JIMMA UNIVERSITY

POST GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Investigating the effect of waste ceramic dust and Jute Geotextile fiber mix on strength of expansive subgrade soil. A Study in Jimma City.

A Thesis submitted to the post Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering).

By

Deneke Desula

March, 2024

Jimma, Ethiopia.

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Advisor: Tarekegn Kumala (Ph.D.)

Co-Advisor: Engr. Oluma Gudina (MSc.)

March,2024

Jimma, Ethiopia.

DECLARATION

I, the undersigned, declare that this thesis entitled "**Investigating the effect of waste ceramic dust and jute geotextile fiber mix on strength of expansive subgrade soil.** It is my original work, and has not been presented by any other person for an award of a degree in this or any other University, and all sources of material used for this thesis work have duly acknowledged.

Candidate: **Deneke Desula** ____________________ ___25_/__03__/_2024____

Signature Date

As Master's Research Advisors, we hear by certifying that we have read and evaluated this MSc research prepare under our guidance, by Deneke Desula entitled: "**Investigating the effect of waste ceramic dust and jute geotextile fiber mix on strength of expansive subgrade soil.** This thesis has been submitted for examination with my approval as university supervisor.

Tarekegn Kumala (Ph.D.) ____________________ ___25____/_03_/__2024_____

Principal advisor Signature Date

Engr. Oluma Gudina (MSc.) ____________________ ____25__/___03___/__2024__

Co-Advisor Signature Date

As member of Board of Examiners of the M.Sc. Thesis Open Defense Examination, We certify that we have read, evaluated the thesis prepared by Deneke Desula and examined the candidate. We recommended that the thesis could be accepted as fulfilling the thesis requirement for the Degree of Master of Science in Highway Engineering.

______________________________ ______________________ _____/____/_______

External Examiner Signature Date

______________________________ ______________________ _______/_____/______

Internal Examiner Signature Date

______________________________ ______________________ ________/_____/_____

Chairperson Signature Date

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ABSTRACT

Expansive clay soil is one of the most abundant soil types in Ethiopia; it primarily causes significant damage to structures like buildings, pavements, and bridges. Some land areas in Jimma City covered with expansive soil. The problematic nature of such soil can improved by employing chemical stabilization techniques such as lime, cement, and bituminous stabilization. However, these methods are expensive to develop and apply in countries such as Ethiopia. The reason enforces to use locally available, low-cost, and environmental friendly materials to get stable subgrade soil. Hence, the objective of this study was to investigate the mixed effect of varying percentages (10% ,15%and20%) of ceramic dust waste and (0.5% to2.5%) of jute geotextile fiber every 0.5%increament by weight of dry soil on the strength of expansive subgrade soil and stabilization conducted as matrix combination. The sampling technique used for this research was a purposive sampling, which is non– probability method. Three sample areas selected and samples were taken, and then their properties were investigated. The Laboratory tests conducted were Moisture content, particle size analysis, Atterberg limits, free swell, specific gravity, compaction, California Bearing Ratio and Unconfined compression strength tests. Shrinkage limit from Liquid Limit vs Plasticity index chart, soil expansiveness by formula and soils swelling potential conducted for untreated while strength tests conducted only for treated soil. The result from laboratory tests have shown that the soil sample belonged to the A-7-5 class according to American Association of State Highway and Transportation officials and CH (inorganic clay of high plasticity) according to the Unified Soil Classification System and the soil rated as poor (unsuitable) subgrade materials.

For stabilized soil by different combinations, the one, in which the maximum improvement was achieved, at a combination of 1.5%Jute Geotextile Fiber with 20%Ceramic Dust Waste Powder. The results revealed that Maximum dry density, California Bearing Ratio, and Unconfined compression strength were increased from 1.3g/cc to 1.57g/cc,1.03% to 8.4% and74.80KPa to 538.88Kpa, respectively for immediate cure while 660.65KPa unconfined compressive strength for seven day curing. Optimum moisture content and Swell decreased from 30.05% to 22.64% and 10.07% to 1.04%, respectively. For enhancing the strength of expansive subgrade soil, a suitable combination of 20% of waste ceramic dust and 1.5% of jute geotextile fiber by weight of dry soil is optimum and recommended.

Key word: *Expansive subgrade soil: Ceramic dust waste: Jute Geotextile fiber: Soil stabilization: California Bearing Ratio.*

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
CDWP	Ceramic Dust Waste Powder
CBR	California Bearing Ratio
CH	Inorganic clay with high plasticity
ESS	Expansive Subgrade Soil
ERA	Ethiopia Road Authority
FS	Free swell
JGTF	Jute Geotextile Fiber
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
SS1	Soil Sample One
SS2	Soil Sample Two
SS3	Soil sample three
UCS	Unconfined Compression strength
USCS	Unified soil classification system

CHAPTER ONE

INTRODUCTION

1.1 Background

Construction activities around the world are significantly impacted by the presence of expansive clay soil, like the southwestern United States, South America, Canada, Africa, Australia, Europe, India, China, and the Middle East [1]. Expansive soil is a worldwide problem and common in our country, Ethiopia which encompasses nearly 40% of the total area of the country and is observed in areas such as central Ethiopia, the Southern, South-west, Southeast and northeast part of Ethiopia. However, Addis Ababa city area and western region of Ethiopia have densely expansive soil coverage[2,3].

Any structure layover on the ground vertical or horizontal needs safe and steady soil. To accomplish this safety and steadiness requirements the Engineering properties of the soil underneath the structure or on the structure must recognized. Construction of Pavement on expansive/sweeping soil is hazardous due to its low subgrade strength [4]. Expansive soils like black cotton soil always create problems with the subgrade of flexible asphalt pavement. The problems are swelling during the wet season, shrinkage during the dry season, and unequal settlement and this leads the pavement failure and distresses [4,5].

Stabilization is a one of subgrade treatment method that aims to create structural stability in order to increase long-term performance. Stabilization with stabilizing agent is the most cost effective method [6,7]. The most frequent enhancements brought about by stabilization include improved soil gradation, decreased plasticity index or swelling potential, increased durability, increased shear strength of soil, and therefore improved subgrade load-bearing capacity to support applying loads on pavements and foundations [8].

It has been a major concern of many authors to use waste materials in geotechnical applications. Meanwhile, bitumen, cement, lime or another chemicals used as soil stabilizer are costly and to overcome it mixing those material with another waste material additives or waste materials only as stabilizer in order to reduce the amount of the costly stabilizers [9]. The amount of waste especially that generated by construction is increasing every year. Utilizing environmental wastes to stabilize soft soil might help to reduce waste production and protect natural resources [9]. Use of local materials can considerably minimize down the construction and maintenance cost [10].

Therefore, this study focused on how to use locally available waste materials; ceramic dust waste and geotextile jute fiber to improve strength of weak subgrade soil. The stabilization using geotextile jute fiber and waste ceramic dust mix was an economical method since they are wastes, relatively cheap and easily available or from different construction sites abandoned as waste. Ceramic dust is produced because of the ceramic stoneware, ceramic bricks, and roofing tile industries.

In ceramic factories around 30% of daily production goes as waste material, which is not recycled, and the disposal of which creates soil, water, and air pollution [11,12] Similarly, Jute takes nearly 3 months, to grow to a height of 12–15 ft, then cut during the season and bundles were maintained in water for ‘‘retting" procedure, in which the outer plant is "individualized" to create a Fiber and the inside stem and outer stem are separated. Then it separated and washed to remove dust from the plant. The fiber after drying and transported to jute mills to be transformed into jute yarn and hessian. There is around 5.6 thousand tons of jute production per year in Ethiopia [13]. Jute Geo-textiles (JGT) are products of jute[14] and from which jute sack or jute bag produce for its purpose and after torn, it used for this study.

In Jimma City there were problems due to the effect of expansive soil [15]. This a problematic soil found In Jimma City needs treatment [5]. To overcome these problems understanding properties of soil and evaluating suitable stabilization methods are very important [16]. The main objective of this research was to investigate the effect of waste ceramic dust and jute geotextile fiber mix to improve the strength of expansive subgrade soil. Its performances were evaluated with laboratory results and the laboratory results Moreover, the statistical analysis from this study were good news to prevent pavements failure due to weak subgrade soil.

1.2 Statements of a problem

Expansive soils are weak, and a soil with poor shear strength and low bearing capacity. When constructing facilities or infrastructure on such soils, engineers have various difficulties [10].The problems of such expansive soil is susceptible to differential settlements, poor shear strength, high compressibility, swell during wet season and shrinks during dry season depending upon the presence of moisture on it. The soil changes in volume as a reason of this behavior and its result increase construction and maintenance cost, less performance of flexible pavement like surface distress (surface deformations, cracking), and failure of the roads before their expected design life [2,13]. Due to their inadequate strength and low bearing capacity, such weak subgrade soils are unable to withstand the stresses placed on them by the structure during road construction or during its service life[17]

The majority areas of Ethiopia was covered in expansive clay soils, and Jimma City is no exception. Distresses in the pavement, such as longitudinal cracking and failure and traverse cracking, High and low severity alligator cracking, and other types are present in the research area, and this was caused by expansive soils that weaken the subgrade layer by existing in it [4, 5].

Researchers conducted using cement or lime stabilization[5,15] within the research area to enhance the strength of expansive subgrade soil but the Application of lime as a stabilizer agent is not helpful in some cases where soil bearing capacity, density, and hydraulic conductivity are very low[16]. Sulfate attack, carbonation, and environmental influences are disadvantages of using lime/cement as soil stabilization additional to the day-to-day cost increment of cement [16].

According to [18], the interfacial shear resistance of the fiber-soil interface, which is influenced by friction, bonding force, matrix suction, and interface morphologies, strongly controls the engineering behavior of fiber-reinforced soil, particularly the strength characteristics. Because of the application of load, the soil particles at the soil-fiber interface may have the potential to rotate, and the fiber in tension may experience plastic deformation at the sharp corners or edges of the particles. The existence of additives in soil, the rotation of particles at the soil-fibre interface is restricted by increased bonding force due to cementation [18] reduces swelling potential, shrinkage, and crack width. For this reason, Investigation were done to increase the subgrade soil's strength, and the experimental result carried out by investigating effect of ceramic dust waste and Geotextile Jute Fiber mix on strength of expansive subgrade soil. Therefore, stabilization of expansive subgrade soils with a mix of ceramic dust waste and geotextile jute fiber combination is a better way; because they are locally available, environmental friendly and cheaper or economically feasible. As well as it is good for environmental clearance other than dropping waste to anywhere.

1.3 Research questions

1. What are the engineering properties of expansive subgrade soil of selected area?
2. What are the stabilizer percentages perform optimum to strengthen expansive subgrade soil?
3. What are the properties of expansive subgrade soil when stabilized with ceramic dust waste and Jute Geotextile fiber mix?
4. What are the effects of stabilizers with respect to the standard specification?

1.4 Objectives of the study

1.4.1 General objectives

The general objective was to investigate the effect of waste ceramic dust and jute geotextile fiber mix on strength of subgrade soil.

1.4.2 Specific Objectives

- To determine the engineering properties of the selected expansive subgrade soils
- To determine the optimum stabilizer percentages that enhance the strength of expansive subgrade soil.
- To investigate the properties of expansive subgrade soil when stabilized with ceramic dust waste and Jute Geotextile fiber mix.
- To compare the effects of the stabilizers with standard specification.

1.5 Significance of the study

For the design of the highway's subgrade, and maybe for others, this study will be highly useful. When the result can be utilized by the Jimma City administration, contractors, consultants to construct a road that has a strong subgrade layer with good pavement condition, it can accessible for service life to users, reduce the design thickness of subgrade to be more economic, conserves aggregate materials and also to reduce total cost of construction, maintenance and increase service life of the pavement structures for design period.

On the other hand, this research can be a reference for Jimma Institute of Technology students special in highway and geotechnical streams those who want to carry out a further study with respect to stabilizing with this material type and helps to show a new combination of locally available material as an alternate way of stabilizer.

1.6. Scope and limitation of the study

The study covered the stabilization of selected expansive soil in Jimma City. To perform this study, three representative expansive soil samples along the road segment from different locations in Jimma City were taken. The collected samples were disturbed or undisturbed and taken depth after 1.5m from natural ground level to exclude organic matter. From three soil samples, one the weakest soil sample was selected based on properties studied, moreover by its CBR value and treated. This study supported by different types of literature and a series of laboratory experiments. The scope of the study was to investigate the combined effect of waste ceramic dust with geotextile jute fiber on strength properties of expansive subgrade soil depending on laboratory tests to construct flexible pavement. Also the study conducted by taking limited parameters like; natural moisture content, specific gravity, particle size analysis, Atterberg limit; free swell, modified Proctor compaction, CBR and CBR swell, and UCS tests and out of laboratory, shrinkage limit from (LL vs PI) chart, classification of soil expansiveness from formula and activity to determine swell potential were conducted to determine engineering properties of natural soils and strength properties (modified compaction, CBR and UCS in both immediate and seven day cured) was studied for treated soil to achieve the objective of the research. Finally, a recommendation was drawn and forwarded. However, the finding of the research was limited to selected weak soil in Jimma City.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

Clayey/expansive soil expands when it is wetted and shrink when dried. These soils are called as expansive soil or swelling soil. These soils are difficult to work with in construction because of their swell able character. Due to the existence of fine clay particles that expand when in contact with water, the soil frequently swells and shrinks excessively with changes in water content, causing differential settlement of structures [19].

The presence of clay causes expansive soils to expand. It contains a significant quantity of clay minerals and can change in volume with varying moisture levels. The structures can suffer damage on a regular basis as the seasons change. Uneven or localized movement is a sign of significant problems. When it rains, expansive soils can expand by 10% or more. The soils contract back to their former size when they dry out. As a result, the foundation and roadbeds experience cracks because of the entire movement against the volume change[20]

In construction of any structures, engineering properties of soil are the important factor to be considered. As soils has also a relation with water and thus the stabilization of soil can increase the rate of tolerance of water into the soil thus making it quite ideal for engineering purpose[21]

2.2 Origin of Expansive Soils

Either volcanic rocks or sedimentary rocks can be the parent materials of expansive soils. Basalt, dolerite, sills, dykes, and gabbro's make up the fundamental igneous rock. Montmorillonite is a component of the sedimentary rock that physically degrades to produce expansive soils[1]. The fine grained soil eventually accumulating in the ocean basin. The second case in which Montmorillonite is formed could be volcanic eruption sending up clouds of ash fall on the plains and sees and through process thus ashes were altered to Montmorillonite [1].

2.3 Mineralogy of clay and expansive Soils

The presence of clay minerals contributes to the soil's expansiveness. 0.002 mm or smaller is the size of clay particles. However, according to [1], the main characteristic of fine-grained soils is their mineralogical composition, which is more significant than the grain size alone in determining clay minerals. By weathering or volcanic eruption, parent rock is transformed into crystalline hydrous alumino-silicates known as clay minerals. The fundamental components of clay minerals are the silica tetrahedron and the alumina octahedron and combine into tetrahedral and octahedral sheets to form the various types of clays. Clays can be divided into three general groups on the basis of their crystalline arrangement. They are; Kaolinite group, Montmorillonite group (also called the smectite group) and Illite group. Montmorillonite is the clay mineral which causes most of the expansive soil problems. The name Montmorillonite is uses currently as a group name for all clay minerals with high expansiveness potential [1].

2.4. Structure of clay minerals

Some clay minerals do contain material that is non-crystalline; clay minerals are generally crystalline in nature. In the emergence of clay mineral structures, two essential building blocks are involved. The two are the octahedral unit and the tetrahedral unit.

The tetrahedral unit consists of four oxygen atoms (or hydroxyls, if needed to balance the structure) placed at the apices of a tetrahedron enclosing a silicon atom which combines together to form a shell-like structure with all the tips pointing in the same direction. The oxygen at the bases of all the units lies in a common plane. Each of the oxygen ions at the base is common to two units [*23,*24,25].

A. Kaolinite

The mineral in the kaolin group that is most prevalent one. The kaolinite layer's structure is determined by how the gibbsite and silica sheet building blocks are organized. The silica tetrahedron tips and one of the layers of the alumina octahedral sheet are merged in units to form the structure, which is made up of a single tetrahedral sheet and a single octahedral sheet of alumina. Since hydrogen bonds are comparatively strong, the kaolinite crystals consist of many sheet stacking that are difficult to dislodge. The mineral is, therefore, stable and water cannot enter easily between the sheets to expand the unit cells. Kaolinite is a typical two layered mineral having a tetrahedral and an octahedral sheet joined to from 1 to 1-layer structure held by a relatively strong hydrogen bond. When in contact with water, kaolinite does not absorb water more and does not afterward expand more [24,25]

B. **Montmorillonite**

The mineral montmorillonite is the most prevalent member of the montmorillonite family. This mineral's structural arrangement consists of two silica tetrahedral sheets with an alumina octahedral sheet in the middle [22]

In stacking of these combined units one above the other, the oxygen layers of each unit are adjacent to the oxygen of neighboring units, resulting in very weak bonding (forced bonding of the Van der walls) and excellent separation between them. Water can penetrate between the plates, causing them to expand significantly and thus the structures can break into pieces. Soils containing significant amounts of the mineral montmorillonite will have high swelling and shrinkage properties. Montmorillonite is a three-layer mineral having a single octahedral sheet sandwiched between two tetrahedral sheets [22]

C. Illite

The fundamental structural component of illit is comparable to that of montmorillonite except that some of the silicons are always replaced by aluminum atoms and the resultant charge deficiency is balanced by potassium ions. The potassium ions occur between unit layers. They form non-exchangeable K+ ion bonds with them that are stronger than the water bond of montmorillonite but weaker than hydrogen bonds. Illite, in comparison to montmorillonite, does not swell as much when exposed to water [22].

2.5 Expansive Soil Distribution in the World

In many parts of the world, expansive soils are common, especially in dry and semi-arid areas and in areas where rainy weather returns after extended droughts. Expansive soil, which is widespread throughout the world, has a significant negative impact on construction activities. Examples include Burma, China, India, Iran, Israel, Japan, and Oman in Asia; Argentina, Canada, Cuba, Mexico, Trinidad, the United States, and Venezuela in the Americas; Cyprus, Germany, Greece, Norway, Romania, Spain, Sweden, Turkey, and UK in Europe; Australia; Ethiopia, Ghana, Kenya, Morocco, South Africa, and Zimbabwe in Africa [24]

2.6 Expansive soils Distribution in the Ethiopia

Expansive soil is well known to be widely distributed in Ethiopia, where it accounts for almost 40% of the country's total land and is frequently seen in places like the country's central region.

The Southern, South-west, southeast, and northeast part has a clay soil distribution. However, the Addis Ababa city area and western Ethiopia has densely clay soil coverage. Some of the places in the central region of Ethiopia along the main routes, such as Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis-Gohatsion, and Addis-Modjo, are covered in expansive soils. Areas like some part of Mekele, Gondor, Bahirdar, Debreberihan and Gambela are also known to be partly covered by expansive soils [2].

2.7 Identification of Expansive Soils

2.7.1 **Field Identification**

It is easy to recognize expansive soils during fieldwork either dry or wet seasons. Their color varies from dark grey to black. During dry seasons, shrinkage cracks are visible on the ground surface with the maximum width of these cracks reaching up to 20 mm or more and they travel deep into the ground. A lump of dry black cotton soil requires a hammer to break. A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail. During rainy seasons, these soils become very sticky and very difficult to traverse. Appearance of cracking in the nearby structures is also indicative [25,27]

2.7.2 Laboratory Identification

There are a number of laboratory tests that are useful in identifying expansive soils. Generally, these can be categorized as mineralogical identification, direct and indirect methods[1]

A. Mineralogical Identification

It is the nature of clay mineral in the soil, which is responsible for the swelling or shrinkage, and therefore, the methods that directly or indirectly enable to identify the types of minerals can be considered to be the most desirable ones [1,2]. For this purpose, tests like X-Ray diffraction, electron microscope, differential thermal analysis, dye absorption, determination of silica-sesquioxide ratio and Base Exchange capacity are usually used to study the amount and type of clay minerals, by which the swelling characteristics of clay can be identified. These methods are mostly used for academic or research purposes. They are time consuming, require expensive test equipment, the outcomes are interpreted by specially trained technicians. As a result, for ordinary engineering works these methods are generally not often used.

B. Direct Methods

Chen (1988) recommended the direct method of expansion potential measurement to recognize expansive soils since the test is simple to perform and does not require any expensive laboratory equipment. According to him X-ray diffraction is principally used in determining the proportion of the various minerals present in colloidal clay and if supported by differential thermal analysis and skimming electron microscopic examination it provides good results.

C. Indirect Methods

This method is used to investigate the expanding potential of fine- grained soil and by examining other parameters, which indirectly give information about the soil property. And hence estimate their swell potential (Sridharn &Praksh, 2016) ,the liquid limit, plastic limit ,shrinkage limit, plasticity index, free swell are useful for determining the swelling characteristics of most of the clays and prepared a chart to support the identification [25] .And other strength and bearing capacity determining parameters.

I. Particle size associated properties

Soil swelling is controlled by the relative particle size distribution of fine-grained soils. The swell potential of fine-grained soils has been predicted by numerous studies using criteria based on the percentage of clay size fraction (i.e., particles smaller than 0.002 mm in size) or colloid content (i.e., the presence of particles smaller than 0.001 mm),presented in table 2.1.

Table 2.1; Grain size and swell potential relation [1, 71]

Degree of expansivity/swell potential	Percent clay size fraction	Colloid content
Low	<30	<15
medium	30-60	13-23
High	60-95	20-31
Very high	>95	>28

II .Liquid Limit related properties

The relationship between moisture content and consistency defined by the atterberg limits recognize based on the liquid limit of fine-grained soils, the probable degree of soil swelling.

Table 2.2 : Liquid limit and swell potential relation [1,28]

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

III Plasticity Index related properties

The plasticity index (PI) is the most frequently utilized factor for estimating soil's potential for shrinking and swelling. The potential for soil swelling will increase with a greater plasticity index and more plastic soil presented in table 2.3

Table 2.3: Expansive soil classification based on their plasticity index (BRGM, 2009)

Swelling potential	Plasticity index (%)
Low	<12
Medium	12-25
High	25-40
Very high	>40

IV. Shrinkage and degree of expansion

Both linear shrinkage and shrinkage limits are considered to be related to the swell potential.

Table2.4: Relationship between linear shrinkage, shrinkage limit, and swelling potential [27]

Shrinkage limit (%)	Linear shrinkage (%)	Degree of expansion
< 10	>8	Critical
10-12	5-8	Marginal
> 12	0-5	Non-critical

The term shrinkage limit, expressed as a water content in percent is often taken to mean the volume of water needed to completely fill the gaps in a particular cohesive soil after drying (normally in an oven). In earthworks incorporating cohesive soils, the idea of shrinkage limit can be utilized to assess the possibilities of both shrinkage and crack development. It has two Standard Test Methods were adapted, Mercury Method (ASTM D-427) and Wax Method (ASTM D-4943). Mercury is a considered harmful chemical, hence Wax Method is offered as an alternative to Test Method D

427, which is used to determine the shrinkage limit and other shrinkage factors of soils using mercury. Since this test method has been performed by only a few organizations to date and the problem was the same for this study. by Wax Method shrinkage limit can be determined from laboratory test result of liquid limit and plasticity indexes by plotting value on chart as according to [28] .this study also determined shrinkage limit by using wax method and the main aim of determining shrinkage limit by this method is to determine the degree of soil expansiveness by using formula according to ERA (2002). Used methods or steps:-

- At the shrinkage limit (border water content), the sample transitions from a semi-solid to a solid state. The sample starts to dry up after this point.
- The figure 2.1 depicts the phenomena of volume change.
- Plot point let A, using the values of LL and PI determined experimentally, and extend it to meet O. The shrinkage limit is determined by the line AO's intersection with the X-axis.

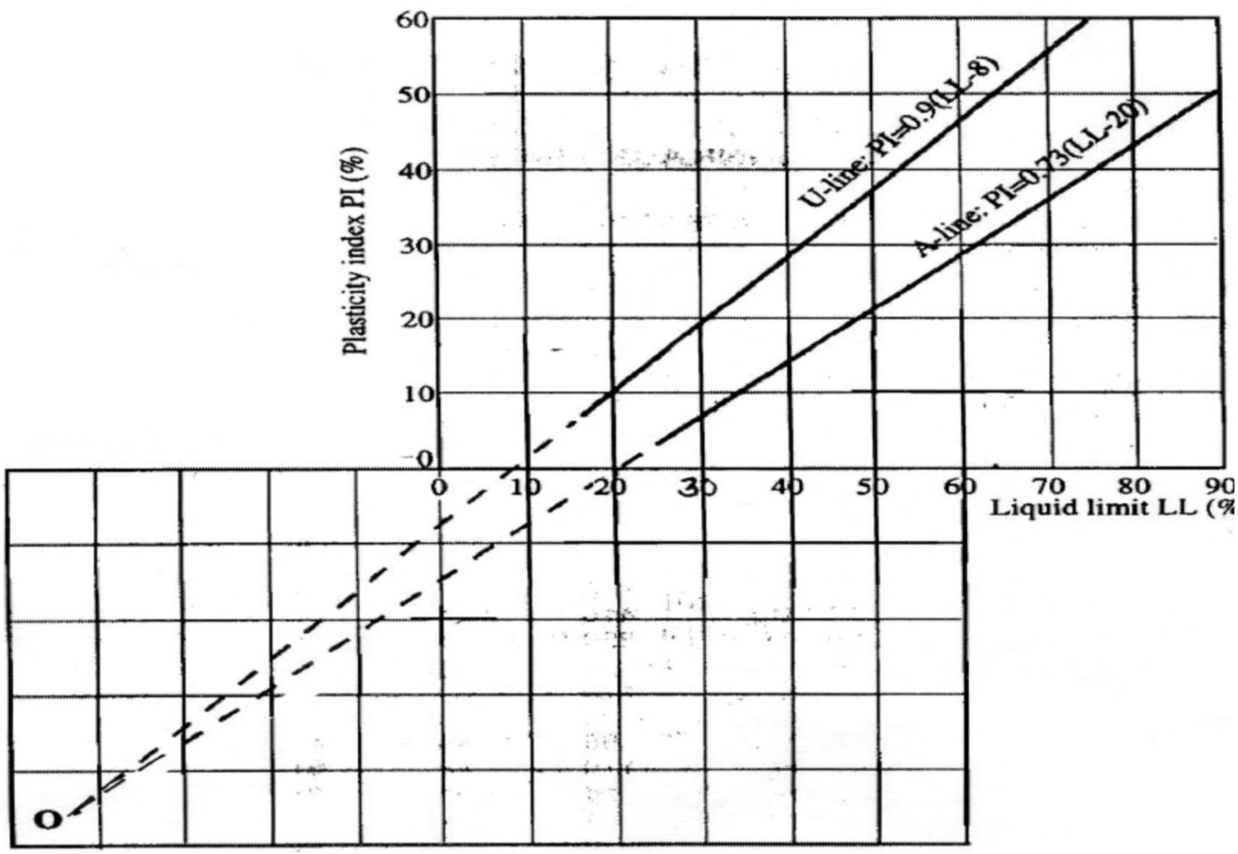


Figure 2.1: determination Shrinkage limit from LL and PI chart

V. Free Swell and degree of expansion

Table 2.5: Swell potential classification an degree of severity [29]

Degree of expansion	Free Swell index (%)
Low	Less than 20
Moderate	20 to 35
High	35 to 50
Very high	Greater than 50

VI. Activity

Skempton (1953) demonstrated that activity may be related to the clay's mineralogy and geotechnical history by defining activity (A) as the ratio of plasticity index (PI) to 0.002mm clay fraction (CF). He divided clay into three categories: "normal," "inactive," and "active" clays. Activity =PI/<0.002%clay (2.1)

Table 2.6: soil swell potential evaluation from activity [1]

Activity	Nature of soil	Swell potential
<0.75	Inactive clay	Low
0.75-1.25	Normal clay	High
>1.25	Active clay	Very High

VII .Expansive Soils Classification by formula, ERA,(2002)

According to ERA,(2002),Extended investigations are advisable if :

- ✓ The results of field reconnaissance indicate expansive soils, and
- ✓ PIw>20%

Where PIw =Plasticity index tested on fraction <0.425mm, weighted for the sample's actual content of particles <0.425mm as follows;

PIw =PI*(%passing 0.425mm)/100 (2.2)

To estimate expansiveness (Єex)= 2.4 wp – 3.9 ws +32.5 (2.3)

Where: Wp= PI *(%passing 0.425mm)/100 (2.4)

Ws = Shrinkage Limit *(%passing 0.425mm)/100 (2.5)

Table 2.7: Classification of expansive soils ERA,(2002)

Expansiveness ,Єex	Classification
<20	Low
20-50	Medium
>50	High

VIII Natural Moisture (Water) Content

Table 2.8: Typical values of water content in a saturated state[30,31]

Soil or sample types	Natural Water Content in a Saturated State (%)
Loose uniform sand	25-30
Dense uniform sand	12-16
Loose angular-grained silty sand	25
Dense angular-grained silty sand	15
Stiff clay	20
Soft clay or clay	30-50
Soft organic clay	80-130
Glacial till	10

XI. Specific Gravity Test

Table 2.9: specific gravity for various soils[32]

Soil types	Range of G_s
Sand	2.63-2.67
Silt	2.65-2.7
Clay and Silty Clay	2.67-2.9
Organic Soil	Less than 2

X. Compaction (Moisture-Density) Test

Compaction is the process of densifying soil by utilizing mechanical force to reduce air voids. It is used to build engineering structures such as earth dams, airport runways, and highway embankments; loose soils must be compacted to improve their strength by increasing their unit weight. The dry unit weight is used to determine the degree of compaction. To determine the correlation between soil moisture content and dry density for a given compaction effort, laboratory tests are carried out. [33]. The quantity of mechanical energy that is exerted on the soil mass is known as the comp active effort. In the field, soil is compacted using a variety of techniques, some of which include tamping, kneading, vibrating, and static load compaction. The test is also known as the Proctor test since it will be performed in this laboratory utilizing the tamping or impact compaction technique and the R. R. Proctor-developed tools and procedures in 1933. This experiment is carried out to ascertain how the soil's dry density and moisture content relate to a given compaction effort[30].

Preparation of soil sample for proctor's compaction test was conducted as per ASTM standards. The optimum water content is the water content that results in the greatest density for a specified compactive effort. Purpose of Compaction are it can increases shear strength of soils, increases the bearing capacity of foundations or embankment, decreases the undesirable settlement of structures reduction in hydraulic conductivity, increasing the stability of slopes on embankments and others.

Table 2.10.: Alternative proctor test methods *[30]*

	Standard Proctor (ASTM 698)			Modified Proctor (ASTM 1557)		
	Method A	Method B	Method C	Method A	Method B	Method C
Material	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve	≤ 20% Retained on No.4 Sieve	>20% Retained on No.4 ≤ 20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve
soil passing	Sieve No.4	3/8" Sieve	¾" Sieve	Sieve No.4	3/8" Sieve	¾" Sieve
Mold	4" DIA	4" DIA	6" DIA	4" DIA	4" DIA	6" DIA

No.of Layers	3	3	3	5	5	5
No. of blows	25/layer	25/layer	56/layer	25/layer	25/layer	56/layer

XI California Bearing Ratio Test

In road constructions, the most widely used laboratory test for evaluating the potential strength of subgrade, sub base, and base course materials is the California Bearing Ratio test.

The California Bearing Ratio (CBR) is a very commonly used laboratory test for predicting the strength of a subgrade layer. This test method was first introduced into the California State Highway Department in the 1920s. CBR test was standardized by American Society Of Testing Materials ASTM D-1883 for laboratory prepared samples and by American Association of State Highway and Transportation officials AASHTO T-193; and by British Standard as BS 1377 part 4 [2,32] Soaked CBR test is used to simulate the worst scenario that would likely happen to the sample due to wet weathers. Hence, before performing the tests, soaking the remolded samples for four days (96 hours) is mandatory.

Subgrade strength for pavement structural design is evaluated in terms of California Bearing Ratio (CBR). Strength indicators other than CBR may be used provided they are adequately correlated on CBR values. Design CBR of the subgrade class under study below 3% is S1 according to ERA pavement design manual and cannot preferable for road construction and support structures constructed on it unless special treatment is made.

The CBR swell of the soil is measured by placing the tripod with the dial indicator on the top of soaked CBR mold. The compacted soil samples of the CBR mold are soaked for 96 hours in a water bath to get the CBR swell of the soil. The initial dial reading of the dial indicator on the soaked CBR of mold is taken immediately after soaking the sample.

Table 2.11: ERA sub grade strength class[35]

Subgrade Class	CBR Range in %	Subgrade quality
S1	<3	Very poor subgrade
S2	3,4	Poor
S3	5,6,7	Poor to fair

S4	8-15	fair
S5	15-30	Fair to Good
S6	>30	Good to Excellent

Xii Unconfined Compression Strength Test

The unconfined compression strength of soil is a load per unit area at which an unconfined cylindrical specimen of soil will fail in simple compression test. It is used to calculate the unconsolidated undrained shear strength of the soil under unconfined conditions. To measure the resistance of the soil by compressibility or shearing deformation, UCS test gives the shear strength of the soil that is useful parameters for computing safe bearing capacity of soil as well as strength of soil. To Determine the bearing capacity of foundations, dams, and other structures using the undrained shear strength [1]

Unconfined Compressive Strength (UCS) of an undisturbed soil specimen is determined, and the test is a special case of a triaxial compression test, notably for cohesive soils alone that can stand alone without confinement. A simple compression axial load is applied quickly in the soil specimen in which the all-round pressure (confined pressure) is equal to zero (i.e., $\sigma 3 = 0$). The test is an undrained test and is based on the assumption that there is no moisture loss during the test. The soil specimen is sheared by applying an axial load of major principal stress ($\sigma 1$) and failure is reached. The Deviator stress ($\sigma 1$-$\sigma 3$) is equal to the major principal stress and minor principal stress ($\sigma 3$) is equal to zero. Gradually increasing axial stress (qu) causes the specimen to break down (Arora KR. Soil Mechanics.2004).

Table 2.12: Quality of subgrade based on UCS(qu) Value [36,37,38]

Consistency description	qu (kn/m^2)	Criteria
Very Soft	0-25	Squishes between finger when squeezed
Soft	25-50	Very easily deformed by squeezing
Medium stiff (firm)	50-100	Thumb makes impression to deform
Stiff	100-200	Hard to deform by hand squeezing (cannot be indented by thumb pressure)
Very stiff	200-400	Very hard to deform by hand (Can be indented by thumb nail)
Hard	>400	Nearly impossible to deform by hand or difficult to indent by thumb nail

Xiii Sensitivity

This determines the strength loss that results from soil disturbance or remolding. The unconfined compression strength is much lower for many naturally deposited clay soils when the soils are assessed following remolding without altering the moisture content. Sensitivity describes this quality of clay soil. Sensitivity is defined as the ratio of the undisturbed strength to the remodeled strength. Table 2.13.

Table 2.13: Sensitivity of soil[39]

Descriptive Term	**Sensitivity**:= $\frac{\text{undisturbed}}{\text{remolded}}$
Insensitive, normal	< 2
Moderately sensitive	2 – 4
Sensitive	4 – 8
Extra sensitive	8 – 16
Quick	> 16

2.8 Engineering Classification of expansive soil

2.8.1 Textural soil classification system

Its classification is based on mechanical examination that quantifies the proportions of sand, silt, and clay-size components in a given sample. This classification method does not reveal any properties of the soil other than the distribution of grain sizes. It is extensively utilized by those who work in agriculture due to its simplicity. According to Arora KR. Soil Mechanics and Purushothama Raj P. Soil Mechanics and Foundation Engineering, the textural categorization obtained from the chart may not always exactly reflect the physical characteristics of the soil.

2.8.2 AASHTO classification system

Hogentogler and Terzaghi first created this method in 1929 under the name Public Roads Classification method. There are then several revisions. The present AASHTO (1978) system is primarily based on the version in 1945. Comprised of 8 Major Groups: A1 through A7 (with subgroups determined from Group Index (GI)) and A8 (Organics) [34]

Table 2.14: AASHTO Classifications System [34]

A1 to A3	A4 to A7
Granular Materials (Less than or equal to 35% passes #200 Sieve) #Using LL and PI separates silty materials from clayey materials (only for A2 group)	Silt-Clay Materials (≥ 36% passes #200 Sieve) #Using LL and PI separates silty materials from clayey materials

Generally**,** A-1 materials are well-graded stone fragments gravel and sands, A-2 materials are silty or clayey gravel and sand; A-3 soils are clean poorly graded sands; A4 & A5 are generally silts, A6 & A7 are generally clays.

The system is based on the Particle-size distribution, Liquid Limit and Plasticity Index. A Group Index is introduced to further differentiate soils containing appreciable fine-grained materials. If plasticity index is equal to or less than (liquid limit -30), the soil is A7-5 (i.e. PL>30%) If plasticity index is greater than (liquid limit -30), the soil is A7-6 (i.e. PL<30%)

2.8.3 Unified Soil Classification System (USCS)

The United States Bureau of Reclamation and the U.S. Army Corps of Engineers later updated and implemented A. Casagrande's 1942 proposal for the Unified Soil Classification System. Currently, almost all geotechnical work uses the system. The Unified Soil Classification System is based on the recognition of the type and predominance of the constituents considering grain-size, gradation, plasticity and compressibility. Divided into three broad categories: the first one is Coarse Grained Soils-Gravels (G) and Sands (S) < 50% passing through #200 sieve (i.e. >50% retained on #200 sieve). The second type, Fine Grained Soils-Silts (M) and Clays (C) ≥ 50% passing through #200 sieve and the third soil type is highly organic (peaty) soils

In the field, identification is accomplished by visual examination for the coarse-grained soils and a few simple hand tests for the fine-grained soils. In the laboratory, the grain-size curve and the Atterberg limits can be used. The peaty soils are readily identified by color, odor, spongy feel and fibrous texture. In the Unified System, the following symbols are used for identification:

Table 2.15: USCS symbol and their description

Symbol	G	S	M	C	O	Pt	H	L	W	P
Descriptions	Gravel	Sand	Silt	Clay	Organic silts and clay	Peat and organic soils	High plasticity	Low plasticity	Well graded	Poor graded

Low plasticity LL<35%; Intermediate plasticity LL=35-50%; High plasticity LL= 50-70% (Fat for Clay, LL>70%;Soil is very high plasticity and Elastic for Silt, e.g. high-quality pottery clay).(Arora KR, 2004.& Purushothama Raj P,2005 Soil Mechanics)

Secondary descriptions of (coarse grained) are: - **M** = Silty > 12% fines, PI<4 or plots below “A” Line; **C** = Clayey> 12% fines, PI>7 and plots on or above “A” line; **P** = Poorly Graded < 5% fines, Cu < 6 and/or 1>Cc>3; **W** = Well Graded: < 5% fines, Cu $\geq$ 6 and 1$\leq$ C_c $\leq$ 3

2.9 Design and construction considerations on the expansive grounds

To mitigate or overcome the problem of expansive soils, the design engineer may consider the following four main approaches; (ERA, 2002).

- ✓ Avoid expansive soils areas by realignment
- ✓ Excavate the weak soils and replace them with suitable material (backfilling)
- ✓ Treat the expansive clays (with lime or materials)
- ✓ Minimize moisture changes and potential swelling in the expansive soils.
- ✓ Stabilize soil by using different additives

2.10. Soil stabilization

The improvement of a soil's engineering qualities and performance by the changing or maintenance of one or other soil characteristic is known as soil stabilization. Stabilization, in its broadest meaning, includes any techniques used to change a soil's characteristics in order to increase its engineering performance. It describes the process wherein a particular soil, cement, or other chemical elements are added to a natural soil in order to increase one or more of its characteristics. A soil deposit can be added a stabilizing agent to stabilize and allowing it to interact by permeating through soil voids, or it can be stabilized by mechanically mixing natural soil and stabilizing material together to create a homogeneous composition[40] .

2.10.1. Soil stabilization techniques for Road construction

A. Mechanical Stabilization

Mechanical stabilization is a procedure that improves the soil's stability and shear strength without changing the soil's chemical qualities in highly organic soils. The fundamental mechanical stabilizing methods are compaction, mixing or blending of two or more gradations, the application of geo-reinforcement, and mechanical remediation [41]

B, Chemical stabilization

Chemical soil stabilization involves altering the physio-synthetic around and inside clay particles so that less water is needed to correct the static imbalance. Chemical stabilization requires adding or injecting chemically active materials into the soil, such as calcium, sodium chloride, fly ash, Portland cement, or lime, or with viscoelastic substances like bitumen. Hygroscopic and deliquescent calcium chloride is utilized as a water-retentive ingredient in mechanically stabilized soil bases and surface [41]. Soil improvement by means of chemical stabilization can be divided into three types of chemical processes. like; cation exchange, flocculation-agglomeration and pozzolanic reactions.

I. Cation Exchange

Due to their natural tendency to have negative charges, soil clay minerals and organic matter use electrostatic forces to attract positively charged ions (cations) to their surfaces. The negatively charged surfaces of the clay minerals and organic matter in soil allow for the electrostatic adsorption and retention of positively charged ions (cations). Soils with a higher clay fraction tend to have a higher CEC. Organic matter has a very high CEC [42]

Cation-exchange capacity (CEC) is a measure of the total negative charges in the soil that absorb cations such as Ca2+, Mg2+, H+, Na+, and K+; it describes how many cations may be kept on soil particle surfaces. Positively charged atoms or molecules (cations) are bound by negative charges on the surfaces of soil particles, but they allow the cations to exchange with other positively charged particles in the soil water nearby. Various negative charges on soil particle surfaces, especially those of clay minerals and soil organic matter, lead to the development of cation-exchange capacity. Soil CEC fluctuates according to clay proportion, clay type, soil pH, and organic matter content. A higher CEC usually indicates that the soil has more clay and organic matter; high CEC soils often have more water holding capacity than low CEC soils.

The capacity of clay to absorb ions from the solution revealed the presence of such charges. Cations (positive ions) are more readily absorbed than anions (negative ions); hence, negative charges must be predominant on the clay surface. A cation, such as Na^+, is readily attracted from a salt solution and attached to a clay surface. However, the absorbed Na^+ ion is not permanently attached; it can be replaced by K^+ ions if the clay is placed in a solution of potassium chloride (KCl). The process of replacement by excess cation is called cation exchange. Some are more strongly attracted than others, and the cations can be arranged in a series in terms of their affinity for attraction as follows:

$Al^{3+} > H^{+} > Ca^{2+} > Mg^{2+} > NH^{4+} = K^{+} > Na+ > Li^{+}$

This series indicates that, for example, Al^{3+} ions can replace Ca^{2+} ions, and Ca^{2+} ions can replace Na^+ ions. The exchangeable cations may be present in the surrounding water or be gained from the stabilizers. H^+ is unique which has small size and high charge density. The process is called cation exchange [22].

An example of the cation exchange;

Na-clay + CaCl2 → Ca-Clay + NaCl

$Ca(OH)_2 \rightarrow Ca_2+ + 2(OH)$

Ca^{2+} + Clay Mineral-M^+ → Clay Mineral-Ca^{2+} + M^+

II. Flocculation and Agglomeration

The diffused water layer's thickness decreases as a result of cation exchange, causing clay particles to approach each other more closely resulting in the development of an edge to face attraction, thereby causing flocculation. The flocculation is also aided by the high concentration of electrolytes in pore fluid and high pH environment. The soil becomes friable and granular, making it easier to work and compact. The soil's flexibility index and ability to expand and shrink significantly reduce at this point.

Flocculated structure occurs in clays. The clay particles have large surface area and, therefore, the electrical forces are important in such soils. The edges of the clay particles are positively charged, whereas the surface is negatively charged. The positively charged edges and negatively charged faces come into contact with one another within the particle. This results in a flocculated structure.

Flocculent structure is formed when there is a net attractive force between particles. When clay particles settle in water, deposits formed have a flocculated structure. The degree if flocculation of a clay deposit depends upon the type and concentration of clay particles, and the presence of salts in water. Clays settling out in a salt-water solution have a more flocculent structure than clays settling out in a fresh water solution. Due to change in texture, a significant reduction in the swelling of the soil occurs. In general, the soils in a flocculated structure have a low compressibility, a high permeability and high shear strength

III. Pozzolanic Reactions

The rate of reaction between a pozzolan and Ca2+, Ca(OH)2, Si(OH)4, or H4SiO4 in the presence of water is measured by the pozzolanic activity. A silica-rich precursor with no cementing properties is changed by the pozzolanic reaction into a calcium silicate with effective cementing properties. By definition, a pozzolanic substance has the ability to bind calcium hydroxide in the presence of water. The evaluation of pozzolanic materials can therefore be done through the chemical measurement of this pozzolanic activity. This can be done by directly measuring the amount of calcium hydroxide a pozzolan consumes over time.

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

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

Clay clumps are initially covered and joined together by the calcium silicate gel that is generated. An interlocking structure is then formed by the gel as it crystallizes, increasing the strength of the soil. Pozzolanas are materials possessing alumina (Al_2O_3), silica (SiO_2), and ferrite (Fe_2O_3), with the sum total of their composition by weight equal to or greater than 70%. Cement, fly ash, ground granulated blast furnace slag, bitumen, quicklime, hydrated lime, ceramic powder, calcite, etc., have been in use as conventional binders having fulfilled the above minimum standard requirements, in various civil engineering and earth works. This is because they possess cementitious properties. These materials have been applied to improve the properties of soil in the earth and improve the properties of concrete, asphalt, etc., in other civil engineering areas of responsibility[43] When in contact with moisture, clayey soils have the ability to instantly swell. The clay minerals become charged during this stage of moisture percolation, with negative ions on the surface and positive ions around the edge. The adsorbed moisture also dissociates to its dipole forming hydrogen (H^+) and hydroxyl (OH^-) ions. This process is called hydration of the clayey soil where the soil is impregnated with moisture to its optimum content. According to previous research results, ceramic powder is an amorphous material known for its high composition of alum inosilicates (Al-Si-Fe), which satisfies the design standard conditions [40,41]

2.11 Jute Fiber

Jute is a type of fiber known as bast, which is long, soft, and shiny and may be spun into strong, coarse threads. It is used for sacking, burlap, and twine as a backing material for tufted carpets. It is the second most produced and useful after cotton which is one of the cheapest natural fibers It has the highest average tensile strength after bamboo, Maguey and Lechuguila. Jute fiber has also the highest modulus of elasticity after Bamboo. Cellulose, lignin, and pectin are generally found in plants and make up jute fibers. Jute is a common name for both the fiber and the plant it comes from. It belongs to the genus Corchorus in the basswood familyTiliacia[45]

Jute grows to a height of 12 to 15 feet in about 3 months during the growing season before being cut, packed, and kept submerged in water for the "retting" process, which separates the inner stem from the outside stem and "individualizes" the outer plant to create a fiber. The plant is then separated and cleaned to remove any dust. The fiber is transferred to jute mills for conversion into jute yarn and hessian after it has dried. There is around 5.6 thousand tons of jute production per year in Ethiopia. Figure 2.3 shows The extracted jute fibers and jute sack are purchased from market in Bahirdar and all Ethiopia respectively [46]. Several fabrics, including Hessian cloth, sacking, scrim, carpet-backing cloth (CBC), and canvas, can be made from jute. For this study, fabrics such as jute sacks were used when it is out of its original function.

A) Jute and is application

Among all Normal/natural Fibers which are abundantly accessible, Jute is guaranteeing because it is less costly, available in numerous forms and can be utilized as a substitute in indoor as well as outdoor application . Due to its potential, it has made interest among industries for substitute to woods and manufactured/synthetic fibers giving an ideal solution. Jute is mostly used in Textile field but it is also a primary material for other commodities. It is used for manufacturing packaging fabrics, carpet backing, mats, bags, ropes, etc.[47]

Asians are used In Automotive Field for ,manufacturing hoses, belts, seals, tires, diaphragms, dashboard, interiors, carpets, seat coach[48]. They are employed in the construction industry to create interiors, doors, windows, door shutters, ceilings, floors, roofing, drainage pipes, and electrical fittingsThey are employed in the transportation industry to produce the sleeper berth backing for railway coaches, seat backing, and bus backrests. They are employed in the furniture industry to create chairs, coat hangers, tables, table tops, shelves, and walls. In Industrial application, Jute along with epoxy is used in manufacturing circuit board laminates, surface coatings, adhesives, potting's, encapsulations for electronic circuit boards, etc[48]. As Natural fibers such as Jute are degradable in nature, nontoxic, nonhazardous have advantages over Synthetic Fibers in terms of Environment alertness and perception. As they have capability in terms of giving mechanical characteristics, they are now considered as alternative to Synthetic fibers[43].

B) Synthetic Jute Geotextiles (JGT)

One of the geosynthetic (GS) materials, geotextiles are mostly formed of polypropylene fiber, and their range of applications is expanding steadily as new scientific and technological advancements are developed. Jute geotextiles (JGT) are an emerging type of technical textile used in the geotechnical and bioengineering industries. These are created using both natural and synthetic or man-made fibers, and come in a variety of forms, sizes, shapes, and compositions to suit the conditions of the specific site as well as the functional requirements. These are a group of commodities, which are used for solving problems related to geotechnical, bio-engineering, agronomic and horticultural requirements by way of consolidation, filtration, separation, management of soil along with agricultural mulching and improvements/reinforcement [50]. Jute geotextiles made of synthetic woven fabric are used to improve the subgrade when building a highway[51]. The properties of soils under load are crucial to the stability of any civil engineering structure and it is to enhance soil stability for which use of geotextiles gained initial recognition as novel geotechnical materials. In more recent years, geotextiles have been employed in the construction of both new and repaired highways to enhance the tensile and mechanical properties of the materials used in civil engineering, such as road surfaces and sub-surface[52]. Jute fibers are usually strong with low extensibility. Moisture content in jute helps increase its frictional property[53]

2.12. *Ceramic Dust*

Materials comprised of clay, different elements, powders, and water are known as ceramic materials. They are inorganic, non-metallic, and non-reactive. Bricks, tiles, plates, glass, and sanitary ware are only a few examples of the materials that are most frequently used in a construction company. Construction and demolition garbage, which produces a million tons of waste annually, is the largest source of ceramic waste. Ceramic materials, which make up around 25% of the waste from construction and demolition, come from both the building process and from industry in the form of rejected bricks and tiles[54]. The ceramics industry's daily production is thought to be wasted to the amount of 30% [55]. It is available in Ethio Ceramic PLC, Tabor Ceramic PLC[56]

A, Mechanical properties of ceramics

Mechanical characteristics of ceramic bodies typically pertain to the characteristics of the materials, including strength, toughness, or thermal characteristics. Pottery is expected to resist several continuous mechanical stresses without experiencing fracture, structural damage and losing functionality[56]. In bending test, a material can be exposed until cracks initiate on the surface under tension. The fracture behavior of the brittle material is affected when the initiation of a crack is usually equivalent to the failure due to unstable crack growth. Besides that, toughness becomes as a highly important mechanical properties of a ceramic product as its measure of the intrinsic fracture energy [57]. Ceramics typically have a strength and modulus of 30-350 GPa.

2.13 *Ceramic Wastes on Soil Stabilization*

A lot of ceramic dust wastage is produced during formation, transportation and placing of ceramic tiles. Hence, its disposal is a problem, which can be removed with the idea of utilizing it as an admixture to stabilization. Utilizing industrial waste combined with weak soil for pavement construction is good since it will lower project costs, lengthen project duration, and reduce waste accumulation. Few types of waste materials namely crusher dust, fly ash and ceramic dust waste are popular as admixtures in improving weak soils[52,56] .

From the available literature, it is found that limited research has been done to study the effects of ceramic dust waste on different geotechnical properties of expansive soil. An expansive soil's index properties, compaction properties, CBR, and swelling pressure were all examined in a previous investigation to determine the effects of dust waste. Strengthening the expansive soil subgrade of a flexible pavement has allowed researchers to study the economics of stabilization. Thus, use of Ceramic dust waste not only improves the soil properties but problem of their disposal can also be solved. The previous study demonstrates that ceramic waste materials have been utilized to enhance the characteristics of clayey soils, and the impact of ceramic dust on various soil parameters has been assessed [57, 58]

2.14. Previous studies on materials to improve expansive Subgrade soil

2.14.1. Previous studies on jute geotextile fiber

The potential of jute geotextile fiber alone and in combination with other materials to enhance expansive subgrade soil has been the subject of numerous studies. The following are a few of them.

[60]Studied that the improvement of engineering properties of soil by using jute fiber treating with the sand. Jute fiber is treated with the sand to enhance the engineering properties in case of pavement and earthen slopes. Analyzed and discussed the potential of fine sand stabilization with jute is cut into approximately 20mm lengths as admixture. The various percent Fine sand with varying densities and moisture contents was combined with jute geotextile pieces in amounts of 0.5%, 1%, 1.5%, and 2%. After conducting all of the Unconfined Compressive Strength Tests, it was determined that 1.0% jute fiber was best for MDD and 1.5% jute fiber was best for UCS. A study used jute geotextile with bitumen-coated/polythene sheets in an experimental investigation on sub-grade soil to raise the CBR value. On clay soil (CI), some experimental experiments were done, but the findings were unreliable for significant compaction. The main use of JGT is to minimize the layers of pavement thickness, and when placed properly on a road sub-grade, designed woven JGT increases the road's bearing capacity (stated as CBR%). A sub-grade treated with JGT typically has CBR enhancement between 150% and 300% of the control value[61]. Studied the use of natural and artificial fibres is a suitable method for homogeneous soil reinforcing. The study looks into how Jute fibers effect the strength and engineering qualities of black cotton soil that has been treated with lime. The qualities of stabilized soil, including their variations with jute fiber content, were assessed.These properties included shrinkage limit, unconfined compressive strength, and California bearing ratio. Index qualities of soil samples with 0%, 1%, and 2% to 5% jute fiber were assessed in accordance with the applicable IS code of practice. According to the test results, the black cotton soil's expansive behavior has significantly decreased. If black cotton soil is combined with 5% lime and jute fibers from 0% to 5% by weight, the shrinkage limit rises from 13.75% to 28.68%. California bearing ratio and unconfined compressive strength have increased significantly [62].

[63] Jute fiber, when chopped into lengths of about 30mm, has been studied for its capacity to stabilize soil. Jute fiber was used in varied amounts, including 0.5%, 1%, 1.5, and 2%, and it was combined with soil. Laboratory tests like the California Bearing Ratio (CBR) test, modified compaction tests, and direct shear strength tests have been carried out to observe the change in the engineering qualities of soil. According to the results of the research, stabilizing soil with 30mm pieces of jute as a stabilizer enhances the soil's strength qualities. In comparison to the CBR of the unreinforced soil sample, which was 28.19%, the result indicates a good improvement by an increase of 25.2%. [64] Jute fibers of various lengths (30mm, 60mm, and 90mm) were studied to stabilize the subgrade soil, and proportions of 0.25 %, 0.50 %, 0.75 %, and 1% were used as the reinforcing agents. . The California Bearing Ratio test was conducted on all the samples, the results have been presented that CBR value of soil increases with the inclusion of jute fibre. The CBR value of soil increases further when jute fiber content rises, and this increase is notable at fiber content of 0.75%. As the length of the fiber grows, so does the CBR value of the soil. At a fiber content of 0.75%, for fiber with a diameter of 2 mm and a length of 90 mm, the greatest increase in CBR value above that of plain soil was found to be more than 200%. It has been determined that employing jute-geotextile for soil reinforcement is economically advantageous because it is a cheap and easily available material.

[65] The expansive soil was found to be stabilized quite well by adding jute reinforcement, as the soil's CBR values improved. After reinforcement, there have been significant changes in the soil. The soil's OMC reduced from 19.54 % to 15.98 %. After the 4-layer reinforcement of the jute layer, the MDD was seen to be lowered to 1.72 g/cc. The MDD increased at 2-layer reinforcement from 1.698 g/cc to 1.74 g/cc. The CBR values increased; the natural sample's CBR value was 2.67%; after the jute layer's 2-layer reinforcement, it improved to 6.07%; and after the jute layer's reinforcement was expanded to 4-layers, it reached a remarkable 11.85% CBR value.

2.14.2. Previous studies on ceramic dust waste

Many researchers had conducted several findings and outcomes on ceramic dust waste as soil stabilizer; to improve the expansive subgrade soil with other materials and alone. Some of them are as follows.

[57] Conducted research on expansive soil blended with tile waste from 0 to 30% in a 10% increment. Analysis of test findings revealed a drop in index features, OMC, and swelling pressure, but an increase in MDD and CBR as tile waste increased. [55] conducted the study and With an increase in the proportion of ceramic dust added, the MDD keeps rising while the OMC keeps falling. With an increase in the percentage of ceramic dust added, the saturated CBR keeps rising. When 30% ceramic dust was added, the soaking CBR value increased by 150% relative to the untreated soil. The Free swell and swelling pressure of the soil also decreased with increase in waste ceramic dust. With increase in percentage of WCD, the MDD of soil goes on increasing. The MDD increases from 15.6 kN/m3 to18.1 kN/m3 when WCD were increased from 0 to30%. This behavior results from the replacement of high specific gravity (2.82) WCD particles with low specific gravity (1.9) soil particles.

[66]added ceramic tile dust wastes up to 40% to find out its impact on swelling pressure and swelling potential of Na-bentonite, and observed that swelling pressure and swelling potential reduced by 86% and 57% with a 40% admixture of used ceramic tile dust.

[67]investigated the impact of adding ceramic waste to clay materials, and it was found through chemical, mineralogical, and morphological investigations that there would be no negative effects on the qualities of the sintered fire-clay products. The test results also show that the ceramic waste can be utilized as a filler in ceramic bricks, increasing the likelihood of its safe and sustainable reuse. [68] Investigated the impacts of ceramic dust waste on an expansive soil's index values, compaction properties, unconfined compressive strength, California bearing ratio, shear strength parameters, and swelling pressure. The local expansive soil was blended with 5% more clay dust, ranging from 0 to 30%. Following a thorough analysis of the test findings, it was found that while swelling pressure, cohesion, and cohesion pressure reduced with an increase in ceramic dust concentrations, MDD, UCS, and CBR increased. According to the analysis of test findings, the amount of tile waste had an impact on the liquid limit, plastic limit, optimal moisture content, swelling pressure, maximum dry density, and California bearing ratio. Additionally, it was found that tile waste up to 20% can be used for strengthening the expansive soil sub grade of flexible pavement with a significant reduction in construction costs. This was discovered from CBR and

swell pressure tests that stabilized expansive soil has shown maximum improvement compared to untreated expansive soil.

[56] Examined the efficacy of waste ceramic powder in expanding clay soil stability. Waste ceramic powder was used to stabilize the soil in concentrations of 5%, 10%, 15%, 20%, 25%, and 30% by dry weight of the soil. For Atterberg limits, compaction, and CBR tests, all stabilized soil samples were additionally given a 7-day curing period. Analysis of the data reveals a small improvement in the soil stabilization characteristics of waste ceramic powder.

2.14.3. Previous studies on combination of ceramic dust waste with jute geotextile fiber

The strength of expansive subgrade soil has not been studied using a combination of ceramic dust waste and jute geotextile fiber, and the studies that have been done using JGTF alone to strengthen expansive subgrade soil are listed above. Compaction tests were not improved because the fiber has a moisture-absorbing behavior, and the impact of curing in the UCS test was not examined [60,61,63,64]. This study used a variety of laboratory tests to demonstrate the effects of adding different percentages of ceramic dust waste and jute geotextile fiber.

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1 Study Area Description

The study was conducted at Jimma town. Jimma town is located at southwestern part of Ethiopia, which is about 354km far from Addis Ababa. According to WGS 84 coordinate reference system which is the latest revision of the World Geodetic System, Jimma is geographically located between 7° 38’52” and 7° 43’ 14” N latitude, and between 36° 48’ 00” and 36° 53’ 24” E longitude. The Town has a temperature that ranges from 20-30°C and the average annual rainfall of 800-2500mm and the City has an altitude of 1718-2000m above sea level. It is in the Weina-Dega climatic region, which is appropriate for both human settlement and agriculture. It is mostly covered with black, gray and red colored plastic clay soils.

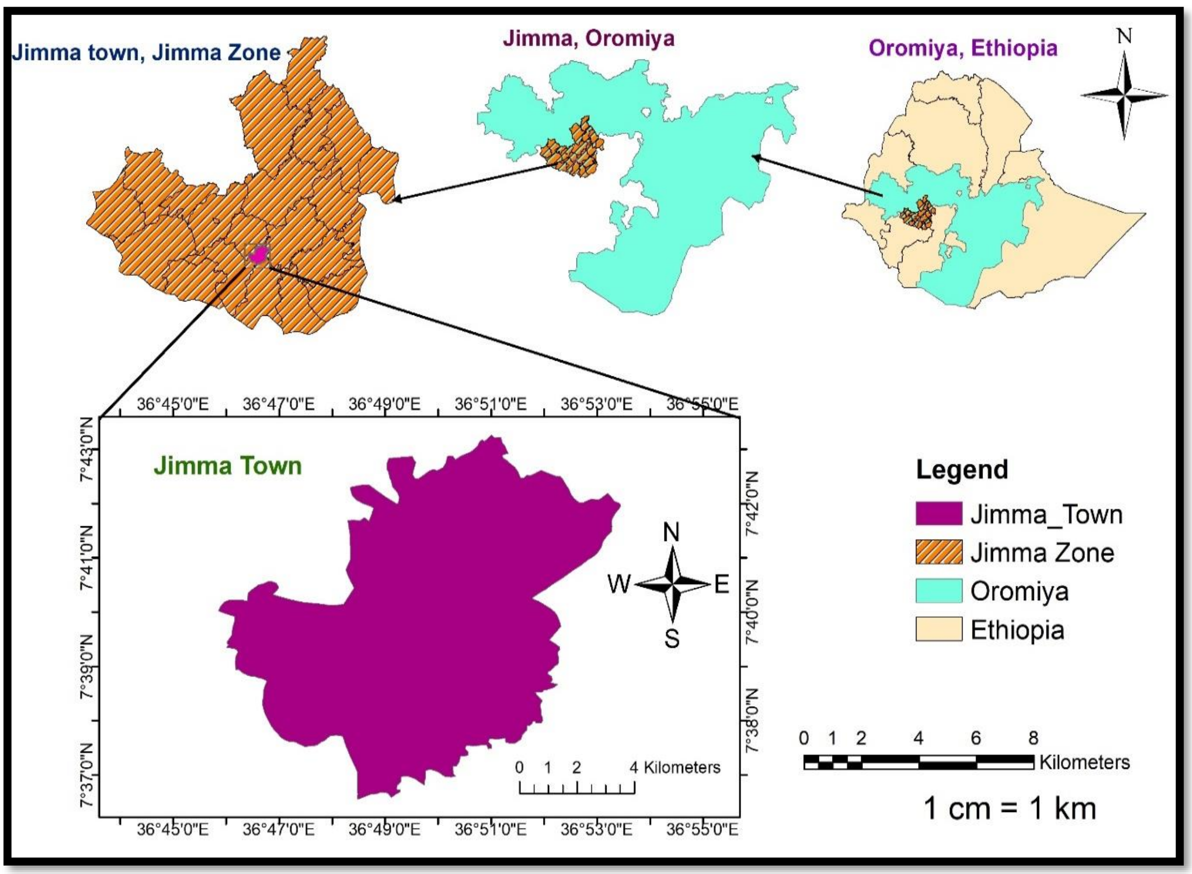


Figure 3.1: study area map (source ARC GIS)

3.2 Study design

The study was designed to answer the research questions and achieve its objectives based on experimental findings. The study began with collecting samples and procedures including Taking and transporting samples to the laboratory and preparation of samples for each laboratory test. For laboratory tests, the sample preparation and the tests procedure was conducted according to the AASHTO and ASTM code of practice and then the tests were performed on natural and treated soil samples. Three soil samples were selected and their index and engineering properties were determined. Then after one, the weakest soil sample was identified by its properties and taken to treatment. For the treated sample, the expansive soil was partially replaced by a mix of varying percentages (10 to20% at an increment of 5%) of ceramic dust waste and geotextile jute fibers (0.5%, 1%, 1.5%, 2% and 2.5% by weight of dry soil) and matrix combination was investigated. The laboratory test conducted was; natural moisture content, particle size analysis, free swell, Atterberg limit, specific gravity, compaction test, California bearing ratio (CBR) and USC tests, and out-of-laboratory shrinkage limit from LL vs PI chart and degree of Soil expansiveness from formula and activity was also determined. All listed laboratory tests and others were conducted for natural subgrade soil samples to determine their index and engineering properties.

Modified Proctor test, California bearing ratio (CBR), and USC for immediate and seven days cured tests were conducted for treated soil samples to determine its strength properties. The purpose of this study was to investigate and evaluate the combined effect of ceramic dust waste with jute geotextile fibers in stabilizing the expansive subgrade soil. After obtaining the results from a laboratory test, it has been analyzed and discussed thoroughly. Then the output from the analysis the optimum improving combination was determined and compered with standard specifications.

Finally, based on the outcomes of the laboratory tests, the research conclusions and recommendations were provided.

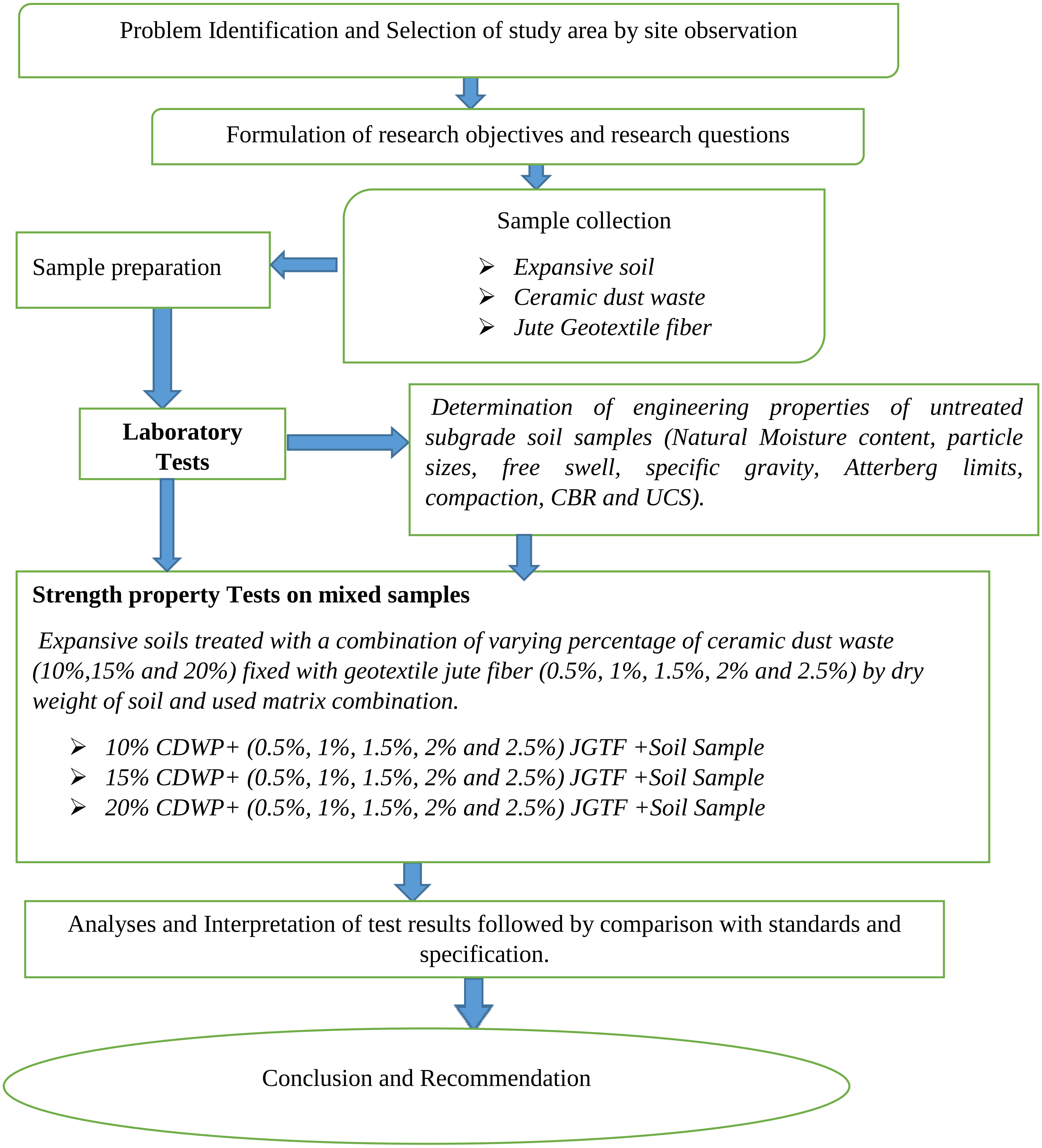


Figure 3.2: Study design flow chart.

3.3 Study variables

There are two type of study variables, which are dependent and independent variables.

3.3.1 Dependent *variable*

The dependent variable was more related with general objective of the study.

- The Strength of subgrade soil stabilized with waste ceramic dust and jute geotextile fiber mix.

3.3.2 Independent variables

The independent variables of this study were-

- Engineering properties of natural and strength properties of treated expansive subgrade Soil:-
- ✓ Natural Moister content
- ✓ Atterberg limit(LL, PL& PI)
- ✓ Free swell
- ✓ Compaction (OMC&MDD),
- ✓ CBR and CBR Swell
- ✓ UCS
- Dosage of ceramic dust waste and Geotextile jute fiber

3.4 Sampling technique

Purposive sampling, which is not a probability method, was the sampling technique used. Because the experimental analysis of the study was specifically performed on an expansive subgrade soil sample, this study selects the samples in accordance with some criterion, which is considered relevant for the particular investigation.

3.5 Sample Size

For this study, three expansive soil samples from different locations; sample one from Bossa kito Addis ketema, which is located latitude of 7° 41’07” and longitude of 36° 49’ 41”, sample two from Bachobore located latitude of 7° 39’49” and longitude of 36° 50’ 27” and sample three from Koch segment around catholic school located latitude of 7° 40’55” and longitude of 36° 50’ 44” along a flexible pavement Road segment in Jimma City at depth of 1.5m-2m to remove organic matter was taken for sampling and other ceramic demolished or building construction sites abandoned as waste when broken by Mishandling, Transportation problem, Dimension defects or incurring problems throughout the industrial process and geotextile jute sack for fiber also collected from the commercial center of garden fruits and cereal veranda of marcato in Jimma City, purchased by a very cheap price. The samples were obtained by inspecting the study area and selecting the required data. Site visiting is the first and foremost investigation of soil nature. It was important to get information about the texture of soils around the vicinity area.

3.6 Data source and collection procedures

The procedures used to collect the data included: field visual inspection, which was followed by a categorization of the area's soil conditions based on previous research. Next, representative locations for sampling were chosen based on the availability of expansive soil. Both primary data and secondary data sources were used. Primary data for this study were a field investigation and laboratory experiment output. Secondary data used for this research were collected from different standards, journals, books, websites, manuals and others.

3.6.1 Field investigation

In the field, expansive soils can be identified by applying several techniques. In this study the method listed under [25]was used to identify expansive soils of the study area during dry season. According to this method, expansive soils have the following properties;

- ✓ Have a color of black or gray and dark gray to brown
- ✓ Polygon pattern of surface Crack during dry season
- ✓ A shiny surface when partly dry piece of soil is polished with finger
- ✓ High dry strength and low wet strength
- ✓ Sickness and low traffic ability when wet

Depending up on the above methods, expansive soils of three different location where identified and collected.

3.6.2 Materials and sources

Table 3.1 Materials Description

No	Material used	Description	Area/sources of materials
1	Expansive Soil	From Three pits, expansive soil samples of subgrade materials along flexible pavement Road segment According to ERA manual at depth after 1.5m from natural ground level to remove organic matter. The disturbed/undisturbed samples were collected. The soil is black gray, black and black gray in color respectively.	**SS1=** From Saris-Ferenj Arada Segment (Bossa kito Addis ketema) **SS2 =**From Bus station to Mizan Segment (Bachobore) and **SS3 =**From Aretu Anbessa to Kochi Segment (around Catholic school) From Jimma City.
2	Jute Geotextile fiber	Synthetic mechanically,woven jute type or thread woven fabrics type and are a cut by using scissors having approximate caliper measured thickness of 0.32-0.43mm with average length of 35mm was used.	JGT Sack Purchased by very cheap price From commercial center of garden fruits and cereal veranda of markato in Jimma City abandoned after torn.
3	Ceramic waste	waste when broken by:- ✓ Mishandling ✓ Transportation problem. ✓ Defects in dimensions or issues that arise throughout the industrial process and it was used after broken to pieces, then crushed and powdered manually by using means of hand.	Collected from different building construction sites abandoned as waste from Jimma City

Figure 3.3: Overviews of test pits and stabilizers (photo captured by Sintayehu B.)

3.6.3 Preparations of Natural soil and stabilizers for laboratory tests

Before treatment and testing, the samples were prepared by the method described in AASHTO T87-86 for soil sample. This process involves:

- ✓ Air drying of samples and oven drying at 105 ±5ºC, air dried sample was used for this study
- ✓ Breaking up the soil aggregates by a rubber covered mallet and adequately pulverized.

- ✓ Then sieve analysis are conducted on properly pulverize natural soil. Sieves are conducted into three groups. The first team are soil samples passing #40 (0.425mm) sieve for Atterberg limits and free swell, 2.00mm sieve for specific gravity and the third group are soil samples passing # 4 (4.75mm) for compaction ,California bearing ratio and Unconfined compression strength (UCS)tests.

Figure 3.4: Samples air-drying and preparation procedure (photo captured by Belete A.)

3.6.4 Samples mixing procedure

- ✓ jute geotextile fiber with ceramic waste powder was first added to air-dried, pulverized, sieved soil samples and dry mixed thoroughly

- ✓ Finally, wet mixing was done by sprinkling according to test types measured amount of water followed by a thorough mixing until a uniform soil-additive matrix was obtained

Figure 3.5: Sample-mixing procedure (photo captured by Mohammed N.)

3.7 Laboratory Test procedures and types of tests

Test Procedures

The soil samples were taken from the site and dried by air. Depending on the test type, the soil sample was sieved after air-drying. The tests were performed on expansive subgrade soil before and after a mix of waste ceramic dust with a geotextile jute fiber combination:-Two test methods are adapted and they are:- Untreated expansive subgrade soil test and Treated expansive subgrade soil by the waste ceramic dust with geotextile jute fiber combination. Untreated expansive subgrade soil test covered in laboratory was like; natural moisture content, grain size analysis, Atterberg limit, specific gravity, free swell, compaction , CBR and CBR swell and UCS tests ; to determine samples geotechnical properties before mixing and after determination of its properties, the most expansive or the weakest soil sample was selected and treated.

This test was conducted on a combination of ceramic dust waste of various proportions (from 10 to 20% at an increment of 5%)by weight of dry soil with various proportions of geotextile jute fiber (0.5%, 1%,1.5%, 2% and 2.5% by weight of dry soil) with expansive soil and its combinations method seen in study design. Ceramic waste collected from building construction sites abandoned as waste and manually hand-crushed and hand-pulverized and Geotextile jute fiber used was synthetic woven geotextile jute made from jute, or thread woven fabrics type having approximate caliper measured thickness of 0.32-0.43mm with an average length of 35mm was used for this study, and Laboratory tests conducted for strength characteristic of bearing and compressive strength were; proctor compaction, CBR, and UCS tests and consistency tests were not suitable because of fiber characteristic(shape, diameter, length, aspect ratio, texture), fiber concentration, distribution and orientation, fiber mixing method with soil[18]. Then the optimum improving combination was determined.

Types of conducted tests

i. *Natural moisture content test*

The natural moisture content of natural subgrade soil was tested using the ASTM D-2216 standard test protocol. A correlation between soil behavior and its index qualities has been established using this test, which is one of the most important index features. The amount of water in a material that is used to indicate the relationship between the phases of solids, water, and air in a particular volume of material. Moisture content was determined by the ratio of mass of water to mass of soil.

- ✓ For this, study the moisture or water, content of natural soil sample determined by oven-drying method.
- ✓ The disturbed sample from the site was placed in the moisture can and covered or secured with a lid. After the samples were brought from the site to the laboratory testing room, the soil sample was weighed, the secured lid removed, and the sample was dried in the oven for 24 hours.
- ✓ The sample was then reweighed, and the weight of dry soil was divided by the difference in weight provided by the soil's moisture content. Moisture content was determined for the soil samples under investigation using oven temperature of 105°c$\pm$5 for 24 hour

Figure 3.6: determining natural moisture content (photo captured by Mohammed N.)

ii. ***Atterberg limits test***

This test method is in line with ASTM D-4318 for the determination of the liquid limit, plastic limit, and plasticity index. When subjected to 25 shocks from the cup being dropped 10 millimeters in a standard liquid limit apparatus run at a rate of two shocks per second, a portion of soil in a standard cup that has been cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 in.). The liquid limit (LL) refers to this. The plastic limit (PL) of a soil is the water content at which it can no longer be rolled into 3.2 mm (1/8 in.) diameter threads without collapsing. The distinction between liquid and plastic limits is represented by the plastic index.

This lab was performed to determine the plastic and liquid limits of a fine-grained soil. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 425µm (No. 40) sieve. The liquid limit of each soil had been determined by using Casagrande apparatus. The plasticity index was then computed for each soil based on the liquid and plastic limit obtained.

Figure 3.7: Atterberg's limit tests procedure (photo captured by Belete A.)

iii. Specific Gravity test

By using a soil sample that passes through a No. 10 sieve (2mm sieve size) and is oven-dried at 105°c±5, the specific gravity, or measure of the heaviness of the soil particles, is calculated. In this lab, a density bottle was used to calculate the soil's specific gravity. The ratio of a material's mass to that of an equivalent volume of water is known as its specific gravity. The purpose of this test was to determine the specific gravity of natural soil and ceramic dust. It was conducted in accordance with ASTM D-854 standard. The significance of finding out the specific gravity in this study was to find out particle sizes in hydrometer analysis. In effect, it tells us how much the material is heavier than (or lighter).

Figure 3.8: Specific gravity test (photo captured by Belete A.)

iv. Particle Size Distribution test

This test was performed to determine the percentage of different particle sizes enclosed within a soil. The determination of grain size analysis can be done in two ways: mechanical sieve analysis and sedimentation (hydrometer analysis).The distribution of the larger, coarser particles is determined using the sieve analysis, while the distribution of the smaller, finer particles is determined using the hydrometer method. For this study, both wet sieve analysis and hydrometer analysis was done according to ASTM D-422.

For wet sieve analysis, 1000 gm of air dried natural soil sample was put in a tray and soaked in water. After 24 hours, the samples were washed on the75-µm sieve and dried it in an oven. Oven dried samples were sieved on mechanical sieve by using sieve shaker on sieve sizes 9.50 mm, 4.75 mm, 2.00 mm, 0.85 mm, 0.425 mm, 0.30 mm, 0.15 mm, 0.075 mm . The retaining samples were weighted. However, this method is only useful to differentiate only gravel and sandy soil of coarse-grained soils.

For fine-grained soils to differentiate silt and clay soil particles sedimentation, (hydrometer) analysis was used. For this materials passing on 75-µm sieve were collected for hydrometer analysis then performed to measure the amount of silt and clay size particles. About 50 gram of soil was taken and 125 ml solution of sodium hex metaphosphate as dispersing agent was added to soil, distilled water was added and treated for 24 hours. Then the soaked soil was transferred to dispersion cup and stirred for 15 minutes. The soil mixture was poured into the standard measuring flask and made total volume of oil suspension exactly by 1000cc. Finally, hydrometer measurements were used to calculate the percentage of silt and clay-sized particles in the sample.

Figure 3.9: Particle size distribution test (photo captured by Mohammed N.)

v. Free Swell test:-

AASHTO and ASTM have not yet standardized this test. The method was suggested by Holtz and Gibbs (1956) and Indian standard IS 2720 (part XL) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample without being loaded.

The procedure consists of pouring very slowly of 10 cm^3 of that part of the dry soil passing No. 40 (0.425 mm) sieve in to a 100 cm^3 graduated measuring cylinder and letting the content stand for approximately 24 hours until all the soil completely settles on the bottom of the graduating cylinder. For each pit, two oven-dried soil samples were taken to conduct the test. One was put in a 100 cm^3 graduated glass cylinder containing kerosene and the other sample was put in a similar cylinder containing distilled water. Both the samples were left undisturbed on quiet area for 24 hours and then their volumes were noted. The test was carried out to determine the natural soil's free swell (%). The degree of expansiveness of the samples is fairly approximated by the free swell test**.** A result calculated by the following formula.

$$FS = \frac{Vf - Vi}{Vi} * 100\% \qquad (3.1)$$

Where: FS (%) = Percent of free swell Vf = Final volume change Vi = Initial volume

Figure 3.10: Free swell test Procedure (photo captured by Sebisbe M.)

VI .Compaction test:-

As a result of soil compaction, future settlement and permeability are reduced by reducing the air space in the soil and increasing its stiffness. Compaction tests come in two different types: Standard Proctor compaction and Modified Proctor compaction test. In this particular study it was performed modified proctor compaction test. Increasing soil density can generally increase the majority of engineering attributes, including soil's strength, stiffness, resistance to shrinkage, and imperviousness. The amount of mechanical energy used to compact the soil is known as the compactive effort. In the field, soil is compacted using a variety of techniques, some of which include vibration, kneading, tamping, and static load compaction. This laboratory was conducted the tamping compaction method using optimum water content and maximum dry density.

The laboratory test was conducted with accordance of ASTM D-1557 using modified effort of hammer 4.54 kg for 457.2 mm height. The test was conducted on disturbed samples of soil that passed through sieve sizes of 19 mm mixed with water to create samples at varying moisture levels ranging from the dry state to the wet state. These samples were compacted in 5 layers at 56 blows per layer in accordance with the specified nominal compaction energy of modified proctor test. Based on the moisture content and unit weight of compacted soil, dry density was determined. It was done to determine the material's optimum moisture content (OMC) and maximum dry density (MDD). The test was done on the natural soil and then various percentages of ceramic dust waste and Jute geotextile fiber added on the Expansive clay soil and MDD and OMC were determined.

Figure 3.11: Compaction test Procedure (photo captured by Mohammed N.)

vii. California Bearing Ratio (CBR) and CBR-Swell Test:

The California bearing ratio (CBR) test is frequently used to determine the durability or strength of subgrade soil, subbase course material, and base course materials for use in roadway and airport pavements. The test is used primarily to determine empirically required thicknesses of flexible pavements for highway and airfield pavements. In this case the test was conducted to determine the strength of subgrade soil. It is expressed by the force exerted by the plunger and the depth of its penetration into the specimen. It is designed to determining the relationship between force and penetration. The CBR and CBR-swell tests were conducted in accordance with ASTM D-1883 for the natural soil, soil stabilized with ceramic dust waste powder and jute geotextile fiber mix. CBR tests were conducted on the compacted specimens at the optimum moisture content using a modified compaction test. A three point CBR tests at 10, 30 and 65 blows were conducted according to ASTM specification and the CBR values at 95% MDD was determined. The samples passing # 4 (4.75mm) sieve size were compacted in CBR molds at optimum moisture content and gauged for swelling characteristics before and after soaking for four days to evaluate the percentage of swell. The CBR swell of the soil was measured by placing the tripod with the dial indicator on the top of the soaked CBR mold. The initial dial reading of the dial indicator on the soaked CBR mold was taken just after soaking the sample. At the conclusion of 96 hours, the final dial reading of the dial indicator was taken.

Figure 3.12: CBR and CBR-swell test (photo captured by Giza T.)

Viii. Unconfined Compression Strength (UCS) test

According to ASTM D-2166 material testing manual, the main aim of performing unconfined compressive strength test is to determine the approximate compressive strength of soils with sufficient cohesion to permit testing in the unconfined state. The unconfined compressive strength (qu) is defined as the compressive stress at which an unconfined cylindrical sample of soil is fail in a simple compression test and shear strength is half of the unconfined compressive strength.

It gives measurements of the soil's stress-strain properties and undrained compressive strength. Since it is one of the quickest and least expensive techniques of determining soil shear strength, the unconfined compression test is a common technique. The specimens were created using the ideal water content and maximum dry unit weight. The study was conducted for both uncured and seven-day cured UCS testes. The strength has been tested for natural and stabilized samples. For natural soil, an undisturbed soil sample was brought from sampling pits by Shelby tube for testing, and sensitivity checked. Disturbed samples were remolded by using previously determined maximum dry density and optimum moisture contents for natural soil samples and soil stabilized with ceramic dust waste and jute geotextile fiber, and the strength was tested for both immediate and seven days cured. Samples were cured by means of packing or enfolded with a low-density Polyethylene plastic bag and put in the desiccator to fix the loss of moisture with the water was added until pores of desiccator and left for seven days.

Figure 3.13: UCS test Procedure (photo captured by Haile T.)

3.8 Properties of Ceramic dust waste

3.8.1 Physical properties of Ceramic dust waste

Geotechnical Properties of Waste Ceramic Powder [56] was used as reference in this study.

Table 3.2***:*** Geotechnical Characteristics of Waste Ceramic Powder

No.	Properties	Value
1	**Grain size Analysis**	
	% passing Sieve No.200	84.8
2	Free Swell (%)	5.02
3	Specific Gravity	3.06
4	**Atterberg Limit**	
	Liquid limit (%)	28.5
	Plastic limit (%)	18.4
	Plastic index (%)	10.1
5	**Compaction Characteristics**	
	Optimum Moisture Content	14.8
	Maximum Dry Density	2.58
6	CBR (%)	12.9
7	CBR Swelling (%)	0.132

Specific gravity of CDW studied by [55] was 2.82 and the studied value specific gravity for this study was 2.81

3.8.2 Chemical properties of Ceramic Dust Waste

Chemical composition of CDW taken from Ethiopia Geological survey that was similarity with Jijo James and Kasinatha Pandian [69] presented the chemical composition of ceramic dust which was obtained from XRF Scanning Electron Microscopy was used in this study.

Table 3.3: Oxide Composition of Waste Ceramic Powder.(Source from Ethiopian Geological Survey)

Constituents (Symbols)	Constituents (Names)	Chemical Composition (% by mass)	Requirement (%) ASTM C-618
SiO_2	Silicon Oxide	67.51	SiO2 + Al_2O_3 + Fe_2O_3 above 70%
Al_2O_3	Aluminum Oxide	16.92	
Fe_2O_3	Ferric Oxide	0.75	
CaO	Calcium Oxide	3.13	
MgO	Magnesium Oxide	1.82	5 and below
K_2O	Potassium Oxide	1.31	
Na_2O	Sodium Oxide	4.8	
P_2O_5	Phosphorus pentoxide	0.023	
SO_3	Sulfur trioxide	0.01	5 and below
Cl_2	Chlorine	0.043	
MnO	Manganese(II) oxide	0.12	
TiO_2	Titanium dioxide	0.04	
L.O.I	Loss on ignition	1.2	Organic content of CDW

3.9 Properties of Jute Fiber

3.9.1 Physical properties of jute fiber

The physical and tensile properties of Jute Fiber studied **(**Abeba Asra &Temesgen Wondimu, 2017"Addis Ababa Science And Technology& R. B. S. Mamo, Yesheneh Jejaw,2020, Bahir Dari) was used in this study [43,44]

Jute fibers typically have a high strength and limited flexibility. Jute's moisture content enhances its ability to resist friction [53]

Table 3.4: Tensile and physical properties of jute fiber

Properties	Descriptions
Density	1.47gm/cc
Tenacity/strength	4.2gm/denier
Average Toughness Index	0.02
Ultimate diameter for JGT woven(mm)	0.32-0.43
Color	white, off white, yellow, brown, grey, golden
Specific heat	0.34 Cal/g/c0
Tensile strength (MPa)	393-773
Ultimate diameter single(mm)	0.015-0.02
Moisture absorption (%)	11
Specific gravity	1.5
Fibers length	5-12ft
Dimensional stability	good

3.9.2 Chemical properties of Jute Fiber

The chemical properties studied by (R. B. S. Mamo, Yesheneh Jejaw,2020, Bahiri Dari & B. S. Praveen Aggarwal,2010) [44,52]**.** used in this study.

Jute is a ligno-cellulosic, composite natural bast/steam Fiber. Cellulose, hemi-cellulose and lignin are its major constituent components & its three dimensional structure is formed by different inter and intra-molecular forces resulted from various physical, chemical, and hydrogen bonds, between them and they makes strong bond when using jute fiber for expansive subgrade stabilization. The lignin and pectin content. Apart from this, the superior engineering properties makes it an excellent material as reinforcement in composites for various applications[46].

Table 3.5: Jute fiber's chemical composition

Properties	Chemical Composition (% by mass)
Holocellulose	82-85
Alpha Cellulose	58-63
Hemicellulose	21-24
Lignin	12-14
Pectin	0.2-0.5
Fat & Wax	0.4-0.8
Protein	0.8-1.5
Mineral Materials	0.6-1.1

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Introduction

This chapter includes a description of the results of the laboratory tests as well as the engineering characteristics of samples of natural soil and soil that has been stabilized using CDW and JGTF mix. Natural moisture content, particle size analysis, Atterberg's limits tests, free swell, specific gravity, compaction, CBR, and UCS tests on samples are among the laboratory tests. Additional tests measure soil expansiveness based on a formula and potential soil swelling based on activity was presented.

4.2. Engineering Properties of expansive subgrade Soil Samples for selected area

4.2.1 Natural Moisture Contents

The natural moisture content of SS1 was 40.01% , SS2 was 43.01% and SS3 was 42,28%. These soils have a high moisture content, and Table 2.9 [31]'s literature review section observed that the soils were poor for subgrade materials because they included 30%–50% soft clay or clay. Results from the laboratory tests are reported in Table 4.1, and tabular experimental results are shown in Appendix A-1.

Table 4.1: Natural moisture content of samples

Samples	Natural moisture content (%)	Description
SS1	40.01	Clay or soft clay
SS2	43.01	Clay or soft clay
SS3	42.28	Clay or soft clay

4.2.2 Atterberg Limits

The result indicates that for three soil samples have very high swelling potential and those were failed to use for subgrade. According to USCS Soil classification System, LL less than 35% indicates low plasticity, between 35-50% intermediate plasticity, between 50-70% high plasticity and greater than 70% is very high plasticity. Hence, LL value indicates that the soils were highly plastic clay. As a result, the soils fall under the USCS soil classification system's CH (inorganic clay of high plasticity) soil class and the A-7-5 soil class according to the AASHTO soil classification system, which indicates that the soils have a plasticity index of equal to or less than (liquid limit -30). This type of soil is typically categorized as a material with weak engineering properties and should not be used as a sub-grade material. These values indicate three soil samples were very high plastic clay; the subgrade is easily deformed and expanded, and it is weak to both internal and external loads. Finally, the structure begins to crack and is quickly destroyed. To avoid this failure Stabilization with various additives should be necessary to improve its engineering properties. The laboratory test data presented in the Table 4.2 and detail tabular test results of atterberg limits shown in Appendix A-2

Table 4.2: Atterberg limit test results of untreated Samples and soil classification

Samples	Atterberg Limits			%passing of sieve #200	Soil classification		Swelling potential	Color	Performance
	LL%	PL%	PI%		AASHTO	USCS			
SS1	83.02	35.85	47.17	94.13	A-7-5 (54)	CH	Very high	black-grey	Very weak soil
SS2	97.51	41.14	56.37	95.48	A-7-5 (67)	CH	Very high	black	Very weak soil
SS3	81.09	35.75	45.34	95.03	A-7-5 (53)	CH	Very high	black-grey	Very weak soil

4.2.3 Specific Gravity

Specific gravity is the measuring of the heaviness of soil particle, was determined by using the soil sample passed 2mm sieve size and it was oven dried and density bottle having a capacity of 100 ml used to determine the three pits of soil samples. An average specific gravity of sample one was **2.7**, sample two was 2.67 and sample three was 2.7**.** According to [32] . The specific gravity of clay and silty clay of most soils between 2.67-2.9. For the most of calculation, the specific gravity can be assumed as 2.65 for cohesion less soils and 2.70 for clay soils. Therefore, the results indicates that the three soil samples were grouped under clay soil. The summary of the test results were tabulated in table 4.3, while the laboratory test analyses were given in Appendix A-3 for Soil samples and Appendix-B for CDW.

Table 4.3: Specific gravity soils and Ceramic dust samples

Sample Types	Specific gravity value
SS1	2.7
SS2	2.67
SS3	2.7
CDW	2.81

4.2.4 Particle size distribution

The soil sample one was black-grey in color and almost 94.13% of the soil was passed through No.200 sieve and 5.15% retained. Fine grained soils; silt and clay contains more percent 39.53% and 54.60% respectively, The soil sample two was black in color and 95.48% of the soil was passing through No.200 sieve (75µm) and 4.05% retains. Samples passed through No.200 sieve (75µm) are fine-grained soils and retained on this sieve are coarse-grained soils. Fine-grained soils; silt and clay contained more percent 27.01% and 68.47% respectively and the soil sample three was black-grey in color, and 95.03% of the soil was passing through No.200 sieve (75µm) and 4.67% retained. Samples passed through No.200 sieve (75µm) are fine grained soils and retained on this sieve are coarse grained soils. Fine-grained soils; silt and clay contained more percent 29.25% and 65.78% respectively. According to [1]the result indicated, the soil contains mostly clay and medium degree of expansive for soil sample one and high degree of expansiveness for soil sample two and three. The tabular experimental results were presented in Appendix A-4 and the combined particle size distribution curves were shown in Figures 4.1.

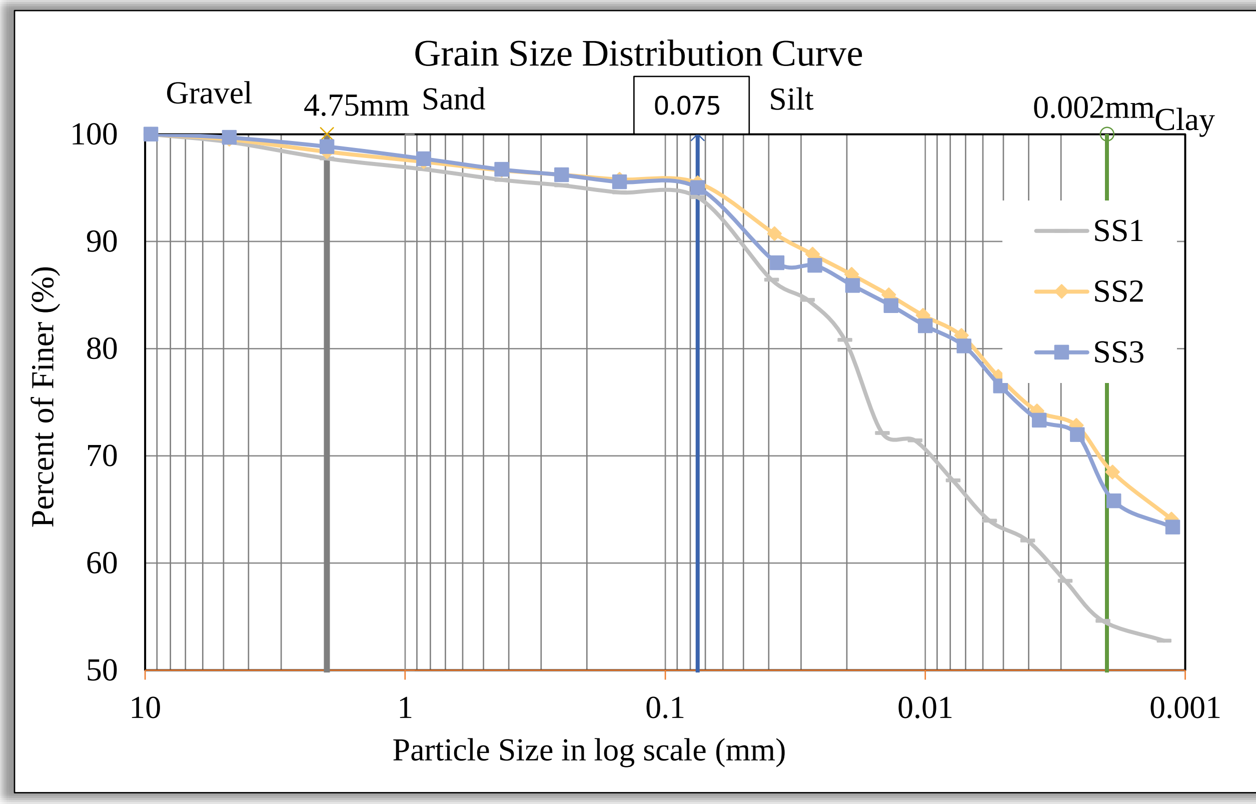


Figure 4.1: Particle size distribution curve of SS1, SS2 and SS3

4.2.5 Activity

Table 4.4: Activity of soil samples

Samples	PI%	C(Percent of clay size friction)%	Activity(A)=PI/C	Nature of soil	Expansion/ Swell potential
SS1	47.17	54.60	47.17/54.60=0.863, b/n (0.75-1.25)	Normal clay	High
SS2	56.37	68.47	56.37/68.47=0.823,b/n (0.75-1.25)	Normal clay	High
SS3	45.34	65.78	45.34/65.78=0.689<0.75	Inactive clay	Low

4.2.6 Free Swell

Free swell of sample one was 89%, sample two was 95% and Sample three was 91%. Soils are called having very high degree of expansion when the free swell index exceeds 50% as (Holtz and Gibbs,1956) cited by[71] , High when the degree of expansion is 35-50%, moderate when it is in 20-35% and low degree of expansion when it is less than 20%. This result indicates that the soils are highly expansive. Three soil samples have a very high degree of expansion, and these soils experience volumetric changes that cause pavement distress, cracking, and overall unevenness as a result of seasonal wetting and drying. The results of the soil's free swell testing were revealed in Table 4.5 and Appendix A-5

Table 4.5: Free swell of Soil Samples

Sample pits	Initial reading	Final reading	% free swell	Soil expansion
SS1	10	18.9	89	Very High
SS2	10	19.5	95	Very High
SS3	10	19.1	91	Very High

4.2.7 Shrinkage Limit

Potential for shrinkage, the potential for the development of cracks, or both, in earthworks using cohesive soils. By using Wax Method plotting plasticity index and liquid limit was determined and shrinkage limit value for Soil Sample one was 19.3%,Soil Sample two was 19.5% and Soil

Sample three was 19%and the SL of three samples were greater than 12 . The result indicates the degree of expansion was non-critical and the plotted value indicated in figure 4.2 and table 4.6.

Table 4.6: Shrinkage limit determined from graph

Samples	LL (%)	PI (%)	SL (%)	Degree of Expansion
SS1	83.02	47.17	19.3	Non-critical
SS2	97.51	56.51	19.5	Non-critical
SS3	81.09	45.34	19.0	Non-critical

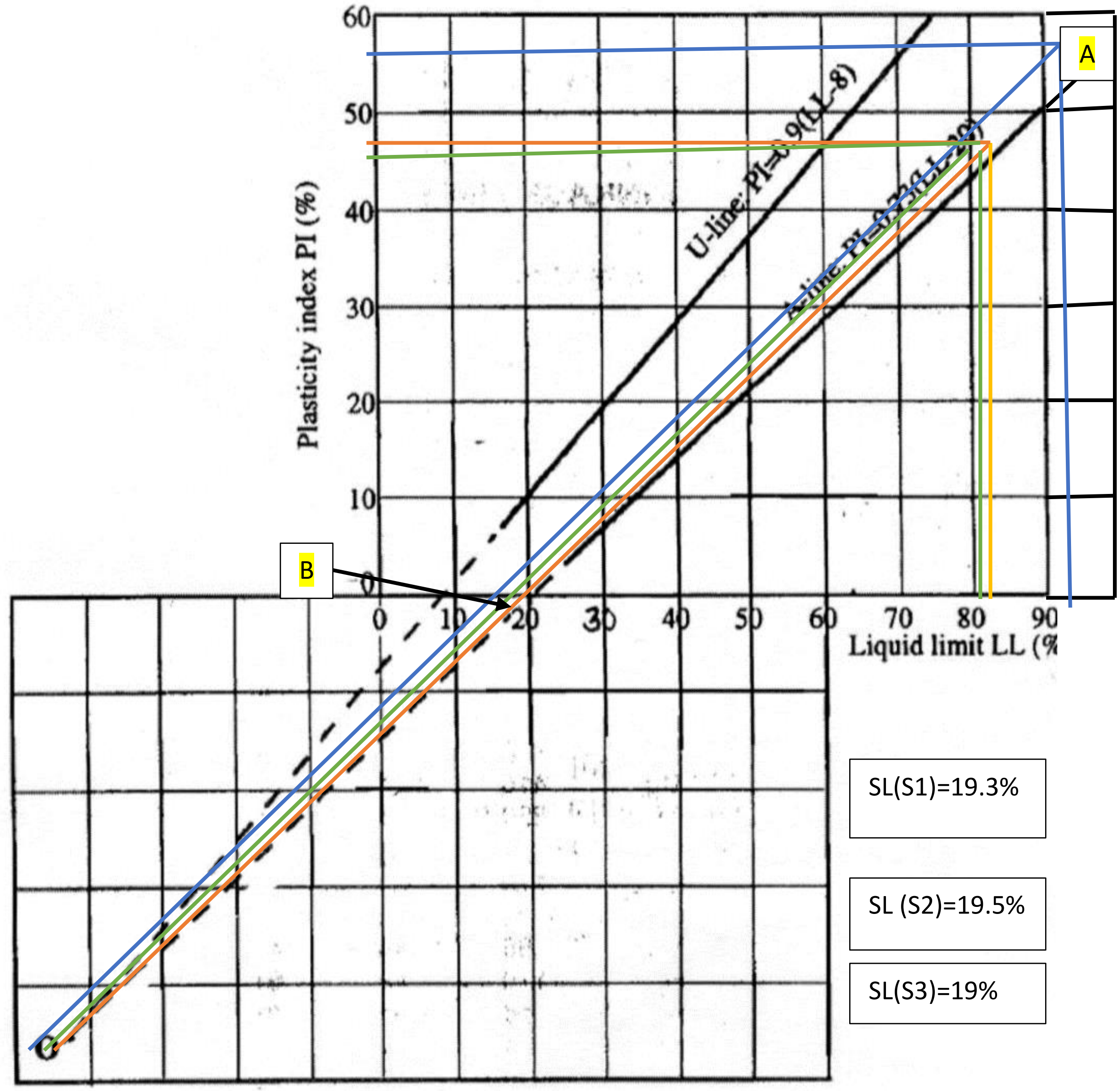


Figure 4.2: Determination of shrinkage limit from LL and PI

4.2.8 Soil Expansiveness from Formula

For the Soil sample one, percentage of passing through sieve size (0.425mm), and Plastic index was 95.75% and 47.17% respectively, for the soil sample two percent of passing through sieve size (0.425mm) and Plastic index was 96.64% and 56.37% respectively, in addition for soil sample three, the percent of passing through sieve size (0.425mm) and Plastic index was 96.72% and 45.34% respectively, The findings show that three soil samples are highly expansive. accordingly to ERA (2002). The summary of the calculated results were tabulated in Table 4.7

Table 4.7: Expansiveness determined for soil samples by formula

Samples	%Passing by sieve size(0.425mm)	PI (%)	SL (%)	Wp	Ws	Expansiveness (€ex)	Classification
SS1	95.75	47.17	19.3	45.17	18.48	68.91	High
SS2	96.64	56.37	19.5	54.48	18.85	89.76	High
SS3	96.72	45.34	19	43.85	18.38	66.08	High

4.2.9 Compactions (Moisture-density)

Modified proctor test method was used to determine the MDD and OMC of natural soils according to ASTM D-1557. The OMC of soil sample one was 22.401% and MDD 1.462 g/cc. In addition the OMC and MDD of sample, two was 30.05% and 1.625 g/cc and Sample three was 20.05% and 1.39g/cc respectively. The outcomes of the lab tests were displayed on Figure 4.3 and the summary of test results also tabulated and attached in Appendix A-6

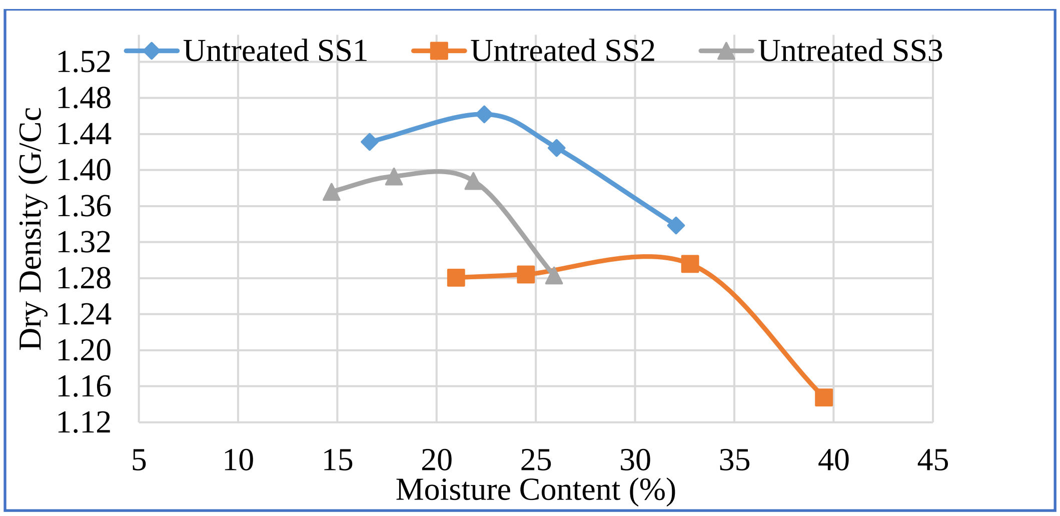


Figure 4.3: OMC and MDD of SS1, SS2 and SS3

4.2.10 Soaked CBR and CBR swell

As result indicated in Table 4.8, the CBR value at 95% MDD compaction of SS1 was 1.54% and CBR swell was 8.08% and CBR of SS2 and CBR Swell was 1.03% and 10.07% respectively, On another hand CBR of SS3 and CBR swell was 1.77% and 7.35% respectively.

CBR values of natural sub-grade soils of the three samples have not fulfilled the requirement of sub-grade soils as per ERA standards. According to ERA manual 2013 specification, It is not permitted to use CBR values less than 3%, because from both a technical and economic perspective. It would normally be inappropriate to lay pavement on soils of such bearing capacity. Based on the suitability of sub-grade soil, the three soil samples were classified as poor and not allowed to use for sub-grade materials. The CBR swell value also indicated that the soils have a critical (very high) degree of expansion. Therefore, the soils need to be treated before use. The test results were shown in Appendix A-7

Table 4.8: CBR and CBR Swell result for untreated Soil samples

Samples	CBR (%) at 95 (%) MDD	CBR swell (%)	ERA (2013) requirement for subgrade materials (CBR value)
SS1	1.54	8.08	Less than 3% and Failed for subgrade
SS2	1.03	10.07	Less than 3% and Failed for subgrade
SS3	1.74	7.35	Less than 3% and Failed for subgrade

4.2.11 Unconfined compressive strengths

It is one of strength property determination test and the result indicated was for undisturbed and remolded natural soil samples.

4.2.12 Undisturbed and Remolded Natural Soil Samples

Undisturbed soil samples were taken directly from site, disturbed samples were remolded at MDD, and OMC were determined from compaction test. For undisturbed site samples, the stress value was taken a minimum value because every design from site must be designed and considered for minimum strength. But for remolded samples the average value was considered because the value difference may be happened from at the time of remolding, from materials and other factors. Sensitivity terms the effect of soil disturbance on shear strength. Sensitivity of soil is a sign of a decrease in shear strength when the soil has been disturbed in any way, in this case when it was remolded and subjected to cyclic loading. Sensitivity of sample one was 0.875, sample two was

0.92 and Sample three was 0.78. The findings show that the three soil Samples were insensitive or normal sensitivity; where the values were less than two.

As it was described in previous section the strength (consistency) of soils according to (Bowels J. 1988), if the value of Qu (KPa) from 0-25 kpa is said very soft soil, 25- 50 kpa is called soft soil, and 50-100 kpa is medium stiff or firm soil and so forth. Depending on those grouping the strength (consistency) of three soil samples were firm soils, which means thumb makes impression to deform or intended only by strong finger pressure.

Table 4.9 shows the laboratory test results of UCS of undisturbed and remolded natural soil samples of three pits and the additional tabular test data were included in Appendix A-8.

Table 4.9: Unconfined compression strength of natural soil samples

Samples	Sample type (Immediate)	Avg. UCS(kpa	Shear strength(kpa)	Consideration for design purpose	Strength(Consistency)	Sensitivity
SS1	Undisturbed	73.36	36.68	The least value was taken into consideration.	Firm soil	0.875
	Remolded	83.83	41.92	The average value was taken	Firm soil	
SS2	Undisturbed	68.99	34.50	The least value was taken into consideration.	Firm soil	0.92
	Remolded	74.80	37.41	The average value was taken	Firm soil	
SS3	Undisturbed	76	38	The least value was taken into consideration.	Firm soil	0.78
	Remolded	97.91	48.96	The average value was taken	Firm soil	

4.3. Soil Sample Taken for Treatment

From three pits of soil samples, by comparing their properties resulted from laboratory with each other and moreover by its CBR value, the weakest soil sample was selected for stabilization; because this study focused on investigating the combined effects of materials on expansive subgrade soil strength and whether they improve soil strength or not rather than stabilizing different and more soil samples. From three soil samples, SS2 was the weakest soil and the stabilization was held on SS2 with CDW and JGT fiber combination for its strength characteristics.

Table 4.10: Summary of engineering Properties of natural subgrade soils and the weakest soil identified for stabilization

Comparison factors		SS1	SS2	SS3	The most weakest SS
Particle sizes		Clay=54.6	Clay=68.4	Clay=65.8	SS2
Soil classifications	AASHTO	A-7-5 (54)	A-7-5 (67)	A-7-5 (53)	SS2
	USCS	CH, Highly plastic inorganic clay	CH, Highly plastic inorganic clay	CH, Highly plastic inorganic clay	
Natural moisture contents (%)		40.01	43.01	42.28	SS2
Free swells (%)		89	95	91	SS2
Atterberg limits (%)	LL	83.02	97.51	81.09	SS2
	PI	47.17	56.37	45.34	
Compaction characters	MDD (g/cc)	1.462	1.30	1.39	SS2
	OMC (%)	22.401	30.05	20.05	
CBR (%)		1.54	1.03	1.77	SS2
Expansiveness from formula		68.91	89.76	66.08	SS2
UCS (KPa)	Undisturbed	73.36	68.99	76	
	Remolded	83.83	74.82	97.91	SS2

4.4 Effect of Ceramic dust waste and Jute Geotextile Fiber mix on compaction

The study conducted by combining various percentage of CDW(10%,15%and 20%) fixed with varying percentage of JGT fiber with dry soil by matrix combination. It was presented and the result indicates that the OMC and MDD in each combination shows decrement and increment from natural soil respectively. Previous study showed CDWP only used for expansive subgrade soil stabilization, OMC decreased and MDD increased at 30%added to soil [53,57],and jute fiber has moisture absorption properties, for this reason, soil stabilized with only jute fiber the OMC increased and MDD decreased [63,64].But in this combination the OMC and MDD of soil stabilized with CDW and JGT by different mix ratio was presented in Figure 4.4, and the value of laboratory data analysis attached in Appendix (C-1).

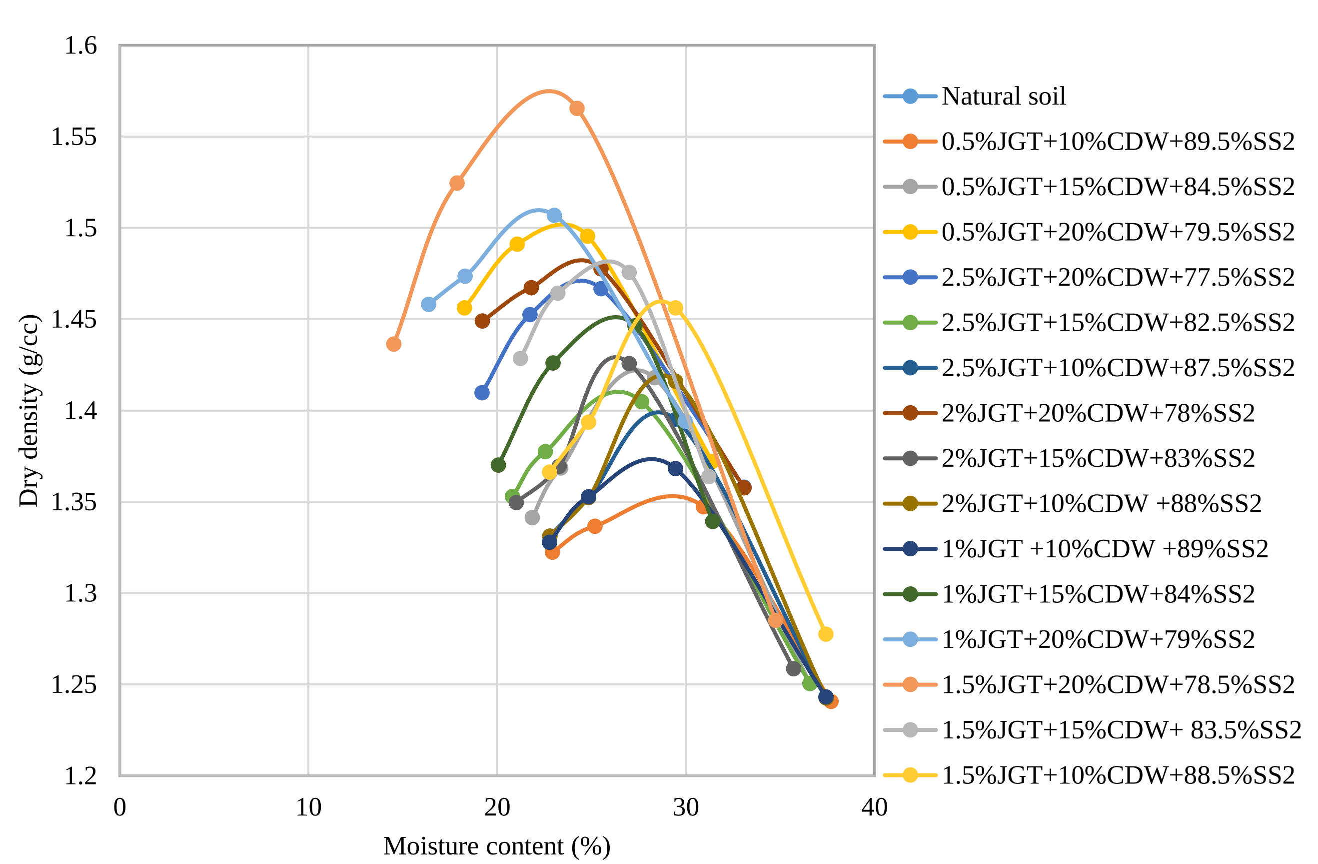


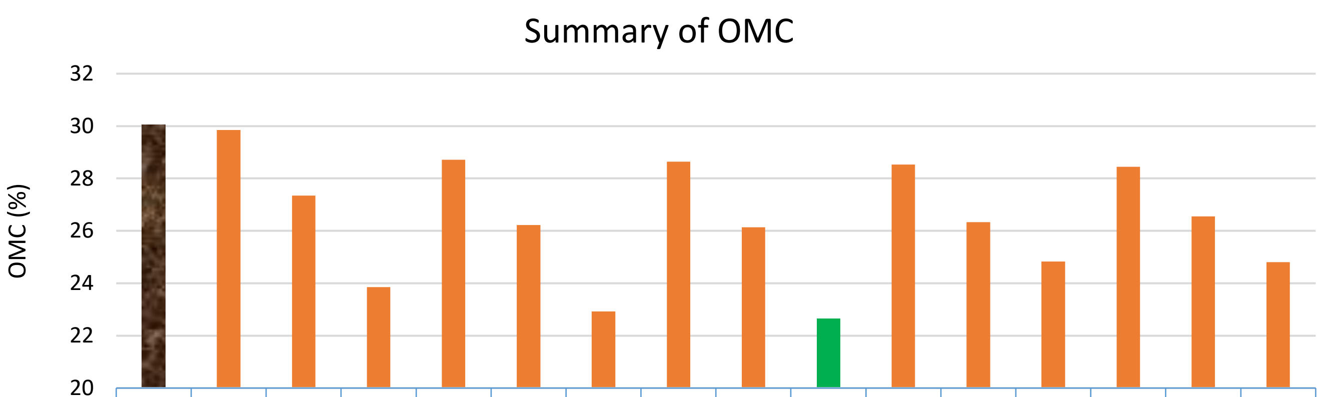
Figure 4.4: Moisture content and dry density of Soil, CDW and JGTF mix with different ratios

From Figure 4.5: it was observed that for fixed percentage of JGTF with an increased percentage of CDW with in each combination, the OMC of soil was slightly declined when compared with the result of natural soil and when comparing the results of stabilized soil by different combinations with each other's, the value shown fluctuated decrement and the optimum decrement for OMC gained at combination of 1.5%JGT+20%CDW with soil and the value reduced from 30.05% to 22.64% when comparing with other combination. In general, this decrement in OMC is because

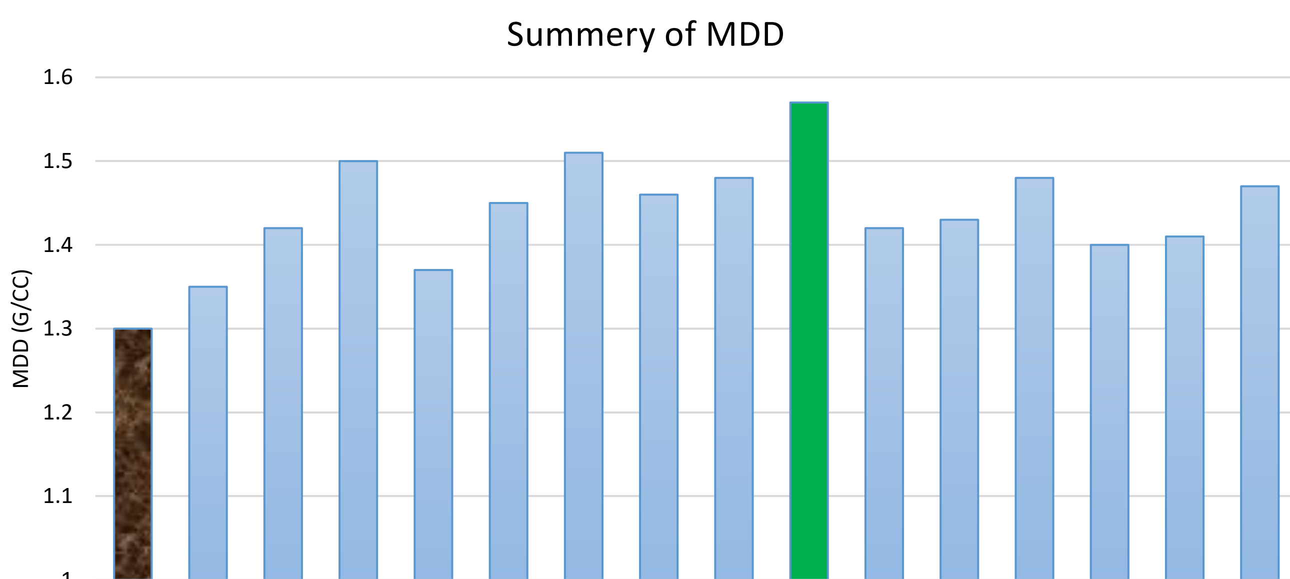
- ✓ Water-absorbing properties of JGTF less than that of natural Jute fiber and the additive or CDW can balanced water absorbing capacity of fiber and clay/expansive soil and reduce plasticity of soil by its self-cementitious properties i.e, high Pozzolanas that materials possessing alumina (Al_2O_3), silica (SiO_2), and ferrite (Fe_2O_3) greater than 70% as ASTM C-618 mentioned and this Pozzolanic constituents produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH).
- ✓ The calcium silicate gel formed initially coats and binds lumps of clay together. The gel then crystallizes to form an interlocking structure, which increases the soil strength.
- ✓ Ceramic dust powder is denser than the soil. The denser the more preventing permeability of water.
- ✓ Quick absorption of added water by waste ceramic powder. This indicates that less water is required to compact the soil when using a JGTF mixture with waste ceramic powder. Ceramic dust powder effectively dries wet soils and provides an initial rapid strength gain.in this cause OMC decrease.

The combined effect of CDW and JGTF on the MDD of Soil as shown in the Figure 4.5, the MDD shown increment from natural soil in each combination and value shown fluctuated increment in each combination when compared with each other's. From combinations optimum stabilized combination was 1.5%JGT+20%CDW with soil and it shows significant increment (from 1.3g/cc to 1.57 g/cc) when comparing with natural and different combinations. In general, this increment in each combination from natural soil was because of-

- ✓ The addition of CDW absorb water amount of the soil as its pozzolanic properties by reducing soil plasticity.
- ✓ The specific gravity of CDW were denser than that of soil sample and the specific gravity of fiber was made positive effect. Where the specific gravity of JGTF was 1.5 [45], CDW was 2.81 and soil was 2.67.
- ✓ Moisture absorbing capacity in jute geotextile fiber helps to increase its fractional properties and when adding the additive with cementitious properties, as result dry density increase [53].



	Natural soil	0.5% JGT+ 10% CDW+SS2	0.5% JGT+ 15% CDW+SS2	0.5% JGT+ 20% CDW+SS2	1%JGT +10 %CDW +SS2	1%JGT+ 15% CDW+SS2	1%JGT+ 20% CDW+SS2	1.5% JGT+ 10% CDW+SS2	1.5% JGT+ 15% CDW+ SS2	1.5% JGT+ 20% CDW+SS2	2%JGT+ 10% CDW +SS2	2%JGT+ 15% CDW+SS2	2%JGT+ 20% CDW+SS2	2.5% JGT+ 10% CDW+SS2	2.5% JGT+ 15% CDW+SS2	2.5% JGT+ 20% CDW+SS2
OMC	30.05	29.85	27.35	23.85	28.72	26.22	22.92	28.64	26.14	22.64	28.53	26.33	24.83	28.45	26.55	24.81

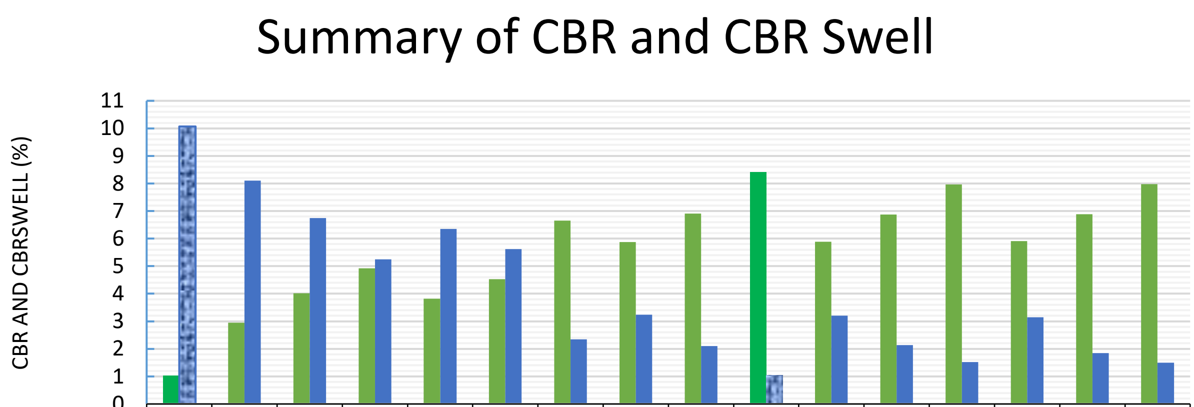


	Natural soil	0.5%JGT+10%CDW+SS2	0.5%JGT+15%CDW+SS2	0.5%JGT+20%CDW+SS2	1%JGT+10%CDW+SS2	1%JGT+15%CDW+SS2	1%JGT+20%CDW+SS2	1.5%JGT+10%CDW+SS2	1.5%JGT+15%CDW+ SS2	1.5%JGT+20%CDW+SS2	2%JGT+10%CDW+SS2	2%JGT+15%CDW+SS2	2%JGT+20%CDW+SS2	2.5%JGT+10%CDW+SS2	2.5%JGT+15%CDW+SS2	2.5%JGT+20%CDW+SS2
MDD	1.3	1.35	1.42	1.5	1.37	1.45	1.51	1.46	1.48	1.57	1.42	1.43	1.48	1.4	1.41	1.47

Figure 4.5: Effect of CDW and JGTF mix on optimum moisture content and maximum dry density of Soil.

4.5 Effect of Ceramic dust waste and Jute Geotextile Fiber mix on CBR and CBR Swell

CBR and CBR-swell tests were conducted and by the soil samples replaced with varying percentage of CDW, and JGTF Combination in weight of dry soil. Three point CBR and CBR swell tests were conducted for each of the combination and the results of the soaked CBR test was shown in Figure 4.6.Detail of laboratory test result was included in Appendix C-2.



	Natural soil	0.5%JGT+10%CDW+SS2	0.5%JGT+15%CDW+SS2	0.5%JGT+20%CDW+SS2	1%JGT+10%CDW+SS2	1%JGT+15%CDW+SS2	1%JGT+20%CDW+SS2	1.5%JGT+10%CDW+SS2	1.5%JGT+15%CDW+SS2	1.5%JGT+20%CDW+SS2	2%JGT+10%CDW+SS2	2%JGT+15%CDW+SS2	2%JGT+20%CDW+SS2	2.5%JGT+10%CDW+SS2	2.5%JGT+15%CDW+SS2	2.5%JGT+20%CDW+SS2
CBR	1.03	2.95	4.02	4.92	3.82	4.53	6.65	5.87	6.91	8.4	5.89	6.87	7.96	5.91	6.89	7.98
CBR Swell	10.07	8.11	6.75	5.25	6.35	5.62	2.34	3.24	2.1	1.04	3.21	2.14	1.52	3.15	1.85	1.5

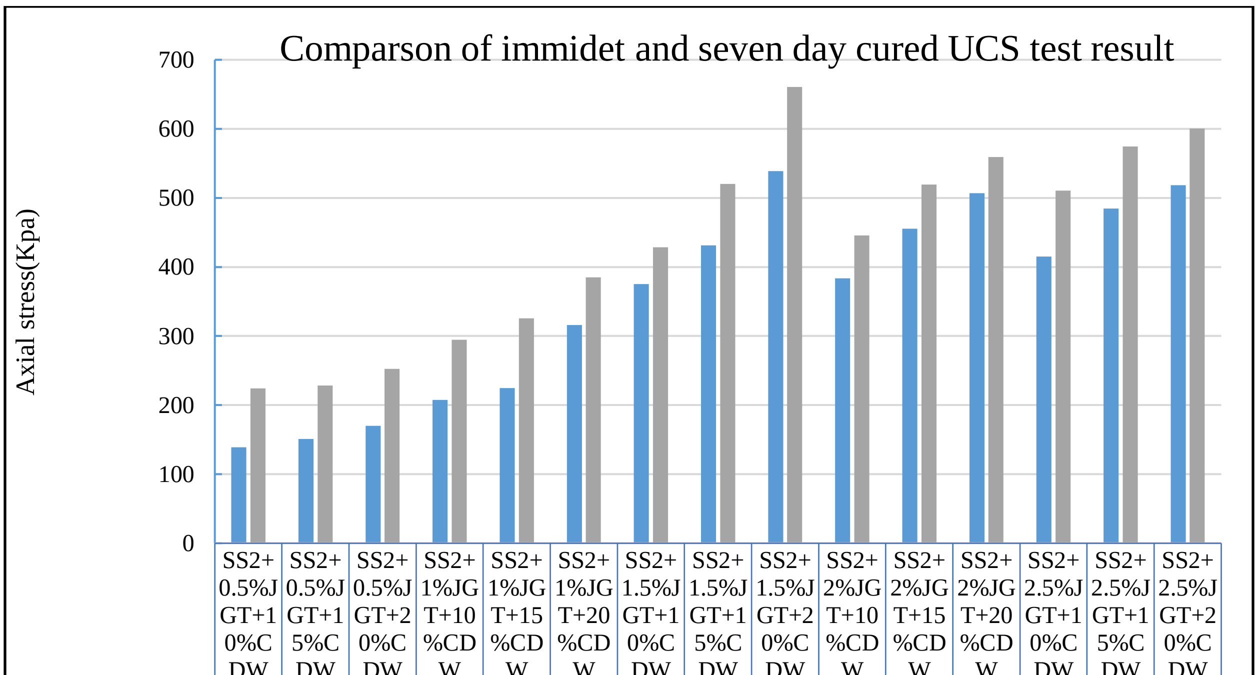
Figure 4.6: effect of CDWP and JGTF mix on CBR and CBR Swell

From laboratory result shown in Figure 4.6, it was summarized that, the CBR values were increased in each combination from 1.03% to the value represented for individual combination and improvement changed subgrade strength class almost from minimum requirement to satisfied requirement in each combination. When comparing the CBR value of different combination with each other and, 78.5%SS2+1.5%JGT+20%CDW combination shows greater or considerable increments on CBR value from 1.03% to 8.4%. This indicates that soil after treated with waste ceramic powder, jute geotextile fiber mix, the soil’s CBR value was significantly enhanced, and CBR swell value decreased from 10.07% to 1.04% for maximum CBR. As the outcome, the expansive subgrade soil treated with waste ceramic powder and jute geotextile fiber satisfies the requirements established in the Ethiopian Roads Authority pavement design manuals, ERA (2013) specification for materials suitable for use as subgrade material of not less than 3% of CBR determined at MDD and OMC as well as CBR swell value not greater than 2%. In general, when

1.5%JGT and 20%CDWP combined, it highly improved and the maximum CBR value was gained. The reason for this improvement is due to fiber and the pozzolanic reactions of waste ceramic powder with soil and also the amount of stabilizer have direct proportional relation at 95% of MDD and a decrease in CBR-Swell also due to cation exchange, flocculation, and agglomeration of the soil particles and the replacement of some of the volume that is previously occupied by expansive clay minerals by waste ceramic dust powder and then it decreased the soil plasticity.

4.6 Effect of Ceramic dust waste and Jute Geotextile Fiber mix on UCS

Tests on remolded specimens treated with various combinations of CDW and JGTF by mass of dry soil were conducted after mixing and compacting the samples at the MDD and OMC of the compaction test. Both immediate and seven day cured UCS tests were conducted for samples of soil mixed with (10%, 15%and 20%) of CDW with varying percentage of JGTF and Samples were extruded from the compaction mold by Shelby tube or cylindrical soil sampler. For cured samples, the specimen was extruded and sealed with impermeable plastic bag to be cured at room temperature for seven days. The summarized UCS test results were shown in figure 4.7 for both immediate and cured tests and detailed UCS laboratory results was presented in Appendix C-3.



	SS2+0.5%JGT+10%CDW	SS2+0.5%JGT+15%CDW	SS2+0.5%JGT+20%CDW	SS2+1%JGT+10%CDW	SS2+1%JGT+15%CDW	SS2+1%JGT+20%CDW	SS2+1.5%JGT+10%CDW	SS2+1.5%JGT+15%CDW	SS2+1.5%JGT+20%CDW	SS2+2%JGT+10%CDW	SS2+2%JGT+15%CDW	SS2+2%JGT+20%CDW	SS2+2.5%JGT+10%CDW	SS2+2.5%JGT+15%CDW	SS2+2.5%JGT+20%CDW
qu (immidet)	138.83	150.88	169.93	207.48	224.81	316.09	375.09	431.52	538.88	383.6	455.5	506.67	414.91	484.47	518.27
qu(7day cured)	224.11	228.21	252.48	294.75	325.52	384.91	428.43	520.17	660.65	445.9	519.28	559.14	510.48	574.27	600.34

Figure 4.7: Comparison of immediate and cured UCS test result

The UCS of soil samples with different percentage of fibers were calculated from the loads at failure and shown in Figure 4.7 and it was presented that, the improvement in UCS values for immediate and cured sample specimen that was stabilized with soil, CDW and JGTF for different combination.

When, compering the value of different combinations by each other the optimum UCS value of soil stabilized with CDW and JGTF for immediate and cured for seven days were at combination of 1.5%JGT and 20%%CDW and its value was 538.88 kpa and 660.65 kpa respectively. In generally stress increased from 74.8KPa to 538.88KPa (immediate test) and from to 660.65KPa for seven days cured test and it was hard. Therefore, the optimum UCS value of soil stabilized with CDW and JGTF for immediate and seven days cured was 1.5%JGTF and 20%CDW. The increase in the strength from untreated soil due to the shear transfer mechanism induced by the fiber inclusions and additive with pozzolanic properties.

4.7. Comparison of the effects of the stabilization results with standard specification

Table 4.11: CBR value Comparison for stabilized soil with standard specification

Combinations	Laboratory Results of CBR in %	ERA (2013) subgrade class	Bowles and ERA subgrade material quality standard
Natural soil	1.03	S1	**Very Poor** and blow requirement
0.5%JGTF+10%CDW+SS2	2.95	S1	186%increased but **Very Poor** and blow requirement
0.5%JGTF+15%CDW+SS2	4.02	S2	290%increased and **Poor** but satisfied requirement
0.5%JGTF+20%CDW+SS2	4.92	S2	377.7%increased and **poor** but satisfied requirement
1%JGTF+10%CDW+SS2	3.82	S2	270.87%increased and **poor** but satisfied requirement
1%JGTF+15%CDW+SS2	4.53	S2	339.8%increased and **poor** but satisfied requirement
1%JGTF+20%CDW+SS2	6.65	S3	545.63%increased,**poor to fair** and satisfied requirement
1.5%JGTF+10%CDW+SS2	5.87	S3	469.9%increased,**poor to fair** and satisfied requirement
1.5%JGTF+15%CDW+SS2	6.91	S3	570.87%increased,**poor to fair** and satisfied requirement
1.5%JGTF+20%CDW+SS2	8.4	S4	715.53%increased, **fair** and satisfied requirement
2%JGTF+10%CDW+SS2	5.89	S3	555.19%increased,**poor to fair** and satisfied requirement
2%JGTF+15%CDW+SS2	6.87	S3	566.99%increased,**poor to fair** and satisfied requirement

2%JGTF+20%CDW+SS2	7.96	S3	672.82%increased,**poor to fair** and satisfied requirement
2.5%JGTF+10%CDW+SS2	5.91	S3	473.78%increased,**poor to fair** and satisfied requirement
2.5%JGTF+15%CDW+SS2	6.89	S3	568.93%increased,**poor to fair** and satisfied requirement
2.5%JGTF+20%CDW+SS2	7.98	S3	674.76%increased,**poor to fair** and satisfied requirement

Except the combination of 0.5% JGTF + 10% CDW + SS2, all another combination satisfied the requirement of ERA subgrade material quality standard, i.e. the CBR value of material greater than 3%. Additionally, the result agreed with Table 2.11 of the literature review states that [35, 36] the quality of the CBR subgrade improved from Poor but satisfied requirement of subgrade to fair subgrade and in subgrade class improved from S1 to S4. For natural soil to optimum value gained combination it was improved CBR value from 1.03% to 8.4%.In the combination, the value for CBR Swell decreased from 10.07% to 1.04% compering different combinations with natural soil.

Table 4.12: UCS value Comparison for stabilized soil with standard specification

Combinations	Laboratory Results of Avg. UCS value in (KPa)		ASTM/AASHTO Specification for consistency or strength	
	For immediate test	For cured test	For immediate test	For cured test
Natural soil	74.80	-	Firm soil	-
0.5%JGTF+10%CDW+SS2	138.83	224.11	Stiff	Very Stiff
0.5%JGTF+15%CDW+SS2	150.88	228.21	Stiff	Very Stiff
0.5%JGTF+20%CDW+SS2	169.93	252.48	Stiff	Very Stiff
1%JGTF+10%CDW+SS2	207.48	294.75	Very Stiff	Very Stiff
1%JGTF+15%CDW+SS2	224.81	325.52	Very Stiff	Very Stiff
1%JGTF+20%CDW+SS2	316.09	384.91	Very Stiff	Very Stiff
1.5%JGTF+10%CDW+SS2	375.09	428.43	Very Stiff	hard
1.5%JGTF+15%CDW+SS2	431.52	520.17	hard	hard
1.5%JGTF+20%CDW+SS2	538.88	660.65	hard	hard
2%JGTF+10%CDW+SS2	383.60	445.90	Very Stiff	hard
2%JGTF+15%CDW+SS2	455.50	519.28	hard	hard
2%JGTF+20%CDW+SS2	506.67	559.14	hard	hard
2.5%JGTF+10%CDW+SS2	414.91	510.48	hard	hard
2.5%JGTF+15%CDW+SS2	484.47	574.27	hard	hard
2.5%JGTF+20%CDW+SS2	518.27	600.34	hard	hard

As it was discussed, in literature review section table 2.12, quality of subgrade based on UCS value stated [36, 37, 38], the strength of soils from 0-25 KPa is said very soft soil, 25- 50 KPa is called soft soil, 50- 100 KPa is medium stiff or firm soil, 100-200 KPa is stiff soil, 200-400 KPa is very stiff soil and 400-1000 KPa is hard soil. Therefore, the consistencies of samples in different combination shows stiff to hard, in immediate and Very stiff to hard in seven days cured test and agreed with ASTM Specification for consistency or strength.At the combination of 1.5%JGTF+20%CDW+SS2 the optimum numerical value of UCS was gained when compared with others.

CHAPTER FIVE

CONCLUSION AND RECOMMANDATIONS

5.1 Conclusion

This research was done on the stabilization of the selected expansive subgrade soil in Jimma City with locally available ceramic dust waste and Jute Geotextile fiber material combination with the help of a series of laboratory investigations. The following conclusions and recommendations were reached.

The laboratory results have shown that the expansive subgrade soils selected and taken for this investigation were A-7-5 (54) for soil sample one, A-7-5(67) for sample two and A-7-5(53) for sample three as per the AASHTO soil classification and CH as USCS for 3 Soil samples. From the grain size determination test , the percentage of passing through sieve No. 200 (0.075mm sieve size) of samples were greater than 35% and soils are fine grained and weak for subgrade materials.

From Consistency properties test results shown that; for soil samples have greater than 70% liquid limit and very high swelling potential, degree of Soil expansiveness from the formula greater than 50%, and soils are high expansive, free swell index greater than 50% and the soil samples have very high swelling potential. The compaction test indicates that the MDD and OMC of soil sample one was 1.462 g/cc and 22.401% respectively, MDD and OMC of soil sample two was 1.3 g/cc and 30.05% respectively and MDD was 1.39g/cc and OMC 20.05% for sample three.

CBR and CBR swell of sample one was 1.54% and 8.08% respectively, CBR and CBR swell of sample two was 1.03% and 10.07% respectively, 1.77%, and 7.35% for sample three and based on ERA (2013) subgrade material CBR value requirement the three soil samples were failed and need improvement. From the Unconfined compression strength test, as result shown that; sensitivity were checked and normal or insensitive, the soil consistency was founded as firm soil, and they were failed to accept as subgrade soil for road pavement construction.

From the three Soil samples, soil sample two was the weakest soil and stabilized with varying percentage of ceramic dust waste and Jute Geotextile fiber mix. A series of tests were performed to study the effects of ceramic dust waste and Jute Geotextile fiber mix on the strength properties of selected subgrade soil samples and the method used was the matrix combination method i.e the varying percentage of waste Ceramic dust powder and Jute geotextile fiber combined one for all each other.

The optimum moisture content decreased and fluctuated while the maximum dry density increased and fluctuated from natural to stabilized soil in each combination. When comparing from all combinations, 1.5%JGTF+20%CDWP with dry soil was optimum for tests i.e (OMC decreased from 30.05% to 22.64% and MDD increased from 1.3g/cc to 1.57g/cc). CBR values were increased from natural soil and increments in value were fluctuated in each combination from 1.03% to the value indicating subgrade strength class almost from the minimum requirement to satisfied requirement in each combination. When comparing the CBR value of different combinations with each other and, 1.5%JGT+20%CDW with dry soil, the combination shows greater or considerable increments in CBR value from 1.03% to 8.4%. Therefore, the 1.5%JGTF and 20% CDW mix was improved expansive soil from S1 to S4, in the maximum value gained combination and the value for CBR Swell decreased from 10.07% to 1.04% natural to stabilized soil.

The unconfined compressive strength (UCS) test conducted and the result shows in each combination, there were significant increments in stress value for both immediate and cured seven days from natural soil. When comparing the value of different combinations with each other the optimum UCS value of soil stabilized with CDW and JGTF immediate test and cured for seven days were a combination of 1.5%JGT and 20%%CDW was 538.88 kpa and 660.65 kpa respectively. In generally, strength increased from 74.8KPa to 538.88KPa (immediate test) and to 660.65KPa for seven days cured test.

Finally, from investigations, it was concluded that the results from this study satisfied ERA and ASTM standard specifications for subgrade materials and Expansive soil treated with 20% of ceramic dust waste and 1.5% of Jute geotextile fiber mix by weight of dry soil is strengthening expansive subgrade soil. The utilization of ceramic waste for soil stabilization helps to increase Environmental aesthetics in addition to economic stabilization than that of using traditional stabilizations.

5.2 Recommendation

Based on the findings of this research, the following recommendations are forwarded:

- It is recommended to use ceramic dust waste and Jute geotextile fiber as soil stabilizer at the indicated percentages for the effective construction of the highway and other road whom facing weak subgrade at the time of construction.
- Jute geotextile fibers investigated in this study is can be used as a stabilizer of expansive sub grade soil in combination with Ceramic dust waste. Therefore, concerned bodies like, Ethiopian Road Authority should be made aware about this potential soil stabilizing materials and promote its standardized production and usage manual.
- Further studies should be carried out in order to identify the long-term effects that Ceramic dust waste and Jute geotextile fibers have on the durability (including decaying properties for JGTF) under the road pavement structures.
- This study was conducted by taking limited parameters and Additional test parameters like PH value test, and mineralogical tests should be performed to have more accurate test results and economic analysis of materials with other mechanical stabilizations to know the best economic stabilization.
- Curing time for this study was conducted for seven days on UCS test. Additional curing time should be performed to check the further effects on the strength of expansive soil.

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APPENDIX

Appendix A: Laboratory Test Results of Untreated Soil Samples

Appendix A-1: Natural Moisture (Water) Content According to ASTM D-2216

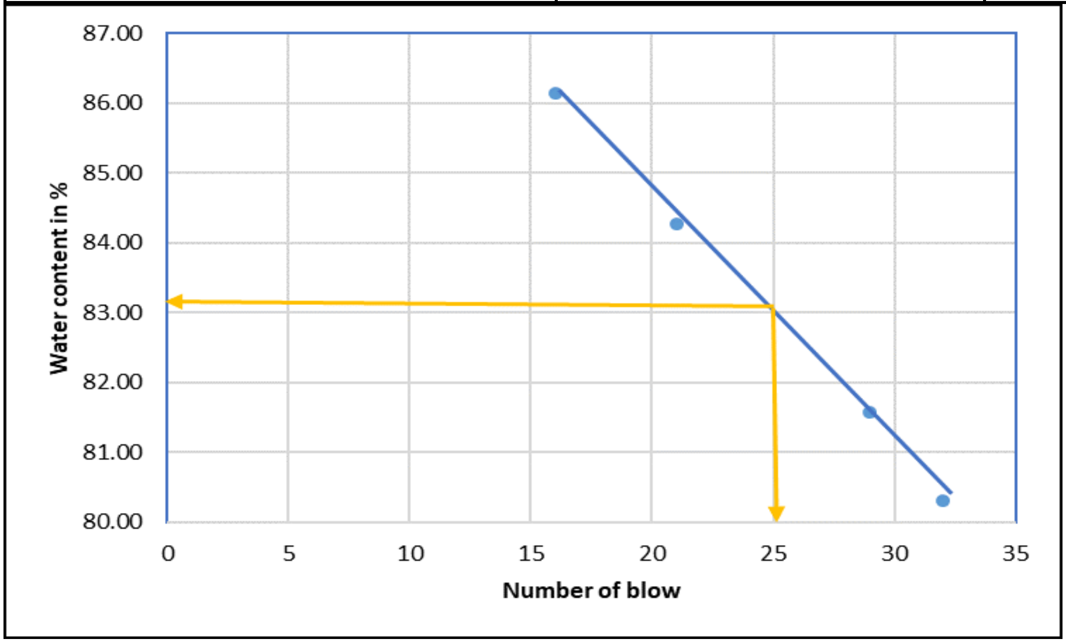
NATURAL MOISTURE CONTENT DETERMINATION									
ASTM D 2216 - Standard Test Method for Laboratory Determination of									
Water (Moisture) Content of natural Soil									
samples	Sample 01 (Bossa Addis kebele)			Sample 02 (Bacho Bore)			Sample 03 (Katholic school)		
can code	C1	F	G3	C8	B22	A29	K	E	A5
Wt. of Container +lid (g)	53.39	44.85	54.47	54.19	45.65	55.27	48.2	51.3	56.2
Wt. of container + wet soil (g)	167.49	153.82	162.18	169.29	154.92	162.88	158.25	162.32	147.86
Wt. of container + dry soil (g)	134.12	123.35	134.05	134.95	122.85	129.5	131.25	125.15	119.85
Wt. of water (g)	33.37	30.47	28.13	34.34	32.07	33.38	27	37.17	28.01
Wt. of dry soil (g)	79.13	76.7	73.78	80.76	77.2	74.23	83.05	73.85	63.65
Moisture container (%)	42.17	39.73	38.13	42.52	41.54	44.97	32.51	50.33	44.01
Average	**40.01**			**43.01**			**42.28**		

Appendix A-2: Atterberg Limits Data Analysis of Natural Soils (According to ASTM D-4318

For Natural soil sample 1

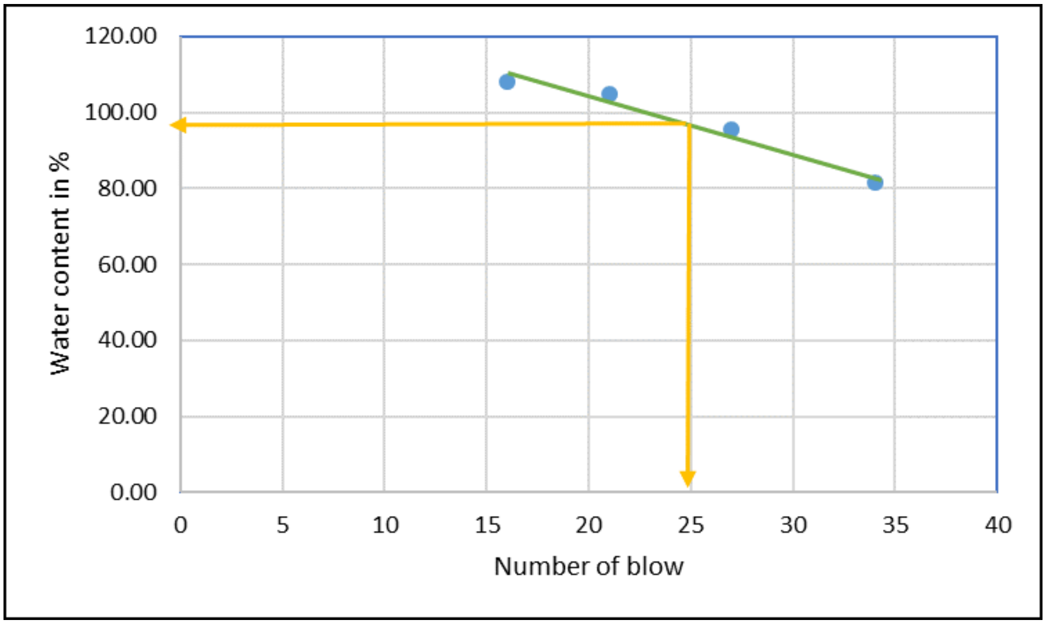
DETERMINATION OF LIQUID LIMIT & PLASTIC LIMIT OF SOIL						
TEST METHOD：ASTM D 4318						
Liquid limit and plastic limit test : natural soil (Sample 1)						
Determinations	Liquid Limit				Plastic Limit	
Number of blows	32	29	21	16		
Test No	1	2	3	4	1	2
Container Code	A5	T2D	3E	T3	NC32	B11
Wt. of container + wet soil (g)	42.62	41.84	40.73	41.95	24.42	25.08
Wt. of container + dry soil (g)	31.61	30.95	30.65	30.45	22.57	22.96
Wt. of container (g)	17.9	17.6	18.69	17.1	17.45	17

Wt. of water (g)	11.01	10.89	10.08	11.5	1.85	2.12
Wt. of dry soil (g)	13.71	13.35	11.96	13.35	5.12	5.96
Moisture content (%)	80.31	81.57	84.28	86.14	36.13	35.57
LL	**83.02**				Average	35.85



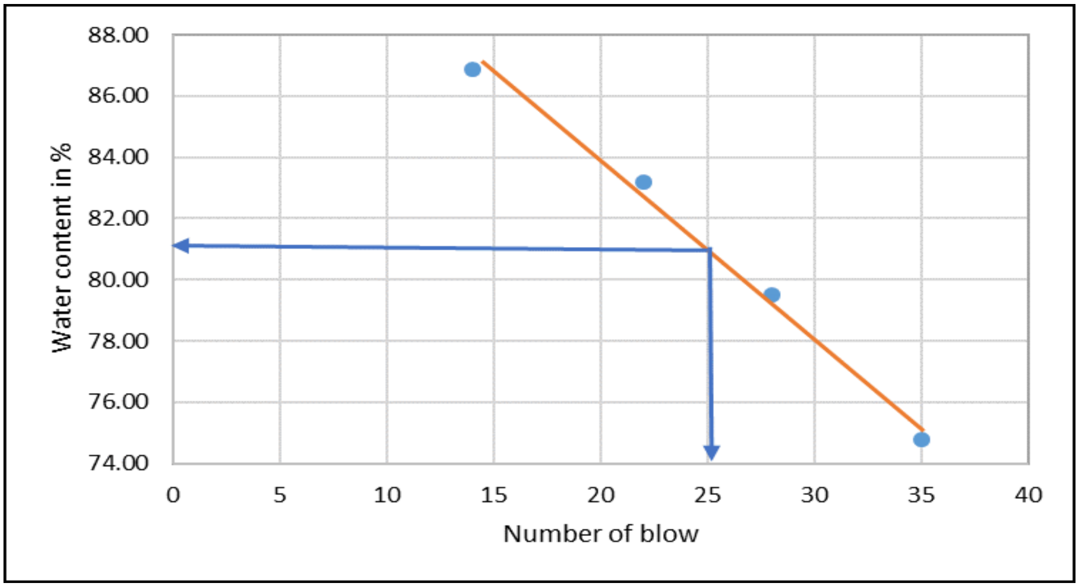
For Natural soil sample 2

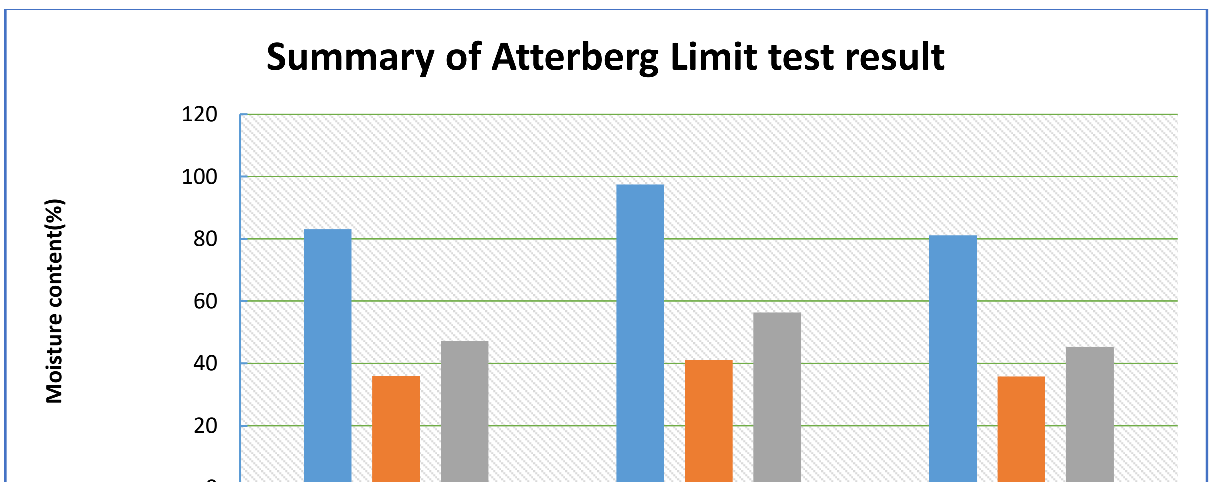
DETERMINATION OF LIQUID LIMIT & PLASTIC LIMIT OF SOIL						
TEST METHOD：ASTM D 4318						
Liquid limit and plastic limit test : natural soil (Sample 02)						
Determinations	Liquid Limit				Plastic Limit	
Number of blows	34	27	21	16		
Test No	1	2	3	4	1	2
Container Code	P15	LB	B12	C1	NC	L32
Wt. of container + wet soil (g)	45.29	47.54	43.63	44.81	25.12	25.08
Wt. of container + dry soil (g)	32.96	32.95	30.87	30.45	22.79	22.97
Wt. of container (g)	17.85	17.7	18.71	17.15	17.46	17.5
Wt. of water (g)	12.33	14.59	12.76	14.36	2.33	2.11
Wt. of dry soil (g)	15.11	15.25	12.16	13.3	5.33	5.47
Moisture content (%)	81.60	95.67	104.93	107.97	43.71	38.57
LL	**97.51**				Average	41.14



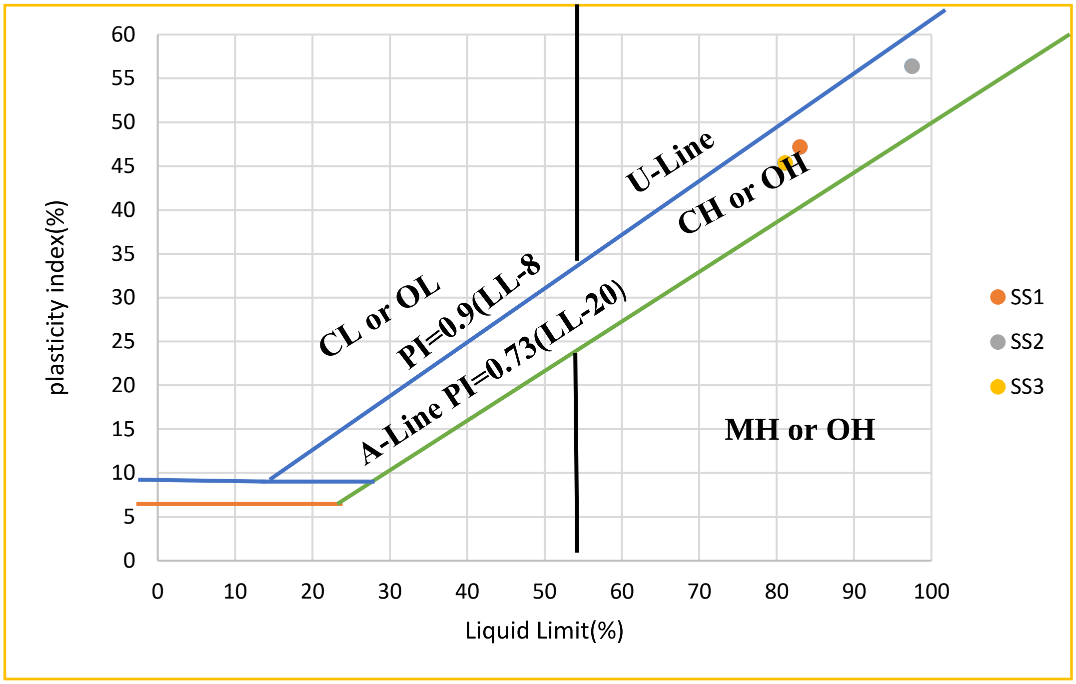
For Natural soil sample 3

DETERMINATION OF LIQUID LIMIT & PLASTIC LIMIT OF SOIL						
TEST METHOD：ASTM D 4318						
Liquid limit and plastic limit test : natural soil (Sample 03)						
Determinations	Liquid Limit				Plastic Limit	
Number of blows	35	28	22	14		
Test No	1	2	3	4	1	2
Container Code	1A	T3	4E	K	A5	C1
Wt. of container + wet soil (g)	39.89	41.54	38.93	41.7	24.31	25.06
Wt. of container + dry soil (g)	30.31	30.98	29.73	30.45	22.51	22.93
Wt. of container (g)	17.5	17.7	18.67	17.5	17.45	17
Wt. of water (g)	9.58	10.56	9.2	11.25	1.8	2.13
Wt. of dry soil (g)	12.81	13.28	11.06	12.95	5.06	5.93
Moisture content (%)	74.79	79.52	83.18	86.87	35.57	35.92
LL	**81.09**				Average	35.75





	Untreated SS1	Untreated SS2	Untreated SS3
Liquid Limit (%)	83.02	97.51	81.09
Plastic Limit (%)	35.85	41.14	35.75
Plasticty Index (%)	47.17	56.37	45.34



Appendix A-3: Specific Gravity Test Analysis (According to ASTM D-854)

Specific gravity test data and result for natural soil 1

Test pit Lacotion	Bossa Kito Addis Ketema in Jimma		
Test Pit No	Sample 1		
Code of Pycnometer	V	15	4
Mass of Pycnometer	30.750	29.151	32.945
Mass of Pycnometer with Dry Soil	43.728	37.684	44.270
Mass of Pycnometer with Dry Soil and Water(W2)	134.170	130.042	135.362
Mass of Pycnometer and Water(W1)	126.037	124.699	128.252
Temperature of Pycnometer with Water (T^0_{ci})	21	21	21
Density of water at (T^0_{ci})	0.99802	0.99802	0.99802
Temperature of Pycnometer with Soil and Water (T^0_{cx})	22	22	22
Density of water at (T^0_{cx})	0.9978	0.9978	0.9978
Corrected Mass of Pycnometer and Water	126.009	124.672	128.224
Correction Factor, K at 20^0_c	0.9996	0.9996	0.9996
Specific Gravity, Gs at 20^0_c	2.693	2.697	2.704
Average Specific Gravity, Gs at 20^0_c			**2.70**

Specific gravity test data and result for natural soil 2

Test pit Lacotion	Bacho Bore in Jimma		
Test Pit No	Sample 2		
Code of Pycnometer	12	2	7
Mass of Pycnometer	28.642	30.628	30.834
Mass of Pycnometer with Dry Soil	38.085	41.879	39.812
Mass of Pycnometer with Dry Soil and Water(W2)	129.311	132.887	132.082
Mass of Pycnometer and Water(W1)	123.432	125.895	126.460
Temperature of Pycnometer with Water ($T^0{}_{ci}$)	23	23	23
Density of water at ($T^0{}_{ci}$)	0.99756	0.99756	0.99756
Temperature of Pycnometer with Soil and Water ($T^0{}_{cx}$)	24	24	24
Density of water at ($T^0{}_{cx}$)	0.99732	0.99732	0.99732
Corrected Mass of Pycnometer and Water	123.402	125.865	126.430
Correction Factor, K at $20^0{}_c$	0.9991	0.9991	0.9991
Specific Gravity, Gs at $20^0{}_c$	2.669	2.658	2.697
Average Specific Gravity, Gs at $20^0{}_c$	**2.67**		

Specific gravity test data and result for natural soil 3

Test pit Lacotion	Catholic school in Jimma Koch		
Test Pit No	Sample 3		
Code of Pycnometer	C	Z	3
Mass of Pycnometer	31.411	28.341	33.925
Mass of Pycnometer with Dry Soil	43.928	37.684	44.270
Mass of Pycnometer with Dry Soil and Water(W2)	133.900	130.540	134.732
Mass of Pycnometer and Water(W1)	126.037	124.699	128.252
Temperature of Pycnometer with Water ($T^0{}_{ci}$)	22	22	22
Density of water at ($T^0{}_{ci}$)	0.9978	0.9978	0.9978
Temperature of Pycnometer with Soil and Water ($T^0{}_{cx}$)	23	23	23
Density of water at ($T^0{}_{cx}$)	0.99756	0.99756	0.99756
Corrected Mass of Pycnometer and Water	126.007	124.669	128.221
Correction Factor, K at $20^0{}_c$	0.9996	0.9996	0.9996
Specific Gravity, Gs at $20^0{}_c$	2.71	2.69	2.70
Average Specific Gravity, Gs at $20^0{}_c$	**2.70**		

Appendix B: Laboratory Test result of Specific gravity for CDWP

Specific gravity test data of CDW			
Code of Pycnometer	1	8	7
Mass of Pycnometer	31.911	32.141	34.925
Mass of Pycnometer with Dry Soil	41.911	42.141	44.925
Mass of Pycnometer with Dry Soil and Water	133.451	135.115	135.672
Mass of Pycnometer and Water	127.037	128.699	129.252
Temperature of Pycnometer with Water (T^0_{ci})	23	24	24
Density of water at (T^0_{ci})	0.99756	0.99732	0.99732
Temperature of Pycnometer with Soil and Water (T^0_{cx})	24	25	25
Density of water at (T^0_{cx})	0.99732	0.99707	0.99707
Corrected Mass of Pycnometer and Water	127.006	128.667	129.220
Correction Factor, K at $20^0{}_c$	0.9991	0.9988	0.9988
Specific Gravity, Gs at $20^0{}_c$	2.8101	2.8121	2.8154
Average Specific Gravity, Gs at $20^0{}_c$	**2.81**		

Appendix A-4: Particle Size Analysis (According to ASTM D-422)

A) Soil sample 1: wet sieve and hydrometer analysis

I Wet Sieve Analysis SS1				
Total washed soil specimen mass, g				1000
Sieve Size	Mass of Retaing (g)	Percentage of Retaing	Commulative Percentage Retaining	Percentage of fine particles
9.50	0	0	0	100.00
4.75	7.224	0.7224	0.7224	99.28
2.00	15.325	1.5325	2.2549	97.75
0.850	10.061	1.0061	3.261	96.74
0.425	9.911	0.9911	4.2521	95.75
0.250	5.088	0.5088	4.7609	95.24
0.150	6.591	0.6591	5.42	94.58
0.075	4.503	0.4503	5.8703	94.13
Pan	941.297	94.1297	100	
Total	1000			

Wet Sieve Analysis SS2				
Total washed soil specimen mass, g				1000
Sieve Size	Mass of Retaing (g)	Percentage of Retaing	Commulative Percentage Retaining	Percentage of fine particles
9.50	0	0	0	100.00
4.75	4.724	0.4724	0.4724	99.53
2.00	11.55	1.155	1.6274	98.37
0.850	9.361	0.9361	2.5635	97.44
0.425	8.01	0.801	3.3645	96.64
0.250	4.085	0.4085	3.773	96.23
0.150	4.391	0.4391	4.2121	95.79
0.075	3.103	0.3103	4.5224	95.48
Pan	954.776	95.4776	100	
Total	1000			

Wet Sieve Analysis SS3				
Total washed soil specimen mass, g				1000
Sieve Size	Mass of Retaing (g)	Percentage of Retaing	Commulative Percentage Retaining	Percentage of fine particles
9.50	0	0	0	100.00
4.75	3.05	0.305	0.305	99.70
2.00	8.415	0.8415	1.1465	98.85
0.850	11.361	1.1361	2.2826	97.72
0.425	10.011	1.0011	3.2837	96.72
0.250	5.088	0.5088	3.7925	96.21
0.150	6.591	0.6591	4.4516	95.55
0.075	5.203	0.5203	4.9719	95.03
Pan	950.281	95.0281	100	
Total	1000			

Combined particle size distribution of SS1and SS2

Grain Size (mm)	% passing	Combined % passing	Grain Size (mm)	% passing	Combined % passing
9.50	100.00	100.00	9.50	100.00	100.00
4.75	99.28	***99.28***	***4.75***	99.53	***99.53***
2.00	97.75	***97.75***	***2.00***	98.37	***98.37***
0.850	96.74	96.74	0.850	97.44	97.44
0.425	95.75	95.75	0.425	96.64	96.64
0.250	95.24	95.24	0.250	96.23	96.23
0.150	94.58	94.58	0.150	95.79	95.79
0.075	94.13	***94.13***	***0.075***	95.48	***95.48***
0.039	91.37	86.42	0.038	94.97	90.72
0.028	89.40	84.55	0.027	92.28	88.82
0.020	85.44	80.81	0.019	89.30	86.92
0.015	76.24	72.11	0.014	87.35	85.02
0.011	75.53	71.43	0.010	85.39	83.11
0.008	71.57	67.69	0.007	85.01	81.21
0.006	67.61	63.95	0.005	81.03	77.41
0.004	65.64	62.08	0.004	77.65	74.17
0.003	*61.68*	*58.34*	*0.003*	*76.25*	*72.84*
0.002	57.73	54.60	0.002	71.67	68.47
0.001	55.77	52.75	0.001	67.10	64.09

Combined particle size distribution of SS3

Grain Size (mm)	% passing	Combined % passing
9.50	100.00	100.00
4.75	99.70	***99.70***
2.00	98.85	***98.85***
0.850	97.72	97.72
0.425	96.72	96.72
0.250	96.21	96.21
0.150	95.55	95.55
0.075	95.03	***95.03***
0.037	95.0	87.99
0.027	93.0	87.77
0.019	91.0	85.89
0.014	89.0	84.01
0.010	87.0	82.13

0.007	85.0	80.25
0.005	81.1	76.49
0.004	77.7	73.30
0.003	75.7	71.98
0.002	69.8	**65.78**
0.001	66.7	63.34

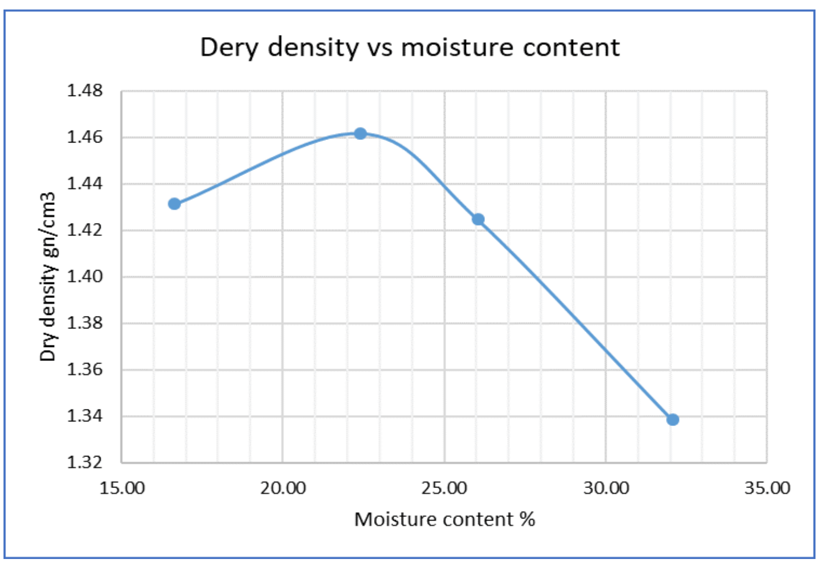
Appendix A-5: Free Swell Index of Natural Soil

Sample pits	Initial reading	Final reading	% free swell	Remarks
SS1	10	18.9	89	Very High
SS2	10	19.5	95	Very High
SS3	10	19.1	91	Very High

Appendix A-6: Compaction Test Data Analysis of Natural Soil (According to ASTM D-1557)

Untreated Natural Soil (Sample 1)								
Mass of sample (g)	4500							
Mass of Mold (g) B	2721							
Volume of Mold (cm^3) D	2124							
Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	6266		6521		6534.5		6475	
Mass of Wet Soil (g) A-B=C	3545.5		3800.5		3814		3754.5	
Bulk Density g/cm^3 C/D=(E)	1.67		1.79		1.80		1.77	
moisture determination								
Container code	RT	6	A5	B	C1	E	K	F
Mass of wet soil + container(g) (F)	61.072	53.795	99.11	84.38	59.13	71.4	95.512	72.369
Mass of dry soil + container (g) (G)	54.91	48.528	84.38	74.62	50.06	60.9	76.91	58.752
Mass of container (g) (H)	17.563	17.109	17.90	31.49	17.29	18.1	17.455	17.287
Mass of moisture (g) F-G=(I)	6.162	5.267	14.73	9.77	9.07	10.4	18.602	13.617
Mass of dry soil (g) G-H=(J)	37.347	31.419	66.48	43.12	32.77	42.8	59.455	41.465
Moisture content (%) (I/J)*100=K	16.499	16.764	22.16	22.65	27.69	24.4	31.288	32.840
Avg. moisture content % (L)	16.632		22.401		26.042		32.064	
Dry density g/cm^3 E/(100+L)*100	1.43		1.46		1.42		1.34	

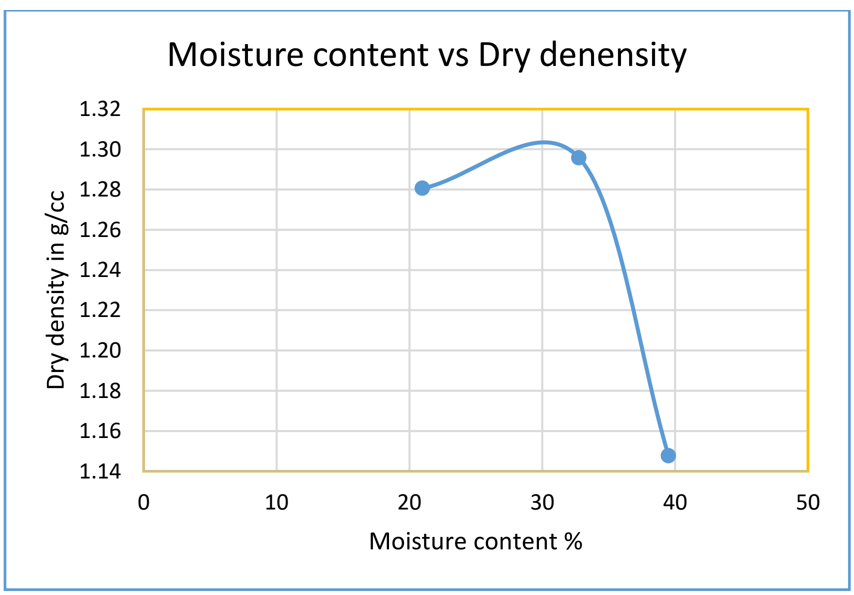
MDD (g/cc)	1.462
OMC (%)	22.401



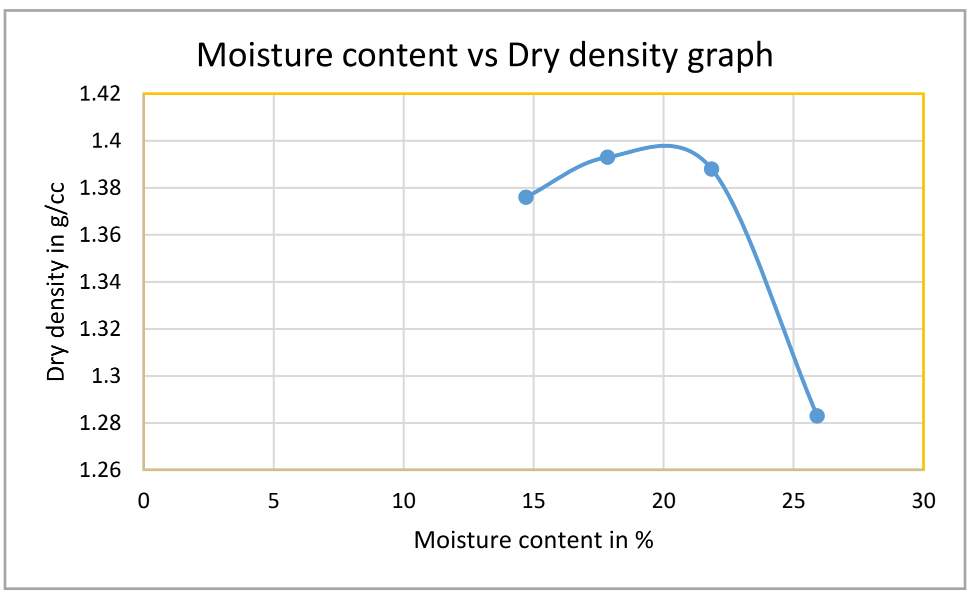
Untreated Natural Soil (Sample 02)	
Mass of sample (g)	4500
Mass of Mold (g) A	2721
Volume of Mold (cm^3) B	2124

Test No.	1		2	3		4	
Mass of mold + wet soil (g) C	6011.5		6117	6375		6122	
Mass of wet soil (g) C-A=D	3290.5		3396	3654		3401	
Bulk Density g/cm^3 D/B=(E)	1.549		1.599	1.720		1.601	
Moisture determination							
Container code	T1C1	H23	G3T3	3C	B8	11D	S1
Mass of wet soil + container (g) (F)	125.57	114.4	88.31	148.82	112	133.85	149.5
Mass of dry soil + container (g) (G)	106.42	97.94	76.54	116.44	89.02	99.22	116.7
Mass of container (g) (H)	17.5	17.5	18	20.5	16.7	17	27.5
Mass of moisture (g) F-G=(I)	19.15	16.42	11.77	32.38	22.98	34.63	32.89
Mass of dry soil (g) G-H=(J)	88.92	80.44	58.54	95.94	72.32	82.22	89.15
Moisture content % (I/J)*100=K	21.54	20.41	20.11	33.75	31.78	42.12	36.89
Avg. moisture content % (L)	20.97		24.50	32.76		39.51	
Dry density g/cm^3 E/(100+L)*100	1.281		1.284	1.296		1.148	

OMC (%)	30.05
MDD(g/cc)	1.3



Untreated Natural Soil (Sample 03)								
Mass of sample (g)	4500							
Mass of Mold (g) A	2720.5							
Volume of Mold (cm^3) B	2124							
Test No.	1		2		3		4	
Mass of mold + wet soil (g) C	6073.5		6208		6314		6152	
Mass of wet soil (g) C-A=D	3353		3487.5		3593.5		3431.5	
Bulk Density g/cm^3 D/B=(E)	1.579		1.642		1.692		1.616	
Moisture determination								
Container code	H-2	TC2	T2C1	HC11	V3	T5C1	C11	A5
Mass of wet soil + container (g) (F)	102.72	107.99	115.02	115.25	114.41	111.97	119.04	114.36
Mass of dry soil + container (g) (G)	92.04	96.16	98.44	100.45	95.75	96.46	99.13	93.63
Mass of container (g) (H)	17.72	17.57	17.08	17.55	17.95	17.78	17.68	17.91
Mass of moisture (g) F-G=(I)	10.68	11.83	16.58	14.8	18.66	15.51	19.91	20.73
Mass of dry soil (g) G-H=(J)	74.32	78.59	81.36	82.9	77.8	78.68	81.45	75.72
Moisture content % (I/J)*100=K	14.37	15.05	20.38	17.85	23.98	19.71	24.44	27.38
Avg. moisture content % (L)	14.71		17.85		21.85		25.91	
Dry density g/cm^3 E/(100+L)*100	1.376		1.393		1.388		1.283	

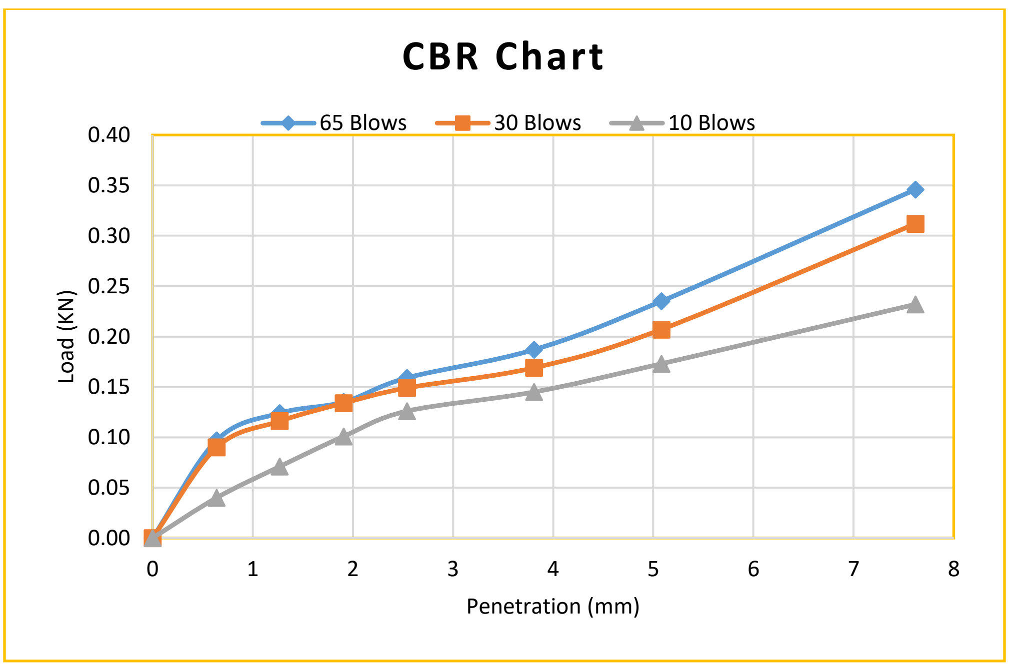


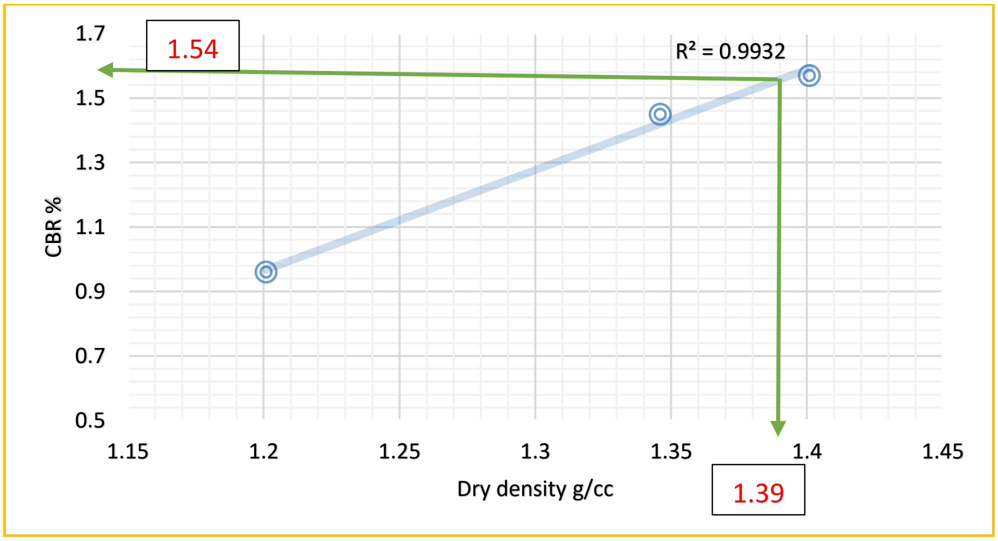
OMC%	20.05
MDD g/cc	1.39

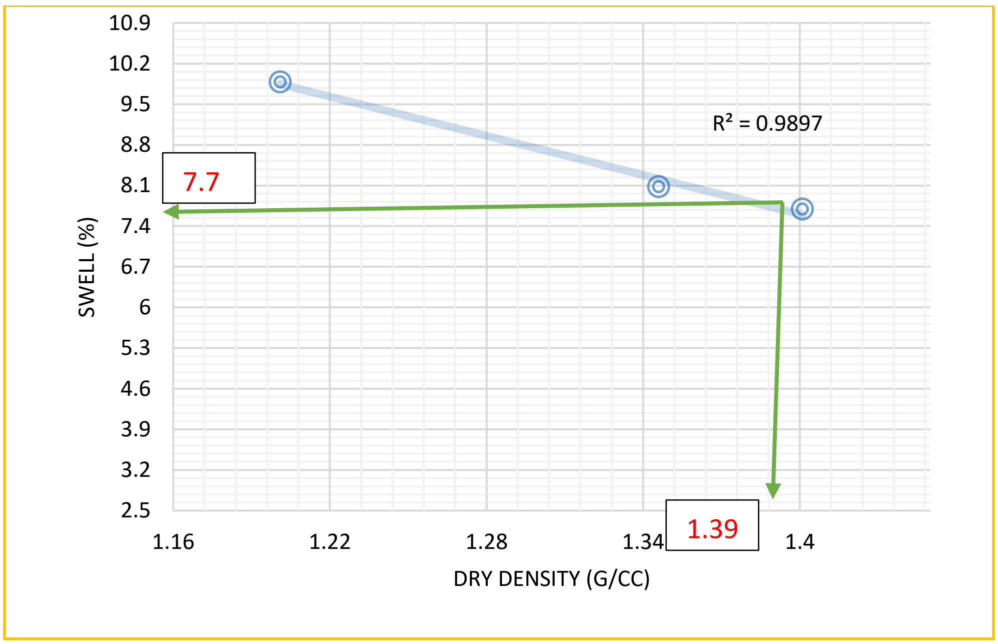
Appendix A-7: California Bearing Ratio Test Data Analysis (ASTM D-1883)

CBR and CBR Swell result summary of natural SS1

Penetration and load determination of untreated SS1						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10 blows	
Penetration (mm)	Load (KN)	CBR %	Load (KN)	CBR %	Load (KN)	CBR %
@2.54	0.209	1.57	0.193	1.45	0.128	0.96
@5.08	0.295	1.48	0.287	1.44	0.179	0.9
Modified MDD (g/cc)				1.46		
Dry Density at 95% of MDD				1.39		
No of Blows				65	30	10
CBR Values (%)				1.57	1.45	0.96
CBR swell (%)				7.69	8.08	9.88
DDBS (g/cc)				1.401	1.346	1.201
CBR (%) at 95% MDD and CBR swell (%)				1.54 and 7.7 respectively		

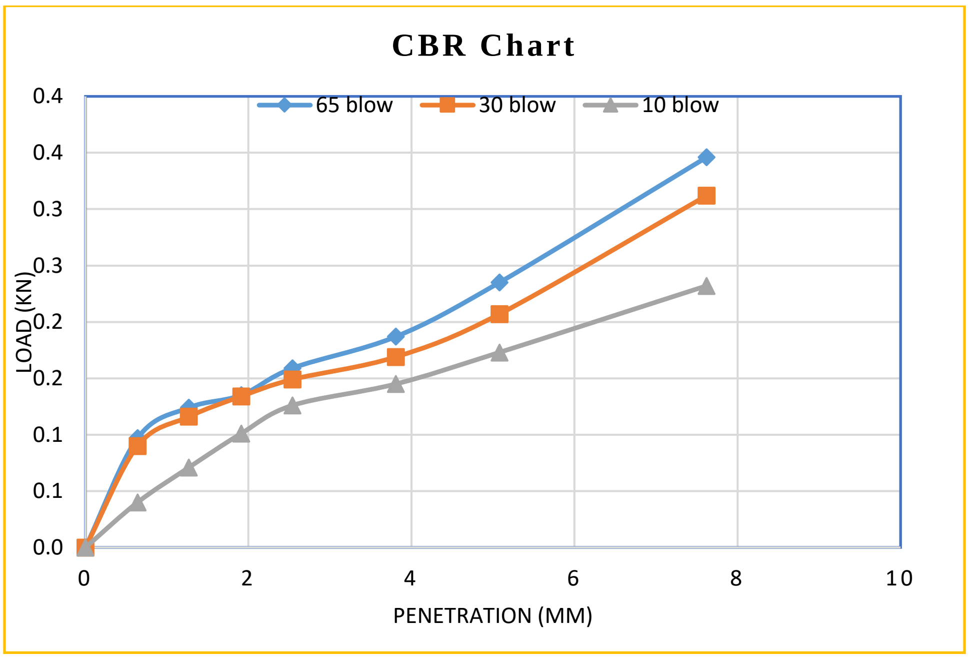


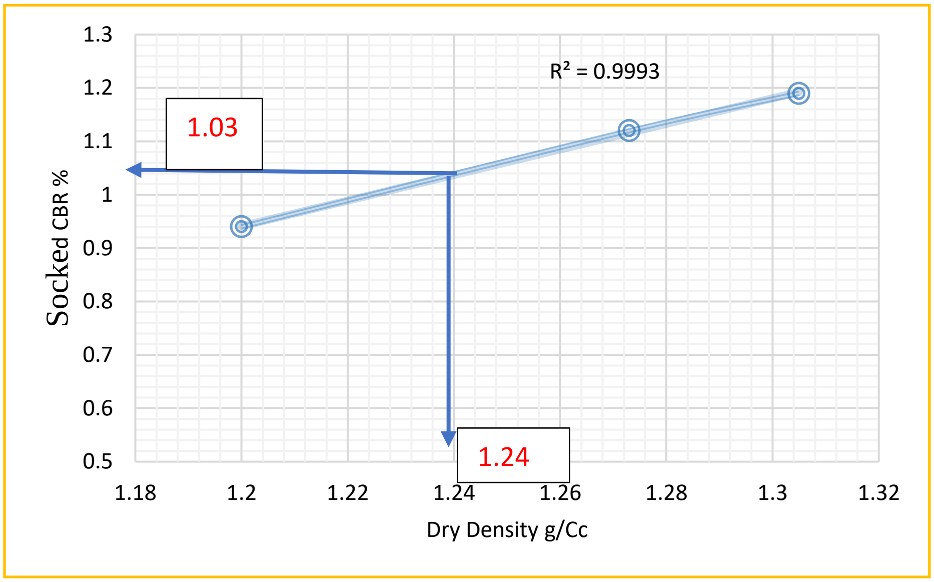

1.54
R² = 0.9932
1.7
1.5
1.3
1.1
0.9
0.7
0.5
CBR %
1.15
1.2
1.25
1.3
1.35
1.4
1.45
Dry density g/cc
1.39


10.9
10.2
9.5
8.8
8.1
7.4
6.7
6
5.3
4.6
3.9
3.2
2.5
SWELL (%)
7.7
R² = 0.9897
1.16
1.22
1.28
1.34
1.39
1.4
DRY DENSITY (G/CC)

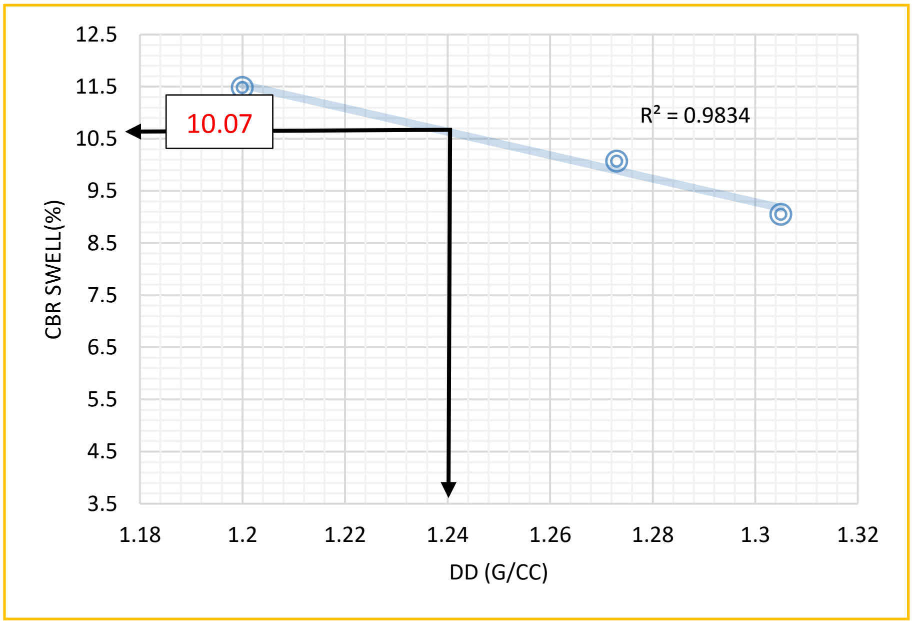
CBR and CBR Swell result summary of natural SS2

Penetration and load determination of untreated SS2						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10 blows	
Penetration (mm)	Load (KN)	CBR %	Load (KN)	CBR %	Load (KN)	CBR %
@2.54	0.159	1.19	0.139	1.12	0.000	0.94
@5.08	0.235	1.18	0.205	1.04	0.000	0.87
Modified MDD (g/cc)			1.3			
Dry Density at 95% of MDD			1.24			
No of Blows			65		30	10
CBR Values (%)			0		1.12	0.94
CBR swell (%)			9.05		10.07	11.48
DDBS (g/cc)			1.305		1.273	1.2
CBR (%) at 95% MDD and CBR swell (%)			1.03 and 10.07 respectively			





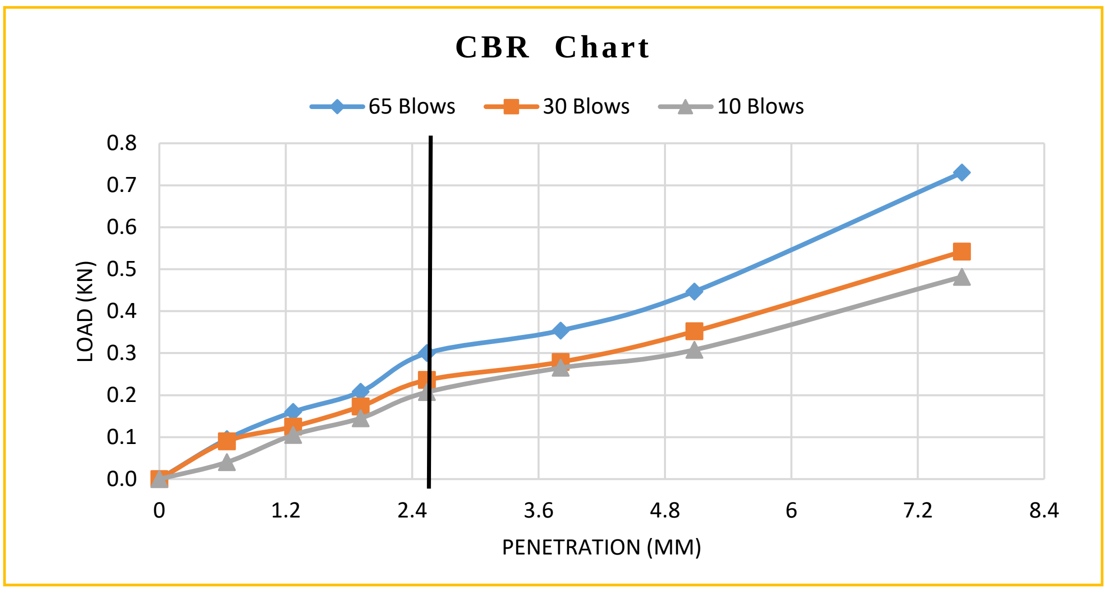
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
Socked CBR %
R^2 = 0.9993
1.03
1.24
1.18
1.2
1.22
1.24
1.26
1.28
1.3
1.32
Dry Density g/Cc

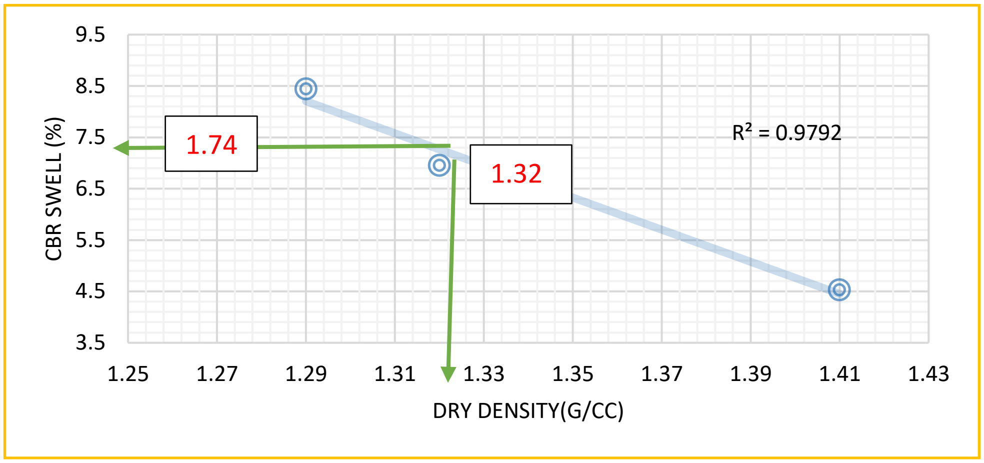


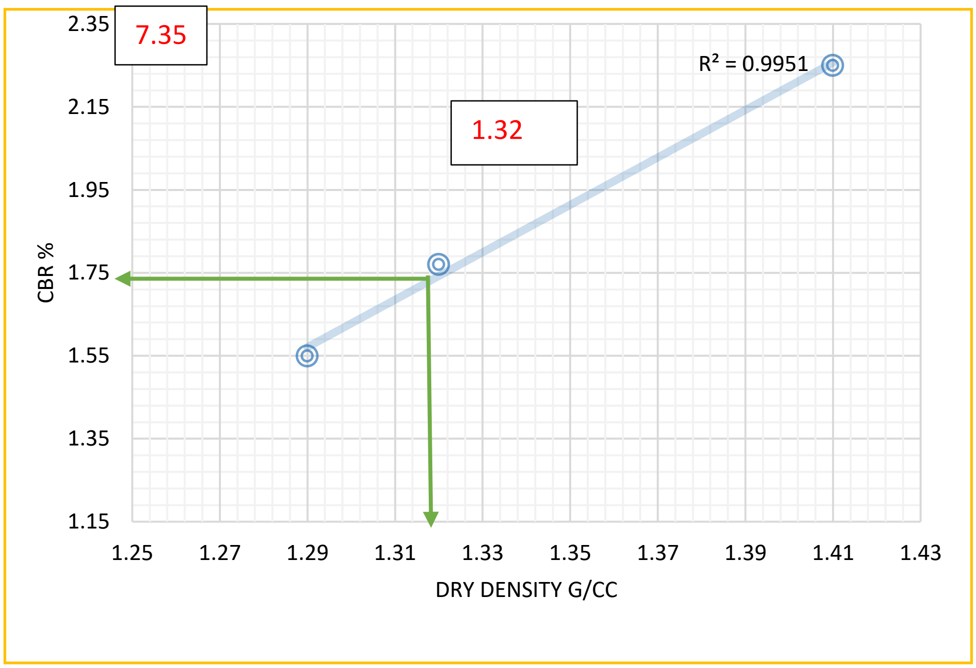
12.5
11.5
10.5
9.5
8.5
7.5
6.5
5.5
4.5
3.5
CBR SWELL(%)
10.07
R^2 = 0.9834
1.18
1.2
1.22
1.24
1.26
1.28
1.3
1.32
DD (G/CC)

CBR and CBR Swell result summary of natural SS

Penetration and load determination of untreated SS3						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10 blows	
Penetration (mm)	Load (KN)	CBR %	Load (KN)	CBR %	Load (KN)	CBR %
@2.54	0.3	2.25	0.236	1.77	0.000	0.00
@5.08	0.447	2.24	0.352	1.76	0.000	1.54
Modified MDD (g/cc)				1.39		
Dry Density at 95% of MDD				1.32		
No of Blows				65	30	10
CBR Values (%)				2.25	1.77	1.55
CBR swell (%)				4.53	6.95	8.44
DDBS (g/cc)				1.41	1.32	1.29
CBR (%) at 95% MDD and CBR swell (%)				1.74 and 7.35 respectively		



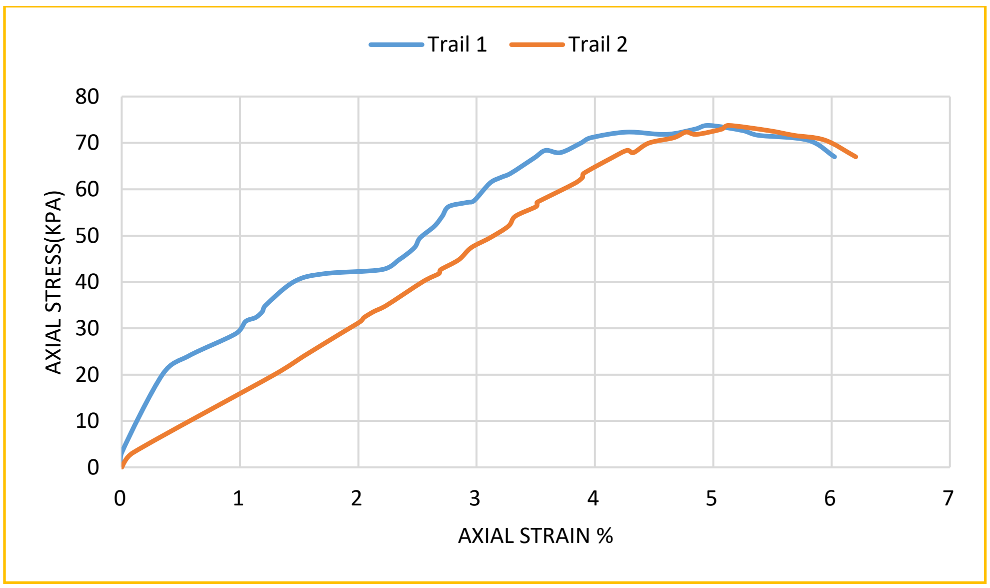

9.5
8.5
7.5
6.5
5.5
4.5
3.5
CBR SWELL (%)
1.74
1.32
R² = 0.9792
1.25
1.27
1.29
1.31
1.33
1.35
1.37
1.39
1.41
1.43
DRY DENSITY(G/CC)


2.35
7.35
R² = 0.9951
2.15
1.32
1.95
1.75
CBR %
1.55
1.35
1.15
1.25
1.27
1.29
1.31
1.33
1.35
1.37
1.39
1.41
1.43
DRY DENSITY G/CC

Appendix A-8: Unconfined Compression Strength Test Data Analysis According to ASTM D-2166

A) Undisturbed natural SS1

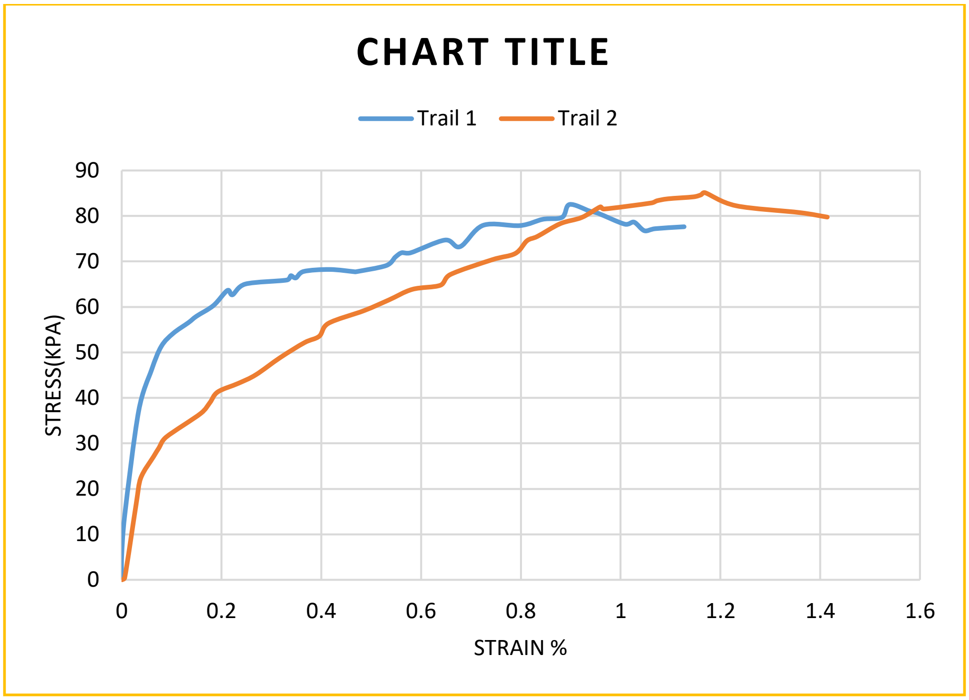
Sample information	Trial number: 01	Trial number:02
	Sample type	
	Undisturbed (Natural SS1)	Undisturbed (Natural SS1)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	7.5	8.0
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	85.06	90.73
Axial stress (qu) (kpa)	72.95	73.77
Shear stress (Su) or Cohesion (C) (kpa)	36.48	36.89
Corrected Area at peak qu (cm^2)	11.947	11.954
Load at peak qu, (N)	87.16	88.18



Trial 01					Trial 02			
strain (%)	AC(cm2)	Load(N)	Stress (kpa)		strain (%)	AC (cm2)	Load(N)	Stress(Kpa)
0.00	11.341	0.00	0.00		0.00	11.341	0.00	0.00
0.00	11.352	3.59	3.161		0.10	11.352	3.59	3.161
0.35	11.490	23.07	20.079		1.30	11.490	23.07	20.079
0.57	11.518	27.68	24.036		1.54	11.518	27.68	24.036
0.96	11.554	33.32	28.841		1.85	11.554	33.32	28.841
1.05	11.574	36.40	31.449		2.02	11.574	36.40	31.449
1.13	11.578	37.43	32.325		2.05	11.578	37.43	32.325
1.19	11.588	38.96	33.625		2.13	11.588	38.96	33.625
1.21	11.600	40.50	34.915		2.24	11.600	40.50	34.915
1.46	11.637	46.65	40.090		2.55	11.637	46.65	40.090
1.72	11.653	48.70	41.795		2.68	11.653	48.70	41.795
2.20	11.655	49.73	42.668		2.70	11.655	49.73	42.668
2.34	11.673	52.29	44.797		2.85	11.673	52.29	44.797
2.47	11.686	55.37	47.381		2.95	11.686	55.37	47.381
2.52	11.705	57.93	49.495		3.11	11.705	57.93	49.495
2.64	11.724	61.01	52.038		3.27	11.724	61.01	52.038
2.71	11.731	63.57	54.191		3.33	11.731	63.57	54.191
2.76	11.753	66.14	56.273		3.50	11.753	66.14	56.273
2.92	11.753	67.16	57.143		3.51	11.753	67.16	57.143
2.98	11.757	67.67	57.561		3.54	11.757	67.67	57.561
3.11	11.793	72.29	61.300		3.83	11.793	72.29	61.300
3.21	11.800	73.83	62.562		3.90	11.800	73.83	62.562
3.29	11.802	74.85	63.425		3.91	11.802	74.85	63.425
3.49	11.831	78.95	66.736		4.14	11.831	78.95	66.736
3.58	11.847	81.00	68.377		4.27	11.847	81.00	68.377
3.71	11.854	80.49	67.904		4.33	11.854	80.49	67.904
3.88	11.870	83.05	69.972		4.46	11.870	83.05	69.972
3.97	11.896	84.59	71.111		4.67	11.896	84.59	71.111
4.27	11.909	86.13	72.325		4.77	11.909	86.13	72.325
4.60	11.920	85.62	71.829		4.86	11.920	85.62	71.829
4.84	**11.947**	**87.16**	**72.955**		5.07	11.947	87.16	72.955
4.96	11.954	88.18	73.767		**5.13**	**11.954**	**88.18**	**73.767**
5.25	11.997	87.16	72.649		5.47	11.997	87.16	72.649
5.38	12.024	86.13	71.634		5.68	12.024	86.13	71.634
5.80	12.058	85.11	70.582		5.94	12.058	85.11	70.582
6.03	12.091	81.00	66.995		6.20	12.091	81.00	66.995

B) Remolded soil sample (sample 1)

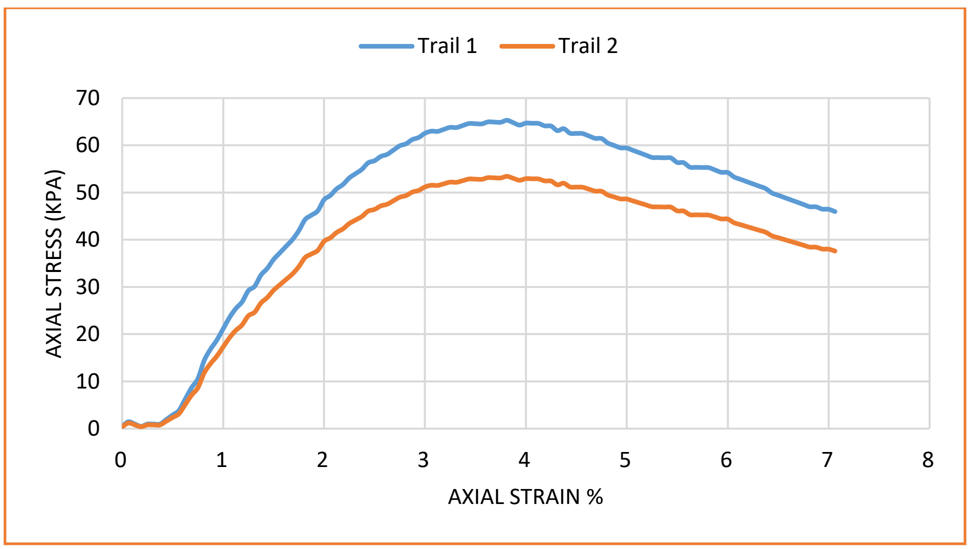
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Natural soil 1)	Remolded (Natural soil 1)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	82.53	85.12
Shear stress (Su) or Cohesion (C) (kpa)	41.26	42.56
Corrected Area at peak qu (cm^2)	11.444	11.475
Load at peak qu, (N)	94.44	97.68



Trail 1					Trail 2			
Strain %	AC (cm2)	Load (N)	Stress(Kpa)		Strain %	AC (cm2)	Load (N)	Stress(Kpa)
0.000	11.3408	-	0.00		0.00	11.3408	0.00	0.00
0.000	11.3408	5.94	5.234		0.01	11.3415	0.54	0.476
0.006	11.3415	16.19	14.275		0.03	11.3441	18.89	16.650
0.034	11.3447	42.09	37.105		0.04	11.3451	25.36	22.357
0.060	11.3476	52.35	46.131		0.06	11.3481	30.76	27.107
0.079	11.3498	58.28	51.352		0.07	11.3493	32.92	29.006
0.103	11.3525	61.52	54.192		0.09	11.3509	35.62	31.379
0.134	11.3560	64.22	56.552		0.16	11.3588	41.55	36.583
0.150	11.3579	65.84	57.968		0.18	11.3609	44.25	38.952
0.184	11.3617	68.54	60.323		0.19	11.3627	46.95	41.320
0.212	11.3649	72.32	63.630		0.23	11.3674	49.11	43.202
0.222	11.3661	71.24	62.674		0.27	11.3714	51.27	45.085
0.248	11.3690	73.93	65.031		0.32	11.3769	55.59	48.858
0.331	11.3785	75.01	65.926		0.37	11.3825	59.36	52.153
0.339	11.3794	76.09	66.869		0.40	11.3859	60.98	53.559
0.349	11.3805	75.55	66.388		0.41	11.3880	64.22	56.393
0.365	11.3823	77.17	67.800		0.49	11.3962	67.46	59.194
0.417	11.3883	77.71	68.238		0.54	11.4019	70.16	61.531
0.467	11.3940	77.17	67.730		0.58	11.4073	72.85	63.867
0.469	11.3943	77.17	67.729		0.64	11.4137	73.93	64.776
0.530	11.4013	78.79	69.107		0.66	11.4160	76.63	67.127
0.549	11.4034	80.95	70.988		0.74	11.4257	80.41	70.377
0.562	11.4049	82.03	71.925		0.79	11.4310	82.03	71.760
0.579	11.4069	82.03	71.912		0.81	11.4337	85.27	74.575
0.648	11.4148	85.27	74.699		0.83	11.4361	86.35	75.503
0.678	11.4183	83.65	73.258		0.88	11.4414	89.58	78.299
0.725	11.4236	89.04	77.948		0.92	11.4464	91.20	79.679
0.799	11.4321	89.04	77.890		0.96	11.4507	93.90	82.005
0.844	11.4374	90.66	79.270		0.97	11.4515	93.36	81.528
0.883	11.4418	91.20	79.711		1.06	11.4625	94.98	82.862
0.898	11.4435	94.44	82.528		1.07	11.4637	95.52	83.324
0.938	11.4482	92.82	81.081		1.10	11.4664	96.06	83.776
0.955	11.4501	92.28	80.595		1.15	11.4723	96.60	84.203
1.008	11.4563	89.58	78.196		1.16	11.4741	97.14	84.660
1.027	11.4585	90.12	78.653		1.17	11.4750	97.68	85.124
1.048	11.4609	87.97	76.753		1.23	11.4822	94.44	82.250
1.070	11.4634	88.51	77.207		1.36	11.4971	92.82	80.736
1.127	11.4701	89.04	77.632		1.41	11.5035	91.74	79.752

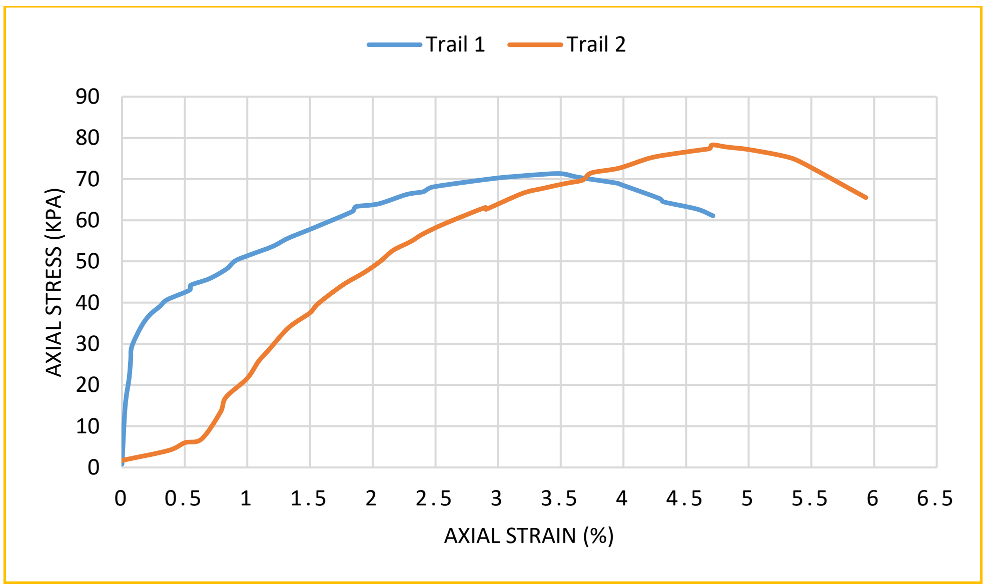
C)Undisturbed natural soil sample (sample 2)

Sample information	Trial number: 01	Trial number:02
	Sample type	
	Undisturbed (Natural soil 2)	Undisturbed (Natural soil 2)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	90.73
Axial stress (qu) (kpa)	65.31	53.43
Shear stress (Su) or Cohesion (C) (kpa)	32.65	26.72
Corrected Area at peak qu (cm^2)	11.791	11.791
Load at peak qu, (N)	77	63



C) Remolded soil sample (sample 2)

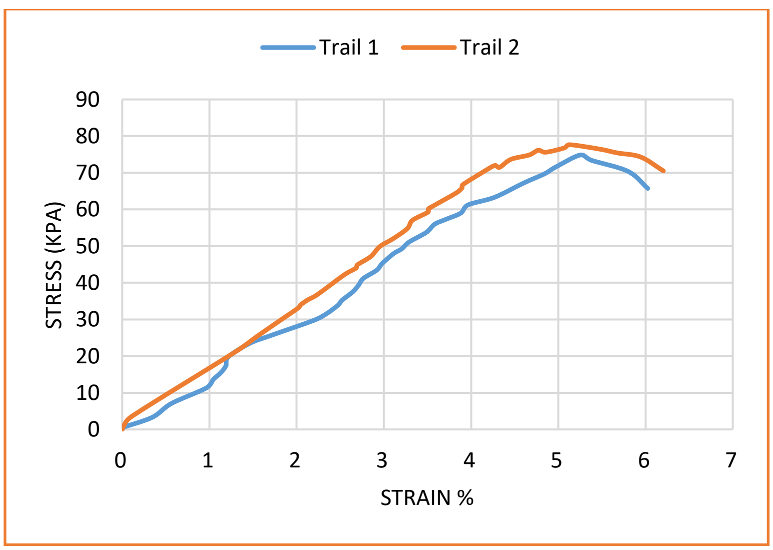
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Natural soil 2)	Remolded (Natural soil 2)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	71.31	78.29
Shear stress (Su) or Cohesion (C) (kpa)	35.66	39.15
Corrected Area at peak qu (cm^2)	11.752	11.590
Load at peak qu, (N)	83.81	90.74



			Trail 1		Trial 02			
strain (%)	Ac (cm2)	Load(N)	stress (kpa)		strain (%)	AC(cm2)	Load (N)	Stress (Kpa)
0.000	11.341	0.93	0.82		0.000	11.095	1.90	1.71
0.025	11.344	16.67	14.69		0.375	11.097	4.61	4.16
0.056	11.347	24.54	21.63		0.500	11.111	6.66	6.00
0.069	11.349	29.63	26.11		0.635	11.115	7.69	6.92
0.078	11.350	33.34	29.37		0.784	11.132	14.87	13.36
0.154	11.358	38.89	34.24		0.829	11.137	18.97	17.03
0.222	11.366	42.13	37.07		0.998	11.156	24.10	21.60
0.301	11.375	44.45	39.08		1.088	11.166	28.71	25.71
0.358	11.382	46.30	40.68		1.168	11.175	31.79	28.44
0.539	11.402	49.08	43.04		1.328	11.193	37.94	33.90
0.549	11.403	50.47	44.26		1.498	11.212	42.04	37.49
0.696	11.420	52.32	45.81		1.564	11.220	44.60	39.75
0.834	11.436	55.10	48.18		1.759	11.242	49.73	44.24
0.904	11.444	57.41	50.17		1.933	11.262	53.32	47.34
1.031	11.459	59.27	51.72		2.060	11.277	56.40	50.01
1.201	11.479	61.58	53.65		2.162	11.288	59.47	52.68
1.322	11.493	63.90	55.60		2.306	11.305	62.03	54.87
1.478	11.511	66.21	57.52		2.398	11.316	64.09	56.63
1.600	11.525	68.06	59.06		2.580	11.337	67.16	59.24
1.836	11.553	71.77	62.12		2.898	11.374	71.78	63.11
1.866	11.556	73.16	63.30		2.904	11.375	71.26	62.65
2.047	11.578	74.08	63.99		3.194	11.409	75.88	66.51
2.268	11.604	76.86	66.24		3.359	11.428	77.42	67.74
2.405	11.620	77.79	66.94		3.542	11.450	78.95	68.96
2.482	11.629	79.18	68.08		3.676	11.466	79.98	69.75
2.961	11.687	81.95	70.12		3.745	11.474	82.03	71.49
3.185	11.714	82.88	70.75		3.967	11.501	83.57	72.66
3.313	11.729	83.34	71.05		4.217	11.531	86.64	75.14
3.500	11.752	83.81	71.31		4.438	11.557	88.18	76.30
3.645	11.770	82.88	70.42		4.613	11.578	89.21	77.05
3.945	11.807	81.49	69.02		4.689	11.588	89.72	77.43
3.986	11.812	81.03	68.60		4.710	11.590	90.74	78.29
4.290	11.849	77.32	65.26		4.829	11.605	90.23	77.75
4.321	11.853	76.40	64.45		4.996	11.625	89.72	77.18
4.587	11.886	74.55	62.72		5.270	11.659	88.18	75.64
4.715	11.902	72.69	61.08		5.410	11.676	86.64	74.21
4.749	11.906	71.77	60.28		5.935	11.741	76.90	65.50

E Undisturbed natural soil sample (sample 3)

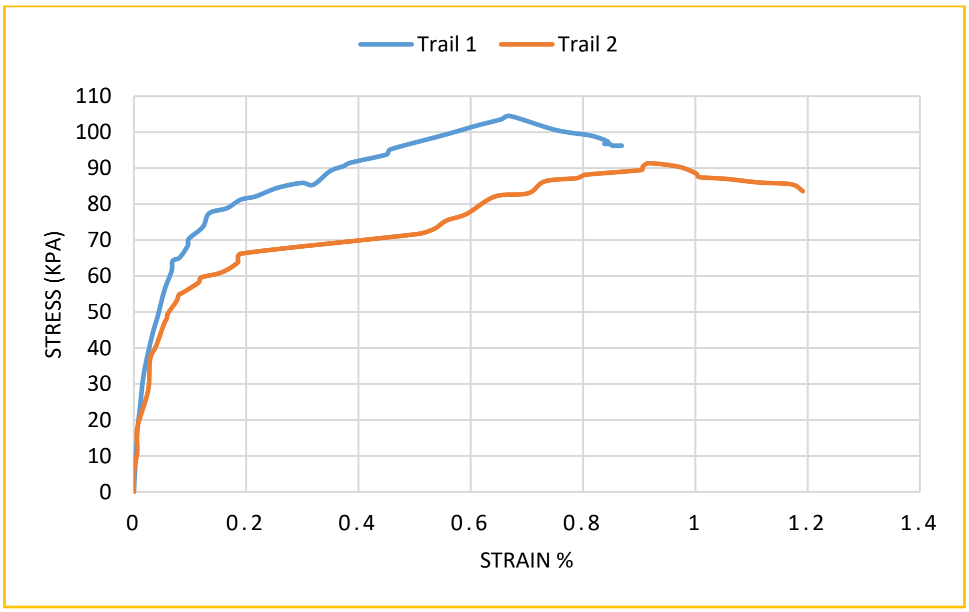
Sample information	Trial number: 01	Trial number:02
	Sample type	
	Undisturbed (Natural soil 3)	Undisturbed (Natural soil 3)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	90.73
Axial stress (qu) (kpa)	74.85	77.65
Shear stress (Su) or Cohesion (C) (kpa)	37.42	38.82
Corrected Area at peak qu (cm^2)	11.97	11.954
Load at peak qu, (N)	89.58	92.82



Trail 1					Trail 2			
Strain %	AC(cm2)	Load (KN)	Stress (Kpa)		Strain %	AC(cm2)	Load (KN)	Stress (Kpa)
0.00	11.34	0.00	0.00		0.00	11.34	-	-
0.00	11.34	0.54	0.48		0.10	11.35	3.78	3.33
0.35	11.38	3.78	3.32		1.30	11.49	24.28	21.14
0.57	11.41	8.09	7.10		1.54	11.52	29.14	25.30
0.96	11.45	12.95	11.31		1.85	11.55	35.08	30.36
1.05	11.46	15.65	13.66		2.02	11.57	38.32	33.10
1.13	11.47	17.81	15.53		2.05	11.58	39.40	34.03
1.19	11.48	19.97	17.40		2.13	11.59	41.01	35.39
1.21	11.48	22.67	19.74		2.24	11.60	42.63	36.75
1.46	11.51	26.98	23.45		2.55	11.64	49.11	42.20
1.72	11.54	29.68	25.72		2.68	11.65	51.27	43.99
2.20	11.60	34.54	29.79		2.70	11.66	52.35	44.91
2.34	11.61	36.70	31.60		2.85	11.67	55.05	47.15
2.47	11.63	39.40	33.88		2.95	11.69	58.28	49.88
2.52	11.63	41.01	35.25		3.11	11.70	60.98	52.10
2.64	11.65	43.71	37.53		3.27	11.72	64.22	54.78
2.71	11.66	45.87	39.35		3.33	11.73	66.92	57.04
2.76	11.66	48.03	41.18		3.50	11.75	69.62	59.24
2.92	11.68	50.73	43.43		3.51	11.75	70.70	60.15
2.98	11.69	52.89	45.24		3.54	11.76	71.24	60.59
3.11	11.70	56.13	47.95		3.83	11.79	76.09	64.53
3.21	11.72	57.74	49.28		3.90	11.80	77.71	65.85
3.29	11.73	59.90	51.08		3.91	11.80	78.79	66.76
3.49	11.75	63.14	53.74		4.14	11.83	83.11	70.25
3.58	11.76	65.84	55.98		4.27	11.85	85.27	71.98
3.71	11.78	67.46	57.28		4.33	11.85	84.73	71.48
3.88	11.80	69.62	59.00		4.46	11.87	87.43	73.65
3.97	11.81	72.32	61.24		4.67	11.90	89.04	74.85
4.27	11.85	75.01	63.32		4.77	11.91	90.66	76.13
4.60	11.89	79.87	67.19		4.86	11.92	90.12	75.61
4.84	11.92	83.11	69.73		5.07	11.95	91.74	76.79
4.96	11.93	85.27	71.46		5.13	11.95	92.82	77.65
5.25	11.97	89.58	74.85		5.47	12.00	91.74	76.47
5.38	11.99	87.97	73.39		5.68	12.02	90.66	75.40
5.80	12.04	84.73	70.38		5.94	12.06	89.58	74.30
6.03	12.07	79.33	65.74		6.20	12.09	85.27	70.52

F, Remolded soil sample (sample 3)

Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Natural soil 3)	Remolded (Natural soil 3)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	104.46	91.35
Shear stress (Su) or Cohesion (C) (kpa)	52.23	45.68
Corrected Area at peak qu (cm^2)	11.417	11.446
Load at peak qu, (N)	119.27	104.57



Trail 1					Trail 2			
Strain %	AC(cm2)	Load(N)	Stress (Kpa)		Strain%	AC(cm2)	Load(N)	Stress (Kpa)
0.000	11.341	0.00	0.00		0.000	11.341	-	0.000
0.006	11.341	18.35	16.18		0.000	11.341	6.44	5.675
0.011	11.342	26.98	23.79		0.006	11.342	12.91	11.385
0.018	11.343	37.24	32.83		0.006	11.342	20.19	17.801
0.032	11.344	49.11	43.29		0.025	11.344	32.06	28.264
0.043	11.346	55.59	48.99		0.029	11.344	41.78	36.826
0.055	11.347	63.68	56.12		0.040	11.346	46.09	40.627
0.067	11.348	69.62	61.34		0.054	11.347	53.11	46.804
0.069	11.349	72.85	64.20		0.059	11.348	54.73	48.228
0.082	11.350	73.93	65.14		0.061	11.348	56.35	49.654
0.096	11.352	77.71	68.46		0.076	11.350	60.13	52.975
0.098	11.352	79.87	70.36		0.080	11.350	62.28	54.874
0.123	11.355	83.65	73.67		0.087	11.351	62.82	55.346
0.134	11.356	87.97	77.46		0.115	11.354	66.06	58.182
0.166	11.360	89.58	78.86		0.120	11.355	67.68	59.605
0.190	11.362	92.28	81.22		0.156	11.359	69.30	61.009
0.217	11.365	93.36	82.15		0.186	11.362	72.54	63.841
0.255	11.370	96.06	84.49		0.190	11.363	75.24	66.213
0.299	11.375	97.68	85.87		0.505	11.399	81.71	71.685
0.320	11.377	97.14	85.38		0.535	11.402	83.33	73.084
0.349	11.381	101.46	89.15		0.557	11.405	86.03	75.433
0.374	11.383	103.08	90.55		0.593	11.409	88.19	77.298
0.385	11.385	104.16	91.49		0.643	11.415	93.73	82.112
0.451	11.392	106.85	93.80		0.702	11.421	94.81	83.008
0.457	11.393	108.47	95.21		0.732	11.425	98.63	86.330
0.556	11.404	113.33	99.37		0.790	11.431	99.71	87.224
0.608	11.410	116.03	101.69		0.805	11.433	100.79	88.155
0.654	11.415	118.19	103.53		0.904	11.445	102.41	89.481
0.670	11.417	119.27	104.46		0.906	11.445	103.49	90.422
0.751	11.427	114.95	100.60		0.918	11.446	104.57	91.355
0.811	11.434	113.33	99.12		0.971	11.452	103.49	90.363
0.833	11.436	112.25	98.16		1.002	11.456	101.33	88.451
0.845	11.437	111.17	97.20		1.007	11.456	100.25	87.504
0.838	11.437	110.63	96.73		1.056	11.462	99.71	86.990
0.846	11.438	110.63	96.73		1.113	11.469	98.63	85.998
0.853	11.438	110.09	96.25		1.172	11.476	98.09	85.477
0.869	11.440	110.09	96.23		1.191	11.478	95.93	83.580

Appendix C: Laboratory test Results of SS2 stabilized with CDWP and JGTF Combination.

Appendix C-1 Compaction Test Data Analysis of Soil Stabilized with CDW and JGTF (According to ASTM D-1557)

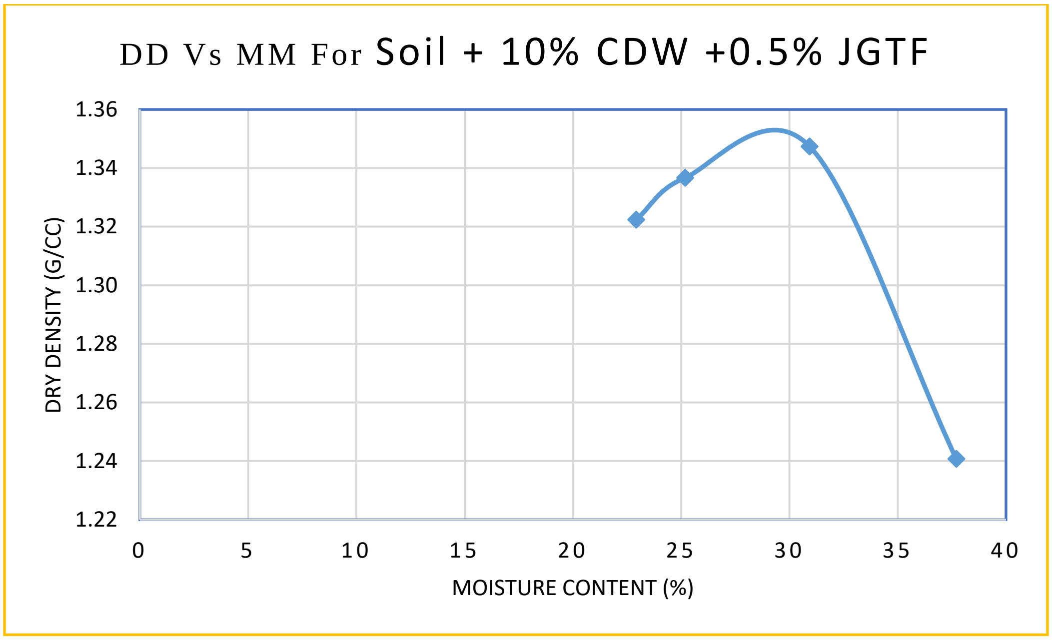
In this section some data presented for sample from each combination to limit un nessocer increment of pages on paper

A) **Soil + 10% CDW +0.5% JGTF**

Soil +10%JGT+0.5%CDW	
Mass of sample (g)	4500
Mass of Mold (g) B	2721
Volume of Mold (cm^3) D	2124

Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	6174		6275		6468		6350	
Mass of Wet Soil (g) A-B=C	3453		3554		3747		3629	
Bulk Density g/cm^3 C/D=(E)	1.63		1.67		1.76		1.71	
moisture determination								
Container code	5HT	5A	4A	B12	P11	4E	P3	K2
Mass of wet soil + container(g) (F)	85.27	84.97	107.92	113.20	93.96	115.75	114.56	116.85
Mass of dry soil + container (g) (G)	72.779	72.21	91.89	95.477	75.92	95.05	93.713	94.395
Mass of container (g) (H)	17.708	17.152	27.92	25.437	17.515	28.2	36.08	37.156
Mass of moisture (g) F-G=(I)	12.491	12.76	16.03	17.723	18.04	20.7	20.847	22.455
Mass of dry soil (g) G-H=(J)	55.071	55.06	63.97	70.04	58.405	66.85	57.633	57.239
Moisture content (%) (I/J)*100=K	22.682	23.18	25.05	25.304	30.888	30.963	36.172	39.230
Avg. moisture content % (L)	22.93			25.18	30.93		37.70	
Dry density g/cm^3 E/(100+L)*100	1.32			1.34	1.35		1.24	

MDD	1.35
OMC	29.85

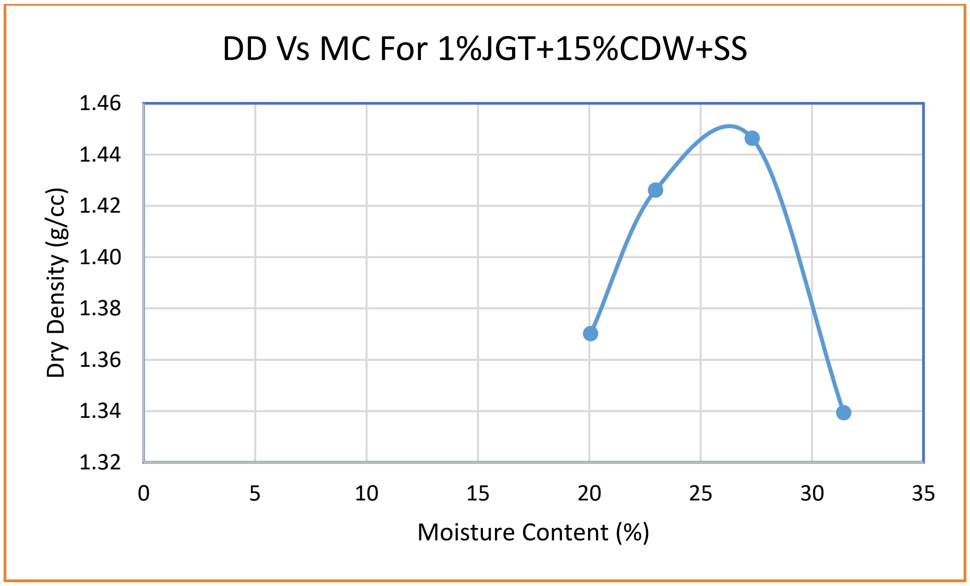


B) Soil+15%CDW+1%JGTF

1%JGT+15%CDW+SS	
Mass of sample (g)	4500
Mass of Mold (g) B	6527
Volume of Mold (cm^3) D	2124

Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	10021		10252		10438		10265.5	
Mass of Wet Soil (g) A-B=C	3494		3725		3911		3738.5	
Bulk Density g/cm^3 C/D=(E)	1.65		1.75		1.84		1.76	
moisture determination								
Container code	B12	A4	T	H	C8	C2	B12	3T2
Mass of wet soil + container(g) (F)	75.8	111.12	102.7	98.68	71.9	68.8	86.65	87.9
Mass of dry soil + container (g) (G)	66.28	96.14	87.7	82.6	58	55	70.5	71.1
Mass of container (g) (H)	20.5	18.64	17.2	17.6	5.8	5.7	19.5	17.2
Mass of moisture (g) F-G=(I)	9.52	14.98	14.95	16.08	13.9	13.8	16.15	16.8
Mass of dry soil (g) G-H=(J)	45.78	77.5	70.5	65	52.2	49.3	51	53.9
Moisture content (%) (I/J)*100=K	20.80	19.33	21.21	24.74	26.63	27.99	31.67	31.17
Avg. moisture content % (L)	20.06		22.97		27.31		31.42	
Dry density g/cm^3 E/(100+L)*100	1.37		1.43		1.45		1.34	

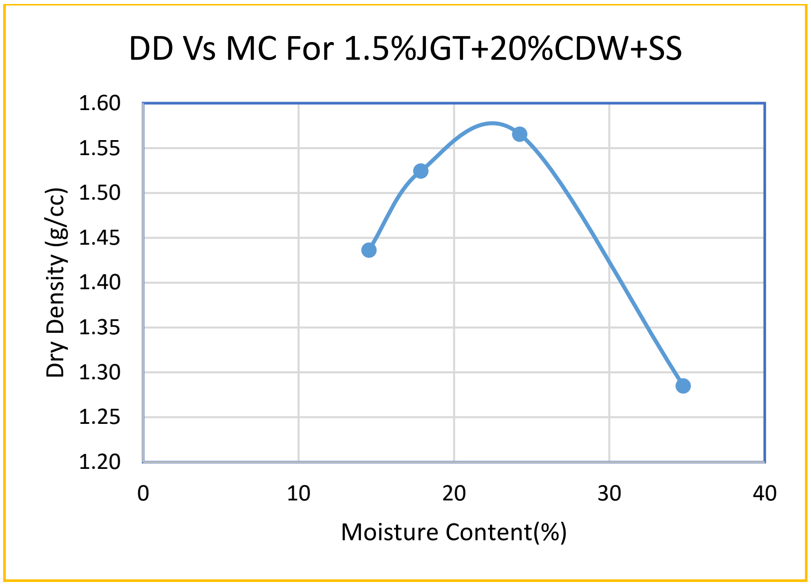
MDD	1.45
OMC	26.22



C) Soil+20%CDW+1.5%JGTF

1.5%JGT+20%CDW+SS	
Mass of sample (g)	4500
Mass of Mold (g) B	6527
Volume of Mold (cm^3) D	2124

Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	10021		10344		10658		10205.5	
Mass of Wet Soil (g) A-B=C	3494		3817		4131		3678.5	
Bulk Density g/cm^3 C/D=(E)	1.65		1.80		1.94		1.73	
moisture determination								
Container code	C14	H	T	4A	C8	A2	B12	3T2
Mass of wet soil + container(g) (F)	72.5	108.1	101.3	93.3	71	67	89	89
Mass of dry soil + container (g) (G)	66.28	96.14	87.7	82.6	58	55	71	71
Mass of container (g) (H)	20.5	18.64	17.2	17.6	6	5.7	20	17
Mass of moisture (g) F-G=(I)	6.22	11.98	13.6	10.7	13	12	19	18
Mass of dry soil (g) G-H=(J)	45.78	77.5	70.5	65	52	49	51	54
Moisture content (%) (I/J)*100=K	13.59	15.46	19.29	16.46	24	24.3	37	32
Avg. moisture content % (L)	14.52		17.88		24.24		34.76	
Dry density g/cm^3 E/(100+L)*100	1.44		1.52		1.57		1.29	

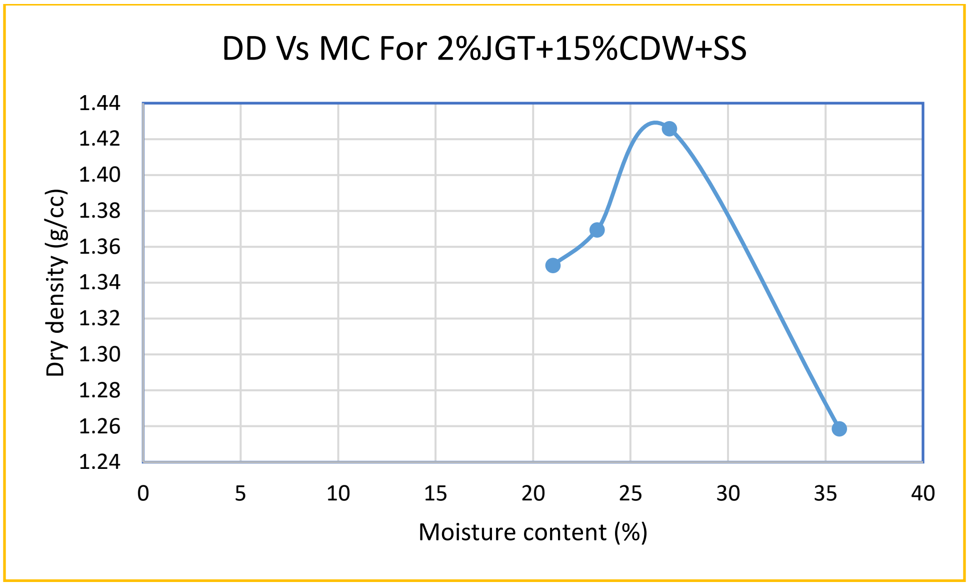


MDD	1.57
OMC	22.64

D) Soil+15%CDW+2%JGTF

2%JGT+15%CDW +SS								
Mass of sample (g)	4500							
Mass of Mold (g) B	2721							
Volume of Mold (cm^3) D	2124							
Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	6190		6307		6567		6349	
Mass of Wet Soil (g) A-B=C	3469		3586		3846		3628	
Bulk Density g/cm^3 C/D=(E)	1.63		1.69		1.81		1.71	
moisture determination								
Container code	W11	A3	5T	B12	B3	4E	P3	K2
Mass of wet soil + container(g) (F)	84.18	83.95	106.81	111.76	90.96	113.94	113.56	115.57
Mass of dry soil + container (g) (G)	72.779	72.21	91.89	95.477	75.92	95.052	93.713	94.4
Mass of container (g) (H)	17.708	17.152	27.92	25.437	17.52	28.204	36.08	37.16
Mass of moisture (g) F-G=(I)	11.401	11.74	14.92	16.283	15.04	18.884	19.847	21.17
Mass of dry soil (g) G-H=(J)	55.071	55.058	63.97	70.04	58.41	66.848	57.633	57.24
Moisture content (%) (I/J)*100=K	20.702	21.323	23.319	23.248	25.751	28.249	34.437	36.985
Avg. moisture content % (L)	21.01		23.28		27.00		35.71	
Dry density g/cm^3 E/(100+L)*100	1.35		1.37		1.43		1.26	

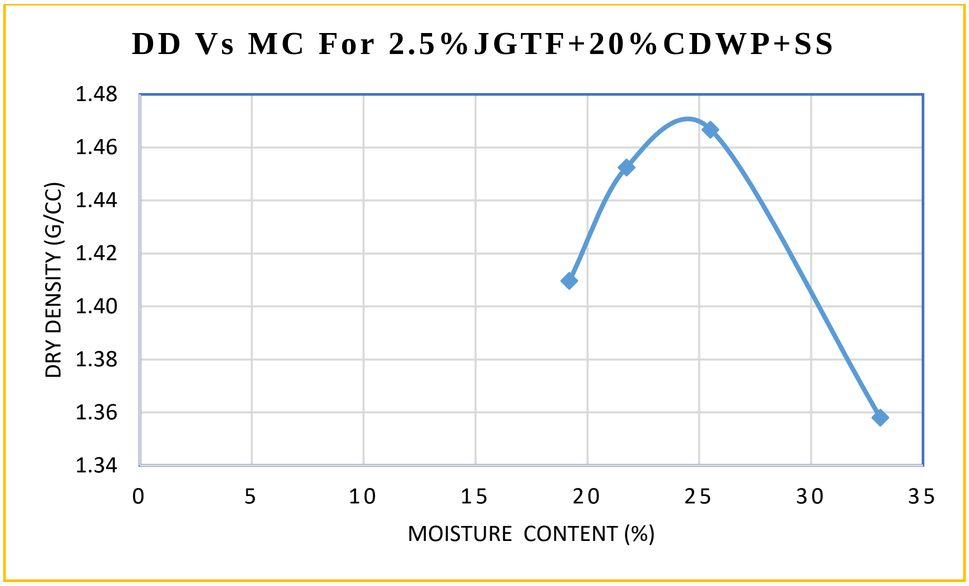
MDD	1.43
OMC	26.33



E) **Soil+20%CDW+2.5%JGTF**

2.5%JGT+20%CDW +SS								
Mass of sample (g)	4500							
Mass of Mold (g) B	2721							
Volume of Mold (cm^3) D	2124							
Test No.	1		2		3		4	
Mass of Mold + Wet soil (g) (A)	6290		6477		6631		6560	
Mass of Wet Soil (g) A-B=C	3569		3756		3910		3839	
Bulk Density g/cm^3 C/D=(E)	1.68		1.77		1.84		1.81	
moisture determination								
Container code	W11	A3	5T	B12	B3	4E	P3	K2
Mass of wet soil + container(g) (F)	82.18	83.95	104.82	111.79	90.96	111.94	113.56	112.57
Mass of dry soil + container (g) (G)	72.779	72.21	91.89	95.477	75.92	95.052	93.713	94.4
Mass of container (g) (H)	17.708	17.152	27.92	25.437	17.52	28.204	36.08	37.16
Mass of moisture (g) F-G=(I)	9.401	11.74	12.93	16.312	15.04	16.884	19.847	18.17
Mass of dry soil (g) G-H=(J)	55.071	55.058	63.97	70.04	58.41	66.848	57.633	57.24
Moisture content (%) (I/J)*100=K	17.071	21.323	20.208	23.290	25.751	25.257	34.437	31.744
Avg. moisture content % (L)	19.20			21.75	25.50		33.09	
Dry density g/cm^3 E/(100+L)*100	1.41			1.45	1.47		1.36	

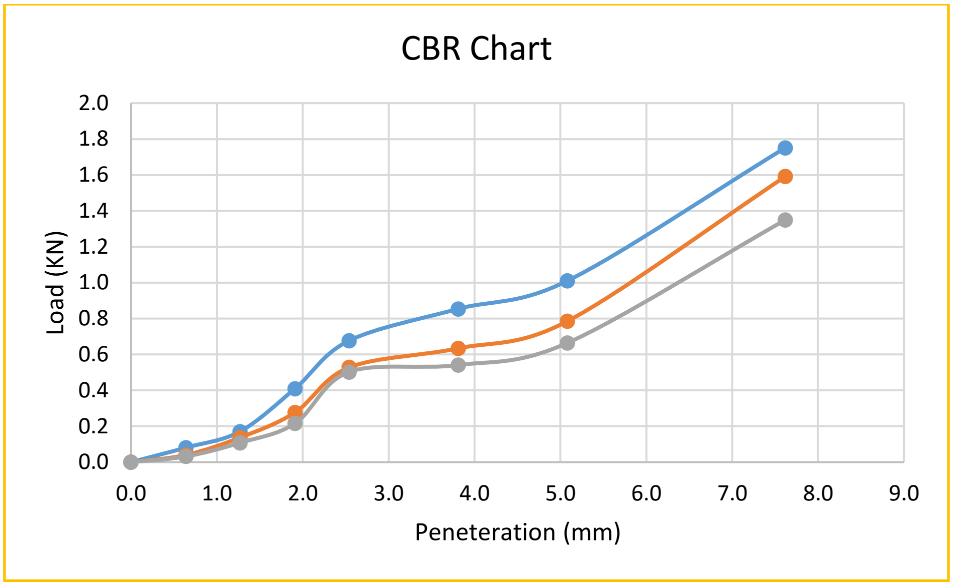
MDD	1.47
OMC	24.81

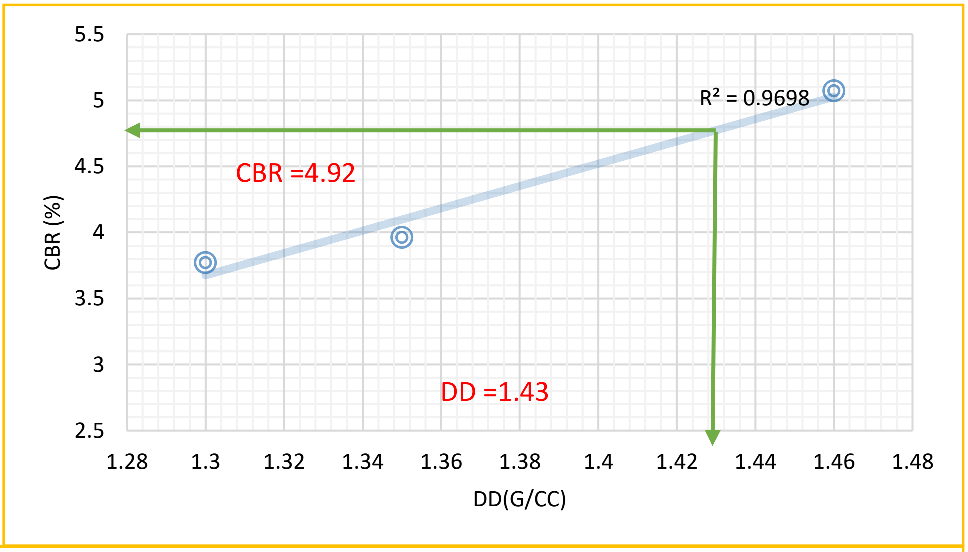


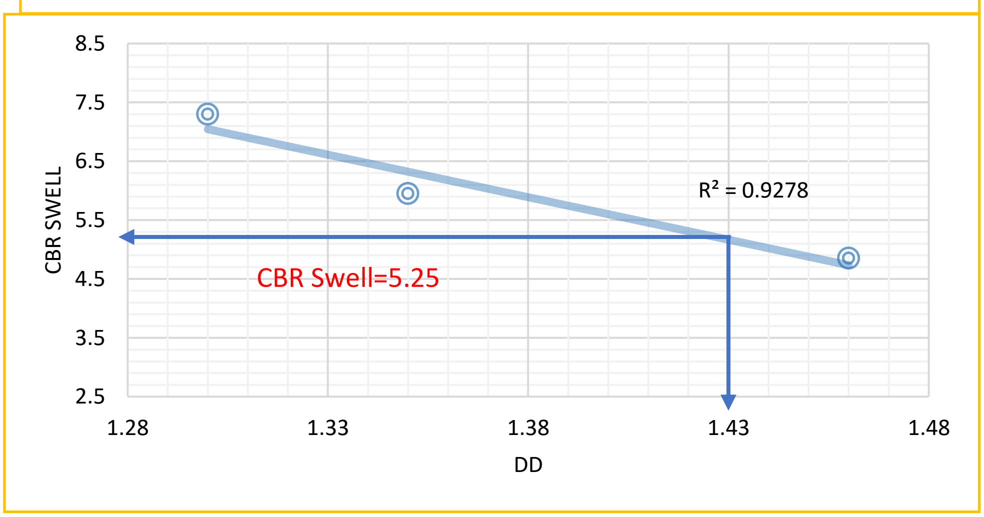
Appendix c-2: California Bearing Ratio (CBR) Test Data Analysis (ASTM D-1883)

A) 20%CDWP+0.5%JGTF +SS2

Penetration and load determination of 0.5%JGT + 20% CDW + SS						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10blows	
Penetration (mm)	Load (kn)	CBR %	Load (kn)	CBR (%)	Load (kn)	CBR (%)
@2.54	0.677	5.07	0.528	3.96	0.503	3.77
@5.08	1.010	5.05	0.785	3.91	0.664	3.32
MMDD (g/cc)	1.5					
Dry Density at 95% of MDD	1.43					
No of Blows	65	30	10			
CBR values (%)	5.05	3.96	3.77			
CBR swell (%)	4.85	5.95	7.30			
DDBS (g/cc)	1.46	1.35	1.30			
CBR (%) at 95% MDD and CBR swell (%)	4.92 and 5.25 respectively					


CBR Chart
Load (KN)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
Peneteration (mm)

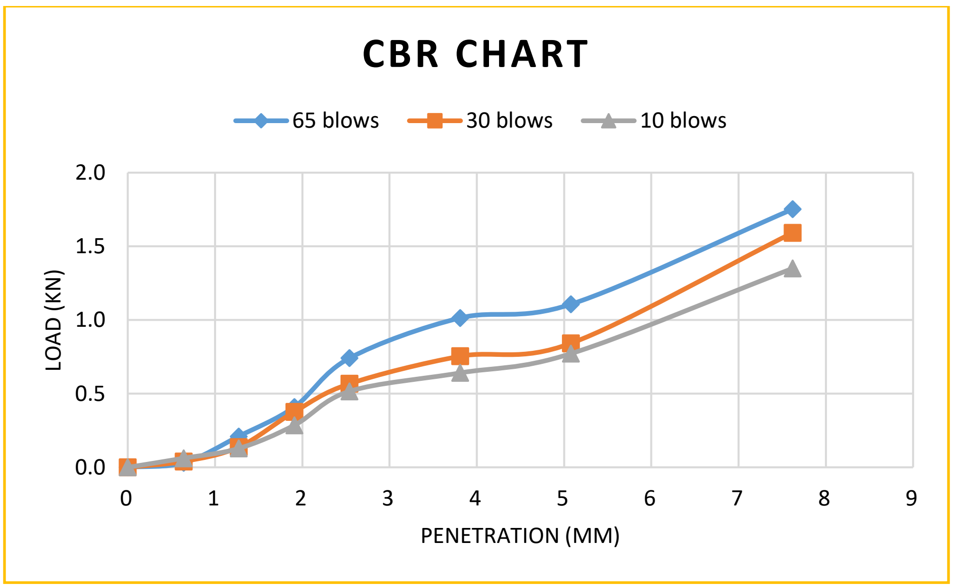

CBR (%)
5.5
5
4.5
4
3.5
3
2.5
R² = 0.9698
CBR =4.92
DD =1.43
1.28
1.3
1.32
1.34
1.36
1.38
1.4
1.42
1.44
1.46
1.48
DD(G/CC)

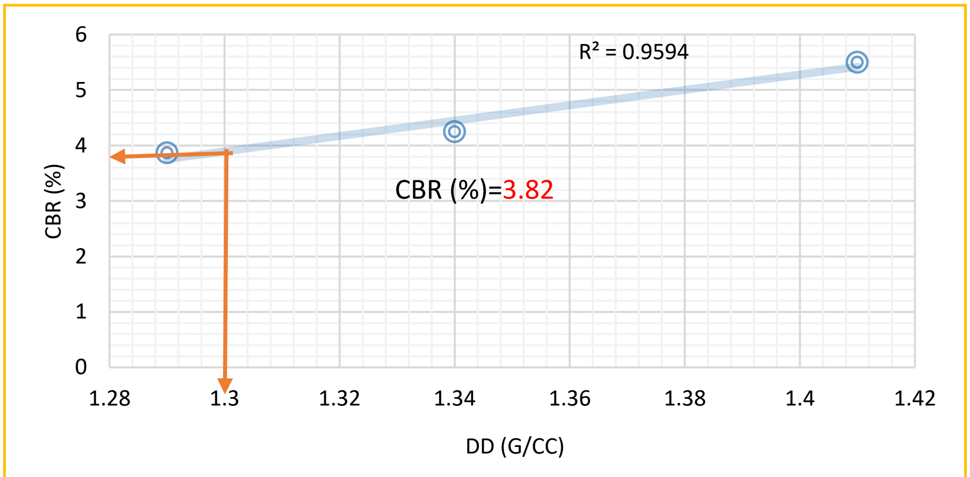

CBR SWELL
8.5
7.5
6.5
5.5
4.5
3.5
2.5
R² = 0.9278
CBR Swell=5.25
1.28
1.33
1.38
1.43
1.48
DD

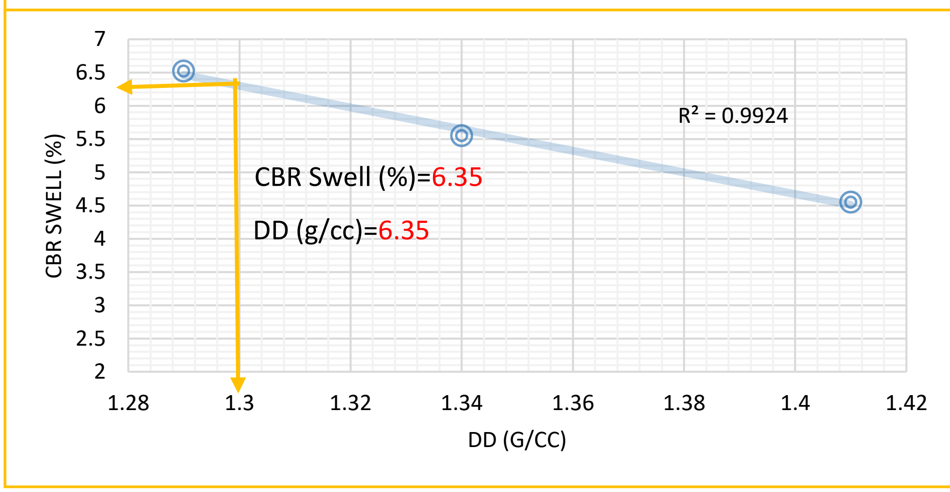
B) 10%CDWP+1%JGTF+SS2

Penetration and load determination of 1%JGT + 10% CDW + SS						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10blows	
Penetration (mm)	Load (kn)	CBR %	Load (kn)	CBR (%)	Load (kn)	CBR (%)
@2.54	0.741	5.55	0.567	4.25	0.516	3.87
@5.08	1.107	5.54	0.842	4.21	0.772	3.86

MMDD (g/cc)	1.37		
Dry Density at 95% of MDD	1.30		
No of Blows	65	30	10
CBR values (%)	5.54	4.25	3.87
CBR swell (%)	4.55	5.55	6.52
DDBS (g/cc)	1.41	1.34	1.29
CBR (%) at 95% MDD and CBR swell (%)	3.82 and 6.35 respectively		

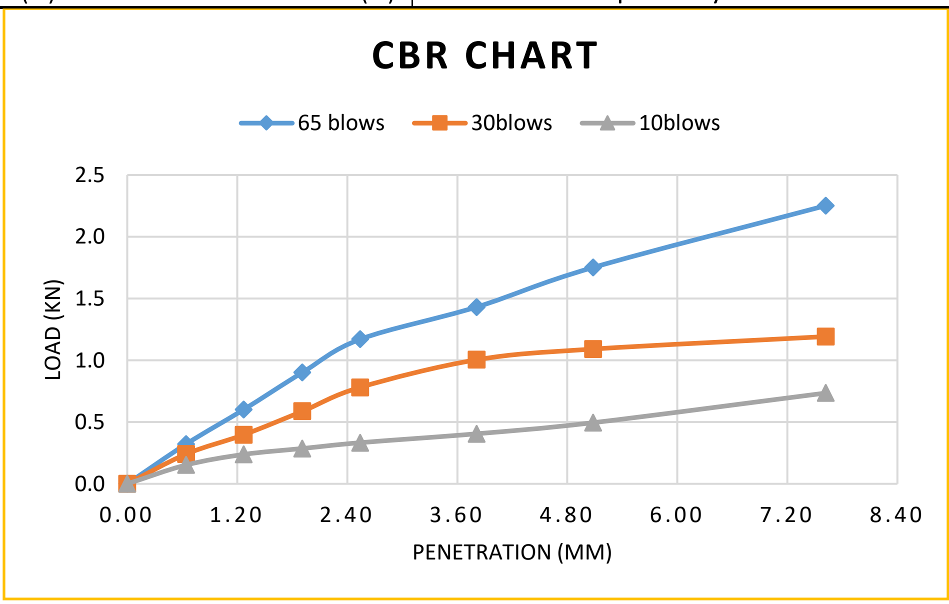


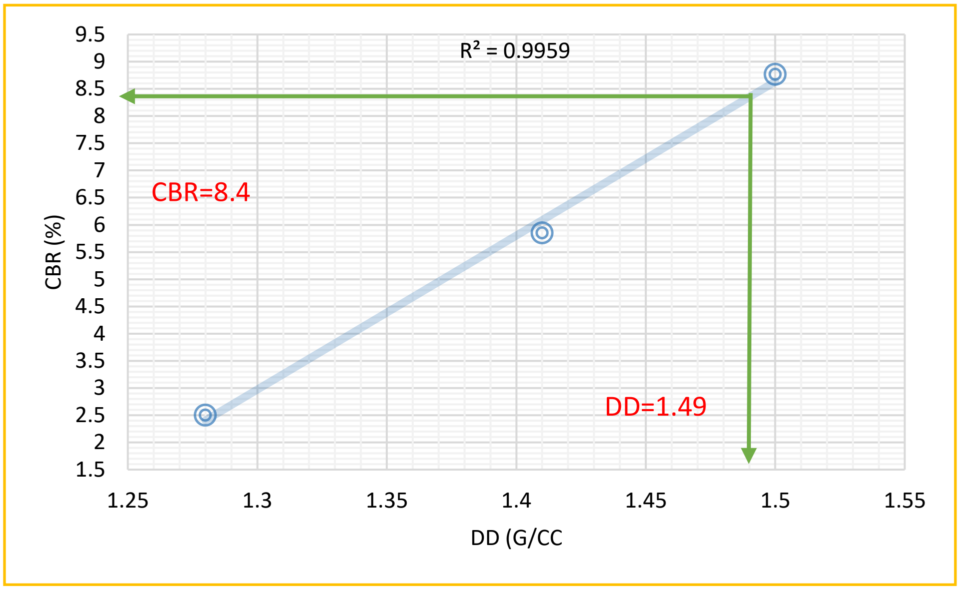

R² = 0.9594
CBR (%)=3.82
CBR (%)
DD (G/CC)
0
1
2
3
4
5
6
1.28
1.3
1.32
1.34
1.36
1.38
1.4
1.42

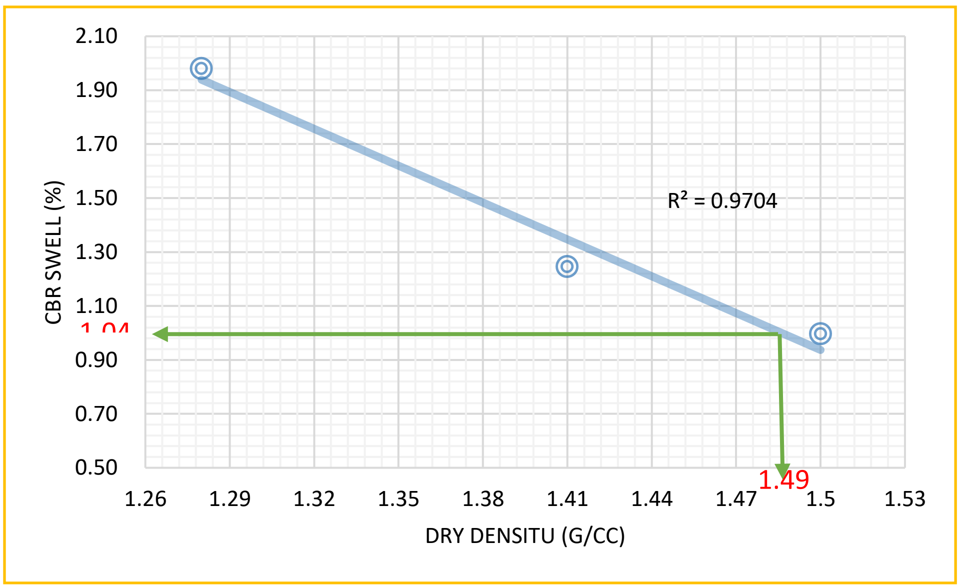

R² = 0.9924
CBR Swell (%)=6.35
DD (g/cc)=6.35
CBR SWELL (%)
DD (G/CC)
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
1.28
1.3
1.32
1.34
1.36
1.38
1.4
1.42

C) 20%CDWP+1.5%JGTF+SS2

Penetration and load determination of 1.5%JGT+20%CDW+SS						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10 blows	
Penetration (mm)	Load (KN)	CBR %	Load (KN)	CBR %	Load (KN)	CBR %
@2.54	1.170	8.77	0.781	5.85	0.333	2.5
@5.08	1.751	8.76	1.091	5.46	0.495	2.48
Modified MDD (g/cc)			1.57			
Dry Density at 95% of MDD			1.49			
No of Blows			65		30	10
CBR Values (%)			8.77		5.85	2.5
CBR swell (%)			1.00		1.25	1.98
DDBS (g/cc)			1.5		1.41	1.28
CBR (%) at 95% MDD and CBR swell (%)	8.4 and 1.04 respectively					

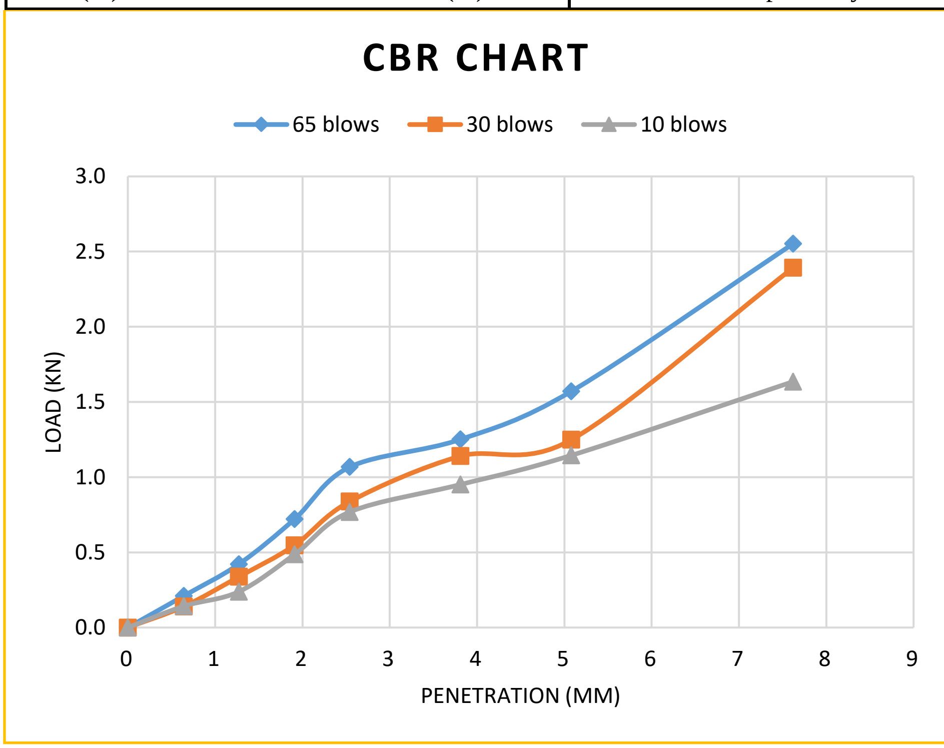


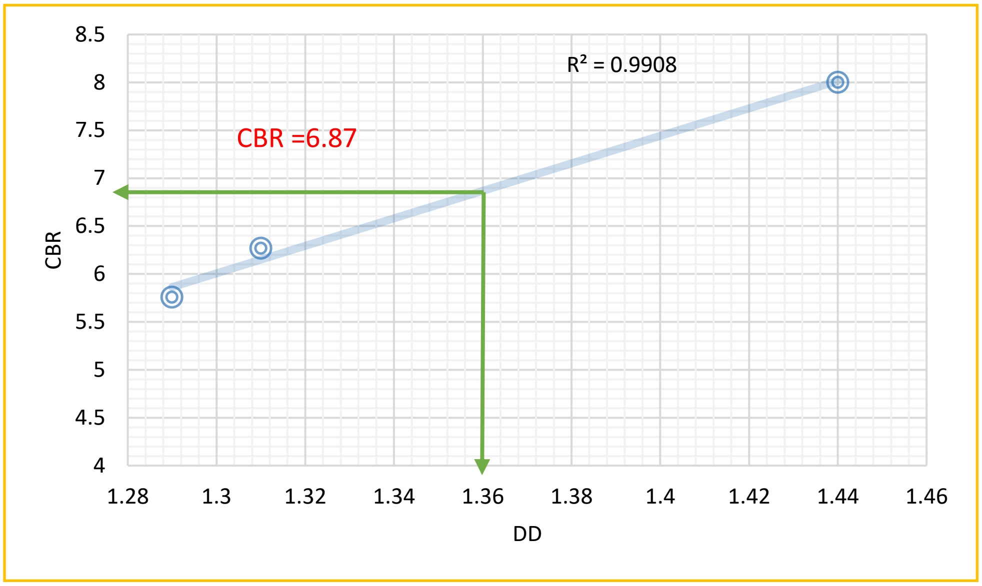

R² = 0.9959
CBR=8.4
DD=1.49
CBR (%)
9.5
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
1.5
1.25
1.3
1.35
1.4
1.45
1.5
1.55
DD (G/CC

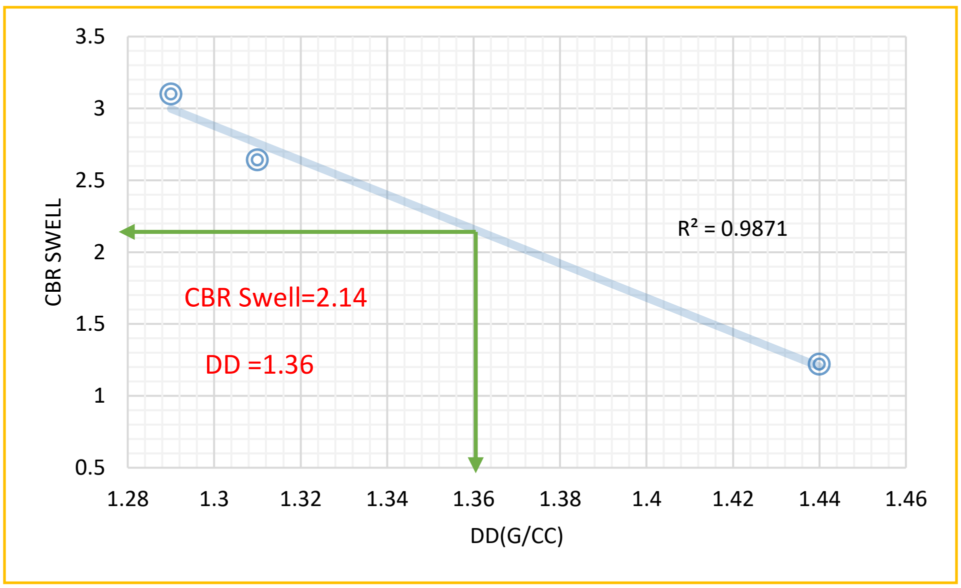

R² = 0.9704
CBR SWELL (%)
2.10
1.90
1.70
1.50
1.30
1.10
1.04
0.90
0.70
0.50
1.26
1.29
1.32
1.35
1.38
1.41
1.44
1.47
1.49
1.5
1.53
DRY DENSITU (G/CC)

D) 15%CDWP+2%JGTF+SS2

Penetration and load determination of 2%JGT + 15% CDW + SS						
Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10blows	
Penetration (mm)	Load (kn)	CBR %	Load (kn)	CBR (%)	Load (kn)	CBR (%)
@2.54	1.067	8.00	0.837	6.27	0.768	5.76
@5.08	1.571	7.86	1.249	6.25	1.145	5.73
MMDD (g/cc)	1.43					
Dry Density at 95% of MDD	1.36					
No of Blows	65		30		10	
CBR values (%)	8.00		6.27		5.76	
CBR swell (%)	1.22		2.64		3.10	
DDBS (g/cc)	1.44		1.31		1.29	
CBR (%) at 95% MDD and CBR swell (%)	6.87 and 2.14 respectively					

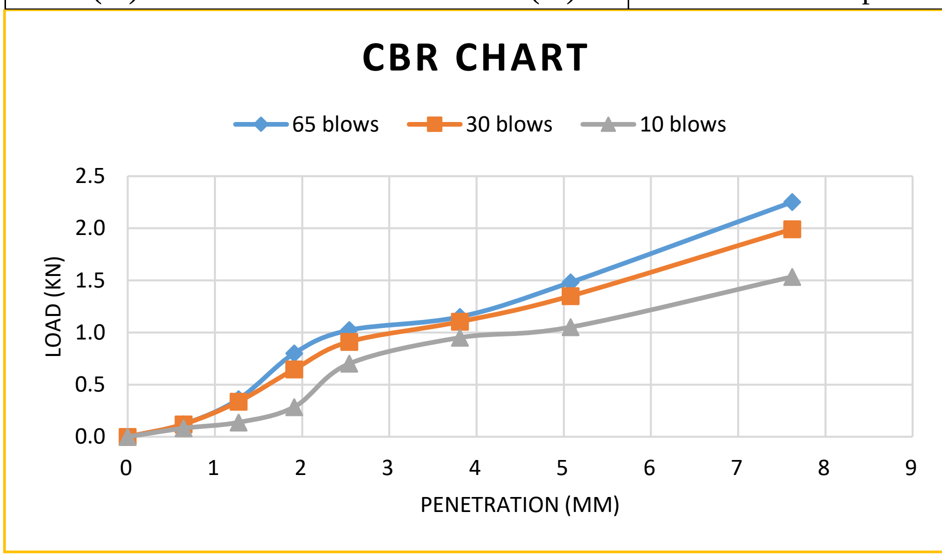


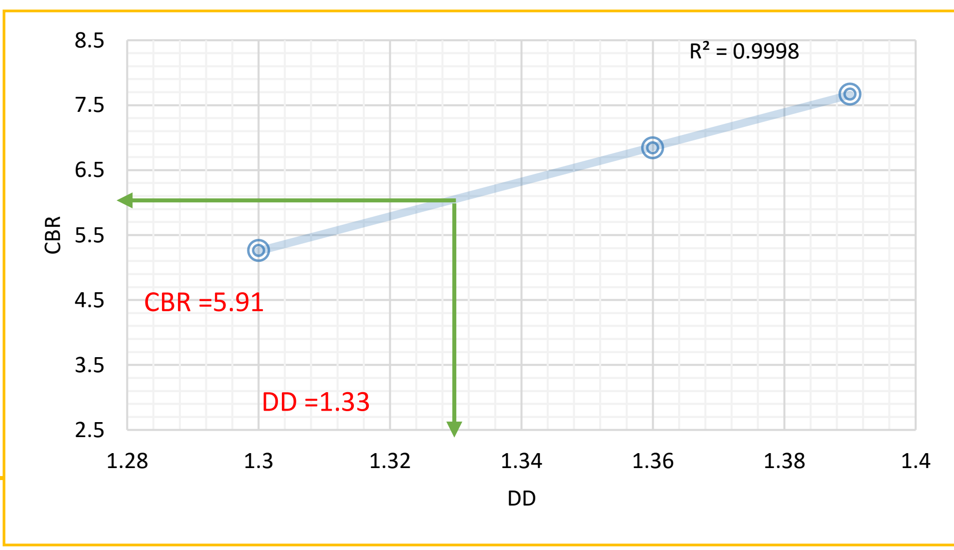

8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
CBR
1.28
1.3
1.32
1.34
1.36
1.38
1.4
1.42
1.44
1.46
DD
R² = 0.9908
CBR =6.87

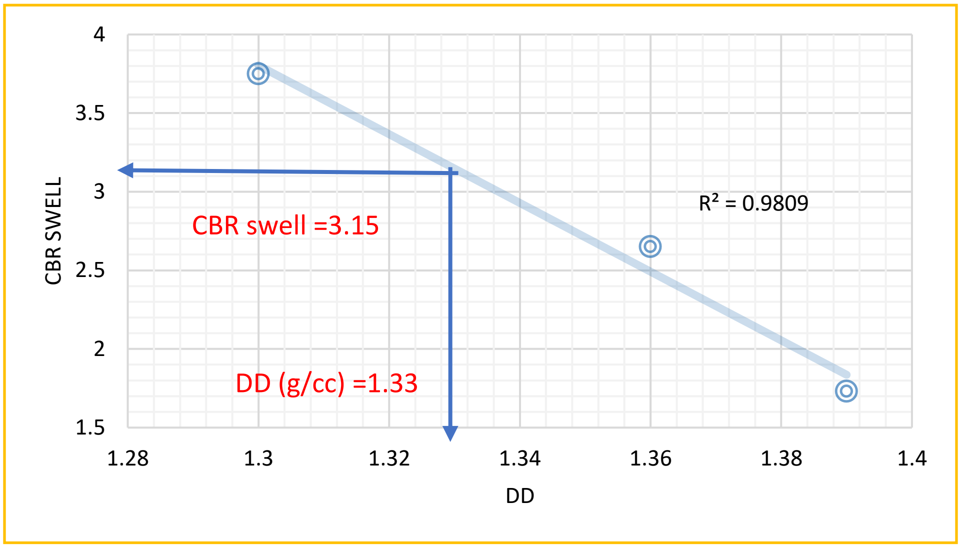

3.5
3
2.5
2
1.5
1
0.5
CBR SWELL
1.28
1.3
1.32
1.34
1.36
1.38
1.4
1.42
1.44
1.46
DD(G/CC)
R² = 0.9871
CBR Swell=2.14
DD =1.36

E) 10%CDWP+2.5%JGTF+SS2

Penetration and load data after 96-hours soaking						
	Number of Blows					
	65 blows		30 blows		10blows	
Penetration (mm)	Load (kn)	CBR %	Load (kn)	CBR (%)	Load (kn)	CBR (%)
@2.54	1.023	7.67	0.912	6.84	0.701	5.25
@5.08	1.482	7.41	1.351	6.76	1.052	5.26
MMDD (g/cc)	1.40					
Dry Density at 95% of MDD	1.33					
No of Blows	65	30	10			
CBR values (%)	7.67	6.84	5.26			
CBR swell (%)	1.73	2.65	3.75			
DDBS (g/cc)	1.39	1.36	1.30			
CBR (%) at 95% MDD and CBR swell (%)	5.91 and 3.15 respectively					

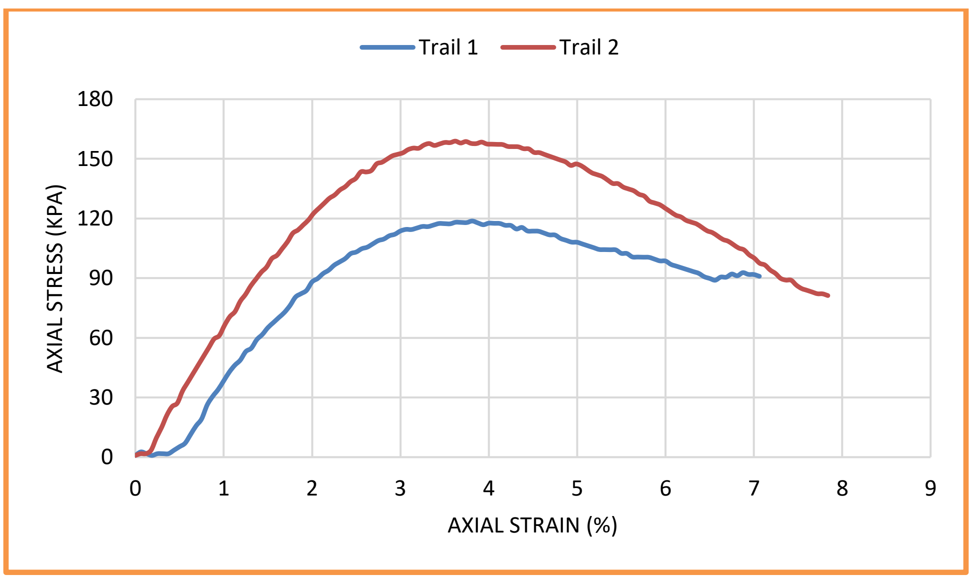






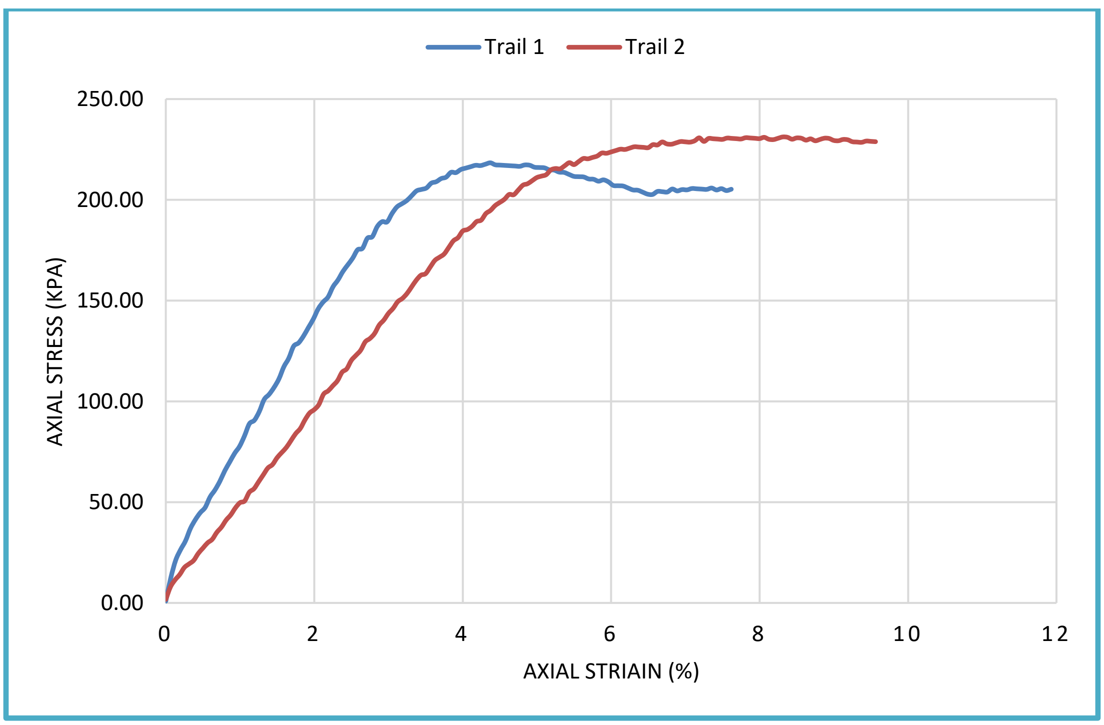
Appendix C-3: Unconfined Compression strength Test Data Analysis According to ASTM D-2166 (Immediate): SS2 Stabilized with CDWP and JGTF

A) UCS test result summary data of SS2+ 10% CDW+0.5%JGTF (immediate test)



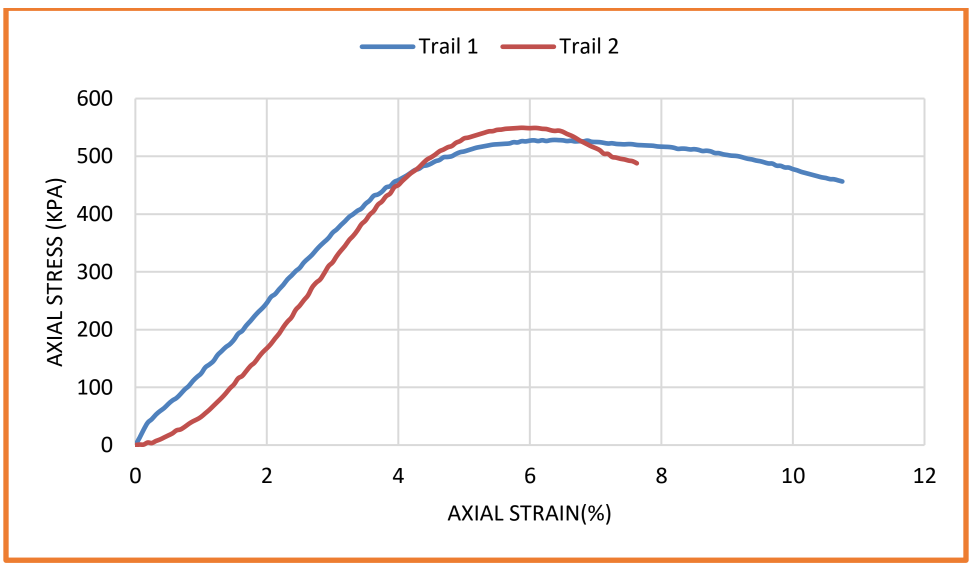
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Immediate) (SS2 + 10% CDW+0.5%JGTF)	**Remolded** (Immediate) (SS2 + 10% CDW+0.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.2
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	93
Axial stress (qu) (kpa)	118.74	158.91
Shear stress (Su) or Cohesion (C) (kpa)	59.37	79.455
Corrected Area at peak qu (cm^2)	11.791	11.767
Load at peak qu, (N)	140	187

B) UCS test result summary data of SS2+ 15% CDW+1%JGTF (immediate test)



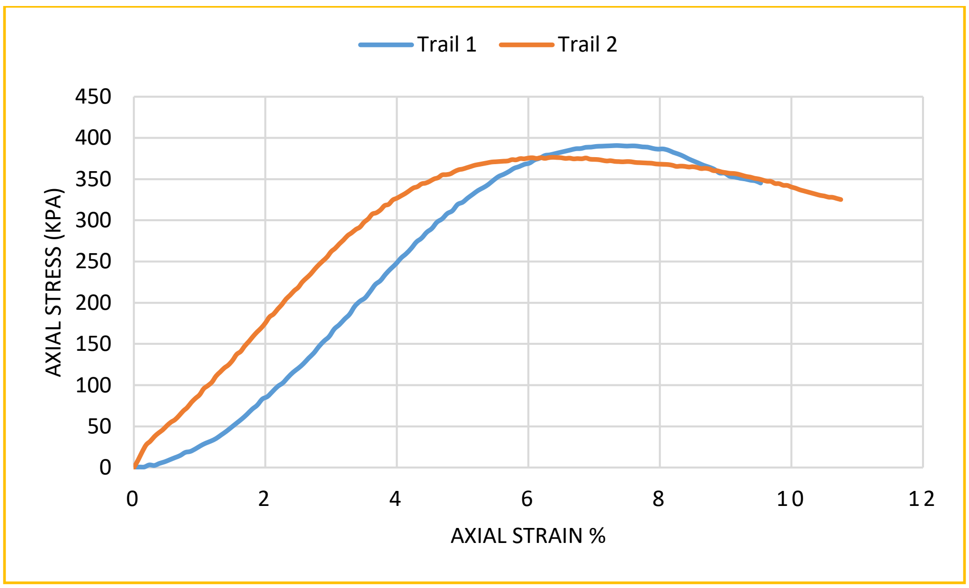
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Immediate) (SS2 + 15% CDW+1%JGTF)	**Remolded** (Immediate) (SS2 + 15% CDW+1%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.2
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	93
Axial stress (qu) (kpa)	218.39	231.22
Shear stress (Su) or Cohesion (C) (kpa)	109.2	115.61
Corrected Area at peak qu (cm^2)	11.860	12.369
Load at peak qu, (N)	259	286

C) UCS test result summary data of SS2+ 20% CDW+1.5%JGTF (immediate test)



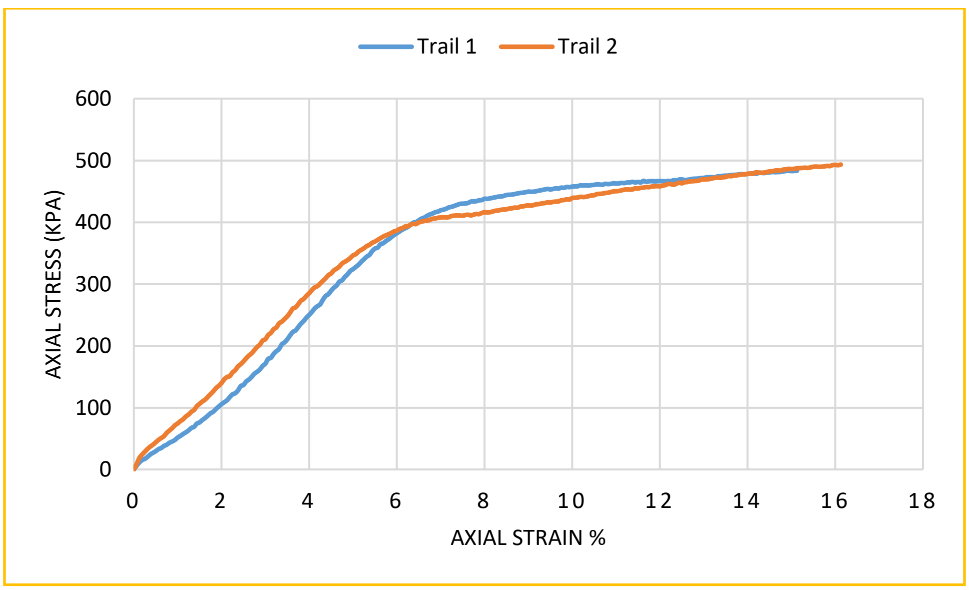
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Immediate) (SS2 + 20% CDW+1.5%JGTF)	**Remolded** (Immediate) (SS2 + 20% CDW+1.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.2
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	93
Axial stress (qu) (kpa)	528.34	549.42
Shear stress (Su) or Cohesion (C) (kpa)	264.17	274.71
Corrected Area at peak qu (cm^2)	12.113	12.049
Load at peak qu, (N)	640	662

D) UCS test result summary data of SS2+ 10% CDW+2%JGTF (immediate test)



Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Immediate) (SS2 + 10% CDW+2%JGTF)	**Remolded** (Immediate) (SS2 + 10% CDW+2%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.2
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	93
Axial stress (qu) (kpa)	390.76	376.44
Shear stress (Su) or Cohesion (C) (kpa)	195.38	188.22
Corrected Area at peak qu (cm^2)	12.240	12.113
Load at peak qu, (N)	478.30	456

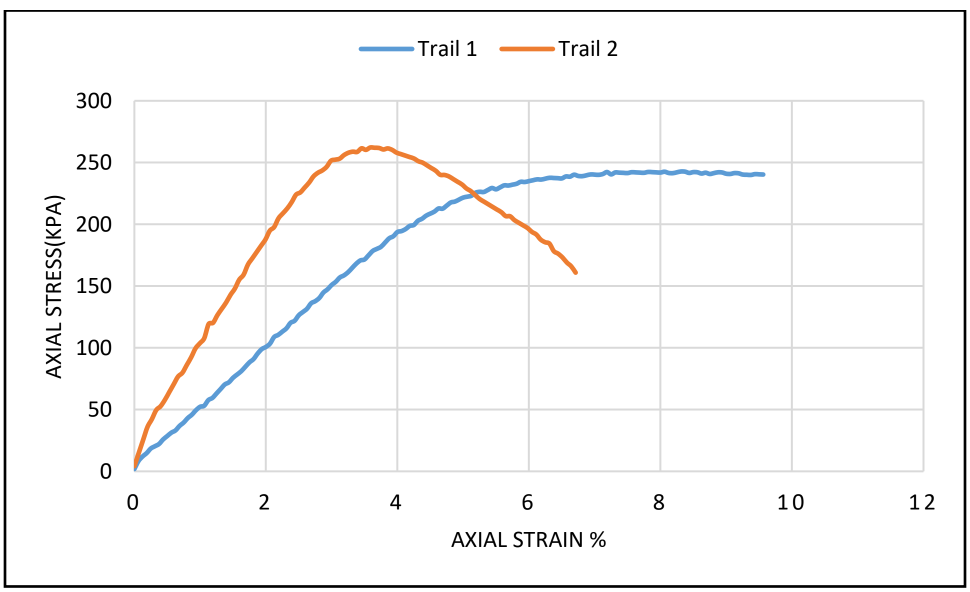
F) UCS test result summary data of SS2+ 15% CDW+2.5%JGTF (immediate test)



Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Immediate) (SS2 + 15% CDW+2.5%JGTF)	**Remolded** (Immediate) (SS2 + 15% CDW+2.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.2
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	93
Axial stress (qu) (kpa)	482.85	486.08
Shear stress (Su) or Cohesion (C) (kpa)	241.425	243.11
Corrected Area at peak qu (cm^2)	13.343	13.343
Load at peak qu, (N)	644.25	648.56

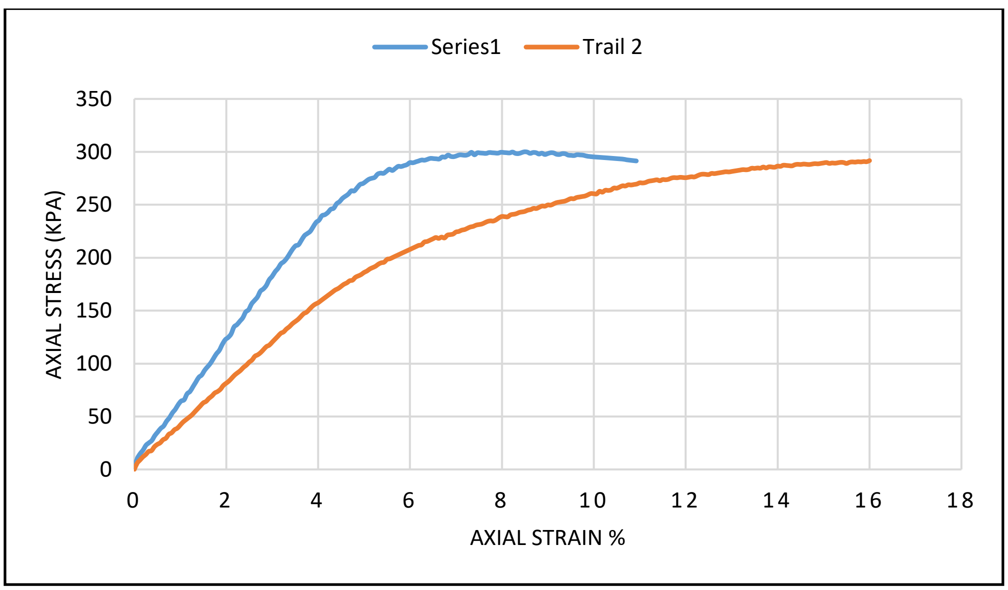
Appendix C-4: Unconfined Compression strength Test Data Analysis According to ASTM D-2166 (Seven day cured): SS2 Stabilized with CDWP and JGTF

A) UCS test result summary data of SS2+ 20% CDW+0.5%JGTF (Seven day cured test)



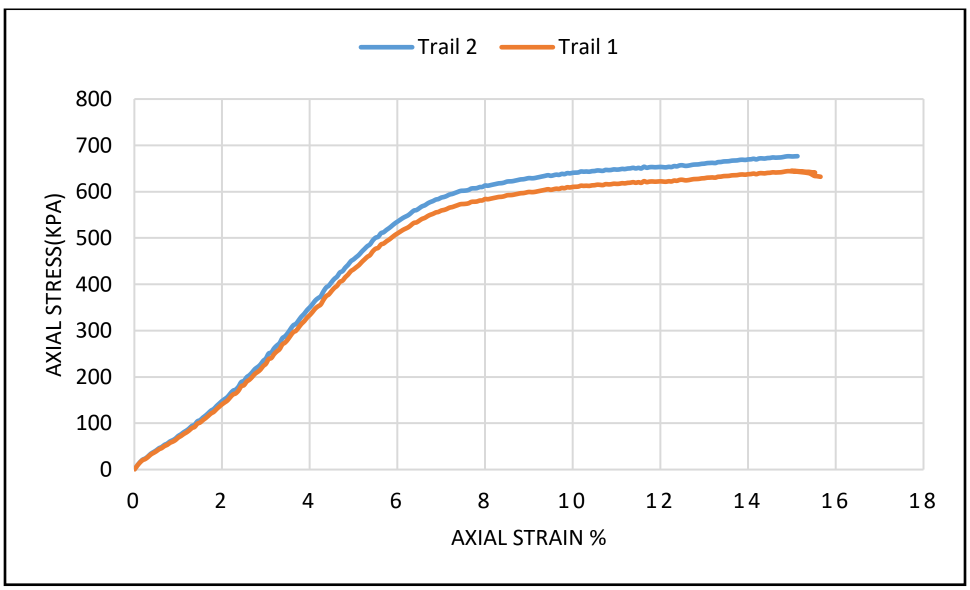
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Seven day cured) (SS2 + 20% CDW+0.5%JGTF)	**Remolded** (seven day cured) (SS2 + 20% CDW+0.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	242.78	262.17
Shear stress (Su) or Cohesion (C) (kpa)	121.213	131.09
Corrected Area at peak qu (cm^2)	12.369	11.763
Load at peak qu, (N)	300.3	308.4

B) UCS test result summary data of SS2+ 10% CDW+1%JGTF (Seven day cured test)



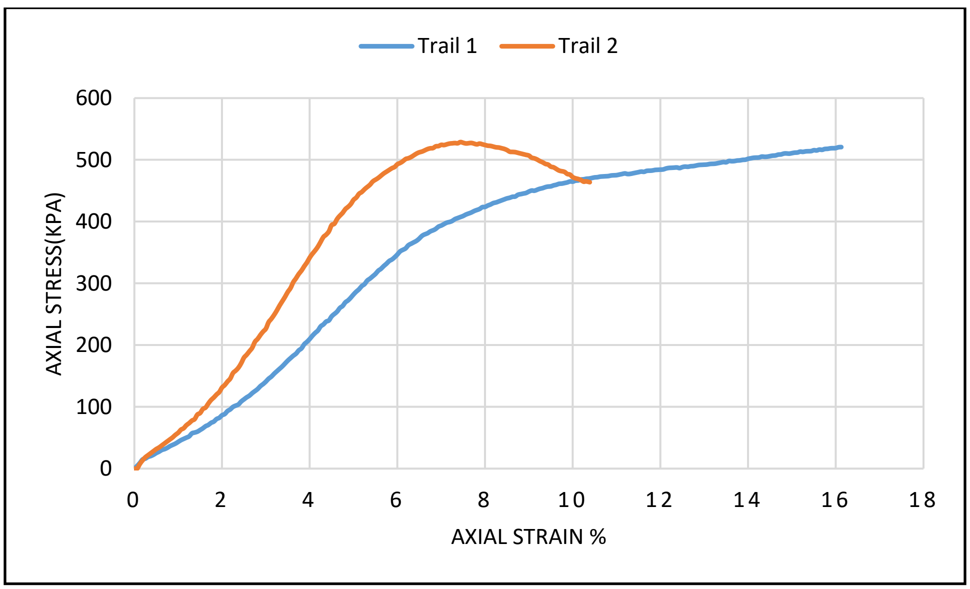
Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Seven day cured) (SS2 + 10% CDW+1%JGTF)	**Remolded** (seven day cured) (SS2 + 10% CDW+1%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	300.01	289.49
Shear stress (Su) or Cohesion (C) (kpa)	150	144.75
Corrected Area at peak qu (cm^2)	12.393	13.343
Load at peak qu, (N)	371.8	386.25

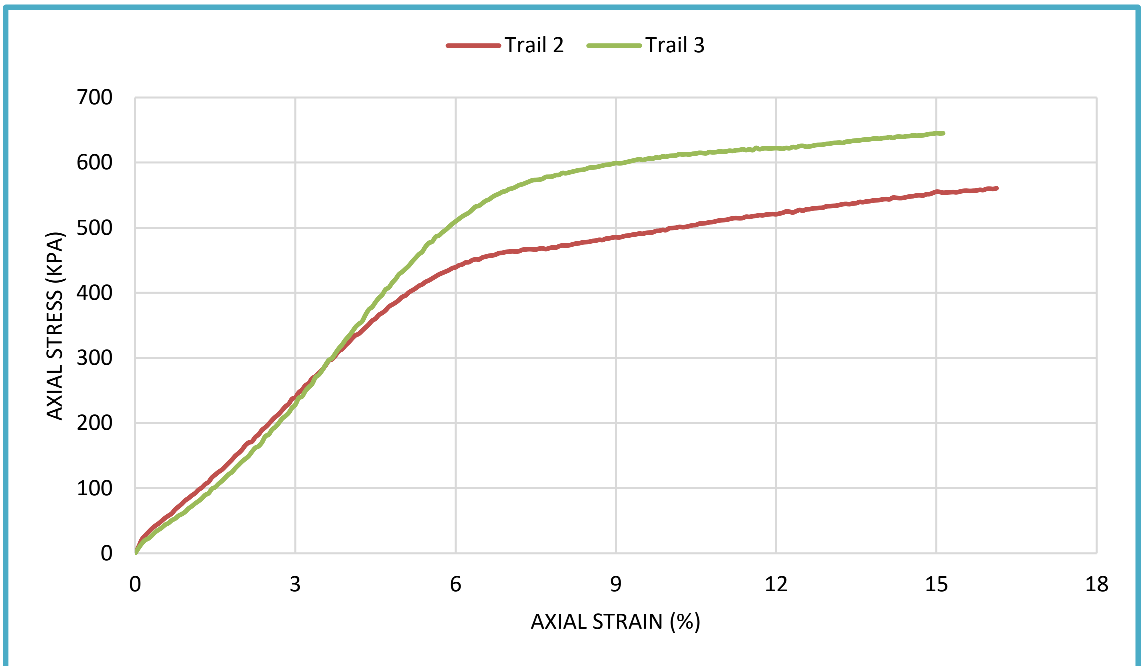
C) UCS test result summary data of SS2+ 20% CDW+1.5%JGTF (Seven day cured test)



Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Seven day cured) (SS2 + 20% CDW+1.5%JGTF)	**Remolded** (seven day cured) (SS2 + 20% CDW+1.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	645.30	676.00
Shear stress (Su) or Cohesion (C) (kpa)	322.65	338
Corrected Area at peak qu (cm^2)	13.343	13.343
Load at peak qu, (N)	861	901.95

D) UCS test result summary data of SS2+ 15% CDW+2%JGTF (Seven day cured test)





Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Seven day cured) (SS2 + 15% CDW+2%JGTF)	**Remolded** (seven day cured) (SS2 + 15% CDW+2%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	510.85	528.79
Shear stress (Su) or Cohesion (C) (kpa)	255.43	264.4
Corrected Area at peak qu (cm^2)	13.343	12.25
Load at peak qu, (N)	681.6	648

E) UCS test result summary data of SS2+ 20% CDW+2.5%JGTF (Seven day cured test)

Sample information	Trial number: 01	Trial number:02
	Sample Type	
	Remolded (Seven day cured) (SS2 + 20% CDW+2.5%JGTF)	**Remolded** (seven day cured) (SS2 + 20% CDW+2.5%JGTF)
Sample Shape	cylindrical	cylindrical
Sample Height (cm)	8	8.3
Sample Diameter (cm)	3.8	3.8
Area (cm^2)	11.341	11.341
Volume (cm^3)	90.73	94.13
Axial stress (qu) (kpa)	645.30	555.37
Shear stress (Su) or Cohesion (C) (kpa)	255.43	264.4
Corrected Area at peak qu (cm^2)	13.343	13.343
Load at peak qu, (N)	861	742

0.5%JGTF+15%CDW +SS2 (immediate)

	Trail 1				Trail 2			
strain, %	AC (cm2)	Load (N)	stress (Kpa)		strain, %	AC (cm2)	Load (N)	stress (Kpa)
0	11.341	0.00	0.00		0	11.34	0.00	0.00
0.0625	11.348	4.87	4.29		0.05625	11.35	3.65	3.22
0.125	11.355	10.29	9.06		0.1125	11.35	7.71	6.79
0.1875	11.362	17.87	15.72		0.16875	11.36	13.40	11.79
0.25	11.370	24.90	21.90		0.225	11.37	18.68	16.43
0.3125	11.377	36.27	31.88		0.28125	11.37	27.20	23.92
0.375	11.384	41.69	36.62		0.3375	11.38	31.26	27.47
0.4375	11.391	48.72	42.77		0.39375	11.39	36.54	32.09
0.5	11.398	52.51	46.07		0.45	11.39	39.38	34.57
0.5625	11.405	60.63	53.16		0.50625	11.40	45.47	39.89
0.625	11.412	66.59	58.35		0.5625	11.41	49.94	43.79
0.6875	11.420	71.46	62.58		0.61875	11.41	53.60	46.96
0.75	11.427	79.58	69.64		0.675	11.42	59.69	52.27
0.8125	11.434	83.37	72.91		0.73125	11.42	62.53	54.73
0.875	11.441	89.32	78.07		0.7875	11.43	66.99	58.61
0.9375	11.448	94.74	82.75		0.84375	11.44	71.05	62.12
1	11.456	98.53	86.01		0.9	11.44	73.90	64.57
1.0625	11.463	104.48	91.15		0.95625	11.45	78.36	68.43

1.125	11.470	109.90	95.81
1.1875	11.477	112.60	98.11
1.25	11.485	118.56	103.23
1.3125	11.492	121.81	106.00
1.375	11.499	128.30	111.57
1.4375	11.507	131.55	114.33
1.5	11.514	133.17	115.66
1.5625	11.521	137.51	119.35
1.625	11.528	140.75	122.09
1.6875	11.536	145.08	125.76
1.75	11.543	147.79	128.03
1.8125	11.551	152.12	131.70
1.875	11.558	155.37	134.43
1.9375	11.565	158.08	136.69
2	11.573	161.33	139.41
2.0625	11.580	165.11	142.58
2.125	11.587	167.82	144.83
2.1875	11.595	171.07	147.54
2.25	11.602	174.32	150.25
2.3125	11.610	177.02	152.48
2.375	11.617	180.27	155.18
2.4375	11.624	182.44	156.94
2.5	11.632	184.06	158.24
2.5625	11.639	187.31	160.93
2.625	11.647	190.02	163.15
2.6875	11.654	191.10	163.97
2.75	11.662	192.18	164.79
2.8125	11.669	194.89	167.01
2.875	11.677	196.51	168.29
2.9375	11.684	198.68	170.04
3	11.692	199.22	170.39
3.0625	11.699	200.30	171.20
3.125	11.707	200.30	171.09
3.1875	11.715	201.39	171.91
3.25	11.722	200.84	171.33
3.3125	11.730	201.39	171.69
3.375	11.737	200.30	170.65
3.4375	11.745	200.84	171.00
3.5	11.752	200.84	170.89
3.56	11.760	202.47	172.17
3.625	11.768	201.93	171.60

1.0125	11.46	82.43	71.94
1.06875	11.46	84.45	73.67
1.125	11.47	88.92	77.52
1.18125	11.48	91.36	79.60
1.2375	11.48	96.23	83.80
1.29375	11.49	98.66	85.87
1.35	11.50	99.88	86.88
1.40625	11.50	103.13	89.66
1.4625	11.51	105.56	91.72
1.51875	11.52	108.81	94.49
1.575	11.52	110.84	96.20
1.63125	11.53	114.09	98.96
1.6875	11.54	116.53	101.01
1.74375	11.54	118.56	102.72
1.8	11.55	121.00	104.77
1.85625	11.56	123.83	107.16
1.9125	11.56	125.87	108.86
1.96875	11.57	128.30	110.90
2.025	11.58	130.74	112.94
2.08125	11.58	132.77	114.63
2.1375	11.59	135.20	116.67
2.19375	11.60	136.83	118.00
2.25	11.60	138.05	118.98
2.30625	11.61	140.48	121.01
2.3625	11.62	142.52	122.69
2.41875	11.62	143.33	123.32
2.475	11.63	144.14	123.94
2.53125	11.64	146.17	125.62
2.5875	11.64	147.38	126.59
2.64375	11.65	149.01	127.92
2.7	11.66	149.42	128.19
2.75625	11.66	150.23	128.81
2.8125	11.67	150.23	128.73
2.86875	11.68	151.04	129.36
2.925	11.68	150.63	128.93
2.98125	11.69	151.04	129.21
3.0375	11.70	150.23	128.44
3.09375	11.70	150.63	128.71
3.15	11.71	150.63	128.63
3.2063	11.72	151.85	129.60
3.2625	11.72	151.45	129.18

3.6875	11.775	201.93	171.49		3.31875	11.73	151.45	129.11
3.75	11.783	199.76	169.53		3.375	11.74	149.82	127.64
3.8125	11.791	198.14	168.05		3.43125	11.74	148.61	126.54
3.875	11.798	194.35	164.73		3.4875	11.75	145.76	124.04
3.9375	11.806	190.56	161.41		3.54375	11.76	142.92	121.55
4	11.814	188.39	159.47		3.6	11.76	141.29	120.10
4.0625	11.821	186.23	157.54		3.65625	11.77	139.67	118.65
4.125	11.829	182.98	154.69		3.7125	11.78	137.24	116.51
4.1875	11.837	180.81	152.75		3.76875	11.79	135.61	115.06

10%CDW+2%JGT+SS2

	Trail 1				Trail 2			
strain, %	Load (N)	AC (cm2)	stress(kpa)		strain, %	Load (N)	AC (cm2)	stress(kpa)
0.000	0.000	11.341	0.0		0.000	0.00	11.341	0.00
0.000	0.000	11.341	0.0		0.063	9.98	11.348	8.79
0.078	0.723	11.350	0.6		0.125	21.38	11.355	18.82
0.156	0.723	11.359	0.6		0.188	31.35	11.362	27.59
0.234	3.613	11.368	3.2		0.250	36.34	11.370	31.96
0.313	2.890	11.377	2.5		0.313	42.75	11.377	37.58
0.391	5.780	11.386	5.1		0.375	47.74	11.384	41.93
0.469	7.948	11.395	7.0		0.438	52.01	11.391	45.66
0.547	10.838	11.404	9.5		0.500	57.71	11.398	50.63
0.625	13.728	11.412	12.0		0.563	62.70	11.405	54.97
0.703	16.618	11.421	14.5		0.625	66.26	11.412	58.06
0.781	20.953	11.430	18.3		0.688	71.96	11.420	63.02
0.859	22.398	11.439	19.6		0.750	78.38	11.427	68.59
0.938	26.010	11.448	22.7		0.813	83.36	11.434	72.91
1.016	30.345	11.458	26.5		0.875	90.49	11.441	79.09
1.094	33.958	11.467	29.6		0.938	96.19	11.448	84.02
1.172	36.848	11.476	32.1		1.000	101.18	11.456	88.32
1.250	40.460	11.485	35.2		1.063	109.73	11.463	95.72
1.328	45.518	11.494	39.6		1.125	114.00	11.470	99.39
1.406	50.575	11.503	44.0		1.188	118.99	11.477	103.67
1.484	56.355	11.512	49.0		1.250	127.54	11.485	111.05
1.563	62.135	11.521	53.9		1.313	133.24	11.492	115.94
1.641	67.915	11.530	58.9		1.375	138.94	11.499	120.82
1.719	74.418	11.539	64.5		1.438	143.21	11.507	124.46

1.797	81.643	11.549	70.7
1.875	87.423	11.558	75.6
1.953	96.093	11.567	83.1
2.031	99.705	11.576	86.1
2.109	106.930	11.586	92.3
2.188	114.155	11.595	98.5
2.266	119.213	11.604	102.7
2.344	127.160	11.613	109.5
2.422	134.385	11.623	115.6
2.500	140.165	11.632	120.5
2.578	146.668	11.641	126.0
2.656	154.615	11.651	132.7
2.734	161.840	11.660	138.8
2.813	171.233	11.669	146.7
2.891	179.180	11.679	153.4
2.969	185.683	11.688	158.9
3.047	196.520	11.698	168.0
3.125	203.023	11.707	173.4
3.203	210.970	11.716	180.1
3.281	218.195	11.726	186.1
3.359	229.755	11.735	195.8
3.438	236.980	11.745	201.8
3.516	242.038	11.754	205.9
3.594	251.430	11.764	213.7
3.672	261.545	11.773	222.1
3.750	267.325	11.783	226.9
3.828	276.718	11.793	234.7
3.906	284.665	11.802	241.2
3.984	291.890	11.812	247.1
4.063	300.560	11.821	254.3
4.141	307.063	11.831	259.5
4.219	315.010	11.841	266.0
4.297	324.403	11.850	273.7
4.375	330.183	11.860	278.4
4.453	338.853	11.870	285.5
4.531	344.633	11.879	290.1
4.609	354.025	11.889	297.8
4.688	359.083	11.899	301.8
4.766	367.030	11.909	308.2
4.844	371.365	11.918	311.6
4.922	380.758	11.928	319.2

1.500	149.63	11.514	129.95
1.563	158.18	11.521	137.29
1.625	162.45	11.528	140.91
1.688	170.29	11.536	147.62
1.750	176.70	11.543	153.08
1.813	183.83	11.551	159.15
1.875	190.24	11.558	164.60
1.938	195.94	11.565	169.42
2.000	203.06	11.573	175.47
2.063	211.61	11.580	182.74
2.125	215.89	11.587	186.31
2.188	223.01	11.595	192.34
2.250	229.43	11.602	197.74
2.313	237.26	11.610	204.37
2.375	242.96	11.617	209.14
2.438	249.38	11.624	214.53
2.500	254.36	11.632	218.68
2.563	262.20	11.639	225.27
2.625	267.90	11.647	230.02
2.688	273.60	11.654	234.76
2.750	280.73	11.662	240.72
2.813	287.14	11.669	246.06
2.875	292.84	11.677	250.78
2.938	298.54	11.684	255.50
3.000	306.38	11.692	262.04
3.063	311.36	11.699	266.13
3.125	317.78	11.707	271.44
3.188	323.48	11.715	276.13
3.250	329.89	11.722	281.42
3.313	334.16	11.730	284.89
3.375	339.15	11.737	288.95
3.438	342.71	11.745	291.80
3.500	349.84	11.752	297.67
3.563	354.83	11.760	301.72
3.625	361.95	11.768	307.58
3.688	364.09	11.775	309.19
3.750	368.36	11.783	312.62
3.813	374.78	11.791	317.86
3.875	376.91	11.798	319.46
3.938	383.33	11.806	324.69
4.000	386.18	11.814	326.89

5.000	384.370	11.938	322.0
5.078	390.873	11.948	327.1
5.156	396.653	11.958	331.7
5.234	402.433	11.968	336.3
5.313	406.768	11.977	339.6
5.391	411.825	11.987	343.5
5.469	418.328	11.997	348.7
5.547	424.108	12.007	353.2
5.625	427.720	12.017	355.9
5.703	432.055	12.027	359.2
5.781	437.113	12.037	363.1
5.859	440.003	12.047	365.2
5.938	443.615	12.057	367.9
6.016	445.783	12.067	369.4
6.094	450.840	12.077	373.3
6.172	453.730	12.087	375.4
6.250	458.065	12.097	378.7
6.328	459.510	12.107	379.5
6.406	461.678	12.117	381.0
6.484	463.845	12.128	382.5
6.563	466.013	12.138	383.9
6.641	468.180	12.148	385.4
6.719	470.348	12.158	386.9
6.797	471.070	12.168	387.1
6.875	473.238	12.178	388.6
6.953	473.960	12.189	388.9
7.031	475.405	12.199	389.7
7.109	476.128	12.209	390.0
7.188	476.850	12.219	390.2
7.266	477.573	12.230	390.5
7.344	478.295	12.240	390.8
7.422	478.295	12.250	390.4
7.500	478.295	12.261	390.1
7.578	479.018	12.271	390.4
7.656	479.018	12.281	390.0
7.734	478.295	12.292	389.1
7.813	478.295	12.302	388.8
7.891	476.850	12.313	387.3
7.969	476.128	12.323	386.4
8.047	476.850	12.334	386.6
8.125	475.405	12.344	385.1

4.063	389.74	11.821	329.69
4.125	393.30	11.829	332.49
4.188	397.58	11.837	335.88
4.250	401.85	11.845	339.27
4.313	403.99	11.852	340.85
4.375	408.26	11.860	344.23
4.438	409.69	11.868	345.21
4.500	412.54	11.876	347.38
4.563	416.10	11.883	350.15
4.625	418.24	11.891	351.72
4.688	422.51	11.899	355.09
4.750	423.23	11.907	355.45
4.813	424.65	11.915	356.41
4.875	428.21	11.922	359.17
4.938	431.06	11.930	361.32
5.000	432.49	11.938	362.28
5.063	434.63	11.946	363.83
5.125	436.76	11.954	365.38
5.188	438.90	11.962	366.92
5.250	440.33	11.970	367.87
5.313	441.75	11.977	368.82
5.375	443.18	11.985	369.76
5.438	444.60	11.993	370.71
5.500	445.31	12.001	371.06
5.563	446.03	12.009	371.40
5.625	446.74	12.017	371.75
5.688	447.45	12.025	372.10
5.750	449.59	12.033	373.63
5.813	449.59	12.041	373.38
5.875	451.73	12.049	374.91
5.938	451.73	12.057	374.66
6.000	453.15	12.065	375.59
6.063	453.86	12.073	375.93
6.125	453.15	12.081	375.09
6.188	454.58	12.089	376.02
6.250	453.86	12.097	375.18
6.313	455.29	12.105	376.11
6.375	456.00	12.113	376.44
6.438	456.00	12.121	376.19
6.500	456.00	12.130	375.94
6.563	455.29	12.138	375.10

8.203	472.515	12.355	382.5		6.625	456.00	12.146	375.44
8.281	470.348	12.365	380.4		6.688	455.29	12.154	374.60
8.359	467.458	12.376	377.7		6.750	456.00	12.162	374.94
8.438	463.845	12.386	374.5		6.813	456.00	12.170	374.68
8.516	460.955	12.397	371.8		6.875	457.43	12.178	375.60
8.594	458.065	12.407	369.2		6.938	456.00	12.187	374.18
8.672	455.175	12.418	366.5		7.000	456.00	12.195	373.93
8.750	453.008	12.429	364.5		7.063	456.00	12.203	373.68
8.828	450.118	12.439	361.9		7.125	455.29	12.211	372.84
8.906	445.060	12.450	357.5		7.188	454.58	12.219	372.01
8.984	445.060	12.461	357.2		7.250	455.29	12.228	372.34
9.063	440.725	12.471	353.4		7.313	454.58	12.236	371.51
9.141	440.003	12.482	352.5		7.375	454.58	12.244	371.26
9.219	438.558	12.493	351.0		7.438	454.58	12.252	371.01
9.297	437.835	12.504	350.2		7.500	455.29	12.261	371.34
9.375	436.390	12.514	348.7		7.563	455.29	12.269	371.09