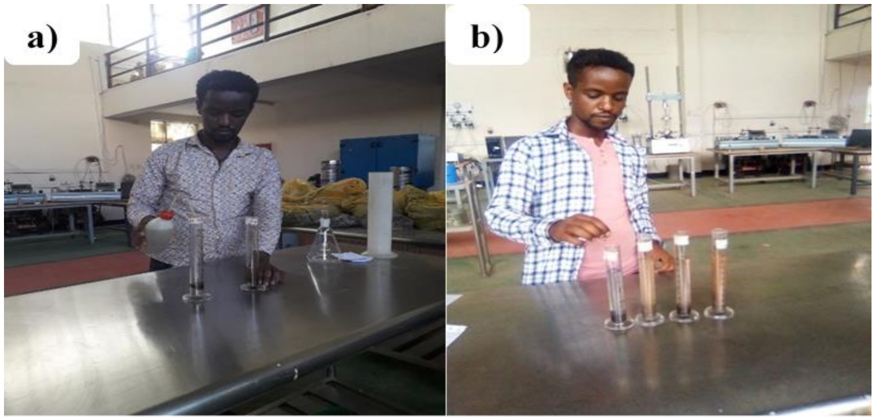


Figure 3-10: Differential free swell test (a) soaking of sample (b) stirring to remove air void

3.6.7. Standard compaction test

Laboratory standard compaction tests were carried out on bare soil, soil-ESA and soil-optimum ESA-CSP mixtures to determine optimum moisture content and maximum dry density of material samples. In this research, standard proctor compaction test was implemented by 600KN-m/m^3 standard effort in the mold having 4 inches (101.6 mm) diameter & internal effective height of 4.6 inches (117 mm) under which the material was compacted into three layers by applying 25 blows in each layer (ASTM D 698, 2003). Moisture was started from 8% (200ml) for the initial sample, and the wet density was measured in each step until it becomes maximum and then decreasing after increasing water content. After measuring wet mass for each soil sample and soil-additives mixtures for different water content, a portion of each sample from top, middle and bottom was taken to an oven dry machine to determine average moisture content that was available in the soil. Then the dry density of each sample at different moisture content was determined using the following equation.

$$\rho_{dry}(g/cm^3) = \frac{\rho_{wet}, g/cm^3}{1 + w} \tag{18}$$

Where: ρ_{dry} is dry density of sample, ρ_{wet} is wet (bulk) density of sample and w is average moisture content in decimal number.

Dry density obtained was limited to be under the linear portion called ZAV line which shows soil mass was below saturated condition and all void volume between particles was occupied by water, and it was determined by the following equation.

$$\gamma_{dry}(ZAV) = \frac{G_s \gamma_w}{1 + wG_s} \quad (19)$$

Where: $\gamma_{dry}(ZAV)$ is dry unit weight of saturated soil, γ_w is unit weight of water and w & G_s are respective average water content and specific gravity of samples.

Figure 3-11: Standard compaction test (a) prepared soil sample (b) dry mixing of soil sample and additive (c) compaction operation (d) compacted sample

3.6.8. Unconfined compressive strength test

According to ASTM D 2166, (2000) standard, unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of fine-grained soil will fail in a simple compression test. The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of clays under unconfined conditions, i.e. lateral pressure (σ_3) is zero. For soils, the undrained shear strength (Su) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (Su) of clays is commonly determined from an unconfined compression test, which is equal to half of unconfined compressive strength (q_u) when the soil's angle of internal friction (ϕ) is zero. The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion and expressed as:

$$S_u = C_u = \frac{q_u}{2} \tag{20}$$

For the case, the test is used to differentiate the weakest natural soil from all three different samples, the sample having the least UCS value is the weakest sample. And then selected the weakest soil sample was mixed with ESA at 2%, 4%, 6% and 8% proportion to be remolded with respective water content and density under both cured and uncured conditions to determine effect and amount of optimum ESA based on UCS criteria. Specimens having height of 76 mm and diameter of 38 mm, used for this test, were prepared through remolding of dry disturbed soil sample finer than 4.75 mm sieve using a mold having circular cross-section at predetermined moisture (OMC) and density (MDD). Then it was put and loaded under UCS machine and result was recorded & analyzed automatically via computerized system. Based on the result obtained, optimum percentage of ESA was selected to be combined with CSP at 5%, 10%, 15% and 20%. Finally, optimum amount of ESA and CSP was proposed based on how combined effect gives better UCS result.

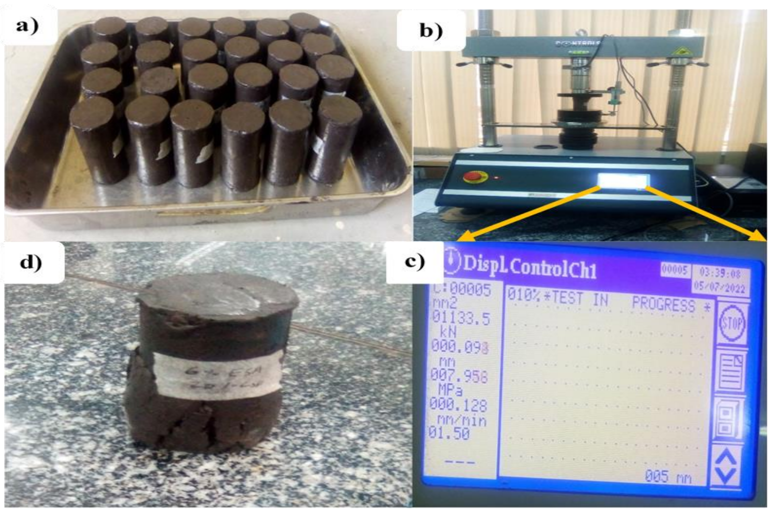


Figure 3-12: Images of (a) remolded specimens (b) UCS loading machine (c) digital screen of UCS machine (d) specimen after loading

3.6.9. Consolidation test

The consolidation test was conducted according to the procedure given by ASTM D 2435 standard method. The samples for the consolidation test were prepared by mixing the soil with the respective optimum moisture content that was obtained from standard compaction test and then compacted to attain MDD in standard odometer specimen ring having 60 mm diameter and 20 mm height. The samples were fixed in the consolidation cell with height of 20 mm and immersed in water to make it under fully saturation. Then consolidation test was conducted by applying incremental loads starting with 0.8 kg/cm^2 and ending with 6.4 kg/cm^2. At each loading stage, the deformation at 0.1, 0.25, 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, and 1440 minutes was recorded on an observation sheet for further analysis using Casagrande's logarithm of time fitting method. When the loading of 6.4 kg/cm^2 reached, unloading stages were start conducting by reducing constant load. This test was conducted for natural soil, soil mixed with 6% ESA, and soil-6% ESA mixed with 5% and 10% CSP to investigate effect on coefficient of consolidation, compressibility index, coefficient of the volume of compressibility, and initial void ratio of samples.

Figure 3-13: Photographic view of (a) specimen preparation (b) specimen mounted in consolidation cell (c) consolidation apparatus (d) data recording

3.6.10. X-ray diffraction

XRD is a technique used to know about the various component of minerals found in materials used for this study. For XRD testing, the fine powder of weakest soil, ESA and CSP was sieved through a sieve size of 75 µm and the portion of the powder passing this sieve was used for X-ray diffraction testing. X-ray diffraction pattern was recorded with X-ray diffractometer with CuKα radiation (λ= 1.5406Å) at diffraction angle 2θ ranged between 10° to 80° in steps of 2θ=0.02°. Finally, different phases of each material were obtained and identified by analyzing the peaks of XRD results patterns with the help of “X-Pert High Score” software tool and inter planar spacing (d) was calculated by using Bragg’s equation shown below.

$$d = \frac{n\lambda}{2\sin\theta} \tag{21}$$

Where: n is Bragg’s constant equal to 1, λ is wave length =1.5406Å, θ is Bragg’s angle in radian and d is interplanar spacing in Å.

3.6.11. Scanning electron microscope

Scanning electron microscope (SEM) is a powerful analytical technique to perform analysis on a wide range of materials, at high magnifications, and to produce high resolution images. Powder of material samples passes through sieve No. 200 was used for the study with resolution of 200 times larger than the real particle size, and it was done for soil, ESA, CSP, soil mixed with 6% ESA and soil mixed with 6% ESA & 15% CSP.

Figure 3-14: SEM testing machine setup

3.6.12. Fourier transform infrared spectroscopy

FTIR is a technique used to obtain an infrared spectrum of absorption or emission of solid materials used in the study, which simultaneously collects high resolution spectral data over a wide spectral range. This confers a significant advantage over a dispersive spectrometer, which measures intensity over a narrow range of wavelengths at a time. The term Fourier transform infrared spectroscopy originates from the fact that a Fourier transform (a mathematical process) is required to convert the raw data into the actual spectrum (Shameer & Nishath, 2019). The test is used to determine the functional group of minerals found in the materials.

CHAPTER FOUR
RESULT AND DISCUSSION

4.1. Overview

Laboratory experiments including natural moisture content, compaction, consistency limit, free swell index, particle size distribution and specific gravity were done on natural soil collected from all three test pits to classify and select the weakest soil for further studies. Therefore, based on its swelling potential and compressive strength, the soil sample collected from TP-3 was selected as the weakest soil needed to be studied for the effect of mixing with ESA and CSP. Based on these, ESA with the proportion of 2%, 4%, 6%, and 8% by dry weight of soil was mixed with natural soil to assess its effect and determine the optimum amount of ESA needed to be mixed with CSP to study their combined effect on natural soil's geotechnical properties. Finally, the combined effect of both additives was studied, and the final dosage that gives better result was proposed as the optimum proportion of admixture.

4.1.1. Moisture content of natural soil

Moisture content is the ratio expressed as a percent of the mass of pore or free water in a given mass of material to the mass of the solid soil. A standard temperature of 110°± 5 °C is used to determine mass of dry solid soil (ASTM D 2216, 1998).

Table 4-1: Natural moisture content of representative samples

Location	Depth of sampling	Moisture content (%)
TP-1	2 m	28.24
TP-2	2 m	29.45
TP-3	2 m	26.28

Natural moisture content determination of soil sample was used to know existing condition of material and depth of groundwater table at the sampling sites. Based on laboratory results, natural moisture contents are 28.24%, 29.45% and 26.28% for TP-1, TP-2 and TP-3 respectively. The result shows sample from TP-2 is the wettest soil due to there may be some moisture infiltration or shallow groundwater table depth and TP-3 material have the least moisture.

4.1.2. Specific gravity of natural soil and additives

The specific gravity of material used for the study were summarized as in table 4-2 which show natural soil sample from TP-1, TP-2 and TP-3 have G_s of 2.609, 2.618 2.606 respectively. ESA and CSP powder also have specific gravity of 2.546 and 2.776 respectively. The result shows CSP is a heavyweight material and ESA is the lightest material. Specific gravity determination of materials for this study follows ASTM D 854, (2002) standard method.

Table 4-2: Specific gravity of soil samples and additives

Location	Source of sampling	Specific gravity, G_s
TP-1	2 m	2.609
TP-2	2 m	2.618
TP-3	2 m	2.606
ESA	Local source	2.546
CSP	Local source	2.776

4.1.3. Particle size distribution analysis of natural soil and additives

Table 4-3: Summary of particle size analysis for soil and additives

Sample	Source	Combination of particles size (%)			
		≥ 4.75 mm (Gravel)	0.075 mm-4.75mm (Sand)	0.075 mm-0.002 mm (Silt)	≤ 0.002 mm (Clay)
TP-1	2 m	1.3	4.9	40.1	53.7
TP-2	2 m	2.2	4	40.2	53.6
TP-3	2 m	1.7	6.7	35.4	56.2
ESA	Local source	0	10	84.2	5.8
CSP	Crusher site	0	88.8	11.2	0

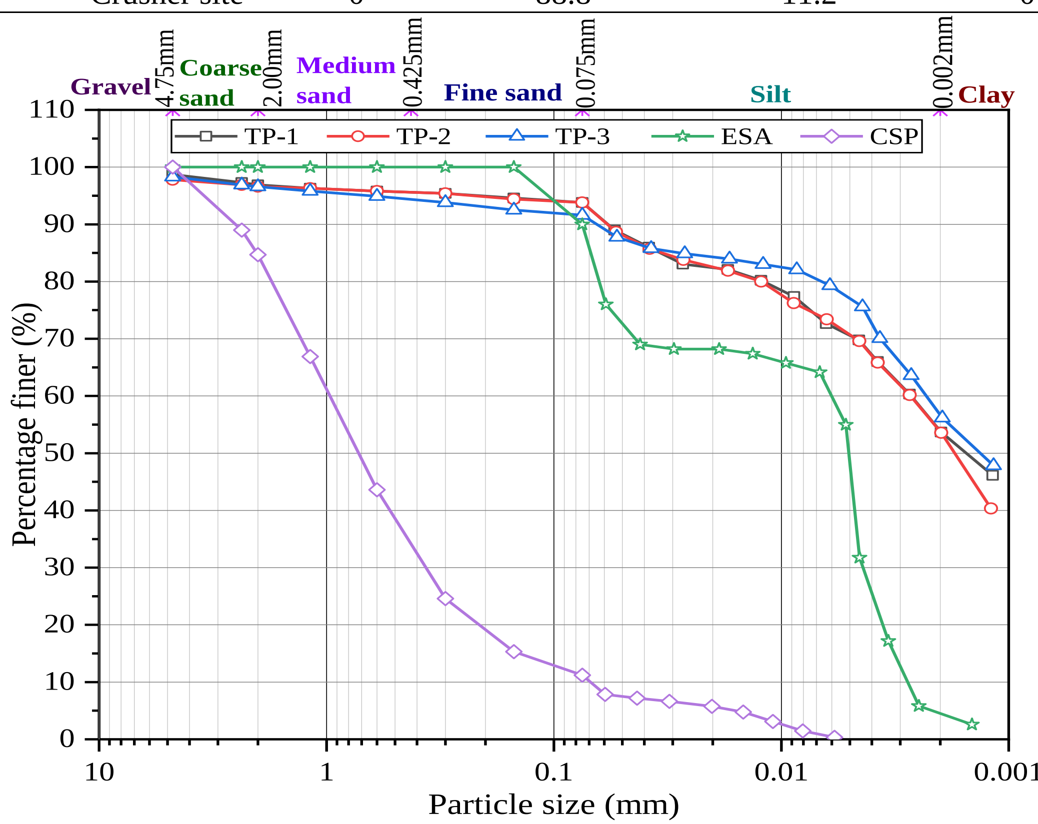


Figure 4-1: Particle size distribution curve of materials used in the study

Particle size analysis is the quantitative determination of the distribution of particle size in soils which is performed under the control of ASTM D 6913, (2004) for coarse-grained soils and ASTM D 7928, (2021) fine-grained soils. From table 4-3, for soil collected from TP-1, TP-2, TP-3 and ESA there were 93.8%, 93.8%, 91.8% and 90% of particles that are passes through sieve size 0.075 mm respectively, which are classified as the fine-grained materials. While 11.2% of CSP sample was passed through sieve No.200, and classified as coarse grained material contains particle size ranges from 0.006 mm to 4.75 mm. Referring plasticity chart under USCS, all soil samples have liquid limit greater than 50% and falls above A-line which categorized as CH, inorganic fat clay having high plasticity group. Again from plasticity chart under AASHTO classification system, all three soil samples fall under A-7-5 group having group index of 20. Crushed stone powder contains a coarser material as compared to soil sample and presents in the size ranging from 0.006 mm to 4.75 mm, from this 88.8% of particles are retained on sieve size of 0.075 mm, while 11.2% of CSP was passed through 0.075 mm. Based on the percent finer at 4.75 mm and 0.075 mm, Cu and Cc, the class of the CSP used in this study can be characterized as SW, well-graded sand as per USCS. But according to AASHTO classification system, CSP is categorized as A-3 (0), fine sand material due to its percent finer in sieve size 0.425 mm and 0.075 mm and non-plastic property. The ESA is a finer material exists in the size ranging from 0.001 mm to 150 μm with 90% of the particles passing the 0.075 mm sieve & 10% of it retained above 0.075 mm sieve and categorized as ML, silt with low plasticity by USCS and A-4 (0) as per AASHTO system.

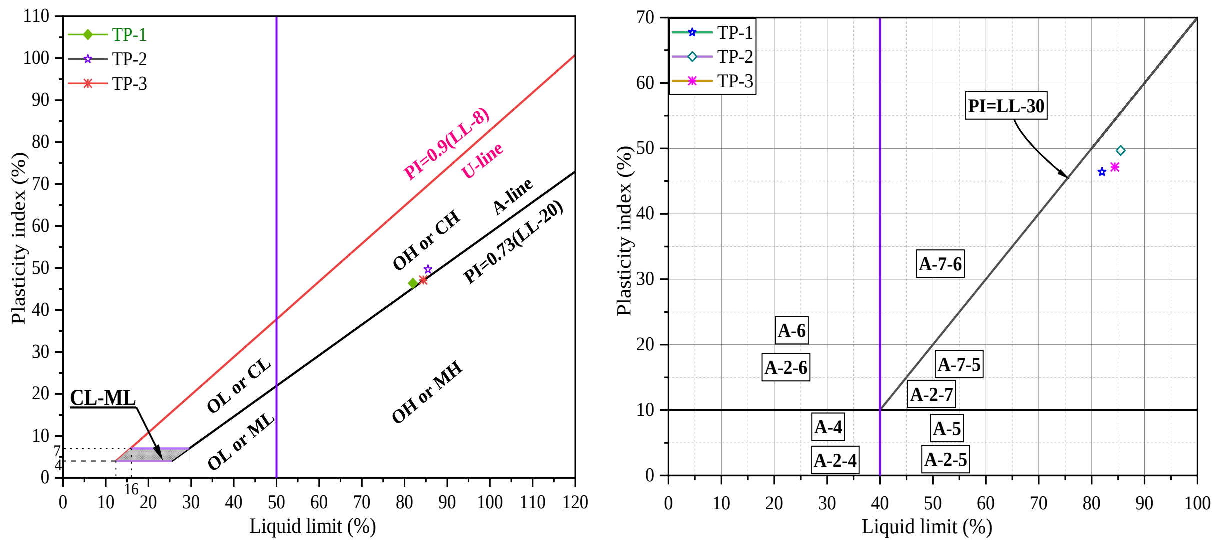
4.1.4. Atterberg limit of natural soil

Atterberg limit test results include liquid limit, plastic limit, plastic index and linear shrinkage result under which all are followed by ASTM D 4318, 2000 except BS 1377-2, (1990) was used to determine linear shrinkage property of soil.

Table 4-4: Atterberg limit test results of natural soils

Designation	Source	Atterberg limit results (%)			
		LL	PL	PI	LS
TP-1	2 m	81.96	35.56	46.4	20.97
TP-2	2 m	85.50	35.83	49.67	21.34
TP-3	2 m	84.38	37.22	47.16	21.96

From test results liquid limit, plastic limit, plasticity index and linear shrinkage of soil sample from TP-1, TP-2 and TP-3 are (81.96%, 35.56%, 46.40%, and 20.97%), (85.50%, 35.83%, 49.67%, and 21.34%), and (84.38%, 37.22%, 47.16%, 21.96%) respectively. Plastic limit and plasticity index result obtained are mainly used in soil classification system and shows soil grouped as CH and A-7-5 (20) as per USCS and AASHTO system respectively as shown on figure 4-2. The size of linear shrinkage is used to characterize soil and interpret cracking size & characteristics occurred due to seasonal moisture variation.



(a) USCS system **(b)** AASHTO system

Figure 4-2: Plasticity chart for natural soil classification system

4.1.5. Free swell index of natural soil

Free swell is the increase in volume of a soil, without any external constraints, due to submergence in water and used to identify the potential of a soil to swell which might need further detailed investigation regarding swelling and swelling pressures under different field conditions. The test was performed under control of IS 2720 part 40, (2002) standard method.

Table 4-5: FSI and FSR result of natural soils and additives

Designation	Source	FSI (%)	FSR
TP-1	2 m	81.00	1.81
TP-2	2 m	85.70	1.86
TP-3	2 m	95.00	1.95
CSP	crusher site	0.20	1.00
ESA	local source	0.50	1.01

The soil samples from TP-1, TP-2 and TP-3 have FSI of 81%, 85.7%, and 95% and the FSR of 1.81, 1.86, and 1.95 respectively which shows material from third test pit have more swelling potential when compared with others. ESA and CSP used for the study also have FSI and FSR of (0.2% & 1) and (0.5% & 1.01) respectively. These values indicate that the admixtures used in the study are almost non-swelling and better material to replace some portion of weak soil and improve its geotechnical properties.

4.1.6. Standard compaction test result of natural soil

Compaction of soils is a procedure in which soil sustains under mechanical stress is densified by reducing the volume of void in the mass. Soil mass consists of solid particles and voids filled with water or/and air. When soil is subjected to stress, soil particles are redistributed within the soil mass and the void volume decreases, resulting in densification. The test follows ASTM D 698, (2000) standard procedures and used to increase strength, decrease compressibility and permeability of soils. The result of compaction test of natural soil was summarized in the table 4.6 as shown below.

Table 4-6: Standard compaction test result of natural soil

Location	MDD(g/cm^3)	OMC (%)
TP-1	1.36	32.22
TP-2	1.34	33.61
TP-3	1.32	35.78

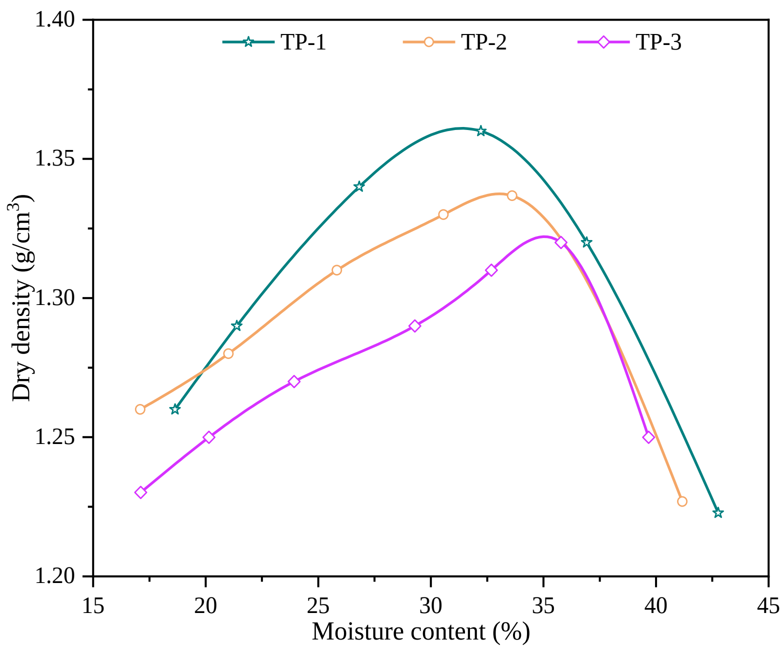


Figure 4-3: Compaction curves of natural soil collected from TP-1, TP-2 and TP-3

Results obtained reveals that natural soil collected from TP-1, TP-2 and TP-3 have optimum moisture content of 32.22%, 33.61% & 35.78% and maximum dry density of 1.36g/cm^3, 1.34g/cm^3, 1.32g/cm^3 respectively.

4.1.7. Unconfined compressive strength result of natural soil

Table 4-7: Unconfined compressive strength result of natural soil

Location	Depth of sampling	UCS (KN/m^2)
TP-1	2 m	136.45
TP-2	2 m	125.45
TP-3	2 m	94.74 (weakest)

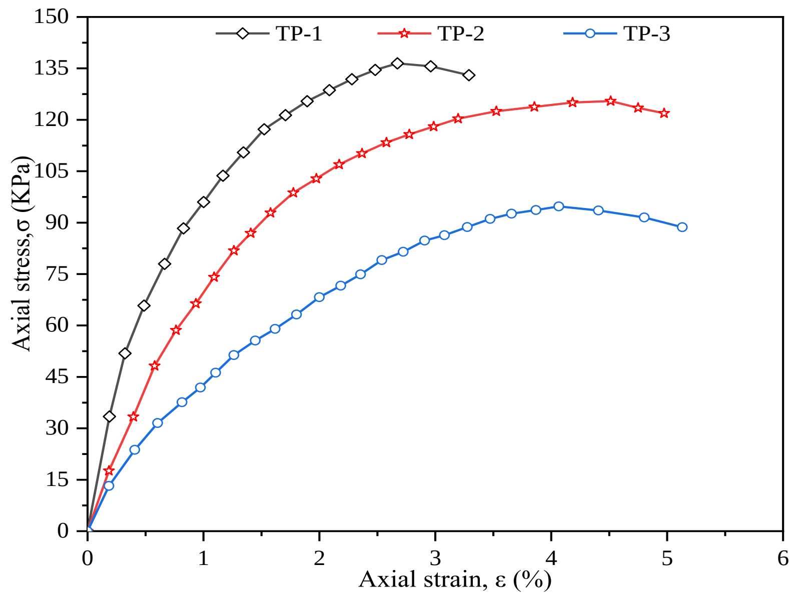


Figure 4-4: Unconfined compressive strength curve of natural soils

Samples of soil collected from all three test pits were also tested for their strength through unconfined compressive strength test following ASTM D 2166, (2000) standard method. Based on this, material collected from TP-1, TP-2 & TP-3 have compressive strength of 136.45Kpa, 125.45Kpa & 94.74Kpa respectively. In addition to swelling property, compressive strength result shows TP-3 material has less UCS value and selected as the weakest sample needed for further study by mixing with respective additives.

Table 4-8: Summary of geotechnical properties of natural soils

Properties of natural soil	TP-1	TP-2	TP-3
Natural moisture content	28.24%	29.45%	26.68%
Optimum moisture content	32.22%	33.61%	35.78%
Maximum dry density	$1.36 g/cm^3$	$1.34 g/cm^3$	$1.32 g/cm^3$
Liquid limit	81.96%	85.50%	84.38%
Plastic limit	37.56%	35.83%	37.22%
Plastic index	44.40%	49.67%	47.16%
Linear shrinkage	20.97%	21.34%	21.96%
Free swell index	81%	85.70%	95%

Free swell ratio	1.81	1.86	1.95
Percent finer of No.4 sieve	98.70%	97.80%	98.30%
Percent finer of No.200 sieve	93.80%	93.80%	91.60%
C_u & C_C	2.69 & 0.372	2.25 & 0.444	1.97 & 0.51
Soil class based on USCS	CH	CH	CH
AASHTO system	A-7-5 (20)	A-7-5 (20)	A-7-5 (20)
Specific gravity, G_s	2.609	2.618	2.606
UCS	136.45KPa	125.45KPa	94.74KPa
Color	Black	Black	Black

Table 4-9: Summary of properties of admixtures

Properties of admixtures	ESA	CSP
Free swell index	0.50%	0.20%
Free swell ratio	1.01	1
Liquid limit	18.95%	16.49
Plastic limit	16.03%	NA
Plastic index	2.92%	Non-plastic
Linear shrinkage	2.38%	1.55%
Percent finer of No.4 sieve	100%	100%
Percent finer of No.200 sieve	90%	11.20%
C_U & C_C	2.16 & 1.13	14.53 & 2.122
Classification based on USCS	ML	SW
AASHTO system	A-4 (0)	A-3 (0)
Specific gravity, G_s	2.546	2.776
Color	Dark gray	Dark silver

4.2. Mineralogical Composition and Microstructure Analysis of Materials

Based on the test results, soil sample collected from the third pit have less strength and more swelling property when compared to other natural soil samples collected from the same study area. Therefore, the effect of mixing this weakest soil with both additives in case of its geotechnical properties was investigated. And also characteristics of natural soil and both admixtures were assessed through qualitative tests like FTIR, XRD and SEM analysis.

4.2.1. FTIR result analysis of materials

Fourier transform infrared spectroscopy is a unique tool for the study of mineral and organic components of soil samples. FTIR spectroscopy offers sensitive characterization of minerals and soil organic matter, and mechanistic and kinetic aspects of mineral-SOM interactions that underlie biogeochemical processes. The molecular resolution of mineral and organic functional groups provided by FTIR has contributed significantly to understanding mineral and SOM structures, and ion and organic molecule sorption to mineral surfaces. The versatility of FTIR spectroscopy makes it a foundational tool for soil scientists, despite challenges in the acquisition and interpretation of soil spectra that stem from chemical heterogeneity (Margenot *et al*., 2017). The functional group of every analysis result in this study was assigned based on standard spectrometry table found in (Mohrig *et al*., 2010).

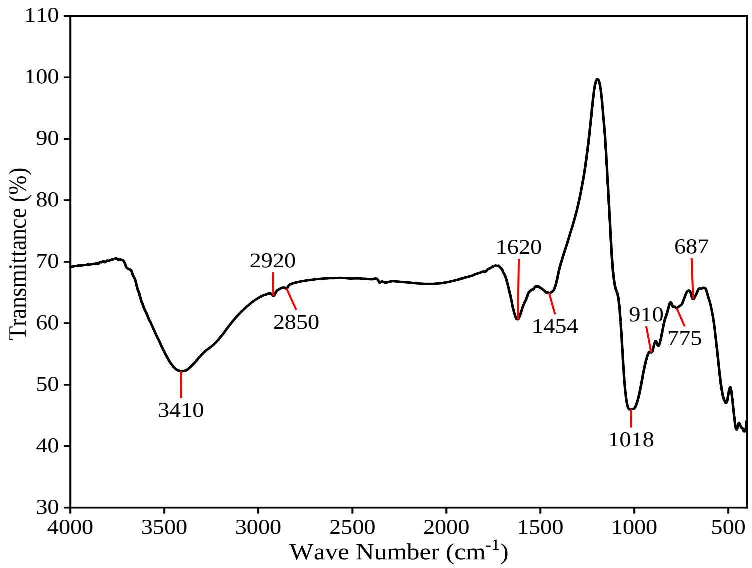


Figure 4-5: FTIR spectra of black cotton soil used for the study

From figure 4-5 the peak at 3410 cm^{-1} is indicative of free O-H (hydroxyl), broad strong hydrogen bonding, probably due to water vapor at the light chamber of the spectrophotometer and that present in the clay. The peaks at 2920 cm^{-1} and 2850 cm^{-1} are assigned to alkyl C-H stretching, which is an indication of organic impurity and agreed closely with the result that obtained by (Işçi *et al*.,2006). The peak at 1620 cm^{-1} was attributed to C=C medium stretching conjugated or cyclic alkene group. Again the peak at 1454 cm^{-1} was assigned to characteristic bands of alkane methyl group which is mostly related to bending vibration C-H functional group. And also the peak found at 1018 cm^{-1} is assigned to C-O strong stretching band. While peak at 910 cm^{-1} is assigned to C=CH2 Trans di substituted alkene group and peaks at 775 cm^{-}1 and 687 cm^{-1} was indicates presence of aromatic ring functional group in the soil sample.

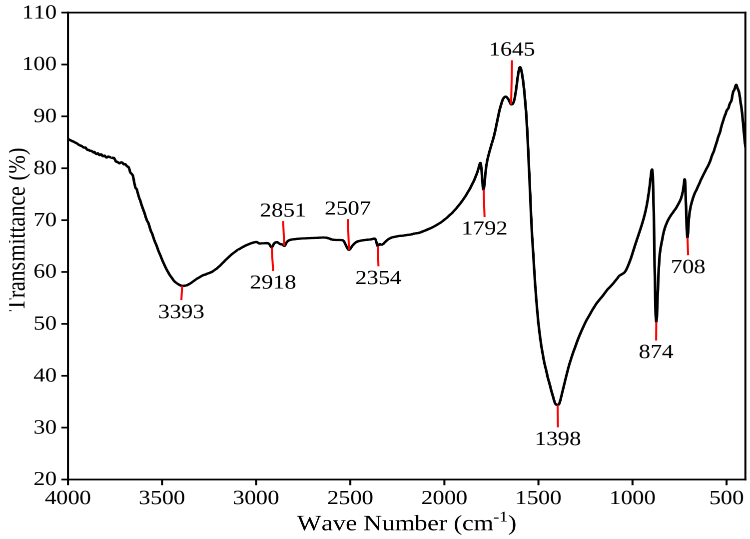


Figure 4-6: FTIR spectra of eggshell ash used in the study

The FTIR pattern of the natural chicken eggshell ash is shown in the figure 4-6 and analyzed as follows. The FTIR spectrum of the eggshell ash was assigned stretching vibration with medium intensity of the O-H or N-H group at, 3393 cm^{-1}. Again peaks at 2918 cm^{-1} & 2851 cm^{-1} were assigned with C-H medium to strong intensity of stretching alkanes group and peak at wave number of 2507 cm^{-1} corresponds to H-C=O:C-H carbonyl stretching of the aldehyde group. O=C=O & C=O stretching carbon dioxide & conjugated acid halide group were observed at 2354 cm^{-1} & 1792 cm^{-1} respectively. The vibration of the C=C stretching of alkenes was depicted at 1645 cm^{-1}. The peak at 1398 cm^{-1} assigned C-H bending vibration with medium to weak intensity of the alkane methyl group. And also peaks at 874 cm^{-1} & 708 cm^{-1} reveals C-H strong bonding of aromatic ring (benzene derivative) groups.

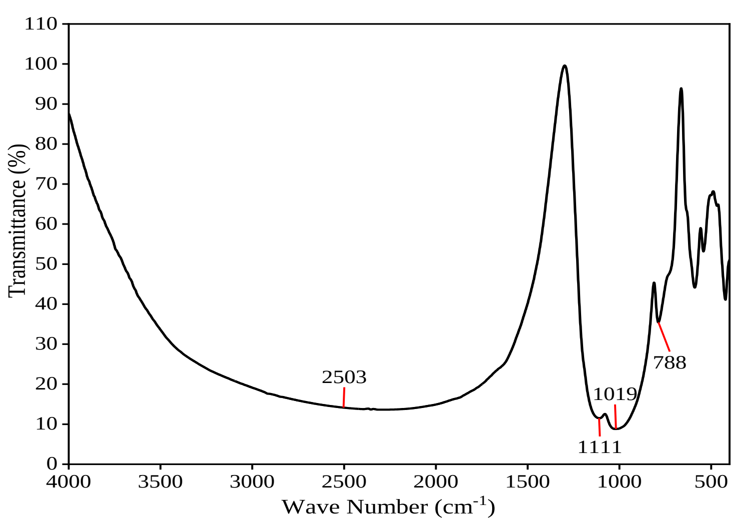


Figure 4-7: FTIR spectra of crushed stone powder used in the study

The FTIR spectrum of crushed stone powder in figure 4-7 shows the broad peak at 2503 cm^{-1} having medium to weak intensity of O-H stretching group for Al-OH and Si-OH. And also peaks observed at 1019 cm^{-1} & 1111 cm^{-1} were shows C-O strong stretching bond having aliphatic ether or secondary alcohol. Again the peak on wave number of 788 cm^{-1} shows C-H strong bending under 1, 3-di-substituted or 1, 2, 3- tri-substituted group. And also peaks found at region of 781 cm^{-1} to 436 cm^{-1} indicating the presence of quartz (El-Sherbiny *et al.*,2019).

4.2.2. XRD and SEM result analysis of materials

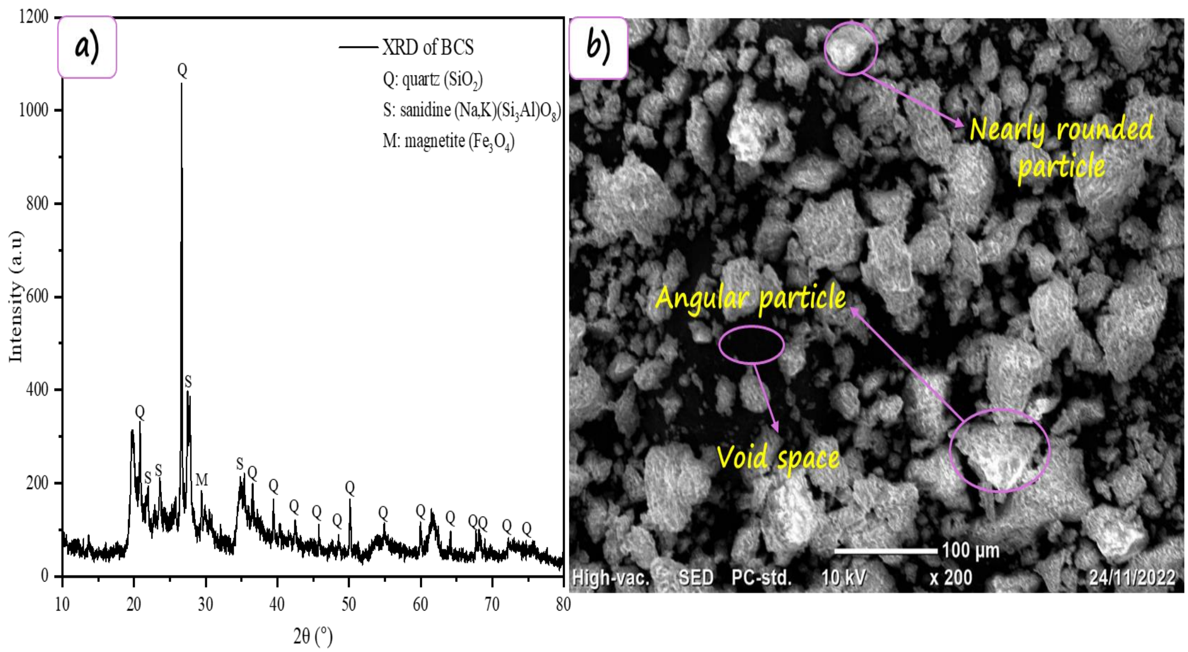


Figure 4-8: Image showing (a) XRD pattern result and (b) SEM photograph of natural soil

The XRD pattern of expansive soil on Figure 4-8, shows that soil sample mainly composed of quartz, sanidine (potassium or sodium feldspar), and magnetite minerals. The characteristic peak of quartz occurred at 2θ=26.62929° or d=3.344789806Å is present as a sharp peak that indicates the pronounced crystallinity of the SiO_2 phase close to the values obtained from the JCPDS (046-1045) standard data. The SEM photograph of natural soil shows the presence of a mixture of angular and nearly rounded particles, with some fines having some void space in the mass.

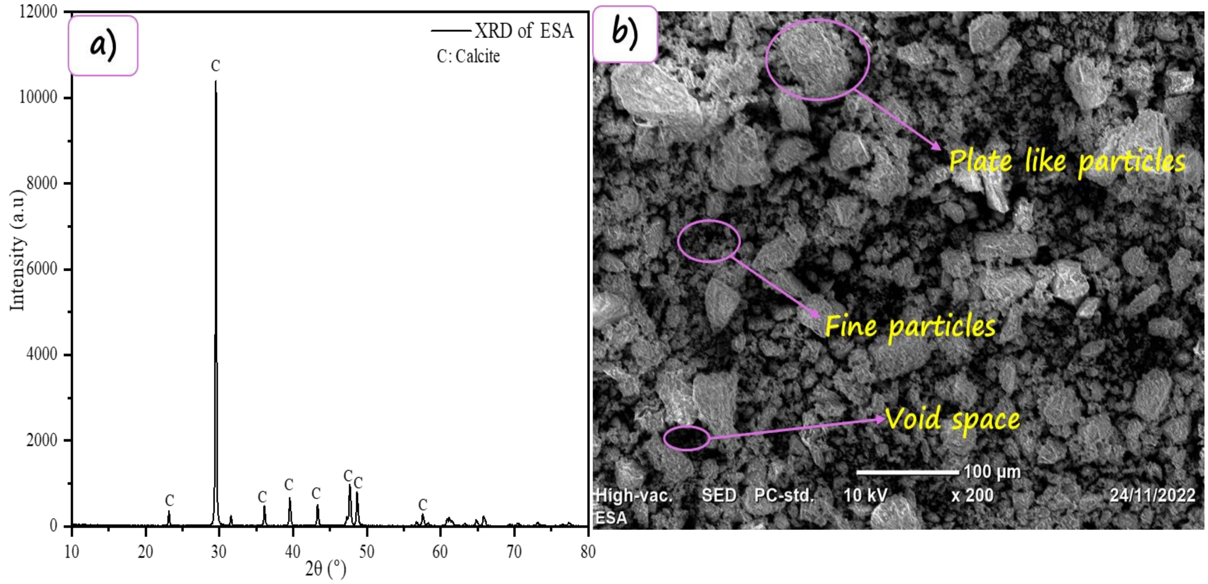


Figure 4-9: Image showing (a) XRD pattern result and (b) SEM photograph of ESA

The XRD pattern of the ESA, on Figure 4-9, shows that it composed of completely calcite minerals. The characteristic peak of calcite at 2θ=29.53535° or d= 3.021966351Å is present as a sharp peak that indicates the pronounced crystallinity of the calcite phase, close to the values obtained from the JCPDS (005-0586) standard data. As per Tchuente *et al*., (2019) the composition of ESA contains around 96.36% of calcite mineral. The SEM photograph of ESA exhibited the presence of a mixture of plate & fine particles with inter particle void space in the mass.

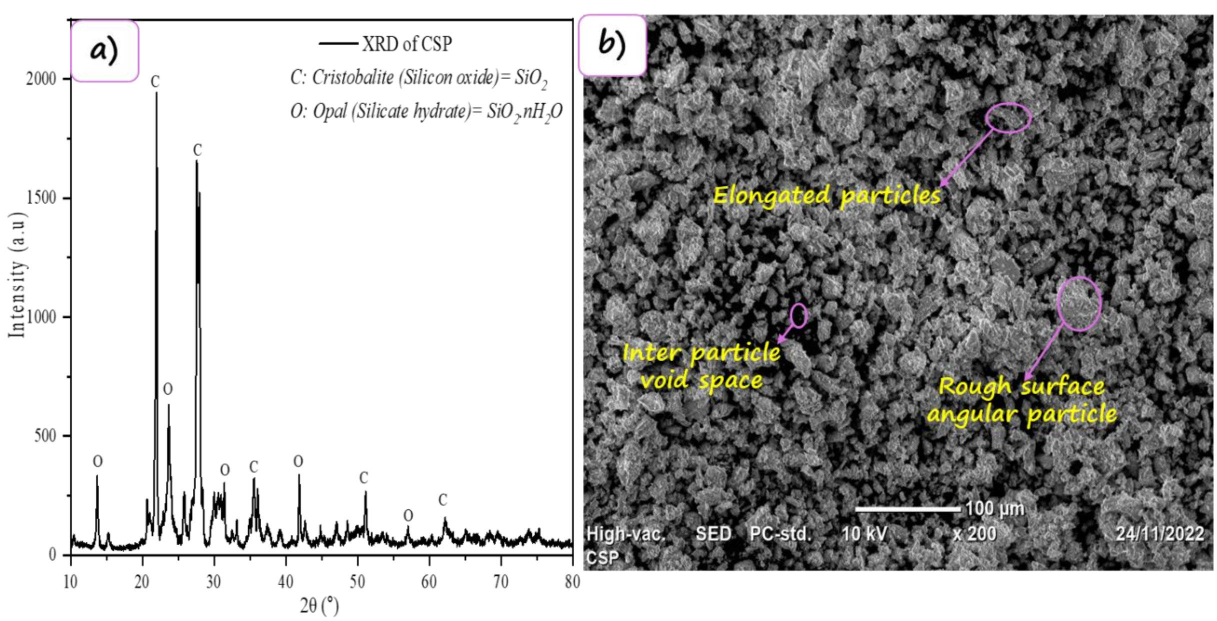


Figure 4-10: Image showing (a) XRD pattern result and (b) SEM photograph of CSP

The SEM photograph and XRD outline of CSP are shown on Figure 4-10. The XRD outline of CSP shows a peak intensity between 20° and 30°, represents the existence of Cristobalite (SiO_2). The characteristic peak of cristobalite at 2θ=21.93702° or d=4.048468941Å and 2θ=27.72983° or d=3.214491052Å is present as a sharp peak which indicates the pronounced crystallinity of the cristobalite phase close to the values obtained from the JCPDS (039-1425) standard data. And beyond that range there is occurrence of opal or silicate hydrate. As per Blayi *et al*., (2021) the composition of SiO_2 in CSP is 70.02% which is clearly justified in XRD result for its availability as quartz and silicate hydrate. The SEM photograph of CSP exhibited the presence of a mixture of elongated or rod like particles with rough surface and angular particles. The sum of Al_2O_3, SiO_2 and Fe_2O_3 in CSP is 85.27% satisfies the requirement of 70%, as per (ASTM C 618, 2000).

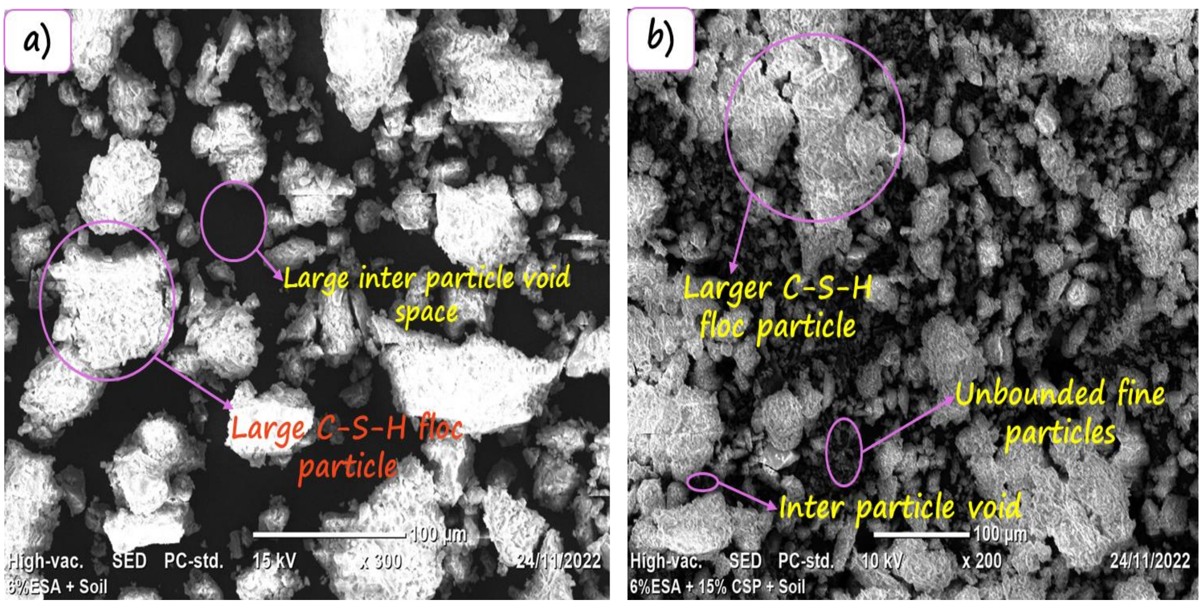


Figure 4-11: SEM photograph of (a) 6% ESA + soil and (b) 6% ESA + 15% CSP + soil

Figure 4-11 shows two different SEM image result of soil sample prepared by mixing with different amount and type of admixtures. The first image (a) shows a SEM image of soil sample mixed with 6% ESA. It was noticed that the sample contains coarser particles when compared to that of pure expansive soil. The reason of formation of coarser grains is due to particle floc formation resulted from pozzolanic chemical reaction occurred between cations raised from ESA and soil that facilitated due to water molecules in the mass. The second image (b) represents SEM of soil sample mixed with 6% ESA & 15% CSP, which shows mixture of coarser particles with finer particles. Coarser particles were formed in the same way as the first case process, and finer particles are probably existed from extra CSP particles.

4.3. Effect of ESA on geotechnical properties of natural soil

4.3.1. Effect of ESA on swelling index of natural soil

Table 4-10: Swelling index of soil mixed with different proportions of ESA

Dose of ESA	Free swell index (%)	Free swell ratio (FSR)
0% ESA	95.00	1.95
2% ESA	60.00	1.60
4% ESA	45.45	1.45
6% ESA	40.91	1.41
8% ESA	36.36	1.36

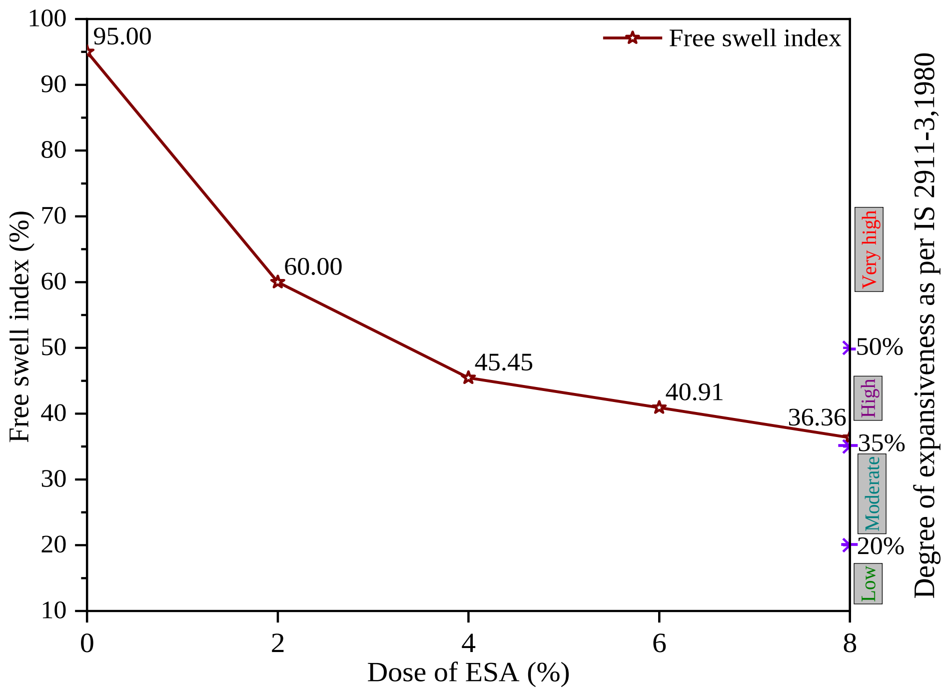


Figure 4-12: Variation in swelling index of soil with eggshell ash content

Free swell index of natural soil collected from TP-3 was observed as 95%. This makes the soil to fall under very high degree of expansion category as per the classification stated in (IS 2911, 1980). Whereas, with mixing of 2%, 4%, 6% and 8% of ESA to soil the swelling property of natural soil reduces to 60%, 45.45%, 40.91% and 36.36% respectively. Hence, after incorporating of 8% ESA, soil sample was improves its swelling potential and falls near to moderate degree of expansion. Therefore, eggshell ash is very effective and efficient admixture in reducing the swelling characteristics of expansive soils.

4.3.2. Effect of ESA on consistency limit of natural soil

Table 4-11: Consistency limit of soil mixed with different proportions of ESA

Dose of ESA	LL (%)	PL (%)	PI (%)	LS (%)
0% ESA	84.38	37.22	47.16	21.96
2% ESA	83.66	38.05	45.61	21.12
4% ESA	82.94	43.40	39.54	20.04
6% ESA	81.64	44.87	36.77	19.11
8% ESA	78.23	45.58	32.65	18.87

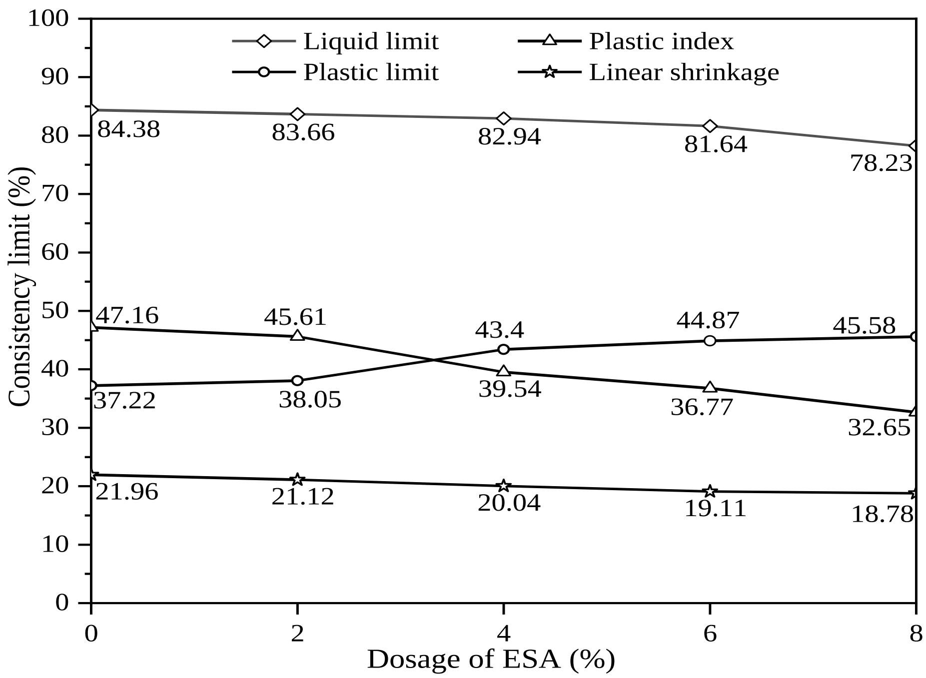


Figure 4-13: Variation of consistency limit of soil with ESA

Figure 4-13 shows the plasticity and shrinkage characteristics of soil stabilized soil with ESA. It is evident that the addition of ESA results in a decrease in the liquid limit, increase in plastic limit, and a consequent reduction in plasticity index of the natural soil. And also mixing of ESA to soil was caused reduction in linear shrinkage of material under study. The addition of ESA results in the decrease of liquid limit of soil from 84.38% to 78.23% for 8% ESA amendment. The plastic limit of the control specimen was increased from 37.22% to 45.58% for 8% ESA addition to the sample. The effect of these two properties, decrease in LL and increase in PL, can be seen as a reduction in the plasticity index of the soil. Plasticity index of soil reduces from 47.16% to 32.65% for 8% ESA incorporation. Decrease in liquid limit was caused by the free Ca^{2+} obtained from ESA and exchanges with the adsorbed cations of the clay mineral, resulting in reduction in size of the diffused water layer surrounding the clay particles. This reduction in the diffused water layer allows the clay particles to come into closer contact with one another, to form a cluster or floc of the clay particles. Thereby, ESA amendment leads to reduced plasticity of the stabilized soil (James *et al*., 2017). The PI recorded shows a decreasing trend along with the increased ESA content, which is attributable to the ability of ESA to clamp, flocculate and coarsening the soil particles.

4.3.3. Effect of ESA on compaction characteristics of natural soil

Table 4-12: MDD and OMC of soil mixed with different percentages of ESA

Dose of ESA	MDD (g/cm^3)	OMC (%)
0% ESA	1.320	35.78
2% ESA	1.310	36.06
4% ESA	1.300	37.21
6% ESA	1.296	37.79
8% ESA	1.290	36.32

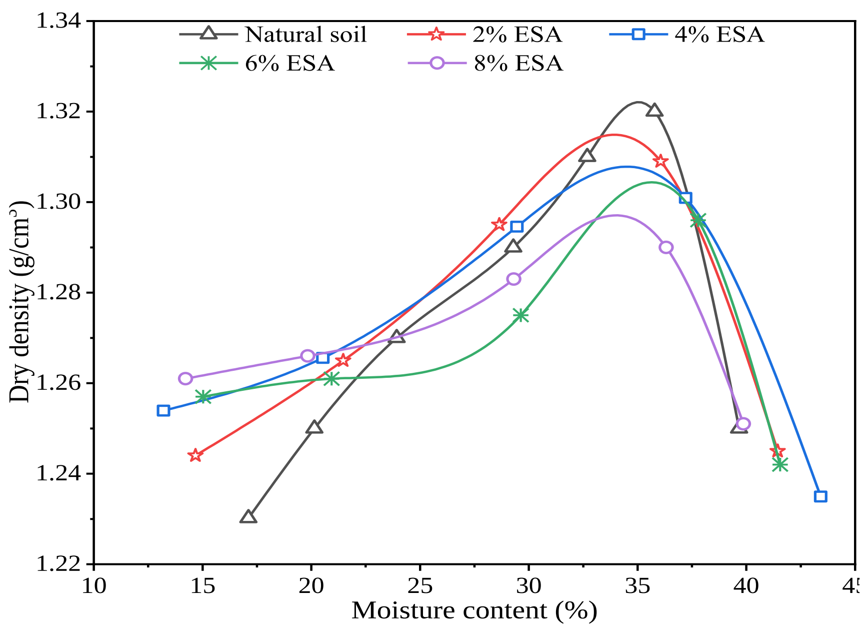


Figure 4-14: Compaction curve of soil mixed with different amount of ESA

Figure 4-14 shows the compaction curve of the soil sample mixed with ESA at 2%, 4%, 6%, and 8% by dry weight of soil. The compaction curve plotted illustrates that the particle arrangement of the sample changed with the addition of ESA having high CaO content, thus the specific surface area of the sample is altered, and more water is required for the chemical reaction among the fine particles, to set off the stabilization process. The MDD of the treated sample was initially decreased to 1.31 g/cm^3 from 1.32 g/cm^3 at 2% of ESA amendment and further decreased to 1.3 g/cm^3, 1.296 g/cm^3 and 1.29 g/cm^3 for 4%, 6% and 8% of ESA content respectively due to the replacement of soil by the ESA which has relatively lower specific gravity (G_s=2.546) compared to soil having G_s=2.606. The OMC of the treated sample increased to 36.32% from 35.78% with the incorporation of 8% ESA due to increase in calcium content in the sample and more water needed for the hydration process and formation of calcium silicate hydrate particles. And also OMC increase is due to the addition of ESA, which decreases the quantity of free silt and clay fraction that means coarser materials were formed and allow water to take place. This implies also that more water is needed in order to compact the soil-ESA mixtures.

4.3.4. Effect of ESA on compressive strength of natural soil

Table 4-13: UCS of soil sample mixed with different proportions of ESA

Dosage of additives	Unconfined compressive strength(KPa)	
	Uncured strength	Cured (7 days) strength
0% ESA	94.74	94.74
2% ESA	107.21	117.22
4% ESA	123.92	150.09
6% ESA	149.17	177.68
8% ESA	130.91	151.21

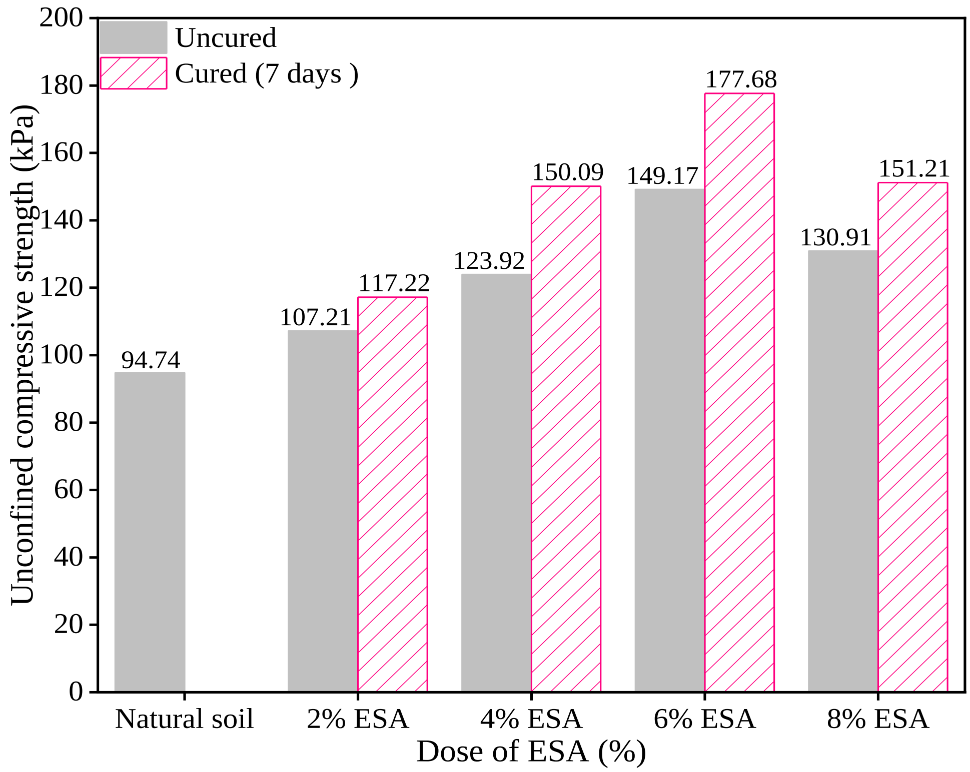


Figure 4-15: Variation of compressive strength of soil with the addition of ESA

Figure 4-15 illustrates the relationship between UCS value and the different percentage of ESA mixed with soil under both uncured and cured conditions. From this, the value of unconfined compressive strength result with and without delayed compaction, up to 6% addition of ESA was increased, and then sudden decrease was observed at 8% addition of ESA to the soil. It was noticed that the strength of the soil was increased approximately by average of 16.40% and 23.4% for both uncured and cured conditions for every addition of 2% ESA to the soil which shows cured condition gives better result than uncured method. Again, it was also inspected that strength of soil was decreased by 12.24% (for uncured) and 14.90% (cured condition) with further addition of 8% eggshell ash to soil sample. The initial increase in the UCS value was expected because of the gradual formation of cementitious compounds (C-S-H) due to the reaction between the calcium present in the ESA, silicon compound in soil and water. The decrease in the UCS values after the addition of 8% ESA was attributable to excess eggshell ash that occupies spaces within the soil mass to form weak bonds between the soils and the cementitious compounds formed by reaction, thus having a negative effect on the cohesive nature of the soil. Increase in strength at 7 days was attained due to there is more time for pozzolanic reaction between expansive soil and ESA minerals.

4.4. Effect of CSP on geotechnical properties soil mixed with 6% ESA

4.4.1. Effect of CSP on swelling property of soil mixed with 6% ESA

Table 4-14: Swelling index of soil mixed with 6% ESA & different proportions of CSP

Dose of additives	FSI (%)	FSR
6% ESA + 0% CSP	40.91	1.41
6% ESA + 5% CSP	33.33	1.33
6% ESA + 10% CSP	31.15	1.31
6% ESA + 15% CSP	28.57	1.29
6% ESA + 20% CSP	22.73	1.23

Figure 4-16 shows free swell index of natural soil mixed with 6% ESA was observed as 40.91%. This makes the soil mixed with optimum ESA fall into high degree of expansion category as per the classification stated on (IS 2911, 1980). Whereas, with incorporation of 5%, 10%, 15% and 20% of CSP to soil mixed with 6% ESA the swelling property of sample was further decreased to 33.33%, 31.15%, 28.57% and 22.73% respectively. Reduction in swelling index of material when it was mixed with both additives at different proportion was due to both have less or non-swelling property and swelling soil is replaced by such materials.

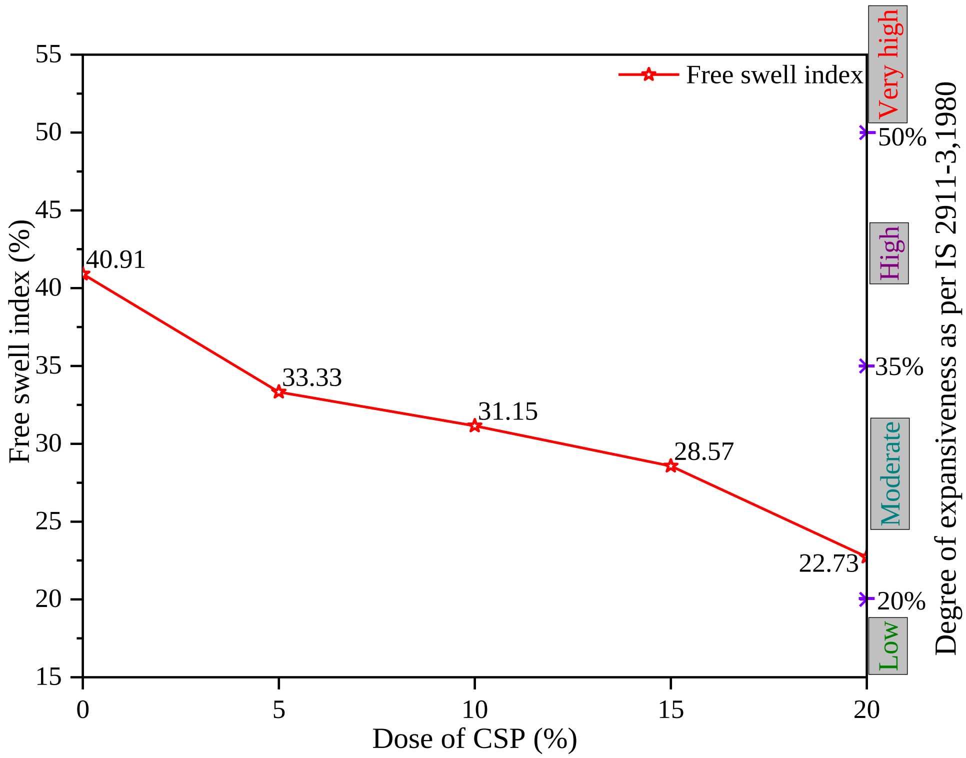


Figure 4-16: Variation of swelling index of soil with 6% ESA and variable CSP content

Hence, after mixing of 20% CSP, soil sample was improves its swelling potential and falls into moderate or nearly low category degree of expansion. Therefore, using a combination of both ESA and CSP is a more effective and efficient method in reducing the swelling characteristics of expansive soils than using ESA alone.

4.4.2. Effect of CSP on Atterberg limit of soil mixed with 6% ESA

Table 4-15: Atterberg limit of soil mixed with 6% ESA & different proportions of CSP

Dose of additives	LL (%)	PL (%)	PI (%)	LS (%)
6% ESA	81.64	44.87	36.77	19.11
6% ESA + 5% CSP	76.55	43.12	33.43	17.81
6% ESA + 10% CSP	75.21	42.27	32.94	16.89
6% ESA + 15% CSP	74.4	42.02	32.38	13.47
6% ESA + 20% CSP	71.91	40.23	31.68	12.62

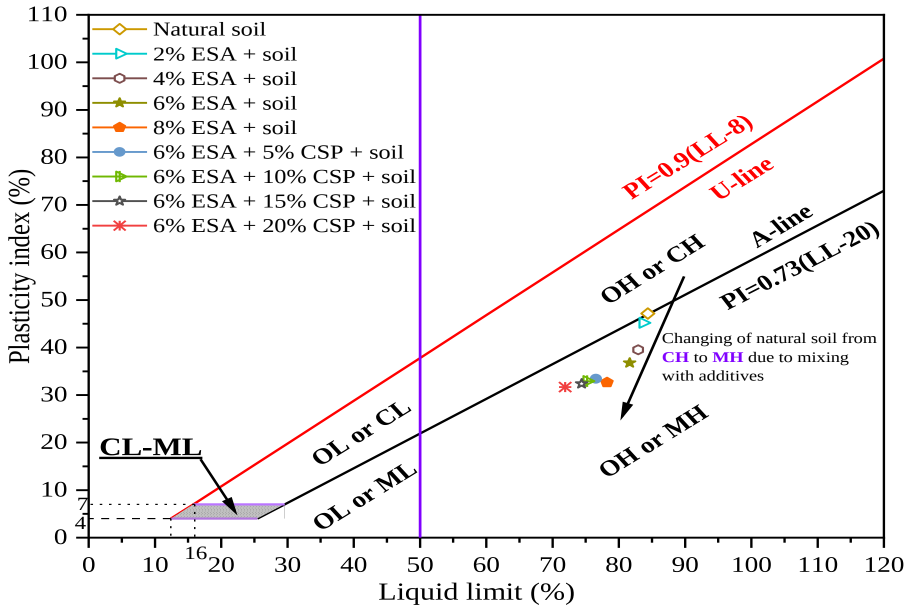


Figure 4-17: USCS Plasticity chart showing effect of additive on soil group

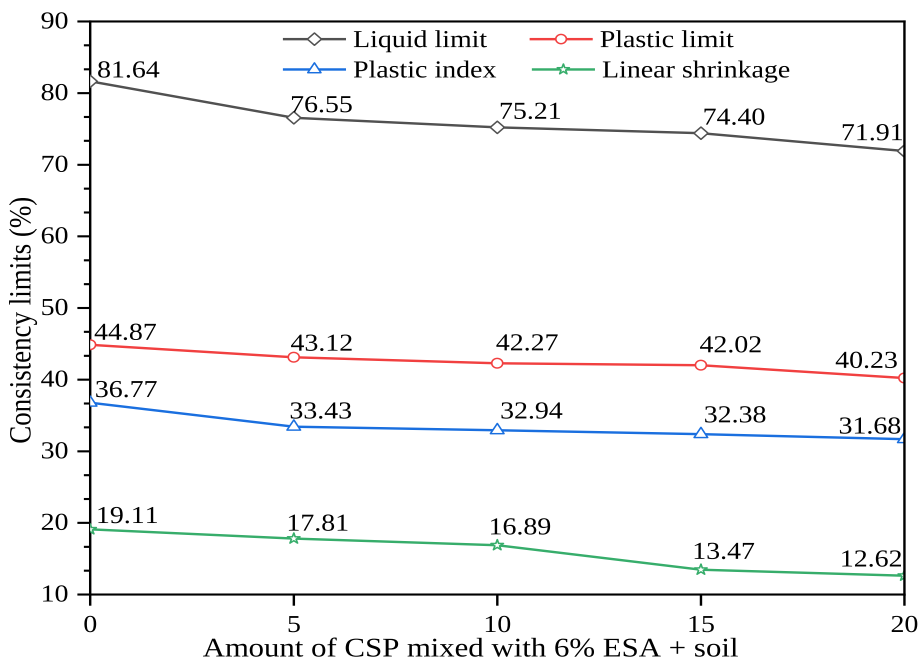


Figure 4-18: Variation of consistency limits of soil mixed additives

Soil sample mixed with 6% ESA was studied for its Atterberg limit and linear shrinkage by mixing with for different amount of crushed stone powder ranges between 5% and 20% at 5% increment interval. Based on this, figure 4-18 shows liquid limit of soil mixed with optimum ESA were 76.55%, 75.21%, 74.40%, and 71.91% as CSP was added at 5%, 10%, 15%, and 20% respectively. Similarly, relevant plastic limit of sample obtained was decreased as 43.12%, 42.27%, 42.02%, and 40.23% for the same proportion of CSP as liquid limit determination.

Plasticity index of the sample was decreased to 33.43%, 32.94%, 32.38%, and 31.68% with addition of 5%, 10%, 15%, and 20% CSP by dry weight of soil respectively. And also linear shrinkage property of natural soil mixed 6% ESA was decreased from 17.81% to 12.62% for mixing proportion increment of CSP from 5% to 20% at 5% increment interval. From all above results, plasticity property and linearly shrinkage characteristics of sample was decreased due to crushed stone powder having less or non-plasticity and less affinity to water absorption property was replaced clay mass having opposite property with additives.

4.4.3. Effect of CSP on compaction characteristics of soil mixed with 6% ESA

Table 4-16: Compaction test result of soil mixed with 6% ESA & varying CSP

Dose of ESA	MDD (g/cm^3)	OMC (%)
6% ESA	1.296	37.790
6% ESA + 5% CSP	1.377	36.087
6% ESA + 10% CSP	1.392	35.864
6% ESA + 15% CSP	1.395	35.010
6% ESA + 20% CSP	1.378	33.930

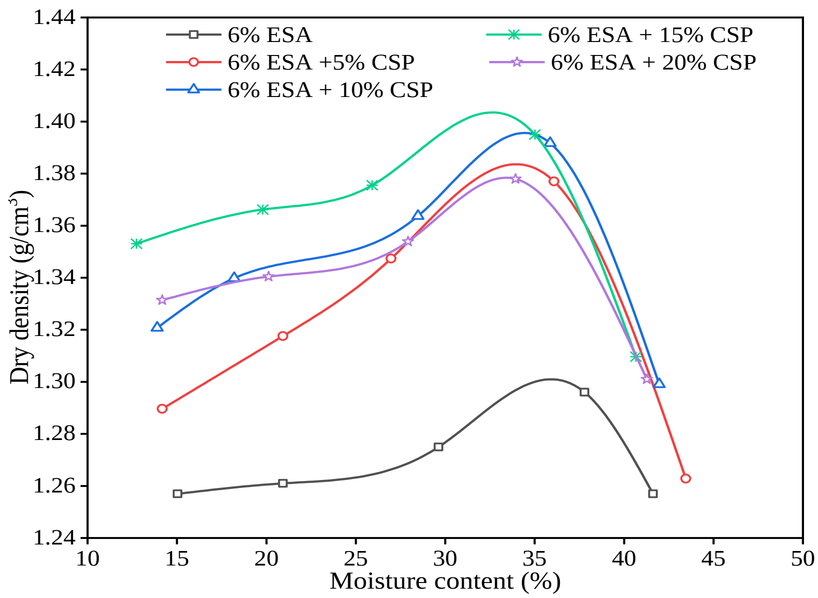


Figure 4-19: Compaction curve of soil mixed with 6% ESA and varied CSP content

Figure 4-19 shows the compaction curve of the soil sample mixed with 6% ESA and varied CSP at 5%, 10%, 15%, and 20% by dry weight of soil. From the obtained result, it was clearly seen that MDD of the treated sample was initially increased to 1.377g/cm^3 from 1.296g/cm^3 at 5% addition of CSP content and further increased to 1.392g/cm^3 and 1.395g/cm^3 for 10% and 15% of CSP content respectively. The reason in increment of maximum dry density of sample when increasing the content of CSP was due to the replacement of clay with material, CSP (G_s=2.776), having higher specific gravity. The OMC of the treated sample decreased to 36.087% from 37.790% with initial inclusion of 5% CSP and then further addition of CSP was decrease moisture content of material to 33.93% with 20% respective mixing due to the reduction in clay content of soil by replacement with crushed stone dust which has less attraction to water molecules; similar result was also observed by (Blayi *et al*., 2021).

4.4.4. Effect of CSP on compressive strength of soil mixed with 6% ESA

Table 4-17: UCS test result of soil mixed with 6% ESA & varying CSP

Dosage of additives	Unconfined compressive strength(KPa)	
	Uncured	Cured (7 days)
6% ESA	149.17	177.68
6% ESA + 5% CSP	172.85	204.82
6% ESA + 10% CSP	242.42	300.74
6% ESA + 15% CSP	269.98	391.22
6% ESA + 20% CSP	192.69	319.16

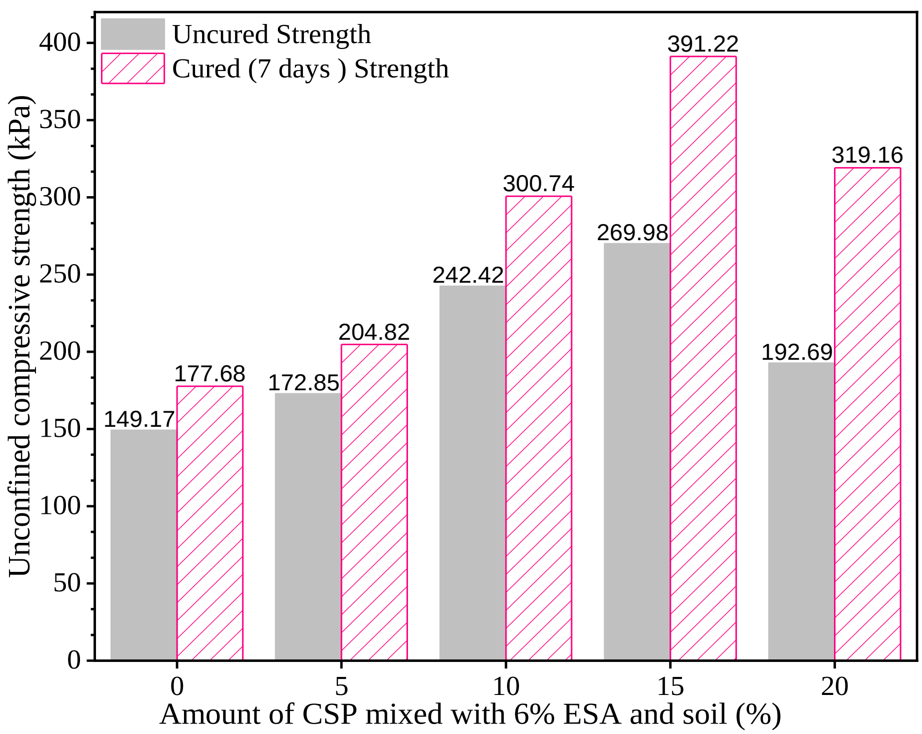


Figure 4-20: Effect of mixing variable CSP content and 6% ESA on UCS of natural soil

Figure 4-20 notices the variation of compressive strength test result with increment of CSP on optimum (6%) ESA content for both uncured and 7 days cured conditions. UCS of natural soil mixed with 6% ESA for both uncured and cured conditions are 149.17KPa and 177.68KPa respectively. But with addition of stone powder to sample mixed with 6% ESA under both uncured & cured conditions, compressive strength of samples is increased up to addition of 15% CSP content and reversed with 20% CSP content. For 5%, 10%, 15% and 20% addition of CSP with 6% ESA UCS value obtained were 172.85KPa, 242.42KPa, 269.98KPa and 192.69KPa for uncured condition and 204.82KPa, 300.74KPa, 391.22KPa and 319.16KPa for 7 days cured condition respectively. The result shows peak UCS of the specimens increases as the percentages of the crushed stone powder with 6% ESA increase up to 15%. The increase in strength of soil is due to cation exchange, flocculation and pozzolanic reaction between available ESA, CSP and soil mineral which is more for cured condition since they get much time to undergoes reaction and form floc of particles. However, beyond 15% of the crushed stone powder, the compressive strength was decreased. The reason of decrement in strength beyond this proportion was due to the reduction of cohesion between the natural soil and stone powder (cohesion less material) particles; similar result was obtained by (Blayi *et al*., 2021). With addition of stone powder, the cohesion property of soil decreases and angle of internal friction increases (Sabat, 2012).

As per Fattah *et al.*, (2014), compressive strength result is the main criteria recommended for the determination of the required optimum amount of additives to be used in soil property improvement. Based on this when ESA mixed with soil alone, 6% of ESA provides better result under cured condition & proposed as optimum amount for the case. Again, when this amount is combined with different proportions of CSP, 6% ESA mixed with 15% CSP under cured state was given the best output in case of property and proposed as final optimum amount used in combination of additives for this study.

4.5. Effect of additives on compressibility characteristics of expansive soil

Consolidation test was conducted for natural soil, soil mixed with optimum (6%) ESA and soil mixed with 6% ESA & different percentages of CSP (5%, 10%, 15% and 20%) to investigate effect of such additives on different settlement parameters like initial void ratio (e_0), coefficient of consolidation (C_v), compressibility index (C_c), coefficient of volume compressibility (m_v) and degree of compressibility of sample prepared.

4.5.1. Initial void ratio, C_c and coefficient of volume compressibility

Table 4-18: Summary of consolidation result and compression parameters of specimens

Cases	Pressure(kPa)	Sample type					
		S	E6-S	E6-C5-S	E6-C10-S	E6-C15-S	E6-C20-S
e (loading)	0	1.285	1.274	1.261	1.216	1.190	1.170
	78.48	1.233	1.235	1.228	1.195	1.167	1.152
	156.96	1.132	1.158	1.186	1.176	1.149	1.135
	313.92	0.947	0.990	0.984	1.023	0.991	0.991
	627.84	0.795	0.840	0.830	0.890	0.884	0.870
e (unloading)	78.48	1.021	0.939	1.003	1.099	1.074	1.060
	156.96	0.895	0.934	0.949	1.031	1.017	0.998
	313.92	0.826	0.868	0.873	0.970	0.929	0.914
	627.84	0.795	0.840	0.830	0.890	0.884	0.870
C_C		0.617	0.557	0.511	0.507	0.454	0.403
e_0		1.285	1.274	1.261	1.216	1.190	1.170
Average m_v (Mpa^{-1})		0.395	0.333	0.302	0.215	0.214	0.202

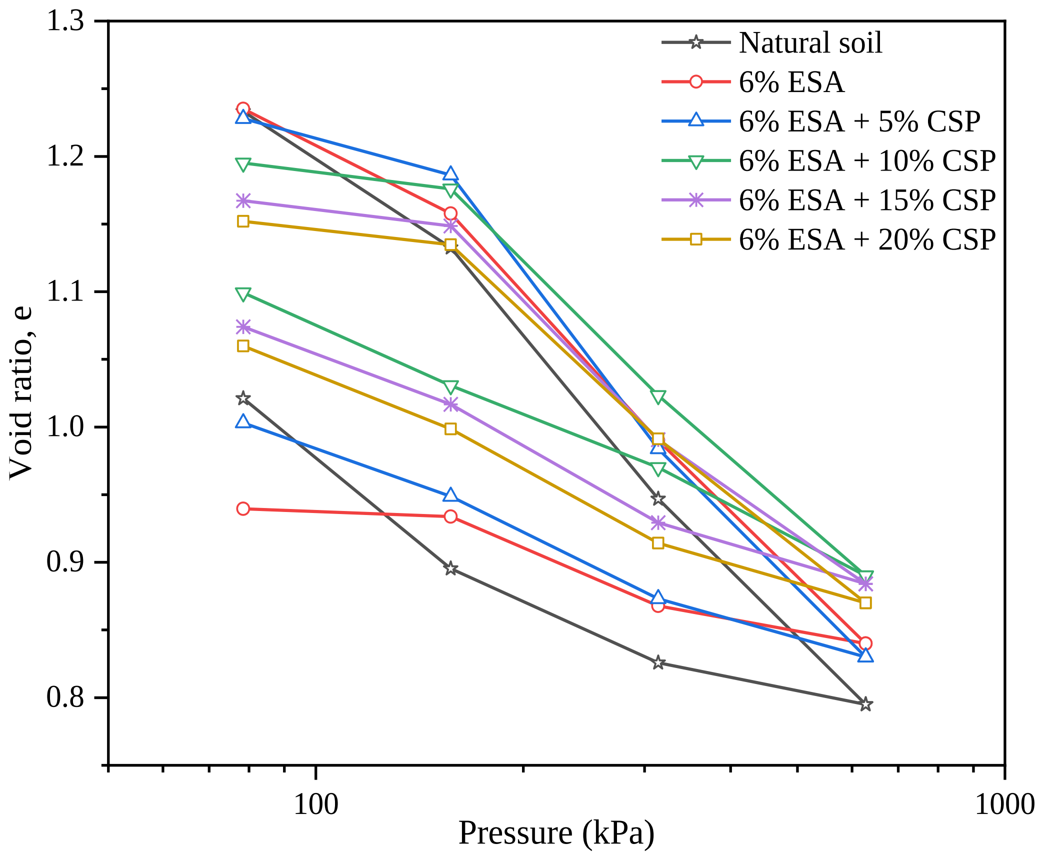


Figure 4-21: Variation of e vs log P of soil sample mixed with additives

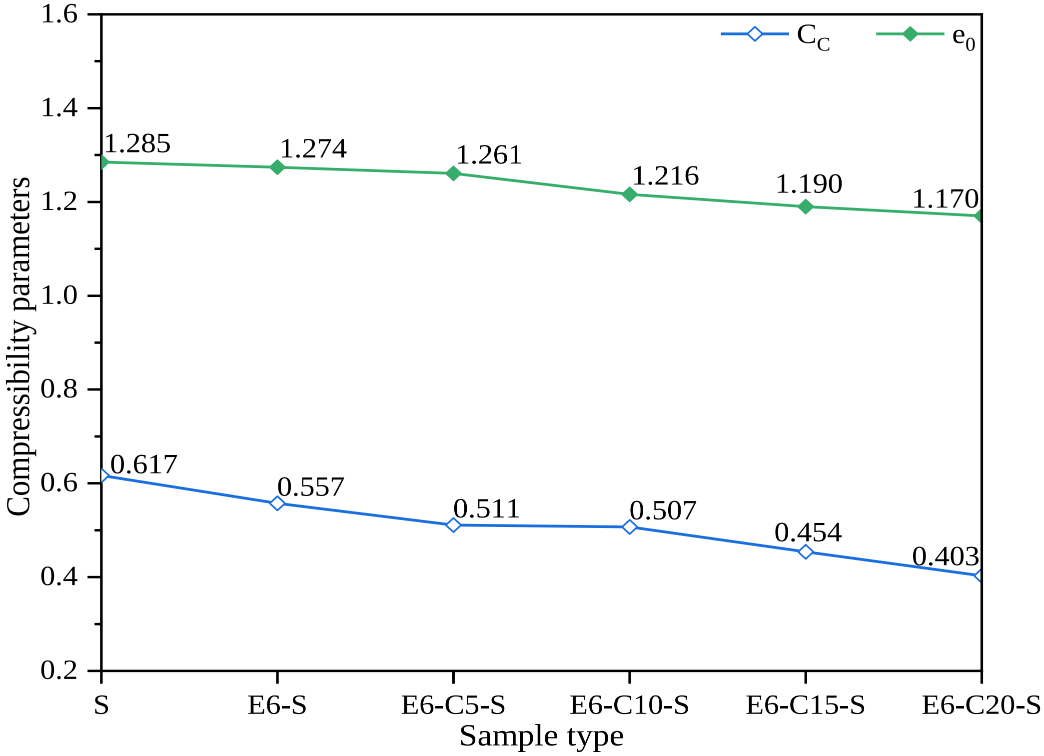


Figure 4-22: Initial void ratio and compressibility index variation with sample type

Initial void ratio is one of the important parameter of one dimensional consolidation test. In this study, it was calculated by incorporating 6% ESA and increasing dosage of CSP up to 20% to clay soil. And from the result obtained, initial void ratio of natural soil was decreased from 1.285 to 1.17 due to mixing it with 6% ESA and 20% CSP as shown on figure 4-22. The decrease in e_0 may be resulted due to reducing moisture content when the percentage of CSP increased. In previous researches conducted by Amiralian *et al*., (2013) argued by increment of lime up to 3% mixed to natural soil led to decrement of void ratio of stabilized soil. And also Omar, (2017) explained by the addition of stone powder to silty soil the initial void ratio decreased with the increase in the percentage of stone powder to 50%.

Figure 4-22 also shows the variation of compressibility index (C_c) of soil with amount of CSP mixed with 6% ESA. The value of C_C of soil sample was decreased from 0.617 to 0.403 with addition percentage of CSP up to 20% with 6% ESA. The decrement in compressibility index of the soil sample was due to replacement of expansive soil with non-plastic material having less or non-swelling characteristics. Primary consolidation settlement of soil is the parameter directly related to compressibility index, and therefore it was decreased for this case. The result obtained from this study was agreed with results found by Omar, (2017) which studied effects of stone powder on some geotechnical properties of a silty soil and showed value of compressibility index was decreased for addition of stone powder up to 50%.

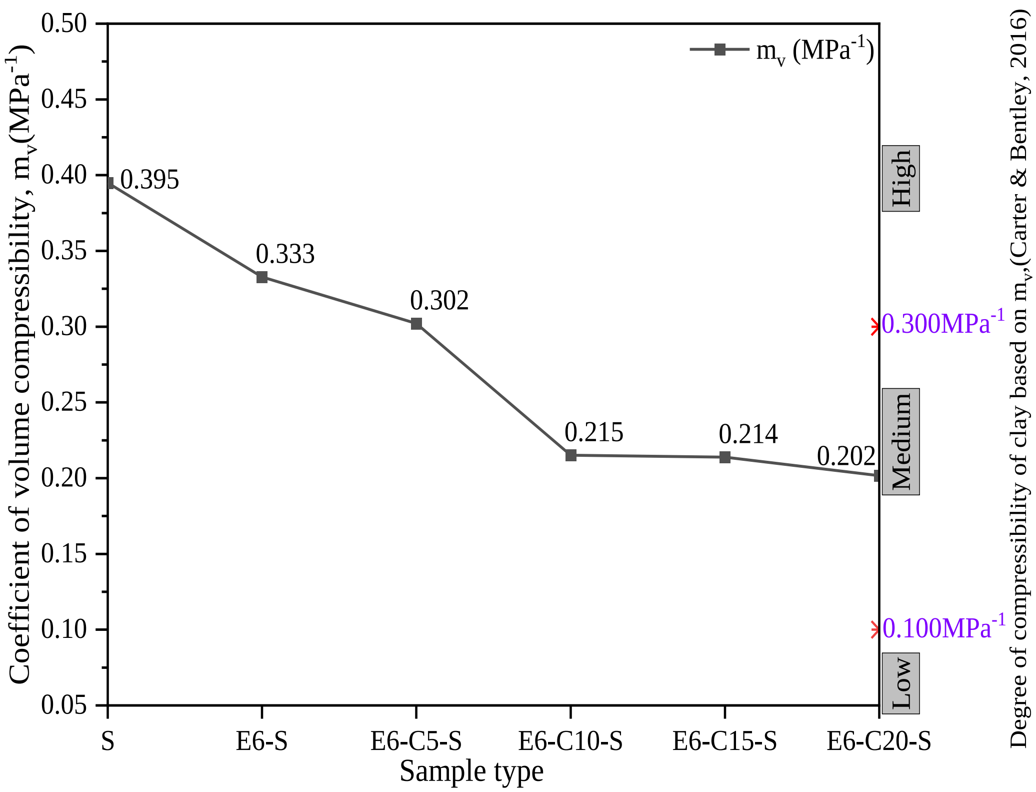


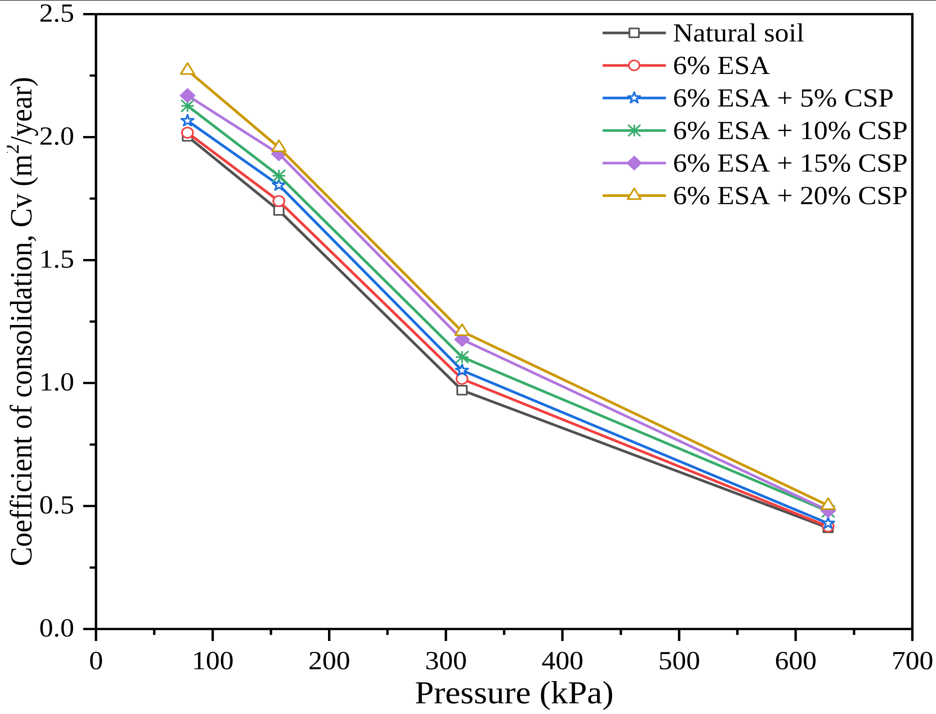
Figure 4-23: Coefficient of volume compressibility of soil variation with dose of additives

Figure 4-23 shows the variation of coefficient of volume compressibility (m_V) of soil samples mixed with 6% ESA and varied (up to 20%) CSP content. The results indicated that the values of m_V decreased from 0.395MPa^{-1} to 0.202 MPa^{-1} which decreased by 48.86% with the increase proportion of CSP up to 20% mixed to fixed (6%) ESA. The reason of decrease in coefficient of volume compressibility characteristics of soil sample was due to the decrease in plastic characteristics of blended samples with the increase in additives (ESA and CSP) content. Similar result was obtained by Mohanty *et al*., (2016) showed that by the increasing percentage of fly ash up to 30%, the m_V values directly decreased. As per Carter & Bentley, (2016) categorization of soil based on m_V value, natural soil used for the study was assigned to material having high compressibility and improved to medium compressibility material when it was mixed with specified additives content.

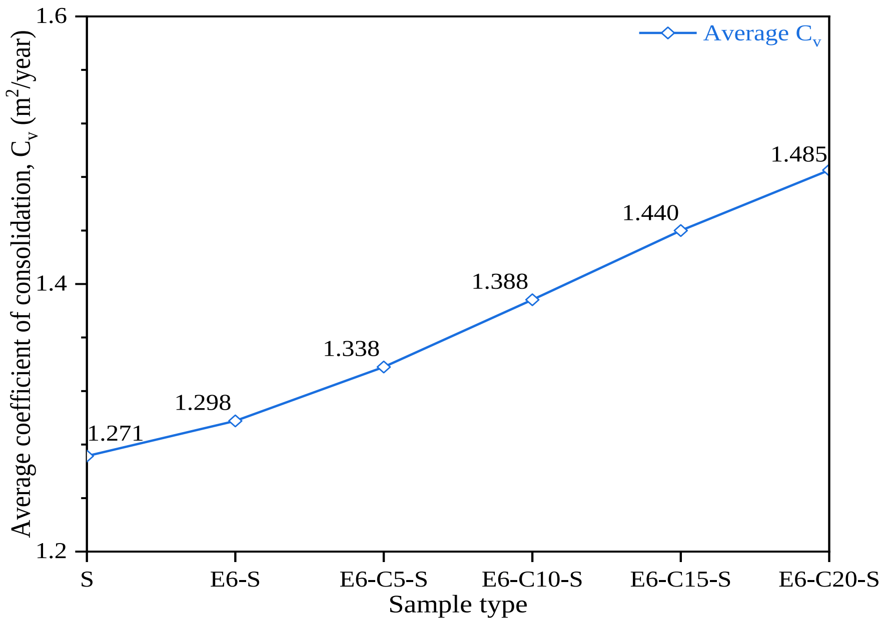
4.5.2. Coefficient of consolidation

Table 4-19: Summary of coefficient of consolidation of samples with varying additives

Pressure(kPa)	Coefficient of consolidation, Cv (m^2/year)					
	S	E6-S	E6-C5-S	E6-C10-S	E6-C15-S	E6-C20-S
78.48	2.0025	2.0177	2.0666	2.1269	2.1690	2.2714
156.96	1.7011	1.7395	1.8044	1.8425	1.9320	1.9571
313.92	0.9708	1.0169	1.0513	1.1053	1.1772	1.2094
627.84	0.4111	0.4165	0.4297	0.4779	0.4816	0.5026
Average Cv	1.2714	1.2977	1.3380	1.3882	1.4400	1.4851



(a)



(b)

Figure 4-24: Variation of C_V of specimen with (a) pressure and (b) dosage of additives

Figure 4-24 shows result of coefficient of consolidation of all specimens under different loadings of one dimensional consolidation test. Based on the analysis of variation of dial gauge readings at various time intervals for a particular stress level with respect to logarithm of time, the coefficient of consolidation (C_V), for all stabilized clay samples have been determined over a range of consolidation pressures. For natural clay, the value of C_V decreases from 2.0025 m^2/year to 0.4111 m^2/year as the pressure increases from 78.48KPa to 627.84KPa, which shows that the time required for the soil to attain a 50% degree of consolidation increases with increase in effective stress. It has been observed that the values of average C_V vary from 1.271 m^2/year to 1.485 m^2/year for soil specimens with various percentages of CSP mixed with 6% ESA at different effective stresses. This is because when the additive content is increased in the blended soil samples, the plasticity characteristics of blended samples are decreased. As a result, the compressibility of blended samples are decreased and hence the time required for 50% consolidation is also decreased. Similar result obtained by Mohanty *et al*., (2016) explained by the addition of fly ash to the soil the coefficient of consolidation increased with the increase in the percentage of fly ash up to 30%. Jain & Puri, (2013) also explained as the consolidation coefficient increases by increasing stone dust at different effective stresses and again consolidation coefficient decreases by rising effective stress at same stone dust contents.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

This paper was aimed to investigate the effect of eggshell ash and crushed stone powder on some geotechnical properties of expansive soils. Clayey soil was mixed with 2%, 4%, 6% and 8% of ESA and 5%, 10%, 15% and 20% of CSP to assess exclusive effect of ESA and combined effect of optimum ESA with varied CSP on properties of soil. Finally, the following results were obtained and concluded as follows.

- For the incorporation of ESA up to 8% by dry weight, FSI decreased from 96% to 36.36%, FSR decreased from 1.95 to 1.36, liquid limit decreased from 84.38% to 78.23%, plastic limit increased from 37.22% to 45.58%, plasticity index decreased from 47.16% to 32.65% and linear shrinkage decreased from 21.96% to 18.86%. These result reveals that mixing soil with eggshell ash reduces expansion potential, plasticity characteristics and shrinkage capacity of samples.
- Maximum dry density of sample was also decreased from 1.32g/cm^3 to 1.29g/cm^3 and optimum moisture content increased from 35.78% to 36.32% when dosage of ESA was introduced by up to 8% by dry weight.
- Unconfined compressive strength of samples was improved to 149.17kPa and 177.68kPa from 94.74kPa for both uncured and cured conditions respectively with addition of 6% ESA, which was the optimum dosage taken for this study.
- Again after mixing of 6% ESA with CSP varied up to 20% dosage to soil sample, properties such as FSI, FSR, liquid limit, plastic limit, plasticity index and linear shrinkage decreases from 40.91% to 22.73%, 1.41 to 1.23, 81.64% to 71.91%, 44.87% to 40.23%, 36.77% to 31.68% and 19.11% to 12.62% respectively.
- Maximum dry density of sample was increased from 1.296g/cm^3 to 1.378g/cm^3 and optimum moisture content decreased from 37.79% to 33.93% when 6% ESA and CSP varied up to 20% was mixed with natural soil.
- Compressive strength of soil sample mixed with 6% ESA and varied CSP up to 15% was increased from 149.17kPa to 269.98kPa & 177kPa to 391.22kPa for both uncured and

cured conditions respectively which shows curing have positive impact on strength of material.

- For the incorporation of 6% ESA mixed with varied CSP (0% to 20%) at 5% increment interval to natural soil parameters like compressibility index (C_C), initial void ratio (e_0), and coefficient of volume compressibility (m_V) was decreased from (0.617 to 0.403), (1.285 to 1.170), and (0.395MPa^{-1} to 0.202MPa^{-1}) respectively and coefficient of consolidation (C_V) was increased from 1.271 m^2/year to 1.485 m^2/year.
- Mix proportion of 6% ESA and 15% CSP with bare soil had provided better material property result and based on this the proportions was proposed as optimum mixing mount for this study.

5.2. Recommendation

- ✓ For this study calcination temperature of eggshell was limited to only 650 $\pm$ 25℃, and it should be recommended for further study by considering larger incineration temperatures to determine its effect on properties of eggshell.
- ✓ The study was tried to investigate only separate mineralogical characterization of soil and both admixtures, but it should also be mentioned to characterize combination of soil with additives.
- ✓ In this research the curing time of samples for compressive strength test was limited to 7 days only and therefore more investigation on the influence of curing time on strength property of materials using ESA and CSP as admixtures should be proposed.

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APPENDICES

Appendix-A: Natural Moisture Content of Soil Sample

Parameters used for the test

Mass of can (gm) = M_c (gm)
Mass of (can + wet soil) (gm) = M_{cw} (gm)
Mass of (can + dry soil) (gm) = M_{cd} (gm)
Mass of water (gm) = M_w (gm)
Mass of dry soil (gm) = M_d (gm)
Natural moisture content (%) = W (%)

Table A-1: Natural moisture content of soil sample collected from TP-1

Can Id.	M_c (gm)	M_{cw} (gm)	M_{cd} (gm)	M_w (gm)	M_d (gm)	W (%)
A	19.56	140	113.52	26.48	93.96	28.18
B	19.65	88.29	73.6	14.69	53.95	27.23
C	19.21	83.82	69.17	14.65	49.96	29.32
				Avg. moisture content		28.24

Table A-2: Natural moisture content of soil sample collected from TP-2

Can Id.	M_c (gm)	M_{cw} (gm)	M_{cd} (gm)	M_w (gm)	M_d (gm)	W (%)
1	18.85	140	111.99	28.01	93.14	30.07
2	19.48	70.55	59.38	11.17	39.90	27.99
3	19.44	61.81	51.96	9.85	32.52	30.29
				Avg. moisture content		29.45

Table A-3: Natural moisture content of soil sample collected from TP-3

Can Id.	M_c (gm)	M_{cw} (gm)	M_{cd} (gm)	M_w (gm)	M_d (gm)	W (%)	W (%)
A		18.85	75.50	64	11.50	45.15	25.47
B		19.48	72	60.50	11.50	41.02	28.04
C		19.44	74.60	63.45	11.15	44.01	25.34
					Avg. moisture content		26.28

Appendix-B: Specific Gravity of Materials Used for the Study

Table B-1: Specific gravity of soil sample collected from TP-1

S. No.	Specific gravity determination parameters	Pycnometer Id.	
		1	2
1	Weight of Pycnometer, M_p (gm)	43.8	43.76
2	Weight of (Pycnometer + water), $M_{a'}$ (gm)	146.95	146.5
3	Temperature of water at Ma' measured, T' (°C)	17.2	17.4
4	Density of water at T' (°C), $P_{w\,(T')}$	0.99874	0.99871
5	Weight of (Pycnometer + material + Water, M_b (gm)	153.6	153.12
6	Temperature of M_b measured, T (°C)	20.6	21.6
7	Density of water at T (°C), $P_{w\,(T)}$	0.99808	0.99786
8	Weight of (Pycnometer + water) at T °C, M_a (gm)	146.88	146.41

9	Weight of dry material, M_s (gm)	10.88	10.86
10	Specific gravity of material at T °C, G_T	2.609	2.61
11	Correction Factor	0.99987	0.99966
12	Specific gravity of material at 20 °C, G_{20}	2.609	2.609
13	Average G_s	2.609	

Table B-2: Specific gravity of soil sample collected from TP-2

S. No	Specific gravity determination parameters	Pycnometer Id.	
		A	B
1	Weight of Pycnometer, M_p (gm)	43.86	43.73
2	Weight of (Pycnometer + water), $M_{a'}$ (gm)	146.65	146.47
3	Temperature of water at Ma' measured, T' (°C)	17.20	17.40
4	Density of water at T' (°C), $P_{w\,(T')}$	0.99874	0.99871
5	Weight of (Pycnometer + material + Water, M_b (gm)	153.22	152.74
6	Temperature of M_b measured, T (°C)	22.50	21.50
7	Density of water at T (°C), $P_{w\,(T)}$	0.99766	0.99789
8	Weight of (Pycnometer + water) at T °C, M_a (gm)	146.54	146.39
9	Weight of dry material, M_s (gm)	10.79	10.265
10	Specific gravity of material at T °C, G_T	2.620	2.619
11	Correction Factor	0.99945	0.99968
12	Specific gravity of material at 20 °C, G_{20}	2.618	2.619
13	Average G_s	2.618	

Table B-3: Specific gravity of soil sample collected from TP-3

S. No	Specific gravity determination parameters	Pycnometer Id.	
		A	B
1	Weight of Pycnometer, M_p(gm)	43.86	43.73
2	Weight of (Pycnometer + water), $M_{a'}$ (gm)	146.65	146.47
3	Temperature of water at Ma' measured, T' (°C)	17.10	17.30
4	Density of water at T' (°C), $P_{w\,(T')}$	0.99876	0.99872
5	Weight of (Pycnometer + material + Water, M_b (gm)	153.20	152.72
6	Temperature of M_b measured, T (°C)	22.50	21.50
7	Density of water at T (°C), $P_{w\,(T)}$	0.99766	0.99789
8	Weight of (Pycnometer + water) at T °C, M_a (gm)	146.54	146.38
9	Weight of dry material, M_s (gm)	10.79	10.265
10	Specific gravity of material at T °C, G_T	2.609	2.607
11	Correction Factor	0.99945	0.99968
12	Specific gravity of material at 20 °C, G_{20}	2.607	2.606

13	Average G_s	2.606	

Table B-4: Specific gravity of ESA

S. No	Specific gravity determination parameters	Pycnometer Id.	
		1	2
1	Weight of Pycnometer,M_p(gm)	33.09	34.11
2	Weight of (Pycnometer + water), $M_{a'}$ (gm)	133.19	134.09
3	Temperature of water at Ma' measured, T' (°C)	17.30	17.40
4	Density of water at T' (°C), $P_{w\ (T')}$	0.99872	0.99871
5	Weight of (Pycnometer + material + Water, M_b (gm)	139.29	140.30
6	Temperature of M_b measured, T (°C)	22.40	21.80
7	Density of water at T (°C), $P_{w\ (T)}$	0.99768	0.99782
8	Weight of (Pycnometer + water) at T °C, M_a (gm)	133.09	134.00
9	Weight of dry material,M_s(gm)	10.21	10.355
10	Specific gravity of material at T °C, G_T	2.543	2.548
11	Correction Factor	0.99947	0.99961
12	Specific gravity of material at 20 °C, G_{20}	2.542	2.547
13	Average G_s	2.544	

Table B-5: Specific gravity of CSP

S. No	Specific gravity determination parameters	Pycnometer Id.	
		A	B
1	Weight of Pycnometer, M_p (gm)	43.90	43.79
2	Weight of (Pycnometer + water), $M_{a'}$ (gm)	146.59	146.47
3	Temperature of water at Ma' measured, T' (°C)	17.80	17.80
4	Density of water at T' (°C), $P_{w\ (T')}$	0.99863	0.99863
5	Weight of (Pycnometer + material + Water, M_b (gm)	153.37	152.87
6	Temperature of M_b measured, T (°C)	22	22
7	Density of water at T (°C), $P_{w\ (T)}$	0.99777	0.99777
8	Weight of (Pycnometer + water) at T °C, M_a (gm)	146.50	146.38
9	Weight of dry material,M_s(gm)	10.70	10.15
10	Specific gravity of material at T °C, G_T	2.786	2.766
11	Correction Factor	0.99957	0.99957
12	Specific gravity of material at 20 °C, G_{20}	2.785	2.765
13	Average G_s	2.775	

Appendix-C: Swelling Properties of Materials

Parameters used for the test

2% ESA + Soil = E_2-S　　　　6% ESA + 5% CSP + Soil = E_6-C_5-S

4% ESA + Soil = E_4-S
6% ESA + Soil = E_6-S
8% ESA + Soil = E_8-S
6% ESA + 10% CSP + Soil = E_6-C_{10}-S
6% ESA + 15% CSP + Soil = E_6-C_{15}-S
6% ESA + 20% CSP + Soil = E_6-C_{20}-S

Table C-1: Free swell index and free swell ratio of soil sample & additives

Sample	V_K (ml)	V_W (ml)	FSI (%)	Degree of swell	FSR	Swell potential
TP-1	10.5	19	81	Very high	1.81	Moderate
TP-2	10.5	19.5	85.7	Very high	1.86	Moderate
TP-3	10	19.5	95	Very high	1.95	Moderate
CSP	10	10.02	0.2	Low	1	Low
ESA	10	10.05	0.5	Low	1.01	Low
E_2-S	10	16	60	Very high	1.6	Moderate
E_4-S	11	16	45.45	High	1.45	Low
E_6-S	11	15.5	40.91	High	1.41	Low
E_8-S	11	15	36.36	High	1.36	Low
E_6-C_5-S	12	16	33.33	High	1.33	Low
E_6-C_5-S	12.2	16	31.15	High	1.31	Low
E_6-C_5-S	10.5	13.5	28.57	Moderate	1.29	Low
E_6-C_5-S	11	13.5	22.73	Moderate	1.23	Low

Appendix-D: Particle Size Distribution Result

Parameters used for test

Time (min) = t (min)
Actual hydrometer reading = R_a
Corrected hydrometer reading = R_c
Percent finer (%) = P (%)
Sieve number = SN
Sieve size(mm) = S_S(mm)
Mass of soil retained (gm) = M_R(gm)

Adjusted percent finer (%) = P_a (%)
Effective depth (Cm) = L (Cm)
Coefficient of temperature adjustment = K
Diameter of particle (mm) = D (mm)
Percent retained (%) = P_R (%)
Cumulative percent retained (%) = CP_R (%)
Percent passed (%) = P_P (%)

Weight of dry soil and CSP sample prepared to be washed = 1000gm and 500gm for ESA

Table D-1: Grain size analysis result of soil sample collected from TP-1

SN.	S_S (mm)	M_R (gm)	P_R (%)	CP_R (%)	P_P (%)
4	4.75	13	1.3	1.3	98.7
8	2.36	14	1.4	2.7	97.3

10	2	4	0.4	3.1	96.9
16	1.18	6	0.6	3.7	96.3
30	0.6	5	0.5	4.2	95.8
50	0.3	4	0.4	4.6	95.4
100	0.15	8	0.8	5.4	94.6
200	0.075	8	0.8	6.2	93.8
	Pan	937.42	93.74	100	0

Table D-2: Hydrometer analysis result of soil sample collected from TP-1

t(min)	R_a	R_c	P	P_a	L (Cm)	$\sqrt{L/T}$	K	D (mm)
0.5	46	44.35	89.55	89.00	8.8	4.2	0.0129	0.054
1	46	44.35	89.55	86.00	8.8	2.97	0.0129	0.038
2	45.5	43.85	88.54	83.05	8.85	2.1	0.0129	0.027
5	45	43.35	87.53	82.11	8.9	1.33	0.0129	0.017
10	44	42.35	85.51	80.21	9.1	0.95	0.0129	0.012
20	42.5	40.85	82.49	77.37	9.3	0.68	0.0129	0.009
40	40	38.35	77.44	72.64	9.7	0.49	0.0129	0.006
80	38.5	36.85	74.41	69.80	10	0.35	0.0129	0.005
120	36.5	34.85	70.37	66.01	10.3	0.29	0.0129	0.004
240	33.5	31.85	64.31	60.33	10.8	0.21	0.0129	0.003
480	30	28.35	57.25	53.70	11.4	0.15	0.0129	0.002
1440	26	24.35	49.17	46.12	12	0.09	0.0129	0.001

Table D-3: Grain size analysis of soil sample collected from TP-2

SN.	S_S (mm)	M_R (gm)	P_R (%)	CP_R (%)	P_P (%)
4	4.75	22	2.2	2.2	97.8
8	2.36	9	0.9	3.1	96.9
10	2	3	0.3	3.4	96.6
16	1.18	3	0.3	3.7	96.3
30	0.6	5	0.5	4.2	95.8
50	0.3	4	0.4	4.6	95.4
100	0.15	10	1	5.6	94.4
200	0.075	6	0.6	6.2	93.8
	Pan	937.98	93.8	100	0

Table D-4: Hydrometer analysis result of soil sample collected from TP-2

t(min)	R_a	R_c	P	P_a	L (Cm)	$\sqrt{L/T}$	K	D (mm)
0.5	47	45.35	91.33	88.67	8.6	4.15	0.01284	0.053
1	46.5	44.85	90.33	85.73	8.7	2.95	0.01284	0.038
2	46	44.35	89.32	83.78	8.8	2.1	0.01284	0.027

5	45	43.35	87.31	81.89	8.9	1.33	0.01284	0.017
10	44	42.35	85.29	80.00	9.1	0.95	0.01284	0.012
20	42	40.35	81.26	76.23	9.4	0.69	0.01284	0.009
40	40.5	38.85	78.24	73.39	9.65	0.49	0.01284	0.006
80	38.5	36.85	74.22	69.61	10	0.35	0.01284	0.005
120	36.5	34.85	70.19	65.84	10.3	0.29	0.01284	0.004
240	33.5	31.85	64.15	60.17	10.8	0.21	0.01284	0.003
480	30	28.35	57.1	53.56	11.4	0.15	0.01284	0.002
1440	23	21.35	43	40.33	12.5	0.09	0.01284	0.001

Table D-5: Grain size analysis of soil sample collected from TP-3

SN.	S_S (mm)	M_R (gm)	P_R (%)	CP_R (%)	P_P (%)
4	4.75	17	1.7	1.7	98.3
8	2.36	14	1.4	3.1	96.9
10	2	3	0.3	3.4	96.6
16	1.18	8	0.8	4.2	95.8
30	0.6	9	0.9	5.1	94.9
50	0.3	11	1.1	6.2	93.8
100	0.15	13	1.3	7.5	92.5
200	0.075	9	0.9	8.4	91.6
	Pan	915.45	91.55	100	0

Table D-6: Hydrometer analysis result of soil sample collected from TP-3

t(min)	R_a	R_c	P	P_a	L (Cm)	$\sqrt{L/T}$	K	D (mm)
0.5	48	46.35	93.66	87.79	8.4	4.1	0.01289	0.053
1	48	46.35	93.66	85.79	8.4	2.9	0.01289	0.037
2	47.5	45.85	92.65	84.87	8.5	2.06	0.01289	0.027
5	47	45.35	91.64	83.94	8.6	1.31	0.01289	0.017
10	46.5	44.85	90.63	83.01	8.7	0.93	0.01289	0.012
20	46	44.35	89.62	82.09	8.8	0.66	0.01289	0.009
40	44.5	42.85	86.59	79.31	9	0.47	0.01289	0.006
80	42.5	40.85	82.54	75.61	9.3	0.34	0.01289	0.004
120	39.5	37.85	76.48	70.06	9.8	0.29	0.01289	0.004
240	36	34.35	69.41	63.58	10.4	0.21	0.01289	0.003
480	32	30.35	61.33	56.18	11.1	0.15	0.01289	0.002
1440	27.5	25.85	52.23	47.85	11.8	0.09	0.01289	0.001

Table D-7: Grain size analysis of ESA

SN.	S_S (mm)	M_R (gm)	P_R (%)	CP_R (%)	P_P (%)
4	4.75	0	0	0	100
8	2.36	0	0	0	100

10	2	0	0	0	100
16	1.18	0	0	0	100
30	0.6	0	0	0	100
50	0.3	0	0	0	100
100	0.15	0	0	0	100
200	0.075	50	10	10	90
	Pan	450	90	100	0

Table D-8: Hydrometer analysis result of ESA

t(min)	R_a	R_c	P	P_a	L (Cm)	$\sqrt{L/T}$	K	D (mm)
0.5	43	42.6	87.36	76.02	9.2	4.29	0.01376	0.059
1	43	42.6	87.36	69.02	9.2	3.03	0.01376	0.042
2	42.5	42.1	86.34	68.21	9.3	2.16	0.01376	0.03
5	42.5	42.1	86.34	68.21	9.3	1.36	0.01376	0.019
10	42	41.6	85.31	67.40	9.4	0.97	0.01376	0.013
20	41	40.6	83.26	65.78	9.6	0.69	0.01376	0.01
40	40	39.6	81.21	64.16	9.7	0.49	0.01376	0.007
80	30	29.6	60.7	54.96	11.4	0.38	0.01376	0.005
120	20	19.6	40.2	31.75	13	0.33	0.01376	0.005
240	11	10.6	21.74	17.17	14.5	0.25	0.01376	0.003
480	4	3.6	7.38	5.83	15.6	0.18	0.01376	0.002
1440	2	1.6	3.28	2.59	16	0.11	0.01376	0.001

Table D-9: Grain size analysis of CSP

SN.	S_S (mm)	M_R (gm)	P_R (%)	CP_R (%)	P_P (%)
4	4.75	0	0	0	100
8	2.36	110	11	11	89
10	2	43	4.3	15.3	84.7
16	1.18	178	17.8	33.1	66.9
30	0.6	233	23.3	56.4	43.6
50	0.3	190	19	75.4	24.6
100	0.15	93	9.3	84.7	15.3
200	0.075	41	4.1	88.8	11.2
	Pan	112	11.2	100	0

Table D-10: Hydrometer analysis result of CSP

t(min)	R_a	R_c	P	P_a	L (Cm)	$\sqrt{L/T}$	K	D (mm)
0.5	36	35.6	70.11	7.852	10.4	4.56	0.01305	0.06
1	33	32.6	64.2	7.19	10.9	3.3	0.01305	0.043
2	30.5	30.1	59.28	6.639	11.3	2.38	0.01305	0.031

5	26.5	26.1	51.4	5.757	11.95	1.55	0.01305	0.02
10	22	21.6	42.54	4.764	12.7	1.13	0.01305	0.015
20	14.5	14.1	27.77	3.11	13.9	0.83	0.01305	0.011
40	7	6.6	13	1.456	15.2	0.62	0.01305	0.008
80	2	1.6	3.15	0.353	16	0.45	0.01305	0.006

Appendix-E: Atterberg Limit Test Results

Parameters and formulas used for LL and PL calculation

Mass of water = (Mass of can +Wet soil) - mass of can

Mass of dry soil = (Mass of can +Wet soil) - mass of can

Moisture content (%) = Mass of water *100/ Mass of dry soil

Liquid limit = Water content at number of blows are equal to 25.

Plastic limit =Average water content of plastic limit test trials

Table E-1: Liquid limit and plastic limit of natural soil from TP-1

Can ID	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass (wet soil + can)(gm)	28.26	30.69	26.74	25.03	27.12	24.35
Mass (dry soil + can)(gm)	25.2	25.6	23.05	23.4	25.09	22.94
Mass of can (gm)	19.47	19.39	19.48	18.89	19.58	18.76
Mass of dry soil (gm)	5.73	6.21	3.57	4.51	5.28	4.18
Mass of water (gm)	3.06	5.09	3.69	1.63	2.03	1.41
Water content (%)	53.4	81.96	103.36	36.14	36.84	33.73
Number of blows (counts)	35	25	19	35.57		
Liquid limit (%)	81.96					

Table E-2: Liquid limit and plastic limit of natural soil from TP-2

Can ID	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	29.15	31.13	30.73	25.29	25.05	24.24
Mass(dry soil + can)(gm)	24.7	25.79	25.58	23.71	23.54	22.96
Mass of can(gm)	19.31	19.49	19.66	19.31	19.29	19.41
Mass of dry soil(gm)	5.39	6.3	5.92	4.4	4.25	3.55
Mass of water(gm)	4.45	5.34	5.15	1.58	1.51	1.28
Water content (%)	82.56	84.76	86.99	35.91	35.53	36.06
Number of blows (counts)	35	28	19	35.83		
Liquid limit (%)	85.5					

Table E-3: Liquid limit and plastic limit of natural soil from TP-3

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	31.31	33.67	30.35	25.05	23	24.21
Mass(dry soil + can)(gm)	25.28	27.2	26.58	23.52	21.92	22.92
Mass of can(gm)	19.28	19.28	19.63	19.67	18.73	19.53
Mass of dry soil(gm)	6	7.92	6.95	3.85	3.19	3.39
Mass of water(gm)	6.03	6.47	3.77	1.53	1.08	1.29
Water content (%)	100.5	81.69	54.24	39.74	33.86	38.05
Number of blows (counts)	19	26	35	37.22		
Liquid limit (%)	84.38					

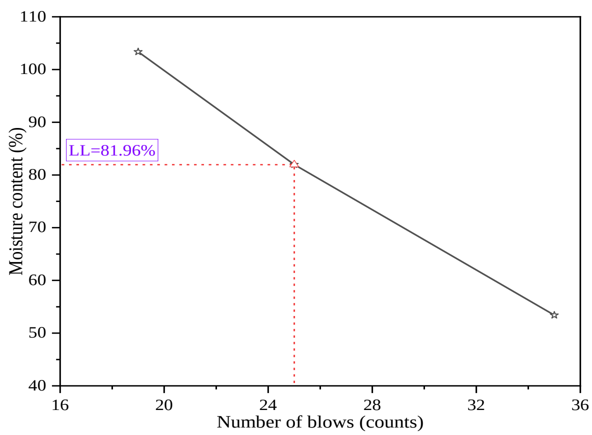


Figure E-1: Moisture content versus number of blows graph for natural soil from TP-1

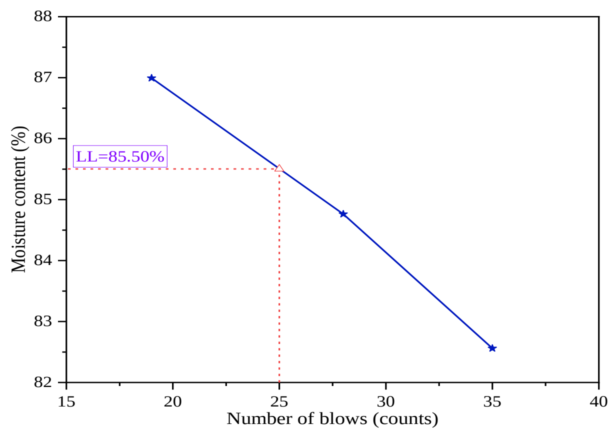


Figure E-2: Moisture content versus number of blows graph for natural soil from TP-2

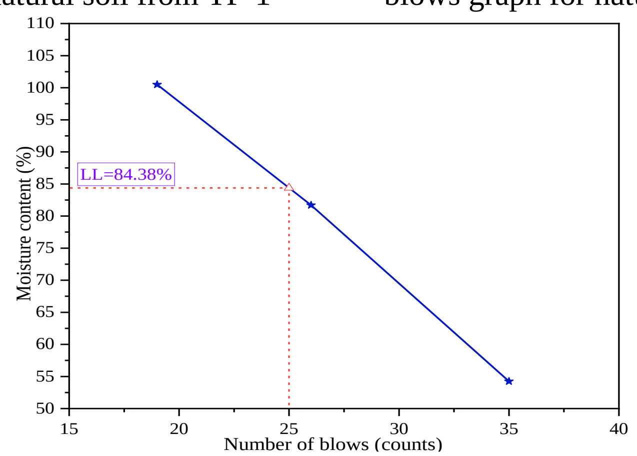


Figure E-3: Moisture content versus number of blows graph for natural soil from TP-3

Table E-4: Liquid limit and plastic limit of 2% ESA + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	45.32	40.14	40.35	23.07	23.14	23.28
Mass(dry soil + can)(gm)	36.99	35.71	36.52	22.05	22.11	22.29

Mass of can(gm)	29.27	30.05	30.03	19.39	19.43	19.64
Mass of dry soil(gm)	7.72	5.66	6.49	2.66	2.68	2.65
Mass of water(gm)	8.33	4.43	3.83	1.02	1.03	0.99
Water content (%)	107.9	78.27	59.01	38.35	38.43	37.36
Number of blows (counts)	16	27	32	38.05		
Liquid limit (%)	83.66					

Table E-5: Liquid limit and plastic limit of 4% ESA + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	30.29	31.89	27.8	34.84	34.95	35.29
Mass(dry soil + can)(gm)	24.72	26.2	24.21	33.35	33.4	33.7
Mass of can(gm)	19.33	19.34	18.72	30.03	29.7	30.04
Mass of dry soil(gm)	5.39	6.86	5.49	3.32	3.7	3.66
Mass of water(gm)	5.57	5.69	3.59	1.49	1.55	1.59
Water content (%)	103.34	82.94	65.39	44.88	41.89	43.44
Number of blows (counts)	19	25	33	43.4		
Liquid limit (%)	82.94					

Table E-6: Liquid limit and plastic limit of 6% ESA + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	44.28	40.05	37.5	34.7	33.62	34.21
Mass(dry soil + can)(gm)	37.62	35.62	34.42	33.38	32.57	32.91
Mass of can(gm)	30.19	29.82	30.11	30.5	30.23	29.95
Mass of dry soil(gm)	7.43	5.8	4.31	2.88	2.34	2.96
Mass of water(gm)	6.66	4.43	3.08	1.32	1.05	1.3
Water content (%)	89.64	76.38	71.46	45.83	44.87	43.92
Number of blows (counts)	19	29	34	44.87		
Liquid limit (%)	81.64					

Table E-7: Liquid limit and plastic limit of 8% ESA + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	41.32	43.67	39.36	35.13	34.84	35.08
Mass(dry soil + can)(gm)	36.20	37.85	35.46	33.56	33.39	33.61
Mass of can(gm)	30.06	30.41	30.09	30.12	30.22	30.37
Mass of dry soil(gm)	6.14	7.44	5.37	3.44	3.17	3.24
Mass of water(gm)	5.12	5.82	3.90	1.57	1.45	1.47
Water content (%)	83.39	78.23	72.63	45.64	45.74	45.37

Number of blows (counts)	18	25	33	45.58
Liquid limit (%)	78.23			

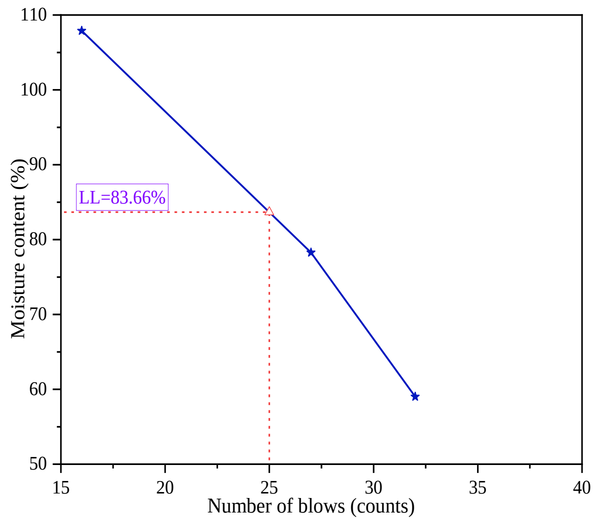


Figure E-4: Moisture content versus number of blows graph of 2% ESA + soil

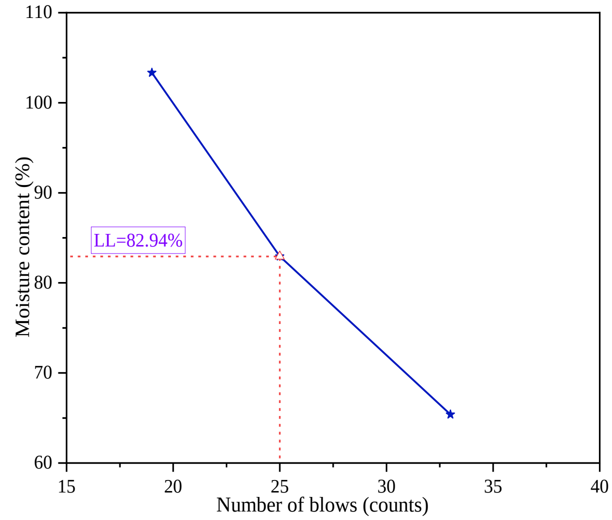


Figure E-5: Moisture content versus number of blows graph of 4% ESA + soil

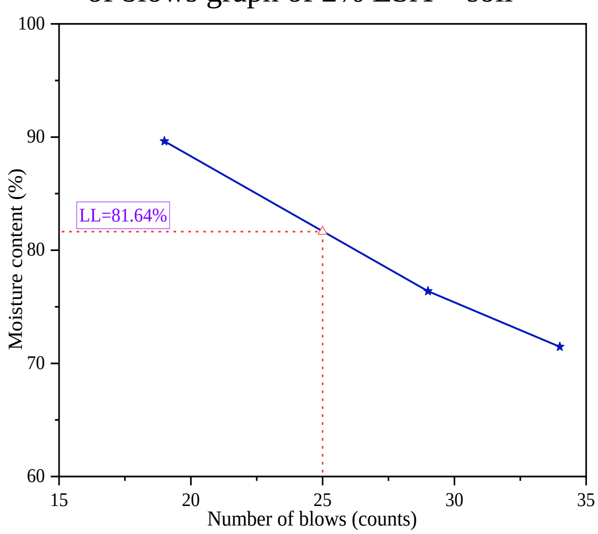


Figure E-6: Moisture content versus number of blows graph of 6% ESA + soil

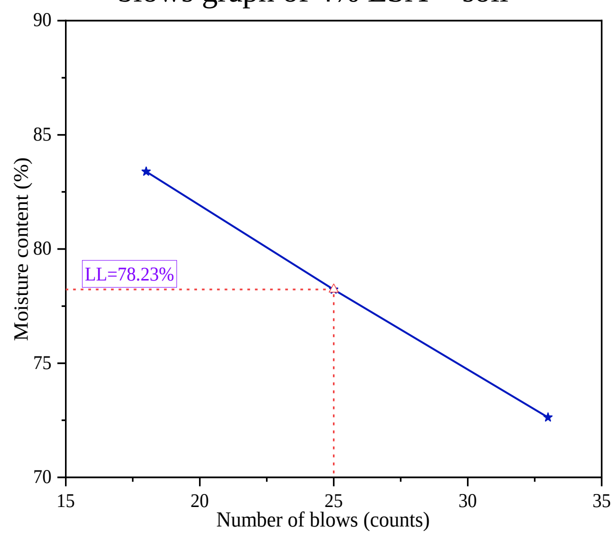


Figure E-7: Moisture content versus number of blows graph of 8% ESA + soil

Table E-8: Liquid limit and plastic limit of 6% ESA + 5% CSP + soil

	Liquid limit			Plastic Limit		
Can Id	1	2	3	4	5	6
Mass(wet soil + can)(gm)	31.14	26.65	29.95	23.78	24.06	24.3
Mass(dry soil + can)(gm)	25.82	23.68	25.65	22.49	22.67	22.87
Mass of can(gm)	19.43	19.8	19.43	19.49	19.44	19.57
Mass of dry soil(gm)	6.39	3.88	6.22	3	3.23	3.3
Mass of water(gm)	5.32	2.97	4.3	1.29	1.39	1.43

Water content (%)	83.26	76.55	69.13	43	43.03	43.33
Number of blows (counts)	18	25	35	43.12		
Liquid limit (%)	76.55					

Table E-9: Liquid limit and plastic limit of 6% ESA + 10% CSP + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	29.16	29.7	29.98	23.9	23.22	23.69
Mass(dry soil + can)(gm)	24.7	25.46	25.7	22.38	22.08	22.45
Mass of can(gm)	19.38	19.52	19.44	18.77	19.4	19.51
Mass of dry soil(gm)	5.32	5.94	6.26	3.61	2.68	2.94
Mass of water(gm)	4.46	4.24	4.28	1.52	1.14	1.24
Water content (%)	83.83	71.38	68.37	42.11	42.54	42.18
Number of blows (counts)	16	29	35	42.27		
Liquid limit (%)	75.21					

Table E-10: Liquid limit and plastic limit of 6% ESA + 15% CSP + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	32.26	29.34	28.23	25.22	24.25	24.89
Mass(dry soil + can)(gm)	26.72	24.92	24.61	23.47	22.82	23.26
Mass of can(gm)	19.46	18.88	19.45	19.31	19.44	19.35
Mass of dry soil(gm)	7.26	6.04	5.16	4.16	3.38	3.91
Mass of water(gm)	5.54	4.42	3.62	1.75	1.43	1.63
Water content (%)	76.31	73.18	70.16	42.07	42.31	41.69
Number of blows (counts)	19	29	35	42.02		
Liquid limit (%)	74.4					

Table E-11: Liquid limit and plastic limit of 6% ESA + 20% CSP + soil

Can Id	Liquid limit			Plastic Limit		
	1	2	3	4	5	6
Mass(wet soil + can)(gm)	32.3	31.8	30.65	24.12	35.01	35.41
Mass(dry soil + can)(gm)	26.7	26.6	26.14	22.8	33.59	33.8
Mass of can(gm)	19.56	19.66	19.37	19.55	30.06	29.76
Mass of dry soil(gm)	7.14	6.94	6.77	3.25	3.53	4.04
Mass of water(gm)	5.6	5.2	4.51	1.32	1.42	1.61
Water content (%)	78.43	74.93	66.62	40.62	40.23	39.85
Number of blows (counts)	18	21	32	40.23		
Liquid limit (%)	71.91					

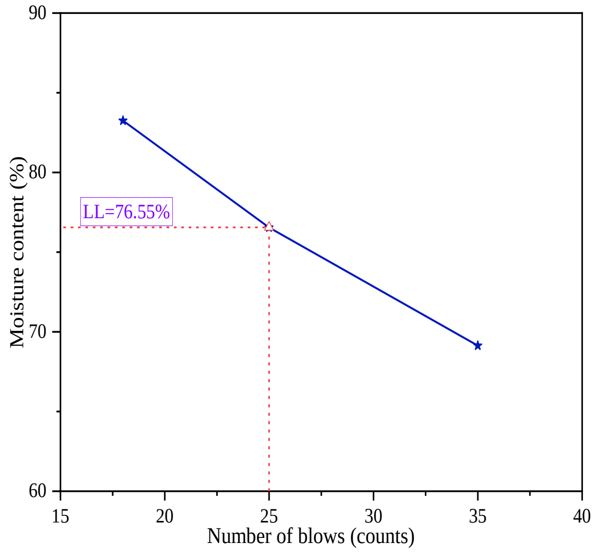


Figure E-8: Moisture content versus number of blows graph of 6% ESA + 5% CSP + soil

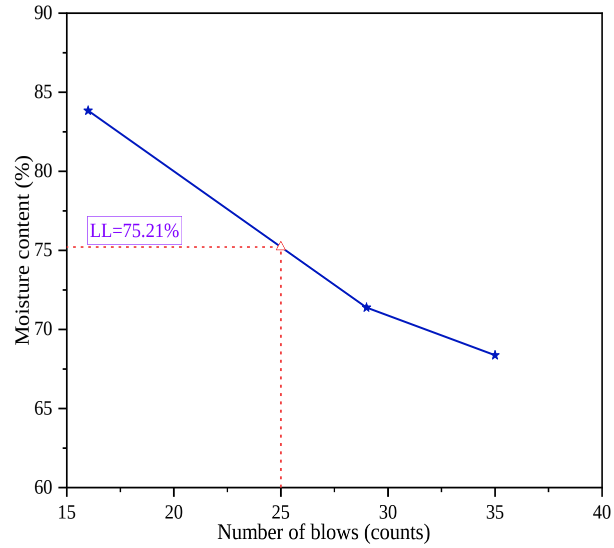


Figure E-9: Moisture content versus number of blows graph of 6% ESA + 10% CSP + soil

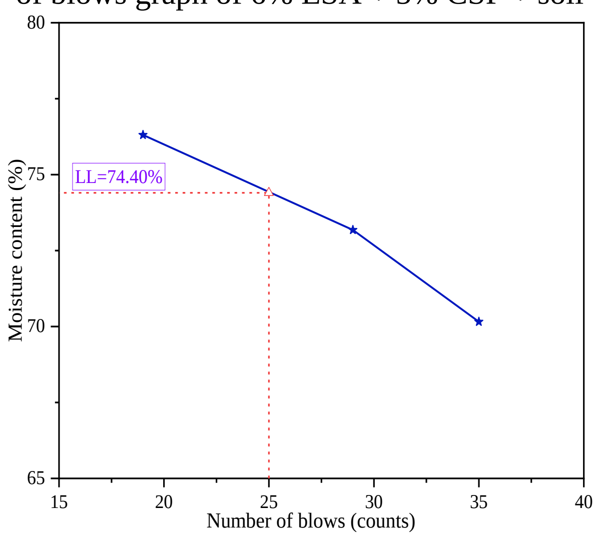


Figure E-11: Moisture content vs number of blows graph of 6% ESA + 20% CSP + soil

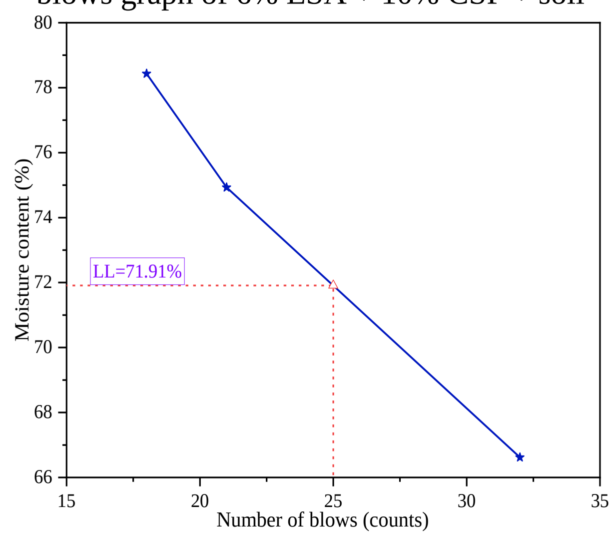


Figure E-11: Moisture content versus number of blows graph of 6% ESA + 20% CSP + soil

Parameters and formulas used for LS calculation

NB. Length was measured in both sides.

Length of wet sample=L_O=134.72mm

Average length of oven dry soil=L_D

Linear shrinkage (%)=(L_O-L_D)/(L_O))*100%

2% ESA + Soil = E_2-S

6% ESA + 5% CSP + Soil = E_6-C_5-S

4% ESA + Soil = E_4-S

6% ESA + 10% CSP + Soil = E_6-C_{10}-S

6% ESA + Soil = E_6-S

6% ESA + 15% CSP + Soil = E_6-C_{15}-S

8% ESA + Soil = E_8-S

6% ESA + 20% CSP + Soil = E_6-C_{20}-S

Table E-12: Linear shrinkage of natural soils

Test pits	TP-1	TP-2	TP-3
Length of wet sample (mm)	134.72	134.72	134.72

Length of oven dry soil after 24 hr,L1 (mm)	106.72	105.72	105.7
Length of oven dry soil after 24 hr,L2 (mm)	106.22	106.22	104.57
Average length of oven dry soil(mm)	106.47	105.97	105.14
Linear shrinkage (%)	20.97	21.34	21.96

Table E-13: Linear shrinkage of soil mixed with changing ESA

Type of sample	E_2-S	E_4-S	E_6-S	E_8-S
Length of wet sample (mm)	134.72	134.72	134.72	134.72
Length of oven dry soil after 24 hr,L1 (mm)	106.32	107.72	108.72	109.12
Length of oven dry soil after 24 hr,L2 (mm)	106.22	107.72	109.22	109.72
Average length of oven dry soil(mm)	106.27	107.72	108.97	109.42
Linear shrinkage (%)	21.12	20.04	19.11	18.78

Table: Linear shrinkage of soil mixed with 6% ESA & changing CSP

Dose of admixtures mixed with soil	E_6-C_5-S	E_6-C_{10}-S	E_6-C_{15}-S	E_6-C_{20}-S
Length of wet sample (mm)	134.72	134.72	134.72	134.72
Length of oven dry soil after 24 hr,L1 (mm)	110.72	112.22	115.22	117.72
Length of oven dry soil after 24 hr,L2 (mm)	110.72	112.22	115.22	117.72
Average length of oven dry soil(mm)	110.72	112.22	115.22	117.72
Linear shrinkage (%)	17.81	16.7	14.47	12.62

Appendix-F: Compaction Test Results

Table F-1: Compaction test results of natural soil collected from TP-1

DENSITY	Trial number	1	2	3	4	5	6
	Mass of mold (gm)	3755	3755	3755	3755	3755	3755
	Mass of (mold + soil) (gm)	5165	5233	5355	5447	5467	5403
	Mass of soil (gm)	1410	1478	1600	1692	1712	1648
	Volume of mold (cm^3)	944	944	944	944	944	944
	Bulk density (gm/cm^3)	1.49	1.57	1.69	1.79	1.81	1.75
MOISTURE CONTENT	Can identity	A	B	C	D	E	F
	Weight of can (gm)	30.12	30.23	30.09	29.96	30.02	30.46
	Weight of (wet soil + can) (gm)	65.26	63.95	63.69	66.19	67.89	67.39
	Weight of (dry soil + can) (gm)	59.72	58.01	56.59	57.36	57.68	56.33
	Weight of water (gm)	5.54	5.94	7.10	8.83	10.21	11.06
	Weight of dry soil (gm)	29.60	27.78	26.50	27.40	27.66	25.87
	Average moisture content (%)	18.64	21.38	26.82	32.22	36.92	42.76
Dry density of soil (gm/cm^3)		1.26	1.29	1.34	1.36	1.32	1.22
ZAV	Specific gravity of soil, G_s	2.609	2.609	2.609	2.609	2.609	2.609
	Density of water (gm/cm^3)	1	1	1	1	1	1
	ZAV dry density	1.76	1.67	1.53	1.42	1.33	1.23

Table F-2: Compaction test results of natural soil collected from TP-2

DENSITY	Trial number	1	2	3	4	5	6
	Mass of mold (gm)	3755	3755	3755	3755	3755	3755
	Mass of (mold + soil) (gm)	5150	5220	5315	5390	5441	5390
	Mass of soil (gm)	1395	1465	1560	1635	1686	1635
	Volume of mold (cm^3)	944	944	944	944	944	944
	Bulk density (gm/cm^3)	1.48	1.55	1.65	1.73	1.79	1.73
MOISTURE CONTENT	Can identity	A	B	C	D	E	F
	Weight of can (gm)	19.39	19.47	19.22	19.23	19.51	19.40
	Weight of (wet soil + can) (gm)	67.14	64.45	64.62	64.06	63.83	64.57
	Weight of (dry soil + can) (gm)	60.17	56.64	55.30	53.56	52.75	51.40
	Weight of water (gm)	6.97	7.81	9.32	10.50	11.08	13.17
	Weight of dry soil (gm)	40.78	37.17	36.08	34.33	33.24	32.00
	Average moisture content (%)	17.09	21.01	25.83	30.56	33.34	41.17
Dry density of soil (gm/cm^3)		1.26	1.28	1.31	1.33	1.34	1.23
ZAV	Specific gravity of soil, G_s	2.618	2.618	2.618	2.618	2.618	2.618
	Density of water (gm/cm^3)	1	1	1	1	1	1
	ZAV dry density	1.81	1.69	1.56	1.45	1.39	1.26

Table F-3: Compaction test results of natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5	6	7
	Mass of mold (gm)	3755	3755	3755	3755	3755	3755	3755
	Mass of (mold + soil) (gm)	5115	5175	5240	5325	5395	5445	5405
	Mass of soil (gm)	1360	1420	1485	1570	1640	1690	1650
	Volume of mold (cm^3)	944	944	944	944	944	944	944
	Bulk density (gm/cm^3)	1.44	1.50	1.57	1.66	1.74	1.79	1.75
MOISTURE CONTENT	Can identity	A	B	C	D	E	F	G
	Weight of can (gm)	19.45	19.46	19.31	19.44	30.37	30.51	30.01
	Weight of (wet soil + can) (gm)	62.71	63.42	63.36	63.10	76.39	84.70	79.04
	Weight of (dry soil + can) (gm)	56.39	56.05	54.86	53.21	65.05	70.42	65.11
	Weight of water (gm)	6.32	7.37	8.50	9.89	11.34	14.28	14.93
	Weight of dry soil (gm)	36.94	36.59	35.55	33.77	34.68	39.91	35.10
	Average moisture content (%)	17.11	20.15	23.92	29.29	32.69	35.78	39.67
Dry density of soil (gm/cm^3)		1.23	1.25	1.27	1.29	1.31	1.32	1.25
ZAV	Specific gravity of soil, G_S	2.606	2.606	2.606	2.606	2.606	2.606	2.606
	Density of water (gm/cm^3)	1	1	1	1	1	1	1
	ZAV dry density	1.80	1.71	1.61	1.48	1.41	1.35	1.28

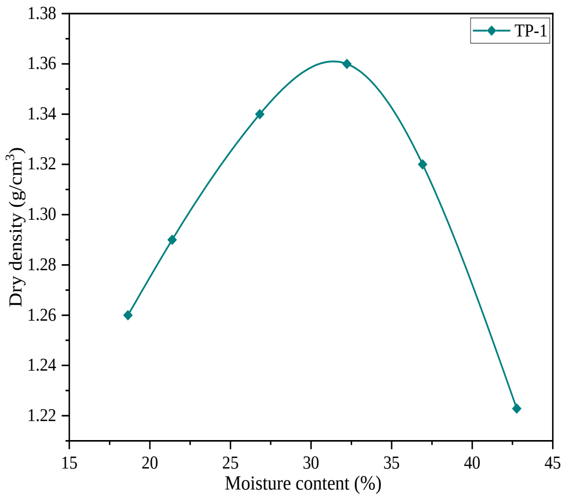


Figure F-1: Compaction curve of natural soil collected from TP-1

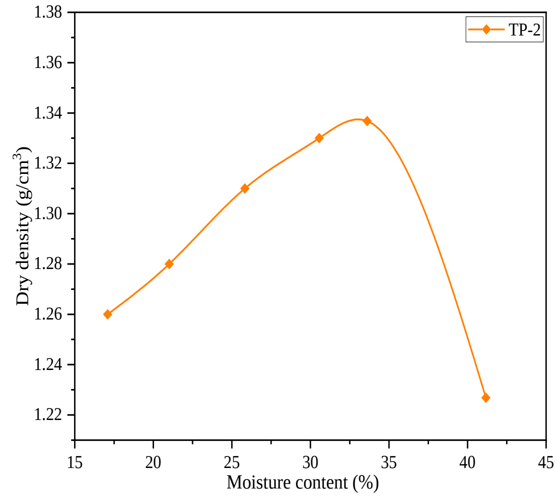


Figure F-2: Compaction curve of natural soil collected from TP-2

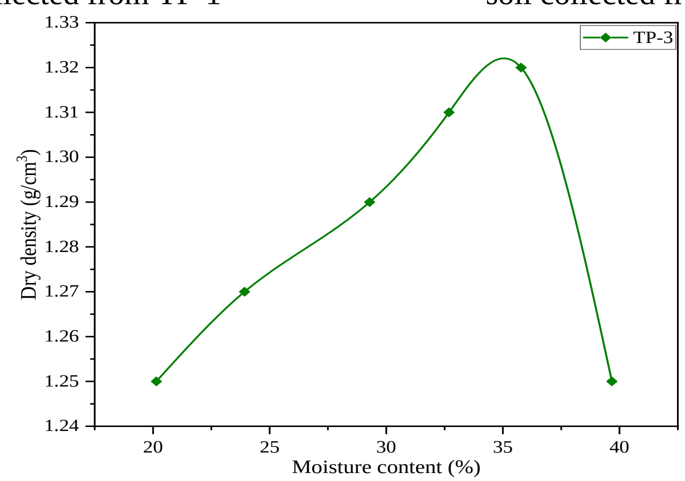


Figure F-3: Compaction curve of natural soil collected from TP-3

Table F-4: Compaction test results of 2% ESA + natural soil collected from TP-3

	Trial number	1	2	3	4	5
DENSITY	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5107	5211	5332	5441	5423
	Mass of soil (gm)	1347	1451	1572	1681	1663
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.43	1.54	1.67	1.78	1.76
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	19.40	23.02	30.36	30.19	30.32
	Weight of (wet soil + can) (gm)	51.03	53.77	72.16	77.46	78.89
	Weight of (dry soil + can) (gm)	46.99	48.32	62.85	64.93	64.66
	Weight of water (gm)	4.04	5.45	9.31	12.53	14.23
	Weight of dry soil (gm)	27.59	25.3	32.49	34.74	34.34
	Average moisture content (%)	14.64	21.54	28.64	36.06	41.43

Dry density of soil (gm/cm^3)		1.244	1.265	1.295	1.309	1.245
ZAV	Specific gravity of soil, G_S	2.605	2.605	2.605	2.605	2.605
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.884	1.671	1.492	1.343	1.253

Table F-5: Compaction test results of 4% ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5100	5200	5342	5445	5432
	Mass of soil (gm)	1340	1440	1582	1685	1672
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.42	1.53	1.68	1.78	1.77
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	30.35	29.99	29.85	29.64	30.23
	Weight of (wet soil + can) (gm)	75.13	73.59	85.37	79.16	78.77
	Weight of (dry soil + can) (gm)	69.92	66.18	72.74	65.84	64.07
	Weight of water (gm)	5.21	7.42	12.63	13.32	14.71
	Weight of dry soil (gm)	39.57	36.19	42.88	36.20	33.84
	Average moisture content (%)	13.21	20.53	29.45	34.70	45.35
Dry density of soil (gm/cm^3)		1.254	1.266	1.295	1.301	1.235
ZAV	Specific gravity of soil, G_S	2.603	2.603	2.603	2.603	2.603
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.937	1.696	1.473	1.368	1.294

Table F-6: Compaction test results of 6% ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5125	5200	5320	5446	5420
	Mass of soil (gm)	1365	1440	1560	1686	1660
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.45	1.53	1.65	1.79	1.76
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	19.35	19.23	19.32	19.39	19.18
	Weight of (wet soil + can) (gm)	47.82	41.76	42.71	62.95	73.07
	Weight of (dry soil + can) (gm)	44.11	37.86	37.38	51.02	57.25
	Weight of water (gm)	3.71	3.90	5.33	11.93	15.82
	Weight of dry soil (gm)	24.76	18.64	18.05	31.63	38.07
	Average moisture content (%)	15.03	20.93	29.63	37.79	41.55
Dry density of soil (gm/cm^3)		1.257	1.261	1.275	1.296	1.242
ZAV	Specific gravity of soil, G_S	2.602	2.602	2.602	2.602	2.602
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.870	1.685	1.469	1.312	1.250

Table F-7: Compaction test results of 8% ESA + natural soil collected from TP-3

DE	Trial number	1	2	3	4	5

	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5120	5192	5326	5420	5412
	Mass of soil (gm)	1360	1432	1566	1660	1652
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.44	1.52	1.66	1.76	1.75
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	19.36	19.58	19.41	19.16	19.27
	Weight of (wet soil + can) (gm)	40.08	49.10	61.64	70.28	78.53
	Weight of (dry soil + can) (gm)	42.73	44.22	52.05	56.66	61.64
	Weight of water (gm)	3.34	4.88	9.59	13.62	16.89
	Weight of dry soil (gm)	23.37	24.64	32.64	37.51	42.37
	Average moisture content (%)	14.22	19.84	29.31	36.32	39.87
Dry density of soil (gm/cm^3)		1.261	1.266	1.283	1.290	1.251
ZAV	Specific gravity of soil, G_S	2.601	2.601	2.601	2.601	2.601
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.899	1.716	1.476	1.337	1.277

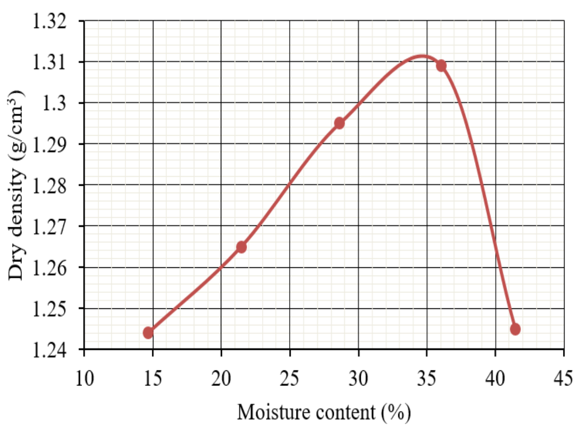


Figure F-4: Compaction curve of 2% ESA + natural soil collected from TP-3

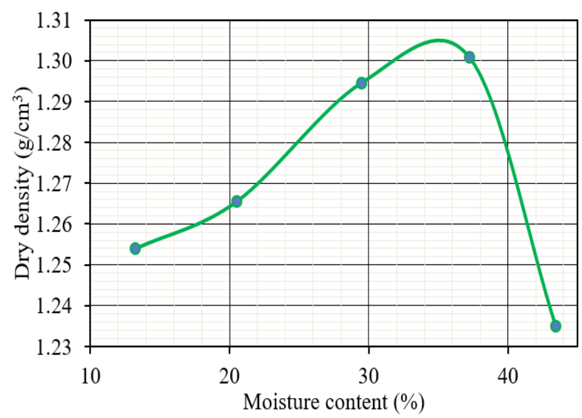


Figure F-5: Compaction curve of 4% ESA + natural soil collected from TP-3

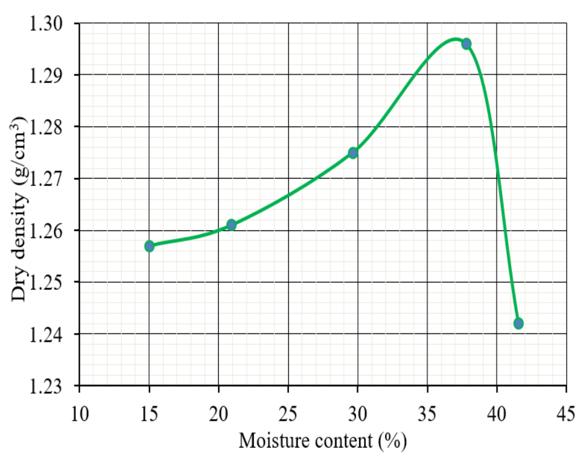


Figure F-6: Compaction curve of 6% ESA + natural soil collected from TP-3

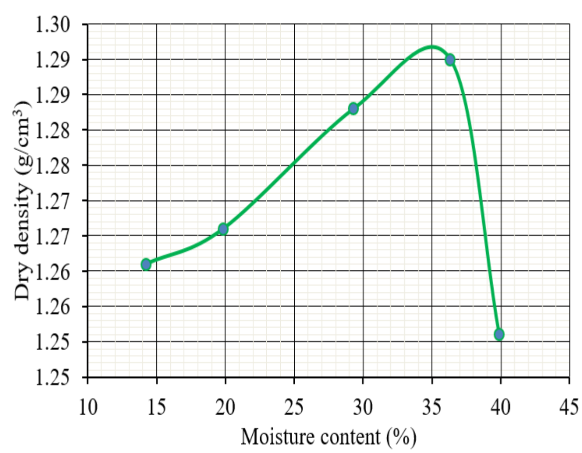


Figure F-7: Compaction curve of 8% ESA + natural soil collected from TP-3

Table F-8: Compaction test results of 5% CSP + 6% ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5150	5264	5375	5529	5470
	Mass of soil (gm)	1390	1504	1615	1769	1710
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.47	1.59	1.71	1.87	1.81
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	30.22	29.70	30.35	30.07	29.82
	Weight of (wet soil + can) (gm)	69.22	72.17	73.68	80.07	80.76
	Weight of (dry soil + can) (gm)	64.32	64.82	64.45	66.81	65.36
	Weight of water (gm)	4.90	7.35	9.23	13.25	15.40
	Weight of dry soil (gm)	34.10	35.12	34.09	36.75	35.53
	Average moisture content (%)	14.18	20.92	26.98	36.09	43.45
Dry density of soil (gm/cm^3)		1.290	1.318	1.347	1.377	1.263
ZAV	Specific gravity of soil, G_S	2.610	2.610	2.610	2.610	2.610
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.905	1.688	1.532	1.384	1.273

Table F-9: Compaction test results of 10% CSP + 6%ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5180	5255	5414	5545	5501
	Mass of soil (gm)	1420	1495	1654	1785	1741
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.50	1.58	1.75	1.89	1.84
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	30.16	30.09	30.17	30.30	30.56
	Weight of (wet soil + can) (gm)	67.82	65.23	73.81	82.04	83.53
	Weight of (dry soil + can) (gm)	63.18	59.84	64.13	68.38	67.86
	Weight of water (gm)	4.65	5.40	9.68	13.66	15.66
	Weight of dry soil (gm)	33.02	29.74	33.97	38.08	37.30
	Average moisture content (%)	13.89	18.20	28.48	35.86	41.97
Dry density of soil (gm/cm^3)		1.321	1.340	1.364	1.392	1.299
ZAV	Specific gravity of soil, G_S	2.618	2.618	2.618	2.618	2.618
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.920	1.773	1.500	1.423	1.324

Table F-10: Compaction test results of 15%CSP +6%ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5200	5305	5395	5538	5530
	Mass of soil (gm)	1440	1545	1635	1778	1770
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.525	1.637	1.732	1.883	1.875

MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	19.45	19.09	19.43	19.15	19.34
	Weight of (wet soil + can) (gm)	52.91	49.98	62.55	65.11	79.34
	Weight of (dry soil + can) (gm)	49.10	44.92	53.67	53.19	62.28
	Weight of water (gm)	3.81	5.06	8.88	11.91	17.06
	Weight of dry soil (gm)	29.65	25.82	34.24	34.04	42.94
	Average moisture content (%)	12.74	19.80	25.91	35.01	39.73
Dry density of soil (gm/cm^3)		1.353	1.366	1.376	1.395	1.342
ZAV	Specific gravity of soil, G_S	2.626	2.626	2.626	2.626	2.626
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.968	1.728	1.563	1.468	1.385

Table F-11: Compaction test results of 20%CSP +6%ESA + natural soil collected from TP-3

DENSITY	Trial number	1	2	3	4	5
	Mass of mold (gm)	3760	3760	3760	3760	3760
	Mass of (mold + soil) (gm)	5195	5280	5395	5502	5495
	Mass of soil (gm)	1435	1520	1635	1742	1735
	Volume of mold (cm^3)	944	944	944	944	944
	Bulk density (gm/cm^3)	1.52	1.61	1.73	1.85	1.84
MOISTURE CONTENT	Can identity	A	B	C	D	E
	Weight of can (gm)	19.27	19.55	19.28	19.43	19.51
	Weight of (wet soil + can) (gm)	55.89	65.21	76.04	75.76	71.75
	Weight of (dry soil + can) (gm)	51.34	57.53	63.64	61.49	56.41
	Weight of water (gm)	4.55	7.68	12.40	14.27	15.34
	Weight of dry soil (gm)	32.07	37.99	44.36	42.06	36.09
	Average moisture content (%)	14.18	20.12	27.92	33.93	41.27
Dry density of soil (gm/cm^3)		1.331	1.340	1.354	1.378	1.301
ZAV	Specific gravity of soil, G_S	2.634	2.634	2.634	2.634	2.634
	Density of water (gm/cm^3)	1	1	1	1	1
	ZAV dry density	1.918	1.722	1.518	1.391	1.321

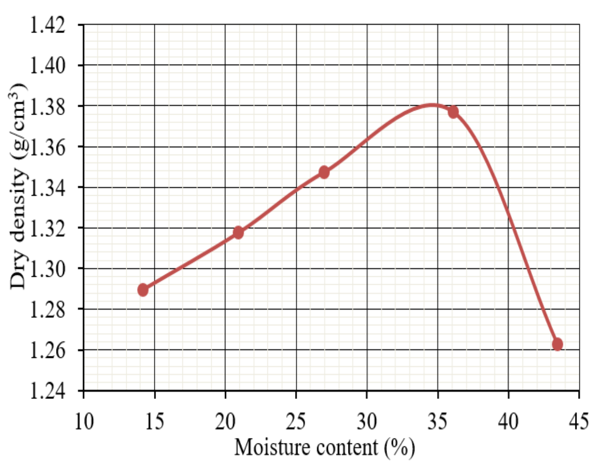


Figure F-8: Compaction curve of 5% CSP + 6% ESA + natural soil

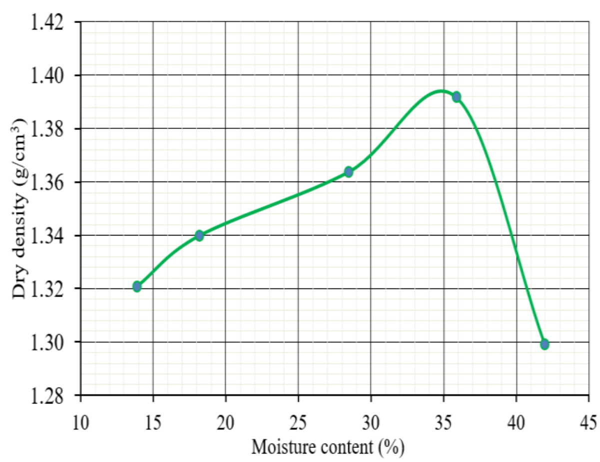


Figure F-9: Compaction curve of 10% CSP + 6% ESA + natural soil

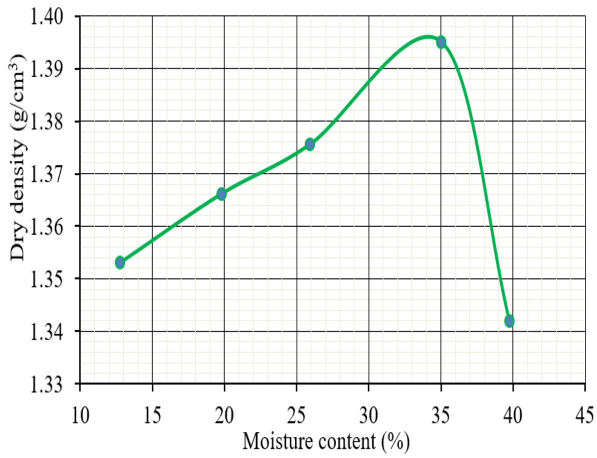


Figure F-10: Compaction curve of 15% CSP + 6% ESA + natural soil

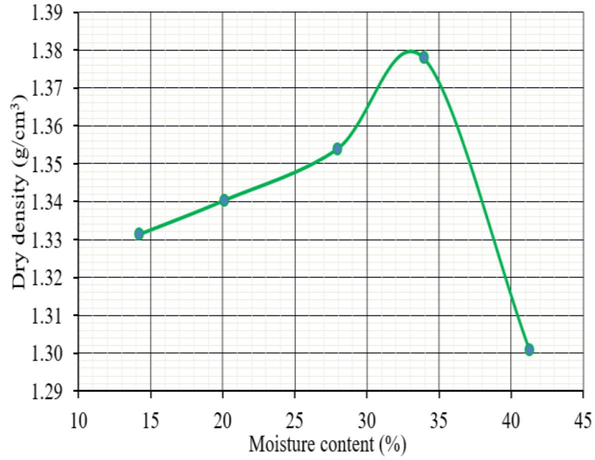


Figure F-11: Compaction curve of 20% CSP + 6% ESA + natural soil

Appendix-G: Unconfined Compressive Strength Test Results

Parameters and constants used for the test

Bulk density =ρ_{bulk}	Axial load = P
Maximum dry density = $\rho_{dry}max$	Corrected area = A_c
Deformation = ΔL	Axial stress = σ
Axial strain = ε	Height to depth ratio of sample =H/D
Optimum moisture content = OMC	Depth of sampling = D
Initial height of sample (mm)	76mm
Initial diameter of sample (mm)	36mm
Initial cross-sectional area of sample (mm^2)	$1134.11mm^2$
Initial volume of sample (mm^3)	$86192.74mm^3$
Type of sample	Remolded at OMC & 95% of MDD

Table G-1: UCS test result sheet for TP-1

ρ_{bulk} (g/cm^3)	1.79	Location		Dukem town
$\rho_{dry}max$ (g/cm^3)	1.36	D (m)		2
OMC (%)	32.22	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.14	0.19	0.038	1136.25	33.44
0.25	0.32	0.059	1137.78	51.86
0.37	0.49	0.075	1139.66	65.81
0.51	0.66	0.089	1141.70	77.95
0.63	0.83	0.101	1143.58	88.32
0.76	1.00	0.110	1145.59	96.02
0.89	1.17	0.119	1147.52	103.70
1.02	1.34	0.127	1149.57	110.48

1.16	1.52	0.135	1151.66	117.22
1.30	1.71	0.140	1153.81	121.34
1.44	1.89	0.145	1156.02	125.43
1.59	2.09	0.149	1158.27	128.64
1.73	2.28	0.153	1160.58	131.83
1.89	2.48	0.157	1162.98	134.57
2.03	2.67	0.159	1165.27	136.45
2.25	2.96	0.159	1168.72	135.62
2.50	3.29	0.156	1172.69	133.03

Table G-2: UCS test result sheet for TP-2

ρ_{bulk} (g/cm^3)	1.79	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.34	D (m)		2
OMC (%)	33.61	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.14	0.18	0.020	1136.21	17.60
0.30	0.39	0.038	1138.61	33.37
0.44	0.58	0.055	1140.72	48.22
0.58	0.76	0.067	1142.84	58.63
0.71	0.93	0.076	1144.81	66.39
0.83	1.09	0.085	1146.64	74.13
0.96	1.26	0.094	1148.62	81.84
1.07	1.41	0.100	1150.31	86.93
1.20	1.58	0.107	1152.31	92.86
1.35	1.78	0.114	1154.62	98.73
1.50	1.97	0.119	1156.95	102.86
1.65	2.17	0.124	1159.28	106.96
1.80	2.37	0.128	1161.63	110.19
1.96	2.58	0.132	1164.14	113.39
2.11	2.78	0.135	1166.50	115.73
2.27	2.99	0.138	1169.03	118.05
2.43	3.20	0.141	1171.57	120.35
2.68	3.53	0.144	1175.57	122.49
2.93	3.86	0.146	1179.59	123.77
3.18	4.18	0.148	1183.64	125.04
3.43	4.51	0.149	1187.72	125.45
3.61	4.75	0.147	1190.67	123.46
3.78	4.97	0.1455	1193.47	121.91

Table G-3: UCS test result sheet for TP-3

ρ_{bulk} (g/cm^3)	1.79	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.32	D (m)		2
OMC (%)	35.78	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00

0.14	0.18	0.015	1136.21	13.20
0.31	0.41	0.027	1138.76	23.71
0.46	0.61	0.036	1141.02	31.55
0.62	0.82	0.043	1143.44	37.61
0.74	0.97	0.048	1145.27	41.91
0.84	1.11	0.053	1146.79	46.22
0.96	1.26	0.059	1148.62	51.37
1.10	1.45	0.064	1150.77	55.61
1.23	1.62	0.068	1152.77	58.99
1.37	1.80	0.073	1154.93	63.21
1.52	2.00	0.079	1157.26	68.26
1.66	2.18	0.083	1159.44	71.59
1.79	2.36	0.087	1161.47	74.91
1.93	2.54	0.092	1163.67	79.06
2.07	2.72	0.095	1165.87	81.48
2.21	2.91	0.099	1168.08	84.75
2.34	3.08	0.101	1170.14	86.31
2.49	3.28	0.104	1172.53	88.70
2.64	3.47	0.107	1174.93	91.07
2.78	3.66	0.109	1177.17	92.59
2.94	3.87	0.111	1179.75	93.66
3.09	4.07	0.112	1182.18	94.74
3.35	4.41	0.111	1186.41	93.56
3.65	4.80	0.109	1191.33	91.49
3.90	5.13	0.106	1195.46	88.67

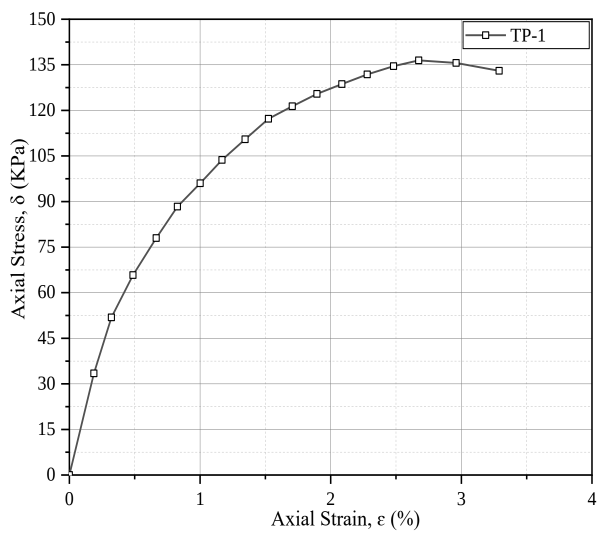


Figure G-1: UCS curve of soil collected from TP-1 (q_u = 136.45kPa)

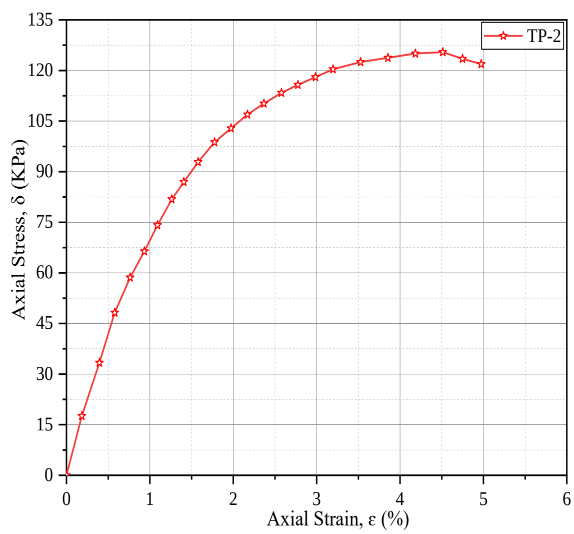


Figure G-2: UCS curve of soil collected from TP-2 (q_u = 125.45kPa)

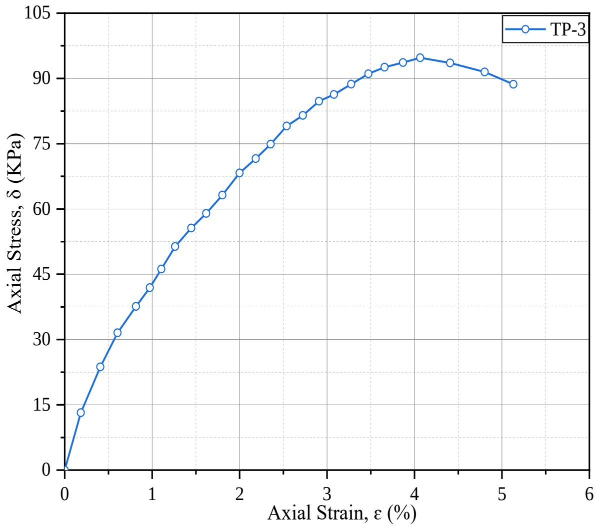


Figure G-3: UCS curve of soil collected from TP-3 (q_u = 94.74KPa)

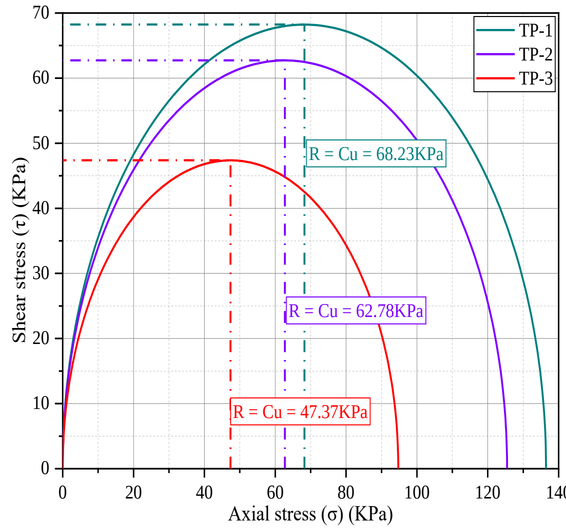


Figure G-4: Mohr circle of soil collected from TP-1, TP-2 and TP-3

Table G-4: UCS test result sheet for 2% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.78	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.309	D (m)		2
OMC (%)	36.06	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.60	0.79	0.009	1143.14	8.27
1.00	1.32	0.023	1149.24	20.10
1.29	1.70	0.043	1153.70	37.31
1.45	1.91	0.058	1156.17	49.95
1.53	2.01	0.066	1157.42	57.15
1.62	2.13	0.074	1158.82	63.43
1.70	2.24	0.080	1160.06	68.79
1.79	2.36	0.087	1161.47	75.03
1.89	2.49	0.093	1163.04	80.35
1.98	2.61	0.098	1164.45	83.86
2.12	2.79	0.103	1166.66	88.20
2.27	2.99	0.108	1169.03	92.51
2.43	3.20	0.112	1171.57	95.90
2.59	3.41	0.117	1174.13	99.27
2.76	3.63	0.120	1176.85	101.71
2.93	3.86	0.123	1179.59	104.15
3.09	4.07	0.125	1182.18	105.69
3.27	4.30	0.127	1185.11	107.21
3.47	4.57	0.125	1188.37	105.14
3.70	4.87	0.122	1192.15	102.17

Table G-5: UCS test result sheet for 2% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.78	Location		Dukem town

ρ_{dry}max (g/cm^3)	1.309	D (m)		2
OMC (%)	36.06	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.45	0.59	0.003	1140.87	2.89
0.75	0.99	0.007	1145.42	5.76
1.00	1.32	0.012	1149.24	10.53
1.12	1.47	0.024	1151.08	21.02
1.24	1.63	0.040	1152.93	34.35
1.38	1.82	0.052	1155.09	44.76
1.53	2.01	0.066	1157.42	57.02
1.67	2.20	0.078	1159.60	67.35
1.82	2.39	0.086	1161.94	73.84
1.97	2.59	0.095	1164.29	81.25
2.13	2.80	0.103	1166.82	88.62
2.28	3.00	0.110	1169.19	94.08
2.46	3.24	0.118	1172.05	100.42
2.64	3.47	0.123	1174.93	104.86
2.81	3.70	0.127	1177.66	107.42
2.98	3.92	0.130	1180.40	109.96
3.15	4.14	0.134	1183.15	113.43
3.37	4.43	0.136	1186.74	114.94
3.61	4.75	0.139	1190.67	116.40
3.90	5.13	0.140	1195.46	116.86
4.24	5.58	0.141	1201.13	117.22
4.47	5.88	0.138	1204.99	114.11
4.75	6.25	0.131	1209.72	108.21

Table G-6: UCS test result sheet for 4% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.78	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.301	D (m)		2
OMC (%)	34.70	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.50	0.66	0.005	1141.63	4.20
0.85	1.12	0.012	1146.94	10.46
1.14	1.50	0.023	1151.39	19.80
1.35	1.78	0.040	1154.62	34.30
1.53	2.01	0.052	1157.42	44.58
1.71	2.25	0.066	1160.22	56.89
1.85	2.43	0.077	1162.41	66.07
1.99	2.62	0.085	1164.61	73.16
2.15	2.83	0.092	1167.13	79.17
2.30	3.03	0.098	1169.51	84.14
2.45	3.22	0.104	1171.89	89.09
2.59	3.41	0.110	1174.13	94.03

2.75	3.62	0.116	1176.69	98.92
2.92	3.84	0.122	1179.43	103.78
3.09	4.07	0.127	1182.18	107.60
3.27	4.30	0.130	1185.11	109.36
3.43	4.51	0.134	1187.72	113.16
3.59	4.72	0.138	1190.34	115.93
3.74	4.92	0.142	1192.81	118.71
3.91	5.14	0.145	1195.63	121.44
4.06	5.34	0.146	1198.12	122.19
4.22	5.55	0.149	1200.79	123.92
4.37	5.75	0.148	1203.30	122.66
4.51	5.93	0.146	1205.66	121.43

Table G-7: UCS test result sheet for 4% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.78	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.301	D (m)		2
OMC (%)	34.70	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.50	0.66	0.007	1141.63	5.69
0.85	1.12	0.012	1146.94	10.20
1.14	1.50	0.017	1151.39	14.90
1.35	1.78	0.026	1154.62	22.52
1.53	2.01	0.041	1157.42	35.04
1.71	2.25	0.058	1160.22	49.75
1.85	2.43	0.070	1162.41	60.39
1.99	2.62	0.084	1164.61	72.33
2.15	2.83	0.098	1167.13	83.54
2.30	3.03	0.108	1169.51	92.26
2.45	3.22	0.117	1171.89	99.84
2.59	3.41	0.125	1174.13	106.29
2.75	3.62	0.131	1176.69	111.36
2.92	3.84	0.140	1179.43	119.04
3.09	4.07	0.148	1182.18	125.36
3.27	4.30	0.156	1185.11	131.63
3.43	4.51	0.161	1187.72	135.72
3.59	4.72	0.167	1190.34	140.23
3.74	4.92	0.170	1192.81	142.55
3.91	5.14	0.173	1195.63	144.83
4.06	5.34	0.176	1198.12	147.13
4.22	5.55	0.179	1200.79	149.40
4.37	5.75	0.179	1203.30	149.09
4.51	5.93	0.181	1205.66	150.09
4.66	6.13	0.178	1208.20	147.41
4.81	6.33	0.177	1210.74	146.03
4.95	6.51	0.174	1213.13	143.60

5.10	6.71	0.172	1215.69	141.15

Table G-8: UCS test result sheet for 6% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.79	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.296	D (m)		2
OMC (%)	37.79	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.30	0.39	0.003	1138.61	2.46
0.60	0.79	0.006	1143.14	4.90
0.87	1.14	0.011	1147.25	9.76
1.12	1.47	0.021	1151.08	18.24
1.24	1.63	0.035	1152.93	30.36
1.37	1.80	0.050	1154.93	43.64
1.49	1.96	0.064	1156.79	55.67
1.57	2.07	0.077	1158.04	66.49
1.72	2.26	0.091	1160.38	78.42
1.88	2.47	0.102	1162.88	87.89
2.05	2.70	0.116	1165.55	99.70
2.22	2.92	0.126	1168.24	107.85
2.39	3.14	0.136	1170.94	115.98
2.58	3.39	0.144	1173.97	122.83
2.75	3.62	0.151	1176.69	128.50
2.94	3.87	0.157	1179.75	132.91
3.14	4.13	0.161	1182.99	136.10
3.32	4.37	0.164	1185.92	138.12
3.51	4.62	0.168	1189.03	141.29
3.70	4.87	0.171	1192.15	143.27
4.00	5.26	0.176	1197.12	147.35
4.25	5.59	0.179	1201.29	149.17
4.60	6.05	0.178	1207.18	147.29

Table G-9: UCS test result sheet for 6% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.79	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.296	D (m)		2
OMC (%)	37.79	H/D		2
ΔL (mm)	ε(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.38	0.50	0.012	1139.81	10.53
0.72	0.95	0.023	1144.96	20.21
1.10	1.45	0.036	1150.77	30.93
1.38	1.82	0.055	1155.09	47.76
1.57	2.07	0.077	1158.04	66.08
1.73	2.28	0.100	1160.53	85.88
1.85	2.43	0.114	1162.41	97.98
1.95	2.57	0.130	1163.98	111.61

2.06	2.71	0.144	1165.71	123.53
2.18	2.87	0.157	1167.61	134.13
2.32	3.05	0.166	1169.83	141.48
2.45	3.22	0.173	1171.89	147.31
2.58	3.39	0.178	1173.97	151.59
2.71	3.57	0.183	1176.05	155.86
2.84	3.74	0.190	1178.14	161.63
2.95	3.88	0.192	1179.91	162.90
3.07	4.04	0.198	1181.86	167.15
3.22	4.24	0.201	1184.29	169.72
3.44	4.53	0.205	1187.88	172.29
3.65	4.80	0.207	1191.33	173.76
3.86	5.08	0.210	1194.80	175.51
4.10	5.39	0.213	1198.79	177.68
4.30	5.66	0.210	1202.13	174.69
4.53	5.96	0.207	1206.00	171.64

Table G-10: UCS test result sheet for 8% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.76	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.290	D (m)		2
OMC (%)	36.32	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
1.12	1.47	0.005	1151.08	4.34
1.31	1.72	0.009	1154.01	7.58
1.53	2.01	0.015	1157.42	12.96
1.74	2.29	0.021	1160.69	18.31
1.94	2.55	0.031	1163.82	26.85
2.12	2.79	0.040	1166.66	34.29
2.28	3.00	0.053	1169.19	44.90
2.42	3.18	0.065	1171.42	55.49
2.56	3.37	0.075	1173.65	63.90
2.69	3.54	0.085	1175.73	72.30
2.83	3.72	0.095	1177.98	80.65
2.97	3.91	0.103	1180.24	86.85
3.11	4.09	0.109	1182.50	91.97
3.25	4.28	0.118	1184.78	99.17
3.39	4.46	0.123	1187.06	103.20
3.53	4.64	0.126	1189.36	106.15
3.67	4.83	0.133	1191.66	111.19
3.82	5.03	0.135	1194.14	113.05
3.99	5.25	0.140	1196.96	116.96
4.15	5.46	0.145	1199.62	120.87
4.31	5.67	0.148	1202.30	122.68
4.47	5.88	0.151	1204.99	125.52
4.69	6.17	0.154	1208.70	127.20

4.90	6.45	0.155	1212.27	127.86
5.10	6.71	0.156	1215.69	128.53
5.25	6.91	0.158	1218.27	129.28
5.48	7.21	0.160	1222.25	130.91
5.70	7.50	0.159	1226.07	129.48
5.84	7.68	0.158	1228.52	128.20
5.98	7.87	0.155	1230.97	125.92

Table G-11: UCS test result sheet for 8% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.76	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.290	D (m)		2
OMC (%)	36.32	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.14	0.18	0.001	1136.21	1.14
0.31	0.41	0.004	1138.76	3.42
0.50	0.66	0.007	1141.63	5.69
0.73	0.96	0.013	1145.11	11.35
0.97	1.28	0.025	1148.78	21.50
1.18	1.55	0.036	1152.00	31.60
1.35	1.78	0.047	1154.62	40.53
1.51	1.99	0.057	1157.10	49.43
1.67	2.20	0.065	1159.60	56.05
1.78	2.34	0.077	1161.31	66.05
1.90	2.50	0.087	1163.19	74.88
2.01	2.64	0.094	1164.92	80.35
2.14	2.82	0.105	1166.97	90.23
2.27	2.99	0.114	1169.03	97.86
2.39	3.14	0.125	1170.94	106.58
2.52	3.32	0.130	1173.01	110.83
2.64	3.47	0.138	1174.93	117.28
2.78	3.66	0.146	1177.17	123.69
2.93	3.86	0.150	1179.59	126.74
3.07	4.04	0.157	1181.86	133.10
3.22	4.24	0.164	1184.29	138.31
3.40	4.47	0.168	1187.23	141.25
3.61	4.75	0.173	1190.67	145.21
3.85	5.07	0.177	1194.63	148.00
4.10	5.39	0.179	1198.79	149.65
4.39	5.78	0.182	1203.64	151.21
4.64	6.11	0.181	1207.86	149.60
4.86	6.39	0.178	1211.59	147.00
5.02	6.61	0.177	1214.32	145.60

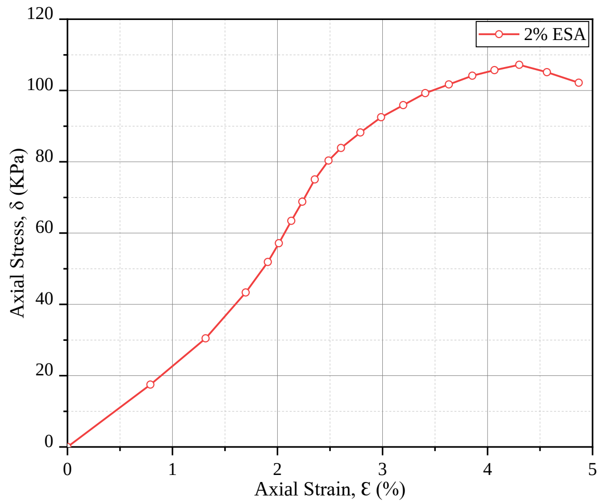


Figure G-5: UCS curve of 2% ESA + soil (q_u (uncured) = 107.21KPa)

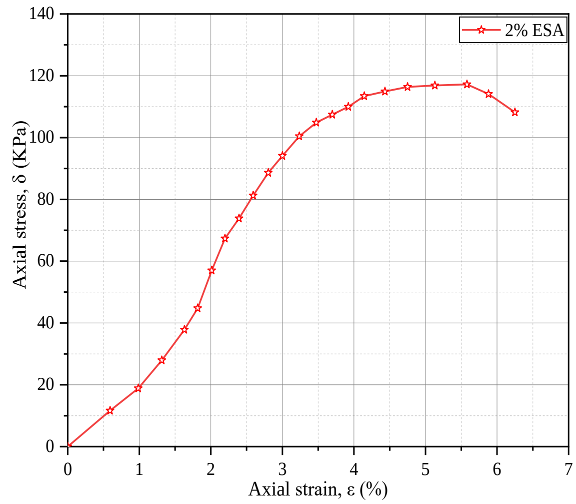


Figure G-6: UCS curve of 2% ESA + soil (q_u (cured) =117.22KPa)

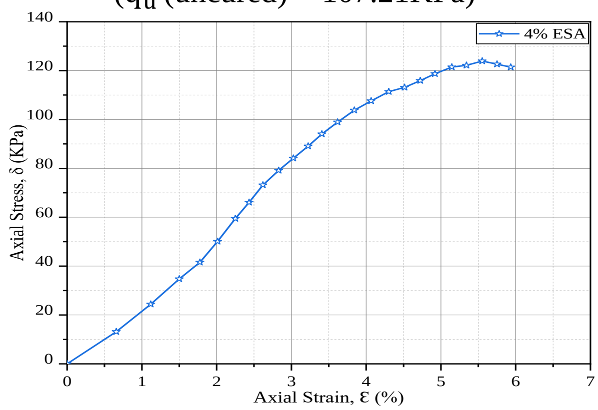


Figure G-7: UCS curve of 4% ESA + soil (q_u (uncured) = 123.92KPa)

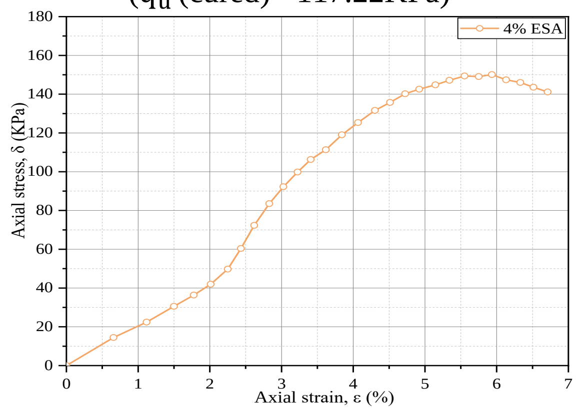


Figure G-8: UCS curve of 4% ESA + soil (q_u (cured) =150.09KPa)

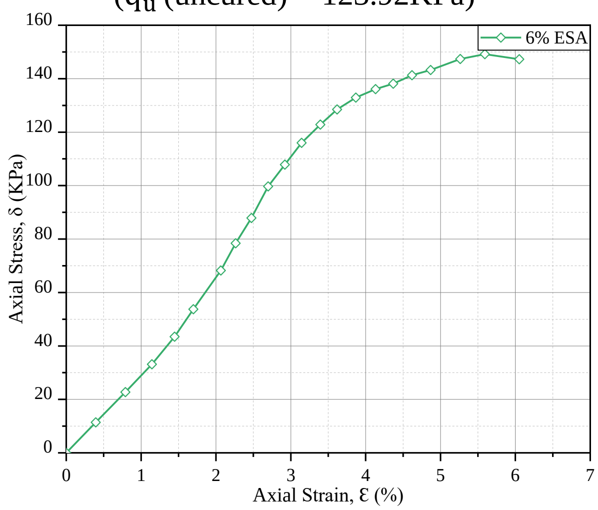


Figure G-9: UCS curve of 6% ESA + soil (q_u (uncured) = 149.17KPa)

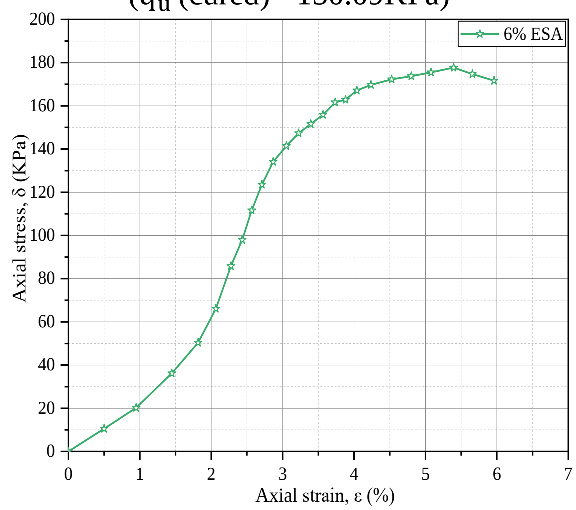


Figure G-10: UCS curve of 6% ESA + soil (q_u (cured) =177.68KPa)

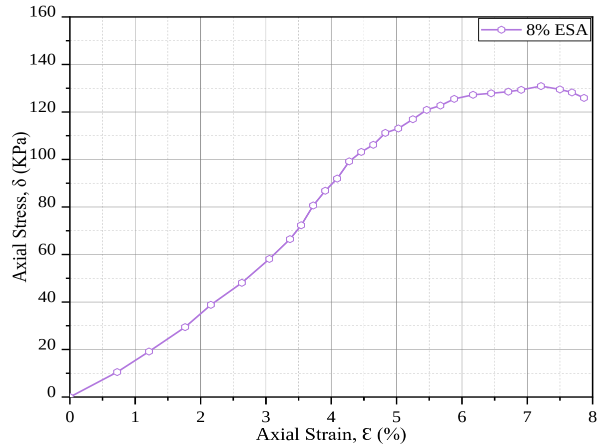


Figure G-11: UCS curve of 8% ESA + soil (q_u (uncured) = 130.91KPa)

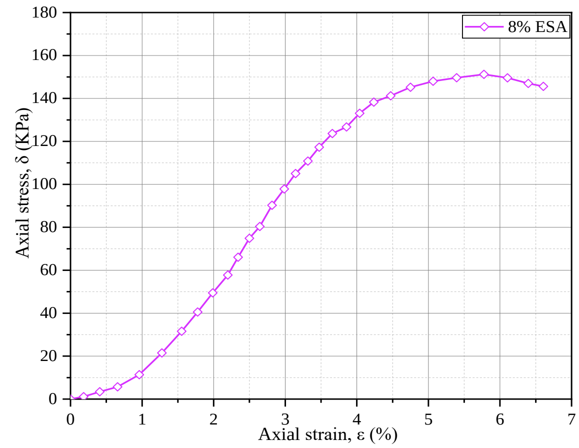


Figure G-12: UCS curve of 8% ESA + soil (q_u (cured) =151.21KPa)

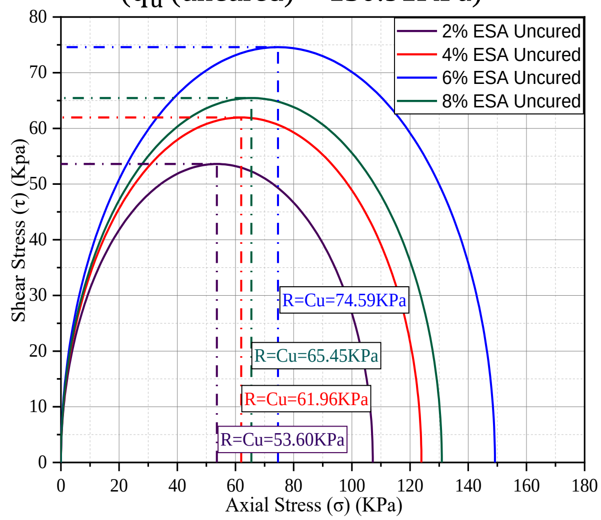


Figure G-13: Mohr circle of soil mixed with ESA (Uncured condition)

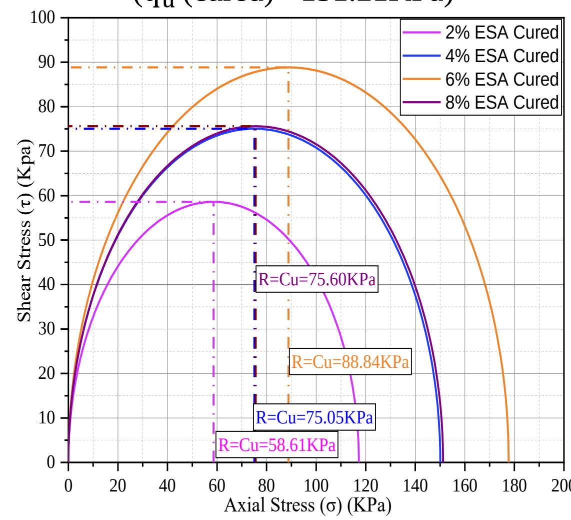


Figure G-14: Mohr circle of soil mixed with ESA (Cured condition)

Table G-12: UCS test result sheet for 5% CSP + 6% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.87	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.377	D (m)		2
OMC (%)	36.09	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.53	0.70	0.009	1142.08	7.88
0.91	1.20	0.020	1147.86	16.99
1.08	1.42	0.030	1150.46	26.08
1.20	1.58	0.041	1152.31	35.15
1.30	1.71	0.051	1153.85	44.20
1.41	1.86	0.066	1155.55	57.12
1.53	2.01	0.075	1157.42	64.80
1.65	2.17	0.086	1159.28	73.75
1.76	2.32	0.098	1161.00	83.98

1.89	2.49	0.111	1163.04	95.44
2.03	2.67	0.128	1165.24	109.42
2.18	2.87	0.140	1167.61	119.48
2.34	3.08	0.156	1170.14	133.32
2.50	3.29	0.167	1172.69	141.98
2.65	3.49	0.176	1175.09	149.35
2.80	3.68	0.186	1177.50	157.96
2.95	3.88	0.194	1179.91	163.99
3.10	4.08	0.198	1182.34	167.46
3.25	4.28	0.201	1184.78	169.65
3.39	4.46	0.204	1187.06	171.85
3.50	4.61	0.206	1188.87	172.85
3.66	4.82	0.204	1191.49	171.21
3.81	5.01	0.200	1193.97	167.09
3.97	5.22	0.195	1196.62	162.96

Table G-13: UCS test result sheet for 5% CSP + 6% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.87	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.377	D (m)		2
OMC (%)	36.09	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.62	0.82	0.003	1143.44	2.97
1.18	1.55	0.017	1152.00	14.76
1.43	1.88	0.034	1155.86	29.42
1.59	2.09	0.051	1158.35	44.03
1.75	2.30	0.071	1160.84	61.51
1.94	2.55	0.095	1163.82	81.80
2.11	2.78	0.117	1166.50	100.56
2.28	3.00	0.133	1169.19	113.41
2.44	3.21	0.146	1171.73	124.77
2.58	3.39	0.158	1173.97	134.67
2.72	3.58	0.168	1176.21	143.09
2.85	3.75	0.182	1178.30	154.37
2.97	3.91	0.190	1180.24	161.32
3.08	4.05	0.196	1182.02	165.40
3.18	4.18	0.201	1183.64	169.48
3.28	4.32	0.211	1185.27	177.85
3.40	4.47	0.218	1187.23	183.28
3.53	4.64	0.223	1189.36	187.24
3.67	4.83	0.230	1191.66	192.59
3.82	5.03	0.233	1194.14	195.04
3.96	5.21	0.236	1196.46	197.50
4.09	5.38	0.240	1198.62	199.98
4.24	5.58	0.241	1201.13	200.98
4.38	5.76	0.247	1203.47	204.82

4.53	5.96	0.245	1206.00	202.99
4.68	6.16	0.245	1208.54	202.56
4.83	6.36	0.241	1211.08	199.33
4.98	6.55	0.238	1213.64	196.10

Table G-14: UCS test result sheet for 10% CSP + 6% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.89	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.392	D (m)		2
OMC (%)	35.86	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.53	0.70	0.013	1142.08	11.56
0.91	1.20	0.038	1147.86	33.06
1.08	1.42	0.101	1150.46	87.49
1.20	1.58	0.140	1152.31	121.71
1.30	1.71	0.162	1153.85	140.14
1.41	1.86	0.180	1155.55	155.64
1.53	2.01	0.195	1157.42	168.22
1.65	2.17	0.211	1159.28	182.18
1.76	2.32	0.224	1161.00	193.28
1.89	2.49	0.238	1163.04	204.29
2.03	2.67	0.251	1165.24	215.23
2.18	2.87	0.262	1167.61	224.69
2.34	3.08	0.271	1170.14	231.25
2.50	3.29	0.279	1172.69	237.79
2.65	3.49	0.282	1175.09	240.11
2.80	3.68	0.285	1177.50	242.42
2.95	3.88	0.281	1179.91	237.73
3.10	4.08	0.277	1182.34	234.45
3.25	4.28	0.271	1184.78	228.40
3.39	4.46	0.262	1187.06	221.01

Table G-15: UCS test result sheet for 10% CSP + 6% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.89	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.392	D (m)		2
OMC (%)	35.86	H/D		2
ΔL (mm)	ε(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.20	0.26	0.011	1137.11	10.03
0.42	0.55	0.032	1140.42	28.32
0.55	0.72	0.076	1142.38	66.53
0.72	0.95	0.143	1144.96	124.46
0.90	1.18	0.192	1147.71	167.20
1.00	1.32	0.215	1149.24	186.82
1.18	1.55	0.247	1152.00	214.41
1.35	1.78	0.268	1154.62	232.02

1.49	1.96	0.289	1156.79	249.66
1.65	2.17	0.304	1159.28	262.23
1.82	2.39	0.319	1161.94	274.71
1.99	2.62	0.329	1164.61	282.24
2.16	2.84	0.338	1167.29	289.73
2.32	3.05	0.346	1169.83	295.60
2.48	3.26	0.350	1172.37	298.20
2.65	3.49	0.353	1175.09	300.74
2.86	3.76	0.348	1178.46	295.05
3.06	4.03	0.344	1181.69	291.02
3.22	4.24	0.336	1184.29	283.97
3.41	4.49	0.329	1187.39	276.83
3.57	4.70	0.323	1190.01	271.43

Table G-16: UCS test result sheet for 15% CSP + 6% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.88	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.395	D (m)		2
OMC (%)	35.01	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.64	0.84	0.007	1143.75	6.30
0.90	1.18	0.018	1147.71	15.68
1.12	1.47	0.047	1151.08	40.66
1.31	1.72	0.090	1154.01	77.99
1.47	1.93	0.133	1156.48	115.18
1.60	2.11	0.167	1158.50	144.50
1.71	2.25	0.198	1160.22	170.66
1.82	2.39	0.223	1161.94	192.09
1.93	2.54	0.241	1163.67	207.28
2.05	2.70	0.257	1165.55	220.84
2.16	2.84	0.270	1167.29	231.30
2.25	2.96	0.277	1168.72	237.18
2.34	3.08	0.283	1170.14	241.51
2.48	3.26	0.292	1172.37	248.73
2.61	3.43	0.299	1174.45	254.42
2.76	3.63	0.304	1176.85	258.49
2.92	3.84	0.308	1179.43	260.97
3.07	4.04	0.313	1181.86	265.01
3.22	4.24	0.315	1184.29	265.98
3.37	4.43	0.320	1186.74	269.98
3.82	5.03	0.315	1194.14	263.79
3.97	5.22	0.310	1196.62	258.73

Table G-17: UCS test result sheet for 15% CSP + 6% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.88	Location	Dukem town
ρ_{dry}max (g/cm^3)	1.395	D (m)	2

OMC (%)	35.01	H/D		2
ΔL (mm)	ε(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.64	0.84	0.019	1143.75	16.52
0.90	1.18	0.055	1147.71	47.57
1.12	1.47	0.101	1151.08	87.57
1.31	1.72	0.143	1154.01	123.74
1.47	1.93	0.197	1156.48	170.69
1.60	2.11	0.244	1158.50	210.27
1.71	2.25	0.284	1160.22	244.35
1.82	2.39	0.321	1161.94	276.52
1.93	2.54	0.347	1163.67	297.77
2.05	2.70	0.368	1165.55	315.30
2.16	2.84	0.389	1167.29	332.82
2.25	2.96	0.401	1168.72	343.20
2.34	3.08	0.412	1170.14	351.75
2.48	3.26	0.422	1172.37	360.04
2.61	3.43	0.428	1174.45	364.77
2.76	3.63	0.439	1176.85	372.94
3.00	3.95	0.447	1180.72	378.84
3.22	4.24	0.454	1184.29	383.01
3.45	4.54	0.460	1188.05	387.11
3.67	4.83	0.466	1191.66	391.22
3.92	5.16	0.460	1195.79	384.60
4.12	5.42	0.454	1199.12	378.28

Table G-18: UCS test result sheet for 20% CSP + 6% ESA + soil (Uncured)

ρ_{bulk} (g/cm^3)	1.85	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.378	D (m)		2
OMC (%)	33.93	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.31	0.41	0.005	1138.76	4.35
0.50	0.66	0.008	1141.63	7.23
0.73	0.96	0.017	1145.11	14.41
0.97	1.28	0.031	1148.78	27.29
1.18	1.55	0.046	1152.00	40.10
1.35	1.78	0.059	1154.62	51.45
1.51	1.99	0.073	1157.10	62.74
1.67	2.20	0.083	1159.60	71.15
1.78	2.34	0.097	1161.31	83.83
1.90	2.50	0.111	1163.19	95.04
2.01	2.64	0.119	1164.92	101.98
2.14	2.82	0.134	1166.97	114.53
2.27	2.99	0.145	1169.03	124.21
2.39	3.14	0.157	1170.94	133.87

2.52	3.32	0.165	1173.01	140.66
2.64	3.47	0.175	1174.93	148.86
2.78	3.66	0.185	1177.17	156.99
2.93	3.86	0.198	1179.59	167.85
3.10	4.08	0.206	1182.34	174.44
3.30	4.34	0.215	1185.59	180.92
3.51	4.62	0.219	1189.03	184.56
3.66	4.82	0.223	1191.49	186.95
3.81	5.01	0.228	1193.97	190.71
3.96	5.21	0.229	1196.46	191.69
4.10	5.39	0.231	1198.79	192.69
4.25	5.59	0.228	1201.29	189.55
4.39	5.78	0.226	1203.64	187.81
4.54	5.97	0.224	1206.17	186.04
4.70	6.18	0.218	1208.87	180.17

Table G-19: UCS test result sheet for 20% CSP + 6% ESA + soil (Cured)

ρ_{bulk} (g/cm^3)	1.85	Location		Dukem town
ρ_{dry}max (g/cm^3)	1.378	D (m)		2
OMC (%)	33.93	H/D		2
ΔL (mm)	ε (%)	P (KN)	A_c (mm2)	σ (KN/m2)
0.00	0.00	0.000	1134.11	0.00
0.50	0.66	0.009	1141.63	7.66
0.87	1.14	0.018	1147.25	15.25
1.15	1.51	0.026	1151.54	22.80
1.39	1.83	0.063	1155.24	54.53
1.50	1.97	0.088	1156.95	75.63
1.67	2.20	0.128	1159.60	110.17
1.81	2.38	0.168	1161.78	144.61
1.97	2.59	0.203	1164.29	174.35
2.15	2.83	0.236	1167.13	202.42
2.31	3.04	0.261	1169.67	222.93
2.47	3.25	0.285	1172.21	243.34
2.61	3.43	0.306	1174.45	260.76
2.76	3.63	0.326	1176.85	276.59
2.92	3.84	0.336	1179.43	284.88
3.07	4.04	0.352	1181.86	297.63
3.23	4.25	0.359	1184.45	302.88
3.39	4.46	0.362	1187.06	305.16
3.56	4.68	0.371	1189.85	311.80
3.72	4.89	0.373	1192.48	312.58
3.88	5.11	0.378	1195.13	316.28
4.05	5.33	0.380	1197.95	317.00
4.22	5.55	0.383	1200.79	319.16
4.38	5.76	0.382	1203.47	317.00
4.55	5.99	0.380	1206.34	314.80

4.73	6.22	0.375	1209.38	309.66

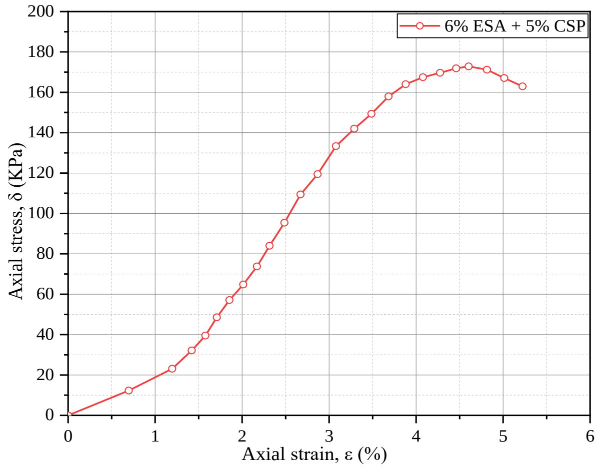


Figure G-15: UCS curve of 5% CSP + 6% ESA + soil (q_u (uncured) = 172.85KPa)

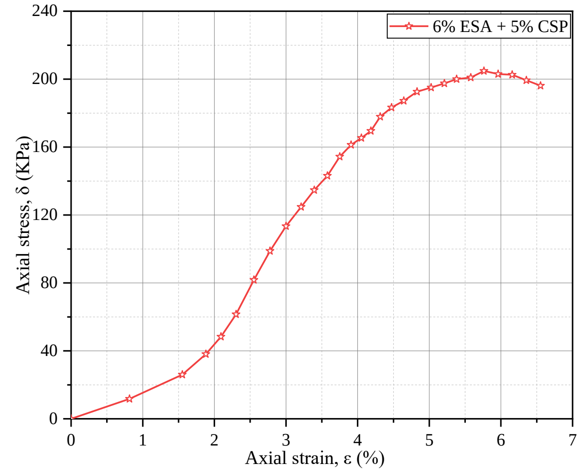


Figure G-16: UCS curve of 5% CSP + 6% ESA + soil (q_u (cured) = 204.82Kpa)

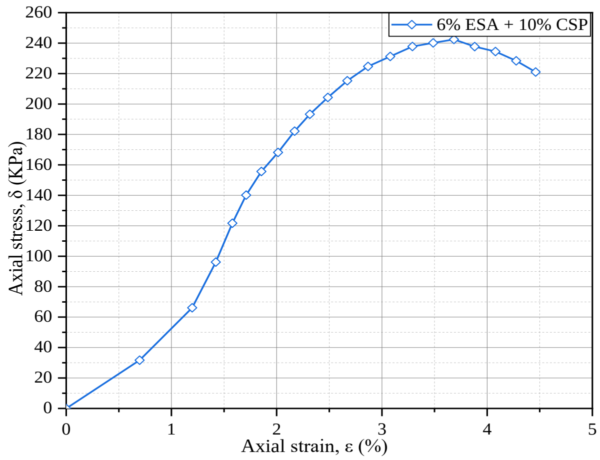


Figure G-17: UCS curve of 10% CSP + 6% ESA + soil (q_u (uncured) = 242.42Kpa)

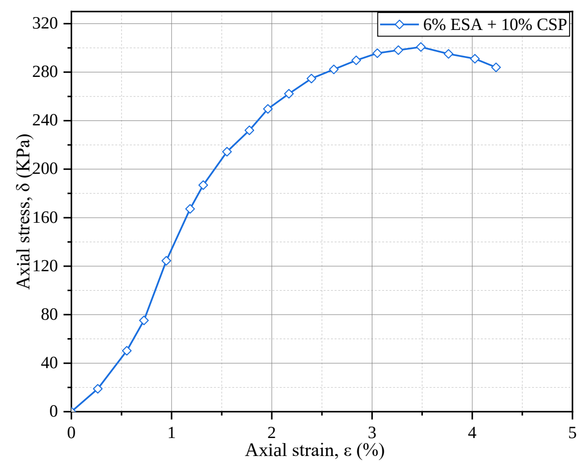


Figure G-18: UCS curve of 10% CSP + 6% ESA + soil (q_u (cured) = 300.74Kpa)

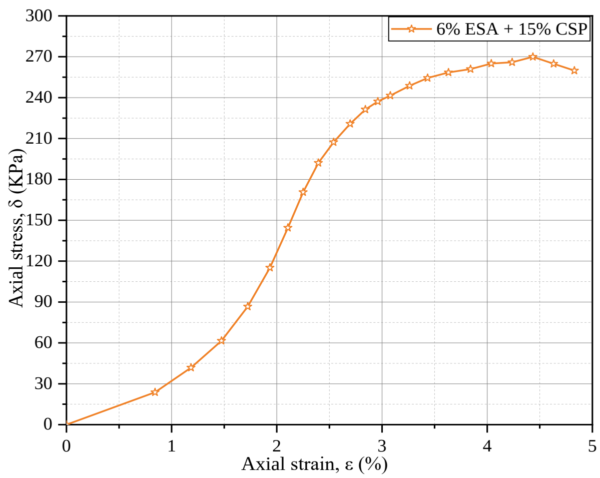


Figure G-19: UCS curve of 15% CSP + 6% ESA + soil (q_u (uncured) = 269.98KPa)

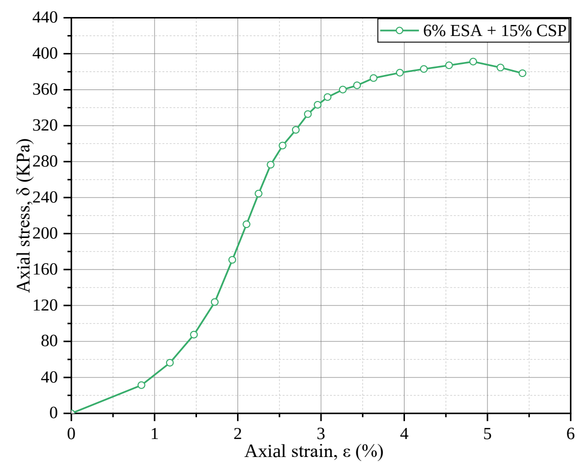


Figure G-20: UCS curve of 15% CSP + 6% ESA + soil (q_u (cured) = 391.22KPa)

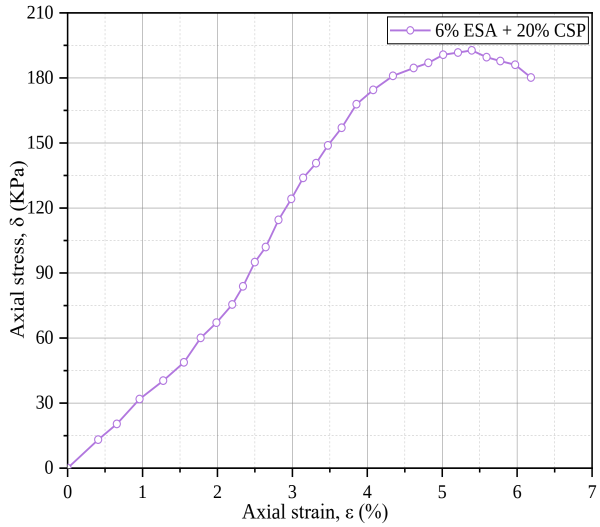


Figure G-21: UCS curve of 20% CSP + 6% ESA + soil (q_u (uncured) = 192.69KPa)

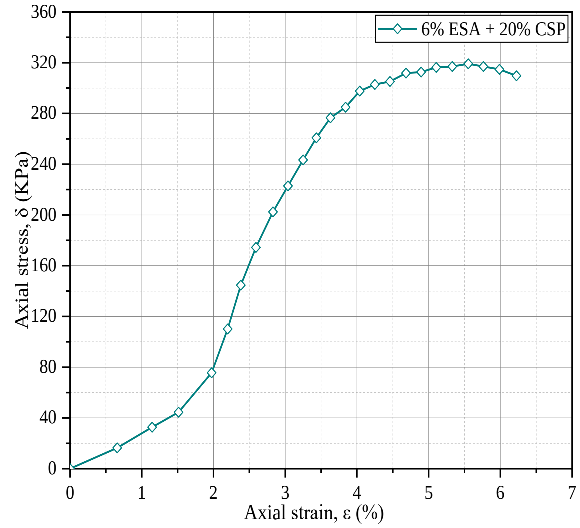


Figure G-22: UCS curve of 20% CSP + 6% ESA + soil (q_u(cured) = 319.16KPa)

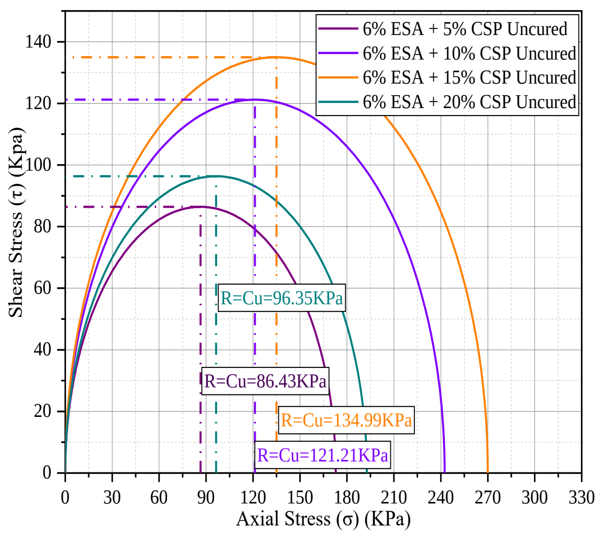


Figure G-23: Mohr circle of soil mixed with 6% ESA and varying CSP (Uncured condition)

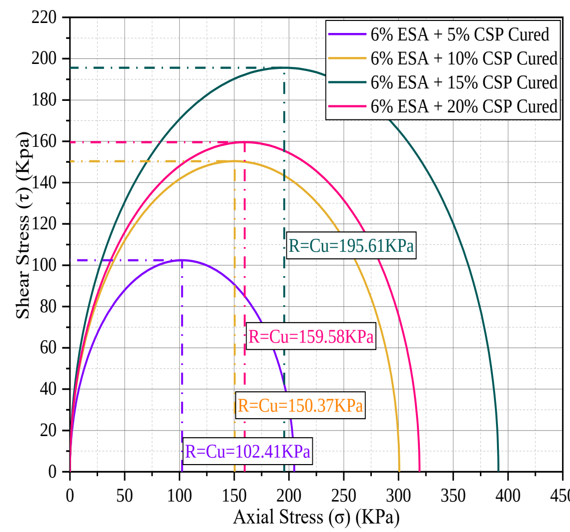


Figure G-24: Mohr circle of soil mixed with 6% ESA and varying CSP (Cured condition)

Appendix-H: Consolidation Test Results

Parameters and formulas used for consolidation test

Weight of specimen ring + soil = M_{rs}

Weight of specimen ring = M_r

Weight of specimen = M_s

Diameter of specimen = D

Initial height of specimen = H

Total volume of specimen = $V = \frac{\pi D^2}{4} * H$

Final moisture content = w_{final}

Initial dry density of specimen = $\rho_{dry} = \frac{\rho_{wet}}{1+w_{initial}}$

Specific gravity = G_s

Density of water = ρ_w

Volume of solid in soil specimen = $V_s = \frac{M_s}{G_s * \rho_w}$

Height of solid soil mass = $H_s = \frac{4V_s}{\pi D^2}$

Initial wet density = $\rho_{wet} = \frac{M_s}{V}$

Initial moisture content = $w_{initial}$

weight of container = w_c

weight of (container + wet soil) = w_{cw}

weight of (container + dry soil) = w_{cd}

Volume of voids in soil specimen = $V_v = V - V_s$

Initial void ratio = $e_0 = \frac{V_v}{V_s}$

weight of water = w_w

weight of soil = w_s

Moisture content = $w = \frac{w_w}{w_s} * 100\%$

S = Natural soil

E6-S = 6% ESA + soil

E6-C5-S = 6% ESA + 5% CSP + soil

E6-C10-S = 6% ESA + 10% CSP + soil

E6-C15-S = 6% ESA + 15% CSP + soil

E6-C20-S = 6% ESA + 20% CSP + soil

Length of drainage path= H_{dr}

Change in void ratio=$\Delta e = \frac{\Delta H*(1+e_0)}{H}$

Void ratio=$e = e_0 - \Delta e$

Deformation at 50% consolidation= d_{50}

Depth of specimen at 50% consolidation=H_{50}

Deformation at 100% consolidation=$\Delta H = d_{100}$

Elapsed time to reach 50% consolidation=t_{50}

Coefficient of consolidation=$C_v = \frac{0.197*{H_{dr}}^2}{t_{50}}$

Table H-1: Moisture content result of samples

Parameters		S	E6-S	E6-C5-S	E6-C10-S	E6-C15-S	E6-C20-S
Initial moisture content	w_c (gm)	27.07	30.06	30.07	30.11	30.11	30.11
	w_{cw} (gm)	66.22	94.8	53.41	67.05	67.05	67.05
	w_{cd} (gm)	55.94	77.2	47.23	57.35	57.47	57.75
	w_w (gm)	10.28	17.6	6.18	9.7	9.58	9.3
	w_s (gm)	28.87	47.14	17.16	27.24	27.36	27.64
	w (%)	35.61	37.34	36.01	35.61	35.01	33.65
Final moisture content	w_c (gm)	30.01	30.01	27.08	29.29	29.29	29.29
	w_{cw} (gm)	118.86	120.64	119.21	124.74	124.74	124.84
	w_{cd} (gm)	94.47	94.69	92.33	96.08	97.08	97.88
	w_w (gm)	24.39	25.95	26.88	28.66	27.66	26.96
	w_s (gm)	64.46	64.68	65.25	66.79	67.79	68.59
	w (%)	37.84	40.12	41.20	42.91	40.80	39.31

Table H-2: Specimen data and properties result

Parameters	S	E6-S	E6-C5-S	E6-C10-S	E6-C15-S	E6-C20-S
M_{rs} (gm)	168.16	168.25	168.14	168.1	168.1	168.1
M_r (gm)	76.36	76.36	76.54	76.57	76.57	76.57
M_s (gm)	91.8	91.89	91.6	91.53	91.53	91.53
D (cm)	6	6	6	6	6	6
H (cm)	2	2	2	2	2	2
V (cm^3)	56.52	56.52	56.52	56.52	56.52	56.52
ρ_{wet} (gm/cm^3)	1.624	1.626	1.621	1.619	1.619	1.619
$w_{initial}$, %	35.61	41.63	35.70	33.65	33.65	33.65
ρ_{dry} (gm/cm^3)	1.198	1.148	1.194	1.212	1.212	1.212
G_s	2.606	2.602	2.61	2.618	2.626	2.634

ρ_w (gm/cm^3)	1	1	1	1	1	1
V_s (cm^3)	24.74	24.86	25.00	25.51	25.81	26.04
H_s (cm)	0.88	0.88	0.88	0.90	0.91	0.92
V_v (cm^3)	31.78	31.66	31.52	31.01	30.71	30.48
e_0	1.285	1.274	1.261	1.216	1.190	1.170
w_{final}, %	37.84	40.12	41.20	42.91	42.91	42.91

Table H-3: Recorded specimen data of natural soil and 6% ESA mixed with soil

Elapsed time	Natural soil				6% ESA + soil			
	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2
	Deformation (mm)				Deformation (mm)			
0.1	0.213	0.64	1.95	3.32	0.183	0.55	1.56	2.87
0.25	0.243	0.73	1.98	3.38	0.197	0.59	1.61	2.88
0.5	0.260	0.75	2.03	3.4	0.203	0.61	1.65	2.92
1	0.273	0.82	2.1	3.435	0.217	0.65	1.721	2.96
2	0.300	0.9	2.163	3.47	0.237	0.71	1.803	3.02
4	0.333	0.998	2.278	3.55	0.253	0.76	1.9	3.101
8	0.365	1.094	2.375	3.67	0.271	0.813	2	3.22
15	0.391	1.172	2.529	3.82	0.286	0.859	2.11	3.31
30	0.417	1.25	2.69	3.952	0.297	0.89	2.22	3.485
60	0.433	1.3	2.81	4.02	0.312	0.937	2.327	3.634
120	0.445	1.335	2.89	4.1	0.32	0.96	2.4	3.75
240	0.453	1.36	2.96	4.19	0.328	0.985	2.45	3.83
480	0.463	1.39	2.99	4.3	0.34	1.02	2.496	3.88
1440	0.477	1.432	3.05	4.36	0.347	1.04	2.531	3.931

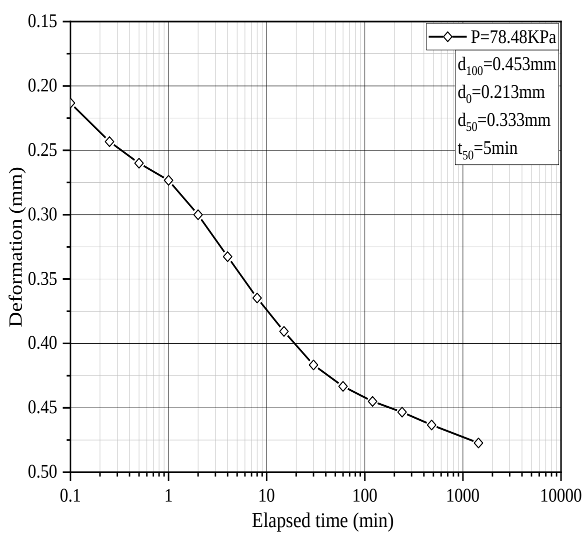


Figure H-1: Deformation versus log time curve of natural soil for P = 0.8kg/cm^2

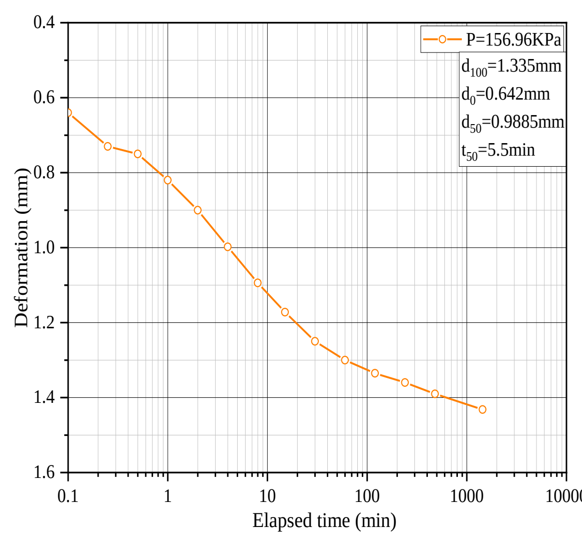


Figure H-2: Deformation versus log time curve of natural soil for P = 1.6kg/cm^2

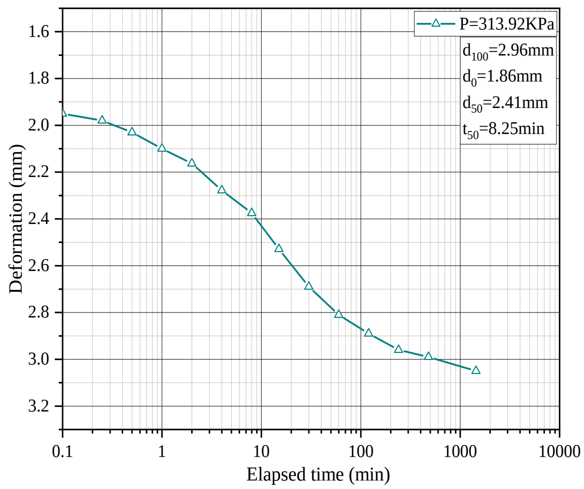


Figure H-3: Deformation versus log time curve of natural soil for P = 3.2kg/cm^2

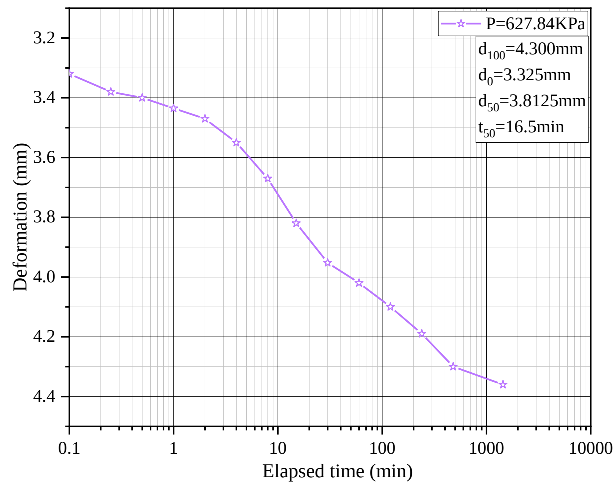


Figure H-4: Deformation versus log time curve of natural soil for P = 6.4kg/cm^2

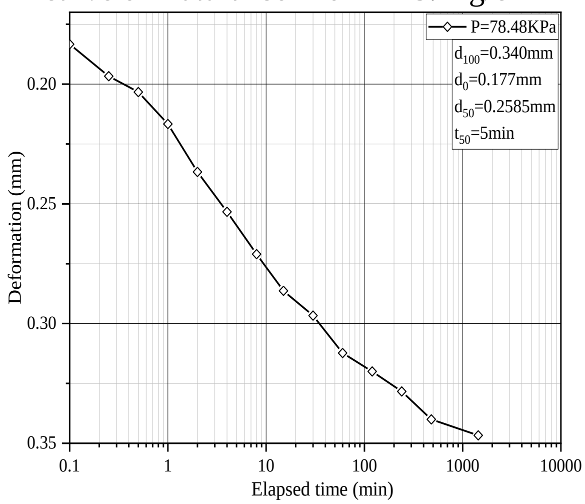


Figure H-5: Deformation versus log time curve of 6% ESA + soil for P = 0.8kg/cm^2

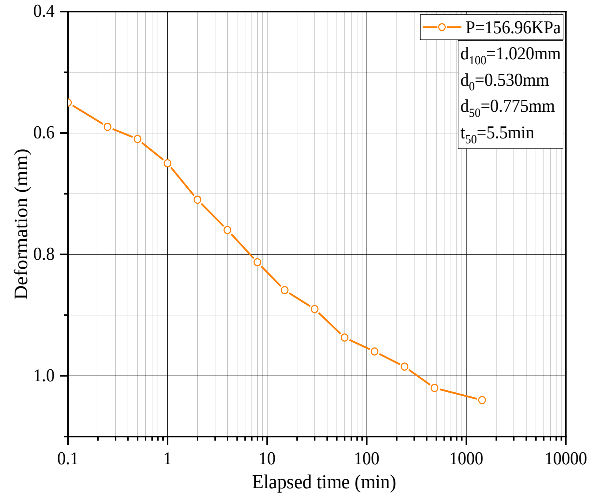


Figure H-6: Deformation versus log time curve of 6% ESA + soil for P = 1.6kg/cm^2

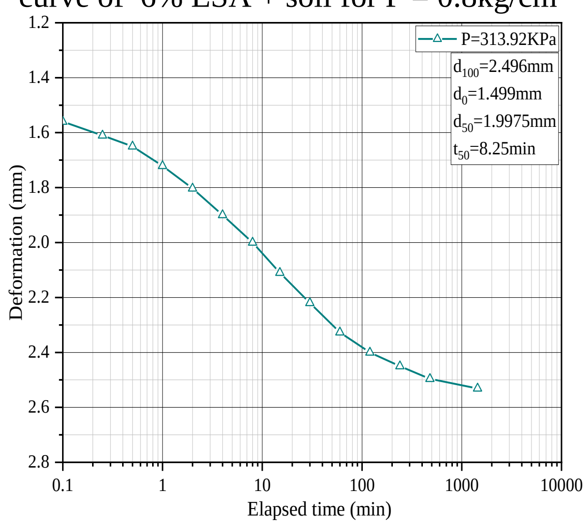


Figure H-7: Deformation versus log time curve of 6% ESA + soil for P = 3.2kg/cm^2

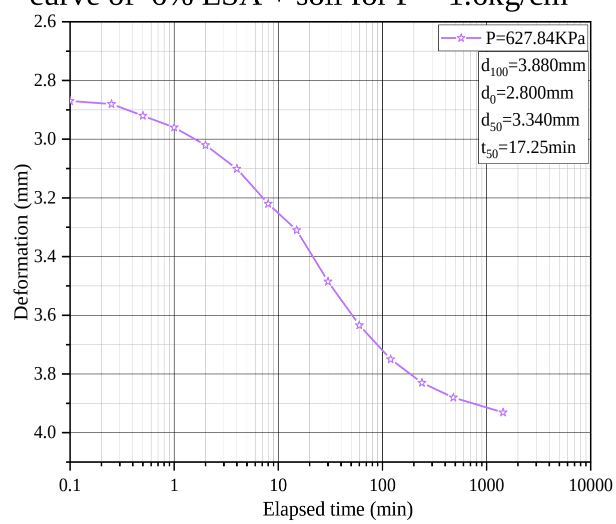


Figure H-8: Deformation versus log time curve of 6% ESA + soil for P = 6.4kg/cm^2

Table H-4: Recorded data of 6% ESA + 5% CSP and 6% ESA + 10% CSP mixed with soil

Elapsed time	6% ESA + 5% CSP + soil				6% ESA + 10% CSP + soil			
	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2
	Deformation (mm)				Deformation (mm)			
0.1	0.157	0.38	1.56	2.75	0.100	0.210	1.008	2.079
0.25	0.169	0.395	1.6	2.84	0.107	0.231	1.050	2.100
0.5	0.174	0.416	1.65	2.9	0.111	0.237	1.103	2.120
1	0.186	0.432	1.7	2.94	0.118	0.247	1.145	2.142
2	0.203	0.455	1.78	2.97	0.129	0.254	1.181	2.174
4	0.217	0.48	1.85	3.02	0.138	0.264	1.230	2.216
8	0.232	0.51	1.94	3.08	0.148	0.273	1.285	2.269
15	0.245	0.542	2.03	3.16	0.156	0.284	1.344	2.342
30	0.254	0.59	2.14	3.27	0.162	0.294	1.422	2.411
60	0.268	0.63	2.25	3.41	0.170	0.309	1.507	2.520
120	0.274	0.66	2.36	3.59	0.175	0.326	1.596	2.663
240	0.281	0.67	2.45	3.75	0.179	0.340	1.670	2.805
480	0.291	0.685	2.51	3.86	0.185	0.357	1.735	2.930
1440	0.297	0.705	2.58	3.94	0.189	0.369	1.785	3.012

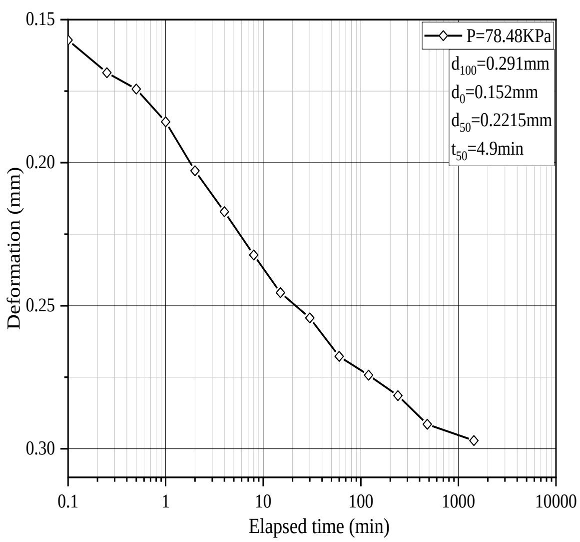


Figure H-9: Deformation versus log time curve of 6%ESA+5%CSP+soil for P = 0.8kg/cm^2

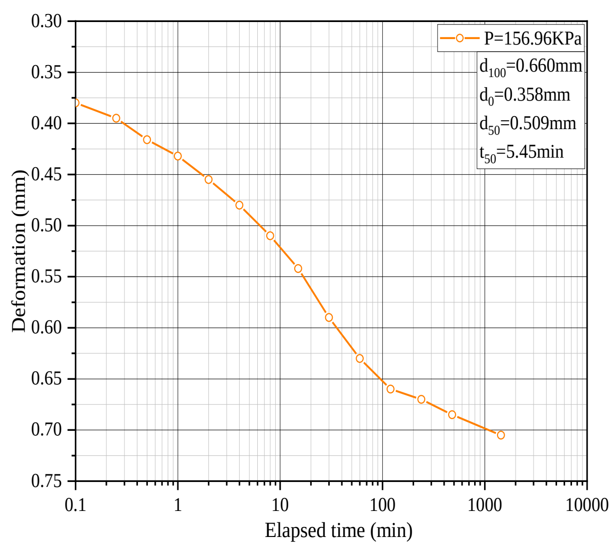


Figure H-10: Deformation versus log time curve of 6%ESA+5%CSP+soil for P = 1.6kg/cm^2

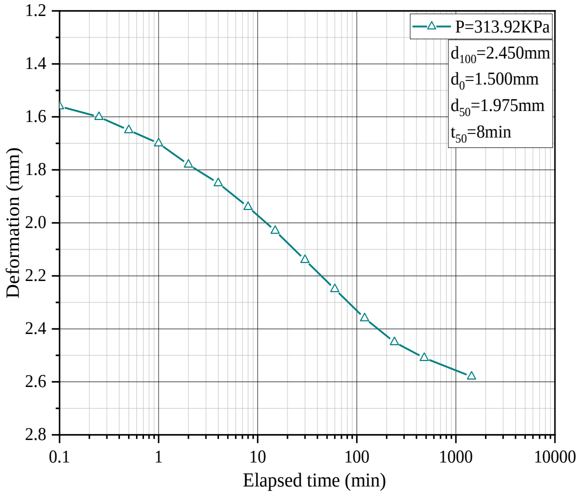


Figure H-11: Deformation vs log t curve of 6%ESA+5%CSP+soil for P = 3.2kg/cm^2

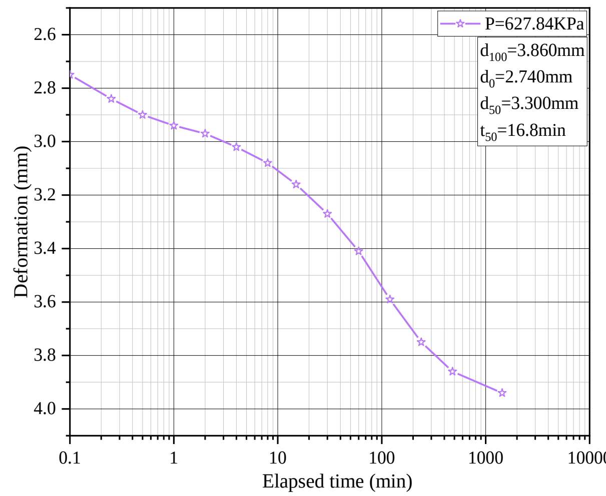


Figure H-12: Deformation vs log t curve of 6%ESA+5%CSP+soil for P = 6.4kg/cm^2

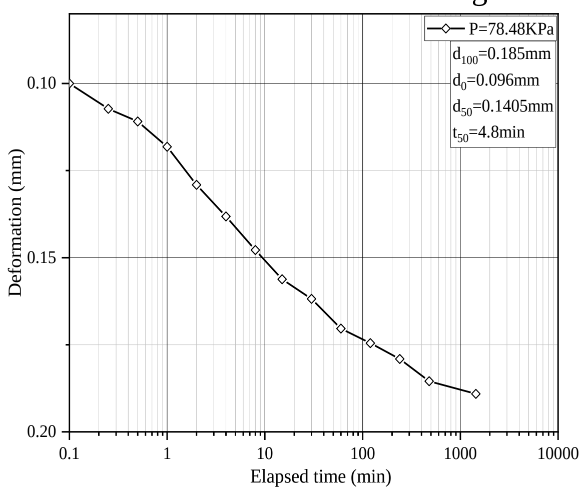


Figure H-13: Deformation vs log t curve of 6%ESA+10%CSP+soil for P= 0.8kg/cm^2

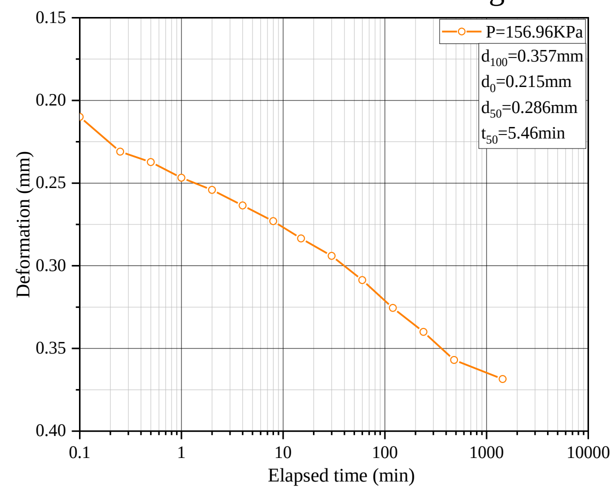


Figure H-14: Deformation vs log t curve of 6%ESA+10%CSP+soil for P= 1.6kg/cm^2

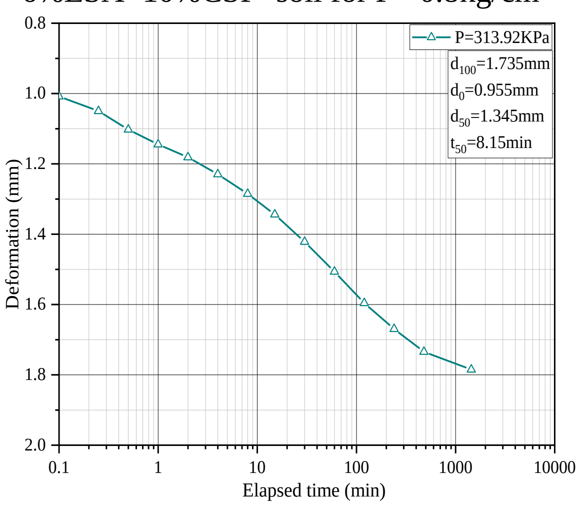


Figure H-15: Deformation vs log t curve of 6%ESA+10%CSP+soil for P= 3.2kg/cm^2

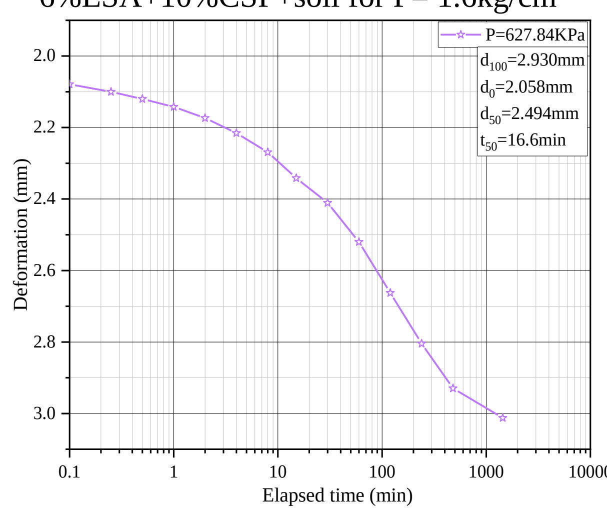


Figure H-16: Deformation vs log t curve of 6%ESA+10%CSP+soil for P= 6.4kg/cm^2

Table H-5: Recorded data of 6% ESA + 15% CSP and 6% ESA + 20% CSP mixed with soil

Elapsed time	6% ESA + 15% CSP + soil				6% ESA + 20% CSP + soil			
	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2	0.8 kg/cm^2	1.6 kg/cm^2	3.2 kg/cm^2	6.4 kg/cm^2
	Deformation (mm)				Deformation (mm)			
0.1	0.110	0.220	1.056	2.178	0.092	0.2	0.96	1.98
0.25	0.118	0.242	1.100	2.200	0.098	0.22	1	2
0.5	0.122	0.249	1.155	2.221	0.102	0.226	1.05	2.019
1	0.130	0.259	1.199	2.244	0.108	0.235	1.09	2.04
2	0.142	0.266	1.238	2.277	0.118	0.242	1.125	2.07
4	0.152	0.276	1.288	2.321	0.127	0.251	1.171	2.11
8	0.163	0.286	1.346	2.377	0.136	0.26	1.224	2.161
15	0.172	0.297	1.408	2.453	0.143	0.27	1.28	2.23
30	0.178	0.308	1.489	2.526	0.148	0.28	1.354	2.296
60	0.187	0.323	1.579	2.640	0.156	0.294	1.435	2.4
120	0.192	0.341	1.672	2.790	0.160	0.31	1.52	2.536
240	0.197	0.358	1.749	2.938	0.164	0.33	1.59	2.671
480	0.204	0.374	1.817	3.069	0.170	0.34	1.652	2.79
1440	0.208	0.386	1.870	3.156	0.173	0.351	1.7	2.869

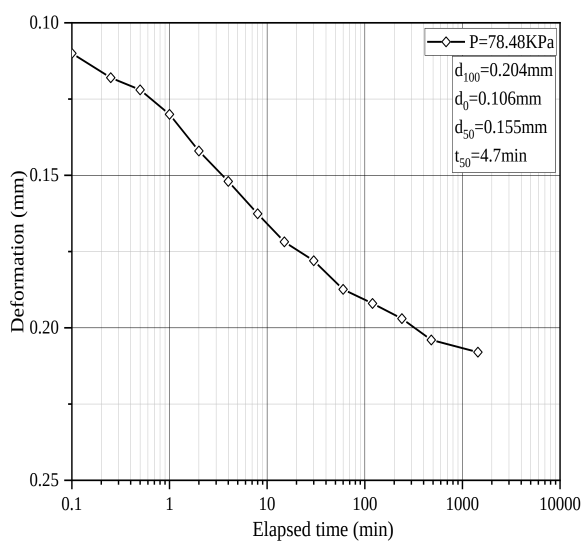


Figure H-17: Deformation vs log time curve of 6%ESA+15%CSP+soil for P= 0.8kg/cm^2

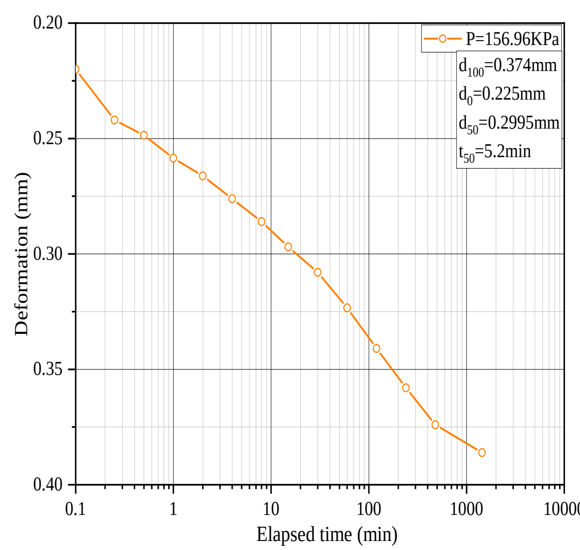


Figure H-18: Deformation vs log time curve of 6%ESA+15%CSP+soil for P= 1.6kg/cm^2

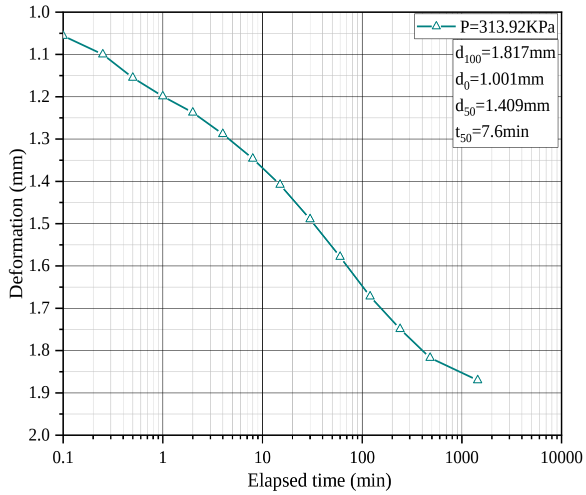


Figure H-19: Deformation vs log t curve of 6%ESA+15%CSP+soil for P= 3.2kg/cm^2

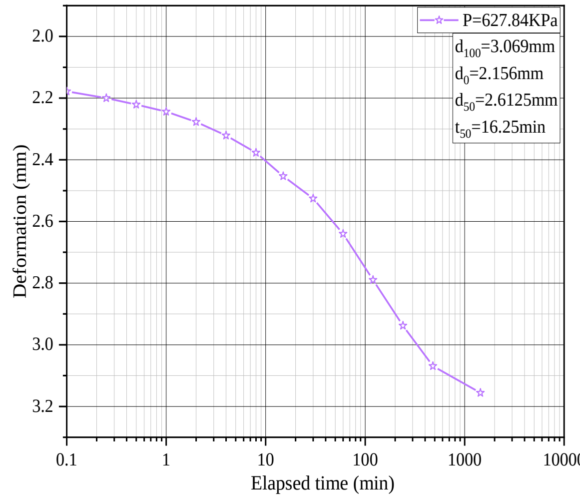


Figure H-20: Deformation vs log t curve of 6%ESA+15%CSP+soil for P= 6.4kg/cm^2

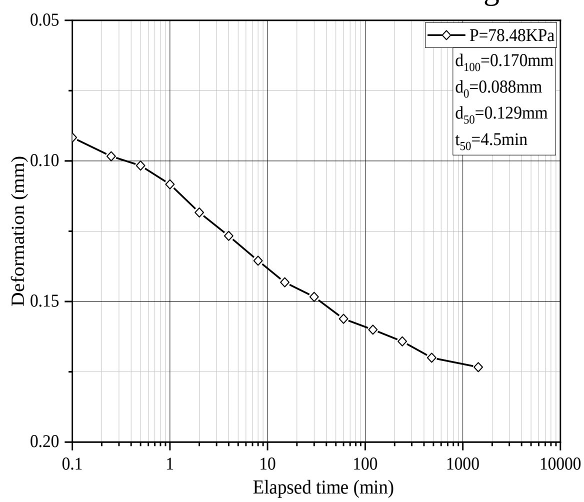


Figure H-21: Deformation vs log t curve of 6%ESA+20%CSP+soil for P= 0.8kg/cm^2

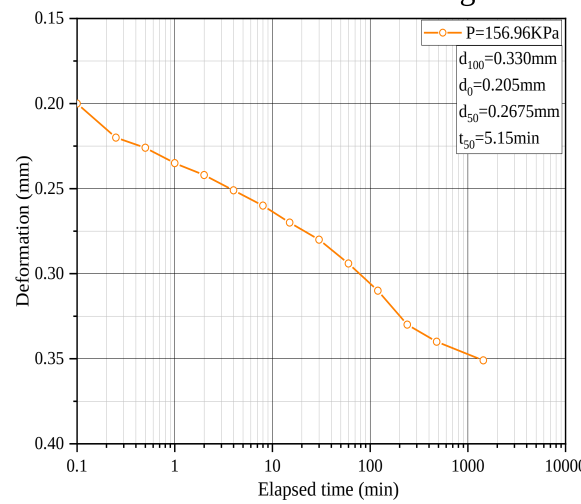


Figure H-22: Deformation vs log t curve of 6%ESA+20%CSP+soil for P= 1.6kg/cm^2

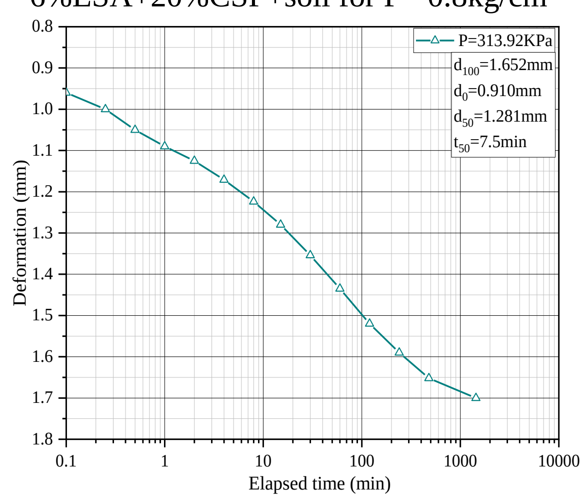


Figure H-23: Deformation vs log t curve of 6%ESA+20%CSP+soil for P= 3.2kg/cm^2

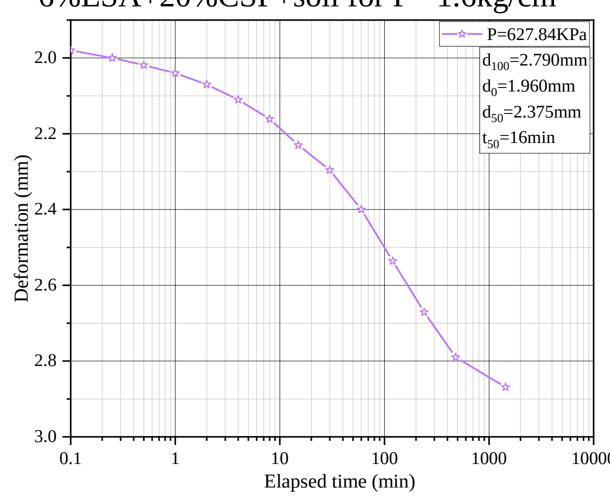


Figure H-24: Deformation vs log t curve of 6%ESA+20%CSP+soil for P= 6.4kg/cm^2

Table H-6: Consolidation test result of natural soil

Conditions	Pressure(Kpa)	$d_{100}=\Delta H$(cm)	H(cm)	e_0	Δe	e	m_v
Loading Case	0	0	2	1.285	0	1.285	-
	78.48	0.0453	2	1.285	0.0518	1.233	0.289
	156.96	0.1335	2	1.285	0.1525	1.132	0.562
	313.92	0.296	2	1.285	0.3382	0.947	0.518
	627.84	0.43	2	1.285	0.4913	0.795	0.212
Unloading case	78.48	0.231	2	1.285	0.2639	1.021	
	156.96	0.341	2	1.285	0.3896	0.895	
	313.92	0.402	2	1.285	0.4593	0.826	
	627.84	0.426	2	1.285	0.4867	0.795	

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	C_v(m^2/year)
78.48	2	0.0333	1.9667	0.98335	5	2.0025
156.96	2	0.09885	1.9012	0.95058	5.5	1.7011
313.92	2	0.241	1.759	0.8795	8.25	0.9708
627.84	2	0.38125	1.6188	0.80938	16.5	0.4111

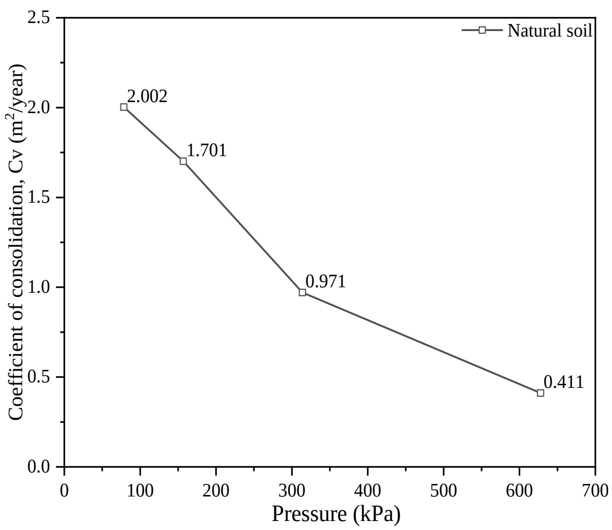


Figure H-25: C_V vs pressure of natural soil

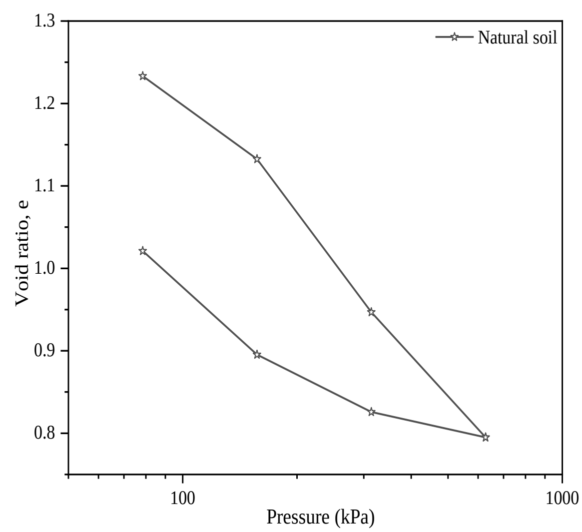


Figure H-26: e vs log p of natural soil

Table H-7: Consolidation test result of soil mixed with 6% ESA

Conditions	Pressure(Kpa)	$d_{100}=\Delta H$(cm)	H(cm)	e_0	Δe	e	m_v
Loading Case	0	0	2	1.274	0	1.274	-
	78.48	0.034	2	1.274	0.0387	1.235	0.218
	156.96	0.102	2	1.274	0.1160	1.158	0.433
	313.92	0.2496	2	1.274	0.2838	0.990	0.470
	627.84	0.388	2	1.274	0.4411	0.840	0.210
Unloading case	78.48	0.294	2	1.274	0.3342	0.939	
	156.96	0.299	2	1.274	0.3399	0.934	

	313.92	0.357	2	1.274	0.4059	0.868
	627.84	0.382	2	1.274	0.4343	0.840

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	$C_v(m^2/year)$
78.48	2	0.02585	1.9742	0.98708	5	2.0177
156.96	2	0.0775	1.9225	0.96125	5.5	1.7395
313.92	2	0.19975	1.8003	0.90013	8.25	1.0169
627.84	2	0.334	1.666	0.833	17.25	0.4165

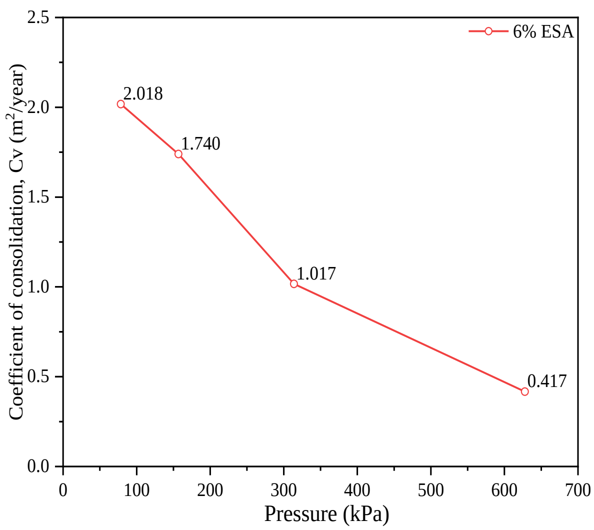
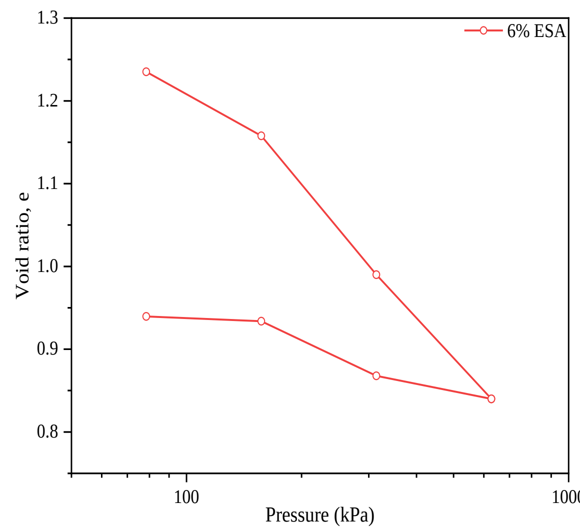


Figure H-27: C_V vs pressure of soil mixed with 6% ESA



FigureH-28: Void ratio vs log pressure of soil mixed with 6% ESA

Table H-8: Consolidation test result of soil mixed with 6% ESA and 5% CSP

Conditions	Pressure(Kpa)	$d_{100}=\Delta H$(cm)	H(cm)	e_0	Δe	e	m_v
Loading case	0	0	2	1.261	0	1.261	-
	78.48	0.0291	2	1.261	0.0329	1.228	0.187
	156.96	0.066	2	1.261	0.0746	1.186	0.235
	313.92	0.245	2	1.261	0.2769	0.984	0.570
	627.84	0.386	2	1.261	0.4363	0.830	0.217
Unloading case	78.48	0.228	2	1.261	0.2577	1.003	
	156.96	0.276	2	1.261	0.3120	0.949	
	313.92	0.343	2	1.261	0.3877	0.873	
	627.84	0.375	2	1.261	0.4239	0.830	

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	$C_v(m^2/year)$
78.48	2	0.02215	1.9779	0.98893	4.9	2.0666
156.96	2	0.0509	1.9491	0.97455	5.45	1.8044
313.92	2	0.1975	1.8025	0.90125	8	1.0513
627.84	2	0.33	1.67	0.835	16.8	0.4297

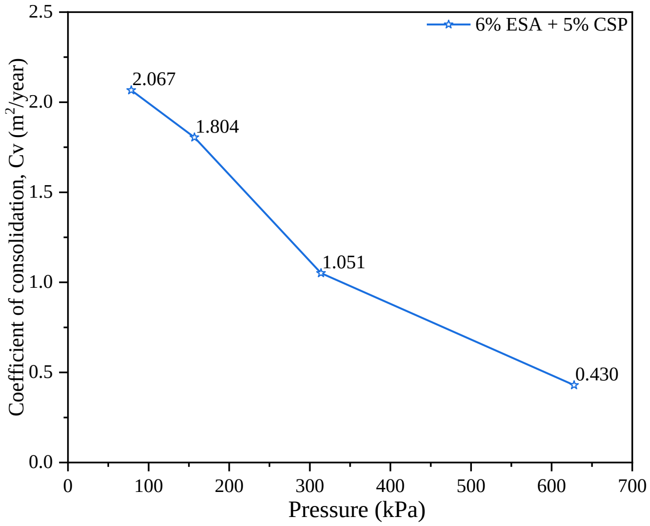


Figure H-29: C_V vs pressure of soil mixed with 6% ESA and 5% CSP

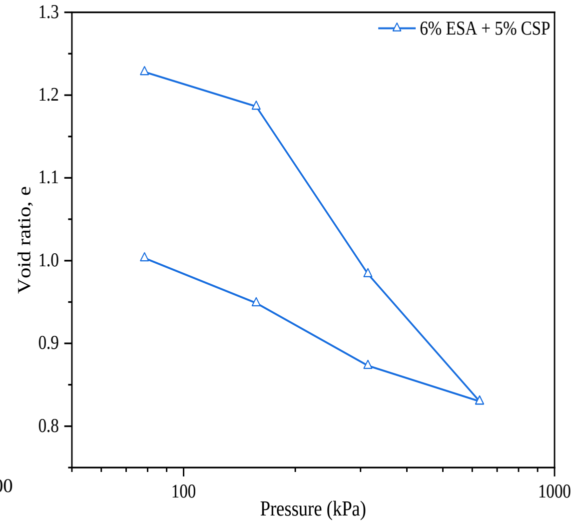


Figure H-30: Void ratio vs log pressure of soil mixed with 6% ESA and 5% CSP

Table H-9: Consolidation test result of soil mixed with 6% ESA and 10% CSP

Conditions	Pressure(Kpa)	d_{100}=ΔH(cm)	H(cm)	e_0	Δe	e	m_v
Loading case	0	0	2	1.216	0	1.216	-
	78.48	0.0185	2	1.216	0.0205	1.195	0.120
	156.96	0.0357	2	1.216	0.0395	1.176	0.110
	313.92	0.1735	2	1.216	0.1922	1.023	0.439
	627.84	0.293	2	1.216	0.3246	0.890	0.192
Unloading case	78.48	0.105	2	1.216	0.1163	1.099	
	156.96	0.167	2	1.216	0.1850	1.031	
	313.92	0.2216	2	1.216	0.2455	0.970	
	627.84	0.2663	2	1.216	0.2950	0.890	

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	C_v(m^2/year)
78.48	2	0.01405	1.9860	0.99298	4.8	2.1269
156.96	2	0.0286	1.9714	0.9857	5.46	1.8425
313.92	2	0.1345	1.8655	0.93275	8.15	1.1053
627.84	2	0.2494	1.7506	0.8753	16.6	0.4779

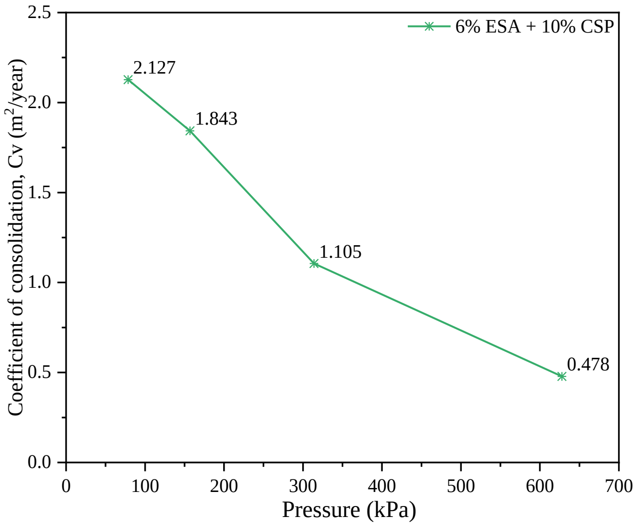


Figure H-31: C_V vs pressure of soil mixed with 6% ESA and 10% CSP

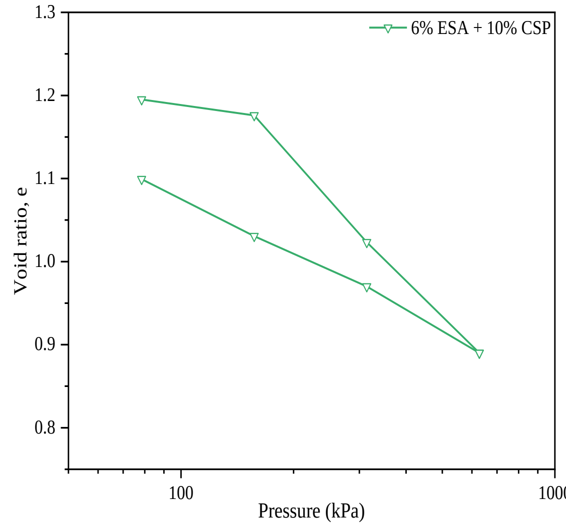


Figure H-32: Void ratio vs log pressure of soil mixed with 6% ESA and 10% CSP

Table H-10: Consolidation test result of soil mixed with 6% ESA and 15% CSP

Conditions	Pressure(Kpa)	d_{100}=ΔH(cm)	H(cm)	e_0	Δe	e	m_v
Loading case	0	0	2	1.190	0	1.190	-
	78.48	0.0204	2	1.190	0.0223	1.167	0.133
	156.96	0.0374	2	1.190	0.0409	1.149	0.108
	313.92	0.1817	2	1.190	0.1989	0.991	0.460
	627.84	0.3069	2	1.190	0.3360	0.884	0.155
Unloading case	78.48	0.1056	2	1.190	0.1156	1.074	
	156.96	0.1579	2	1.190	0.1729	1.017	
	313.92	0.2377	2	1.190	0.2602	0.929	
	627.84	0.279	2	1.190	0.3054	0.884	

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	C_v(m^2/year)
78.48	2	0.01550	1.9845	0.99225	4.7	2.1690
156.96	2	0.02995	1.9701	0.98503	5.2	1.9320
313.92	2	0.14090	1.8591	0.92955	7.6	1.1772
627.84	2	0.26125	1.7388	0.86938	16.25	0.4816

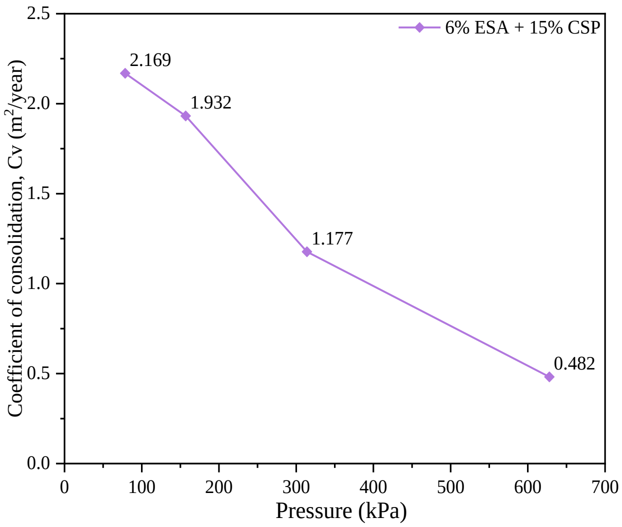


Figure H-33: C_V vs pressure of soil mixed with 6% ESA and 15% CSP

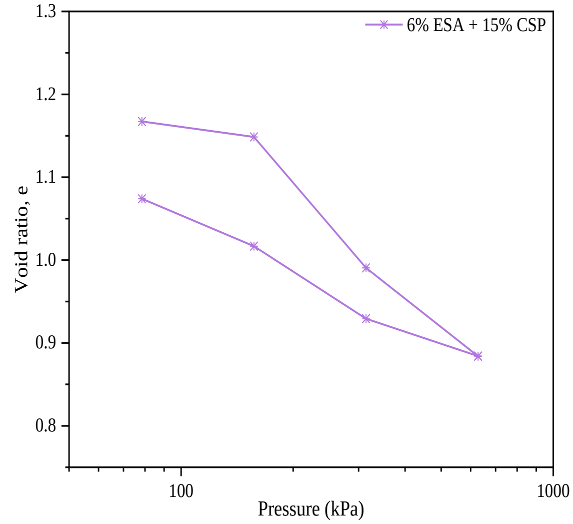


Figure H-34: Void ratio vs log pressure of soil mixed with 6% ESA and 15% CSP

Table H-11: Consolidation test result of soil mixed with 6% ESA and 20% CSP

Conditions	Pressure(Kpa)	d_{100}=ΔH(cm)	H(cm)	e_0	Δe	E	m_v
Loading case	0	0	2	1.170	0	1.170	-
	78.48	0.017	2	1.170	0.0184	1.152	0.105
	156.96	0.033	2	1.170	0.0358	1.135	0.102
	313.92	0.1652	2	1.170	0.1793	0.991	0.421
	627.84	0.279	2	1.170	0.3028	0.870	0.178
Unloading case	78.48	0.102	2	1.170	0.1107	1.060	
	156.96	0.1585	2	1.170	0.1720	0.998	
	313.92	0.2362	2	1.170	0.2563	0.914	
	627.84	0.271	2	1.170	0.2941	0.870	

Coefficient of consolidation, C_v Calculation

Pressure(KPa)	H(cm)	d_{50}(cm)	H_{50}	H_{dr}	t_{50}(min)	C_v(m^2/year)
78.48	2	0.0129	1.9871	0.99355	4.5	2.2714
156.96	2	0.02675	1.9733	0.98663	5.15	1.9571
313.92	2	0.1281	1.8719	0.93595	7.5	1.2094
627.84	2	0.2375	1.7625	0.88125	16	0.5026

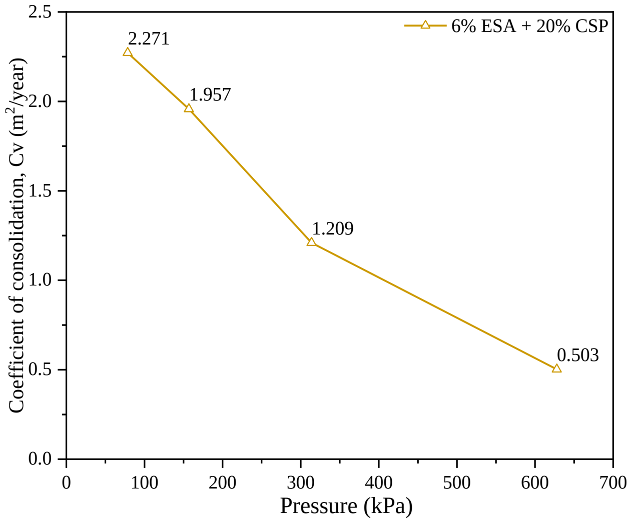


Figure H-35: C_V vs pressure of soil mixed with 6% ESA and 20% CSP

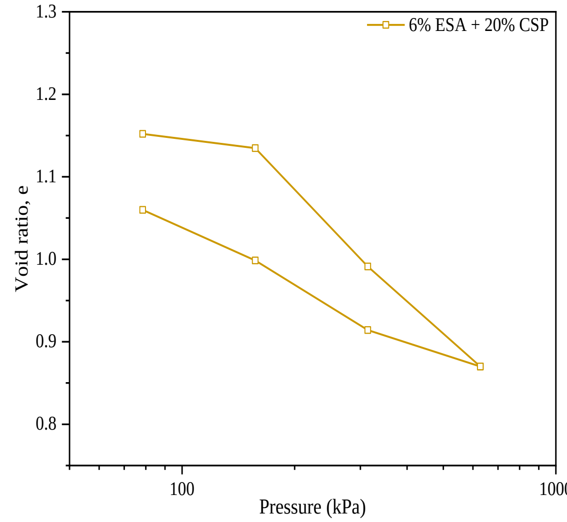


Figure H-36: Void ratio vs log pressure of soil mixed with 6% ESA and 20% CSP