A STUDY ON SUBGRADE SOIL TREATED WITH CATTLE BONE ASH AND WHEAT HUSK ASH: IN CASE OF ALI-AGARFA WABE ROAD PROJECT, SOUTH EAST ETHIOPIA

Henok Shimelis Alemu

A Thesis Submitted to the Department of Civil Engineering,

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Masters in Geotechnical Engineering

Office of Graduate Studies

Adama Science and Technology University

June, 2023

Adama, Ethiopia

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DECLARATION

I declare that this final thesis entitled "A study on subgrade soil treated with cattle bone ash and wheat husk ash: in case of Ali-Agarfa Wabe road project, South East Ethiopia" is my work and has not been submitted to any university for a similar purpose. The references used in this thesis are duly recognized by proper citations.

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I, the advisor of this thesis, hereby certify that I have closely advised the student while developing this final thesis and read the draft thesis entitled “A study on subgrade soil treated with cattle bone ash and wheat husk ash: in case of Ali-Agarfa Wabe road project” prepared under my guidance by Henok Shimelis. Therefore, I recommend the submission of the thesis to the department for further review and evaluation.

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APPROVAL PAGE

I hereby certify that the recommendation and suggestion given by the thesis review committee are appropriately incorporated into the final thesis entitled "A study on subgrade soil treated with cattle bone ash and wheat husk ash: in case of Ali-Agarfa Wabe road project" by Henok Shimelis.

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We, the undersigned, members of the board of reviewers of the final thesis open defense by Henok Shimelis have read and evaluated the thesis entitled "A study on subgrade soil treated with cattle bone ash and wheat husk ash: in case of Ali-Agarfa Wabe road project" and assessed the understanding of the candidate about the research. This is, therefore, to certify that the thesis is accepted and we recommend the implementation of the final thesis.

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ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of States Highway and Transport Officials
ASTM	American Society of Testing and Materials
BS	Black Soil
CBA	Cattle Bone Ash
CBR	California Bearing Ratio
CC	Coefficient of Curvature
CH	High plasticity Clay
CL	Low plasticity Clay
CCCC	China Communications Construction Corporation
CU	Coefficient of Uniformity
ES	Expansive soil
ERA	Ethiopian Roads Authority
EGS	Ethiopian Geological Survey
FSI	Free Swell Index
FSR	Free Swell Ratio
Gs	Specific gravity
LL	Liquid Limit
LS	Linear Shrinkage
MDD	Maximum Dry Density
MH	High plasticity Silt
ML	Low Plasticity Silt
OH	High plasticity Organic
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
PSD	Particle size distribution
UCS	Unconfined Compressive strength
WHA	Wheat Husk Ash

ABSTRACT

The soils that cause effective damages to the pavement structures are expansive soil. These damages occur due to poor or unfavorited engineering properties, such as low bearing capacity, high liquid limit, high compressibility, high volume changes, etc. The roads on this type of subgrade soil fail before their expect design life. To minimize damage and overall costs, it is essential to improve the strength properties of such soils through the use of soil stabilization methods. In this study, alternative local waste materials, cattle bone ash (CBA) and wheat husk ash (WHA), were used as additives to help improve the properties of expansive soil from Bale Agarfa. The weakest soil samples were selected based on their plasticity, swelling properties, CBR, and compressive strength. The research also evaluated the effects of blending CBA and WHA at various percentages (2.5%, 5%, 7.5%, and 10%) on the soil's index and engineering properties, such as Atterberg limits, linear shrinkage, free swelling, compaction, CBR, and unconfined compressive strength. The results showed a decrease in liquid limit, plastic index, free swelling index (FSI), free swell ratio (FSR), and linear shrinkage when CBA and WHA were blended up to 7.5%. Conversely, maximum dry density, CBR, and UCS increased with the addition of both CBA and WHA. Based on the overall test results, the optimum ratio of the blended CBA and WHA for improving expansive soil in this study was proposed to be 7.5%.

Keywords*:* Cattle bone ash, California bearing ratio, Consistency, Unconfined compressive strength, Wheat husk ash.

CHAPTER ONE
INTRODUCTION

1.1 Background of the Study

Many road pavements are constructed on subgrades that are not suitable for the soil conditions present at a specific site. This is often due to a lack of appropriate land for construction, leading to homes and roads being built on land that cannot bear the required traffic load and structural weight. As urban areas continue to grow, the need to build on land with unfavorable soil conditions becomes increasingly unavoidable (Canakci *et al*., 2015).

One type of soil that presents difficulties for construction is clay. Clay is a natural material made up of fine-grained minerals that exhibit plasticity when wet but harden permanently when fired (Ochieng, 2016). One characteristic of clay soil that makes it problematic for construction is its tendency to swell and shrink with changes in the volume of the soil, leading to heaving and cracking when subjected to traffic loads on roads and other structures (Ariana, 2016).

To address these issues, road construction on clay-rich subgrades, such as black cotton soil, often involves excavating and replacing the problematic soil with more suitable materials. This process, however, can be costly and time-consuming. Consequently, various stabilization methods have been developed to improve the soil's load-bearing capacity, using chemical stabilizers like cement and lime. Yet, these conventional stabilizers have environmental impacts, leading researchers to explore more eco-friendly and sustainable alternatives.

A potential solution comes in the form of wheat husk ash (WHA) and cattle bone ash (CBA), which are proposed to be mixed with expansive clay soils to improve their characteristics for construction purposes. WHA and CBA are waste materials generated from agricultural and slaughter industries, and their disposal can be challenging.

When combined in specific proportions, WHA and CBA interact chemically because of the presence of silicon (Brooks *et al*., 2019) and calcium oxides (Adeboje *et al*., 2017), respectively. This chemical reaction can improve the soil's properties, making it more suitable for construction and reducing the need for excavation and replacement of the soil.

By utilizing these waste materials from agriculture and the slaughter industry, researchers aim not only to help alleviate disposal problems but also to enhance the engineering properties of problematic soils in a more environmentally friendly and cost-effective manner.

In summary, the construction industry frequently faces challenges when building on sites with unfavorable soil conditions. The use of agricultural and industrial waste materials, such as WHA and CBA, can provide a more sustainable and environmentally friendly solution to improve these soils and reduce the need for expensive and labor-intensive excavation and replacement processes. This innovative approach has the potential to make construction on marginal lands more feasible while also addressing waste disposal problems in other industries

1.2 Statement of the Problem

The Ali-Agarfa-Wabe Road Project is currently facing significant challenges in its construction process due to the presence of unsuitable subgrade soil material. This soil, which forms the foundation of the road, requires removal and replacement with higher quality material to ensure the longevity and stability of the road. This need for remediation not only significantly increases the overall construction costs but also necessitates the excavation of additional borrow areas to obtain suitable replacement materials. This further excavation contributes to the depletion of nonrenewable natural resources, exacerbating concerns over environmental sustainability. Moreover, the disposal of the removed soil presents its own set of difficulties, adding another layer of complexity to the project. To overcome these challenges, a remedial solution must be devised which addresses the issues of soil quality and resource depletion while also promoting sustainable and efficient road construction practices.

1.3 Research Questions

- ✓ What are the geotechnical properties of the soil at Ali-Agarfa Wabe road project?
- ✓ How does the addition of cattle bone ash and wheat husk ash to the natural soil affect its engineering properties?
- ✓ What proportions of cattle bone ash and wheat husk ash create the optimal blend for improving soil properties in Ali-Agarfa Wabe road project?

1.4 Objectives

1.4.1 General objective

The general objective of this thesis is to study a subgrade soil treated with cattle bone ash and wheat husk ash: In Case of Ali-Agarfa Wabe Road Project, South East Ethiopia.

1.4.2 Specific objective

- ✓ To study the engineering properties of the soil at Ali-Agarfa Wabe road project.
- ✓ To evaluate the effectiveness of blending cattle bone ash and wheat husk ash with the natural soil in terms of improving soil stability and strength.
- ✓ To determine the optimal blend ratio of cattle bone ash and wheat husk ash required to achieve the desired improvements in soil properties for construction projects in Ali-Agarfa Wabe road project.

1.5 Significance of the Study

In Ethiopia, numerous studies have been conducted on enhancing the performance of problematic subgrade soil through chemical stabilization techniques, such as cement, lime, marble dust, and fly ash. However, the increasing cost of these materials due to their use in various industries has led to the need for alternative solutions. As a result, blending locally available agricultural waste, specifically WHA and CBA, has been proposed as a more economical method to improve subgrade soil performance. This study, which focuses on the combined effect of WHA and CBA, is not only significant for the Ali Agarfa Wabe road project but also offers potential benefits for other regions with similar soil types and can serve as reference material for future research.

1.6 Scope of the Study

This research aims to examine the impact of combining cattle bone ash and wheat husk ash on the geotechnical properties of natural subgrade soil. This quantitative experimental study investigates the effect on the engineering characteristics of the subgrade soil. Soil samples were collected from four pits of the Ali-Agarfa-Wabe road project, at depths ranging from 0.3 to 0.9 meters below the surface.

1.7 Limitation of the Study

The unconfined compressive strength is solely utilized to evaluate the shear strength of fine-grained soil. Soil specimens are collected from a maximum of four pits, and the influence of curing is not considered in the unconfined compressive strength assessment.

1.8 Organization 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. 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.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

Soil can be described as a complex and varied substance containing diverse minerals that originate from the disintegration or weathering of parent rock due to various physical forces (Bethel, 2021). The geotechnical properties of soil are influenced by the parent rock, the extent of weathering, and the climate in the region (Tiwari *et al*., 2020). These factors can act alone or together to create earth materials that may pose significant design and construction challenges due to water absorption and the resulting increase in the material's volume. The phrase "expansive material" refers to any earth materials that exhibit substantial volume changes when exposed to water (Antenane, 2020).

Expansive soils, which are usually clayey, experience significant volume fluctuations in response to variations in soil moisture content. When the soil's moisture content increases, expansive soil swells (increases in volume), and when the moisture content decreases, it contracts (reduces in volume) (Hosseini, 2020).

2.2 Mineralogy of clay soil

Soils rich in clay content are known as expansive soils. They typically contain three prevalent types of clay minerals. These clays are composed of silicates that contain two fundamental components: silicate tetrahedron and aluminum or magnesium octahedron. The diameter of clay particles is usually less than 2 microns (0.002 mm). The structure of clay minerals is formed by combining tetrahedral and octahedral layers and sheets. One can think of a tetrahedron sheet as a layer of silicon ions situated between an oxygen layer and a hydroxyl ion layer.

I. Kaolinite

Kaolinite is a stable clay with minimal swelling and shrinking properties. It is composed of silica and gibbsite sheets, which are firmly connected by shared oxygen ions, as depicted in figure 2.1. The chemical bonds within each sheet are covalent bonds, making them quite strong. The sheets of kaolinite are held together by weaker hydrogen bonds between the silica sheet's oxygen and the alumina sheet's hydroxyl groups. Consequently, the particles have well-defined cleavage parallel to the sheets. However, the hydrogen bonds are robust enough to provide significant reinforcement between the sheets, allowing the kaolinite flakes to grow relatively large in nature (often exceeding 100 sheets in thickness). Water cannot penetrate between the sheets to cause expansion or contraction of the particles, making the mineral relatively stable.

II. Illite

As depicted in Figure 2.1, each individual illite sheet comprises an octahedral layer sandwiched between two tetrahedral layers. The sheets feature exceptionally robust covalent bonds. The octahedral layer can include Al, Mg, Fe, or other cations, while in the tetrahedral layer, 1 out of every 7 Si^{4+} ions is replaced by an Al^{3+} ion, resulting in a significant positive charge deficit. Additionally, the octahedral layer's insufficient charges contribute to this deficiency. As a result, not only must the illite sheet adsorb cations to reconcile broken bonds at the sheet's edges, but it must also adsorb cations to equalize charge deficiencies within each sheet. To counteract the positive charge deficit in the tetrahedral layers, potassium ions (K^+ ions) are introduced between the two layers.

III. Montmorillonite

The swelling behavior of clay soils can be attributed to the presence of the montmorillonite mineral, which has an expandable framework structure. This structure allows water to be easily absorbed, causing the clay to expand. The specific surface area (surface area per unit mass) of montmorillonite is also greater than other clay minerals. Montmorillonite's structure consists of an octahedral layer sandwiched between two tetrahedral layers. The charge deficiency in montmorillonite arises from the octahedral layer and must be balanced in a similar manner as in illite, i.e., by incorporating cations between the montmorillonite sheets. However, due to the larger distance between the charge deficit and the cation in montmorillonite compared to illite, the bond between montmorillonite sheets is significantly weaker than that of illite sheets.

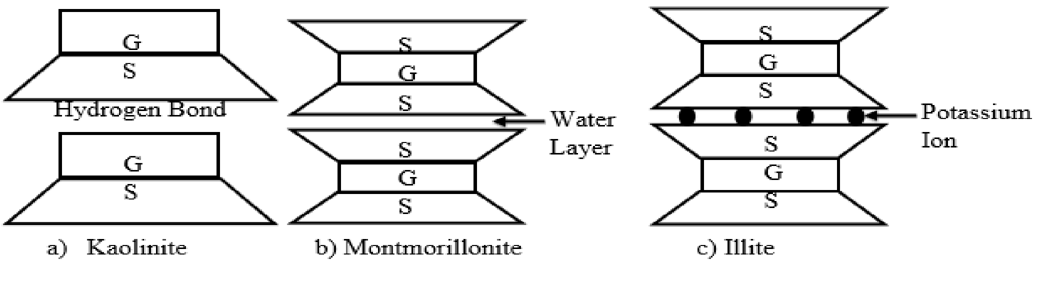


Figure 2.1 Idealized illustration of clay minerals redrawn from (Busscher, 1994)

2.3 Identification of Expansive Soil

Identification of expansive soil usually consists of two important phases. The first is the visual/field identification and the second is sampling and measurement of material properties to be used as the basis for design (keftaw, 2021).

2.3.1 Field identification

Site reconnaissance is a crucial phase in all construction projects, including buildings, roads, railways, and more. The primary goal of the site visit is to conduct a thorough examination and survey of the area to gather adequate information about the location where the structure will be built. During this phase, geotechnical engineers can visually detect the presence of potentially expansive soils along the project section, allowing them to establish preconditions for additional laboratory investigations. According to (Lancellotta, 1995), several key indicators can help identify expansive soils during field investigations, which experienced engineers frequently employ. Some of these indicators include:

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

2.3.2 Laboratory identification

Generally, there are three different methods of identifying expansive soil in the laboratory. These are mineralogical identification, indirect methods and direct methods.

A. Mineralogy identification

(Tseganeh, 2012) suggested that mineralogical identification can be useful in the evaluation of the material but is not sufficient in itself, which is for better and reliable results it should be used in combination. The fundamental factor controlling expansive soil behavior are clay mineralogy and it can be identified using a variety of techniques. The techniques that can be used are X-ray diffraction, chemical analysis, differential thermal analysis, and Scanning electron microscopy.

The various methods of mineralogical identification are important in a research laboratory to explore the basic properties of clay, but it is not suitable for routine tests because it is time consuming, requires expansive test equipment and, the results are interpreted by specially trained technicians

B. Indirect methods

Indirect methods in which one or more of the related intrinsic properties are measured and complemented with experience to provide indicators of potential volume change of expansive clay soil (Dennis, 1994). These methods include, such as the index property, potential volume change (PVC) method, and activity method, etc.

c. Direct methods

These methods present the most useful data by direct measurement; which involves actual measurement of volume change in an odometer type testing (swell pressure test) apparatus. These tests are required to get measurable properties that can be used to predict or estimate the magnitude of volume change (keftaw, 2021). To avoid erroneous conclusions, testing should be done on a variety of samples.

2.4 Classification of Expansive Soil

The categorization of various soils with similar attributes into groups and subgroups is based on their engineering properties. Classification systems offer a concise overview of the general characteristics of diverse soils, without providing in-depth descriptions. Parameters from tests identifying expansive soils have been incorporated into several classification methods to evaluate the likelihood of expansion qualitatively. However, a standardized classification procedure has not been established yet, resulting in the utilization of different schemes in different locations (Zumrawi, 2015). A few classification methods mentioned in the literature are discussed in the subsequent section.

2.4.1 Classification of expansive soil by AASHTO and USCS

I. AASHTO

The AASHTO system of soil classification was developed in 1929 as the Public Road Administration Classification System (Labena, 2020). This system of classification takes into account grain size, liquid limit and plasticity characteristics of soils. According to the system, soil is classified into seven major groups from A-1 through A-7.

Table 2.1 AASHTO soil classification system

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

Table 2. 2 Swelling potential of soil as per USCS

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

II. USCS

The USCS was initially created by Casagrande and later adapted by the USBR and the US Army Corps of Engineers, expanding its usefulness across various disciplines. According to the USCS, soils with group symbols CL or OH have potential expansiveness and exhibit significant volume changes when moisture fluctuates (Tseganeh, 2012).

Where, CH refers to inorganic high plasticity clays (fat clays), CL denotes inorganic low plasticity clays, MH signifies inorganic high plasticity silts, ML represents inorganic low plasticity silts, OL means organic low plasticity silts, OH stands for organic high plasticity clays, Pt alludes to peat and other highly organic soils, etc. Furthermore, SC corresponds to clayey sands, SM refers to silty sands, SW indicates well-graded sands, SP represents poorly-graded sands, GC designates clayey gravels, GM means silty gravels, GW stands for well-graded gravels, and GP distinguishes poorly-graded gravels.

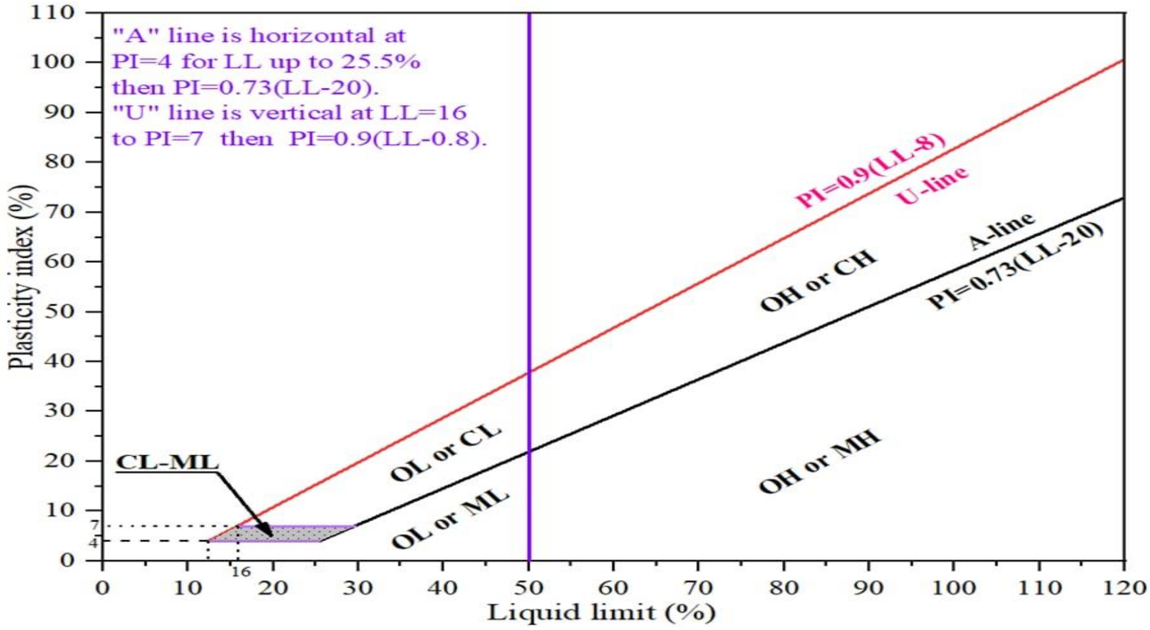


Figure 2. 2 Plasticity chart (Eden Asres, 2017)

2.4.2 Classification of soil based on ERA manual (2013)

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

Table 2.3 Expansive soil classification based on ERA manual (2013)

Expansiveness index, $Єex$	Classification
< 20	Low expansive
20-50	Moderately expansive
>50	Highly expansive

The equation is given by:

$$\epsilon ex = 2.4wp - 3.9ws + 32.5 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; Eqn. (1)$$

Where, Єex= Expansiveness index

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

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

PI = Plasticity Index, and SL = Shrinkage limit

2.4.3 Expansive soil classification based on plasticity index

Expansive soils can be classified from low expansive to very high expansive based on its plasticity index (Dennis, 1994).

Table 2.4 Swelling potential of expansive soils based on plasticity index

Swelling potential	Plasticity index
Low	0-15
Medium	10-35
High	20-55
Very High	35 and above

2.4.4 Expansive soil classification based on liquid limit

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

Table 2.5 Swelling potential of an expansive soil based on liquid limit

Swelling potential	Liquid limit
Low	20 < LL < 35
Medium	25 < LL < 50
High	50 < LL < 70
Very high	> 70

2.4.5 Skempton's method

Skempton classified clays into three classes according to their activities as indicated in Table 2.6.

$$Activity = \left({}^{PI}/_{\boldsymbol{percent\ of\ clay} < \mathbf{0.002}\boldsymbol{mm}} \right)$$

Based on the above classification, montmorillonite clay (Expansive Clay) is defined as active, illite clay as normal and Kaolinitic clay as inactive (Genet, 2021).

Table 2.6 Table of clay based on activity

Degree of Activity	Activity
Inactive clay	$A_c < 0.75$
Normal clay	$0.75 < Ac < 1.25$
Active clay	$Ac > 1.25$

2.5 Distribution of Expansive Soil in Ethiopia

Expansive soils can be found in numerous regions across the globe, including countries such as Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, the United States, the Soviet Union, Venezuela, and Zimbabwe, according to Tefera & Yohannes (1986). In Ethiopia, these soils can be observed in areas like central Ethiopia, along major roads such as Addis Ababa - Ambo, Addis Ababa - Weliso, Addis Ababa – Debere Berehan, Addis Ababa-Gohatsion, and Addis Ababa - Mojo, as well as in regions like Mekelle, Bahirdar, Gambela, Arba Minch, and the southern, southwestern, and southeastern parts of the capital Addis Ababa, where significant recent construction is taking place.

Expansive soil parent materials can be classified into two categories. The first group consists of basic igneous rocks, where the decomposition of feldspar and pyroxene minerals leads to the formation of montmorillonite, the primary mineral in expansive soils, along with other secondary minerals. The second group is made up of sedimentary rocks containing montmorillonite. Ethiopian expansive soils are derived from both of these groups (Tefera & Yohannes, 1986).

2.6 Soil Stabilization Techniques

When the soil at a specific location does not meet the standards established by regulatory agencies, it is necessary to incorporate pozzolan additives to improve its properties (Tseganeh, 2012). This stabilization process can affect various characteristics, such as reducing the plasticity index (PI), minimizing swelling potential, increasing compressive strength, and enhancing durability. Soil stabilization is often more cost-effective than replacing materials or relocating a project, which is typically not feasible. Generally, agencies will specify which soil subgrade requires stabilization, and contractors bidding on projects are aware of this requirement beforehand. As a result, soil stabilization serves as an economically viable method for improving soil at construction sites through the use of stabilizing agents. These stabilizers are materials employed to enhance the engineering properties of soil and are applied when the soil lacks the necessary qualities for a specific construction project.

2.6.1 Types of stabilization techniques

I. **Mechanical stabilization**: Mechanical stabilization is the most prevalent technique that depends on the material's innate characteristics for stabilization, typically achieved through compaction. Soil stability is enhanced by mixing the existing soil with imported soil or aggregate to achieve the desired particle size distribution, and by compressing the blend to a targeted density. Compacting soil at the appropriate moisture content serves as a form of mechanical stabilization itself (Labena, 2020).

II. **Chemical stabilization**: Combining or introducing additives like lime, cement, sodium silicate, calcium chloride, bituminous substances, and resinous materials into the soil can enhance its stability. The process of using chemicals to improve stability is known as chemical stabilization. There are three primary chemical reactions involved in soil enhancement through chemical stabilization: cation exchange, flocculation-agglomeration, and pozzolanic reactions.

a. Cation Exchange

Exchangeable ions refer to the surplus of ions with the opposite charge (compared to the surface) over those with the same charge present in the diffuse double layer. These ions can be substituted by a group of different ions with the same total charge by modifying the chemical composition of the equilibrium electrolyte solution (Fang, 2013). Negatively charged clay particles attract specific types and amounts of cations. The ease of replacement or exchange of these cations depends on several factors, primarily the cation's valence. Cations with a higher valence can easily replace those with lower valence.

For ions with the same valence, the size of the hydrated ions becomes crucial; the larger the ion, the greater the replacement power. If other conditions are equal, trivalent cations are held more firmly than divalent ones, and divalent cations are held more firmly than monovalent cations (Busscher, 1994). A typical sequence of replicability is as follows:

$$Na+ < Li+ < k+ < Rb+ < Cs+ < Mg^2+ < Ca^2+ < Ba^2+ < Cu^2+ < Al^3+ < Fe^3+ < Th^4+$$

The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. An example of cation exchange (Jaleh Forouzan, 2016).

$$\mathrm{Ca}^{2+} + \mathrm{Na}^{+} - \mathrm{Clay} \Rightarrow Ca^{2+}\,\mathrm{Clay} + (Na^{+})$$

b. Flocculation and Agglomeration :- According to Labena (2020), the process of cation exchange causes alterations in the electrical charge surrounding clay particles. Consequently, this leads to increased attraction between particles, ultimately resulting in flocculation and agglomeration. This process causes a reduction in both clay-sized particles and the overall soil surface area, thereby leading to decreased soil plasticity and swelling**.**

c. **Pozzolanic Reactions**:- Pozzolanic reactions that are time-dependent significantly contribute to soil stabilization (Labena, 2020). These reactions lead to the enhancement of various soil characteristics. Components of pozzolanic materials can generate compounds such as calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH), which possess the ability to strengthen the soil.

2.7 Applications of Soil Stabilization

Soil stabilization involves treating soil to improve its engineering properties, particularly for use as a road construction material (Labena, 2020). The soil stabilization process has several benefits, including such as decreasing soil permeability, enhancing the load-bearing capacity of foundational soils, Strengthening the shear resistance of soils, increasing durability under unfavorable moisture and stress conditions, improving natural soils for building highways and regulating the grading of soils and aggregates in constructing bases and sub-bases for highways and airfields.

2.8 Minimum Requirements for Pavement Stabilization in Ethiopia

ERA’s pavement design manual (2013) states, subgrade materials with CBR values < 3%, swelling potential > 2%, liquid limit $>$ 60% and plastic index $>$ 30% need to be replaced or treated with stabilizing agents.

Table 2.7 Stabilized subgrade soil

No	Test	Stabilized sub-grade requirement
1	Atterberg Limits	Reduction of the plasticity index to less than 30
2	Shrinkage Limits	Considerable increase of the shrinkage limit
3	Swell	Reduction of the swell to negligible values
4	CBR	Increase of the CBR to a minimum of 5
5	Particle size distribution	Modification of the particle size distribution (by agglomeration of the clay particles), the final grading being similar to that of a silt.

2.9 Cattle bone Ash

The ash of animal bones comprises basically of calcium phosphate. For a long time, bones are the chief wellspring of phosphorus and ground bones or bone feast is as yet utilized as manure for its phosphorus and calcium substance(Ariyo *et al*., 2021).

Tricalcium phosphate in the form of hydroxyapatite $Ca_5(OH)(PO_4)_3$ is contained in bone ash. The special cellular structure of bones, which is preserved by calcination, is one of the most significant properties of cattle bone ash. Cattle bone ash is non-wetting, chemically inert, free of organic matter, and has a high heat transfer resistance. Investigated the effects of bone ash on soil shear ability. The findings revealed that bone ash played a fascinating role in improving the soil's shear strength.Worked on the chemical stabilization of clay soils for road construction by looking at a case study of the clay soil(Ariyo *et al*., 2021).

2.9.1 Review of previous work on cattle bone ash

Labena (2020) investigated the use of alternative local waste materials, such as cattle bone ash and plastic bottle chips, to enhance the engineering properties of soil and reduce environmental impact. The study examined soil properties both in its natural state and when stabilized with 5, 7, and 10% cattle bone ash (CBA) by dry weight. California bearing ratio (CBR) tests and CBR swell tests were conducted with the optimal CBA percentage and when plastic chips (PC) were added at 0.4, 0.6, and 0.8% concentrations to the soil sample. The results revealed that the soil sample was classified as A7-5 (28) and MH according to the AASHTO and USCS systems, respectively. The addition of CBA reduced the plasticity index (PI), free swell, linear shrinkage, and CBR swell from 45 to 27%, 122.2 to 44.4%, 23.3 to 19.5%, and 9.02 to 0.87%, respectively, indicating an improvement in the soil's swelling property. The specific gravity of the natural soil decreased from 2.63 to 2.46 with 10% CBA content, which can be attributed to the lightweight nature of CBA. The study concluded that the optimal CBR value was achievedat 7% CBA content, resulting in a 31% increase from the original value.

Table 2.8 Chemical composition of Cattle bone ash from different literature

Oxide composition	CaO	P_2O_5	MgO	SiO_2	Fe_2O_3	Al_2O_3	Authors
Percentage (%)	52.2	48.07	2.07	-	0.03	0.61	Obianyo *et al*., (2020)
	41.4	40.32	2.08	2.78	< 0.01	2.56	Labena (2020)
	45.53	38.66	1.18	0.09	0.1	0.06	Ojuri *et al*., (2022)

Table 2.9 Review about Cattle bone ash

Authors	**Title of the study**	**CBA (%)**	**Major finding**	**Burning System**
Labena (2020) (Ethiopia)	Utilization of plastic waste and bone ash to improve the engineering properties of expansive Soils	5, 7 and 10%	Due to addition of bone ash, the plasticity index (PI), free swell, Linear shrinkage, and CBR swell decreased from 45 to 27% ,122.2 to 44.4%, 23.3 to 19.5% and 9.02 to 0.87% respectively. the optimum CBR value obtained at 7% of cattle bone ash content.	Open air burning
Adetayo *et al*., (2022) (Nigeria)	Evaluation of pulverized cow bone ash and waste glass powder on the geotechnical properties of tropical laterite	2%, 4%, 6%, 8% and 10%	up to 10 % dosage revealed a positive improvement in maximum dry density (MDD) and CBR.	Open air burning
(Ariyo et al., 2021)	Evaluation of cow bone ash (CBA) as additives in	2%, 4%, 6%, 8%,	The study concludes that cattle bone ash (CBA) is a good additive in Stabilization of	Controlled temperatur e at 750

(Nigeria)	stabilization of lateritic and termitaria soil	and 10%	Lateritic and Termitaria soils provided that the 8% additive threshold of CBA.	^{O}C for 90 minutes
(Adeboje et al., 2017) (South Africa)	Utilization of pulverized cow bone (CBA) for stabilizing lateritic soil for road work	5–12.5% CBA with a constant increment of 2.5%	only 5% CBA be used to stabilize lateritic soil in order to prevent ponding within the lateritic soil	Open air burning

Adetayo *et al*., (2022) introduced Pulverized Cattle Bone Ash (PCB) to soil samples in varying proportions of 2%, 4%, 6%, 8%, and 10% along with the optimal cement weight. They assessed the soil's strength index by examining its maximum dry density, optimum moisture content, California Bearing Ratio (CBR), and shear stress to determine the improvement in soil consistency. The results showed a positive improvement in the maximum dry density (MDD) with the addition of optimum cement and up to a 10% dosage of PCB. Based on these findings, the researchers concluded that PCB is an effective additive for enhancing geotechnical properties in cement-stabilized lateritic soil.

Ariyo *et al.* (2021) investigated the impact of cattle bone ash (CBA) on the soaked California Bearing Ratio (CBR), Maximum Dry Density (MDD), and Unconfined Compressive Strength (UCS) of soil samples. The study used varying CBA concentrations of 2%, 4%, 6%, 8%, and 10% and focused on termitaria and lateritic soil types. The findings revealed that the optimal CBA amount for increased soaked CBR value was 20.6 at 6% and 24.2 at 8%. Meanwhile, the unsoaked CBR value reached 26.3 and 40.2 both at 8%. Furthermore, the addition of CBA led to an increase in the UCS of all soil samples. At a 10% CBA concentration, UCS values of 200.7 kN/m2 for termitaria soil and 160.7 kN/m2 for lateritic soil were recorded. The study concluded that CBA is an effective additive for stabilizing lateritic and termitaria soils as long as the 8% CBA threshold is not surpassed.

Adeboje *et al.* (2017**)** examined the impact of PCB on lateritic soil samples by incorporating 5-12.5% PCB in increments of 2.5%. The research suggested that only 5% PCB should be used for stabilizing lateritic soil to avoid ponding and potential negative effects caused by excessive PCB, which can cause an overreaction between calcium oxide and silicon oxide, producing calcium silicate or calcium metasilicate, potentially harming the engineering properties of the lateritic soil.

2.10 Wheat Husk Ash

In the Bale zone of Agarfa, approximately 87.36% of the population resides in rural areas, primarily relying on rain-dependent, subsistence agriculture and livestock rearing for their livelihood. Wheat production is particularly abundant in this region (Turi *et al*., 2019).

Wheat husk ash (WHA) has good pozzolanic properties and is used for various purposes. It is a major food source produced in large quantities for both living and non-living beings and has a high calorific value of around 3.5 kcal/g. Farmers typically burn the waste in the fields after extracting the grains, resulting in its byproduct. This study examines the influence of WHA on soil quality. The wheat husk is collected from agricultural fields and burned at an uncontrolled temperature to transform it into fine ash.

Table 2. 10 Chemical composition of wheat husk ash

Oxide composition of WHA (%)						Authors
SiO_2	K_2O	CaO	MgO	Fe_2O_3	Al_2O_3	
42.33	11.30	5.46	0.99	0.84	-	Sharma *et al*., (2018)
43.22	11.3	5.46	0.99	0.84	-	Vageesha *et al*., (2020)
56	-	12.2	0.5	6.5	21	Brooks *et al*., (2019)

Table 2. 11 Review about wheat husk ash

Authors	**Title of the study**	**Percentage of WHA**	**Major finding**	**Burning system**
Monjuddin (2018) (India)	Stabilization of Expansive Soil Using Wheat Husk Ash and Granulated Blast Furnace Slag	3%, 6%, 9% and 12%.	As strain increases stress also increases up to at (9%) of WHA	Open air burning
Muhammad & Marri (2018) (Pakistan)	Immediate and long-term effects of lime and wheat straw on consistency characteristics of clayey soil	1%, 2%, 4%, 6%, 8%, 10%	10% wheat straw content resulted to a 120% increase in the plasticity index.	Open air burning
Fadmoro *et al*., (2022)	Environmental and economic impact	5%, 10%, 15%, 20%	The CBR increase from 4.3 to 27.9 with 10%	Open air burning

(India)	of mixed cow dung and husk ashes in subgrade soil stabilization		addition of ash. UCS values was also from2.4 kg/sq.cm (natural soil) to 6.3 kg/sq.cm (15% ash)	
Vageesha *et al*., (2020) (India)	Stabilisation of Expansive Soil Using Wheat Husk Ash and Sugarcane Straw Ash	3%, 7%, 9% and 11%	The CBR and UCS value is optimum at 7%.	Burned at 600 ^{o}C temp.

In a study by Moniuddin (2018), the engineering characteristics of clay soil and the unconfined compressive strength of expansive soil stabilized with different WHA percentages (3%, 6%, 9%, and 12%) were investigated. The results showed that the stress increased with strain up to a 9% WHA addition, and by optimally adding 9% WHA, the optimum water content was reduced, and the dry density was effectively increased.

Muhammad & Marri, (2018) conducted a study on the impact of wheat husk ash on the consistency of clay soil in Pakistan at varying percentages. The research demonstrated that adding wheat straw significantly increased the plasticity index; for example, a 10% wheat straw content led to a 120% increase in the plasticity index. Consequently, they concluded that properly managing the mixture of appropriate wheat straw could serve as an effective soil stabilizing agent for both immediate and long-term effects.

Fadmoro *et al.*, (2022) examined the impact of wheat husk ash on the geotechnical characteristics of soft clay soil mixed with cow dung ash. The study revealed that incorporating husk ash into the soil led to a minor increase in dry densities and a decrease in optimal moisture content. Additionally, the CBR values significantly increased from 4.3 to 27.9 with a 10% ash addition. The UCS values also showed an increase from 2.4 kg/sq.cm (natural soil) to 6.3 kg/sq.cm (15% ash).

Vageesha *et al.*, (2020)) studied the effect of sugarcane straw ash and wheat husk ash on the expansive soil by applied 3%, 7%, 9% and 11% of SCSA and WHA. The mixing of different percentages of SCSA and WHA gives increasing values of maximum dry density (MDD) for the 3%, 5% and 7% and further in proportion of both ashes, there is decrease in MDD because of stiffness of the soil. At 7%, OMC was achieved and after adding stabilizer soil shows decrease in OMC. The OMC achieved at 7% is used in various projects as index and the CBR value shows maximum value at 7% enhancement of WHA and SCSA then further decline. Test results shows that there is gradual increase in strength when ashes are added and highest value is achieved at 7%.

2.11 Research Gaps

Upon reviewing literature related to the use of cattle bone ash and wheat husk ash in geotechnical engineering, several research gaps have been discovered that warrant further exploration. These research gaps include the following such as no studies have been conducted that utilize a combination of cattle bone ash and wheat husk ash to enhance the quality of subgrade soil and in Ethiopia, only a single researcher has examined the use of cattle bone ash for improving subgrade soil performance. Furthermore, areas like the Bale zone, which have high crop productivity, especially in wheat containing pozzolanic materials, have not yet seen any investigations into the effects of wheat husk ash on soil consistency.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

This section briefly discusses the study area, soil sampling techniques, materials, laboratory testing methods, and tools used. Soil laboratory tests were carried out at the China Communication Construction Corporation CCCC.Lo geotechnical laboratory in Bale Agarfa, Addis Ababa Science and Technology university. According to the ERA Site Investigation Manual (2013), essential parameters for subgrade materials include moisture content, specific gravity, sieve analysis, plasticity index, compaction, free swell, CBR and UCS. Consequently, an experimental program was implemented to determine these parameters for both native and stabilized soils.

3.2 Description of the Study Area

The study has been carried out in Agarfa district, Bale zone, Oromia National Regional State; Southeast Ethiopia, which is located 446 km and 31 km away from Addis Ababa the capital of the country and the center of the zone, Robe respectively. The capital/center of district was Agarfa town. It is found in the extreme North Western Corner of the zone, which is bounded by Shirka district of Arsi zone in the North, West Arsi zone in south West, Dinsho district in south, Sinana district in south East and Gasera in North East. Its absolute location falls between Latitude 59˚09'06"N and Longitude 80˚32'19"E. The total geographical area of the district was about 1258 km^2 (Feyissa & Gebbisa, 2021). Agro ecologically Agarfa district is classified in to 17.01% of tropical (500 m - 1500 m), 58.61% of Sub tropical (1500 m - 2500 m), 23.31% of temperate (2500 m - 3500 m) and 1.07% of alpine/cool (The lowest and highest Altitude of the district is extended from 1000 m and 3000 m above sea level respectively. The mean annual temperature of the district is 17.5˚C.

3.3 Study Design

This study focused on a quantitative experimental approach to assess the index and engineering properties of soil, as well as the enhancement of soil properties through the combined effects of cattle bone ash and wheat husk ash. Four representative disturbed soil samples were gathered from various locations along a road section at a depth of 0.3 to 0.9 m where the soil was most susceptible, using systematic random sampling techniques based on field observations and prior laboratory test results. A lab experiment plan was developed to perform all essential lab tests to examine the subgrade soil property and the impact of cattle bone ash and wheat husk ash on these properties. The disturbed soil samples were initially air-dried, and various laboratory tests were carried out following the American Society for Testing and Materials (ASTM) and AASHTO soil testing protocols.

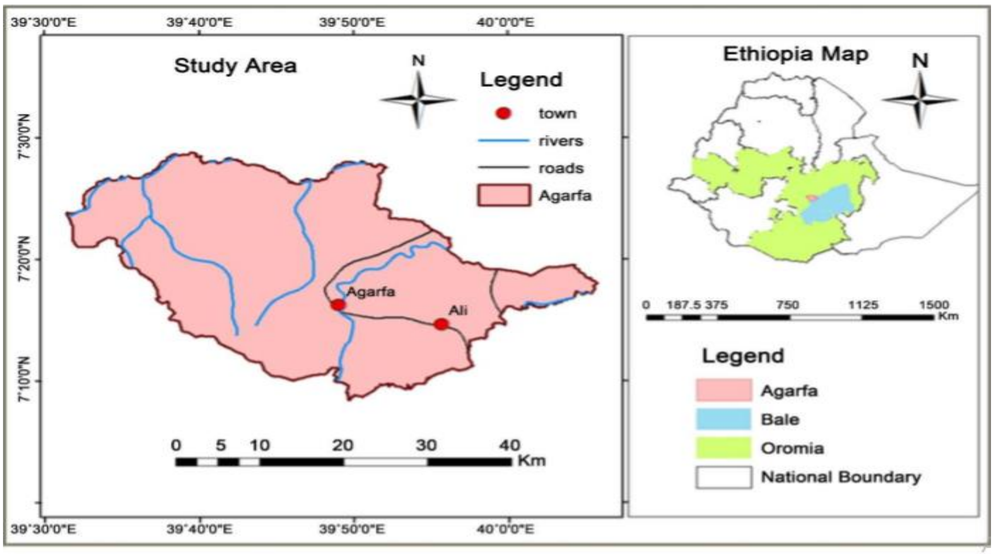


Figure 3. 1 Location map of the study area

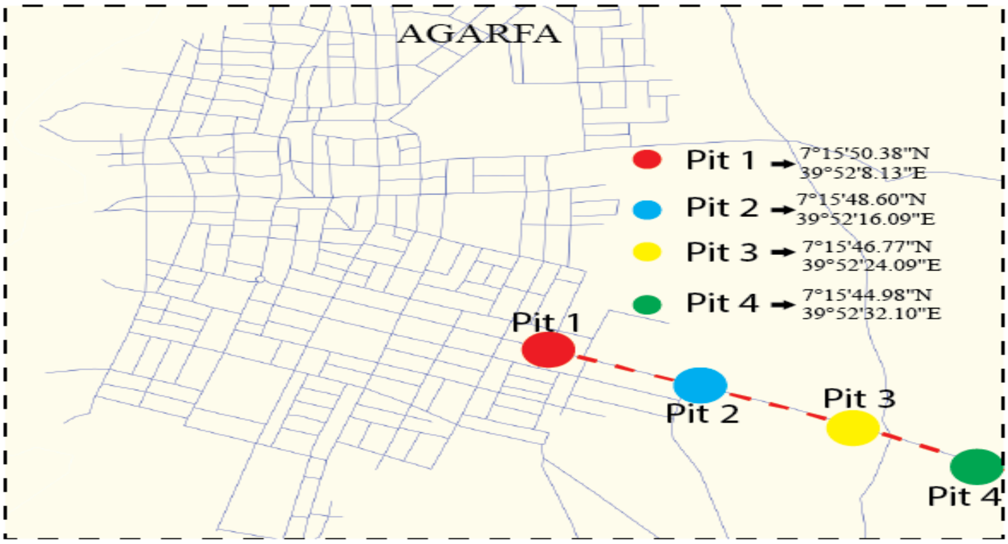


Figure 3. 2 Geographical location of the study area

3.4 Materials

3.4.1 Soil

The soil sample used in this study was obtained by manually excavating pits at a depth of 0.3 to 0.9 m from the natural ground level by a method of disturbed sampling. Then soil sample was tagged and stored in separate sacks and taken to laboratory for the evaluation of soil properties as shown in figure below.

3.4.2 Cattle bone ash

The cattle bone ash was sourced from different regions within the waste areas of Agarfa central abattoirs. The bones were cleaned and rid of any dirt or contaminants. They were then dried outdoors for 48 hours before being burned at an unregulated temperature for 24 hours to produce the ash. This ash was then processed in a milling machine to obtain the required cattle bone ash suitable for the study.

Figure 3.3 soil sample collection and taken to laboratory

Figure 3.4 Cattle bone before and after undergoing open air burning

3.4.3 Wheat husk ash

The wheat husk was gathered from agricultural fields near Agarfa. WHA is an excellent pozzolanic substance. It is a by-product of food that includes both living and non-living organisms. It also has a high calorific value of 3.5 kcal/gm. Typically, farmers burn it after removing the grains. The husk is taken from the field, heated with uncontrolled temperature. It is generally considered crop waste and is non-plastic in nature.

Figure 3.5 WHA before burning and after burning.

Table 3.1 Mix-design and material proportion

Phase-2 (Expansive soil with different percentage of CBA and WHA) using the weakest value of the natural soil			
Percentage of addition (%)	Natural soil (%)	CBA (%)	WHA (%)
0	100	0	0
2.5	97.5	2.37	0.13
5	95	4.75	0.25
7.5	92.5	7.12	0.38
10	90	9.5	0.5

3.5 Experimental Methodology

3.5.1 Methods of testing

During the initial stage of the research, lab experiments were performed on natural soil, evaluating its inherent moisture content, specific gravity, free swell index, Atterberg limits, linear shrinkage, particle size distribution, Modified proctor compaction, California

Bearing Ratio and Unconfined compressive strength in accordance with ASTM and AASHTO standards. The second stage involved determining the blend effect of CBA and WHA on the engineering properties of soils using the appropriate percentages (2.5%, 5%, 7.5%, and 10%), based on the select range of Admixture proportions from the critical review of literature.

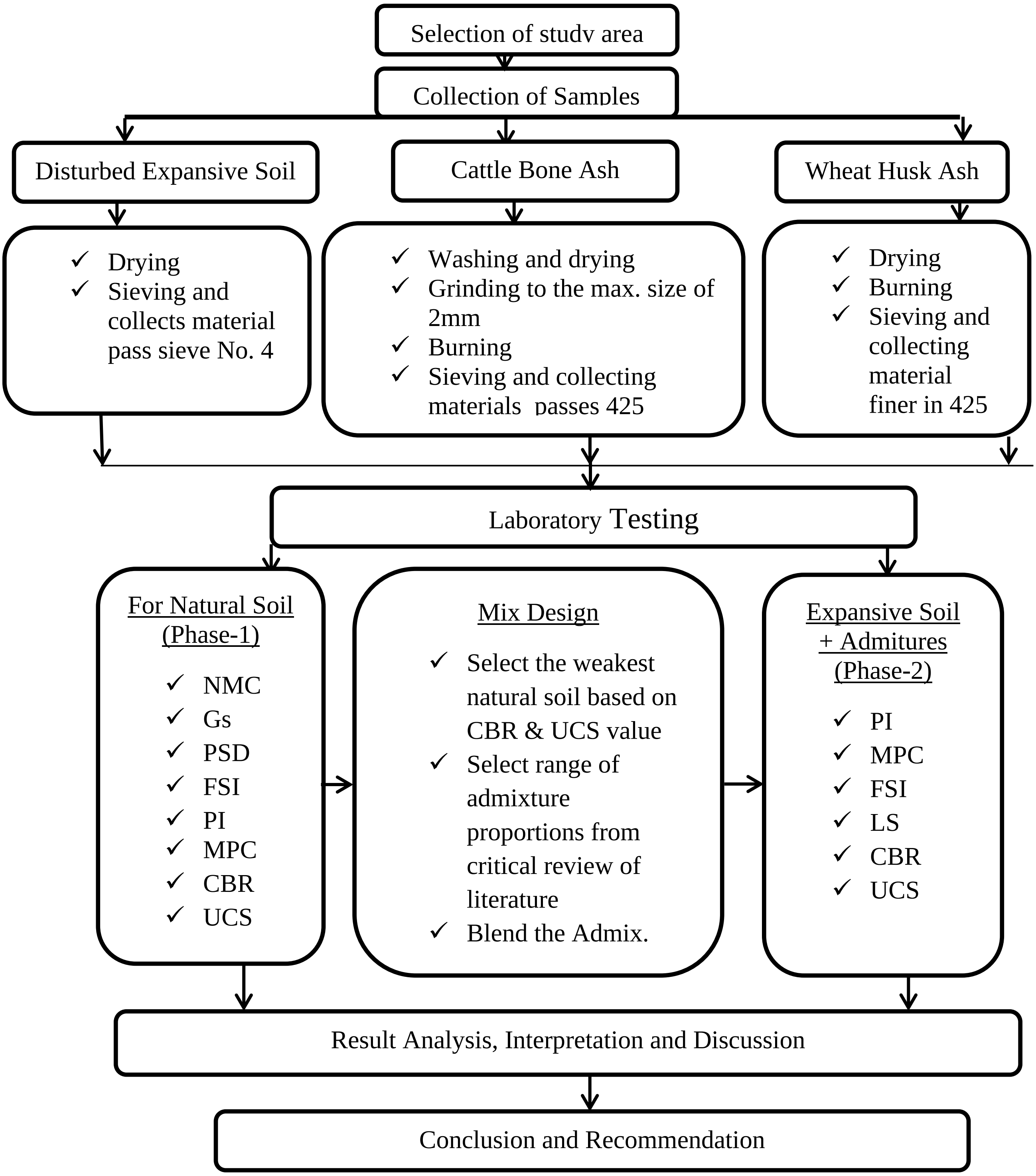


Figure 3.6 Flow chart of methodology

3.5.2 Laboratory Test

To determine the engineering properties of the collected natural soil, various preliminary tests were performed on controlled soil samples, including grain size analysis, Atterberg limits, shrinkage limits, specific gravity, free swell, modified proctor compaction, California Bearing Ratio (CBR), CBR Swell, and linear shrinkage. These laboratory tests for the native soil were carried out at the China Communication Construction Company Ltd. (CCCC), located in the Agarfa district. Additionally, the mineralogical (oxide) composition of the soil was analyzed at the Ethiopian Geological Survey (EGS) Laboratory. The data sheets for all natural soil tests conducted in this study can be found in Appendix.

3.5.3 Grain size distribution

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.

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 hexametaphosphate, (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.

$$\mathbf{C_u = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10} * D_{60}}} \qquad (2)$$

Where: D_{10}, D_{30}, and D_{60} are the particle size that corresponds to the percent finer of 10%, 30%, and 60%, respectively. As shown in Figure 3.5, a hydrometer analysis was then performed using 151H Hydrometer to measure the amount of silt and clay size particles.

3.5.4 Atterberg limits

Samples representing each type of soil were tested for Atterberg limits to assess their plasticity. A device for Atterberg limits was employed to establish the liquid limit of each soil by testing the material that passed through a 425 µm (No. 40) sieve. As shown in Figure 3.6, the liquid limit was identified as the moisture content at which a section of soil in a standard cup, cut by a groove with standard measurements, merged together at the base of the groove for a length of 13 mm (1/2 inch) when exposed to 25 shocks. These shocks were generated by dropping the cup 10 mm in a standard liquid limit apparatus, operating at a rate of two shocks per second. In this research, the ELE International Casagrande apparatus was utilized to find the liquid limit using method A.

To determine the plastic limit of each soil, a 425 µm sieve was used, and 3-mm diameter soil threads were rolled until they started to fracture. The plasticity index was then calculated for each soil based on the obtained liquid and plastic limits. Finally, the liquid limit and plasticity index were employed to categorize each type of soil.

3.5.5 Specific gravity

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

Mathematically; $Gs = (W2-W1)/(W4-W1) - (W3-W2)$ ………………………… (3)

Where, Gs = Specific gravity of soil solids

$W1$ = Weight of dry and clean pycnometer, g

$W2$ = Weight of dry and clean pycnometer + dry soil, g

$W3$ = Weight of dry and clean pycnometer + dry soil + water, g

$W4$ = Weight of dry and clean pycnometer + Water, g

3.5.6 Moisture density relation of the soils

The Modified Proctor Test was conducted to identify the maximum dry density (MDD) and ideal moisture content (OMC) of natural soil, following the AASHTO T-180 standard. A sufficient amount of air-dried soil was acquired in a large mixing pan, then pulverized and sieved through a No. 4 (4.75mm) sieve. Five representative samples of approximately 6000 grams each were prepared for a single Proctor Test using a 6-inch mold. Each sample was compacted using a 4.5kg hammer falling from a distance of 18 inches in five equal layers, with each layer receiving 56 blows. In every proctor test, five trials were performed, with each trial increasing the water content by 2% compared to the previous one, starting with 4% water content. This sequence of measurements continued until the wet unit mass (g/cc) of the compacted soil decreased or remained unchanged.

3.5.7 California Bearing Ratio

Three-point CBR tests were carried out following the AASHTO T-193 guidelines to assess the strength of the sample soil and its behavior under loading. Approximately 18kg of air-dried soil was sieved through a #4 (4.75mm) sieve and mixed with optimum moisture content in a large mixing pan, as depicted in Figure I.4a. The sample was then divided into three equal portions, each weighing around 6.0kg, intended for compaction with 10, 30, and 65 blows per layer across five equal layers in a single CBR mold. Moisture content data were collected before and after compaction.

The compacted soil in the CBR mold was submerged in water for four days with an applied surcharge load of 4.5kg (Figure I.4b). Upon completion of the soaking period, the CBR mold was removed from the water and drained for 15 minutes prior to CBR penetration. The CBR test was conducted using a 28 kN mechanical press manufactured by ELE, as illustrated in Figure I.4(c).

CBR samples penetration were performed at a strain rate of 1mm per minute (at a plunger force of 250N) and the readings of the force were taken at intervals for 0.5mm to a total of penetration not exceeding 7.5 mm. The CBR value was calculated at penetration of 2.5 and 5.0 mm and the higher value was taken. A graph was drawn for penetrations against dry densities in mould and CBR value at 95% of MDD was taken as the design CBR value. The design CBR value is used as the index measurement of soil strength.

3.5.8 Free swell values test

The main problem of expansive soil its swell-shrink behaviour due to moisture fluctuation. Thus, it was led to differential settlement and unevenness in road pavements and railway track structures. Free swell index test was conducted following (IS 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 100 ml and gently shaking or stirring the mixtures with a glass 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.

$$\mathrm{FSI} = \frac{(\mathrm{Vw} - \mathrm{Vk})}{\mathrm{Vk}} * 100\% \quad \& \quad \mathrm{FSR} = \frac{\mathrm{Vw}}{\mathrm{Vk}} \qquad (4)$$

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

3.5.9 Linear shrinkage

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.

$$\text{Linear shrinkage(LS)} = \frac{(\mathrm{L_O} - \mathrm{L_D})}{\mathrm{L_O}} * 100\% \qquad (5)$$

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

3.5.10 Unconfined compressive strength

According to (ASTM D2166−16, (2016) standard, unconfined compressive strength (q_u) refers to the compressive stress at which a cylindrical, unconfined sample of fine-grained soil fails during a simple compression test. This test's main objective is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of clays under unconfined conditions, where lateral pressure (σ3) is zero. For soils, determining the undrained shear strength (Su) is essential for calculating bearing capacity for foundations, dams, etc. The undrained shear strength (Su) of clays can be determined from an unconfined compression test and is equal to half of the unconfined compressive strength (q_u) when the soil's internal friction angle (ϕ) is zero. The most critical condition for soil usually occurs immediately after construction, representing undrained conditions, with undrained shear strength equal to cohesion and expressed as:

$$S_u = C_u = \frac{q_u}{2} \tag{6}$$

In this case, the test is used to identify the weakest natural soil from four different samples, with the sample exhibiting the lowest UCS value being the weakest. The selected weakest soil sample was then mixed with CBA and WHA at proportions of 2.5%, 5%, 7.5%, and 10% and remolded with respective water content and density under both cured and uncured conditions to determine the optimal soil amount based on UCS criteria. Specimens measuring 76 mm in height and 38 mm in diameter were prepared using a mold with a circular cross-section at predetermined moisture (OMC) and density (MDD) levels, using remolded dry, disturbed soil samples finer than a 4.75 mm sieve. The sample was then placed and loaded under a UCS machine, with results recorded and analyzed automatically via a computerized system.

3.6 Chemical Composition of Cattle Bone Ash and Wheat Husk Ash

The chemical properties of the bone ash; code (GLD/F5.10.2) were determined in Geological Insitute of Ethiopia laboratory using LiBO2 FUSION, HF attack, GRAVIMERTIC, COLORIMETRIC and an atomic absorption spectrophotometer (AAS) to know the percentage composition of the major and minor elements. For this test a sample that passed 200 sieve size was used.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 Introduction

Laboratory experiments including consistency limit, free swell index, particle size distribution, natural moisture content, compaction, and California bearing ratio were done on natural soil collected from four representative test pits to classify and select the weakest soil for further studies. Therefore, based on CBR and UCS, the soil sample collected from pit-4 was found to be the weakest soil needed to be studied for the effect of mixing with cattle bone ash and wheat husk ash. Based on these, Finally, combined effect of both additives was studied, and final dosage that gives better result was proposed as optimum proportion of admixture.

4.2 Phase-1 Investigation of the Natural Soil

4.2.1 Particle size distribution of the natural soil

Particle size analysis involves the quantitative assessment of particle size distribution in soils, adhering to the ASTM D6913, (2018) standard for coarse-grained soils and ASTM D7928, (2021) standard for fine-grained soils. In Table 4.1, soil samples from pit-1, pit-2, pit-3, and pit-4 exhibited 93.94%, 94.08%, 59.5%, and 84.24% of particles passing through a 0.075 mm sieve, respectively, identifying them as fine-grained materials. In contrast, 12.8% of WHA and 2.8% of CBA samples passed through sieve No.200, classifying them as coarse-grained materials. According to the USCS plasticity chart, all soil samples had a liquid limit above 50% and were situated above the A-line, categorizing them as CH, except for pit-3, which was classified as MH (inorganic fat clay with high plasticity). The AASHTO plasticity chart identified all four soil samples as A-7-5, possessing group indices of 12, 36, 22, and 33, respectively.

The grain size of wheat husk ash and cattle bone ash was larger compared to the soil samples. From table 4.1, we observe that 87.8% of WHA and 97.2% of CBA remained on a 0.075 mm sieve, while 12.8% of WHA and 2.8% of CBA went through it. Based on the percentage of particles smaller than 4.75 mm and 0.075 mm, the USCS classifies WHA and CBA as well-graded sand (SW). Nevertheless, the AASHTO classification system categorizes WHA and CBA as A-3 (0) fine sand due to their proportion of particles smaller than 0.425 mm and 0.075 mm sieves and their non-plastic properties.

Table 4.1 Summary of particle size distribution of the natural soil and admixture

Sample	Particles size distribution			
	≥ 4.75 mm (Gravel)	0.075 - 4.75mm (Sand)	0.075 - 0.002 mm (Silt)	≤ 0.002 mm (Clay)
Pit-1	0.94	5.12	31.44	62.50
Pit-2	0.22	4.11	34.91	60.76
Pit-3	0.70	39.85	40.55	18.90
Pit-4	1.40	12.53	35.07	51.01
WHA	0.0	87.8	12.8	0.0
CBA	0.0	97.2	2.8	0.0

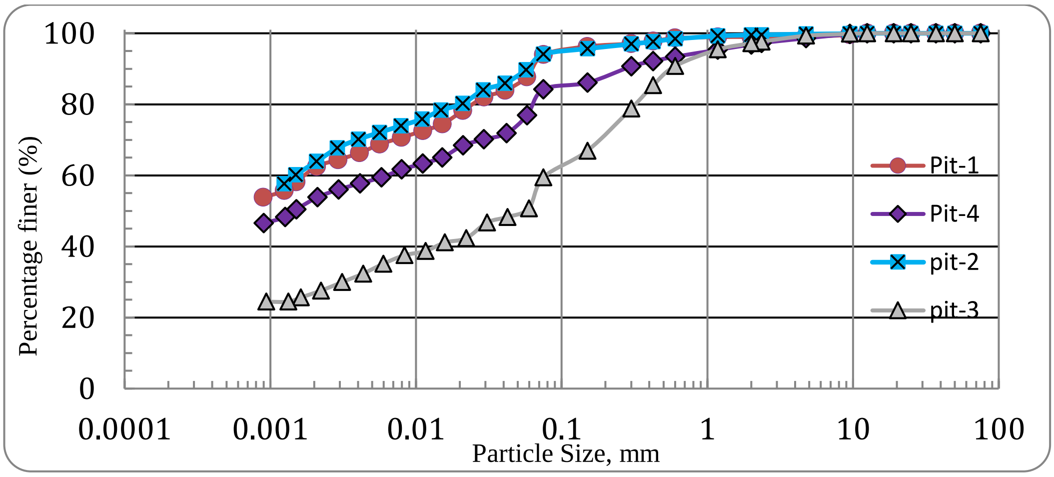


Figure 4.1 Particle size distribution of natural soil

4.2.2 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).

Table 4.2 Atterberg limit test results of natural soils

Designation	Atterberg limit results (%)			
	LL	PL	PI	LS
Pit-1	82.1	35.1	47.0	16.8
Pit-2	89.7	54.6	57.0	10.20
Pit-3	106.5	45.5	61.0	12.02
Pit-4	116.0	37.0	79.0	18.6

From test results liquid limit, plastic limit, plasticity index and linear shrinkage of soil sample from pit-1, pit-2, pit-3 and pit-4 are (82.1%, 35.1%, 47.0%, and 18.60%), (89.7%, 54.6%, 57.0%, and 10.20%), (106.5%, 45.5%, 61.0%, and 12.02%) and (116.0%, 37.0%, 79.0%, 16.80%) respectively. liquid limit and plasticity index result obtained are mainly used in soil classification system and shows soil grouped as CH except for pit-3 and A-7-5 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.

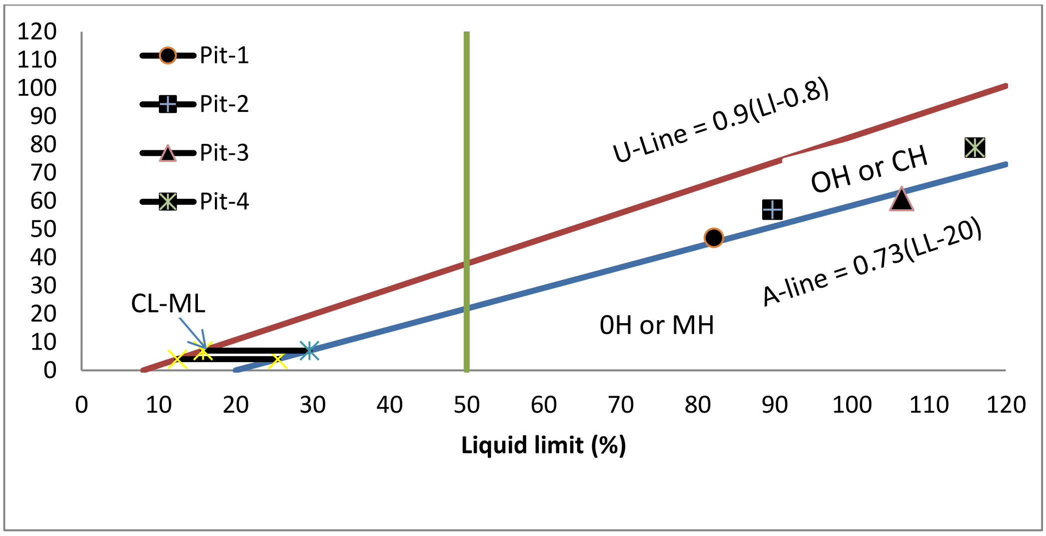


Figure 4.2 Plasticity chart for natural soil classification system

The ERA (2013) guidelines for fill materials specify that the liquid limit should not exceed 60% and the plasticity index should not exceed 30, as determined by AASHTO T-90 (2020). However, the soil samples analyzed in this study exhibit high liquid limits and plasticity indexes, as discussed in Table 4.2, which do not comply with ERA specifications. Soils with elevated liquid limits often have a higher propensity for swelling and shrinking (Sharma *et al*., 2018).

4.2.3 Specific gravity of natural soil

Specific gravity determination of materials for this study follows (ASTM D 854 – 02, 2006) standard method.

Table 4.3 Specific gravity of soil samples and additives

Location	Specific gravity, G_s
pit-1	2.620
pit-2	2.617
pit-3	2.656
Pit-4	2.677
WHA	2.50
CBA	2.366

The specific gravity of the materials used in the study is summarized in Table 4.3, which displays the natural soil samples from pits 1, 2, 3, and 4 with G_s values of 2.613, 2.608, 2.643, and 2.639, respectively. These values fall within the range for clay minerals. As mentioned by Tefera & Yohannes (1986), the typical specific gravity range for soil is between 2.6 and 2.8, though it is dependent on the soil's mineral composition. Silt and clay soils containing alumina minerals have a lower specific gravity than those with iron oxide. In this particular soil sample, the alumina concentration is higher than the iron oxide concentration (Labena, 2020).

4.2.4 Moisture content of natural soil

Moisture content 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 105°± 5 °C is used to determine mass of dry solid soil (ASTM-D-2216-98, 1998).

Natural moisture content determination of soil sample was used to know existing condition of material and depth of ground water table at the sampling sites. Based on laboratory result natural moisture contents are 9.49%, 9.12%, 11.52% and 12.92% for pit-1, pit-2, pit-3 and pit-4 respectively. The result shows sample from pit-4 is wettest soil due to there may be some moisture infiltration or shallow groundwater table depth and pit-2 material have the least moisture.

Table 4.4 Natural moisture content of samples

Location	Moisture content (%)
pit-1	9.49
pit-2	9.12
pit-3	11.52
Pit-4	12.92

4.2.5 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, 1977) standard method. The soil samples from pit-1, pit-2, pit-3, and pit-4 exhibit FSI values of 70.83%, 79.17%, 91.67%, and 95.83%, and FSR values of 1.79, 1.71, 1.92, and 1.95, respectively, indicating that these four test pits have a higher swelling potential compared to others. The free swell test results surpass 50%, signifying that the soil samples used in this study can be categorized as highly expansive soils. These types of soils experience volumetric alterations, causing pavement warping, cracking, and unevenness due to seasonal wetting and drying processes (Busscher, 1994). The WHA and CBA utilized in this research possess FSI and FSR values of (3% & 0.5%) and (1.2% & 1.01), respectively, suggesting that they are nearly non-swelling admixtures. These materials are more suitable for replacing a portion of weak soil and enhancing its geotechnical properties.

Table 4.5 FSI and FSR result of natural soils and additives

Designation	FSI (%)	FSR
pit-1	79.17	1.79
pit-2	70.83	1.71
pit-3	91.67	1.92
Pit-4	95.83	1.96
WHA	3	1.03
CBA	0.5	1.01

4.2.6 Modified proctor compaction test result of natural soil

Soil compaction is a process where soil becomes denser due to a decrease in the volume of empty spaces or voids within the soil mass under mechanical stress. The soil mass is composed of solid particles and voids containing air and/or water. As stress is applied to the soil, its particles reorganize within the mass, leading to a reduction in the void volume and an increase in soil density.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 Modified proctor compaction test result of natural soil

Location	MDD (g/cc)	OMC (%)
pit-1	1.43	26.95
pit-2	1.48	25.80
pit-3	1.41	27.86
Pit-4	1.39	28.14

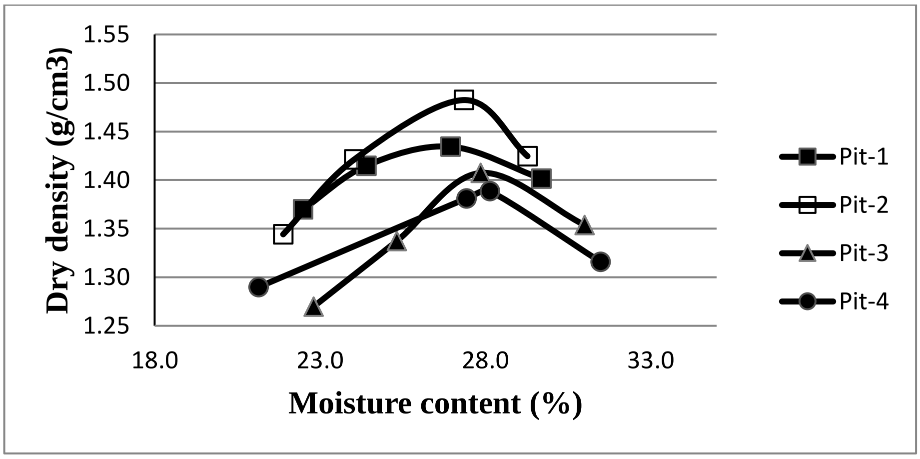


Figure 4.3 Compaction curves of natural soil collected from pit-1, pit-2, pit-3 and pit-4

Results obtained reveals that natural soil collected from pit-1, pit-2, pit-3 and pit-4 have optimum moisture content of 26.95%, 25.80%, 27.86% & 28.14% and maximum dry density of 1.43 g/cc, 1.48 g/cc, 1.41 g/cc, and 1.39 g/cc respectively. The maximum dry density of the sample lies in the interval of 1.39g/cc and 1.48g/cc. Thus the sample can be described as highly plastic clay (Davidson, 2016).

4.2.7 California Bearing Ratio of natural soil

This test method was executed to estimate the potential strength of subgrade soil samples which are used in pavement constructions. CBR estimates the strength of cohesive materials having maximum particle sizes less than 4.75mm. This test method provides for the determination of the CBR of materials at OMC from a specified compaction test and a specified dry unit mass.

Table 4.7 CBR value of natural soil

Location	CBR at 95% MDD
pit-1	1.28
pit-2	1.30
pit-3	1.13
Pit-4	0.94

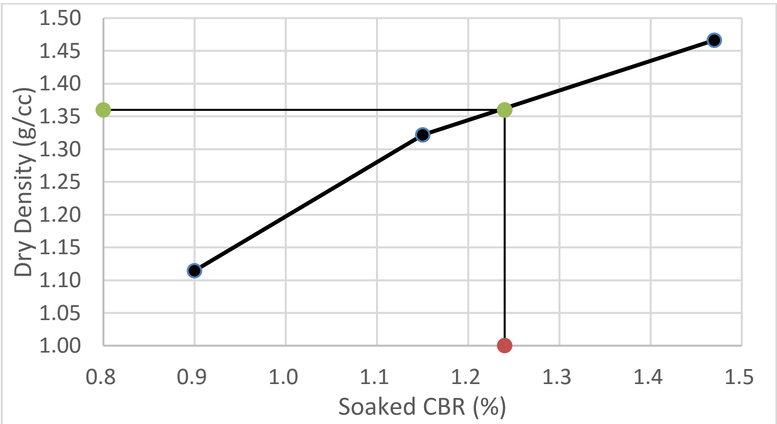


Figure 4.4 CBR value of natural soil for pit-1

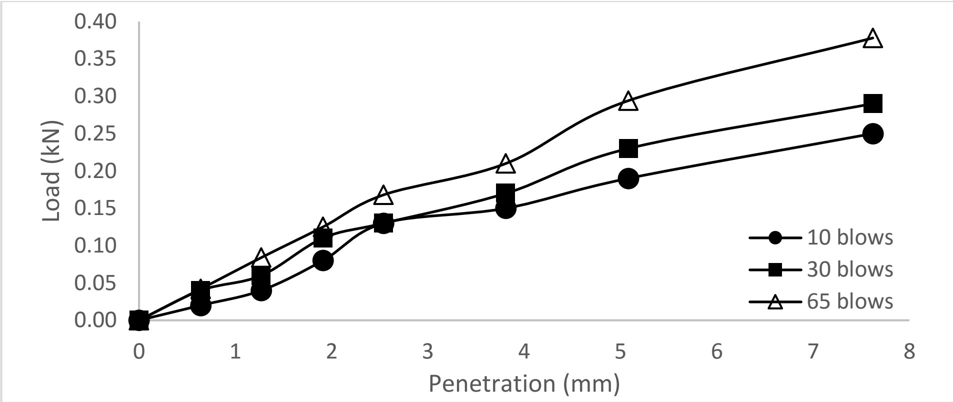


Figure 4.5 Load vs penetration of natural soil for pit-1

4.2.8 Unconfined compressive strength of natural soil

Samples of soil collected from all four test pits were also tested for their strength through unconfined compressive strength test following (ASTM D2166−16, 2016) standard method. Based on this, material collected from Pit-1, Pit-2, Pit-3 & Pit-4 have compressive strength of 139.501 kN/m^2, 152.098 kN/m^2, 116.224 kN/m^2 & 106.673 kN/m^2 respectively. In addition to swelling property, compressive strength result shows Pit-4 material has low UCS value and was selected as the weakest sample needed for further study by mixing with respective additives.

Table 4.8 Unconfined compressive strength result of natural soil

Location	UCS (kN/m^2)
Pit-1	139.501
Pit-2	152.098
Pit-3	116.224
Pit-4	106.673

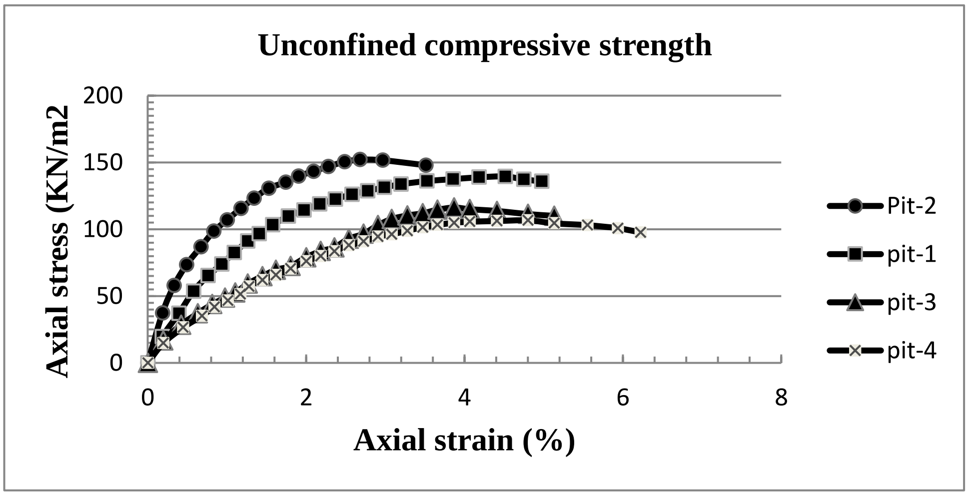


Figure 4.6 Unconfined compressive strength of natural soil

Table 4.9 Summary of geotechnical properties of natural soils

Properties of natural soil	pit-1	pit-2	pit-3	Pit-4
Natural moisture content	9.12	9.49	11.52	12.92
Optimum moisture content	26.95	25.8	27.86	28.12
Maximum dry density	1.43	1.48	1.41	1.39
Liquid limit	82.1	89.7	106.5	116.0
Plastic limit	35.1	54.6	45.5	37.0
Plastic index	47.0	57.0	61.0	79.0
Linear shrinkage	16.8	10.20	12.02	18.6
Free swell index	79.17	70.83	91.67	95.83
Free swell ratio	1.79	1.71	1.92	1.96
Percent finer of No.4 sieve	100%	100%	100%	100%
Percent finer of No.200 sieve	83%	64%	46%	50%
Soil class based on USCS	CH	CH	MH	CH
AASHTO system	A-7-5 (12)	A-7-5 (36)	A-7-5 (22)	A-7-5 (33)
Specific gravity, G_s	2.620	2.617	2.656	2.677
CBR	1.28	1.30	1.13	0.94
UCS (kN/m^2)	139.501	152.098	116.224	106.673
Color	Black	Black	Black	Black

4.3 Properties of Cattle Bone Ash and Wheat Husk Ash

As seen in Table 4.10, bone ash contains a small percentage of Alumina (2.38%) and Silica (4.68%) minerals and more than 25% of CaO and P2O5. The chemical analysis results align with previous literature and are more consistent with the findings of Obianyo *et al.*, (2020). The CaO in bone ash can react with fine soil particles to help stabilize the soil. Similarly, P2O5 has the potential to act as a binding agent that cements soil particles together, increasing stability.

However, when compared to the chemical composition of natural pozzolanas detailed in (ASTM C618, 2014), the sum of the three oxides (silica, alumina, and iron oxide) in cattle bone ash is less than 50%. This suggests that cattle bone ash alone cannot meet the minimum percentage requirements to be considered a calcined natural pozzolana. As a result, it requires other pozzolanic materials to fulfill the criteria of (ASTM C618, 2014).

Hence, it is useful to note here that during mixing of wheat husk ash and cattle bone ash, the pozzolanic reaction takes place as the silica and alumina present in wheat husk ash react with calcium from bone ash to form cementitious products like calcium-silicate-hydrates (CSH) and calcium aluminate hydrate (CAH);which improves the properties of soil in long- term (Chethan, 2018).

Therefore, the combination of CaO from cattle bone ash and the three oxides (silica, alumina, iron oxide) from the wheat husk ash (47.71%) was expected to have good potential to produce class C quality pozzolana requirement.

Table 4.10 Chemical composition of cattle bone ash

Oxide	Composition
SiO_2	4.68
Al_2O_3	2.38
Fe_2O_3	0.02
CaO	38.30
MgO	0.54
Na_2O	1.88
K_2O	0.01
MnO	0.01
P_2O_5	26.08
TiO_2	0.01
H_2O	3.11
LOI	22.32

Table 4.11 Chemical composition of wheat husk ash

Oxide	Composition
SiO_2	44.32
Al_2O_3	0.01
Fe_2O_3	0.38
CaO	3.32
MgO	0.96
Na_2O	1.72

K_2O	16.18
MnO	0.04
P_2O_5	1.71
TiO_2	0.01
H_2O	8.11
LOI	22.59

4.4 Phase-Two Effect of Blended CBA and WHA on the Natural Soil

4.4.1 Effect of blended CBA and WHA on Atterberg limit of soil

Table 4.12 Atterberg limit of soil mixed with different proportions of CBA and WHA

Percentage of addition (%)	BCS & different percentage of WHA and CBA		
	Liquid limit	Plastic limit	Plastic index
0	116	37.36	79
2.5	87.1	43.36	44
5	71.7	38.55	33.2
7.5	59.8	34.01	26
10	67.7	38.9	28.8

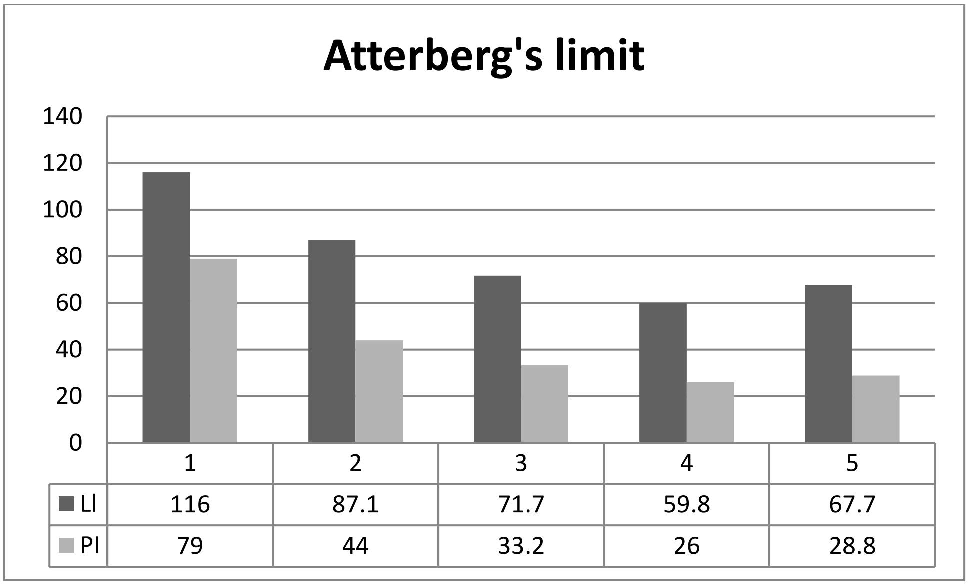


Figure 4. 7 Liquid limit Vs Plastic index

Figure 4.7 shows the plasticity characteristics of soil stabilized soil with CBA and WHA. It is clear that incorporating CBA and WHA leads to a reduction in the liquid limit and a subsequent decrease in the natural soil's plasticity index, up to 7.5%. This is attributed to the pozzolanic material, which binds soil particles and disperses the clay fraction in both CBA and WHA. The liquid limit reduction is due to the free $Ca^{(2+)}$ from CBA, which exchanges with the clay mineral's adsorbed cations, causing a decrease in the size of the water layer surrounding the clay particles. This smaller water layer enables the clay particles to be in closer proximity to each other, forming a cluster or floc of clay particles, similar to what has been observed by Singh & Sharma (2017).

4.4.2 Effect of blended CBA and WHA on compaction characteristics of soil

Table 4.13 MDD and OMC of soil mixed with different percentages of CBA and WHA

Percentage of addition (%)	Expansive soil & WHA & CBA	
	MDD (g/cc)	OMC (%)
0	1.39	28.12
2.5	1.43	26.91
5	1.45	26.32
7.5	1.47	25.24
10	1.43	22.03

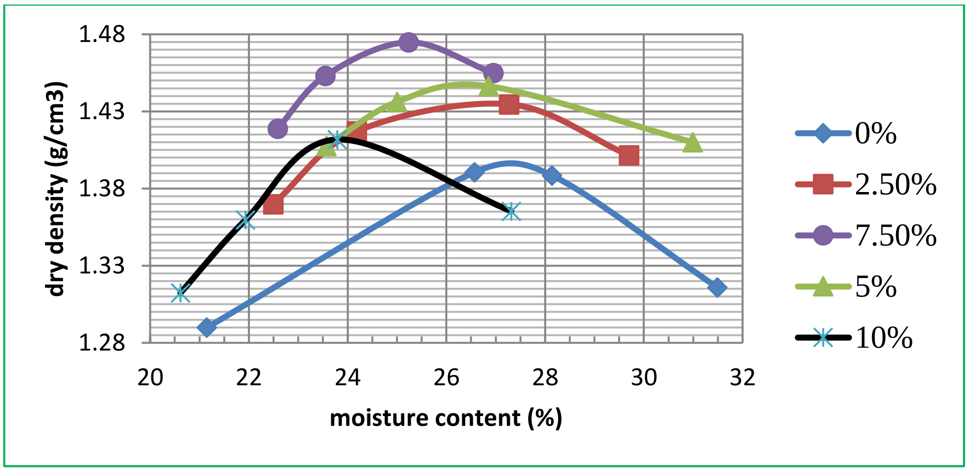


Figure 4.8 Compaction curve of soil mixed with different amount of CBA and WHA

As shown from table 4.13 and figure 4.8 improvement in maximum dry density was seen at 2.5%, 5% and 7.5% including addition of CBA and WHA but with further increase in CBA & WHA maximum dry density decreases because of stiffness of the soil. The decrease OMC is due to specific gravity of WHA which replaces higher specific gravity of expansive soil because its fibrous nature similar case reported by Vageesha *et al*.(2020)

4.4.3 Effect of blended CBA and WHA on California Bearing Ratio of soil

Table 4.14 CBR of soil mixed with different percentages of CBA and WHA

percentage of addition (%)	CBR (%)
0	0.96
2.5	2.63
5	4.19
7.5	5.43
10	2.26

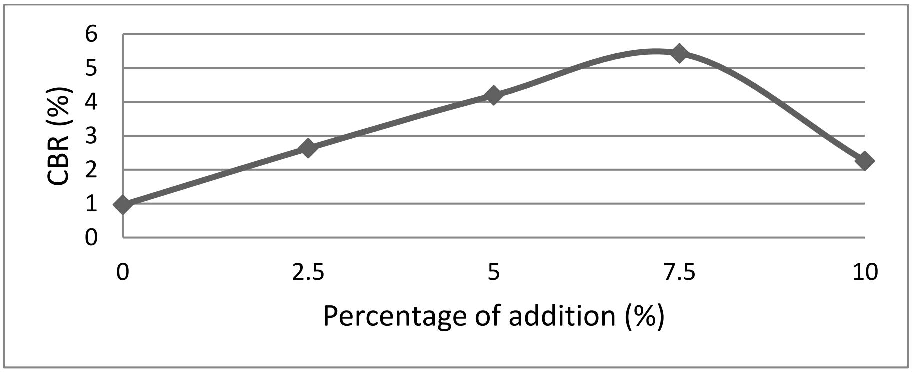


Figure 4.9 CBR (%) of CBA and WHA stabilized under soaked condition

According to Table 4.14 and Figure 4.9, the observed results displayed two patterns. CBR values increased as CBA and WHA concentrations rose from 0 to 10%, reaching an optimal point at 7.5% stabilization for soil samples. This initial CBR increase is anticipated due to the gradual formation of cementitious compounds between cattle bone ash and the alkalis in wheat husk ash. Consequently, the bonds within soil particles grew stronger as more CBA was added (Alhassan, 2008). However, after 7.5% stabilization, CBR values in the soils began to decrease due to the excess CBA occupying spaces within the soil matrix and reducing the bonding in the soil, as reported by Yadu *et al*. (2011). as well. The maximum CBR values for the soils at 7.5% stabilization are approximately 5.65 times higher than the corresponding unstabilized soils.

4.4.4 Effect of blended CBA and WHA on swelling of soil

Free swell index of natural soil collected from Pit-4 was observed as 95.83%. This makes the soil to fall under very high degree of expansion category as per the classification stated in (IS, 1977). Whereas, with mixing of 2.5%, 5%, 7.5% and 10% of CBA & WHA to soil the swelling property of natural soil reduces to 73.91%, 63.63%, 54.5% and 40.9% respectively. Hence, after incorporating of 10% blend, soil sample was improving its swelling potential and falls near to moderate degree of expansion. Therefore, blend materials CBA and WHA are very effective and efficient admixture in reducing the swelling characteristics of expansive soils.

Table 4.15 Swelling index of soil mixed with different proportions of CBA and WHA

Sample	Percentage of addition (%)	FSI (%)	FSR
BS,100	0	95.83	1.96
CBA 2.37+WHA 0.13+BS 97.5	2.5	73.91	1.74
CBA 4.75+WHA 0.25+BS 95	5	63.63	1.64
CBA 7.13+WHA 0.38+BS 92.5	7.5	54.5	1.55
CBA 9.5+WHA 0.5+BS 90	10	40.9	1.41

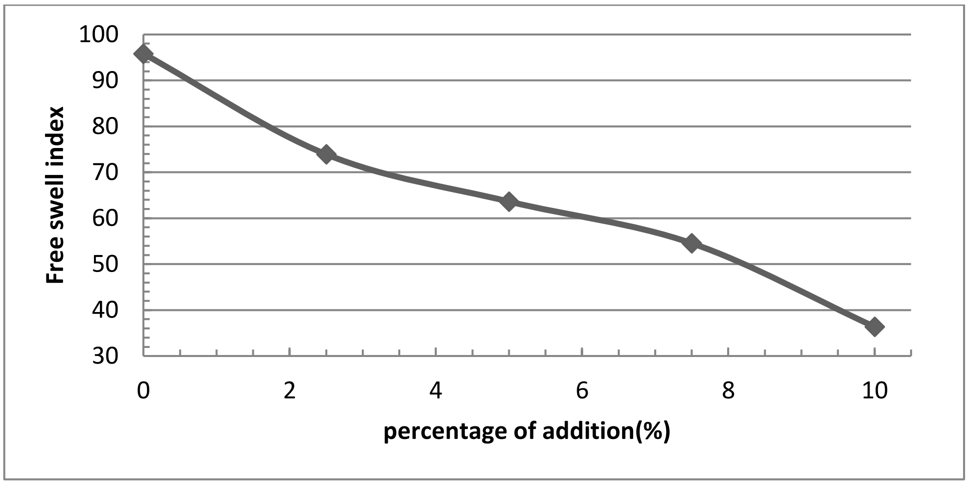


Figure 4.10 Variation of free swell index with additives

4.4.5 Effect of blended CBA and WHA on linear shrinkage of soil

As shown in Table 4.16 and Figure 4.11, the linear shrinkage result of stabilized sample is decreasing when the percentage of cattle bone ash and wheat husk amount increasing from 0 to 10%. With increasing of CBA and WHA amount, the linear shrinkage value decrease from 20.5 % to 13.5% at 10% of CBA and WHA.

Table 4.16 Linear shrinkage of soil with different percentage of CBA and WHA

Test for additives	percentage of additives for different % of CBA & WHA			
	2.5	5	7.5	10
Length of wet sample (mm)	137	137	137	137
Length of oven dry soil after 24 hr,L1 (mm)	120	119	121	123
Length of oven dry soil after 24 hr,L2 (mm)	115.85	119	122	124
Average length of oven dry soil(mm)	117.925	119	121.5	123.5
Linear shrinkage (%)	19	18	15.5	13.5

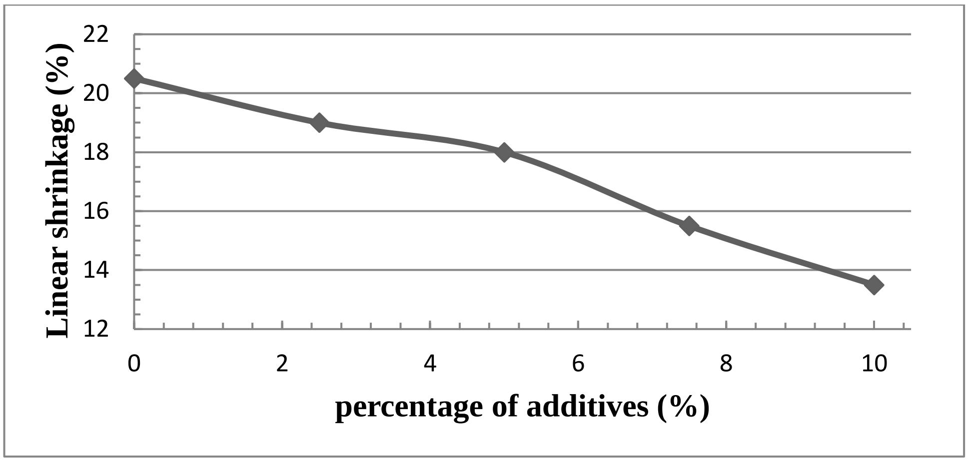


Figure 4.11 Variation of CBA and WHA and linear shrinkage relation

4.4.6 Effect of blended CBA and WHA on UCS of soil

Table 4.17 UCS of soil sample mixed with different proportions of CBA and WHA

percentage of addition (%)	UCS (kN/m^2)
0	106.673
2.5	119.462
5	138.325
7.5	166.107
10	145.931

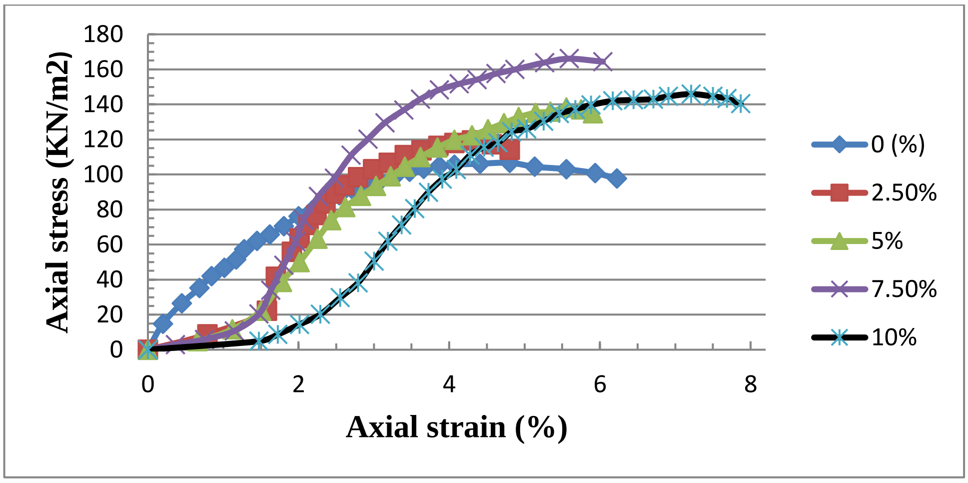


Figure 4.12 Variation of unconfined compressive strength of soil with the addition of CBA & WHA

Figure 4.12 illustrates the relationship between UCS value and the different percentage of CBA and WHA mixed with soil. From this, the value of unconfined compressive strength results up to 7.5% addition of CBA and WHA was increased, and then sudden decrease was observed at 10% addition of CBA & WHA to the soil. It was noticed that the strength of the soil was increased approximately by average of 19.8% for every addition of 2.5% CBA and WHA to the soil. Again, it was also inspected that strength of soil was decreased by 20.17% with further addition of 10% cattle bone ash and wheat husk ash to the soil sample.The initial rise in strength can be attributed to cation exchange, flocculation, and the pozzolanic reaction among CBA, WHA, and soil minerals, as these elements have more time to react and create particle flocs the results are consistent with observation reported by (Alhassan, 2008) & (Labena, 2020). However, when the proportion of CBA and WHA exceeds 7.5%, there is a decrease in compressive strength. This decline in strength can be traced back to the diminished cohesion between natural soil and WHA (a cohesionless material) particles, as noted by Sinha *et al*., (2019).

CHAPTER FIVE
CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the laboratory observed experimental results of the study the following conclusions were drawn.

- The most fragile expansive soil among the four pits identified is classified as A-7-5(33) soil according to the AASHTO (1986) soil classification system and is also considered high plasticity clay (CH) based on the unified soil classification system (USC).
- When the soils vary the incorporation of CBA and WHA up to 7.5% by dry weight, FSI decreased from 95.83% to 59.09%, FSR decreased from 1.96 to 1.36, liquid limit decreased from 116.6% to 59.8%, plasticity index decreased from 79% to 34.01%. These result reveals that mixing soil with cattle bone ash and wheat husk ash reduces expansion potential and plasticity characteristics.
- Combining various proportions of CBA and WHA results in rising maximum dry density (MDD) values for 2.5%, 5%, and 7.5%, but beyond these proportions, MDD decreases due to soil stiffness. At 7.5%, optimum moisture content (OMC) is attained, and after incorporating a stabilizer, the soil demonstrates a reduction in OMC.
- The CBR value primarily helps determine the soil's shear strength and bearing capacity. Test results indicate that the maximum value occurs at a 7.5% increase in CBA and WHA, followed by a decline. This decrease in value is primarily attributed to fluctuations in OMC, which impacts the CBR value.
- The UCS test was performed to determine the soil's shear strength, and the results showed a steady increase in strength with the addition of ashes, peaking at 7.5%. However, adding more ashes beyond that point led to a decline in strength.
- The findings of this research suggest that incorporating a combination of cattle bone ash and wheat husk ash as stabilizers in expansive soils can effectively improve their geotechnical properties, enhancing their suitability for the engineering applications.

5.2 Recommendation

- Further investigate the impact of varying mixing ratios of CBA and WHA in expansive soils, to potentially optimize the stabilizing effect and achieve even better geotechnical properties for a wider range of applications.
- Explore the long-term durability and performance of the stabilized soil using the recommended mixture of CBA and WHA, including potential environmental factors (such as temperature fluctuations and exposure to water) that might affect the soil properties over time.
- Conduct additional studies on a larger variety of expansive soils, to further validate the applicability and effectiveness of the proposed stabilizing mixture of CBA and WHA.

REFERENCES

AASHTO T-90. (2020). Determining the Plastic Limit and Plasticity Index of Soils. *Fop Library*, *90*(October), 1–4.

Adeboje, A., Kupolati, W., Sadiku, R., Ndambuki, J., Yussuf, D., & Kambole, C. (2017). Utilization of Pulverized Cow Bone (PCB) for Stabilizing Lateritic Soil for Road Work. *African Journal of Science, Technology, Innovation and Development*, *9*(4), 411–416. https://doi.org/10.1080/20421338.2017.1340395

Adetayo, O., Umego, O., Amu, O., Faluyi, F., Odetoye, A., & Bucknor, A. (2022). Performance Evaluation of Ternary Blends of Pulverized Cow Bone Ash and Waste Glass Powder on the Strength Properties of Concrete. *Trends in Sciences*, *19*(8), 2097–2106. https://doi.org/10.48048/tis.2022.3222

Alhassan, M. (2008). Potentials of Rice Husk Ash for Soil Stabilization. *A.U.J.T*, *11*(4), 246–250.

Antenane, N. (2020). Experimenting Expansive Soils Stablized with Wood Ash and Lime Targeting Geothenical Properties for Highway. In *Bahir Dar Institute of Technology*.

Ariana, R. (2016). Investigating Black Cotton Soil Blended with Basaltic Quarry Dust and Portland cement for Road Sub-grade Material. In *Addis Ababa Institute of Technology* (Issue September).

Ariyo, A., Joseph, A., Ayobami, B., Ebenezer, F., & Temidayo, F. (2021). Evaluation of Cow Bone Ash (CBA) as Additives in Stabilization of Lateritic and Termitaria Soil. *American Academic Scientific Research Journal for Engineering*, *80*(1), 136–144. http://asrjetsjournal.org/

ASTM-D-2216-98. (1998). Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. *ASTM International*, *January*, 1–5. https://doi.org/10.1520/D2216-10.1.5

ASTM C618, 99. (2014). Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use. *American Society for Testing and Materials International Standard*, 1–5. https://doi.org/10.1520/C0618

ASTM D 4318. (2000). Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. In *American Society for Testing and Materials International* *standard* (Vol. 04).

ASTM D 854 – 02. (2006). Standard Test Methods for of Soil Specific Gravity Solids by Water Pycnometer. *ASTM Standards*, *24*(1), 120432. https://doi.org/10.1016

ASTM D2166‒16. (2016). Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. *ASTM International, West Conshohocken, PA, D2166-16*(Reapproved), 1–6.

ASTM D6913. (2018). Particle Size Analysis. *Geo Testing Express*, *4*(0).

ASTM D7928. (2021). Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis. *ASTM International*, *May 2016*, 1–25. https://doi.org/10.1520/D7928-17

Bethel. (2021). Investigation on the Application of Natural Fiber (Katcha) As A Weak Subgrade Soil Reinforcement Blended with Sand: in Case of Koyefeche Project 18 Road Project. In *Adama Science and Technology University*.

Brooks, R. M., Udo-Inyang, P., & Al-Ayaydah, H. (2019). Soil Stabilization with Fly Ash and Wheat Husk Ash – Optimization. *International Journal of Applied Engineering Research*, *14*(7), 1539–1545.

Busscher, W. (1994). Fundamentals of Soil Behavior. In *Soil Science* (Vol. 158, Issue 1). United States of America. https://doi.org/10.1097/00010694-199407000-00009

Canakci, H., Aziz, A., & Celik, F. (2015). Soil Stabilization of Clay with Lignin, Rice Husk Powder and Ash. *Geomechanics and Engineering*, *8*(1), 67–79. https://doi.org/10.12989/gae.2015.8.1.067

Chethan, K. (2018). *L UPINE PUBLISHERS Stabilization of Expansive Soil Using Wheat Husk Ash and Granulated Blast Furnace Slag*. *1*(2), 29–35. https://doi.org/10.32474/TCEIA.2018.01.000108

Davidson, D. T. (2016). *· a State · iversity Bulletin*. *23*.

Dennis, K. C. (1994). Expansive Soils. *Construction Specifier*, *47*(11), 413–441. https://doi.org/10.1007/978-3-642-41714-6_52078

Eden Asres. (2017). Stabilization of Expansive Clay Soils Using Quarry Waste. *Addis Ababa Institute of Technology*, 77.

ERA. (2013). *Pavement Design Manual*.

Fadmoro, O. F., Kar, S. S., Tiwari, D., & Singh, A. (2022). Environmental and Economic Impact of Mixed Cow Dung and Husk Ashes in Subgrade Soil Stabilization. *International Journal of Pavement Research and Technology*, *15*(4), 835–846. https://doi.org/10.1007/s42947-021-00056-8

Fang, H.-Y. (2013). The Foundation Engineering Handbook. *The Foundation Engineering Handbook*. https://doi.org/10.1201/b15592

Feyissa, G. F., & Gebbisa, M. B. (2021). Choice and Determinants of Saving in Rural Households of Bale Zone: The Case of Agarfa District, Oromia, South East Ethiopia. *OALib*, *08*(02), 1–20. https://doi.org/10.4236/oalib.1107167

Genet. (2021). Stabilization of Expansive Subgrade Soils with Cement and Wood. In *Addis Ababa Institute of Technology*.

Hosseini, M. A. S. S. N. E. A. F. M. N. M. M. M. (2020). *Abbabspour*. ICE Virtual Library. https://doi.org/https://doi.org/10.1680/jgein.20.00010

IS, part 40 2720. (1977). IS 1977 Determination of Free Swell Index of Soils. *In Bureau of Indian Standard*, *3*, 1–6.

Jaleh Forouzan, A. (2016). *Prediction of Swelling Behavior of Expansive Soils Using Modified Free Swell Index, Methylene Blue and Swell Oedometer Tests*. *February*, 159.

keftaw, H. (2021). a Study on Strength Characteristics of Subgrade Soil. *Adama Science and Technology University*.

Labena, E. (2020). *Utilization of Plastic Waste and Bons as to Improve the Engineering Properties of Expansive soils* (Issue May).

Lancellotta, R. (1995). Geotechnical Engineering. In *Geotechnical engineering*. https://doi.org/10.1016/b978-0-12-409548-9.12508-4

Moniuddin, K. (2018). Stabilization of Expansive Soil Using Wheat Husk Ash and Granulated Blast Furnace Slag. *Trends in Civil Engineering and Its Architecture*, *1*(2), 29–35. https://doi.org/10.32474/tceia.2018.01.000108

Muhammad, G., & Marri, A. (2018). Immediate and Long-term Effects of Lime and Wheat Straw on Consistency Characteristics of Clayey Soil. *Geomechanics and Engineering*, *16*(3), 217–231. https://doi.org/10.12989/gae.2018.16.3.217

Obianyo, I. I., Onwualu, A. P., & Soboyejo, A. B. O. (2020). Mechanical Behaviour of Lateritic Soil Stabilized with Bone Ash and Hydrated Lime for Sustainable Building Applications. *Case Studies in Construction Materials*, *12*, e00331. https://doi.org/10.1016/j.cscm.2020.e00331

Ochieng, O. (2016). Characterization and Classification of Clay Minerals for Potential Applications in Rugi Ward, Kenya. *African Journal of Environmental Science and Technology*, *10*(11), 415–431. https://doi.org/10.5897/ajest2016.2184

Ojuri, O. O., Osagie, P. O., Oluyemi-Ayibiowu, B. D., Fadugba, O. G., Tanimola, M. O., Chauhan, V. B., & Jayejeje, O. O. (2022). Eco-friendly Stabilization of Highway Lateritic Soil with Cow Bone Powder Admixed Lime and Plastic Granules Reinforcement. *Cleaner Waste Systems*, *2*(December 2021), 100012. https://doi.org/10.1016/j.clwas.2022.100012

Sharma, V., Rathore, P. K., & Sharma, A. (2018). Soil Stabilization by Using Wheat Husk Ash. *Journal of Civil Engineering and Environmental Technology*, *5*(1), 31–35. http://www.krishisanskriti.org/Publication.html

Singh, M., & Sharma, R. (2017). Soil Stabilization Using Industrial Waste (Wheat Husk and Sugarcane Straw Ash). *International Research Journal of Engineering and Technology(IRJET)*, *4*(9), 589–596. https://irjet.net/archives/V4/i9/IRJET-V4I9100.pdf

Sinha, A. K., Jha, J. N., & Choudhary, A. K. (2019). A study on CBR Behaviour of Waste Pet Strip Reinforced Stone Dust. *Emirates Journal for Engineering*, *21 LNCE*(1), 302–312. https://doi.org/10.1007/978-3-030-02707-0_36

Tefera, T., & Yohannes, S. (1986). Investigations on the Expansive Soils of Addis Ababa. In *EAEA* (Vol. 7).

Tiwari, N., Satyam, N., & Kumar Shukla, S. (2020). An Experimental Study on Micro-structural and Geotechnical Characteristics of Expansive Clay Mixed with EPS Granules. *Soils and Foundations*, *60*(3), 705–713. https://doi.org/10.1016/j.sandf.2020.03.012

Tseganeh, A. B. (2012). Stabilization of Expansive Soil Using Biomedical Waste Using Incinerator Ash. In *Addis Ababa Institute of Technology* (Issue November).

Turi, T., Hayicho, H., & Kedir, H. (2019). Evaluating Land Use/Land Cover Change and

Its Socioeconomic Implications in Agarfa District of Bale Zone, Southeastern Ethiopia. *Journal of Environmental Protection*, *10*(03), 369–388. https://doi.org/10.4236/jep.2019.103022

V. Dakshanamurty, V. R. (1973). *A simple method of identifying an expansive soil*. https://doi.org/https://doi.org/10.3208/sandf1972.13.97

Vageesha S. Mathada, Sharankumar, A. A. K. 1Professor. (2020). Stabilisation of Expansive Soil Using Wheat Husk. *Emerging Technologies and Innovative Research*, *7*(7), 491–496. www.jetir.org

Yadu et al. (2011). Comparison of Fly Ash and Rice Husk Ash Stabilized Black Cotton Soil. In *International Journal of Earth Sciences and Engineering* (Vol. 4, Issue 6).

Zumrawi, M. M. E. (2015). Stabilization of Pavement Subgrade by Using Fly Ash Activated by Cement. *American Journal of Civil Engineering and Architecture*, *3*(6), 218–224. https://doi.org/10.12691/ajcea-3-6-5

APPENDICES

Appendix A. Natural Moisture Content of Soil Sample

Table A.1 Natural moisture content of soil sample collected from pit-1

Moisture Content Determination	Natural moisture content
Weight of Wet soil + cont. (g)	343.0
Weight of Dry soil + cont. (g)	319.0
Weight of Container (g)	66.0
Weight of water (g)	24.0
Weight of Dry soil (g)	253.0
Moisture content (%)	9.49

Table A.2 Natural moisture content of soil sample collected from pit-2

Moisture Content Determination	Natural moisture content
Weight of Wet soil + cont. (g)	461.0
Weight of Dry soil + cont. (g)	427.0
Weight of Container (g)	54.0
Weight of water (g)	34.0
Weight of Dry soil (g)	373.0
Moisture content (%)	9.12

Table A.3 Natural moisture content of soil sample collected from pit-3

Moisture Content Determination	Natural moisture content
Weight of Wet soil + cont. (g)	329.0
Weight of Dry soil + cont. (g)	339.0
Weight of Container (g)	70.0
Weight of water (g)	31.0
Weight of Dry soil (g)	269.0
Moisture content (%)	11.52

Table A.4 Natural moisture content of soil sample collected from pit-4

Moisture Content Determination	Natural moisture content
Weight of Wet soil + cont. (g)	375.0
Weight of Dry soil + cont. (g)	340.0
Weight of Container (g)	69.0
Weight of water (g)	35.0
Weight of Dry soil (g)	271.0
Moisture content (%)	12.92

Appendix B. Specific Gravity of Materials Used for the Study

Table A.5 Specific gravity of soil sample collected from pit-1

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	43.79	43.77
Weight of bottle+ sample, W2(gm)	53.79	17.4
Weight of (bottle + sample + Water), W3 (gm)	154	153.12
Weight of (bottle + water), W4 (gm)	147.8	147.41
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	3.8	4.29
Specific gravity of material = WS/Vs	2.6316	2.61
Correction Factor	0.9999	0.9997
Specific gravity of material at 20 °C, G_{20}	2.631	2.609
Average	2.620	

Table B.2 Specific gravity of soil sample collected from pit-2

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	43.85	43.7
Weight of bottle+ sample, W2(gm)	53.79	17.4
Weight of (bottle + sample + Water), W3 (gm)	154.1	153.82
Weight of (bottle + water), W4 (gm)	147.9	147.66
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	3.8	3.84
Specific gravity of material = WS/Vs	2.6316	2.6042
Correction Factor	0.9999	0.9997
Specific gravity of material at 20 °C, G_{20}	2.631	2.603
Average	2.617	

Table B.3 Specific gravity of soil sample collected from pit-3

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	43.85	43.7
Weight of bottle+ sample, W2(gm)	53.79	17.4
Weight of (bottle + sample + Water), W3 (gm)	153.9	153.76
Weight of (bottle + water), W4 (gm)	147.71	147.48
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	3.81	3.72
Specific gravity of material = WS/Vs	2.624672	2.6882
Correction Factor	0.99987	0.9997
Specific gravity of material at 20 °C, G_{20}	2.624	2.687
Average	2.656	

Table B.4 Specific gravity of soil sample collected from pit-4

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	43.85	43.7
Weight of bottle+ sample, W2(gm)	53.79	17.4
Weight of (bottle + sample + Water), W3 (gm)	154.15	153.83
Weight of (bottle + water), W4 (gm)	147.9	147.55
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	3.75	3.72
Specific gravity of material = WS/Vs	2.666667	2.6882
Correction Factor	0.99987	0.9997
Specific gravity of material at 20 °C, G_{20}	2.666	2.687
Average	2.677	

Table B.5 Specific gravity of WHA

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	44.35	44.75
Weight of bottle+ sample, W2(gm)	53.79	52.86
Weight of (bottle + sample + Water), W3 (gm)	153.5	153.8
Weight of (bottle + water), W4 (gm)	147.6	147.7
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	4.1	3.9
Specific gravity of material = WS/Vs	2.439	2.5641
Correction Factor	0.9999	0.9997
Specific gravity of material at 20 °C, G_{20}	2.439	2.563
Average	2.50	

Table B.6 Specific gravity of CBA

Specific gravity determination parameters	Bottle number	
	1	2
Weight of bottle, W1(gm)	43.85	43.75
Weight of bottle+ sample, W2(gm)	53.8	53.5
Weight of (bottle + sample + Water), W3 (gm)	153.2	153.6
Weight of (bottle + water), W4 (gm)	147.6	148.1
Weight of sample,Ws (gm)	10	10
Volume of sample, Vs = Ws+W4-W3	4.4	4.5
Specific gravity of material = WS/Vs	2.272727	2.2222
Correction Factor	0.99987	0.9997
Specific gravity of material at 20 °C, G_{20}	2.272	2.221
Average	2.247	

Appendix C. Swelling Properties of Materials

Table C.1 Free swell ratio and free swell index of soil sample and additives

Sample	V_K (ml)	V_W (ml)	FSI (%)	Degree of swell	FSR	Swell potential
pit-1	12	21.5	79.17	Very high	1.79	Moderate
pit-2	12	20.5	70.83	Very high	1.71	Moderate
pit-3	12	23	91.67	Very high	1.92	Moderate
pit-4	12	23.5	95.83	Very high	1.96	Moderate
WHA	10	10.3	3	Low	1.03	Low
CBA	10	10.05	0.5	Low	1.01	Low
CBA2.37+WHA0.13+BS97.5	11.5	20	73.91	Very high	1.74	Moderate
CBA4.75+WHA0.25+BS95	11	18	63.63	High	1.64	Moderate
CBA7.13+WHA0.38+BS92.5	11	17.5	59.09	High	1.59	Moderate
CBA9.5+WHA0.5+BS90	11	16.8	52.72	High	1.53	Moderate

Appendix D. Particle Size Distribution and Hydrometer Result

Table D.1 Particle size distribution and hydrometer result for pit-1

Wet Sieve Analysis — Total mass of sample, g — 1000

Sieve No	Sieve Opening (mm)	MS (g)	MS +MR (g)	MR (g)	(%)R	Cum. %R	Perc. Passing (%)
3/4''	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	533.9	538.4	4.5	0.5	0.5	99.6
No 4	4.75	563.7	568.6	4.9	0.5	0.9	99.1
No 8	2.36	532.5	533.4	0.9	0.1	1.0	99.0
No 10	2	519.4	519.7	0.3	0.0	1.1	98.9
No 16	1.18	502.9	506.0	3.1	0.3	1.4	98.6
No 30	0.6	451.7	460.3	8.6	0.9	2.2	97.8
No 40	0.425	431.8	438.8	7.0	0.7	2.9	97.1
No 50	0.3	430.9	438.8	7.9	0.8	3.7	96.3
No 100	0.15	402.5	425.9	23.4	2.3	6.1	93.9
No 200	0.075	390.8	410.7	19.9	2.0	8.1	92.0
pan	-------	409.3	409.3	0.0	0.0	8.1	92.0

Corr.Hydr.Reading (Rc)	Effe. Depth of Hydr. L (cm), from table	Value of K	Diameter of soil Particle D(mm)	% finer insuspension [P]	Adjusted Percen.Finner(%) PA
45	9.2	0.0135	0.057908549	91.3248	76.93201152
42	9.6	0.0135	0.04182822	85.2768	71.83717632
41	9.4	0.0135	0.029267303	83.2608	70.13889792
40	9.7	0.0135	0.021022756	81.2448	68.44061952
38	10.1	0.0135	0.015168738	77.2128	65.04406272
37	10.2	0.0135	0.011132385	75.1968	63.34578432
36	10.5	0.0135	0.007986708	73.1808	61.64750592
35	10.7	0.0137	0.005785446	70.56	59.439744
34	10.9	0.0137	0.004128984	68.544	57.7414656
33	11.1	0.0137	0.002946296	66.528	56.0431872
32	11.2	0.0138	0.002107985	63.9072	53.83542528
30	11.5	0.0138	0.001510401	59.8752	50.43886848
28	11.7	0.014	0.001261943	57.2544	48.23110656
27	11.9	0.014	0.000899923	55.2384	46.53282816

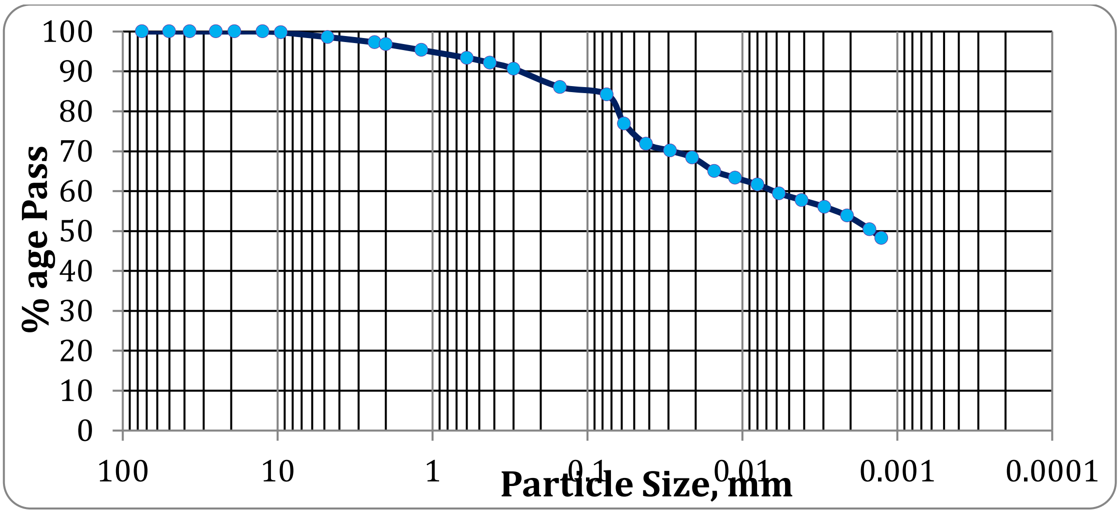


Figure D.1 Particle size distribution for pit-1

Table D.2 Particle size distribution and hydrometer result for pit-2

Wet Sieve Analysis			Total mass of sample, g				1000
Sieve	Sieve	MS	MS + MR	MR	(%)R	Cum. %R	(%)
No	(mm)	(g)	(g)	(g)			Passing
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	533.9	535.2	1.3	0.1	0.1	99.9
No 4	4.75	563.7	564.6	0.9	0.1	0.2	99.8
No 8	2.36	532.5	534.7	2.2	0.2	0.4	99.6
No 10	2	519.4	519.7	0.3	0.0	0.5	99.5
No 16	1.18	502.9	505.7	2.8	0.3	0.8	99.3
No 30	0.6	451.7	460.2	8.5	0.9	1.6	98.4
No 40	0.425	431.8	439.1	7.3	0.7	2.3	97.7
No 50	0.3	430.9	438.0	7.1	0.7	3.0	97.0
No 100	0.15	402.5	415.4	12.9	1.3	4.3	95.7
No 200	0.075	390.8	406.7	15.9	1.6	5.9	94.1
pan	-------	409.3	409.3	0.0	0.0	5.9	94.1

Actual Hyd. Reading (Ra)	(Rc)	L (cm)	Values of K	D(mm)	% finer insuspension [P]	Adjusted Percen.(%) PA
45	47	8.9	0.0135	0.056957	95.3568	89.71167744
44	45	9.1	0.0135	0.040724	91.3248	85.91837184
43	44	9.2	0.0135	0.028954	89.3088	84.02171904
41	42	9.6	0.0135	0.020914	85.2768	80.22841344
40	41	9.7	0.0135	0.014865	83.2608	78.33176064
39	40	9.7	0.0137	0.011017	80.64	75.866112
38	39	9.9	0.0137	0.00787	78.624	73.9694592
37	38	10.1	0.0137	0.005621	76.608	72.0728064
36	37	10.4	0.0137	0.004033	74.592	70.1761536
35	36	10.5	0.0138	0.002886	71.9712	67.71050496
33	34	10.9	0.0138	0.00208	67.9392	63.91719936
31	32	11.2	0.0138	0.001491	63.9072	60.12389376
30	30	11.4	0.014	0.001246	61.2864	57.65824512
29	29	11.5	0.014	0.000885	59.2704	55.76159232

Actual Hyd. Reading (Ra)	(Rc)	L (cm)	Values of K	D(mm)	% finer [P]	Adjusted %) PA
45	47	8.9	0.0135	0.056957	95.357	89.712
44	45	9.1	0.0135	0.040724	91.325	85.918
43	44	9.2	0.0135	0.028954	89.309	84.022
41	42	9.6	0.0135	0.020914	85.277	80.228
40	41	9.7	0.0135	0.014865	83.261	78.332
39	40	9.7	0.0137	0.011017	80.640	75.866
38	39	9.9	0.0137	0.00787	78.624	73.969
37	38	10.1	0.0137	0.005621	76.608	72.073
36	37	10.4	0.0137	0.004033	74.592	70.176
35	36	10.5	0.0138	0.002886	71.971	67.711
33	34	10.9	0.0138	0.00208	67.939	63.917
31	32	11.2	0.0138	0.001491	63.907	60.124
30	30	11.4	0.014	0.001246	61.286	57.658
29	29	11.5	0.014	0.000885	59.270	55.762

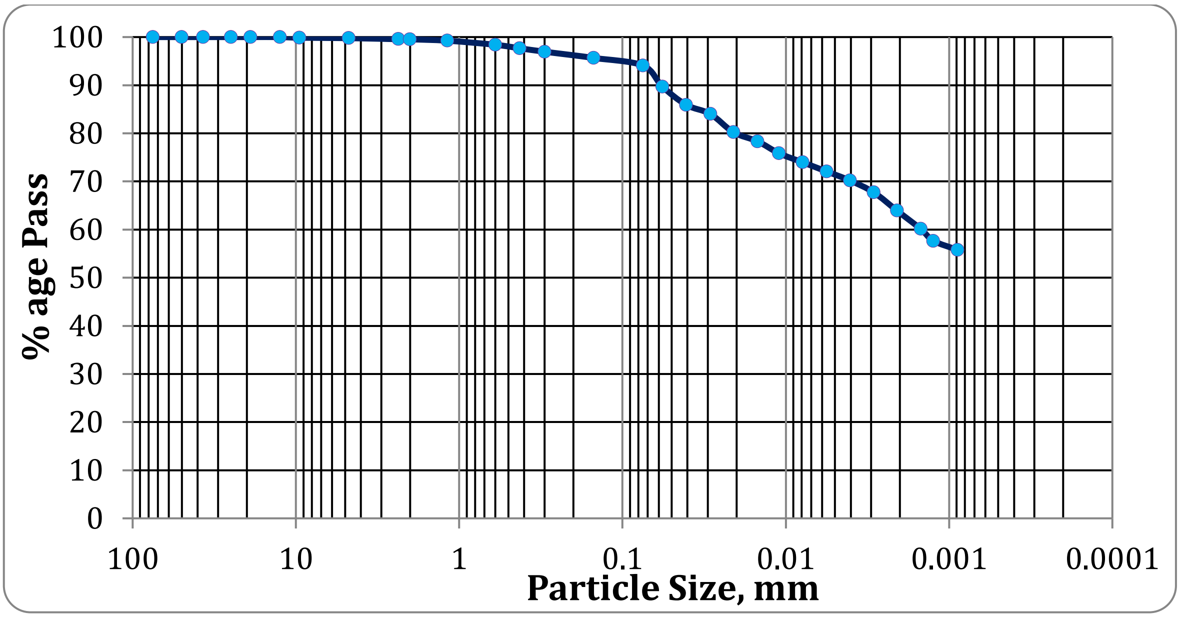


Figure D.2 Particle size distribution for pit-2

Table D.3 Particle size distribution and hydrometer result for pit-3

Sieve No	Sieve Opening (mm)	MS (g)	MS + MR (g)	MR (g)	Percentage Retained (%)	Cum. %R (%)	Perc. Passing (%)
3/4''	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	533.9	535.5	1.6	0.2	0.2	99.8
No 4	4.75	563.7	569.1	5.4	0.5	0.7	99.3
No 8	2.36	532.5	548.9	16.4	1.6	2.3	97.7
No 10	2	519.4	524.2	4.8	0.5	2.8	97.2
No 16	1.18	502.9	520.4	17.5	1.8	4.6	95.4
No 30	0.6	451.7	498.5	46.8	4.7	9.3	90.8
No 40	0.425	431.8	485.3	53.5	5.4	14.6	85.4
No 50	0.3	430.9	496.9	66.0	6.6	21.2	78.8
No 100	0.15	402.5	521.3	118.8	11.9	33.1	66.9
No 200	0.075	390.8	465.5	74.7	7.5	40.6	59.5
pan	-------	409.3	409.3	0.0	0.0	40.6	59.5

Actual Hyd. Reading (Ra)	(Rc)	L (cm),	Values of K	D(mm)	% finer [P]	Adjusted Percen.Finner(%) PA
40.00	42	9.7	0.0135	0.059461	85.2768	50.697
39	40	9.9	0.0135	0.042477	81.2448	48.300
38	39	10.1	0.0137	0.030787	78.624	46.742
34	35	10.7	0.0135	0.02208	71.1648	42.307
33	34	10.9	0.0135	0.015758	69.1488	41.109
31	32	11.2	0.0135	0.011665	65.1168	38.712
30	31	11.4	0.0135	0.008322	63.1008	37.513
28	29	11.7	0.0135	0.005961	59.0688	35.116
26	27	12	0.0137	0.004332	54.432	32.360
24	25	12.4	0.0137	0.003114	50.4	29.963
22	23	12.7	0.0137	0.002228	46.368	27.566
21	21	12.9	0.014	0.001623	43.1424	25.648
20	20	13	0.014	0.00133	41.1264	24.450
20	20	13	0.014	0.000941	41.1264	24.450

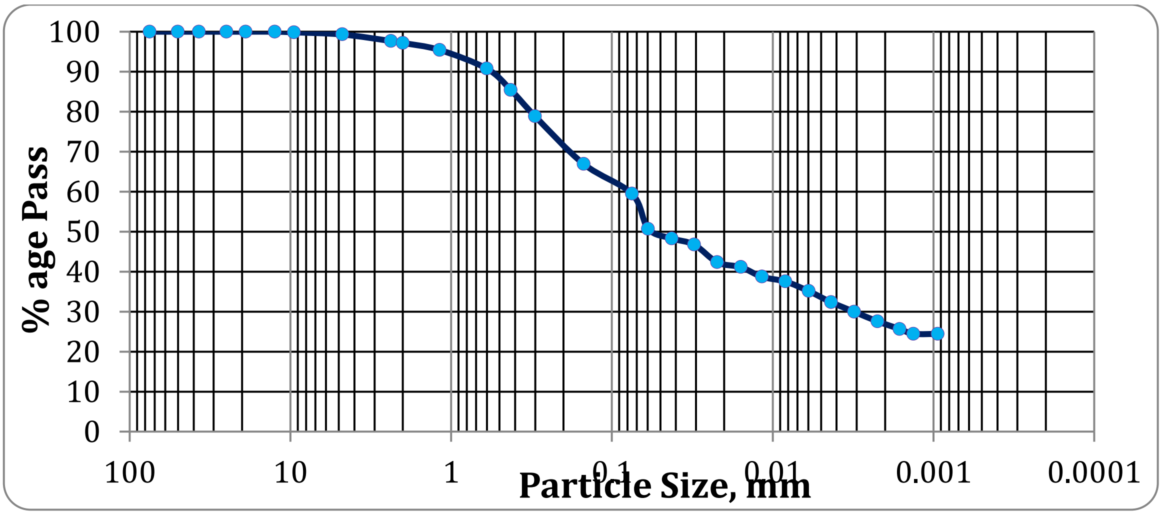


Figure D.3 Particle size distribution for pit-3

Table D.4 Particle size distribution and hydrometer result for pit-4

Wet Sieve Analysis			Total mass of sample, g				1000
Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	MS + MR (g)	MR (g)	%R (%)	%Cum. (%)	Perc. Passing (%)
3/4"	19.0	555.4	555.4	0.0	0.0	0.0	100.0
1/2"	12.5	560.7	560.7	0.0	0.0	0.0	100.0
3.8"	9.5	533.9	536.0	2.1	0.2	0.2	99.8
No 4	4.75	563.7	575.6	11.9	1.2	1.4	98.6
No 8	2.36	532.5	545.6	13.1	1.3	2.7	97.3
No 10	2	519.4	523.9	4.5	0.5	3.2	96.8
No 16	1.18	502.9	517.8	14.9	1.5	4.7	95.4
No 30	0.6	451.7	471.2	19.5	2.0	6.6	93.4
No 40	0.425	431.8	444.1	12.3	1.2	7.8	92.2
No 50	0.3	430.9	445.9	15.0	1.5	9.3	90.7
No 100	0.15	402.5	448.5	46.0	4.6	13.9	86.1
No 200	0.075	390.8	409.1	18.3	1.8	15.8	84.2
pan	-------	409.3	409.3	0.0	0.0	15.8	84.2

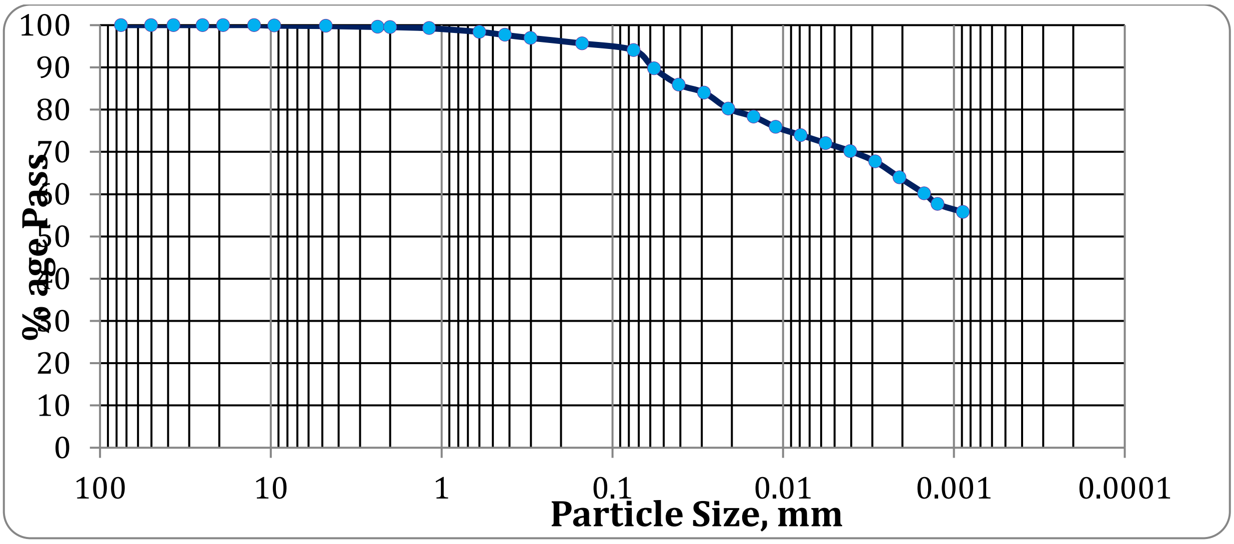


Figure D.4 Particle distribution for pit-4

Appendix E. Atterberg Limit Test of Results

Table E.1 Liquid limit and plastic limit of natural soil from pit-1

	Liquid Limit			Plastic Limit	
Wt. of cont. + wet soil (g) = (w_1)	37.57	36.31	38.81	26.51	26.51
Wt. of cont. + dry soil (g.) = (w_2)	30.52	29.68	30.93	25.50	25.55
Wt. of container (g.) = (w_3)	21.84	21.57	22.67	22.65	22.87
Mass of moisture (g.) (w_1-w_2) = x	7.05	6.63	6.88	1.01	0.96
Wt. of dry soil (g.) (w_2-w_3) = y	8.68	8.11	8.26	2.85	2.68
Moisture Content (%) = (100x/y)	81.22	81.75	83.29	35.44	35.82
No. of Blows	33	28	19		
		LL	**82.2**	**PL**	**35.63**
		Plasticity Index		**46.57**	

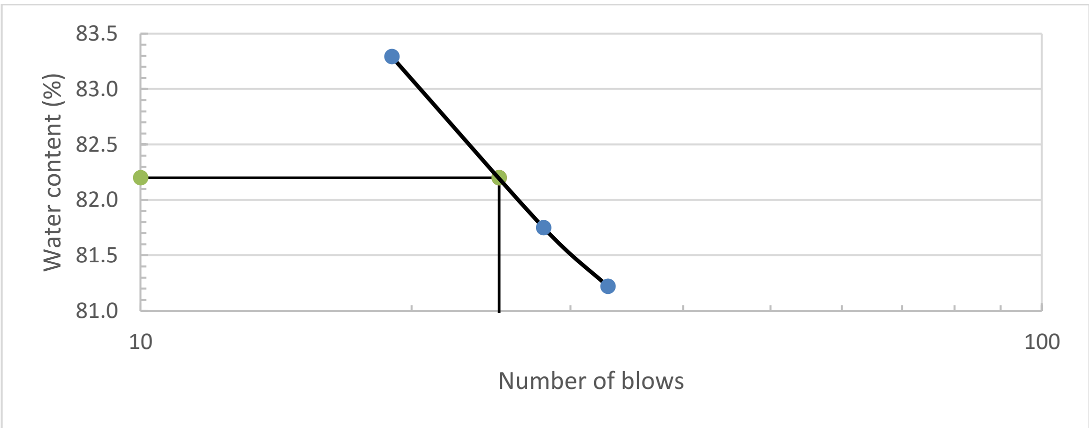


Figure E.1 Moisture content versus number of blows graph for natural soil from pit-1

Table E.2 Liquid limit and plastic limit of natural soil from pit-2

	Liquid Limit			Plastic Limit	
Wt. of cont. + wet soil (g) = (w_1)	46.75	42.62	41.33	26.29	26.29
Wt. of cont. + dry soil (g.) = (w_2)	35.21	32.94	32.40	25.23	25.20
Wt. of container (g.) = (w_3)	22.12	22.08	22.63	22.00	21.93
Mass of moisture (g.) (w_1-w_2) = x	11.54	9.68	8.93	1.06	1.09
Wt. of dry soil (g.) (w_2-w_3) = y	13.04	10.86	9.77	3.23	3.27
Moisture Content (%) = (100x/y)	88.50	89.13	91.40	32.82	33.33
No. of Blows	33	26	18		
	89.35			**33.08**	
	Plasticity Index			**56**	

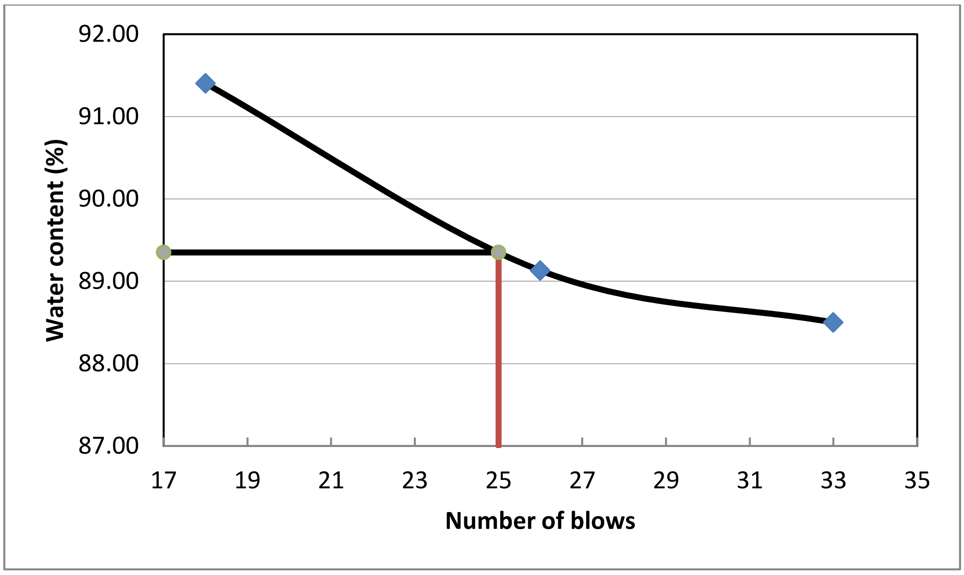


Figure E.2 Moisture content versus number of blows graph for natural soil from pit-2

Table E.3 Liquid limit and plastic limit of natural soil from pit-3

	Liquid Limit			Plastic Limit	
Wt. of cont. + wet soil (g) = (w_1)	46.39	43.37	43.26	26.59	28.47
Wt. of cont. + dry soil (g.) = (w_2)	34.21	32.44	37.47	25.38	26.61
Wt. of container (g.) = (w_3)	22.42	22.18	32.09	22.71	22.58
Mass of moisture (g.) (w_1-w_2) = x	12.18	10.93	5.79	1.21	1.86
Wt. of dry soil (g.) (w_2-w_3) = y	11.57	10.26	5.38	2.67	4.03
Moisture Content (%) = (100x/y)	105.27	106.53	107.62	45.32	46.15
No. of Blows	34	28	19		
	106.93			**45.74**	
	Plasticity Index			**61.2**	

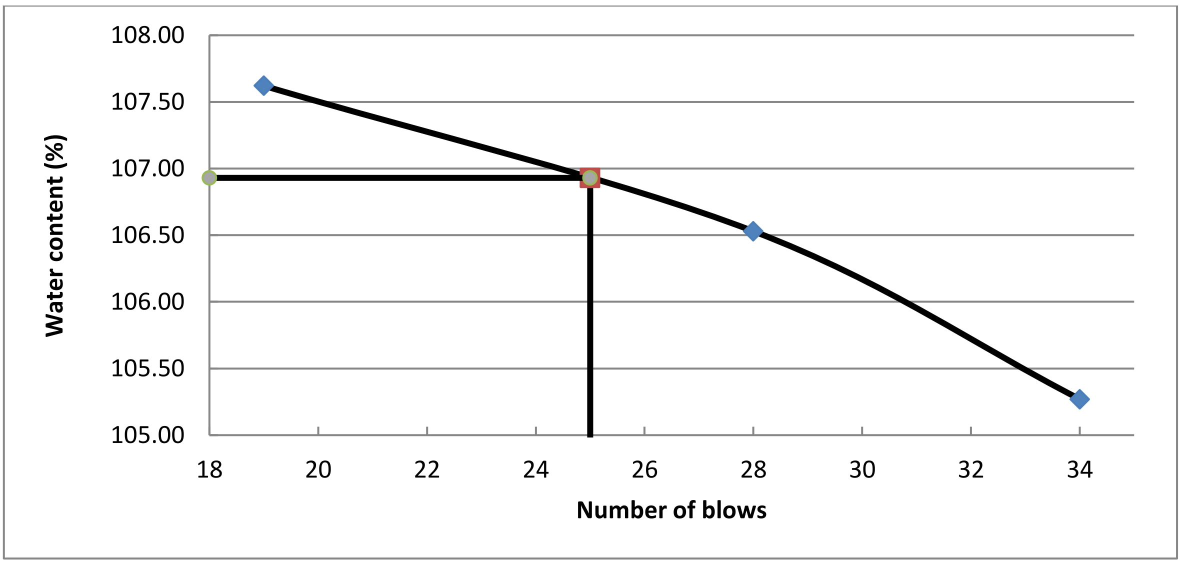


Figure E.3 Moisture content versus number of blows graph for natural soil from pit-3

Table E.4 Liquid limit and plastic limit of natural soil from pit-4

	Liquid Limit			Plastic Limit	
Wt. of cont. + wet soil (g) = (w_1)	41.76	44.88	42.25	29.65	10.89
Wt. of cont. + dry soil (g.) = (w_2)	31.42	32.79	31.68	27.69	28.45
Wt. of container (g.) = (w_3)	22.42	22.41	22.61	22.45	21.91
Mass of moisture (g.) (w_1-w_2) = x	10.34	12.09	10.57	1.96	2.44
Wt. of dry soil (g.) (w_2-w_3) = y	9.00	8.11	9.07	5.24	6.54
Moisture Content (%) = (100x/y)	114.89	116.47	116.54	37.40	37.31
No. of Blows	34	27	18		
	116.6			**37.36**	
	Plasticity Index			**79**	

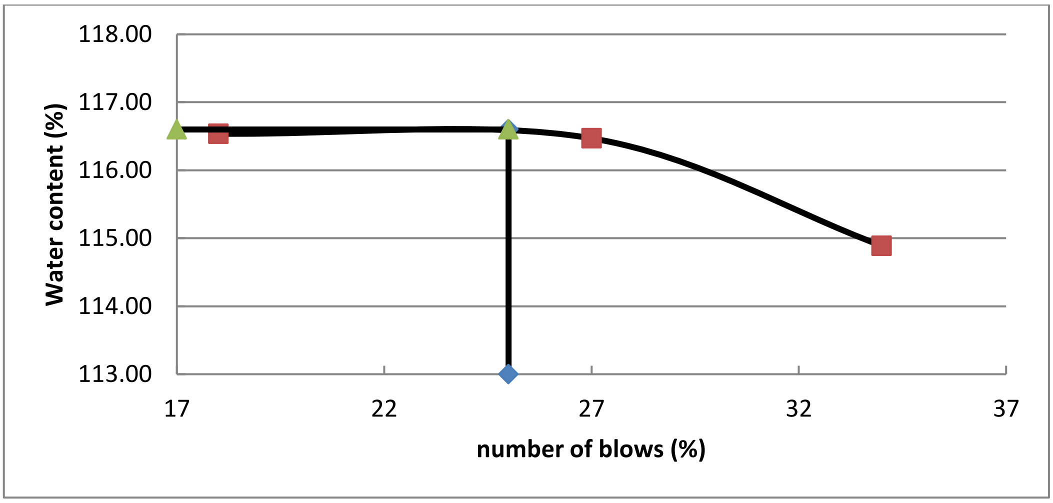


Figure E.4 Moisture content versus number of blows graph for natural soil from pit-4

Table E.5 Atterberg limits of 2.5% blended CBA and WHA on Soil

	Liquid Limit			Plastic Limit	
No. of Blows	34	28	23		
Wt. of cont. + wet soil (g) = (w_1)	55.91	57.95	59.91	26.60	26.80
Wt. of cont. + dry soil (g.) = (w_2)	40.45	41.90	42.20	25.47	25.45
Wt. of container (g.) = (w_3)	22.64	22.63	22.79	22.61	22.59
Mass of moisture (g.) (w_1-w_2) = x	15.46	16.05	17.71	1.13	1.35
Wt. of dry soil (g.) (w_2-w_3) = y	17.81	19.27	19.41	2.86	2.86
Moisture Content (%) = (100x/y)	86.81	83.29	91.24	39.51	47.20
	87.1			**43.36**	
	Plasticity Index			**44**	

Table E.6 Atterberg limits of 5% blended CBA and WHA on soil

	Liquid Limit			Plastic Limit	
No. of Blows	33	28	26		
Wt. of cont. + wet soil (g) = (w_1)	54.41	56.85	58.51	26.70	26.70
Wt. of cont. + dry soil (g.) = (w_2)	41.55	42.50	43.20	25.57	25.55
Wt. of container (g.) = (w_3)	22.64	22.63	22.79	22.61	22.59
Mass of moisture (g.) (w_1-w_2) = x	12.86	14.35	15.31	1.13	1.15
Wt. of dry soil (g.) (w_2-w_3) = y	18.91	19.87	20.41	2.96	2.96
Moisture Content (%) = (100x/y)	68.01	72.22	75.01	38.18	38.85
	71.7			**38.51**	
	Plasticity Index			**33**	

Table E.7 Atterberg limits of 7.5% blended CBA and WHA on soil

	Liquid Limit			Plastic Limit	
No. of Blows	33	28	22		
Wt. of cont. + wet soil (g) = (w_1)	50.72	50.70	50.75	26.54	26.54
Wt. of cont. + dry soil (g.) = (w_2)	40.37	40.14	40.03	25.50	25.55
Wt. of container (g.) = (w_3)	22.48	22.52	22.60	22.53	22.55
Mass of moisture (g.) (w_1-w_2) = x	10.35	10.56	10.72	1.04	0.99
Wt. of dry soil (g.) (w_2-w_3) = y	17.89	17.62	17.43	2.97	3.00
Moisture Content (%) = (100x/y)	57.85	59.93	61.50	35.02	33.00
	59.8			**34.01**	
	Plasticity Index			**26**	

Table E.8 Atterberg limits of 10% blended CBA and WHA on soil

	Liquid Limit			Plastic Limit	
No. of Blows	33	30	27		
Wt. of cont. + wet soil (g) = (w_1)	53.85	55.73	58.13	26.15	26.10
Wt. of cont. + dry soil (g.) = (w_2)	41.30	42.38	43.77	25.16	25.25
Wt. of container (g.) = (w_3)	22.64	22.70	22.76	22.45	22.83
Mass of moisture (g.) (w_1-w_2) = x	12.55	13.35	14.36	0.99	0.85
Wt. of dry soil (g.) (w_2-w_3) = y	18.66	19.68	21.01	2.71	2.42
Moisture Content (%) = (100x/y)	67.26	67.84	68.35	36.53	35.12
	67.8			**35.83**	
	Plasticity Index			**32**	

Appendix F. Modified proctor compaction

Table F.1 Modified proctor compaction for pit-1

Trial No.	1	2	3	4	
Weight of Mould + Wet soil (g)	8296	8470	8600	8593	
Weight of Mould (g)	4732	4732	4732	4732	
Weight of Wet soil (g)	3564	3738	3868	3861	
Volume of Mould (cc)	2124	2124	2124	2124	
Bulk density (g/cc)	1.68	1.76	1.82	1.82	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	380.000	427.00	424.00	428.00	
Weight of Dry soil + cont. (g)	324.000	356.00	348.00	346.00	
Weight of Container (g)	75.000	65.000	66.000	70.000	
Weight of water (g)	56.000	71.000	76.000	82.000	
Weight of Dry soil (g)	249.000	291.00	282.00	276.00	
Moisture content (%)	22.490	24.399	26.950	29.700	
Dry Density (g/cc)	1.370	1.415	1.434	1.402	
	MDD, g/cc	**1.435**	**OMC, %**		**26.7**

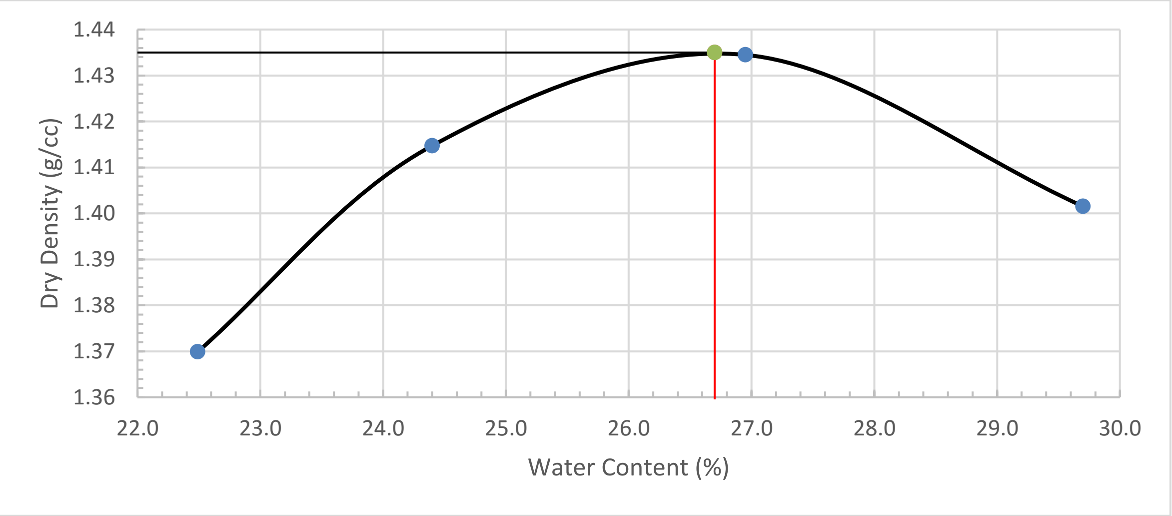


Figure F.1 Modified proctor compaction for pit-1

Table F.2 Modified proctor compaction for pit-2

Trial	1	2	3	4
Weight of Mould + Wet soil (g)	8218	8482	8748	8650
Weight of Mould (g)	4738	4738	4738	4738
Weight of Wet soil (g)	3480	3744	4010	3912
Volume of Mould (cc)	2124	2124	2124	2124

Wet density (g/cc)		1.64	1.76	1.89	1.84
Moisture Content Determination					
Weight of Wet soil + cont. (g)		359.0	358.0	358	360
Weight of Dry soil + cont. (g)		308.0	302.0	297	293
Weight of Container (g)		75.0	69.0	74	68
Weight of water (g)		51	56.0	61	67
Weight of Dry soil (g)		233	233	223	225
Moisture content (%)		21.89	24.03	27.35	29.28
Dry Density (g/cc)		1.34	1.42	1.48	1.42
		MDD, g/cc	**1.482**	**OMC, %**	27.20

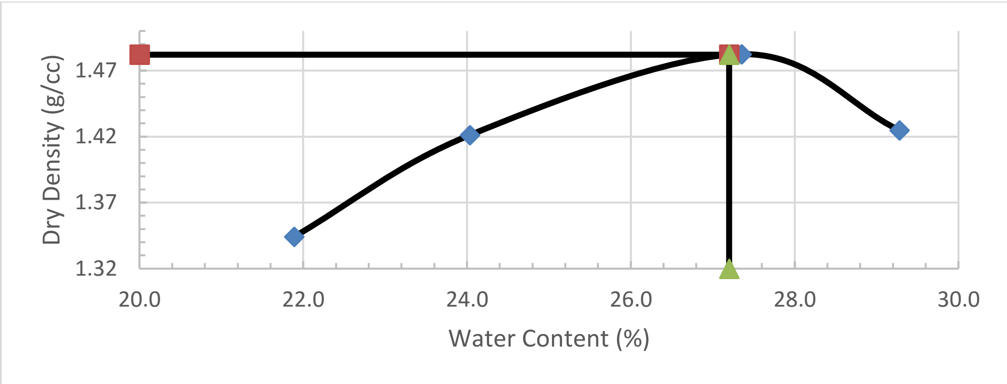


Figure F.2 Modified proctor compaction for pit-2

Table F.3 Modified proctor compaction for pit-3

Trial No.		1	2	3	4
Weight of Mould + Wet soil (g)		8050	8298	8560	8505
Weight of mould (g)		4738	4738	4738	4738
Weight of Wet soil (g)		3312	3560	3822	3767
Volume of Mould (cc)		2124	2124	2124	2124
Wet density (g/cc)		1.56	1.68	1.80	1.77
Moisture Content Determination					
Weight of Wet soil + cont. (g)		355.0	354.0	366	357
Weight of Dry soil + cont. (g)		303.0	296.0	299	286
Weight of Container (g)		75.0	67.0	58.5	57
Weight of water (g)		52	58.0	67	71
Weight of Dry soil (g)		228	229	240.5	229
Moisture content (%)		22.81	25.33	27.86	31.00
Dry Density (g/cc)		1.27	1.34	1.41	1.35
		MDD, g/cc	**1.41**	**OMC, %**	27.86

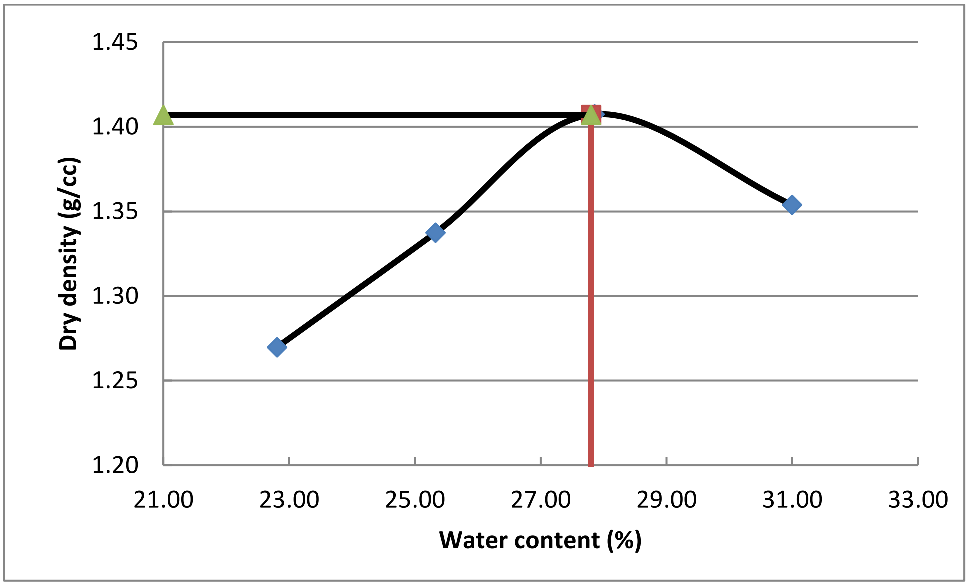


Figure F.3 Modified proctor compaction for pit-3

Table F.4 Modified proctor compaction for pit-4

Trial No.	1	2	3	4
Weight of Mould + Wet soil (g)	8050	8350	8510	8406
Weight of mould, (g)	4731	4731	4731	4731
Weight of Wet soil (g)	3319	3738	3779	3675
Volume of Mould (cc)	2124	2124	2124	2124
Wet density (g/cc)	1.56	1.76	1.78	1.73
Moisture Content Determination				
Weight of Wet soil + cont. (g)	350.0	367.0	367	374
Weight of Dry soil + cont. (g)	302.0	303.5	302	300
Weight of Container (g)	75.0	72.0	71	65
Weight of water (g)	48	63.5	65	74
Weight of Dry soil (g)	227	232	231	235
Moisture content (%)	21.15	27.43	28.14	31.49
Dry Density (g/cc)	1.29	1.38	1.39	1.32
	MDD, g/cc	1.39	OMC, %	28

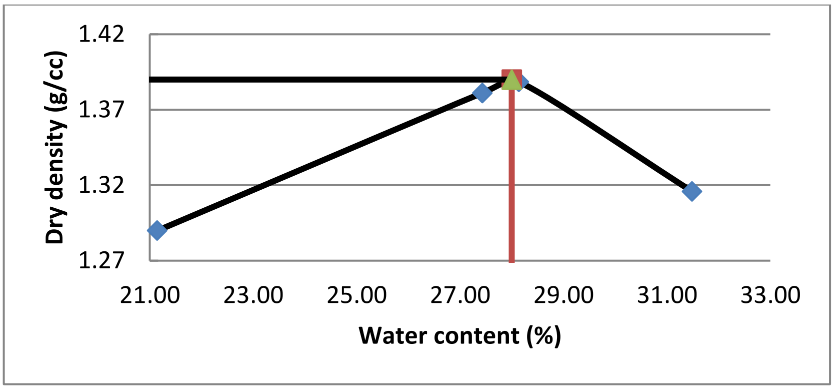


Figure F.4 Modified proctor compaction for pit-4

Table F.5 MPC of 2.5% blended CBA and WHA on soil

Trial No.	1	2	3	4	
Weight of mould + wet soil (g)	8296	8470	8610	8593	
Weight of Mould (g)	4732	4732	4732	4732	
Weight of Wet soil (g)	3564	3738	3878	3861	
Volume of Mould (cc)	2124	2124	2124	2124	
Bulk density (g/cc)	1.68	1.76	1.83	1.82	
Moisture Content Determination					
Weight of Wet soil + cont. (g)	380.000	427.000	427.000	428.000	
Weight of Dry soil + cont. (g)	324.000	356.500	348.800	346.000	
Weight of Container (g)	75.000	65.000	62.000	70.000	
Weight of water (g)	56.000	70.500	78.200	82.000	
Weight of Dry soil (g)	249.000	291.500	286.800	276.000	
Moisture content (%)	22.490	24.185	27.266	29.700	
Dry Density (g/cc)	1.370	1.417	1.435	1.402	
	MDD, g/cc	**1.435**	**OMC, %**		**27.266**

Table F.6 MPC of 5% blended CBA and WHA on soil

Trial No.	1	2	3	4
Weight of Mould + Wet soil (g)	8665	8784	8868	8895
Weight of mould(g)	4971	4971	4971	4971
Weight of Wet soil (g)	3694	3813	3897	3923
Volume of Mould (cc)	2124	2124	2124	2124
Wet density (g/cc)	1.74	1.80	1.83	1.85

Moisture Content Determination					
Weight of Wet soil + cont. (g)		452.0	455.0	456.5	467
Weight of Dry soil + cont. (g)		378.0	377.0	371.5	370
Weight of Container (g)		64.0	65.0	55	57
Weight of water (g)		74	78.0	85	97
Weight of Dry soil (g)		314	312	316.5	313
Moisture content (%)		23.57	25.00	26.86	30.99
Dry Density (g/cc)		1.41	1.44	1.45	1.41
		MDD, g/cc	**1.446**	**OMC, %**	**26.86**

Table F.7 MPC of 7.5% blended CBA and WHA on soil

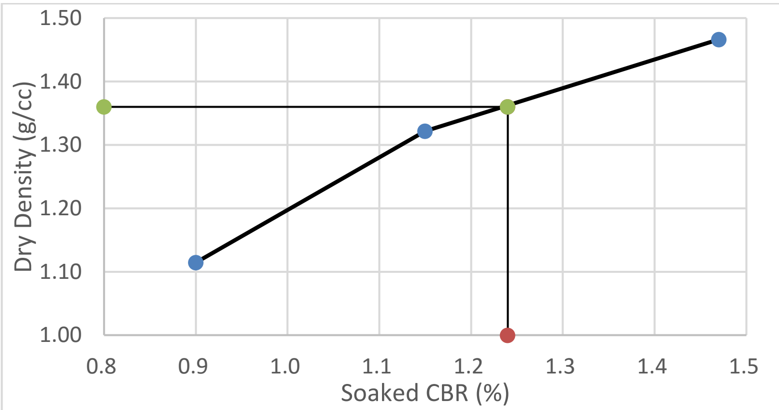
Trial		1	2	3	4
Weight of Mould + Wet soil (g)		8665	8784	8858	8895
Weight of Mould (g)		4971	4971	4971	4971
Weight of Wet soil (g)		3694	3813	3923	3923
Volume of Mould (cc)		2124	2124	2124	2124
Wet density (g/cc)		1.74	1.80	1.85	1.85
Moisture Content Determination					
Weight of Wet soil + cont. (g)		445.0	450.0	452	462
Weight of Dry soil + cont. (g)		375.0	377.0	372	376
Weight of Container (g)		65.0	67.0	55	57
Weight of water (g)		70	73.0	80	86
Weight of Dry soil (g)		310	310	317	319
Moisture content (%)		22.58	23.55	25.24	26.95
Dry Density (g/cc)		1.42	1.45	1.47	1.45
		MDD, g/cc	**1.475**	**OMC, %**	**25.24**

Table F.8 MPC of 10% blended CBA and WHA on soil

Trial	1	2	3	4
Weight of Mould + Wet soil (g)	8100	8259	8450	8430
Weight of mould (g)	4738	4738	4738	4738
Weight of Wet soil (g)	3362	3521	3712	3692
Volume of Mould (cc)	2124	2124	2124	2124

Wet density (g/cc)		1.58	1.66	1.75	1.74
Moisture Content Determination					
Weight of Wet soil + cont. (g)		350.0	345.0	349	348
Weight of Dry soil + cont. (g)		303.0	295.0	295	286
Weight of Container (g)		75.0	67.0	68	59
Weight of water (g)		47	50.0	54	62
Weight of Dry soil (g)		228	228	227	227
Moisture content (%)		20.61	21.93	23.79	27.31
Dry Density (g/cc)		1.31	1.36	1.41	1.37
		MDD, g/cc	**1.412**	**OMC, %**	**23.79**

Appendix G. California Bearing Ratio for Natural Soil



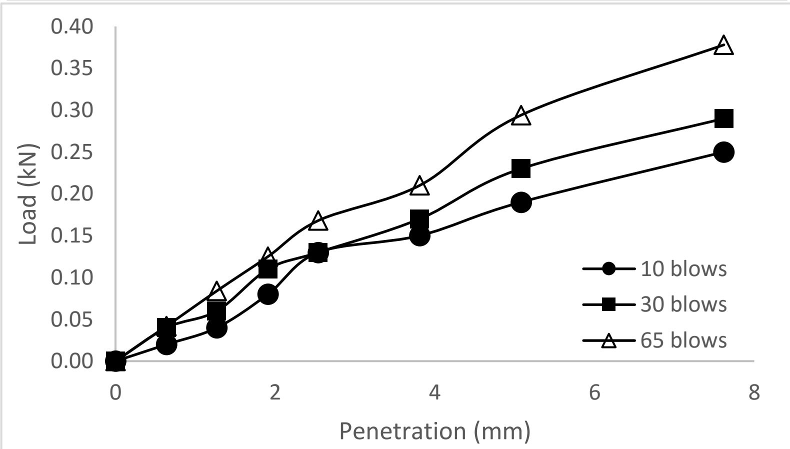
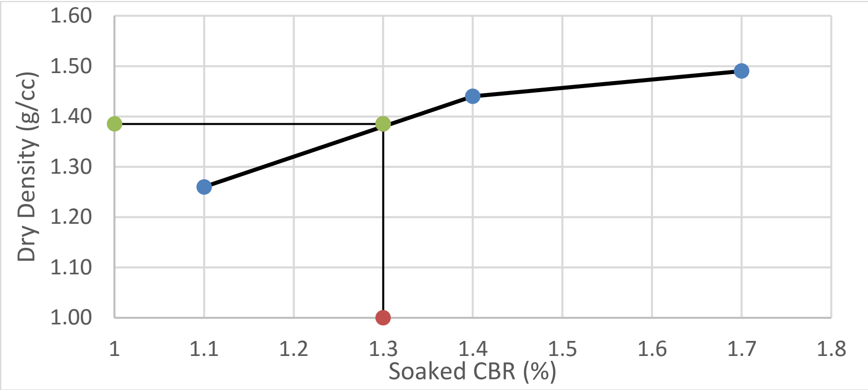


Figure G.1 Soaked CBR and load penetration chart for pit-1



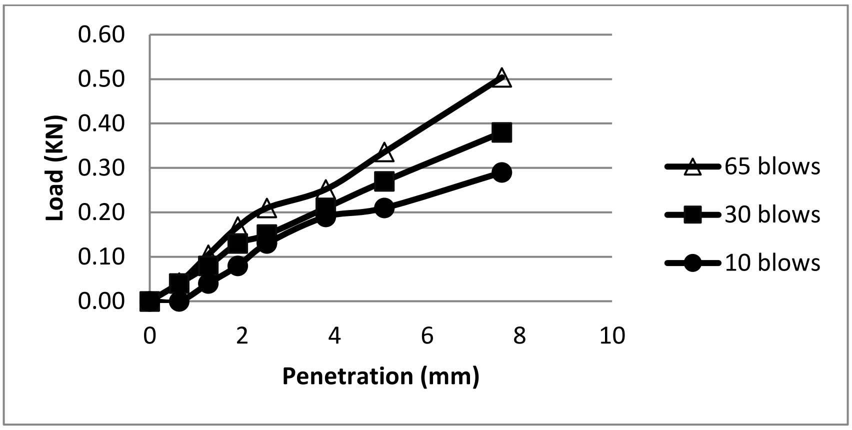
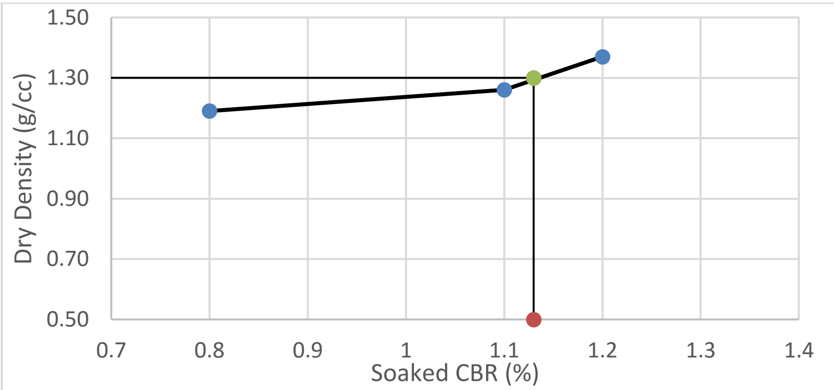


Figure G.2 Soaked CBR and Load penetration for pit-2



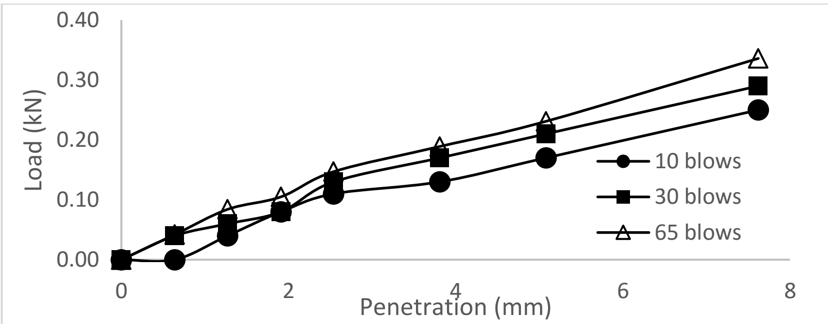


Figure G.3 Soaked CBR and Load penetration for pit-3

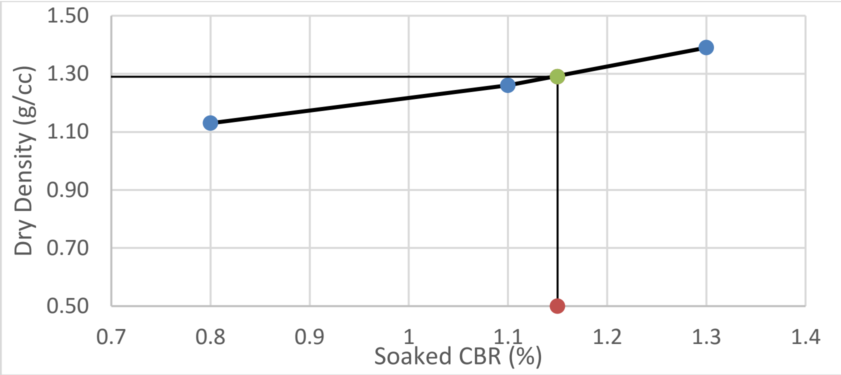


Figure G.4 Soaked CBR for pit-4

Table G.1 Soaked CBR Value for 2.5% blended CBA and WHA on soil

No. of Blows per Layer		10		30		65	
CONDITION OF SAMPLE		Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt.of wet sample + mould, g	W1	9150	9758	9685	10189	10488	10870
Wt.of mould, g	W2	5678	5678	5849	5849	6521	6521
Wt.of wet sample, g	W3 = W1 - W2	3472	4080	3836	4340	3967	4349
Volume of mould, cc	V	2123	2123	2123	2123	2123	2123
Wet unit weight, g/cc	γw = W3 / V	1.64	1.92	1.81	2.04	1.87	2.05
Dry unit weight, g/cc	γw /(1+ W8/100)	1.33	1.35	1.45	1.51	1.50	1.52
Moisture Content Determination							
Wt. of wet sample + cont., g	W3	416	440	469	441	409	442
Wt. of dry sample + cont., g	W4	350	327	390	342	340	345
Wt. of water, g	W5 = W3 - W4	66	113	79	99	69	97
Wt. of container, g	W6	64	57	64	61	56	64
Wt. of dry sample, g	W7 = W4 - W6	286	270	326	281	284	281
Moisture Content, %	W8 = W5/W7*100	23.1	41.9	24.2	35.2	24.3	34.5

CBR DATA (4 days Soaked)

Penet-ration	Std load (kN)	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %
0		0	0.00			0	0.00			0	0.00		
0.64		0.5	0.06			1	0.11			1	0.11		
1.27		1	0.11			1.5	0.17			2.5	0.28		
1.91		2	0.22			2.5	0.28			3.5	0.39		
2.54	13.35	2.5	0.28	0.28	**2.1**	3	0.33	0.33	**2.5**	4.5	0.50	0.50	**3.7**
3.81		3.5	0.39			4	0.44			6	0.66		
5.08	20	4.5	0.50	0.50	**2.5**	5	0.55	0.55	**2.8**	7	0.77	0.77	**3.9**
7.62		5.5	0.61			7	0.77			8	0.88		
Soaked CBR, %		2.5				2.8				3.9			
Dry Density, g/cc		1.33				1.45				1.50			
Swell, %		7.76				6.03				5.17			
Density Requirement:			95%		Target Density:			1.40		CBR		2.63	

Table G.2 Soaked CBR value for 5% blended CBA and WHA on soil

No. of Blows per Layer		10		30		65	
CONDITION OF SAMPLE		Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt.of wet sample + mould,	W1	9145	9767	9665	10199	10498	10880
Wt.of mould, g	W2	5678	5678	5849	5849	6521	6521
Wt.of wet sample, g	W3 = W1 - W2	3467	4089	3816	4350	3977	4359
Volume of mould, cc	V	2123	2123	2123	2123	2123	2123
Wet unit weight, g/cc	γw = W3 / V	1.63	1.93	1.80	2.05	1.87	2.05
Dry unit weight, g/cc	γw /(1+ W8/100)	1.33	1.36	1.45	1.52	1.52	1.54
Moisture Content Determination							
Wt. of wet sample + cont.,	W3	415	438	468	440	407	439
Wt. of dry sample + cont., g	W4	350	327	390	342	340	345
Wt. of water, g	W5 = W3 - W4	65	111	78	98	67	94
Wt. of container, g	W6	64	57	64	61	56	64
Wt. of dry sample, g	W7 = W4 - W6	286	270	326	281	284	281
Moisture Content, %	W8 = W5/W7*100	22.7	41.1	23.9	34.9	23.6	33.5

CBR DATA (4 days Soaked)

Penet-ration	Std load (kN)	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %	Gauge reading	Load kN	Corrected CBR kN	Corrected CBR %
0		0	0.00			0	0.00			0	0.00		
0.64		0.5	0.06			1	0.11			1	0.11		
1.27		1.5	0.17			2.5	0.28			3	0.33		
1.91		2.5	0.28			3.5	0.39			4.5	0.50		
2.54	13.35	5	0.55	0.55	**4.1**	5.5	0.61	0.61	**4.5**	6	0.66	0.66	**4.9**
3.81		4.5	0.50			7	0.77			7	0.77		
5.08	20	7	0.77	0.77	**3.9**	8	0.88	0.88	**4.4**	9	0.99	0.99	**5.0**
7.62		8	0.88			10.5	1.16			11	1.21		
Soaked CBR, %		4.1				4.5				5.0			
Dry Density, g/cc		1.33				1.80				1.50			
Swell, %		7.50				5.60				4.74			
Density Requirement:			95%		Target Density:				1.41	CBR		4.19	

Table G.3 Soaked CBR value for 7.5% blended CBA and WHA on soil

No. of Blows per Layer		10		30		65	
CONDITION OF SAMPLE		Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt.of wet sample + mould,	W1	9900	10457	9814	10201	9982	10304
Wt.of mould, g	W2	6432	6432	5991	5991	6003	6003
Wt.of wet sample, g	W3 = W1 - W2	3468	4025	3823	4210	3979	4301
Volume of mould, cc	V	2123	2123	2123	2123	2123	2123
Wet unit weight, g/cc	γw = W3 / V	1.63	1.90	1.80	1.98	1.87	2.03
Dry unit weight, g/cc	γw /(1+ W8/100)	1.30	1.37	1.44	1.41	1.51	1.56
Moisture Content Determination							
Wt. of wet sample + cont.,	W3	423	436	428	447	432	435
Wt. of dry sample + cont., g	W4	349	332	355	337	358	348
Wt. of water, g	W5 = W3 - W4	74	104	73	110	74	87
Wt. of container, g	W6	58	64	65	64	55	56
Wt. of dry sample, g	W7 = W4 - W6	291	268	290	273	303	292
Moisture Content, %	W8 = W5/W7*100	25.4	38.8	25.2	40.3	24.4	29.8

CBR DATA (4 days Soaked)													
Penet-ration	Std load (kN)	Gauge reading	Load kN	Corrected CBR kN	%	Gauge reading	Load kN	Corrected CBR kN	%	Gauge reading	Load kN	Corrected CBR kN	%
0		0	0.00			0	0.00			0	0.00		
0.64		1	0.11			2	0.22			1.5	0.17		
1.27		3	0.33			3	0.33			3	0.33		
1.91		4	0.44			5	0.55			4	0.44		
2.54	13.35	5	0.55	0.55	**4.1**	6	0.66	0.66	**4.9**	7	0.77	0.77	**5.8**
3.81		7.5	0.83			8	0.88			8	0.88		
5.08	20	8.5	0.94	0.94	**4.7**	10	1.10	1.10	**5.5**	12	1.32	1.32	**6.6**
7.62		10	1.10			12	1.32			13.5	1.49		
Soaked CBR, %		4.7				5.5				6.6			
Dry Density, g/cc		1.30				1.44				1.51			
Swell, %		7.33				5.17				4.31			
Density Requirement:			95%		Target Density:			1.43		CBR		5.42	

Table G.4 Soaked CBR value for 10% blended CBA and WHA on soil

No. of Blows per Layer		**10**		**30**		**65**	
CONDITION OF SAMPLE		Before soaking	After soaking	Before soaking	After soaking	Before soaking	After soaking
Wt.of wet sample + mould, g	W1	9200	9800	9625	10220	10530	10900
Wt.of mould, g	W2	5678	5678	5849	5849	6521	6521
Wt.of wet sample, g	W3 = W1 - W2	3522	4122	3776	4371	4009	4379
Volume of mould, cc	V	2123	2123	2123	2123	2123	2123
Wet unit weight, g/cc	γw = W3 / V	1.66	1.94	1.78	2.06	1.89	2.06
Dry unit weight, g/cc	γw /(1+ W8/100)	1.36	1.38	1.44	1.53	1.51	1.53
Moisture Content Determination							
Wt. of wet sample + cont., g	W3	414	438	467	439	410	443
Wt. of dry sample + cont., g	W4	350	327	390	342	340	345
Wt. of water, g	W5 = W3 - W4	64	111	77	97	70	98
Wt. of container, g	W6	64	57	64	61	56	64
Wt. of dry sample, g	W7 = W4 - W6	286	270	326	281	284	281
Moisture Content, %	W8 = W5/W7*100	22.4	41.1	23.6	34.5	24.6	34.9

CBR DATA (4 days Soaked)													
Penet-ration	Std load (kN)	Gauge reading	Load kN	Corrected CBR kN	%	Gauge reading	Load kN	Corrected CBR kN	%	Gauge reading	Load kN	Corrected CBR kN	%
0		0	0.00			0	0.00			0	0.00		
0.64		0.5	0.06			0.5	0.06			1	0.11		
1.27		1	0.11			1	0.11			2	0.22		
1.91		2	0.22			2.5	0.28			3	0.33		
2.54	13.35	2.25	0.25	0.25	**1.9**	3	0.33	0.33	**2.5**	3.5	0.39	0.39	**2.9**
3.81		3.5	0.39			4	0.44			5	0.55		
5.08	20	4	0.44	0.44	**2.2**	5	0.55	0.55	**2.8**	6	0.66	0.66	**3.3**
7.62		6	0.66			7	0.77			8	0.88		
Soaked CBR, %		2.2				2.8				3.3			
Dry Density, g/cc		1.33				1.44				1.51			
Swell, %		6.90				5.17				3.23			
Density Requirement:			95%		Target Density:			1.34		CBR		2.26	

Appendix H. Unconfined compressive strength

Table H.1 Unconfined compressive strength for pit-2

Area of original, mm2	1017.36			
Wet density (g/cm^3)	1.82	Location	Agarfa town	
Dry density (g/cm^3)	1.434	Diameter	36	
OMC (%)	26.7	Height	76	
Sample deformation,ΔL (mm)	Axial strain(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0	0	0	1017.360	0.000
0.142	0.187	0.038	1019.264	37.282
0.253	0.333	0.059	1020.758	57.800
0.372	0.489	0.075	1022.364	73.359
0.511	0.672	0.089	1024.247	86.893
0.634	0.834	0.101	1025.918	98.448
0.763	1.004	0.11	1027.677	107.037
0.894	1.176	0.119	1029.470	115.593
1.02	1.342	0.127	1031.200	123.158
1.161	1.528	0.135	1033.143	130.669
1.324	1.742	0.14	1035.398	135.214
1.447	1.904	0.145	1037.106	139.812
1.593	2.096	0.149	1039.141	143.388
1.732	2.279	0.153	1041.086	146.962
1.89	2.487	0.157	1043.305	150.483
2.037	2.680	0.159	1045.379	152.098
2.256	2.968	0.159	1048.483	151.648
2.67	3.513	0.156	1054.403	147.951

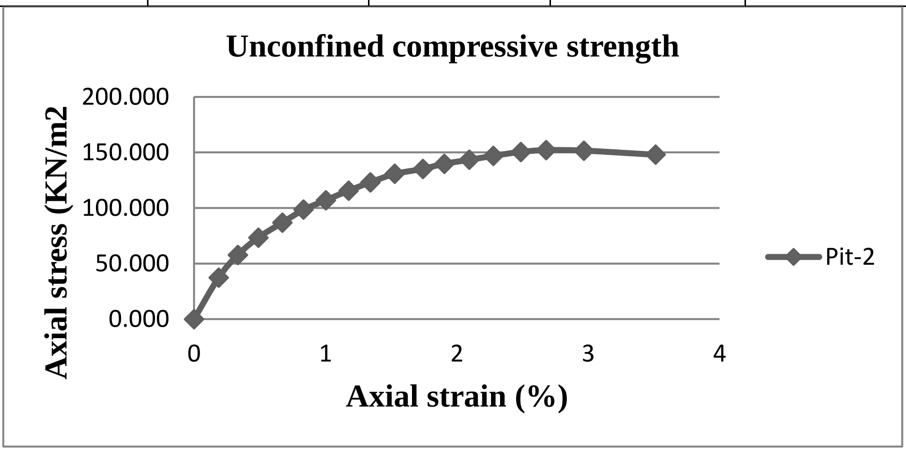


Figure H.1 Unconfined compressive strength for pit-2

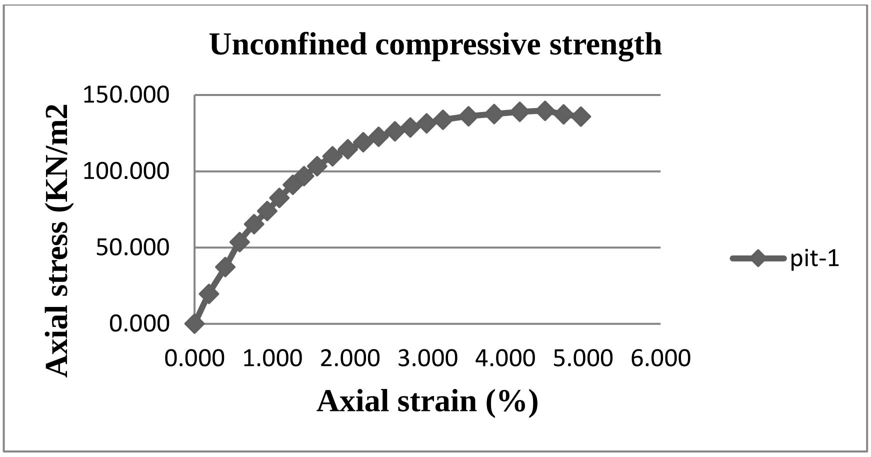


Figure H.2 Unconfined compressive strength for pit-1

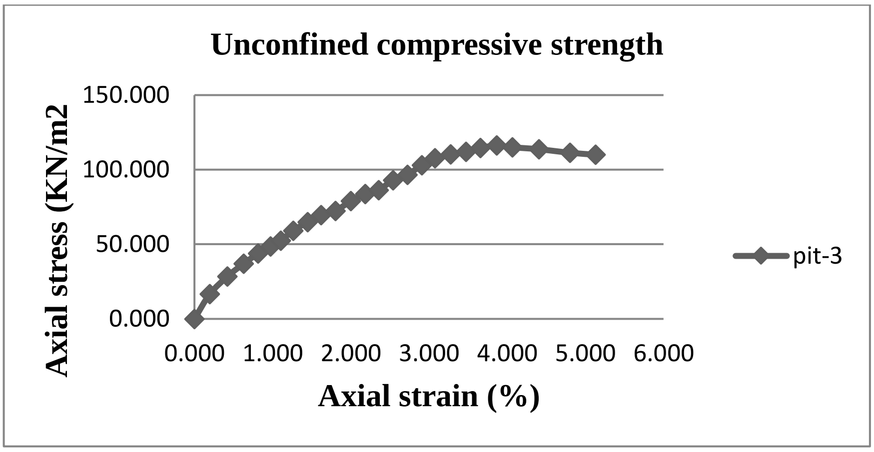


Figure H.3 Unconfined compressive strength for pit-3

Table H.2 Unconfined compressive strength for pit-4

Area of original, mm2	1017.36			
Wet density (g/cm^3)	1.8	Location	Agarfa town	
Dry density (g/cm^3)	1.41	Diameter	36	
OMC (%)	27.86	Height	76	
Sample deformation,ΔL (mm)	Axial strain(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0	0.000	0.000	1017.360	0.000
0.15	0.197	0.015	1019.372	14.715
0.34	0.447	0.027	1021.932	26.421
0.52	0.684	0.036	1024.369	35.144
0.64	0.842	0.043	1026.000	41.910
0.77	1.013	0.048	1027.773	46.703

0.89	1.171	0.053	1029.415	51.486
0.97	1.276	0.059	1030.513	57.253
1.1	1.447	0.064	1032.301	61.997
1.23	1.618	0.068	1034.096	65.758
1.37	1.803	0.073	1036.036	70.461
1.52	2.000	0.079	1038.122	76.099
1.66	2.184	0.083	1040.077	79.802
1.79	2.355	0.087	1041.899	83.501
1.93	2.539	0.092	1043.869	88.134
2.07	2.724	0.095	1045.846	90.836
2.21	2.908	0.099	1047.830	94.481
2.34	3.079	0.101	1049.679	96.220
2.49	3.276	0.104	1051.821	98.876
2.64	3.474	0.107	1053.972	101.521
2.78	3.658	0.109	1055.987	103.221
2.94	3.868	0.111	1058.299	104.885
3.09	4.066	0.112	1060.477	105.613
3.35	4.408	0.113	1064.272	106.176
3.65	4.803	0.114	1068.685	106.673
3.9	5.132	0.112	1072.391	104.440
4.22	5.553	0.111	1077.171	103.048
4.51	5.934	0.109	1081.541	100.782
4.73	6.224	0.106	1084.879	97.707

Table H.3 Unconfined compressive strength for pit-1

Area of original, mm2	1017.36			
Wet density (g/cm^3)	1.89	Location	Agarfa town	
Dry density (g/cm^3)	1.475	Diameter	36	
OMC (%)	27.2	Height	76	
Sample deformation,ΔL (mm)	Axial strain(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0	0.000	0.000	1017.360	0.000
0.14	0.184	0.020	1019.238	19.581
0.3	0.395	0.038	1021.392	37.135
0.44	0.579	0.055	1023.284	53.649
0.58	0.763	0.067	1025.184	65.241
0.71	0.934	0.076	1026.954	73.887

0.83	1.092	0.085	1028.593	82.494
0.96	1.263	0.094	1030.375	91.095
1.07	1.408	0.100	1031.888	96.742
1.2	1.579	0.107	1033.681	103.306
1.35	1.776	0.114	1035.758	109.843
1.5	1.974	0.119	1037.844	114.430
1.65	2.171	0.124	1039.938	118.997
1.8	2.368	0.128	1042.040	122.571
1.96	2.579	0.132	1044.292	126.145
2.11	2.776	0.135	1046.412	128.733
2.27	2.987	0.138	1048.682	131.308
2.43	3.197	0.141	1050.963	133.707
2.68	3.526	0.144	1054.547	136.086
2.93	3.855	0.146	1058.155	137.504
3.18	4.184	0.148	1061.787	138.909
3.43	4.513	0.149	1065.445	139.501
3.61	4.750	0.147	1068.094	137.305
3.78	4.974	0.146	1070.609	135.904

Table H.4 Unconfined compressive strength for pit-3

Area of original, mm2	1017.36			
Wet density (g/cm^3)	1.8	Location	Agarfa town	
Dry density (g/cm^3)	1.41	Diameter	36	
OMC (%)	27.86	Height	76	
Sample deformation,ΔL (mm)	Axial strain(%)	P (KN)	A_c (mm2)	σ (KN/m2)
0	0.000	0.000	1017.360	0.000
0.15	0.197	0.017	1019.372	16.677
0.32	0.421	0.029	1021.662	28.385
0.48	0.632	0.038	1023.826	37.116
0.62	0.816	0.045	1025.728	43.871
0.74	0.974	0.050	1027.363	48.668
0.84	1.105	0.054	1028.730	52.492
0.96	1.263	0.061	1030.375	59.202
1.1	1.447	0.067	1032.301	64.904
1.23	1.618	0.072	1034.096	69.626
1.37	1.803	0.075	1036.036	72.391
1.52	2.000	0.082	1038.122	78.989

1.66	2.184	0.087	1040.077	83.648
1.79	2.355	0.090	1041.899	86.381
1.93	2.539	0.097	1043.869	92.924
2.07	2.724	0.101	1045.846	96.573
2.21	2.908	0.108	1047.830	103.070
2.34	3.079	0.113	1049.679	107.652
2.49	3.276	0.116	1051.821	110.285
2.64	3.474	0.118	1053.972	111.957
2.78	3.658	0.121	1055.987	114.585
2.94	3.868	0.123	1058.299	116.224
3.09	4.066	0.122	1060.477	115.043
3.35	4.408	0.121	1064.272	113.693
3.65	4.803	0.119	1068.685	111.352
3.9	5.132	0.118	1072.391	110.035

Appendix I. Laboratory works

Figure I.1 Determination of Atterberg limits

Figure I.2 Modified proctor compaction a) sample prepared, b) compaction operation

Figure I.4 CBR process a) materials b) penetrating CBR specimen, c) Soaked

Figure I.5 Free swell test process

Figure I.6 Linear shrinkage process a) mixing of sample, b) filling wet sample with mould c) sample prepared for drying

Figure I.7 Unconfined compressive strength a) remolded specimen b) tamping the soil c) UCS loading machine