JIMMA UNIVERSITY
JIMMA INSTITUTE OF TECHNOLOGY
SCHOOL OF GRADUATE STUDIES
FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING
HIGHWAY ENGINEERING STREAM

STABILIZATION OF EXPANSIVE SUBGRADE SOIL USING PUMICE AND CRUSHED STONE DUST: (A CASE JIMMA TO CHIDA ROAD SEGMENT)

A Final Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Highway Engineering)

by:

Kebede Dedino Dercha

October, 2021
Jimma, Ethiopia

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Advisor: Dr. Ing. Fekadu Fufa
Co-Advisor: Ing. Markos Tsegaye (Ph. D candidate)

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(A Case of Jimma to Chida Road Segment)

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Kebede Dedino Dercha

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DECLARATION

I, the undersigned, declare that this thesis entitled "**Stabilization of Expansive Subgrade Soil Using Pumice and Crushed Stone Dust: (A Case Jimma to Chida Road Segment)"** is my original work, and has not been presented by any other person for any award of a degree in this or any other University, and all sources of material used for this thesis have been duly acknowledged. There, whatever the result of my thesis final defense based on the criteria as evaluated by examiners will be accepted by good faith.

Candidate

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Name Signature Date

As master's research advisors, hereby certify that we have read and evaluated this thesis paper prepared under our guidance, by **Kebede Dedino Dercha** entitled **"Stabilization of Expansive Subgrade Soil Using Pumice and Crushed Stone Dust: (A Case Jimma to Chida Road Segment)"** and recommend would be accepted as a fulfilling requirement for the Degree of Masters of Science in Highway Engineering.

Advisor: Dr. Ing. Fekadu Fufa _______________ ______/ ______/ ______

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Abstract

Expansive soils are the most abundant soils in the world, and in Ethiopia, this expansive soil covers more than 40% of the total land area. Road construction along in such types of soils causes significant damage to road pavement due to its characteristics of high shrinkage, low bearing capacity, and high swelling potential. These problems need a wider application of cost-effective and environmentally friendly technology to improve this type of soil properties to be adopted in the current highway construction industries.

The engineering properties of expansive soil can be improved by one of the methods, through stabilization. Therefore, this study focused on stabilizing expansive subgrade soil using pumice blended with crushed stone dust. The purposive sampling technique which is a non-probability method was used. Samples of soil were collected from Jimma to Chida road below 1.5 m from the original ground level. Pumice from Konta Special district and crushed stone dust from crusher plant. The research was done on an expansive soil sample from Jimma to Chida road, stabilized with the addition of the equal amount of pumice and crushed stone dust by varying content of stabilizers in a steeped concentration of 10, 20, 30, and 40% by dry weight of the soil, was used to stabilize the natural soil. For the analysis of the effect of the stabilizer on expansive soil, a comparison was made on the engineering properties of the native soil and stabilized soil.

The engineering properties of natural soil shows that the sampled soils were very weak clay soil and the parameter, CBR (%), = 1.1 and 1.7 and PI (%) = 53 and 41 for sample-1 and sample -2 respectively. The result from stabilization shows that the addition of pumice and crushed stone dust reduces linear shrinkage, free swell, PI, Swelling, and the OMC, increase in MDD& CBR with an increase of pumice and crushed stone dust by considered amount increase of 5, 10, 15, and 20 % of each pumice and crushed stone dust by dry weight of soil for sample -1 and sample -2 respectively. About 26.77% amount of cost savings is also possible when the expansive subgrade soil is stabilized with pumice and crushed stone dust.

Stabilizing expansive subgrade soil using pumice blended with crushed stone dust is an economical and effective option for the improvement of expansive subgrade soil properties.

Keywords: *expansive subgrade soil; pumice; crushed stone dust; Stabilization; engineering property; and cost.*

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AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt concrete
Asl	Average Sea Level
CBR	California bearing ratio
CES	Central statistics agency
cm^3	Cubic centimeter
CSD	Crushed stone dust
ERA	Ethiopian Road Authority
ESS	Expansive Subgrade Soil
FRS	Free swell
g	Gram
G_s	Specific gravity
ha	Hectares
hr	Hour
JIT	Jimma Institute of Technology
km	Kilometer
kpa	kilopascal
LL	Liquid Limit
m	Meter
MC	Moisture contents
MDD	Maximum Dry Density
Mpa	Mega pascal
NMC	natural moisture content
NMSA	National meteorological services agency
No.	Number
NRS	National Regional State
OMC	Optimum Moisture Content
P	Pumice
PDM	Pavement design manual
PI	Plastic Index
PL	Plastic Limit
SNNPRS	South Nation and National People Regional State

SP	Swelling Pressure
UCS	Unconfined Compressive Strength
UK	United Kingdom
USA	United States of America

CHAPTER ONE

INTRODUCTION

1.1 Background

Worldwide the availability of natural construction materials within reasonable hauling distance is one of the major factors that have a direct impact on the investment cost of road projects. In areas where natural construction materials are readily available, roads can be constructed on a sound economic basis. However, the availability of construction material in some regions, natural construction materials are either not available or do not fulfill the quality requirements of road construction materials. Problems associated with these construction materials have been reported in Africa, Australia, Europe, India, and South America, the United States as well as some regions in Canada. The damage due to the expansive soils every year is expected to be $1 billion in the USA, £150 million in the UK, and many billions of pounds worldwide annually (Firoozi *et al.*, 2017).

In many areas of the tropics, especially Africa and India, tropical expansive soils often known as black cotton soils and are the major problematic soils. The total area covered by black cotton soil is 250 million hectares and Australia, India, Sudan, Chad, and Ethiopia contain more than 80% of the total area of the world. These soils show very strong swelling and shrinkage characteristics under changing moisture conditions (Azam, *et al*., 2013).

All roads, use naturally occurring subgrade soils and rocks as the foundation layers and also as the construction materials. As a result, now day highway engineers are faced with the challenge of using soils and rocks available near the project site, whose properties are often unknown and of variable quality (ERA, 2013).

Most of the time expansive subgrade soil is one of the challenges in which highway engineers are facing in road subgrade construction. Expansive subgrade soils are one of those clays with a high content of montmorillonite. Expansive soils have very high plasticity leading to high rates of expansion on wetting flowed by shrinkage and considerable cracking on drying. The distribution of expansive soil in Ethiopia covers about 40% of the total surface area of the country (Sites, Geremew, and Fayissa, 2019). Therefore, most of the road constructed in expansive soil shows different types of distress and this leads to failure before there design life due to the swelling- shrinkage properties of the expansiveness of the soil. Thus, the selection process of route corridor influences the pavement structure and the construction costs, a thorough investigation should be done on the characteristics of subgrade soil.

Generally, treatment of unsuitable subgrade soils is accomplished by modification, stabilization, or excavation and replacement.

Soil stabilization improves the engineering properties of soils and thus making expansive soil more stable. It is essential when the soil accessible for construction is not suitable for the anticipated purpose (Vaijwade *et al.*, 2018). The process of soil stabilization helps to achieve the required properties in soil which are needed for the construction work. From the beginning of the construction work, the necessity of enhancing soil properties has come to the light. So, to improve the engineering properties of soils and make the soil suitable for engineering works soil stabilization is needed.

The main objective of soil stabilization is to improve the strength and stability of the soils and mainly to lower the construction cost. Stabilization with a stabilizing agent, and minimizing of water content change (implementing measure to prevent water infiltration). The stability and bearing capacity of a soil depend on the shear strength, which is directly proportional to the type and conditions of the soil (Vaijwade *et al.*, 2018). Removal or and replacement, as the name indicates, involves removal of the unsuitable subgrade soil and replacement with a select material (usually granular backfill). According to (ERA, 2013), the manual proposes alignment improvement (avoiding the area of expansive soil), excavation/soil replacement (replacing expansive soil with good quality material along the road route).

Many researchers have worked to improve or stabilize its expansive behavior with various admixtures. They have used waste materials as admixtures and showed their best results in soil stabilization (Mudgal *et al.*, 2014). According to the researchers crushed stone dust is material obtained from aggregate crushing industries, use of such stone dust materials creates a lot of problems in the environment and the public due to excess storage and dust accumulation. Considering this aspect this research was conducted on expansive soil by mixing it with locally available crushed stone dust. The use of pumice for the stabilization of clayey subgrades has economic, environmental, and engineering benefits. Locally available pumice has little commercial value. Because it is readily available, the transportation costs involved in using this resource are minimal (Saltan and Keskin, 2015).

Pumice has been used to stabilize expansive soil due to its mineralogical content which will act to have a cementitious property and will undergo a pozzolanic reaction and able to alter the expansive soil property to a certain degree. The natural pumice material excavated from quarry source and passing 4.75mm sieve was used as stabilizing material. In the process of testing the soil-pumice mix, the pumice material was subjected to compaction effort which causes crushing of the pumice to a certain extent (Saltan and Keskin, 2015).

Therefore, this research investigated that the effects of pumice and crushed stone dust for the improvement of expansive subgrade soil. Also, the research investigated that the cost analysis and cost comparison of stabilization using pumice and crushed stone dust with respect to the currently used cart away expansive soil and replacement this expansive soil by quality selected material from the quarry site. It proves as it reduces environmental pollution.

1.2 Statement of problem

A huge amount of investment is spent every year to construct new pavements and maintain existing pavements. For new pavement construction, the overall cost depends strongly on the performance of on subgrade. Subgrades in the selected routes may consist of expansive soil. This expansive soil causes destructive damage to the pavements constructs on it. The damage due to the expansive soil every year is expected to be a billion dollars and pounds worldwide annually (Firoozi *et al.*, 2017). The areal coverage and damage due to expansive soils in Ethiopia are much more. specifically, the southwestern part of Ethiopia including the Jimma zone and Konta Special district were affected by this expansive soil. This damage is due to the volume changing of expansive soil when subjected to moisture variation (ERA, 2013).

The availability of high-quality selective materials and increased costs associated with the use of high-quality materials, and cart away and replacement of weak subgrade by selected borrow materials, the increased cost of disposal land, acquiring selected quarry sites, and transport costs have led to the need for treatment local expansive soils by new technology used in highway construction.

The best alternative is without changing the expansive subgrade materials, stabilizing or upgrading with the least cost stabilizer is the best and economical way of constructing a subgrade layer. Soil stabilization by adding stabilizing agent is one of the best methods of upgrading the performance of materials. The strength and bearing capacity of the soil are impressively enhanced by soil stabilization through controlled compaction, proportioning, and the expansion of reasonable admixtures (Reddy, *et al*., 2018).

Accordingly, roads between the Jimma zone and Konta Special district experienced many types of failures due to the above-mentioned cases. At present construction, the expansive soil is planned to excavate and replaced with selected quarry materials. Therefore, to prevent the problems, engineers need to stabilize the existing expansive subgrade soils before commencing the construction activities. This research investigated the stabilization of expansive subgrade soil using pumice and crushed stone dust for the improvement and cost-effective method for expansive subgrade soil. That existing natural expansive subgrade soil was improved and suitable for construction by mixing expansive soil with pumice and crushed stone dust as a cost-effective stabilizer. Also, this research investigated that the cost analysis and cost comparison of stabilization with respect to cart away and replacing expansive subgrade soil with selected quality materials from quarry site which is currently used in construction.

1.3 Research Questions

1. What are the engineering properties of natural expansive subgrade soil?
2. What are the effects of the mix of pumice with crushed stone dust on soil engineering properties and comparison with standard and specification?
3. What is the cost-effectiveness of using pumice and crushed stone dust with respect to cart away and borrow fill?

1.4 Objective

1.4.1 General Objective

The general objective of the study is to evaluate the effect of blended pumice-crushed stone dust mix for the improvement of expansive subgrade soil.

1.4.2 Specific Objectives

The specific objectives of the study are:

1. to identify the engineering properties of natural expansive subgrade soil.
2. to investigate the effect of using pumice with crushed stone dust on the engineering properties of expansive subgrade soil, and compare with the standard and specification.
3. to analyze and compare the cost of stabilization with respect to cart away and borrow fill.

1.5 The Significance of the Study

The increasing world population number and peoples demand for infrastructure will increases, acquiring more land for settlement, for other industry and this has a direct impact on the construction industry. Due to the above and other cases, the construction cost and construction material cost increased dramatically today. So, it is important to find other economical and environmentally friendly options. On another side, the extraction of locally naturally available construction material for construction projects creates a significant impact on the environment. Also, the construction industry especially the crusher plant can produce dust which affects the environment we live in. Therefore, the construction techniques implemented to solve the socio-economic problem must be environmentally friendly.

The significance of this research was to use pumice and crushed stone dust as stabilizing material for expansive subgrade soil. It is also used as an alternative stabilizer that is environmentally friendly.

The positive result that can be obtained from this study indicates that pumice and crushed stone dust are used as the stabilizing agent for expansive subgrade by the road contractors, that have strong subgrade layers and good pavement conditions. This research also serves as a reference guide for users, and researchers who want to study further the related area for the application of pumice and crushed stone dust use as an innovative stabilizer.

1.6 Scope of the Study

The scope of this study was limited to the effects of pumice- crushed stone dust mix for the use of stabilization expansive subgrade soil. The finding of this study was limited to two representative samples of expansive subgrade soil from the entire Jimma to Chida road segment. This study investigated that the effect of pumice–crushed stone dust on the engineering properties of expansive soils and cost comparison of soil stabilization with a currently used cart away and replacement with quality material from the quarry site. The study was supported by different works of literature, books, laboratory manuals, and lab experiments. To develop the conclusion and recommendation based on laboratory results after conducting different laboratory tests such as grain size analysis, specific gravity, Atterberg's limit, free swell, liner shrinkage, maximum dry density, optimum moisture content, CBR and CBR swell, and unconfined compressive strength for each percentage of respective pumice and crushed stone dust.

The results were analyzed according to ERA, and AASHTO standards.

1.7 Limitation of the study

The soil classification of national or regional and soil engineering maps does not exist in Ethiopia. Hence, the soil was identified by the laboratory investigation. Also, the finding of the research was limited to selected expansive soil in Jimma to Chida road segment.

CHAPTER TWO

LITERATURE REVIEW

2.1 Road Construction

Since practically all civil engineering works are built either on soil, under the soil, or above the soil. A deep understanding of the soil type, soil nature, and soil characteristics is most valuable, especially to engineers, planners, and decision makers. Road pavements are one of civil engineering work which are layered systems with better quality materials on top where the intensity of stress is high and inferior materials at the bottom where the intensity is low. Construction of roads involves the new construction, reconstruction or upgrading, paving, rehabilitation, and/or reclamation of degraded pavements to achieve a state of good repair and increase road traffic safety. Road construction involves the use of asphalt, liquid asphalt, concrete, soil stabilization, rebar, paving and pavement recycling machines, and other road repair materials. Typical bituminous pavements are composed of wearing course, base course, subbase, capping, and subgrade layers (ERA, 2013).

The road subgrade is the pavement layer on which the overall load of traffic and pavement layers lies. The performance of roads depends on many factors that include the subgrade soil. The subgrade must be able to resist loads transmitted from the pavement structure, the density of traffic, and even the weather conditions to which the road is subjected. The load bearing capacity of subgrade soil is often affected by the degree of compaction, moisture content, and especially soil type (ERA, 2013).

Types of pavement subgrade soil affect the selection of road project routes because of problematic characteristics. In such cases, either the highway route is changed or the layer that has problematic soil is replaced with a new layer that has the proper soil type. Sometimes, these kinds of problems will increase the route length of a road construction project and hence project costs. The selection of an appropriate subgrade stabilization process to improve subgrade resistance to permanent deformation is very important for overall pavement performance (Saltan and Keskin, 2015). A subbase course is one pavement layer that serves as an aid to disperse the load from the base course before transmitting it to the subgrade. The base course which is overlying the subbase course plays a prominent role in the support and dispersion of the traffic loads. The surface course consists of a binder course and a wearing course. The Binder course layer works as a supporting, dispersing traffic load and resists shear, while the topmost layer (wearing course) resists abrasion and prevents skidding.

2.2 Subgrade Soil

The type of subgrade soil is largely determined by the location of the road. However, where the soils within the possible corridor for the road vary significantly in strength from place to place, it is desirable to locate the pavement on the stronger soils if this does not conflict with other constraints. For this reason, amongst others, the pavement engineer should be involved in the route selection process (ERA, 2013).

The subgrade is consisting of the naturally occurring material on which the road is built, or the imported fill material used to create an embankment on which the road pavement is constructed (Schaefer, *et al*., 2008).

Undisturbed soil beneath the pavement is called a natural subgrade. The compacted subgrade is the soil compacted by the controlled movement of heavy compactors. A stable subgrade and properly draining subbase help produce a long-lasting pavement. The subgrade or embankment soil on which pavement is built is the most important part of the pavement structure because it is the layer on which the remainder of the structures are supported and helps to resist the destructive effects of traffic and weather, and it acts as a construction platform for building subsequent pavement layers.

In determining the suitability of a subgrade, the following factors should be considered: general characteristics of the subgrade soil, depth to bedrock, depth to the water table, compaction that can be attained in the subgrade, CBR values of compacted and uncompacted subgrades, presence of weak or soft layers or organics in the subsoil, susceptibility to detrimental frost action or excessive swell (Schaefer *et al*,, 2008).

Subgrade performance generally depends on three basic characteristics: Strength, Moisture content, and Shrinkage and/or swelling.

The strength of the road subgrade for flexible pavements is commonly assessed in terms of the California Bearing Ratio (CBR) and this is dependent on the type of soil, its density, and its moisture content. Direct assessment of the likely strength or CBR of the subgrade soil under the completed road pavement is often difficult to make. Its value, however, can be inferred from an estimate of the density and equilibrium (or ultimate) moisture content of the subgrade together with knowledge of the relationship between strength, density, and moisture content for the soil in question. This relationship must be determined in the laboratory. The density of the subgrade soil can be controlled within limits by compaction at a suitable moisture content at the time of construction (ERA, 2013).

The moisture content (MC) of the subgrade soil is governed by the local climate and the depth of the water table on the road surface. According to ERA (2013) manual, the strength of the subgrade soil is assessed by the type of soil, its density, and moisture content. According to ERA (2013) manual subgrades are classified from S_1 to S_6 based on the California bearing ratio (CBR), and are illustrated in Table 2.2.

Table 2.1 CBR range subgrade class (ERA, 2013)**.**

Serial No.	Class	CBR Range (%)
1	S1	< 3
2	S2	3,4
3	S3	5,6,7
4	S4	8-14
5	S5	15 - 30
6	S6	>30

2.2.1 Desirable properties of subgrade soil

The advantageous properties of subgrade soil as a highway material are stability, incompressibility, permanency of strength, minimum changes in volume due to climate superior drainage, and ease of compaction.

Since pavement design is ultimately an attempt to minimize future pavement distress and, thereby maximizing pavement performance, it is important to understand how geotechnical factors impact this distress. Pavement failure may occur due to intrusion of subgrade soil into the granular base, which results in inadequate drainage and reduced stability, also occur due to excessive loads that cause a shear failure in the subgrade, and especially differential of the subgrade. If the subgrade is saturated, excess pore pressures will develop under traffic loads, resulting in subsequent softening of the subgrade. Under dynamic loading, fines can be pumped up into the subbase and/or base.

The subgrade in the flexible pavement is more vulnerable to failure under the vehicular traffic loading due to the non-uniform distribution of the load from overlying layers and the presence of high moisture contents. This layer gets less emphasis compared to other layers in the pavement, although most of the pavement failure is being caused due to the bearing capacity failure of the subgrade layer. Some subgrade soils, especially clayey soils, have great strength at low moisture content; however, they become very weak and less workable with the increase in water content beyond the optimum value. Such soil should be either replaced with superior quality fill material or treated with a suitable treatment process (Prusinski and Bhattacharja, 1999). The replacement of the subgrade soil might not always be the best option due to the associated hauling cost of the excavated materials as well as the imported quality materials. In some developing regions or even urban areas, the unavailability of the aggregate or the shortage of suitable fill materials makes the replacement of weak subgrade soil uneconomical. In such conditions, the strength/stiffness properties of the existing weak subgrade soil can be improved by the use of proper compaction techniques as well as by using some chemical stabilizers. Portland cement, lime, and fly ash are the most common types of chemical stabilizers used by most states to stabilize the weak subgrades; thus creating a proper working platform and/or subbase layer for pavement construction (Pundir and Prakash, 2015).

2.3 Expansive soil

Expansive soils are those that exhibit particularly large volumetric changes, both shrinkage and swell, due to variations in their moisture content. They exhibit poor bearing capacity (similar to some stability problems). Particular care is needed with such expansive soils and, if the construction in these soils cannot be avoided, earthworks must be designed to minimize subsequent changes in moisture content and consequent volume changes. When the subgrade is particularly expansive soil, it may be necessary to replace the expansive material with non-expansive impermeable soil to the depth affected by seasonal moisture changes. However, the measures to minimize the effect of expansive soils must be both economic and proportionate to the risk of pavement damage and increased maintenance costs (ERA, 2013).

Expansive soils are those which experience significant volume changes associated with changes in water content. These volume changes can either be in the form of swell or shrinkage and are sometimes known as swell–shrink soils. Key aspects that need identification when dealing with expansive soils include soil properties, suction/water conditions, temporal and spatial water content variations that may be generated, for example, by trees, and the geometry/stiffness of foundations and associated structures. Expansive soils can be found both in humid environments where expansive problems occur with soils of high plasticity index and in arid/semi-arid soils where soils of even moderate expansiveness can cause significant damage (Jones and Geological, 2012).

Essentially, expansive soil changes in volume in relation to changes in water content. The focus here is on soils that exhibit significant swell potential and, in addition, shrinkage potential. There are many cases where expansion can occur because of chemically induced changes (e.g., swelling of lime-treated sulfate soils). However, many soils that exhibit swelling and shrinking behavior contain expansive clay minerals, such as smectite, that absorb water. The more of this clay soil contains, the higher its swell potential and the more water it can absorb. As a result, these materials swell and thus increase in volume when they become wet, and shrink when they dry. The more water they absorb, the more their volume increases for the most expansive clays. Fine-grained clay-rich soils can absorb large quantities of water after a rainfall, becoming sticky and heavy. Conversely, they can also become very hard when dry, resulting in shrinking and cracking of the ground. This hardening and softening are known as 'shrink–swell' behavior. Expansions of 10% are not uncommon (Chindris, *et al.,* 2017). The primary problem with expansive soils is that deformations are significantly greater than those that can be predicted using classical elastic and plastic theory. As a result, a number of different approaches have been developed to predict and engineer expansive soils (Vaijwade *et al.*, 2018) (Jones and Geological, 2012).

During construction, the roadbed of expansive soil should be kept moist and covered with earthworks prior to any drying. Attempts to process and compact the soil beyond normal density requirements are not required. Fill material over the expansive soils shall be impermeable soils with a plasticity index of greater than 15% (ERA, 2013).

2.3.1 Expansive soil mineralogy

The predominant mineral in expansive soil is montmorillonite. Its basic structure is the aluminum octahedral sheet sandwiched between two silica tetrahedral sheets. Experience shows that swelling problem arises when the soil contains more than 20% montmorillonite or mixed layer montmorillonite, illite vermiculite (C.Of, *et al*., 2018)

The expansive soils are characterized by the presence of expanding lattice type of clay minerals belonging to the smectite group, montmorillonite being an important member of that group. These clay minerals are characterized by;

- Very weak van der Waals' forces in between the adjacent unit cells of the mineral.
- Appreciable isomorphous substitution during the clay mineral formation, leading to very high negative surface charges.
- Very high cation exchange capacity (i.e., 80 –150 m/100 g)
- Large specific surface (i.e., 400 – 900 m^2/g) (Visalakshi *et al.*, 2018).

2.3.2 Characterization of expansive soils

Various criteria adopted to recognize the presence of expanding lattice type clay minerals in natural soil can be broadly classified into two categories namely, mineralogical identification and inferential testing methods.

1. **Mineralogical Identification Methods: -**

The techniques belonging to this category of methods are X-ray diffraction analysis, differential thermal analysis, dye adsorption, chemical analysis, and scanning electron microscope.

2. **Inferential Testing Methods (Jones and Geological, 2012)**

These methods try to link some of the index properties of fine-grained soils with the soil clay mineralogical composition.

They can be classified as indirect methods and direct methods.

a. Indirect Methods

These methods make use of soil index properties such as liquid limit, shrinkage limit, percent clay size composition of soils, and also some of the indices such as plasticity index, shrinkage index, and the like to estimate the swell potential of soils.

b. Direct Method

The methods coming under this category measure the swell potential of soil directly by:

1. **Oedometer Tests**

According to Winterkorn and Fang (1986), the most useful and reliable assessment of the swell potential of soil could be obtained from the conventional Oedometer swell tests (Jones and Geological, 2012).

2. **Free swell tests:**

Free swell value (FSV) test: This test was first proposed by Holtz and Gibbs (1956). This test consists of pouring slowly 10 cm^3 of oven-dried soil passing 425 μm sieve into a 100 cm^3 measuring jar filled with distilled water and noting the equilibrium volume of the sediment formed. The free swell value is then calculated as the increase in the volume of the soil expressed as a percentage of the initial volume.

2.4 Soil Stabilization

Soil stabilization is a method of improving soil properties by blending and mixing other materials. Soil stabilization is the process of improving the shear strength parameters of soil and thus increasing the bearing capacity of the soil. It is required when the soil available for construction is not suitable to carry a structural load. Soil stabilization is used to reduce the permeability and compressibility of the soil mass in earth structures and to increase its shear strength. Thus, reducing the settlement of structures. Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve their geotechnical properties such as compressibility, strength, permeability, and durability.

The researchers explore the Advantages of soil stabilization as follow: stabilized soil functions as a working platform for the project, stabilization waterproofs the soil, stabilization improves soil strength, stabilization helps reduce soil volume change due to temperature or moisture, stabilization improves soil workability, stabilization reduces dust in a work environment, stabilization upgrades marginal materials, stabilization improves durability, stabilization dries wet soils, stabilization conserves aggregate materials and stabilization reduces cost (Navy and Force, 1994).

2.4.1 Soil stabilization methods

In road construction projects, soil or gravelly material is used as the road main body in pavement layers. To have the required strength against tensile stresses and strains spectrum, the soil used for constructing pavement should have a special specification. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen, or a combination of these). The stabilized soil materials have a higher strength, lower permeability, and lower compressibility than the native soil.

Stabilization can be derived from thermal, electrical, mechanical, or chemical means. The first two options are rarely used (Chindris, *et al.*, 2017).

Mechanical stabilization, or compaction, is the densification of soil by the application of mechanical energy (Little, 2009). The method can be achieved in two ways, namely;

1. In situ stabilization and
2. Ex - situ stabilization.

Note that, stabilization is not necessary as a magic wand by which every soil property can be improved for the better. The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability, and durability.

Some stabilization techniques are listed below: - mechanical stabilization, stabilization by using different types of admixes, stabilization by using different types of admixes are: -

lime stabilization, cement stabilization, chemical stabilization, fly ash stabilization, rice husk ash stabilization, bituminous stabilization, thermal stabilization, electrical stabilization, stabilization by geotextile and fabrics and recycled and waste products, etc.

2.4.2 Mechanical stabilization

Mechanical stabilization can be defined as a process of improving the stability and shear strength characteristics of the soil without altering the chemical properties of the soil. The main method of mechanical stabilization can be categorized into compaction, mixing, or blending of two or more gradations, applying geo-reinforcement, and mechanical (Afrin, 2017).

Soil stabilization can be achieved through the physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing (Makusa, 2017).

2.4.3 Chemical stabilization

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium, or sodium chloride or with viscoelastic materials such as bitumen. Chemical stabilizers can be broadly divided into three groups, traditional stabilizers such as hydrated lime, Portland cement, and fly ash; non-traditional stabilizers comprised of sulfated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and by-product stabilizers which include cement kiln dust, lime kiln dust, etc. Among these, the most widely used chemical additives are lime, Portland cement, and fly ash. Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable (Hanuma *et al.*, 2017).

2.5 Components of stabilization

Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve their geotechnical properties such as compressibility, strength, permeability, and durability. The components of stabilization technology include soils and or soil minerals and stabilizing agents or binders (cementitious materials) (Afrin, 2017).

2.5.1 Soil

Most of the stabilization has to be undertaken in soft soils (silty, clayey peat, or organic soils) to achieve desirable engineering properties. Fine-grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter.

A clay soil compared to others has a large surface area due to flat and elongated particle shapes. On the other hand, silty materials can be sensitive to a small change in moisture and, therefore, may prove difficult during stabilization. Peat soils and organic soils are rich in water content of up to about 2000%, high porosity, and high organic content (Makusa, 2017).

2.5.2 Stabilizing Agents

These are hydraulic (primary binders) or non-hydraulic (secondary binders) materials that when in contact with water or in the presence of pozzolanic minerals react with water to form cementitious composite materials (Makusa, 2017). The commonly used binders are Cement, lime, fly ash, blast furnace slag, and pozzolanas.

2.6 Factors Affecting the Strength of Stabilized Soil

The presence of organic matters, sulfates, sulfides, and carbon dioxide in the stabilized soils may contribute to the undesirable strength of stabilized materials (Afrin, 2017).

2.6.1 Organic Matter

In many cases, the top layers of most soil constitute a large amount of organic matter. However, in well-drained soils, organic matter may extend to a depth of 1.5 m (Makusa, 2017). Soil organic matters react with hydration products e.g., calcium hydroxide (Ca (OH)2) resulting in a low pH value. The resulting low pH value may retard the hydration process and affect the hardening of stabilized soils making them difficult or impossible to compact.

2.6.2 Sulfates

The use of a calcium-based stabilizer in sulfat rich soils causes the stabilized sulfate rich soil in the presence of excess moisture to react and form calcium sulphoaluminate (ettringite) and or thamausite, the product which occupies a greater volume than the combined volume of reactants. However, excess water to one initially present during the time of mixing may be required to dissolve sulfate in order to allow the reaction to proceed (Bandara, 2016).

2.6.3 Sulfides

In many waste materials and industrial by-products, sulfides in form of iron pyrites (FeS_2) may be present. Oxidation of FeS_2 will produce sulphuric acid, which in the presence of calcium carbonate, may react to form gypsum (hydrated calcium sulfate) according to the reactions (i) and (ii) below

I. 2FeS2 + 2H2O +7O2= 2FeSO4 + 2H2SO4

II. CaCO3 + H2SO4 + H2O = CaSO4.2 H2O + CO2

The hydrated sulfate so formed, and in the presence of excess water may attack the stabilized material in a similar way as sulfate (Makusa, 2017).

2.6.4 Compaction

This laboratory test is performed to determine the relationship between the moisture content and the dry density of soil for a specified compaction effort. The compactive effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact the soil in the field, and some examples include tamping, kneading vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, therefore, the test is also known as the Proctor test.

Two types of compaction tests are routinely performed: (1) The standard proctor test, and (2) the modified proctor test. Each of these tests can be performed in three different methods as outlined in the attached Table 2.3. In the standard proctor test, the soil is compacted by a 5.5 lb. hammer falling a distance of one foot into a soil-filled mold. The mold is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The modified proctor test is identical to the standard proctor test except it employs, a 10 lb. hammer falling a distance of 18 inches, and uses five equal layers of soil instead of three. There are two types of compaction molds used for testing. The smaller type is 4 inches in diameter and has a volume of about 1/30 ft), and the larger type is 6 inches in diameter and has a volume of about 1/13.333 ft. (2123 cm^3). If the larger mold is used each soil layer must receive 56 blows instead of 25 (See Table 2.3).

In practice, the effect of the addition of a binder to the density of soil is of significant importance. The stabilized mixture has a higher maximum dry density than that of untreated soil for a given degree of compaction. The optimum moisture content increases with increasing binders (Makusa, 2017). In cement stabilized soils, the hydration process takes place immediately after cement comes into contact with water. This process involves the hardening of soil mix which means that it is necessary to compact the soil mix as soon as possible. Any delay in compaction may result in hardening of stabilized soil mass and therefore, extra compaction effort may be required to bring the same effect. That may lead to serious bond breakage and hence loss of strength. Stabilized clay soils are more likely to be affected than other soils due to the alteration of plastic properties of clays (Makusa, 2017). In contrary to cement, delay in compaction for lime-stabilized soils may have some advantages. Lime stabilized soil requires a mellowing period to allow the lime to diffuse through the soil thus producing maximum effects on plasticity. After this period, lime stabilized soil may be remixed and given its final compaction resulting in remarkable strength than otherwise (AASHTO, 2018).

Table 2.2 alternative proctors test methods

	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
For test sample, use 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/layer	25	25	56	25	25	56

2.6.5 Moisture Content

In stabilized soils, enough moisture content is essential not only for the hydration process to proceed but also for efficient compaction. Fully hydrated cement takes up about 20% of its weight of water from the surrounding (Afrin, 2017); on other hand, quicklime (CaO) takes up about 32% of its weight of water from the surrounding (Makusa, 2017). Insufficient moisture content will cause binders to compete with soils to gain these amounts of moisture. For soils with great soil water affinity (such as clay, peat, and organic soils), the hydration process may be retarded due to insufficient moisture content, which will ultimately affect the final strength. Standard reference: ASTM D 2216 standard test method for laboratory determination of water (moisture) content of soil, rock, and soil-aggregate mixtures.

2.6.6 Temperature

The pozzolanic reaction is sensitive to changes in temperature. In the field, temperature varies continuously throughout the day. Pozzolanic reactions between binders and soil particles will slow down at low temperatures and result in lower strength of the stabilized mass. In cold regions, it may be advisable to stabilize the soil during the warm season (Makusa, 2017).

2.7 Pumice

Pumice is a common rock of volcanic origin used as lightweight aggregate, which occurs in many parts of the world, and returns its useful properties only when it is young and unaltered. The low density is due to their cells with cavities being formed by gases expanding with the release of pressure. It was first introduced by the Romans in the second century where 'The Pantheon' has been constructed using pumice. Pumice stone is a lightweight aggregate of low specific gravity. Its water absorption is as high as 55% since it is a highly porous material. The density of pumice is 0.25 g/cm^3 and it is a natural raw material (Ramasamy, 2015).

Pumice is the only rock that floats on water, although it will eventually become waterlogged and sink. It is usually light colored, indicating that it is a volcanic rock high in silica content and low in iron and magnesium.

Moreover, Pumice consists of fine grain or ash. Worldwide more than 50 countries produce pumice and related products of Volcanic ash in a large amount. Italy is the largest producer, and other major countries are Chile, Canada, Spain, Turkey, and the United States. In Africa, natural pozzolanas are present in 6 countries. These are Burundi, Cape Verde, Ethiopia, Rwanda, and Tanzania (Urio and Mwemezi, 2018).

2.7.1 Effects of Crushed Pumice on Expansive Subgrade Soil

Stabilization of high-plasticity clayey subgrade road pavements using the Isparta-Karakaya pumice waste was investigated. Once the index properties were determined, standard compaction, constant volume swelling, unconfined compression, shrinkage, and atterberg limit tests were conducted to examine the effects of pumice on the compaction properties and the unconfined compressive strength of compacted clay. The Isparta-Karakaya pumice is composed of silica and aluminum. The natural unit weight of the pumice is extremely low. However, its permeability coefficient is high. The durability value against freezing of the pumice is 26. The Los Angeles abrasion and CBR values prove the limit values of Turkish Standards (Saltan and Keskin, 2015).

The shrinkage, Atterberg's limit values decreased with increasing pumice ratio because pumice is a non-plastic material. Standard compaction tests were performed by adding varying amounts of pumice to clay. With increasing pumice ratio in the mixture, it was observed that the maximum unit weight increased, whereas the optimum water content decreased. Moreover, the swelling pressure decreased with increasing amounts of pumice in the mixture. The unconfined compressive strength increased with pumice additive ratios of up to 30%. In contrast, the unconfined compressive strength decreased with pumice additive ratios of 40% – 50%. The CBR degree increased as the pumice ratio was increased up to 40%, but it decreased as the pumice ratio reached 50%. Therefore, the ideal amount of the Isparta-Karakaya pumice additive to use for clayey subgrades is 30% by weight. The use of pumice for the stabilization of clayey subgrades has economic, environmental, and engineering benefits. Locally available pumice has little commercial value. Because it is readily available, the transportation costs involved in using this resource are minimal (Saltan and Keskin, 2015).

2.8 The crushed stone dust

Due to the increase in construction activities the demand for crushed stone for buildings, roads, railway ballast and concrete work are increased. For the crushing of stones, a large number of crusher units are installed. The aggregates are produced by blasting and crushing stones in mechanical crushers, the stone dust is formed in primary and secondary crushing, screening, and stock piling. Stone dust is mainly produced during crushing operations. This is a waste product and leads to pollution as well as problem for stock piling it on crusher site to reduce the pollution as well as a disposal problem. The present paper deals with the study to be carried out to assess the utilization of stone dust in road, highway construction the experimental program consisting of evaluating firstly the properties of original expansive soils collected from a different location and the stone dust collected from the same locations for modifications of soils. The modification of soil is carried out by the addition of stone dust to original expansive soils in the range of 0 to 60% by the percentage increase of 10, 20, 30, 40, 50, and 60% the effect of stone dust on the liquid limit, plastic limit, plasticity index, dry density, optimum moisture content, and CBR values is considered (Dixit and Patil, 2017).

2.8.1 The effects of crushed stone dust on expansive soil

2.8.1.1 Effect of Crushed stone dust on Atterberg's limit

According to the researcher (Jemal *et al.*, 2019) justified the results of expansive clay with crusher dust mixes are with varying percentages on addition. It shows the consistency limit such as liquid limit, plastic limit, and plasticity index. It was found that as the percentage of crusher dust increases the liquid limit and plastic limit decreases. Consequently, the plasticity index also decreased followed by an increase in crusher dust content. After modification PI reduced from a value of 44.81% to a value of 14.43% after an improvement with 50% crusher dust. Hence crusher dust has a great impact on the reduction of PI. The probable reason for the reduction of the liquid limit of modified soil may be due to mechanical stabilization and the addition of non-plastic material. Blending expansive soil with crushed stone dust was satisfying ERA standard specification for Subgrade construction. Blending expansive soil with 30% crushed stone dust and above was satisfying ERA standard specification for subgrade construction (Jemal *et a*l., 2019).

2.8.1.2 Effect of Crushed stone dust on free swell index

The free swell index of expansive soil decreases when the ratio of crusher dust increases. The free swell of the samples has decreased with an increase in crusher dust ratio. But slight reduction is observed with a higher ratio of crusher dust added. Except for 5, 10, 15, and 20%

of crusher dust soil mix all ratios were under the specification. As more percent of Crusher dust is added to the soil the swell and shrink properties of the affected soil lower. Besides, more crusher dust content slightly reduces the expansiveness of the soil. As a whole, the quantum of replacement of quarry dust is found to be in the range of 40% to 50% in laying road pavements for the in-situ expansive clay soil which is marginally higher. For economic considerations and for laying local pavements inside streets and villages 30% replacement of clayey soil can be sorted (Jemal *et al.* , 2019).

2.8.1.3 Effect of crushed stone dust on compaction

The values for the maximum dry densities were noted to significantly increase with the addition of crusher dust from a value of 1.323 g/cm^3 to a maximum value of 1.735 g/cm^3 attained in the blend 50% crusher dust. Whereas, the optimum moisture content values are continuously decreasing. The optimum moisture content (OMC) decreases from 30.91% to 18.16% when crusher dust is increased from 0 to 50% (Jemal *et al*., 2019).

2.8.1.4 Effect of crushed stone dust on CBR

The expansive soil was modified by the addition of crusher dust in the proportion stated in the methodology. There is an increase in the CBR value as the dust percentage increases up to 35% and slightly reduces when further increases the dust content. The probable reason for the increase in CBR value of soil is by the addition of stone dust in comparison with original soil may be due to an increase in density of modified soil mass having more strength(Jemal, *et al.,* 2019).

2.8.1.5 Effect of crushed stone dust on CBR Swell

The crusher dust and soil mixtures compacted in CBR molds at optimum moisture content and maximum dry density gauged for swelling properties before and after soaking for four days to evaluate the percent swell. The soil sample had a 3.181% value of CBR swell but when 30% crusher dust was added it reduces to 1.478%. These indicate highly reduction in CBR swell. When it mixes with crushed stone dust beyond 30% it improves the expansive soil strongly but there is slight reduction was observed. Therefore, using crusher dust stabilizers improve the stability and strength of the subgrade soils.

The strength of the subgrade is the principal factor in determining the thickness of the pavement, but deterioration due to frost action must also be taken into account. The strength of the subgrade is associated with the CBR scale *(Jemal, et al., 2019).*

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Study Area

The Jimma to Chida road is located in the Seka Chekorsa and Dedo district of the Jimma Zone of the Oromia national regional state, and the Konta Special district of the SNNPRS in the southwestern part of Ethiopia. It starts at the junction connecting the Jimma to Chida and the Jimma to Mizan Roads at the outskirt of Jimma city which is about 346 km southwest of Addis Ababa, and it terminates in Chida town. The total length of the project road is 79.4 km. The terrain traversed by the project road is dominantly rolling and mountainous comprising about 97.3% and the remaining portion is flat (2.7%) (Environmental and Assessment, 2016).

The road has varying subgrade soils as one travels in the corridor. It was observed that flat sections form generally black, swampy clays while rolling and mountainous terrain form well-drained red, silty clays. As can be expected, the black swampy clays make weak subgrade while the red clays and the weathered rock make strong subgrade. The gray clays make intermediate strength subgrade. Area of swamps needs special treatment (Environmental and Assessment, 2016).

In general, soils along the flat sections are generally black, swampy clays while the soils along the rolling and mountainous sections are well-drained red, silty clays (Environmental and Assessment, 2016)

The samples for laboratory investigation were collected from two different pits which are located between along Jimma and Chida road segment. The specific locations of the study pits are; the first sample was collected from Chida town near to crusher plant area, and the second was collected from Jimma city, Jimma to Mizan roads at the outskirt of Jimma city.

Figure 3.1 Study area (source: www.google) May 20, 2021

3.2 The Study Design and Period

The study was followed by the experimental tests, the discussion on the results obtained and the purposive sampling method was applied to collect soil samples. To have satisfactory data, the stabilization of expansive subgrade soil using pumice and crushed stone dust, field observation, sample collection, and some literature from the study area were used.

The samples were prepared for each laboratory tests. Two representative soil samples were used in this study that would represent the whole study area. Laboratory experimental tests were conducted by the researcher, analysis of results that can be obtained from a laboratory experiment. The tests conducted for both natural expansive soil and stabilized soil samples were particle size analysis (wet sieve analysis and hydrometer), Atterberg's limits (LL, PL, and PI), modified proctor compaction test, (CBR), shrinkage limit, free swell, and unconfined compressive strength (UCS) test with an increment percentage mixture of 10, 20, 30 and 40%, each equal amount of pumice and crushed stone dust content by dry weight of soil.

Then, a discussion was conducted on laboratory results. Under the discussion of the obtained results, the effects of blended pumice and crushed stone dust on expansive soil samples were examined and the result was compared with AASHTO and ERA standards.

The procedure included in this study is sampling, air drying, Preparation of sample for each laboratory test, conducting laboratory tests of natural soil, blending the additive with natural expansive soil, conducting laboratory tests for blended expansive soil, and cross-checked whether satisfies the requirement of the standard specification.

The study was conducted from, April 2021 up to July 2021.

Finally, the cost of stabilization and respect to cart away was analyzed then a cost comparison was made. A general conclusion and recommendation were carried out. The overall research design has shown in Figure 3.2.

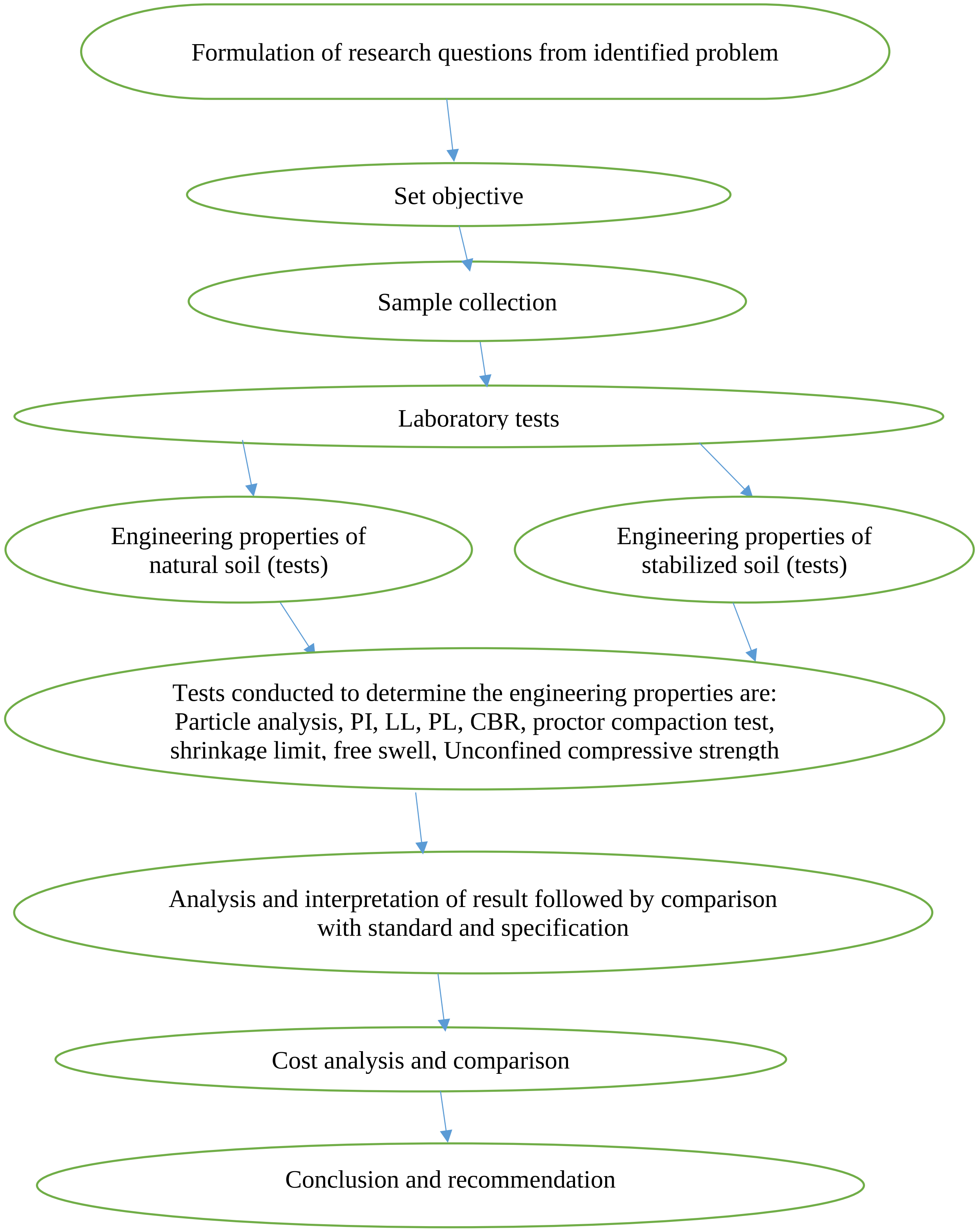

Figure 3.2 Flow chart of study design

3.3 Study variables

3.3.1 Dependent Variable

The dependent variable in this study was the performance improvement of expansive subgrade soil by stabilization using pumice and crushed stone dust.

3.3.2 Independent Variables

The independent variables in this research were engineering properties of expansive subgrade soil, Atterberg's limits, CBR, Compaction, Unconfined compressive strength.

3.4 Source of data

3.4.1 Primary Data

The primary data sources researcher has used for the study are field observation and laboratory test results.

3.4.2 Secondary Data

The secondary data sources that the researcher has used for this study are reference books, encyclopedias, journals, literature reviews, standards (ERA and AASHTO), and specifications.

3.5 Population, Sampling Techniques, and Sample Size

The populations under study are expansive soil, pumice, and crushed stone dust.

The sampling technique used for this research was purposive sampling, which is a non-probability method. This sampling technique was based on the laboratory test on the expansive subgrade soil and investigated the effect of pumice and crushed stone dust on expansive subgrade soil. The researcher takes two sample test pits of disturbed samples from each 1.5 m depth below the original ground. For each test quartering and weighting were used for the sampling technique.

3.6 Methods of Sample Collection

In order to achieve the purpose of this study work ethical considerations were considered and preparation of necessary documentation formats, manuals, prepared all materials for recording and observations, an official letter was taken from JIT to collect existing data from Ethiopia road authority Jimma district to collect the actual data about the study area. Continuous reviewing of related literature on relevant reference books, research papers, standards specifications like ERA, and ASHTO. Necessary data collection, organization, comparison, and analysis were obtained, and then subsequently compared the results with standard specifications.

Conclusion and recommendations were forwarded based on the results.

3.6.1 Materials

3.6.1.1 Expansive subgrade soil

Based on observation, expansive soil samples were sampled from the Chida town, and Jimma city, Jimma to Chida road segment. Two boreholes were excavated by hand. According to

AASHTO and ERA, 200 kilograms disturbed sample was collected at the depth of 1.5 m to avoid the inclusion of organic matter.

3.6.1.2 Crushed stone dust

85 KG crushed stone dust was obtained from Chida town China Zhongmei plc crusher plant.

Figure 3.3 crushed stone sample (Haile Zenebe June 05/2021)

3.6.1.3 Pumice

Three bags of pumice were obtained from Konta Special district each 50KG from three pumice sites, Genji, Yora, and Cheta Kechikecha

Figure 3.4 pumice sample from the quarry and in a laboratory (Abebe Geleso, May 11/2021)

3.6.2 Methods for Preparing and Testing Specimens

The standardized test procedure synthesizes ASTM procedures for mixing specimens with stabilizers. The large bulk samples to create one batch of specimens any boulder in sizes were removed. The soil sample pass sieve 19.5 mm was then mixed by hand until becoming uniform.

3.6.3 Mixing of Soil and Stabilizer

3.6.3.1 Mixing Procedure

After the required amount of stabilizer was mixed with soil, the mixing should be thoroughly mixed until the color of the mixture is uniform. If necessary, to achieve the desired moisture content for the batch, additional water was added into the blended soil sample. before water addition, the appropriate amounts of stabilizer were then added to the mixture and mixed thoroughly for three to five minutes.

The mixing of soil and stabilizer in this research was carried out by hand mixing, and the stabilizer and water were each added slowly to promote uniform mixing and to prevent clumping of the soil and/or stabilizer.

Figure 3.5 material mixing and its procedure (Giza Teshale, June 7/2021)

3.6.3.2 Dosage Rates

The dosage rate is based on the dry weight of natural expansive soil. The pumice–crushed stone dust amount used in this research was given as a percentage of the dry weight of the expansive soil. Accordingly, the amount of pumice–crushed stone dust to be used was found as; 10, 20, 30, and 40% by dry weight of sample soil.

3.7 Laboratory tests

Characteristics of the soils before and after being stabilized with pumice and crushed stone dust in the laboratory were investigated.

General standards the researcher used in conducting laboratory experiments.

Table 3.1 Laboratory test as per standard

Laboratory test	Standard
Free Swell Index test	IS 2720-part 40
Shrinkage limit	Das, 2002
Moisture Content	AASHTO T-256
Grain Size Analysis	AASHTO T-88
Specific Gravity	ASTM D 854-00
Atterberg Limits	AASHTO T90
Soil Classification	AASHTO M-145
Proctor compaction test	AASHTO T-180
California Bearing Ratio and CBR Swell	AASHTO T-193 and AASHTO T-180

3.7.1 Moisture content (AASHTO T-256)

The oven-drying method was used to determine the moisture contents of the samples. For the oven-drying method, small, representative specimens obtained from large bulk samples were weighed as received, then oven dried at 105 °C for 24 hours. The sample was then reweighed, and the difference in weight was assumed to be the weight of the water driven off during drying. The difference in weight was divided by the weight of the dry soil, giving the water content on a dry weight basis (Das, 2002).

$MC = \frac{W-Wd}{Wd}$ …Equation 3.1

Where MC= moisture content

W= weight of wet soil specimen

Wd= weight of oven dried soil

Typical values of water content for various types of natural soils in a saturated state are shown in Table 3.2.

Table 3.2 Typical values of water content in a saturated state (Das, 2002)

Soil	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	30-50
Soft organic clay	80-130
Glacial till	10

3.7.2 Particle Size Distribution (AASHTO T-88)

3.7.2.1 Sieve Analysis

Sieve size analysis and Hydrometer analysis were the two most common tests used for coarse-grain and fine-grain soils respectively. For coarse-grained materials, the grain size distribution is determined by passing soil sample either by wet or dry shaken through a series of sieves placed in the order of decreasing standard opening sizes and a pan at the bottom of the stock. Then the percent passing on each sieve is used for further identifying the distribution and gradation of different grain sizes. Since the samples were containing a high number of fine-grained soils; the wet sieve method was used in sieve analysis to remove the silt and clay content in the sample by washing the dry sample which was soaked for 24r and retain on 0.075 mm sieve size. During the Hydrometer test, a Sodium hex metaphosphate dispersing agent was used to disperse the sticky particles in fine cohesive soils. Tests were performed on disturbed soil samples for both analyses following the ASTM standard (ASTMD422-63, 2000) designations. Finally, the combination of wet sieve analysis and hydrometer test results was used in a plotted graph as a figure.

3.7.2.2 Atterberg Limits (AASHTO T90)

Representative samples of each soil are subject to Atterberg limits testing to determine the plasticity of the soils. An Atterberg limits device is used to determine the liquid limit of each soil using the material passing through a 475 µm (No. 40) sieve. The plastic limit of each soil will be determined by using soil passing through a 475 µm sieve and rolling 3 mm diameter threads of soil until they begin to crack. The plasticity index will then compute for each soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index are then used to classify each soil (ASTMD318).

Table 3.3 Soil expansivity predictions by liquid limit

Degree of expansion	LL (%)	
	Chen	IS 1498
Low	<30	20-35
Medium	30-40	35-50
High	40-60	50-70
Very High	>60	70-90

Table 3.4 Relationship between Atterberg's limit and swelling potential (Pitts 1984, Kalantari 1991).

LL (%)	PL (%)	Swelling potential
<50	<25	Low
50 – 60	25–35	Marginal
>60	>35	High

3.7.2.2.1 Determination of plastic index

The Plasticity index was calculated from plastic limit and liquid limit as follows:

$PI = LL - PL \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \, equation$ 3 .2.

Where PI = plastic index

PL= plastic limit

LL= liquid limit

3.7.2.3 Soil Classification (AASHTO M-145)

Both sample soils were classified according to the Unified Soil Classification System (USCS) and AASHTO. Using the particle size distribution and the Atterberg limits, the USCS designates a two-letter symbol and a group name for each soil. A visual-manual procedure can also be used to identify soils easily in the field.

All classifications provided in this research were based on the laboratory testing-based procedure.

3.7.2.4 Specific Gravity (ASTM D 854-00)

Values for specific gravity of the soil solids were determined by placing a known weight of oven-dried soil in a flask, then filling the flask with water. The weight of displaced water was then calculated by comparing the weight of the soil and water in the flask with the weight of the flask containing only water. The specific gravity was then calculated by dividing the weight of the dry soil by the weight of the displaced water.

$$Gs = \frac{Ws}{Vs * \rho w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{equation } 3.3$$

Where Gs=specific gravity

Ws= weight of solid soils

Vs = volume of solid soil

ρw = Density of water at 4℃

Table 3.5 Specific gravity value for different types of soil

Soil type	Gs
Sand	2.63 –2.67
Silts	2.65–2.7
Clay and silty clay	2.67–2.9
Organic soil	Less than 2

3.7.2.5 Proctor Compaction Test (AASHTO T-180)

The modified Proctor method was used to determine optimum water content at the maximum dry density of soil. This was done for natural soil samples and stabilized soil samples with various percentages of pumice and crushed stone dust which is to determine maximum dry density and optimum moisture contents of both natural and treated soil samples by adding various water contents.

The laboratory modified proctor tests are performed as per (AASHTO T 180 or ASTM D 1557). The tests are performed on disturbed samples of soil particles passing sieve sizes 4.75 mm or 19 mm mixed with water to form samples at various moisture contents ranging from the dry state to the wet state. These samples are compacted in five layers at 56 blows per layer per the specified nominal compaction energy of the modified proctor test.

Figure 3.6 Modified compaction and CBR compaction (Haile Zenebe July 1/2021)

3.7.2.6 California Bearing Ratio Test (AASHTO T-193 and AASHTO T-180)

California Bearing Ratio (CBR) tests were performed according to ASTM (Compactors, current edition approved Feb. 10, 1999. Published May 1999.) and AAASHTOT-180 on the mix ratios selected for stabilization samples. The method used for the preparation and compaction of soil specimens was ("ASTM-D-698 standard compaction," July 1988)ASTM D698. Fifty-six blows were applied to each of the five layers. The natural soil and treatments materials were mixed with water to achieve an OMC as determined by the modified proctor test. A 2-inch diameter penetration piston was used to penetrate the soil during the test. A load was applied on the penetration piston so that the rate of penetration was approximately 0.05 inch/min (1.27 mm/min). A 4.5 kg surcharge load was applied to the specimen to prevent the heaving of the soil. The same surcharge was used during 96 hours of specimen soaking in preparation for the soaked CBR test. At the end of the soaking period the penetration test is carried out at a rate of 1.27 mm/min and the force or load required to cause the penetration will be recorded with respect to the standard penetration depths at each 0.5 mm penetration, including the load value at 2.54 mm and 5.08 mm until the total penetration is 12.7 mm. The penetration resistance load is then plotted against the penetration depth and correction is made for the load-penetration curve. A typical load penetration curve correction graph is presented in the figure. Using the corrected value taken from the load-penetration curve for 2.54 mm and 5.08 mm penetration, the bearing ratio is calculated by dividing the corrected load by the corresponding standard load, multiplied by 100. Its value ranges from 0 (worst) to 100 (best). If the bearing ratio of 2.54 mm is greater than that of 5. 08 mm, the bearing ratio that should be reported for the soil is normally the one at 2.54 mm penetration. When the ratio at 5.08 mm penetration is greater, the test is entirely repeated on a fresh specimen. If the repeated result of

5.08 mm is again greater, the design bearing ratio will be that of 5.08 mm, or else, if the bearing ratio of 2.54 mm is greater the design bearing ratio will be that of 2.54 mm penetration. CBR = (Test load/Standard load) X100.

The following Table 3.6, gives the standard loads adopted for different penetrations for the standard material with a CBR value of 100%.

Table 3.6 Standard loads for different penetrations(“ASTM-D-1557 modified compaction,” July 1988)

Penetration of plunger (mm)	Standard load (kg)
2.5	1370
5.0	2055
7.5	2630
10.0	3180
12.5	3600

Table 3.7 ERA, CBR range for subgrade quality

Serial No.	CBR (%) Range	Subgrade Quality
1	0-3	Very poor subgrade
2	3-7	Poor to fair subgrade
3	7-20	Fair subgrade
4	20-50	Good subgrade
5	50+	Excellent subgrade

Figure 3.7 CBR reading (Giza Teshame July 5/2021)

3.7.2.7 Free Swell Tests (IS 2720 part 40)

The test was conducted on soil samples consisting of pouring slowly 10 cm^3 of oven-dried soil passing 425 µmm sieves into a 100 cm^3 measuring jar filled with distilled water and noting the equilibrium volume of the sediment formed. The free swell value is then calculated as the increase in the volume of the soil expressed as a percentage of the initial volume (ASTMD4546).

Finally, the data collected were organized and interpreted using excels according to the set objective. The result obtained from the laboratory were also analyzed both qualitatively and quantitatively. The free swell index is calculated using equation 3.2

Free swell index (%) = $\frac{Vd-Vk}{Vk} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots Equation$ 3 .4

Where, Vd = the volume of soil specimen read from the graduated cylinder containing distilled water; Vk = the volume of soil specimen read from the graduated cylinder containing kerosene. The soils having a high free swell index value may show considerable volume Changes as compared to the soils having lower free swell index values. Mohan and Goel (1959) gave the following classification of degree of expansion based on the Free swell index values as given

Table 3.8 Classification of degree of expansion based on the Free swell index

Free swell index	Degree of expansion	Degree of severity
> 200	Very high	Severe
100 – 200	High	Critical
50 –100	Medium	Marginal
< 50	Low	Non-critical

3.7.2.8 Shrinkage limit

Saturated clayey soil, when gradually dried, will lose moisture and subsequently, there will be a reduction in volume or linear length of the soil mass or linear length. During the drying process, a condition will be reached when any further drying will result in the reduction of moisture contents without any reduction in volume or linear length. The moisture content of the soil, in percent, which the decrease in soils volume or linear length ceases is defined as the shrinkage limit ((“Das,” 2002).

The linear shrinkage limit is calculated using equation 3.5

$LS = \left(\frac{Li-Lf}{Li}\right) * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$...Equation 3.5

Where, LS = the linear shrinkage, Li= initial length of soil, and Lf = final length of soil specimen after 24 hr. drying.

Table 3.9 Shrinkage limit, linear shrinkage, and degree of expansion relationship

Shrinkage limit	Linear shrinkage	Degree of expansion
<10	> 8	Critical
10 –12	5 – 8	marginal
>12	0 – 5	Non-critical


Figure 3.8 Linear shrinkage for natural soil (Eyuel Yosef July 12/2021)

3.7.2.9 Unconfined compressive strength test (AST D2166-00)

The unconfined compression test is a laboratory test used to derive the unconfirmed compressive strength (UCS) of a rock or soil specimen. Unconfirmed compressive strength (UCS) stands for the maximum axial compressive stress that a specimen can bear under zero confining stress. Because stress is applied along the longitudinal axis, the unconfined compression test is also known as uniaxial compression test. UCS is a parameter widely used in geotechnical design. It is not always possible to conduct the bearing capacity test in the field. Sometimes it is cheaper to take the undisturbed soil sample and test its strength in the laboratory.

Also, to choose the best material for the embankment, one has to conduct strength tests on the samples selected. Under these conditions, it is easy to perform the unconfined compression test on undisturbed and remoulded soil sample soil.

The shear strength of soil can be determined by the Mohr-coulomb failure criteria.

$S = c + \delta + tan\emptyset$ ….Equation 3.6

Where S= shear strength

C=cohesion

δ = normal stress

$\emptyset$ = angle of friction

For undrained tests of saturated clayey soil ($\emptyset$ =0),

$S = Cu$ ……………………………………………………………………………….Equation 3.7

Where Cu= undrained cohesion

For unconfined compressive strength, the test is a quick method of determining the value of Cu for clayey soil.

$qu = \frac{Cu}{2}$ ……………………………………………………………………………...Equation 3.8

For each set of readings, the vertical strain is calculated as follows

$\varepsilon = \frac{\Delta L}{L}$ ……………………………………………………………………………….Equation 3.9

Where ε = vertical strain

ΔL = total vertical deformation of the specimen

L = original length of the specimen

Table 3.10 unconfined compressive strength of different clay soil ((“Das” 2002)

Consistency	qu (Ib/ft^2)
Very soft	0 – 500
soft	500 – 1,000
medium	1,000 – 2,000
stiff	2,000 – 4,000
Very stiff	4,000 – 8,000

Figure 3.9 unconfined compressive test for remoulded soil sample (Misgana. T July 16/2021)

3.8 Data quality assurance

During a field study verification was checked, also the researcher was carefully given attention during data collection, testing, and recording.

A Laboratory test and fieldwork manual were prepared to avoid the error of data. The reliability and accuracy of data were checked.

CHAPTER FOUR

RESULTS AND DISCUSSION

Introduction

This chapter presented that the result of laboratory tests of the natural expansive soil samples, as well as stabilized expansive soil samples, and a discussion on the result of the laboratory tests. The most important parameters that were conducted to identify engineering properties of expansive subgrade soil are; natural moisture content, grain size distribution, free swell, specific gravity, Atterberg's limits, MDD, OMC, UCS, and CBR. Results were analyzed and interpreted while using the statistical description method. The laboratory tests were conducted on the highway and geotechnical laboratories of Jimma University Institutes of Technology.

Laboratory experimental test investigation was used to determine engineering properties of natural expansive soil and the effects of pumice – crushed stone dust as a stabilizing agent for expansive soil.

Finally, a cost-effectiveness analysis of using stabilization with respect to cart away and borrow fill material was done by using the current bid of ERA unit rate cost.

4.1 Engineering Properties of Expansive Subgrade Soil

The tests which were carried out on the samples of natural expansive soil were as follows: natural moisture content, grain size analysis; sieve analysis and hydrometer test, Atterberg's limits, modified proctor compaction test, California bearing ratio test, specific gravity, free swell, unconfined compressive strength, and shrinkage limit

4.1.1 Natural Moisture Content

The values of NMC were 43.63% and 38.47% for samples -1 and sample-2 respectively.

Table 4.1 Natural water contents of soil sample -1 and sample-2

Types of samples	The depth that sample taken	NMC (%)	Soil types
Sample -1	Below 1.5 m	43.63	Soft clay
Sample -2	Below 1.5 m	38.47	Soft clay

This is similar to the range of clay observations made by Das. (2002), since the natural water contents of soft clay soils vary from 30% to 50%.

The results presented in table 4.1 show that soil samples can hold the highest level of moisture content. These results show all soil samples are categorized as clay soils. It can be observed that the soil samples at pit two have more moisture and it is classified as most clay soil. The samples were taken during the dry season, the NMC may be greater than these values. The study area was affected by high rain during the winter season and natural water content is high during the rainy season. The results of this study agreed with the specified standard. Hence, these findings result suggested that the soil samples are clay soils, unsuitable for road construction as subgrade materials and it needs modification to serve as good quality materials.

4.1.2 Specific gravity

This test was performed for two sample soils, which are described shown in Table 4.2. The specific gravity determined for both soil sample-1 and sample-2 were 2.72, and 2.716, respectively.

Table 4.2 Specific gravity of natural soil sample -1 and sample-2

Types of soil sample	Depth (m)	(Gs)	Types of soil
Sample -1	Below 1.5	2.7162	Clay
Sample -2	Below 1.5	2.72	Clay

As the result of this test dedicate that soil samples of the area were clay and weak soil. This result agrees with the observations made by Das (2002), since the author state that specific gravity values ranging from 2.67 to 2.90 were assigned to clay and silty clay soils. The values from laboratory tests were found in this range. The soil has a specific gravity of less than 2.5 was inorganic according to Das (2002). From the result, the soil sample-1 (from Chida) and soil sample-2 (from Jimma), both are clay soil according to specific gravity values. The results of this study agreed with the specified standard. Therefore, these findings result suggested that the soil samples are clay soils, unsuitable for road construction as subgrade materials and it needs modification to serve as good quality materials.

4.1.3 The free swell of natural soil

The free swell index result value obtained for soil samples of the study area was 81.82% and 54.55% for samples -1 and -2 respectively.

According to Mohan and Goel, from free swell index results, the soils that have high swell index shown considerable volume changes as compared to the soil having lower free swell index values. Since the free swell index value is between 50% - 100% and it exceeds 50% for the study area shows that the area soils are expansive soil. The free swell index value is between 50% and 100%, according to ISO 2720 part 40, the degree of expansion is medium and the degree of severity is marginal. The natural soil sample-1 and sample-2 are expansive soil and show marginal volume change. The results of this study agreed with the specified standard.

Therefore, these findings result suggested that the soil samples are clay soils, unsuitable for road construction as subgrade materials and it needs modification to serve as good quality materials.

4.1.4 Grain size analysis

The gradation of the soil sample was conducted by both mechanical (wet sieve) and hydrometer tests. The grain size distribution curve is shown in Figure 4.1 for sample-1 and sample -2. The mechanical analysis is carried out to evaluate the distribution of the coarser particles and the hydrometer method is used to decide the distribution of the finer particles that passed on 0.075mm sieve size.

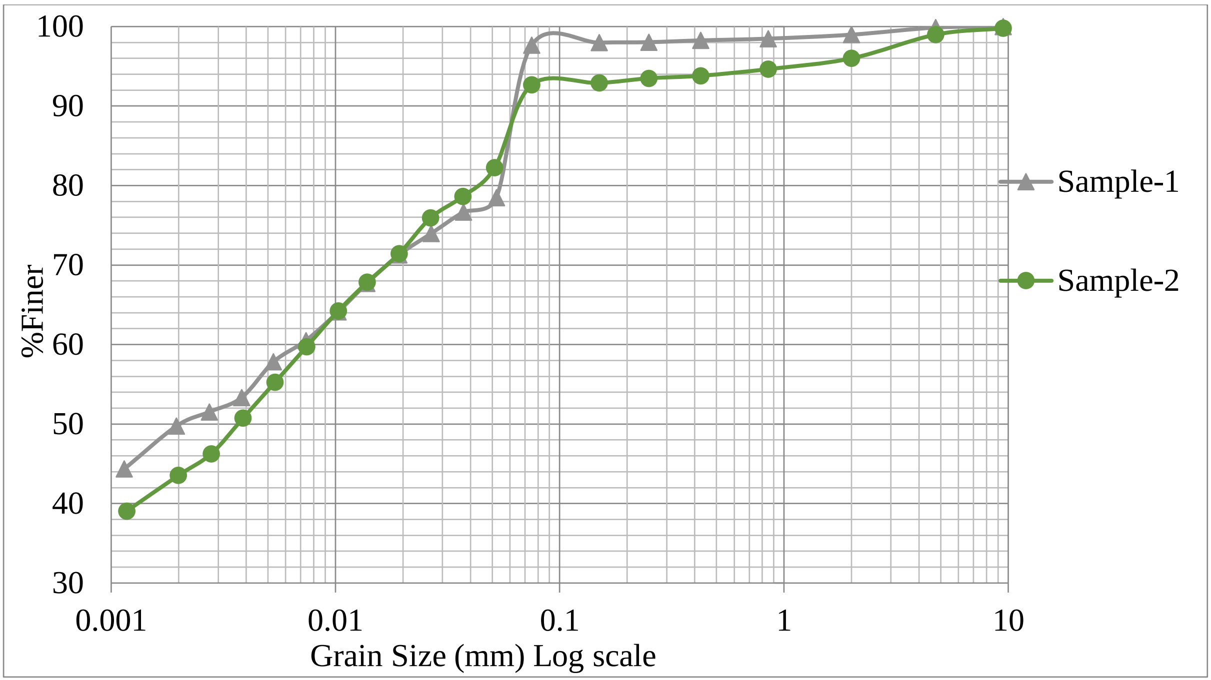

Figure 4.1 Grain size distribution curve of natural soil sample-1 and sample-2.

From the grain size analysis result the contents of coarse gravel contained, 0 % (retained above 9.5 mm), contents of coarser sand retained on 4.75 mm to 2 mm ranged from 0 % - 1.2%, medium sand retained on 1.18 mm to 0.4252 mm ranged from 1 % - 1.4 %, fine sand (retained on 0.3 mm to 0.075 mm) 1.51%, and 95.9% silt and clay (pass on 0.075 mm), for sample-1, and the contents of coarse gravel contained, 0 % (retained above 9.5 mm), contents of coarser sand retained on 4.75 mm to 2 mm ranged from 0% - 0.57%, medium sand retained on 1.18 mm to 0.4252 mm ranged from 0.83% - 2.03%, fine sand (retained on 0.3 mm to 0.075 mm) 2.6 to 6.24%, and 93.76% silt and clay (pass on 0.075 mm) for sample-2 by weight. It can be observed that the soil sample-1 and sample-2 are classified as clay soil since 95.9% and 93.76% passing on 0.075 mm sieve size more than 50%. From hydrometer analysis of 95.9%, 0.28%

(gravel), 3.83% (sand), 39.46% (silt) and 56.43% (clay) for sample-1 and, of 93.76% 0.01% (gravel), 6.23% (sand), 43.35% (silt) and 50.21% (clay) for sample-2.

Percent passing No.200 (75 μm) for soils, both Sample-1 and sample-2 are greater than 35%, which indicates that these soil samples are categorized as fine-grained soil (clay material) according to AASHTO M145. Fine grained soil type is less than 30% but greater than 12% of the total sample, the soil is described as "silty" or "clayey", depending on which particle size is dominating. The percent passing of each test is not only used to categorize soil as coarse grained and fine grained but it also helps to determine the soils class together with the Atterberg limits. Therefore, both soil samples were clay and have weak engineering properties to subgrade materials and required some level of improvement for them to be used as subgrade material based on ERA standard.

4.1.5 Atterberg Limits Test Results

This test was performed on two samples, as shown in Table 4.3. The results reveal that the LL, PL, and PI of the expansive clay soil were determined at 87%, 34%, 53%, and 62%, 21%, and 41% for sample-1 and sample-2 respectively.

Table 4. 3 Atterberg's limit test results for natural soil samples -1 and -2

Atterberg's limits	Sample- 1	Sample- 2
LL (%)	87	62
PL (%)	34	21
PI (%)	53	41

As a result of PI, both soil samples were poor subgrade material. A high numerical value of PI is an indication of the presence of a high percentage of clay in the soil sample. Based on the liquid limit, pit one and pit two soil sample is categorized to clay soil with high plasticity or high swelling potential.

Since the PI value of both sample-1 and sample-2 was greater than 30%, according to ERA specification the subgrade soil is poor. And soil whose PI value is greater than 40% shows that the soil is very plastic according to the (ERA site investigation manual, 2013). This soil dry strength is very high; can't be broken between a thumb and a hard surface. A decrease in particle size leads to an increase in total surface area and, as a result, an increase in the PI. A soil with a high LL and a high PI indicates that the soil is dominated by clay content, whereas a soil with a low LL and a low PI was seen for silty soils due to the impact of particle sizes. Also, the more plastic a soil, the more likely it is to be compressible. Since LL>60%, and PI

>35%, it has high swelling potential (according to Pitts,1984 and Kalantari,1991). It will have a greater potential to shrink and swell and it will be less permeable. Both sample-1 and sample-2 soils are expansive soil. The results of this study agreed with the specified standard. Hence, these findings result suggested that the soil samples are expansive soils, unsuitable for road construction as subgrade materials and it needs modification to serve as good quality materials.

4.1.6 Soil classification

Based on gradation and Atterberg's limit of the sample soils were classified as CH as per the USCS system as shown in Figure 4.2, and A-7-6 as per the AASHTO classification system as shown in Figure 4.3. This indicates that the subgrade soil was clay, highly expansive material, week subgrade and it required treatment to be used as a subgrade material.

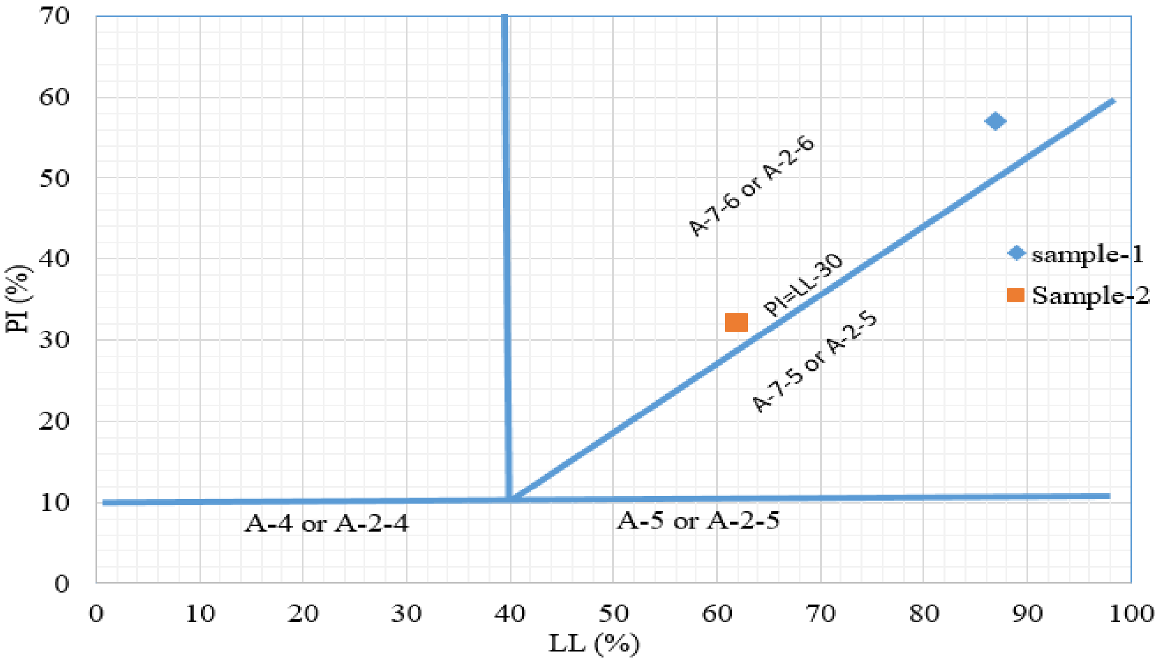

Figure 4.2 Liquid Limit and Plastic Index ranges for silty-clay minerals by AASHTO system

The basic limits needed for this research are the LL and the PL. The LL test is conducted as per AASHTO T89 whereas the PL test is conducted as per AASHTO T 90. The classification of soils was obtained from test sample-1 and sample-2, according to the AASHTO soil classification, both samples were A-7-6.

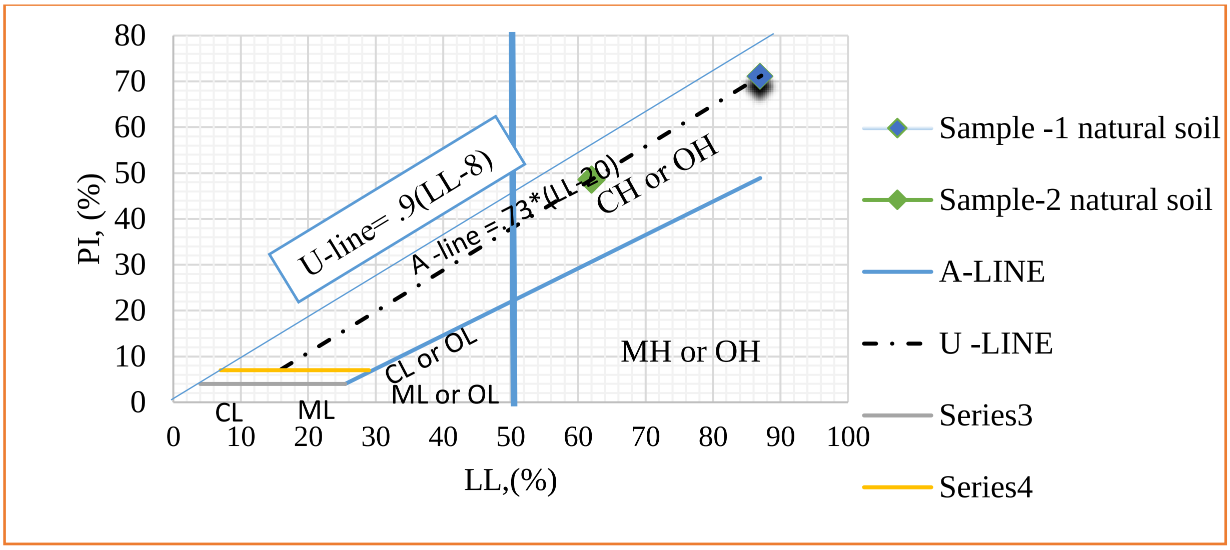

Figure 4.3 Plasticity chart of natural soil samples according to USCS

The classification of soils obtained from test sample-1 and sample-2, according to the USCS scheme, both samples were clay soils (CH).

The soil classification result performed explains that both A-7-6 soil sample-1 and A-7-6 soil sample-2 were fine grained clay soil. Such kind of classification helps to provide information on which group symbol the soil lies. Besides this, these group symbols inform the quality of the soil which and where to use as a highway material.

Soil classification according to the Unified classification technique is based on the plot of the liquid limit against a plastic limit to detect the category of the soils following Casagrande chart. Casagrande chart recommends that the soil with plastic index is greater than four (PI>4) and plots on A-line or above A-line it the soil is classified to clay soils. In this study soil sample one and two have PI>4 and the plot is above A-line with a liquid limit is greater than 50%, they are categorized as clay soil with high plasticity (CH)

As a result, the soils under study have a high probability of losing shear strength when in close contact with water, because the shear strength of fine-grained soils (silt and clay) is primarily determined by inter-particle forces, and this bonding force is easily broken down when exposed to high moisture content. Generally, according to (ERA, 2013) clay material having a LL (%) exceeding 60; and a PI (%) exceeding 30; of weak soils are not fair to use as the subgrade. If the PI is greater than 35%, the material must be treated to minimize the problem or it should be discarded.

4.1.7 Linear Shrinkage

This test was performed on two samples and is shown in Table 4.6. The linear shrinkage, of the natural soil samples, were 13.4 % and 12.1 % for sample-1 and sample-2 respectively.

Table 4.4 Linear Shrinkage test results of the study area.

Types of samples	LS (%)	Date
Sample -1	13.4	June 24/2021
Sample -2	12.1	June 24/2021

The result shows that LS values are greater than 8% that means it is critical (according to Altemeyer, 1955). From the result, the soil samples were easily expanded when it waters and shrink when it gets dry. The soil sample-1 and sample-2 show that it is expansive soil. The results of this study agreed with the specified standard. Hence, these findings result suggested that the soil samples are expansive soils, unsuitable for road construction as subgrade materials and it needs modification to serve as good quality materials.

4.1.8 Compaction characteristics of the natural soils

The relationship between Moisture and density was determined while using the standard Proctor compaction technique using the ASTMD698 procedure. When 20% or less by weight soil sample/material is retained on the 4.75mm sieve the standard compaction technique is applied to produce a well-defined maximum dry unit weight for non-free draining soils. Hence, the soil sample retained on 4.75mm of sieve size is equal to zero, standard compaction test is applied. The results for the relationship of maximum dry density and optimum moisture content are shown in Figure 4.4.

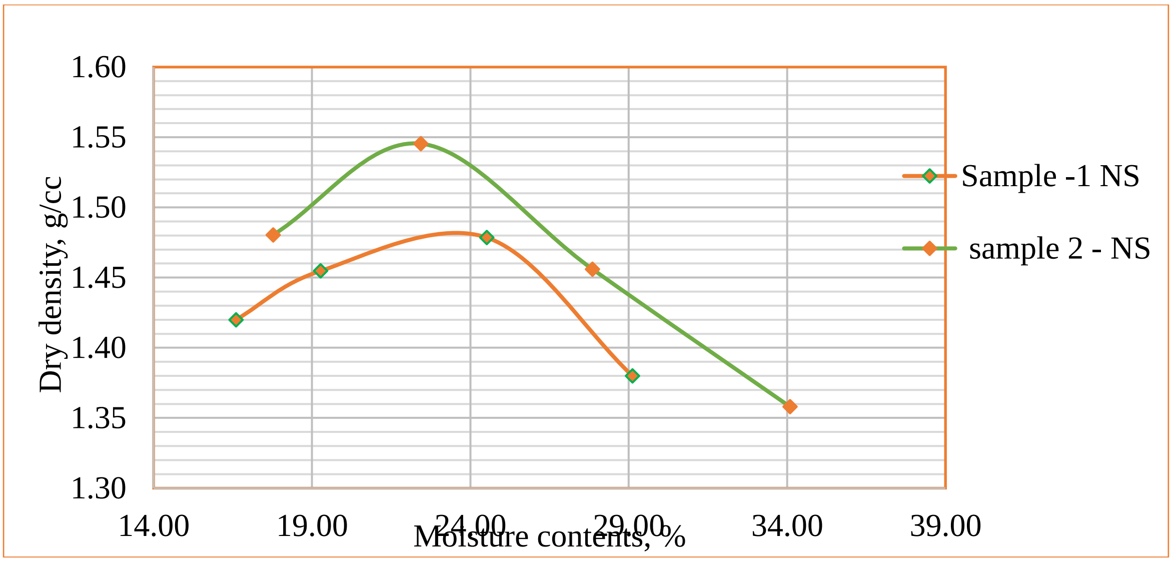

Figure 4.4 Maximum dry density versus optimum moisture content of natural soil sample- 1 and -2.

From the laboratory test results, it can be seen that the maximum dry density of this study site ranges from the weakest 1.48g/cm³ to 1.55 g/cm³, and the optimum moisture content ranged from 23.44 to 24.51%. These results indicated that all soil samples are clay soil. The maximum dry density at 95% is the most common method that can be useful to determine the void ratio and CBR value of subgrade soil to be used for roadway pavement design. From Figure 4.4, sample-1 and sample-2 soil samples have less maximum dry density with high moisture contents, which have less density or specific gravity. Compaction for clay soil MDD range is between 1.45- 1.6 g/ g/cm³, and optimum moisture content is between 22 - 30% (according to Harvard Miniature).

From this, the compaction result shows that both soil samples have relatively low maximum dry density at optimum moisture contents, which are not suitable as subgrade materials as roadbed of pavement design. Result obtained from laboratory test shows the soil samples were categorized as clay soil.

4.1.9 California Bearing Ratio (CBR) And (CBR) Swell of Natural Soil

This test was performed on two samples as shown in Figures 4.5 and 4.6. The CBR values of the natural soil samples are determined after 4 days soak; which indicates that a total loss of strength of the natural subgrade soil on soaked CBR value is one of the parameters used to designate the load bearing capacity of subgrade soil for roadway pavement design. The sample had CBR and CBR swells were 1.1% and 20.3%, 1.7% and 6.8% for samples -1 and sample -2 respectively.

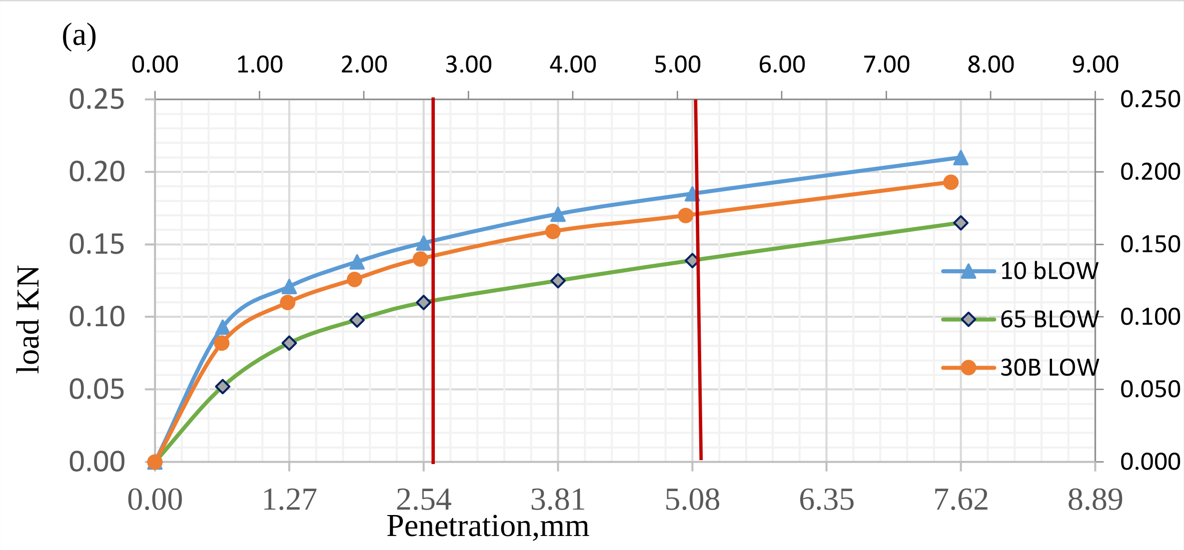

Figure 4.5 California Bearing Ratio graph of the natural soil sample-1

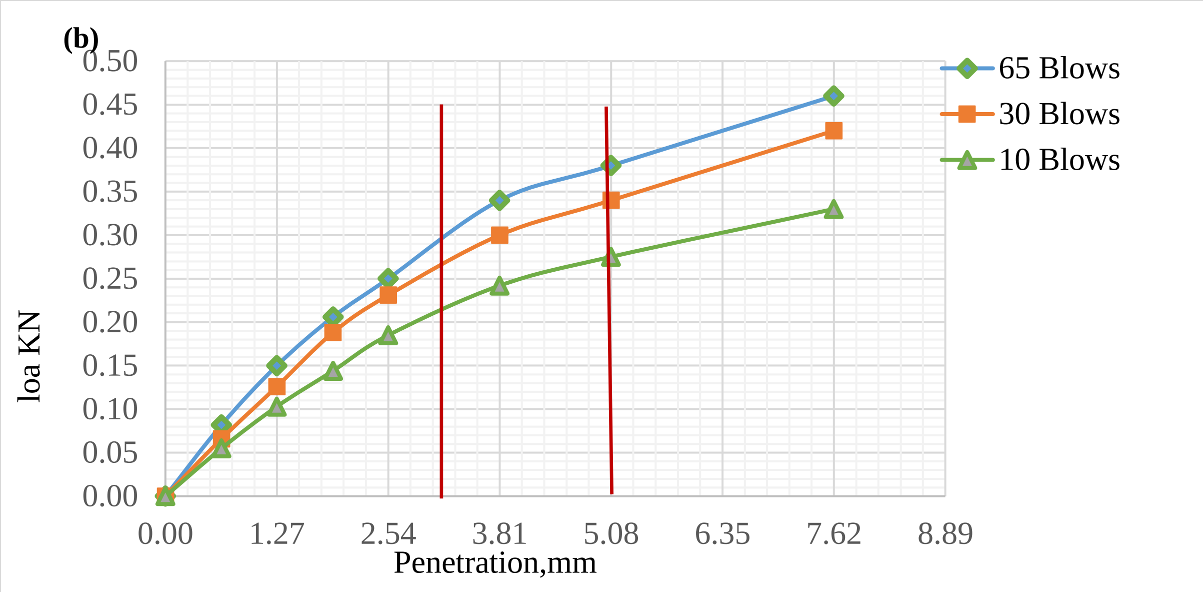

Figure 4.6 California Bearing Ratio graph of the natural soil sample-2

As a result, soil sample-1 and sample-2 exhibit the least CBR value, which is the most expansive soil. Hence subgrade soil samples exhibit low bearing capacity and are not suitable for road construction while the CBR value of subgrade soil samples are less than 3% and CBR swellings are greater than 2%, that the minimum requirements recommended by ERA, 2013 standard. Based on CBR value and CBR swelling value, the material was classified as very poor subgrade materials (ERA, 2013). The results show that the soil samples have a very low load bearing capacity, which required additives materials as the soil to be used as subgrade materials of a roadway. To achieve the objective of this study, both soil sample-1 and sample-2 should need special treatment. Therefore, from the results, the soils are expansive soil.

4.1.10 Unconfined compressive strength

4.1.11 Unconfined compressive strength of natural soil

The result of the unconfined compressive strength test for soil samples is shown in Table 4.5. The unconfined strength of the soil sample in this study area ranged weakest 33 kpa for sample-1 and 42.75 kpa for sample-2.

Table 4.5 Summary of unconfined compression strength test result of natural soil sample

Types of soil	UCS	Shear strength (Su)	Soil type according to Das, 2002
Sample-1	33.0 kpa	16.5 kpa	Soft clay
Sample-2	42.75 kpa	21.65 kpa	Soft clay

The result of the unconfined compressive strength test for soil samples is shown in Table 4.5 The result indicated that all extracted soil samples are categorized as soft soils since the unconfined compression strength test results of specimens failed at a pressure of less than 50 kpa. This failure indicates the deterioration of soil samples starts at the stage of the axial stress become decreases as the axial deformation increases. The unconfined strength of the soil sample in this study area ranged from weakest to relative strongest ranged from 33 to 42.75Kpa. The results showed that the UCS-values of the pit-two soil sample is 33kpa, which is the weakest soil and requires improvement to be used as subgrade materials. The clay soils with UCS less than 50kPa are categorized as clay soils (Alemineh Sorsa, 2019).

The result obtained from the laboratory test shows that both sample -1 and sample -2 were soft soil (weakest clay soil) in which the result was between 0-50 kpa according to (Das,2002). This failure indicates the deterioration of soil samples starts at the stage of the axial stress become decreases as the axial deformation increases.

The results showed that the UCS-values of the soil sample-1 and sample-2, which are the weakest soils and require improvement to be used as subgrade materials. The clay soils with UCS less than 50kPa are categorized as clay soils (Das,2002). Therefore, the results show that the soil samples meet the general principles of soft clay soil and it indicated that the soil samples have low strengths that are not suitable as subgrade materials for roadway pavement design.

Table 4.6 Summary of laboratory results for natural soil samples

Parameter	Laboratory result			
	Sample-1		Sample-2	
Percentage of Passing through No.200	95.89		93.73	
Liquid Limit (%)	86.7		62	
Plastic Limit (%)	34		21	
Plasticity Index (%)	52.7		41	
Linear shrinkage (%)	13.4		12.1	
Soil classification	USCS	CH	USCS	CH
	AASHTO	A-7-6	AASHTO	A-7-6
Specific Gravity of Soil	2.67		2.7	
Natural Moisture Content (%)	43.63		38.47	
Free swell	81.82		54.55	
Compaction test result	MDD	1.48	MDD	1.55
	OMC	24.51	OMC	22.44
California Bearing Ratio (CBR), % (95% MDD)	1.1		1.7	Poor subgrade according to ERA standard.
CBR Swell, % (95% MDD)	20.38		6.81	
UCS test result	33kpa		16.5kpa	Soft clay
Shear strength test result	42.75kpa		21.65kpa	Soft clay

Generally, from summary Table 4.6, the result shows that natural subgrade soils are expansive soil according to ERA and AASHTO standards. Natural soil has high degree of expansion and shrinkage rate, and clay soil. And also, the natural expansive soil shows that it has low bearing capacity.

4.2 Laboratory Tests and Results for Stabilized Soil

4.2.1 Effect of Pumice and Crushed stone dust on Expansive Subgrade Soil

Both sample soils were mixed with equal amounts of each; 10, 20, 30, and 40% of pumice and crushed stone dust by dry weight of soil. The tests mentioned in the above section were repeated on each representative soil - pumice and crushed stone dust stabilization.

4.2.1.1 Effects of Pumice - Crushed Stone Dust on Grain Size Analysis

For both soil samples, grain size analysis was performed for each 10, 20, 30, and 40% of pumice - crushed stone dust stabilizing with two samples of expansive soils by dry weight. Grain size analysis of stabilized soil with different proportions of pumice and crushed stone dust is summarized in Table 4.7

Table 4.7 gradation of stabilized soil samples

Type of sample		Sample -1			Sample-2			
% Of blending	5%P+5 %CSD	10%P+10 %CSD	15%P+15 %CSD	20%P+20 %CSD	5%P+5 %CSD	10%P+10 %CSD	15%P+15 %CSD	20%P+20 %CSD
%Course	7.73	14.96	17.27	24.08	5.32	14.33	19.97	23.57
%Fine	92.27	85.04	82.73	75.92	94.68	85.67	80.03	76.43
%Gravel	0.46	2.07	2.07	1.4	0.39	2.29	1.66	5.11
%Sand	7.27	12.89	15.20	22.68	4.93	12.04	18.31	18.46
%Silt	44.22	36.78	32.99	11.38	38.25	42.12	19.02	16.17
%Clay	48.05	48.26	49.74	64.54	56.43	43.55	61.01	60.26

From the summarized result, the grain size analysis for 10, 20, 30, and 40% of pumice and crushed stone dust by dry weight of soil samples, the fine grain size, which pass on 0.075mm sieve results obtained were 92.27, 85.04, 82.73, and 75.92%, for sample -1, for sample-2, 88.76, 85.67, 80.03, and 76.43%. The result shows that the fine grain size contents were decreased from 95.89% to 75.92% and 93.76% to 76.43% for sample-1 and sample-2 respectively.

The engineering characteristics of a soil mass depend on the proportion of the coarse- and fine-grained size distribution. The densification of the well graded soil cause due to the fine particles move to enter between coarse particles. This can reduce voids of compacted soil, increases the strength of the soil. When compaction is attempted inadequate distribution of particles sizes prevents reduction of the volume of voids. This was due to the coarser effect of pumice. In the present work, when the changes in the fine grain size and the economic cost were considered, the optimum pumice and crushed stone dust were selected to be 40% by weight. Therefore, pumice and crushed stone dust have an effect on grain size analysis. This is because the optimum amount of pumice and crushed stone dust can reduce the fineness of fine-grained clay expansive soil.

4.2.1.2 Effects of Pumice – Crushed Stone Dust on Atterbegr's Limit

The basic Atterberg's limit laboratory tests like LL, PL, and PI were conducted to this study on the effect of the pumice and crushed stone dust. Experimental results of liquid limit plastic limit and plastic index while stabilized with the percentage of marble dust are shown in Table 4.8 and 4.9 discussed each percentage of stabilizing and its effects on consistency.

Table 4.8 Atterberg's limit data for pumice- crushed stone dust stabilized soil sample-1

% Of pumice- crushed stone dust	LL	PL	PI	USCS soil classification
Natural soil	87	34	53	A-7-6
5% pumice + 5% crushed stone dust	68	28	40	A-7-6
10% pumice + 10% crushed stone dust	63	33	30	A-7-5
15% pumice + 15% crushed stone dust	38	19	19	A-6
20% pumice + 20% crushed stone dust	44	34	10	A-5

Table 4.9 Atterberg limit data for pumice- crushed stone dust stabilized soil sample-2

% Of pumice - crushed stone dust	LL	PL	PI	USCS soil classification
Natural soil	62	21	41	A-7-6
5% pumice + 5% crushed stone dust	60	26	34	A-7-6
10% pumice + 10% crushed stone dust	43	21	22	A-7-5
15% pumice + 15% crushed stone dust	40	23	17	A-6
20% pumice + 20% crushed stone dust	36	25	9	A-5

As shown in Table 4.8 and 4.9, the LL decrease from a control value of 87% to 44% and from 62% to 36% for sample-1 and sample-2 respectively. Also, the PI value decreased from a control value of 53% to 10% and 41% to 9% for both sample-1 and sample-2 respectively. From Table 4.11 and 4.12, it was indicated that the highest decrease in PI value was observed at 20% of pumice combined with 20% crushed stone dust for both sample-1 and sample-2. The effect of pumice and crushed stone dust mixes in varying proportions with natural expansive subgrade soil had been studied and the variation in consistency limit for various additive mix-ratio. It was found that as increases the percentage of pumice and crushed stone dust, decreases the LL and the PI for both sample-1 and sample -2 respectively. As a result, the PI also decreased followed by an increase in the addition of pumice and crushed dust contents.

This is due to the addition of non-plastic properties of pumice and crushed stone dust. The non-plastic property of pumice and the large surface area of stone dust caused a decrease in the PI value of stabilized soil.

For validation, from previous study the LL and PI were reduced by 31% and 35% respectively at ideal amount of 30% crushed stone dust blended with expansive clayey subgrade soil and also it decreased by 40% and 44% respectively at ideal amount of 30% of pumice alone blended with expansive subgrade soil. However, the liquid limit and Plastic Index of the stabilized expansive soil is reduced by 51% (LL) and 81% (PI) with the addition of 20% of pumice and 20% of crushed stone dust blende with expansive subgrade soil.

This result shows that the plastic index of the expansive soil blended with pumice and crushed stone dust mixture reduced further as compared to soil treated with pumice and crushed stone dust alone.

From the result, the PI values 10% and 9% and percentage passing greater than 35%, according to AASHTO and ERA manuals, LL minimum 41%, PI maximum 10%, soil classified as A-5 and generally rating subgrade as fair subgrade soil (AASHTO, 2005). In the present work, when the changes in the plasticity index values and the economies cost were considered, the optimum pumice and crushed stone dust amount was selected to be 40% by weight of soil. Generally, both samples were fair to use as subgrade after being stabilized. Hence, pumice and crushed stone dust have significant effects on Atterberg's limits of expansive natural soil samples.

4.2.1.3 Effects of Pumice-Crushed Stone Dust on Moisture- Density Relation

The tests were stabilizing of 10, 20, 30, and 40% of pumice and crushed stone dust with expansive subgrade soil samples by dry weight. The obtained results of sample-1 and sample-2 of moisture contents versus dry density graph to determine MDD and OMC are shown in Figures 4.7 and 4.8. From laboratory test results the MDD and OMC for the mix ratio of 10% of pumice and crushed stone dust is 1.53 and 23.57%, for 20% of pumice and crushed stone dust is 1.56 and 20.1%, for 30% of pumice and crushed stone dust is 1.64, and 18.52%, and for 40% of pumice and crushed stone dust is 1.68, and 13.78% for sample-1. For sample-2 the MDD and OMC values are 1.55 and 22.44% for natural soil, 1.6 and 18.68% for 10% of pumice and crushed stone dust, 1.64 and 18.26% for 20% of pumice and crushed stone dust, 1.67 and 15.68% for 30% of pumice and crushed stone dust, and 1.71 and 14.7% for 40% of pumice and crushed stone dust each equal amount of pumice -crushed stone dust mix ratio by weight of dry soil respectively.

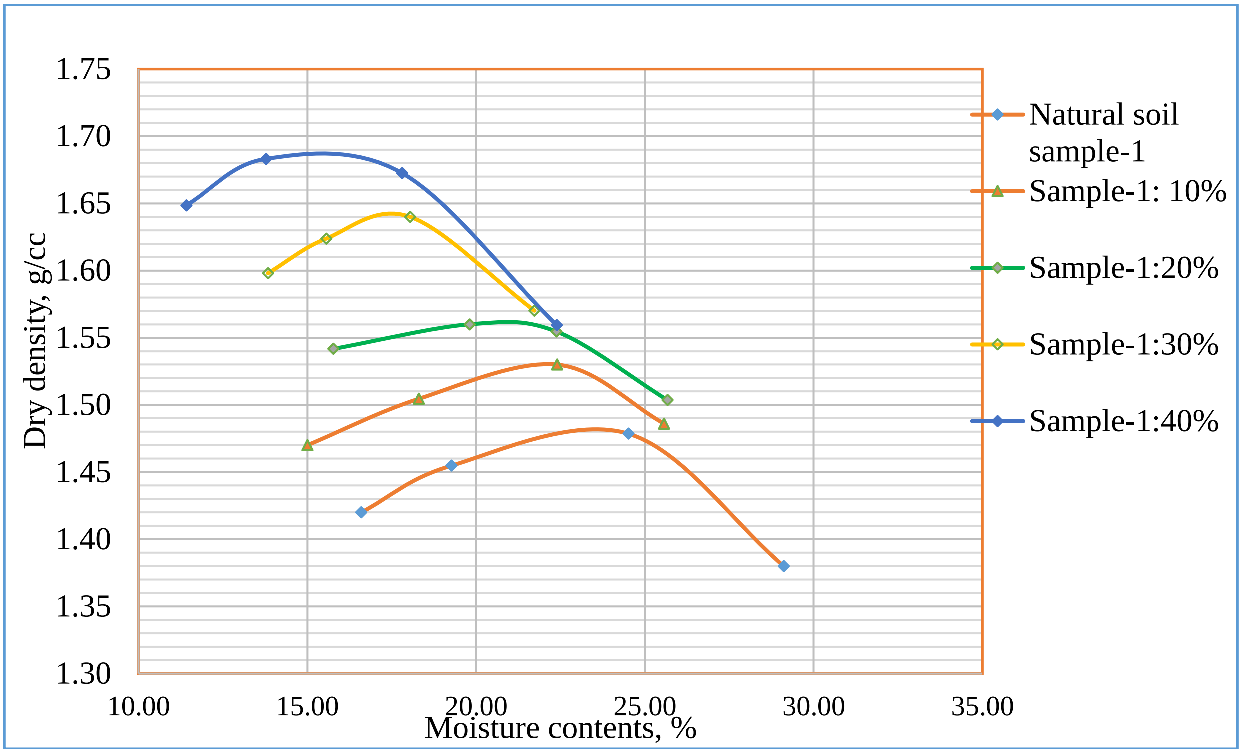

Figure 4.7 Summary of OMC and MDD curve for pumice and crushed stone dust stabilized soil sample-1

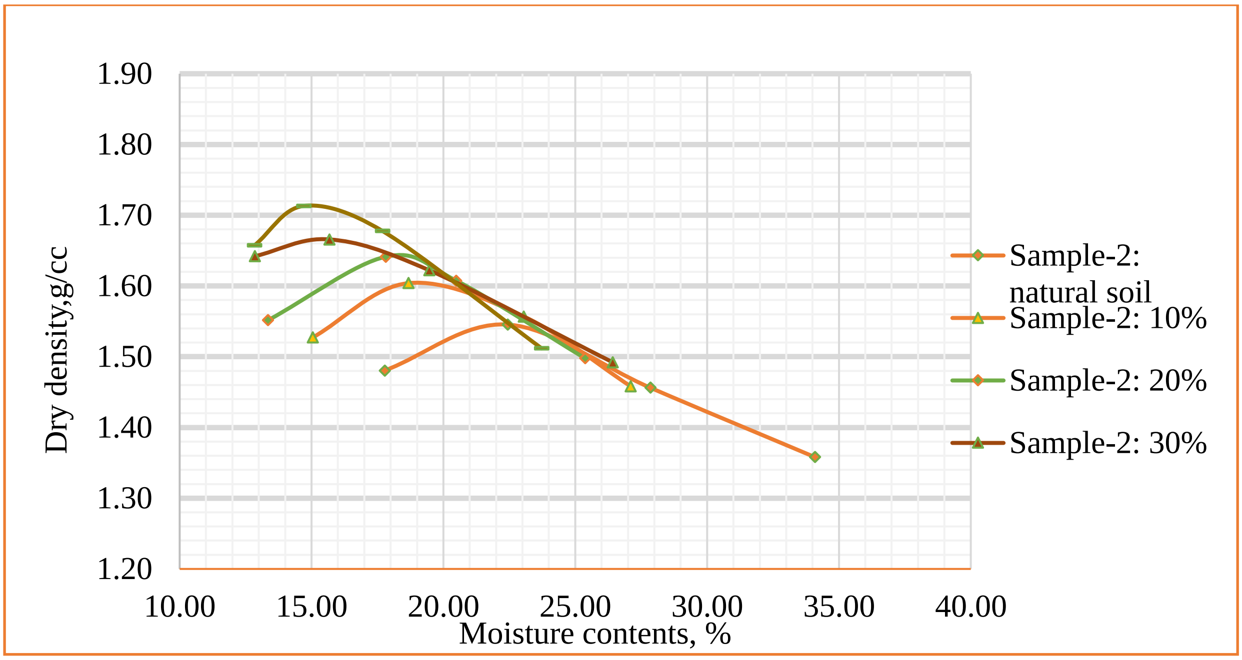

Figure 4.8 Summary of OMC and MDD curve for pumice and crushed stone dust stabilized soil sample-2

The effect of pumice – crushed stone dust stabilization on MDD and OMC of both samples were obtained by modified compaction tests. The results obtained from the laboratory show that soil sample-1 and sample-2, the pumice, and crushed stone dust were added in the proportion of 10, 20, 30, and 40% by weight of dry soil. The MDD shows a continual increase from 1.48 g/cm^3 to 1.68 g/cm^3 for sample-1 and 1.55 g/cm^3 to 1.71 g/cm^3 for sample-2 and decreases OMC from 24.51% to 13.78% for sample-1 and from 22.44% to 14.7% for sample-2 with increasing pumice and crushed stone dust content. Figure 4.9 and 4.10 shows the relation between optimum moisture contents and maximum dry density. From results, it is found that by addition of pumice and crushed stone dust in the proportion of 10, 20, 30, and 40% by dry weight of soil sample, the percentage increase in maximum dry density is found to be 3.38, 5.405, 10.8, and 13.51 %, and 3.22, 5.81, 7.74 and 10.3% for sample-1 and ample-2 respectively. Thus, as the percentage of pumice and crushed stone dust increase maximum dry density increases. The reason for these increments of MMD with the addition of pumice and crushed stone dust is, due to the heavier property and larger surface area of crushed stone dust with porous pumice by rearrangement of modified materials by compaction, increases its maximum dry density. And also due to the crushed stone dust occupied part of the pore space within the soil samples, thus reducing the volume of voids within it. This reduction of void space is responsible for the reduction in the ease with which water flows through the soil. It may be also the addition of non-plastic materials which improve the binding capacity of modified material. Hence, based on the required strength and economic cost, it is concluded that the maximum dry density and OMC were attained when soils blended with 20% of pumice and 20% of crushed stone dust. Therefore, it is concluded that both admixtures have a significant effect on MDD and OMC and the experimental result is valid.

4.2.1.4 Effects of pumice - crushed stone dust on CBR and CBR swell

The soaked CBR test was carried out on the prepared soil samples with different percentages of pumice and crushed stone dust. The results obtained here, shows the relationship between California Bearing Ratio, CBR value, and percentage of pumice and crushed stone dust. The results in Tables 4.10 and 4.11 show that the CBR value increase as the percentage proportion of pumice and crushed stone increased. Prove reading ring load graphs are above the natural soil, which indicated that pumice and crushed dust can improve the strength of expansive subgrade soil in Table 4.10.

Table 4.10 CBR and CBR swell result for pumice- crushed stone dust stabilized sample-1

%Of pumice + crushed stone dust	penetration	Aver. Prove ring reading load	CBR (%)	Max CBR (%)	ERA mini stand. CBR (%)	CBR swelling	Remark
NS (0%)	2.54	0.15	1.14	1.1	>3%	5.60	Poor
	5.08	0.19	0.93				
5% P+ 5% CSD	2.54	0.25	1.86	1.7		4.51	Poor
	5.08	0.29	1.47				
10% P+ 10% CSD	2.54	0.35	2.72	2.7		3.72	Poor
	5.08	0.46	2.30				
15% P+1 5% CSD	2.54	0.49	3.71	3.8		3.58	Satisfied
	5.08	0.84	4.21				
20% P+ 20% CSD	2.54	0.61	4.62	5		1.86	Satisfied
	5.08	1.11	5.55				

Table 4.11 CBR and CBR swell to result for pumice- crushed stone dust stabilized sample-2

Pumice + crushed stone dust	penetration	Aver. Prove ring reading load	CBR (%)	Max CBR (%) at 95% compaction	ERA mini. stand. CBR	CBR swelling	Remark
Natural soil	2.54	0.25	1.89	1.7	>3%	6.81	Poor
	5.08	0.38	1.90				
5% P+ 5% CSD	2.54	0.32	2.45	2.2		6.49	Poor
	5.08	0.62	3.10				
10% P+ 10% CSD	2.54	0.44	3.33	3.4		5.93	Satisfied
	5.08	0.88	4.39				
15% P+1 5% CSD	2.54	0.51	3.87	4.7		3.59	Satisfied
	5.08	1.03	5.13				
20% P+ 20% CSD	2.54	0.48	3.60	5.5		1.92	Satisfied
	5.08	1.16	5.78				

The California Bearing Ratio (CBR) is an indirect measure of the strength of the subgrade. This is also the most widely used method for designing pavement structures. The performance of a road pavement surface is significantly affected by the characteristics of the subgrade. Desirable properties that the subgrade should possess include high strength and stiffness, good drainage, ease of compaction, and low compressibility and swelling.

Generally, the study shows that the increase in the percentage of the proportion of 10, 20, 30, and 40% of pumice -crushed stone dust by dry weight of soil sample, the laboratory result indicates that increase in CBR (%) value from 1.1 to 5 and 1.7 to 5.5 for sample-1 and sample-2 respectively.

According to (Saltan and Keskin, 2015) the CBR value increased by 75% using pumice alone blended with expansive subgrade soil at optimum blend ratio of 30% pumice and according to (Jemal, . *et, al, 2019*) the CBR value increased by 66% using crushed stone dust alone blended with expansive subgrade soil at optimum blend ratio of 30% crushed stone dust. However, the CBR of the stabilized expansive soil is increased by 78% with the addition of 20% of pumice and 20% of crushed stone dust blende with expansive subgrade soil.

From the results the percent increment by increases of pumice- crushed stone dust, the CBR value increased by 35.29, 59.26, 71, and 78%, and 22.73, 50, 63.83, and 69.1% for sample-1 and sample-2 respectively. As shown from Figures 4.11 and 4.12, soil sample-1 and soil sample-2 stabilized by 10, 20, 30, and 40% of pumice and crushed stone dust can strongly improve the strength of expansive soils according to ERA standard (CBR % > 3).

This is due to the increase in the compaction property of stabilized soil with the increased modified mass of soil particles and compaction characteristics of non-plastic materials. Crushed stone dust has a very large surface area than modified soil this increases the bond between modified soil samples. Non- cohesiveness of pumice increased the compaction strength of cohesive soil by increasing the bondage between the soil and the stabilizing agents. When the changes in the CBR values and the economic cost were considered, the optimum amount was selected to be 20% of pumice and 20% of crushed stone dust by weight. This value was used suitable for road construction as subgrade materials in all field studies. Therefore, pumice and crushed stone have an effect on stabilized subgrade soil.

4.2.1.5 Effects of Pumice - Crushed Stone Dust on Shrinkage Limit

Linear shrinkage tests were conducted on this study for 10, 20, 30, and 40% pumice and crushed stone dust by weight of dry soil. The result can be obtained from two sample soils with each 5, 10, 15, and 20% of stabilized pumice- crushed stone dust as shown in Figure 4.9.

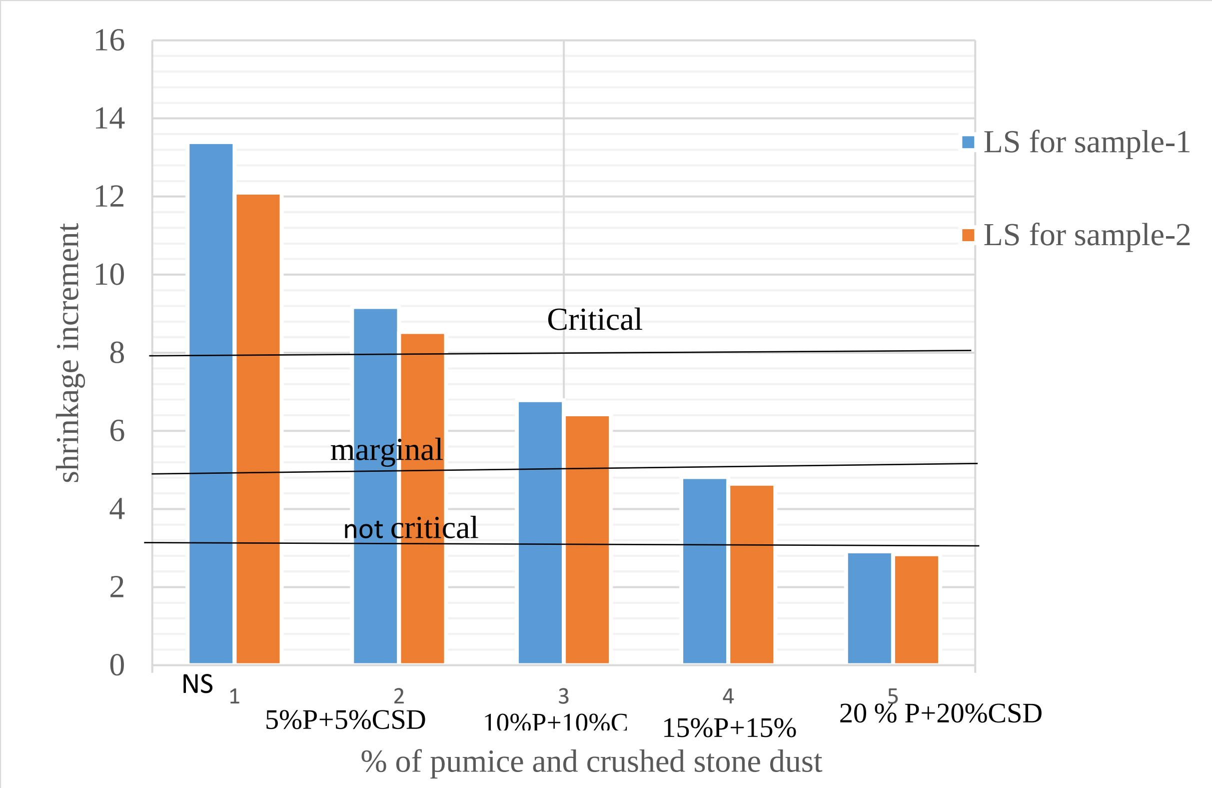

Figure 4.9 summary of linear shrinkage limit graph

The LS shows a continual decrease from 13.4 to 2.92% for sample-1 and 12.1 to 2.84% for sample-2 with increasing pumice and crushed stone dust. Figure 4.9 shows the relation between LS and percentage increase of pumice- crushed stone dust. According to (Saltan and Keskin, 2015) the LS value reduced by 64% using pumice alone blended with expansive subgrade soil at optimum blend ratio of 30% pumice. However, the LS of the stabilized expansive subgrade soil is increased by 78.2% with the addition of 20% of pumice and 20% of crushed stone dust blende with expansive subgrade soil.

Poor geotechnical properties of the subgrade are responsible for the failure of the road pavements. Volume changes in soil can be some very dangerous problems on the highway. The natural soils undergo a large amount of linear shrinkage. After stabilization by 20% pumice and 20% crushed stone dust, the linear shrinkages were decreased to 2.92% and 2.84% which is less than 8% and (not critical in highway design and construction according to Altemeyer, 1955). This decrease in linear shrinkage is due non-plastic effect of pumice and crushed stone dust. The non-shrinkage and expanding behavior of pumice and crushed stone dust lead to decrease linear shrinkage of expansive subgrade soil. Therefore, it is concluded that pumice and crushed stone dust have a significant effect on linear shrinkage and the experiment result is valid.

4.2.1.6 Effects of Pumice - Crushed Stone Dust on Free Swell

The result from the study conducted on the effect of pumice and crushed stone dust on the free swell index of the natural and stabilized soil samples was tabulated in Table 4.12.

Table 4.12 Summary of free swell result for pumice- crushed stone dust stabilized sample-1

Types of samples	Sample-1		Sample-2	
Mix proportion	FSI (%)	IS 1498 requirement	FSI (%)	IS 1498 requirement
Natural soil	81.64	Medium	54.55	Medium
5% P +5 %CSD	63.64	Low	45.5	Low
10 %P+ 10%CSD	45.45	Low	36.4	Low
15 %P+15% CSD	30	Low	27	Low
20 %P+20% CSD	20	Low	9.1	Low

For soil sample-1, the free swell index decreased from 81.64% to 20%. For soil sample-2 the free swell index decreased from 54.55% to 9.1% with an increasing percentage of pumice and crushed stone dust 10, 20, 30, and 40% by dry weight of soil sample respectively. The lowest result was at 20% mix of pumice blended with 20% crushed stone dust for both sample-1 and sample-2 respectively. This indicates that 20% pumice and 20% crushed stone dust was the optimum ratio of additive content to achieve a remarkable free swell index value. according to (Jemal, . *et, al, 2019*) the FS value reduced by 68% using crushed stone dust alone blended with expansive subgrade soil at optimum blend ratio of 30% crushed stone dust. However, the FS of the stabilized expansive subgrade soil is reduced by 77.5% with the addition of 20% of pumice and 20% of crushed stone dust blende with expansive subgrade soil.

According to Is 1498:1970, (2016), soils having a free swell value above 100 can cause damage whereas free swell as low as 100% can cause considerable damage to lightly loaded structures, and soils having an FSI value below 50% seldom exhibits appreciable volume change even under light loads. The decrease in the free swell index is due to the non-plastic property of pumice and crushed stone dust which had no expanding behavior in itself. So, the decrease in the free swell index is due to the non-swelling behavior of pumice and crushed stone dust. Therefore, it is concluded that pumice and crushed stone dust have a significant effect on the free swell and the experiment result is valid.

4.2.1.7 Effects of Pumice - Crushed Stone Dust on Unconfined Compressive Strength

Unconfined compressive strength tests were carried out on soil pumice – crushed stone dust mixture by varying percentages of pumice and crushed stone dust each by 10, 20, 30, and 40% by dry weight expansive soil samples. The summary of the unconfined compressive strength is shown in Table 4.13.

Table 4.13 Axial stress versus axial deformation of stabilized soil with combined pumice and crushed stone dust.

Types of samples	Sample-1		Sample-2	
Mix proportion	UCS	Shear strength	UCS	Shear strength
Natural soil	33 kpa	16.5kpa	42.75 kpa	21.65 kpa
5% P +5 %CSD	37.26 kpa	19 kpa	51.4 kpa	25.7 kpa
10 %P+ 10%CSD	43.3 kpa	21.65 kpa	60 kpa	30 kpa
15 %P+15% CSD	64.78 kpa	32.4 kpa	110 kpa	55 kpa
20 %P+20% CSD	99 kpa	45 kpa	94 kpa	47 kpa

The result from the conducted test showed that the increases in percentage mix of pumice-crushed stone dust increased the compressive strength of stabilized soil from 33.0044 Kpa to 99 Kpa and from 42.75 Kpa to 110 Kpa for samples -1 and sample-2 respectively.

As it can be indicated in Table 4.13, while the percentage of pumice and crushed stone dust increase the unconfined compressive strength becomes increased. For sample-1 the increase of mix from 10% to 40% the compressive strength of specimen increase continues up to 40% and for sample-2 the increases in contents of pumice and crushed stone dust the unconfined compressive strength of specimen increased up to 30% then slightly decreased. This indicated that partial replacement of 20%, pumice, and 20% crushed stone dust has a significant effect on the improvement of unconfined compressive strength of specimens for sample-1 and 15% pumice, and 15% crushed stone dust improves the unconfined compressive strength of specimens for sample-2. As the percentage of both pumice and crushed stone dust increase, axial-strain peak failure (UCS) increased because the shear resistance of specimens increased. The experimental result summarized in Table 4.13, indicated that UCS increased, with an increased percentage of pumice and crushed stone dust. The occurrence of maximum axial stress (peak failure) of pumice-crushed stone dust stabilized soil is corresponding to 20 % pumice content and 20% crushed stone dust dosage for sample-1. The occurrence of maximum axial stress (peak failure) of pumice-crushed stone dust-stabilized soil is corresponding to 15% pumice content and 15% crushed stone dust dosage for sample-2. However, regarding the economic cost, 20% of pumice and 20% crushed stone dust with 99 kpa for sample-1 and 15% of pumice and 15% crushed stone dust with 110 kpa for sample-2 of unconfined compressive strength was selected in this study since it satisfies the minimum requirement, 50 kpa.

The laboratory result proved that the added percentages of both admixtures increase the unconfined compressive strength of stabilized soil. The reason for the increasing unconfined compressive strength of stabilized soil is maybe due non-plastic property of blending materials and the rearrangement of modified soil during remolding time. This is due to the porous materials of pumice materials were rearranged by large surface area materials of clayey soil and fine dust filled porous pumice surface by compaction that increases the unconfined compressive strength of modified soil. The UCS increased, with an increased percentage of pumice and crushed stone dust. Therefore, it is concluded that pumice and crushed stone dust have a significant effect on expansive subgrade soil and the experiment result is valid.

4.3 Cost Analysis of Stabilization with Respect to Cart away and Borrow Fill Material.

4.3.1 Cost Analysis of Selective Borrow Materials

In this study, the thickness of the natural subgrade was determined from charts B1 by correlating subgrade CBR value versus the traffic class which was determined from traffic survey in this study case T4, and S_1 from laboratory test the researcher obtained value CBR (%) = 1.1 and 1.7 for sample 1 and sample 2 respectively, which is classified as subgrade S_1 (CBR<3). The researcher assumed the design road is asphalt concrete. From chart B1 (ERA, 2013) the researcher got each thickness of thin (50 mm) AC, granular base course, granular subbase courses, and capping layer respectively, 175 mm, 250 mm, and 300 mm. The total depth for un stabilized subgrade soil is 725 mm. After stabilization, the subgrade increased from S1 to S3 and the total depth is decreased to 500 mm. The top of embankment or bottom of excavation prior to construction of the pavement structure. Where very weak soils and/or expansive soils are encountered, a capping layer is sometimes necessary. This consists of better-quality subgrade material imported from elsewhere or subgrade material improved by stabilization (usually mechanical), and may also be considered as a lower quality subbase. From this, the capping layer of 300 mm replacement by filling material was calculated and compared with the stabilizing of that weak subgrade by pumice- crushed stone dust. The replacement of selected borrow material to be used as subgrade for the paved road should reveal a CBR (%) value of 5-7 and have a very low swell potential of less than 2%, (LL<30% and PI < 20%) (ERA, 2013).

To compare this replacing materials cost with stabilizing material cost the researcher gathers the information about the road segment that the stabilization had taken on. So, the road segment has two lanes with the median at Zone administration and two lanes without median at district administration and including a walkway of 2.5 m in both directions. In kebele administration, the road has a shoulder of 1.5m on both sides. To calculate the estimated quantity within a 1 km strip of road section the researcher used a one-lane road of a kilometer. Though the selected material pits are available near the site, taking into account the shortest distance at an average distance of 10 km is selected for this analysis. During the process for replacing cost analysis, the work breakdown structures should be conducted. When the unit rate of the selected borrow material, the production and hauling distance of the soil were considered. The unit and performance rate of replaced selective materials are detailed in Table 4.14.

Table 4.14 cost of replacement of natural expansive soil.

Items	description	unit	Rate (ETB)	Length (m)	Width(m)	Depth(m)	Quantity (m^3)	Total rate
1	Bulk excavation in soft soil	m^3	44	1000	7	0.3	2100	92,400
2	Hauling soft soil	m^3	48	1000	7	0.3	2100	100,800
4	Roadbed preparation and compaction to 93% MDD	m^2	41	1000	7	-	7000	287,000
5	Cost for acquiring quarry site	m^2	16 for 15 years	50	50	-	2,500	600,000
6	Excavation of borrow materials to compact 300 mm	m^3	91.53	1000	7	0.3	2100	192213
7	Transportation of excavated borrow material to the construction site	m^3	48	1000	7	0.3	2100	100,800
8	Total cost						1,373,213 ETB	

4.3.2 Cost of pumice – crushed stone dust stabilized soil

In the present study, the suitability and workability of this soil were under question for highway construction as subgrade material. Therefore, the decision to stabilize the soil with the percentage of pumice 20% blending with crushed stone dust 20% was conducted to enhance the properties of soft soil. For comparison pumice from 20 km, Konta Special district, and crushed stone dust from Chida crusher plant.

Cost analysis of flexible pavement, designed to only a single layer of subgrade soils was carried out for laying a kilometer road length with a road width of the driving lane. The construction cost analysis obtained from the current bid price of the study area involves the construction material, labor, equipment, transport, fuel, mixing, laying, loading, unloading, and hauling distance. During cost analysis, the unit rate of the stabilized material, work breakdown structure, and performance rate of each activity were considered. The detailed cost analysis is summarized in Table 4.15.

Table 4.15 he unit and performance rate of stabilized materials

Items	description	unit	Rate (ETB)	Length (m)	Width(m)	Depth(m)	Quantity (m^3)	Total rate
1	The purchase cost of pumice	m^3	230	1000	7	0.08	560	200000.02
2	Transport with a hauling distance of 20km	m^3	48	1000	7	0.08	560	26,880
3	Mixing and preparing for compaction	m^3	23	1000	7	0.08	560	12880
4	The purchase cost of crushed stone dust	m^3	1428.571	1000	7	0.08	560	799999.76
5	Transport cost with hauling distance of 5km	m^3	43	1000	7	0.08	560	24080
6	Mixing and preparing for compaction	m^3	23	1000	7	.08	560	12880
7	Total cost						1,005,519	

4.3.3 Cost Comparison

A comparison of the cost was done for a flexible pavement construction (single subgrade layer) based on two options. The first approach is the replacement of the expansive soil by available selective borrow materials and the second alternative is one of the innovative non-conventional technologies that utilize stabilized soil of 20% pumice and 20% of crushed stone dust by weight of the expansive soil dry weight. While in cost comparison, the basic assumption was taken. The surfacing layer (above subgrade layer) cost was not considered while the comparison. Quantity tabulation was carried out for laying a kilometer road Length with a width of the driving lane (7 m). From cost comparison, the stabilizing of expansive subgrade soil by using 20% pumice and 20% crushed stone dust, the result obtained 80mm thick pumice and 80mm crushed stone dust for generally required capping layer (removal and replaced by selected material) of 300mm this is calculated from the compaction concept 25% compacted by heavy compacted from gross depth. Generally, to produce a 300 mm capping layer it needs 400 mm gross depth before compaction. So, the result reduced the overall thickness of the pavement structure layer thickness from 725 mm to 500 mm, for sample-1 and sample-2 respectively after heavy compaction. This was by increasing the subgrade soil strength CBR (%) of stabilized subgrade soil from 1.1 to 5 and 1.7 to 5.5 respectively. From this value of CBR (%), the subgrade (S_1) was increased to the corresponding S_3 for sample -1 and sample -2 after stabilization.

Generally, it can be observed that the total cost of constructed flexible road pavement using replacement of the expansive soil by non-expansive soil is equal to 1,373,213 ETB, whereas stabilized soil with admixture (20% pumice and 20% of crushed stone dust) by dry weight of soil is 1,005,519ETB. Hence the construction of pavement layers (subgrade) with pumice and crushed stone dust bound is cheaper than the pavement structure with the replacement of the expansive soil by non-expansive soil. The cost savings obtained using stabilized soil with both admixtures is relatively equal to 26.77%.

From cost savings, it can be detected that utilization of pumice – crushed stone dust-stabilized soil, is an economically feasible non-conventional stabilization option for the design and construction of roads, while they are readily available.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

5.1 CONCLUSIONS

Based on the laboratory test results, all the engineering properties of natural soil coincided with the property of fine-grained soils (clay). From grain size analysis of natural soil sample, the fine grain sizes were 95.76% and 93.89% for sample-1 and sample-2 respectively and the result shows that soil samples were highly clay.

The natural soil samples were used in this study were expansive, high plastic index, poor in strength. From the test results of Atterberg's limit and grain size analysis, soil samples are classified as A-7-6 and CH according to AASHTO and USCS for samples -1 and sample-2 respectively.

The engineering properties of the studied expansive subgrade soil revealed that it was not suitable to use as a subgrade layer material unless otherwise, its poor engineering properties were improved. Stabilized expansive subgrade soil samples with pumice and crushed stone dust decrease free swell from 81.82 to 20% sample-1 and 54.55 to 9.1% sample-2. The PI of stabilized soil sample decrease from 53 to 10% for sample -1 and 41 to 9% for sample-2. The MDD of stabilized soil increased from 1.48 to 1.68% and 1.55 to 1.71% and OMC decreased from 24.51 to 13.78% and 22.44 to 14.7% for both soil samples 1- and sample-2 respectively. The CBR (%) of stabilized soil increased from 1.1 to 5 and 1.7 to 5.5 for sample-1 and sample-2 respectively. Generally, most parameters of ERA, (2013) specification requirement was achieved. The engineering properties of expansive soil were improved by pumice with crushed stone dust in different mix-proportion. The optimum amount for adequate stabilization was determined to be 20% P + 20% CSD. Therefore, it was deduced that 20% pumice blended with 20% crushed stone dust was optimum and value cost comparison, the cost of stabilized soil with pumice and crushed stone dust is cheaper than that of the replacement of the expansive soil by selected material. The cost savings obtained using stabilized soil with both admixtures are relatively equal to 26.77%.

5.2 RECOMMENDATION

- As investigated in this study, the stabilization of expansive soil with pumice and crushed stone dust by the only mechanical effect on stabilized soil. Therefore, it is recommended to further study on chemical effects of pumice on expansive soil by curing the specimen. This is because the researcher did not consider the pozzolanic effect of pumice.
- The current study was conducted by taking limited parameters such as CBR, CBR swell, linear shrinkage, free swell, moisture density relation, of expansive soil sample. The addition test parameter like PH value, volumetric mineralogical, and sensitivity of remolded soil sample should also be performed to have more accurate test results.
- Since soil characteristics value varies from one location to the next location through route, the benefits of soil stabilization to achieve higher uniformity is inferred, but not quantified in this study. Further studies to investigate the effect of roadbed support uniformity on pavement performance and to quantify the benefits of more uniform support are recommended.
- From study findings stabilization of expansive soil by 20% pumice and 20% crushed stone dust, the cost analysis was done for the area where pumice availability is not in long distance. For further study, is recommended cost analysis before using this research where the pumice availability is in long distance.
- In research, the effect of crushed stone dust on the environment is not counted in cost analysis so, in further study, it is possible to include the cost-effectiveness of stone dust on the environment.

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APPENDIXES

Appendix A: Natural moisture content

Moisture Content Determination for sample -1

Sample location	Sample-1		
Trial numbers	1	2	3
can code	A	9	J41
A= Mass of can (Mc), gm	36.52	17.38	33.04
B= mass of can +moisture soil (Mcms), gm	99.35	98.35	102.58
C=mass of water can + mass of oven dried soil (Mcds), gm	80.21	73.8	81.4
D= mass of water (Ms) Mw= C-B	19.14	24.55	21.18
E = mass of dry soil (Md) Md =C-A	43.69	56.42	48.36
water content (w), % Wc=Mw/Md	43.81	43.513	43.7966
average water content (w), %		43.7	

Moisture Content Determination for sample-2

sample-2 natural water content	Sample-2		
can code	P1	C15	T2
A= Mass of can (Mc), gm	18.8	25.5	26.5
B= mass of can +moisture soil (Mcms), gm	99.35	98.35	102.58
C=Mass of water can + mass of oven dried soil (Mcds), gm	76.94	78.17	81.42
D= mass of water (Ms) Mw= C-B	22.41	20.18	21.16
E = mass of dry soil (Md) Md =C-A	58.14	52.67	54.92
water content (w), % Wc=Mw/Md	38.545	38.315	38.52
average water content (w), %		38.47	

APPENDIX B: Sample Specific Gravity Test Analysis Data

Sample -1 Natural soil		
code	P1(black)|	P2(White)
mass of pycno	27.09	22.53
mass of pycno + soil	52.18	47.53
mass of pycno + soil +water (after 24 hr)	137.2	137.5
mass of pycno +water	121.81	126.29
Ti (temperature)	23	23
Distilled Ti	0.99757	0.99757
Tx	22	22
D Tx	0.9978	0.9978
Corrected mass of pycno +water	121.83808	126.31912
K	0.9996	0.9996
GS (specific gravity)	2.5791305	2.677
Average specific gravity	2.628	
	clay	
	Gs=2.67	

Sample-2, Natural soil		
code	P1(black)|	3
mass of pycno	27.14	22.57
mass of pycno + soil	52.4	47.63
mass of pycno + soil +water (after 24 hr)	141.99	140.97
mass of pycno +water	125.59	121.84
Ti (temperature)	21	21
Distilled Ti	0.99757	0.99757
Tx	22	22
D Tx	0.99802	0.99802
Corrected mass of pycno +water	125.65	121.9
K	0.9998	0.9998
GS (specific gravity)	2.833	2.574
Average specific gravity	2.703	
	clay	
	Gs=2.70	

APPENDIX C: Grain Size Distribution Test Analysis Data

Wet Sieve and Hydrometer analysis

Wet sieve analysis for sample-1 natural soil				
Sieve size (mm)	Mass of retain on each seive(g)	Percentage of retained soil	Cumulative % of retain soil	Percentage of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	2.50	0.28	0.28	99.72
2.36	5.50	0.62	0.90	99.10
2	1.70	0.19	1.10	98.90
1.18	5.40	0.61	1.71	98.29
0.85	1.80	0.20	1.91	98.09
0.6	4.60	0.52	2.43	97.57
0.425	1.50	0.17	2.60	97.40
0.3	2.10	0.24	2.84	97.16
0.15	3.80	0.43	3.27	96.73
0.075	7.40	0.84	4.11	95.89
pan	847.80	95.89	100.00	0.00
Sum	884.1			

Hydrometer analysis sample-1

Elapsed time, min	temp. 0c	Rh	Meniscus Correction Cm = +1	Hydrometer reading corrected for meniscus (RC)	Effective Depth, L (mm)	K (Table)	Particle Diameter (mm)	Ct from Table	Cd	Corr. Hydr. Rdg. Rc	a	% Finer P	% Adjusted Finer PA
0.5	23	53.8	1	54.78	0.0	0.01297	0.00	0.7	7	48.48	0.989	95.88	87.05
1	23	51	1	52	0.0	0.01297	0.000	0.7	7	45.7	0.989	90.38	82.06
2	23	50.5	1	51.5	0.0	0.01297	0.000	0.7	7	45.2	0.989	89.39	81.16
4	23	48	1	49	0.0	0.01297	0.000	0.7	7	42.7	0.989	84.45	76.67
8	23	47	1	48	0.0	0.01297	0.000	0.7	7	41.7	0.989	82.47	74.87
15	23	45.5	1	46.5	0.0	0.01297	0.000	0.7	7	40.2	0.989	79.50	72.18

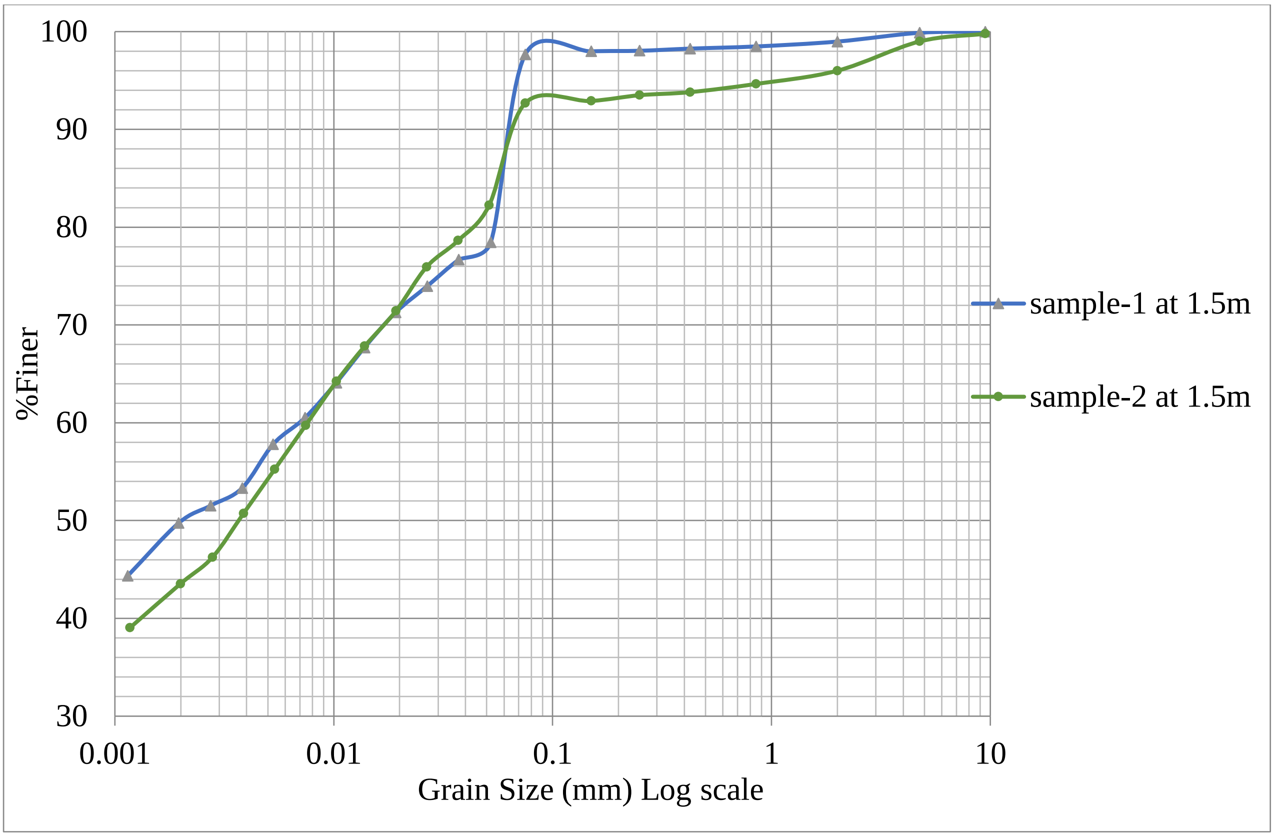
30	23	43	1	44	0.0	0.01297	0.000	0.7	7	37.7	0.989	74.56	67.69
60	23	42	1	43	0.0	0.01297	0.000	0.7	7	36.7	0.989	72.58	65.90
120	23	41.5	1	42.5	0.0	0.01297	0.000	0.7	7	36.2	0.989	71.59	65.00
240	23	40	1	41	0.0	0.01297	0.000	0.7	7	34.7	0.989	68.63	62.30
480	23	37	1	38	0.0	0.013	0.000	0.7	7	31.7	1	62.69	56.92
1440	23	35	1	36	0.0	0.01297	0.000	0.7	7	30	1.0	58.74	53.33

Wet sieve analysis for sample-2, natural soil

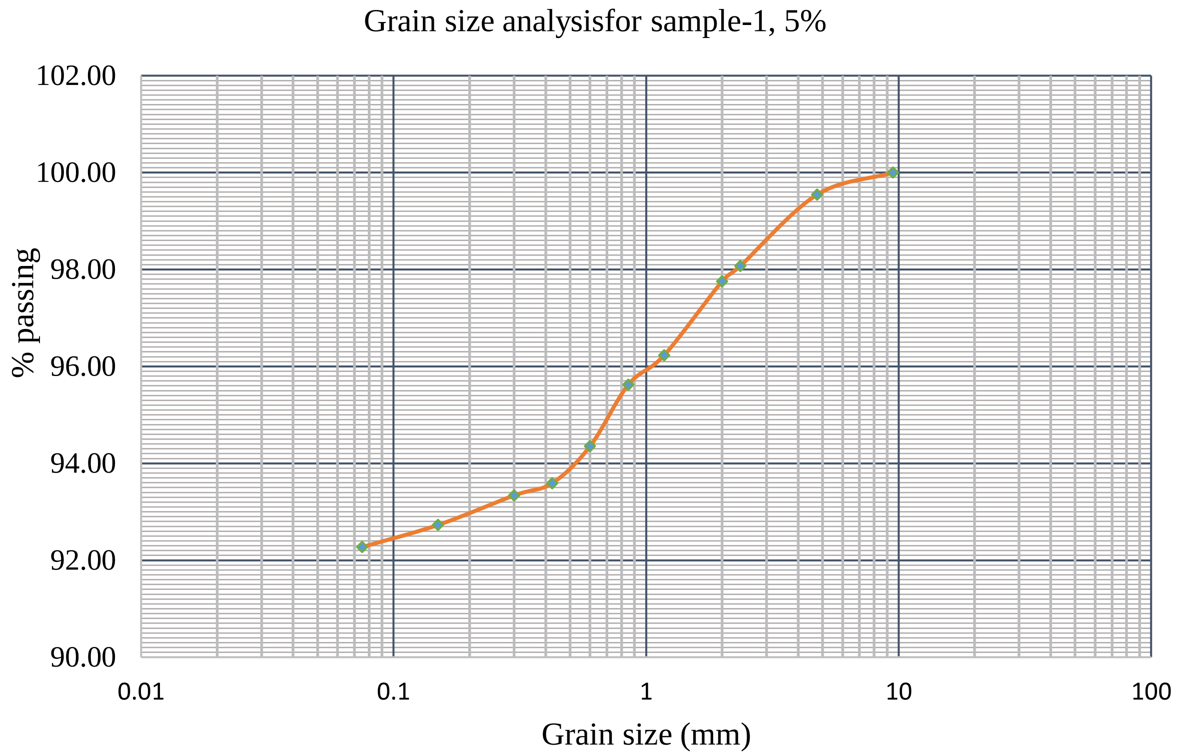
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	0.10	0.01	0.01	99.99
2.36	0.80	0.08	0.09	99.91
2	4.60	0.47	0.57	99.43
1.18	2.50	0.26	0.83	99.17
0.85	2.41	0.25	1.07	98.93
0.6	6.50	0.67	1.74	98.26
0.425	2.80	0.29	2.03	97.97
0.3	5.50	0.57	2.60	97.40
0.15	12.40	1.28	3.88	96.12
0.075	22.90	2.36	6.24	93.76
pan	908.60	93.76	100.00	0.00
Sum	969.1			

Sample-2 hydrometer analysis

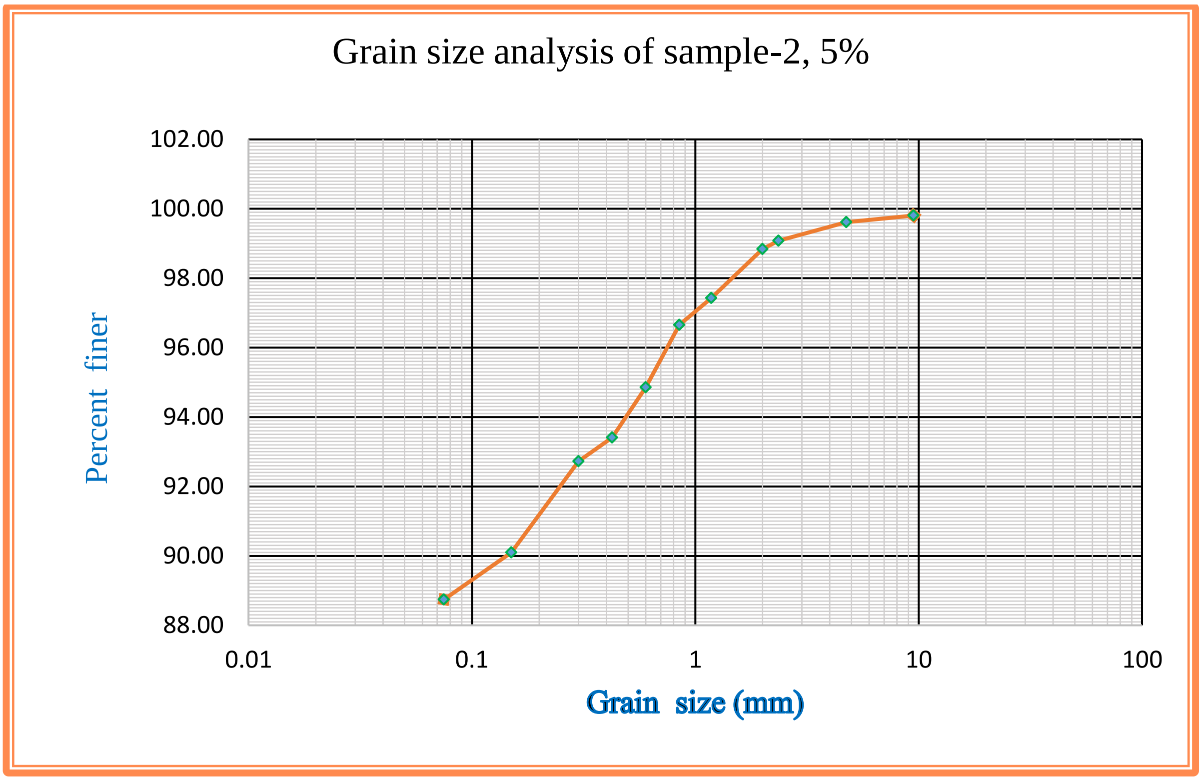
Elapsed time, min	temp. 0c	Rh	Meniscus Correction Cm = +1	Hydrometer reading corrected for meniscus (RC)	Effective Depth, L (mm)	K (Table)	Particle Diameter (mm)	Ct from Table	Cd	Corr. Hydr. Rdg. Rc	a	% Finer P	% Adjusted Finer PA
0.5	23	52.71	1	53.71	0.0	0.01297	0.000	0.7	7	47.41	0.989	93.76	85.13
1	23	51.5	1	52.5	0.0	0.01297	0.000	0.7	7	46.2	0.989	91.37	82.95
2	23	50.5	1	51.5	0.0	0.01297	0.000	0.7	7	45.2	0.989	89.39	81.16
4	23	49	1	50	0.0	0.01297	0.000	0.7	7	43.7	0.989	86.42	78.46
8	23	47	1	48	0.0	0.01297	0.000	0.7	7	41.7	0.989	82.47	74.87
15	23	46	1	47	0.0	0.013	0.00	0.7	7	40.7	0.989	80.49	73.08
30	23	44	1	45	0.0	0.01297	0.000	0.7	7	38.7	0.989	76.54	69.49
60	23	41.5	1	42.5	0.0	0.01297	0.000	0.7	7	36.2	0.99	71.59	65.00
120	23	40.5	1	41.5	0.0	0.01297	0.000	0.7	7	35.2	0.989	69.61	63.20
240	23	38.5	1	39.5	0.0	0.01297	0.000	0.7	7	33.2	0.989	65.66	59.61
480	23	36	1	37	0.0	0.01297	0.000	0.7	7	30.7	0.989	60.71	55.12
1440	23	30	1	31	0.0	0.01297	0.000	0.7	7	24.7	0.989	48.85	44.35



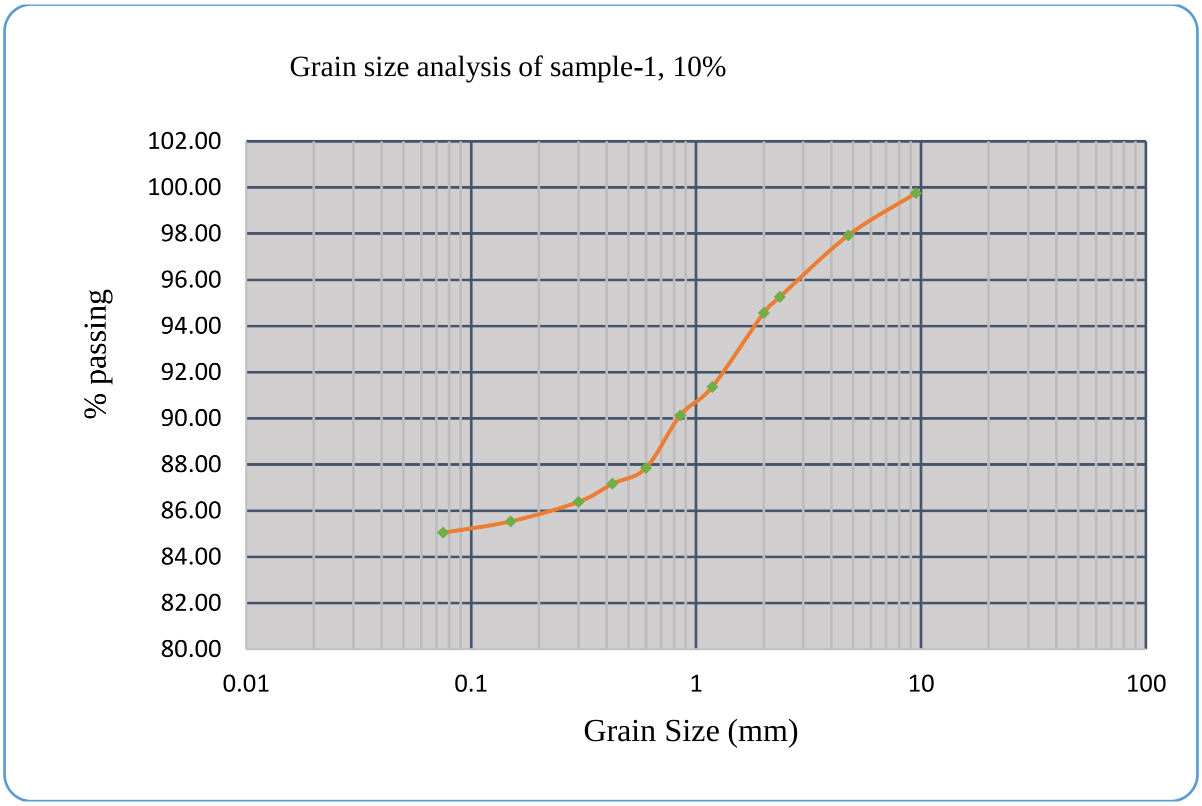
Sample-1: 5%P+5%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve (g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	0.00	0.00	0.00	100.00
4.75	4.50	0.46	0.46	99.54
2.36	14.50	1.47	1.93	98.07
2	3.10	0.31	2.24	97.76
1.18	15.00	1.52	3.77	96.23
0.85	6.00	0.61	4.38	95.62
0.6	12.50	1.27	5.65	94.35
0.425	7.50	0.76	6.41	93.59
0.3	2.50	0.25	6.66	93.34
0.15	6.00	0.61	7.27	92.73
0.075	4.50	0.46	7.73	92.27
Pan	908.60	95.76	100.00	0.00
Sum	984.7			



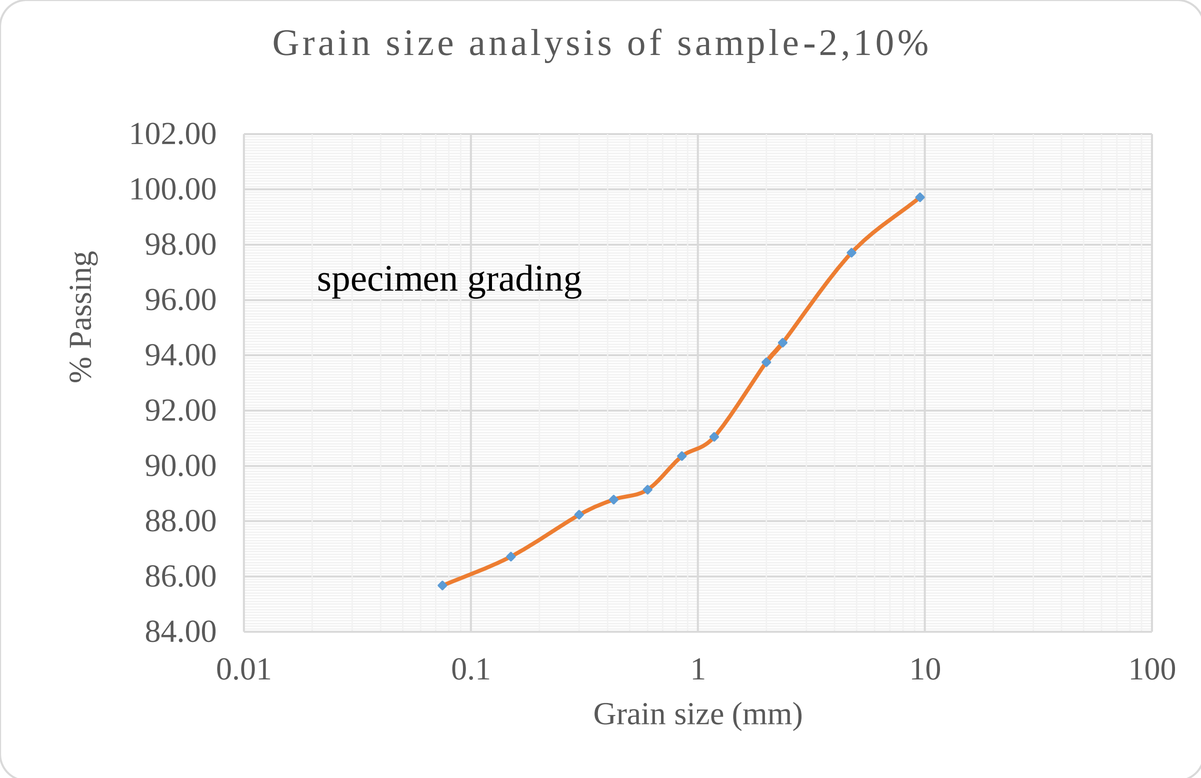
Grain Size Analysis of sample -2: 5%				
Sieve size (mm)	Mass of retaining on each sieve (g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	2.00	0.19	0.19	99.81
4.75	2.00	0.19	0.39	99.61
2.36	5.50	0.53	0.92	99.08
2	2.50	0.24	1.16	98.84
1.18	14.50	1.41	2.57	97.43
0.85	8.00	0.78	3.35	96.65
0.6	18.50	1.79	5.14	94.86
0.425	15.00	1.45	6.59	93.41
0.3	7.00	0.68	7.27	92.73
0.15	27.10	2.63	9.90	90.10
0.075	14.00	1.36	11.26	88.74
pan	908.60	88.10	99.36	0.64
Sum	1031.3			



Sample-1: 10%P+10%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	2.50	0.25	0.25	99.75
4.75	18.50	1.83	2.07	97.93
2.36	27.00	2.67	4.74	95.26
2	7.00	0.69	5.43	94.57
1.18	32.50	3.21	8.64	91.36
0.85	12.50	1.23	9.87	90.13
0.6	23.00	2.27	12.14	87.86
0.425	7.00	0.69	12.83	87.17
0.3	8.00	0.79	13.62	86.38
0.15	8.50	0.84	14.46	85.54
0.075	5.00	0.49	14.96	85.04
pan	848.50	83.76	98.72	1.28
Sum	1013.0			

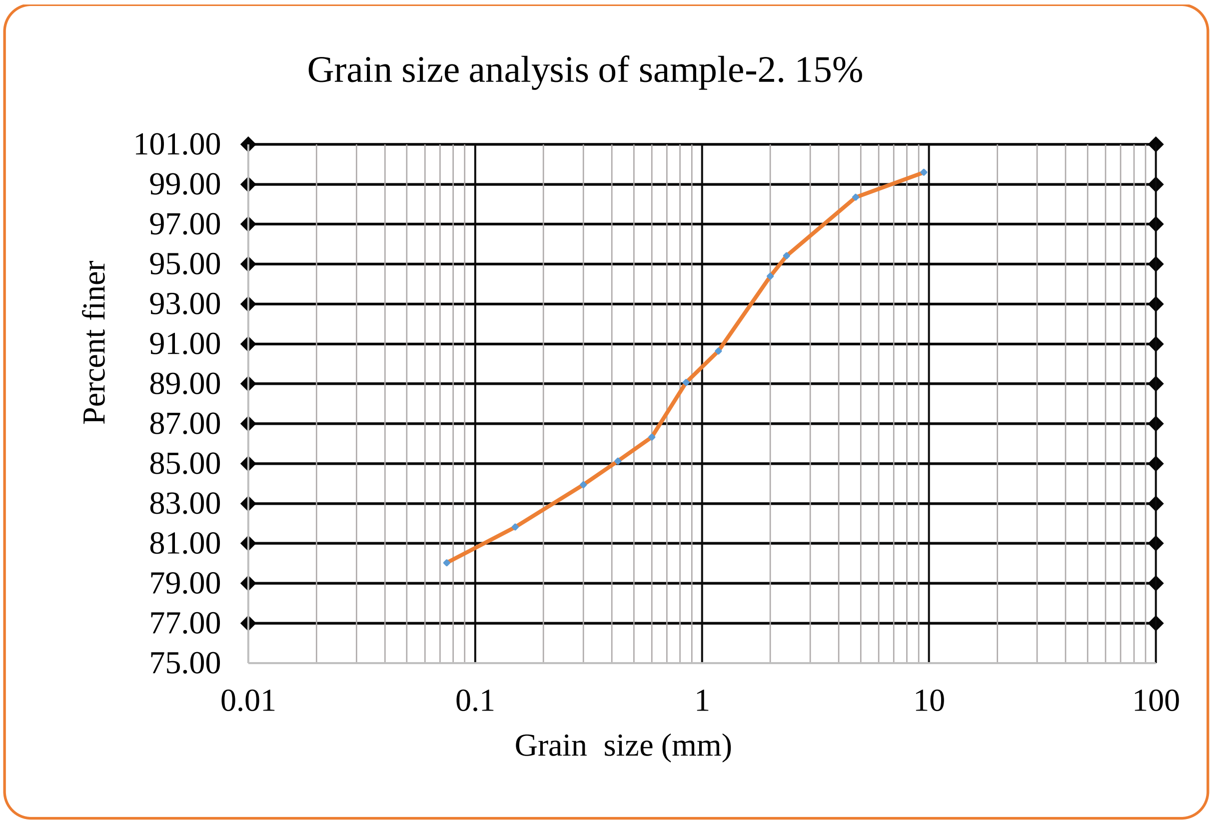


Sample-2: 10%P+10%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	2.90	0.29	0.29	99.71
4.75	20.00	2.00	2.29	97.71
2.36	32.60	3.26	5.55	94.45
2	7.00	0.70	6.25	93.75
1.18	27.00	2.70	8.95	91.05
0.85	7.00	0.70	9.65	90.35
0.6	12.10	1.21	10.86	89.14
0.425	3.60	0.36	11.22	88.78
0.3	5.50	0.55	11.77	88.23
0.15	15.10	1.51	13.28	86.72
0.075	10.50	1.05	14.33	85.67
pan	1000-143.3	85.67	100.00	0.00
Sum	1000.0			

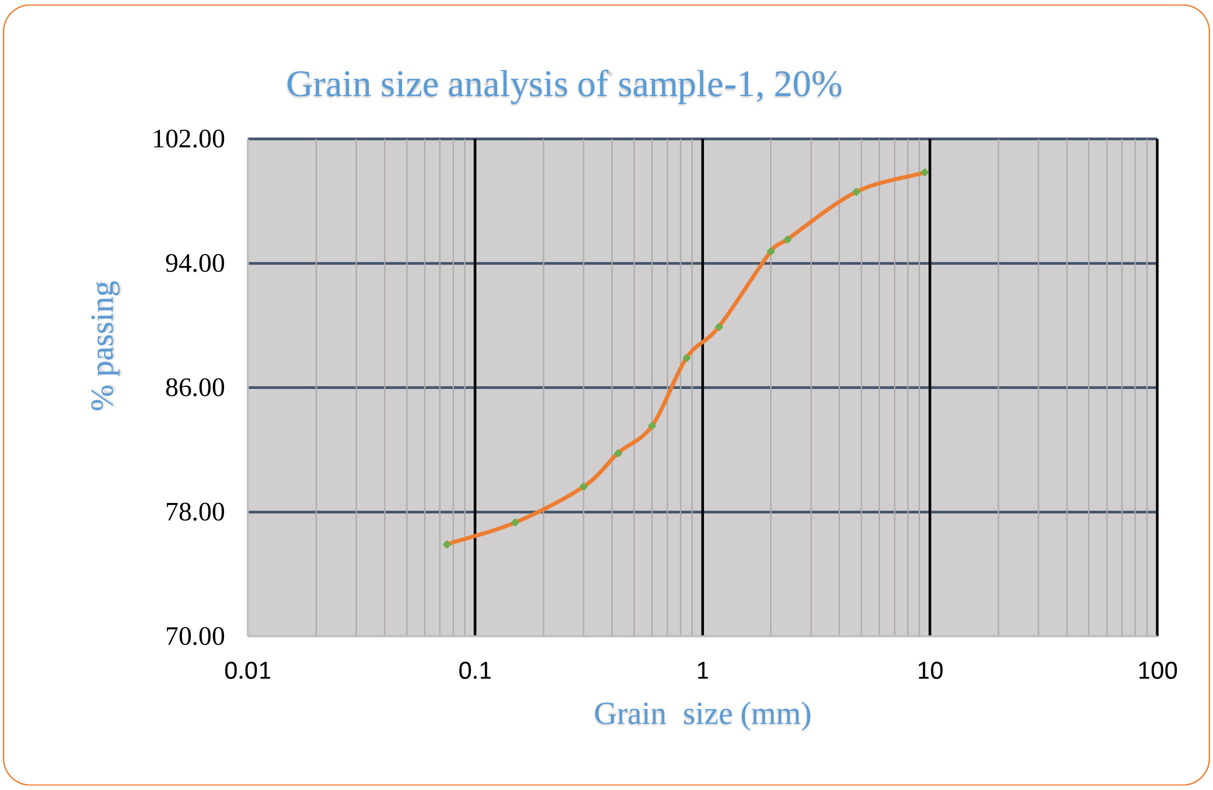


Sample-1: 15%P+15%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	1.60	0.16	0.16	99.84
4.75	18.90	1.86	2.02	97.98
2.36	44.70	4.40	6.42	93.58
2	7.50	0.74	7.16	92.84
1.18	42.10	4.15	11.31	88.69
0.85	13.20	1.30	12.61	87.39
0.6	18.60	1.83	14.44	85.56
0.425	4.70	0.46	14.90	85.10
0.3	4.50	0.44	15.35	84.65
0.15	7.10	0.70	16.05	83.95
0.075	12.40	1.22	17.27	82.73
pan	824.70	81.24	98.51	1.49
Sum	1015.1			

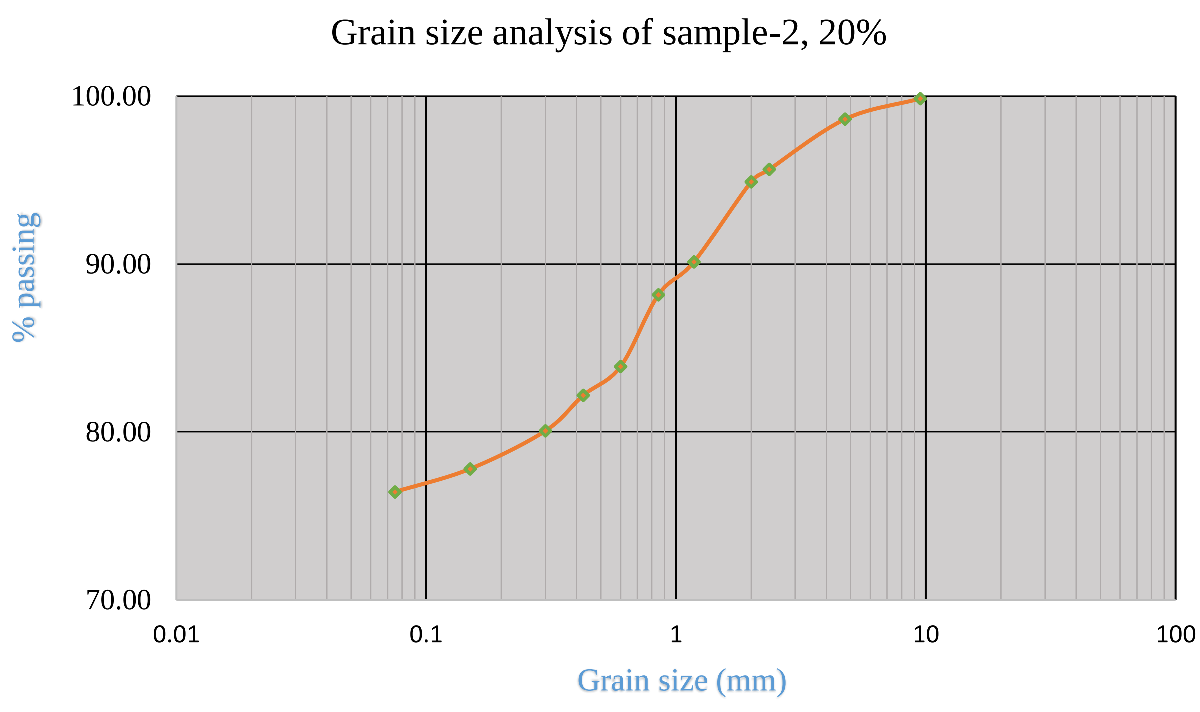
Sample-2: 15%P+15%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	4.10	0.40	0.40	99.60
4.75	12.70	1.25	1.66	98.34
2.36	29.70	2.93	4.58	95.42
2	10.40	1.02	5.61	94.39
1.18	38.10	3.75	9.36	90.64
0.85	16.00	1.58	10.93	89.07
0.6	27.80	2.74	13.67	86.33
0.425	12.20	1.20	14.88	85.12
0.3	12.00	1.18	16.06	83.94
0.15	21.60	2.13	18.19	81.81
0.075	18.10	1.78	19.97	80.03
pan	797.30	78.54	98.51	1.49
Sum	1015.1			



Sample-1: 20%P+20%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	1.50	0.15	0.15	99.85
4.75	12.50	1.25	1.40	98.60
2.36	30.50	3.06	4.47	95.53
2	7.50	0.75	5.22	94.78
1.18	48.50	4.87	10.08	89.92
0.85	20.00	2.01	12.09	87.91
0.6	43.50	4.37	16.46	83.54
0.425	17.50	1.76	18.21	81.79
0.3	21.50	2.16	20.37	79.63
0.15	23.00	2.31	22.68	77.32
0.075	14.00	1.40	24.08	75.92
pan	738.00	74.05	98.14	1.86
Sum	996.6			



Sample-1: 20%P+20%CSD Wet sieve analysis				
Sieve size (mm)	Mass of retaining on each sieve(g)	Percentage of the retained soil	Cumulative % of retaining soil	Percentage of passing particle
9.5	1.50	0.15	0.15	99.85
4.75	12.50	1.23	1.38	98.62
2.36	30.50	3.00	4.37	95.63
2	7.50	0.74	5.11	94.89
1.18	48.50	4.76	9.87	90.13
0.85	20.00	1.96	11.84	88.16
0.6	43.50	4.27	16.11	83.89
0.425	17.50	1.72	17.83	82.17
0.3	21.50	2.11	19.94	80.06
0.15	23.00	2.26	22.20	77.80
0.075	14.00	1.38	23.57	76.43
pan	760.00	74.65	98.22	1.78
Sum	1018.2			



APPENDIX D: Unconfined compressive strength

Test Type:	Unconfined Compression Test (ASTM D-2166)	
Type of Sample:	Remolded Soil Sample	
Sample:	Sample -1: natural soil	
Pit Code:	Natural	
Sample Height (mm)	Peak UCS, (kPa)	Cohesion, (kPa)
77	690	345

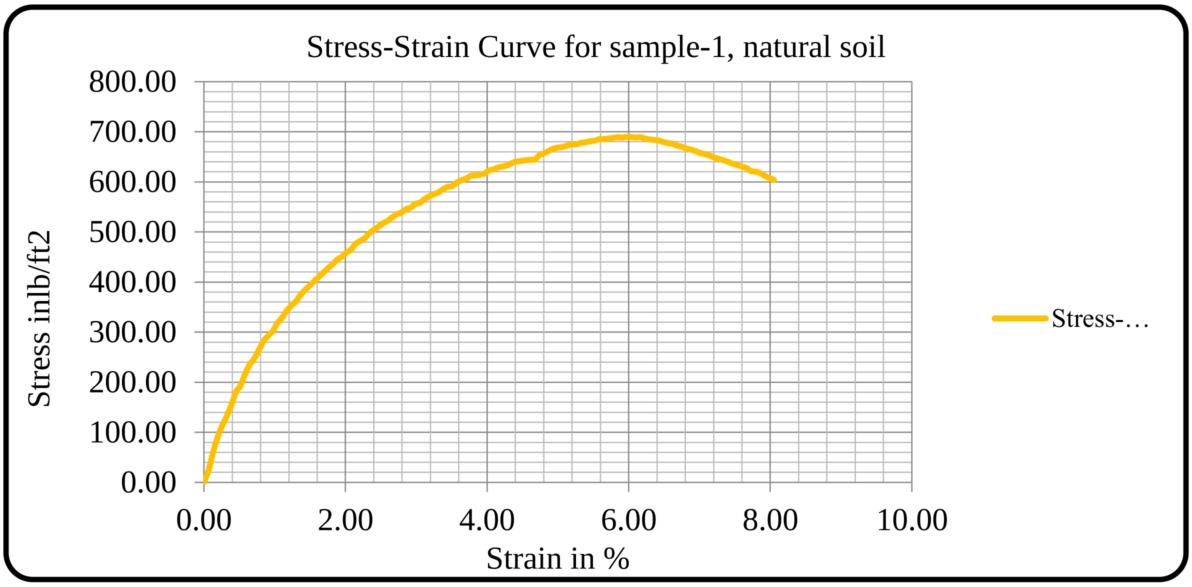
Trial 1 for sample only

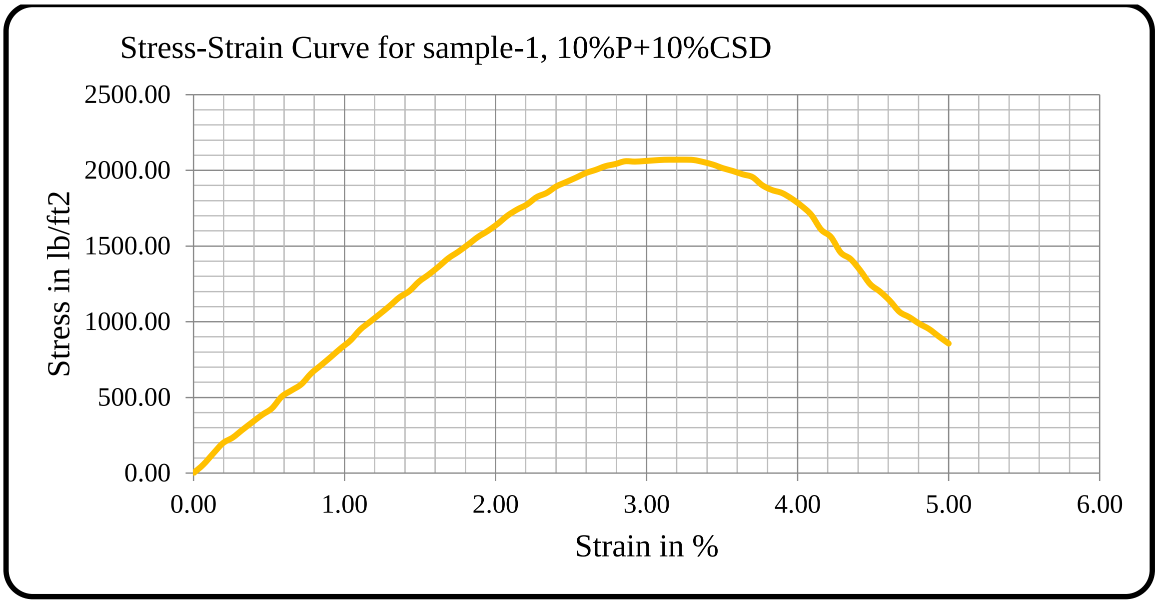
Sample Deformation ΔL (mm)	Load In (N) Trial-1	Sample Height (mm)	Sample Actual Area (cm^2)	Strain	Strain in %	Corrected Area (cm^2)	Stress (kPa)
0.00	0.00	77.00	11.34	0.00	0.00	11.34	0.00
0.05	27.00	77.00	11.34	0.00	0.06	11.35	23.79
0.10	68.00	77.00	11.34	0.00	0.13	11.36	59.88
0.15	103.00	77.00	11.34	0.00	0.19	11.36	90.64
0.20	129.00	77.00	11.34	0.00	0.26	11.37	113.45
0.25	151.00	77.00	11.34	0.00	0.32	11.38	132.71
0.30	177.00	77.00	11.34	0.00	0.39	11.39	155.46
0.35	206.00	77.00	11.34	0.00	0.45	11.39	180.81
0.40	221.00	77.00	11.34	0.01	0.52	11.40	193.85
0.45	248.00	77.00	11.34	0.01	0.58	11.41	217.39
0.50	269.00	77.00	11.34	0.01	0.65	11.42	235.65
0.55	283.00	77.00	11.34	0.01	0.71	11.42	247.75
0.60	303.00	77.00	11.34	0.01	0.78	11.43	265.09
0.65	323.00	77.00	11.34	0.01	0.84	11.44	282.40
0.70	335.00	77.00	11.34	0.01	0.91	11.45	292.70
0.75	346.00	77.00	11.34	0.01	0.97	11.45	302.11
0.80	365.00	77.00	11.34	0.01	1.04	11.46	318.49
0.85	377.00	77.00	11.34	0.01	1.10	11.47	328.75
0.90	393.00	77.00	11.34	0.01	1.17	11.48	342.48
0.95	405.00	77.00	11.34	0.01	1.23	11.48	352.70
1.00	415.00	77.00	11.34	0.01	1.30	11.49	361.17
1.05	430.00	77.00	11.34	0.01	1.36	11.50	373.98
1.10	442.00	77.00	11.34	0.01	1.43	11.51	384.16
1.15	452.00	77.00	11.34	0.01	1.49	11.51	392.60
1.20	463.00	77.00	11.34	0.02	1.56	11.52	401.89
1.25	473.00	77.00	11.34	0.02	1.62	11.53	410.29
1.30	483.00	77.00	11.34	0.02	1.69	11.54	418.69
1.35	494.00	77.00	11.34	0.02	1.75	11.54	427.95

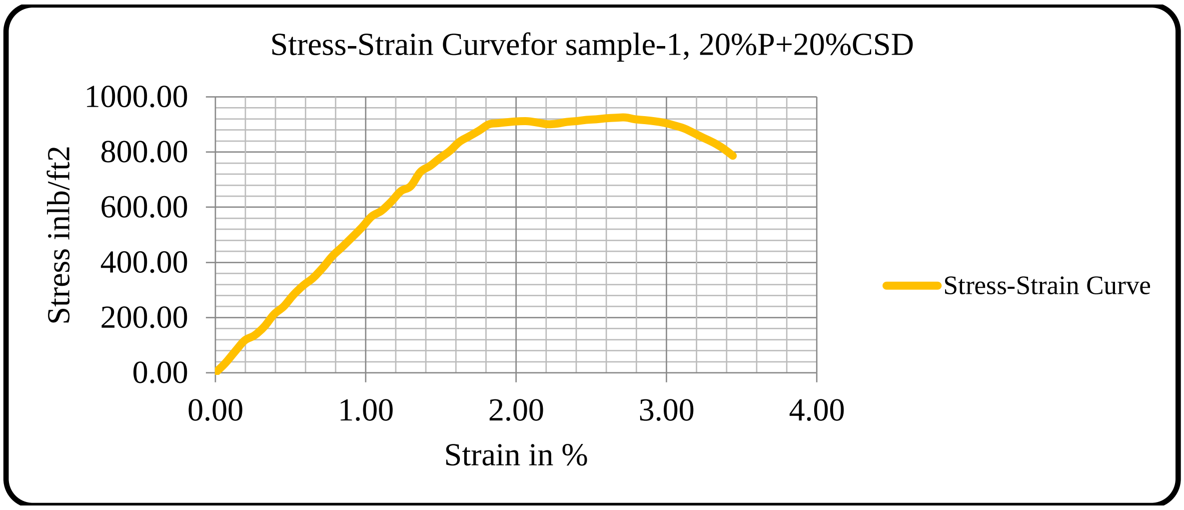
1.40	503.00	77.00	11.34	0.02	1.82	11.55	435.45
1.45	514.00	77.00	11.34	0.02	1.88	11.56	444.68
1.50	521.00	77.00	11.34	0.02	1.95	11.57	450.44
1.55	531.00	77.00	11.34	0.02	2.01	11.57	458.78
1.60	538.00	77.00	11.34	0.02	2.08	11.58	464.52
1.65	552.00	77.00	11.34	0.02	2.14	11.59	476.29
1.70	559.00	77.00	11.34	0.02	2.21	11.60	482.01
1.75	566.00	77.00	11.34	0.02	2.27	11.60	487.73
1.80	578.00	77.00	11.34	0.02	2.34	11.61	497.73
1.85	586.00	77.00	11.34	0.02	2.40	11.62	504.29
1.90	594.00	77.00	11.34	0.02	2.47	11.63	510.83
1.95	602.00	77.00	11.34	0.03	2.53	11.64	517.37
2.00	608.00	77.00	11.34	0.03	2.60	11.64	522.18
2.05	617.00	77.00	11.34	0.03	2.66	11.65	529.55
2.10	624.00	77.00	11.34	0.03	2.73	11.66	535.20
2.15	629.00	77.00	11.34	0.03	2.79	11.67	539.13
2.20	637.00	77.00	11.34	0.03	2.86	11.67	545.62
2.25	641.00	77.00	11.34	0.03	2.92	11.68	548.68
2.30	650.00	77.00	11.34	0.03	2.99	11.69	556.01
2.35	653.00	77.00	11.34	0.03	3.05	11.70	558.21
2.40	662.00	77.00	11.34	0.03	3.12	11.71	565.52
2.45	669.00	77.00	11.34	0.03	3.18	11.71	571.12
2.50	674.00	77.00	11.34	0.03	3.25	11.72	575.00
2.55	679.00	77.00	11.34	0.03	3.31	11.73	578.88
2.60	687.00	77.00	11.34	0.03	3.38	11.74	585.30
2.65	693.00	77.00	11.34	0.03	3.44	11.75	590.02
2.70	695.00	77.00	11.34	0.04	3.51	11.75	591.32
2.75	704.00	77.00	11.34	0.04	3.57	11.76	598.58
2.80	710.00	77.00	11.34	0.04	3.64	11.77	603.27
2.85	714.00	77.00	11.34	0.04	3.70	11.78	606.26
2.90	721.00	77.00	11.34	0.04	3.77	11.79	611.79
2.95	723.00	77.00	11.34	0.04	3.83	11.79	613.08
3.00	725.00	77.00	11.34	0.04	3.90	11.80	614.36
3.05	728.00	77.00	11.34	0.04	3.96	11.81	616.48
3.10	736.00	77.00	11.34	0.04	4.03	11.82	622.84
3.15	739.00	77.00	11.34	0.04	4.09	11.82	624.95
3.20	744.00	77.00	11.34	0.04	4.16	11.83	628.76
3.25	747.00	77.00	11.34	0.04	4.22	11.84	630.86
3.30	750.00	77.00	11.34	0.04	4.29	11.85	632.97
3.35	756.00	77.00	11.34	0.04	4.35	11.86	637.60
3.40	760.00	77.00	11.34	0.04	4.42	11.87	640.54

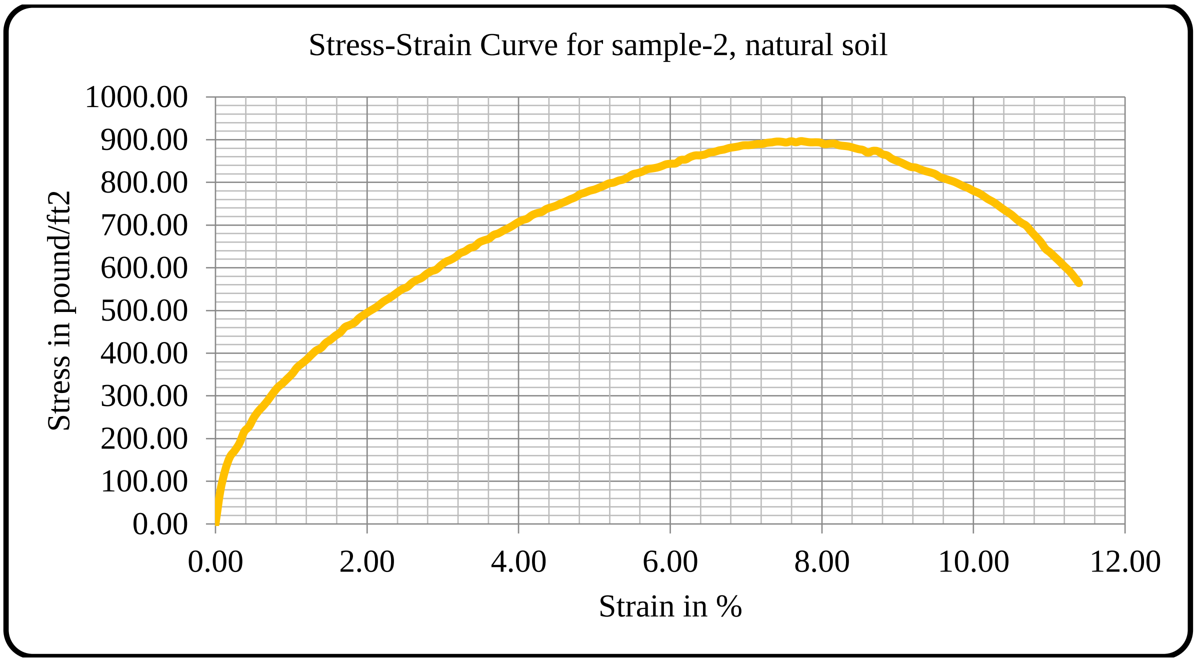
3.45	762.00	77.00	11.34	0.04	4.48	11.87	641.79
3.50	764.00	77.00	11.34	0.05	4.55	11.88	643.03
3.55	766.00	77.00	11.34	0.05	4.61	11.89	644.28
3.60	767.00	77.00	11.34	0.05	4.68	11.90	644.68
3.65	778.00	77.00	11.34	0.05	4.74	11.91	653.48
3.70	783.00	77.00	11.34	0.05	4.81	11.91	657.23
3.75	789.00	77.00	11.34	0.05	4.87	11.92	661.82
3.80	795.00	77.00	11.34	0.05	4.94	11.93	666.39
3.85	798.00	77.00	11.34	0.05	5.00	11.94	668.45
3.90	800.00	77.00	11.34	0.05	5.06	11.95	669.67
3.95	804.00	77.00	11.34	0.05	5.13	11.95	672.56
4.00	807.00	77.00	11.34	0.05	5.19	11.96	674.60
4.05	808.00	77.00	11.34	0.05	5.26	11.97	674.98
4.10	812.00	77.00	11.34	0.05	5.32	11.98	677.85
4.15	814.00	77.00	11.34	0.05	5.39	11.99	679.06
4.20	817.00	77.00	11.34	0.05	5.45	12.00	681.09
4.25	819.00	77.00	11.34	0.06	5.52	12.00	682.29
4.30	823.00	77.00	11.34	0.06	5.58	12.01	685.15
4.35	824.00	77.00	11.34	0.06	5.65	12.02	685.51
4.40	826.00	77.00	11.34	0.06	5.71	12.03	686.70
4.45	828.00	77.00	11.34	0.06	5.78	12.04	687.89
4.50	830.00	77.00	11.34	0.06	5.84	12.05	689.08
4.55	830.00	77.00	11.34	0.06	5.91	12.05	688.60
4.60	832.00	77.00	11.34	0.06	5.97	12.06	689.79
4.65	832.00	77.00	11.34	0.06	6.04	12.07	689.31
4.70	832.00	77.00	11.34	0.06	6.10	12.08	688.83
4.75	833.00	77.00	11.34	0.06	6.17	12.09	689.18
4.80	830.00	77.00	11.34	0.06	6.23	12.10	686.23
4.85	829.00	77.00	11.34	0.06	6.30	12.10	684.93
4.90	828.00	77.00	11.34	0.06	6.36	12.11	683.62
4.95	827.00	77.00	11.34	0.06	6.43	12.12	682.33
5.00	824.00	77.00	11.34	0.06	6.49	12.13	679.38
5.05	822.00	77.00	11.34	0.07	6.56	12.14	677.26
5.10	821.00	77.00	11.34	0.07	6.62	12.15	675.97
5.15	817.00	77.00	11.34	0.07	6.69	12.15	672.20
5.20	815.00	77.00	11.34	0.07	6.75	12.16	670.09
5.25	811.00	77.00	11.34	0.07	6.82	12.17	666.34
5.30	809.00	77.00	11.34	0.07	6.88	12.18	664.23
5.35	806.00	77.00	11.34	0.07	6.95	12.19	661.31
5.40	802.00	77.00	11.34	0.07	7.01	12.20	657.57
5.45	800.00	77.00	11.34	0.07	7.08	12.21	655.47

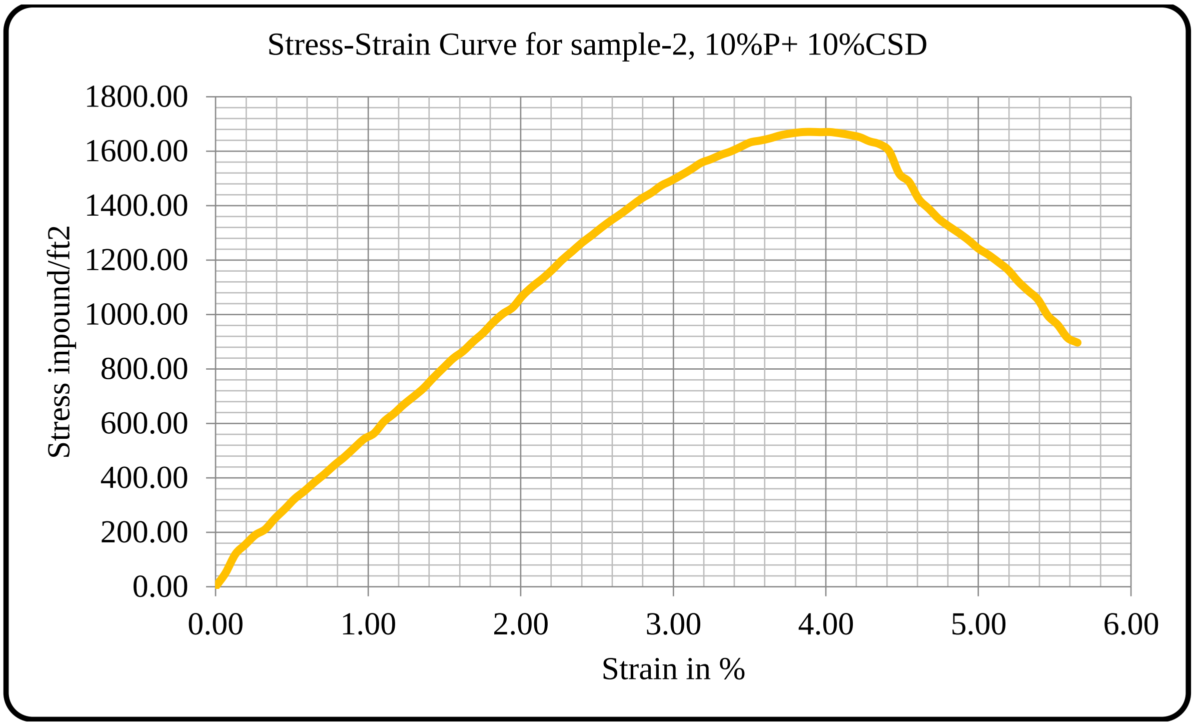
5.50	797.00	77.00	11.34	0.07	7.14	12.21	652.55
5.55	793.00	77.00	11.34	0.07	7.21	12.22	648.83
5.60	790.00	77.00	11.34	0.07	7.27	12.23	645.92
5.65	787.00	77.00	11.34	0.07	7.34	12.24	643.01
5.70	784.00	77.00	11.34	0.07	7.40	12.25	640.11
5.75	780.00	77.00	11.34	0.07	7.47	12.26	636.40
5.80	777.00	77.00	11.34	0.08	7.53	12.27	633.51
5.85	774.00	77.00	11.34	0.08	7.60	12.27	630.62
5.90	771.00	77.00	11.34	0.08	7.66	12.28	627.73
5.95	764.00	77.00	11.34	0.08	7.73	12.29	621.60
6.00	763.00	77.00	11.34	0.08	7.79	12.30	620.35
6.05	760.00	77.00	11.34	0.08	7.86	12.31	617.47
6.10	754.00	77.00	11.34	0.08	7.92	12.32	612.17
6.15	749.00	77.00	11.34	0.08	7.99	12.33	607.68
6.20	745.00	77.00	11.34	0.08	8.05	12.33	604.01

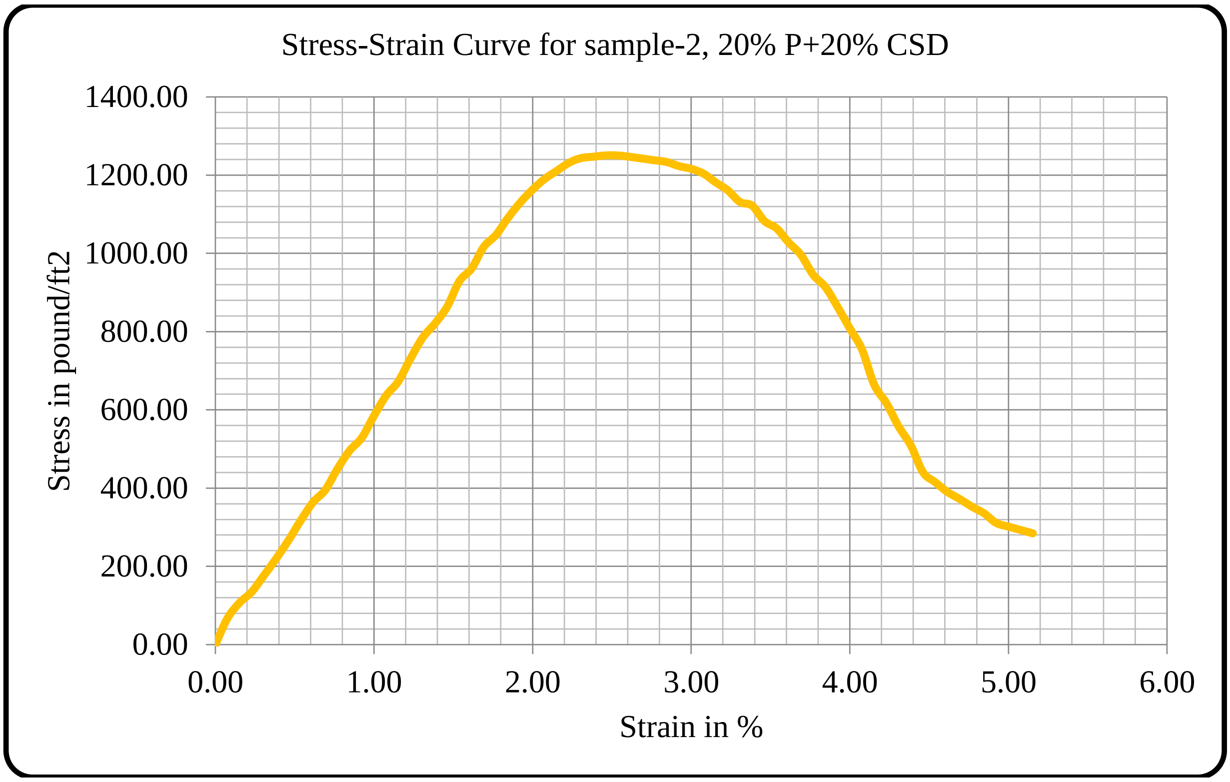



Stress-Strain Curve for sample-1, 10%P+10%CSD
2500.00
2000.00
1500.00
1000.00
500.00
0.00
Stress in lb/ft2
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Strain in %


Stress-Strain Curvefor sample-1, 20%P+20%CSD
1000.00
800.00
600.00
400.00
200.00
0.00
Stress inlb/ft2
0.00
1.00
2.00
3.00
4.00
Strain in %
Stress-Strain Curve

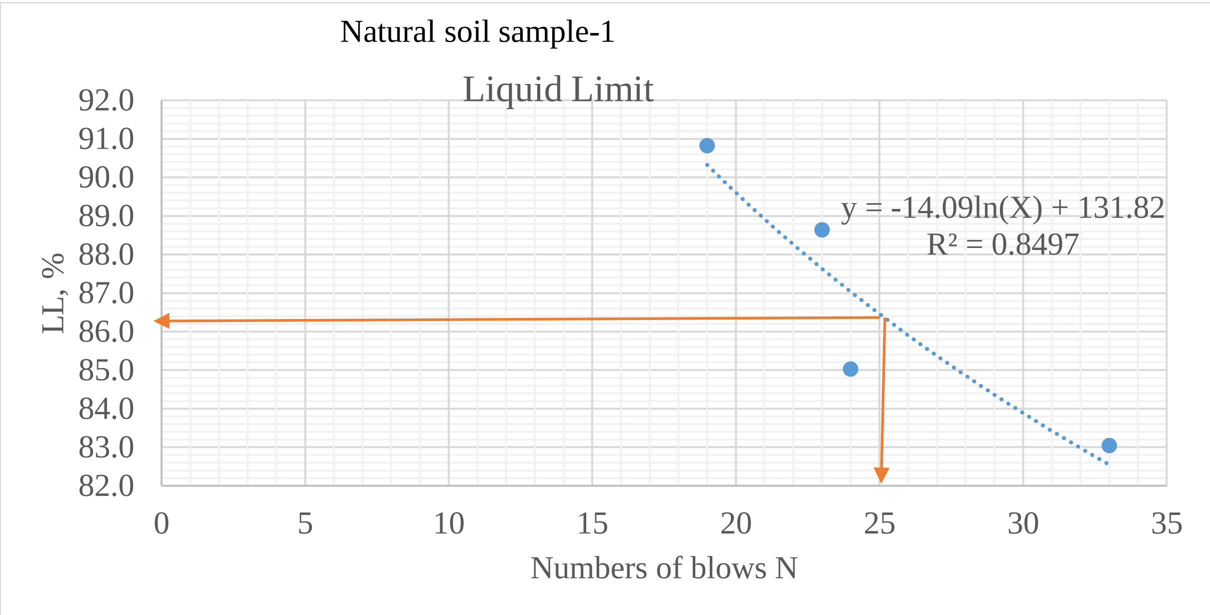

Stress-Strain Curve for sample-2, natural soil
1000.00
900.00
800.00
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
Stress in pound/ft2
0.00
2.00
4.00
6.00
8.00
10.00
12.00
Strain in %


Stress-Strain Curve for sample-2, 10%P+ 10%CSD
Stress inpound/ft2
1800.00
1600.00
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Strain in %

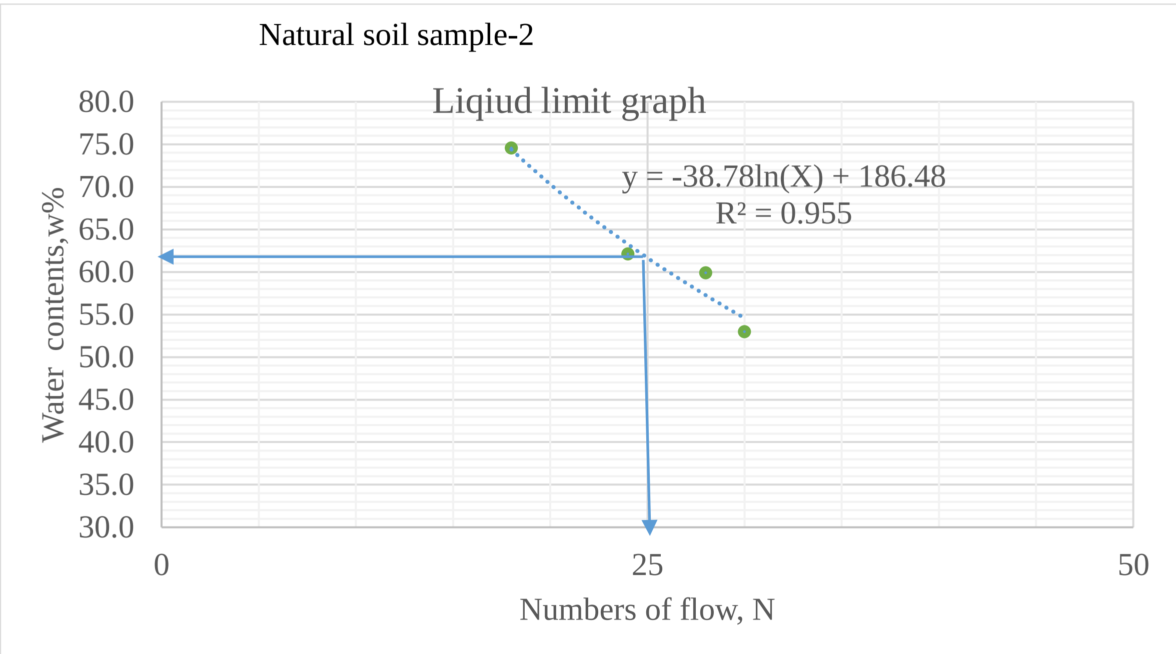

Stress-Strain Curve for sample-2, 20% P+20% CSD
Stress in pound/ft2
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Strain in %

APPENDIX E: Sample Atterberg's Limit Test Analysis Data

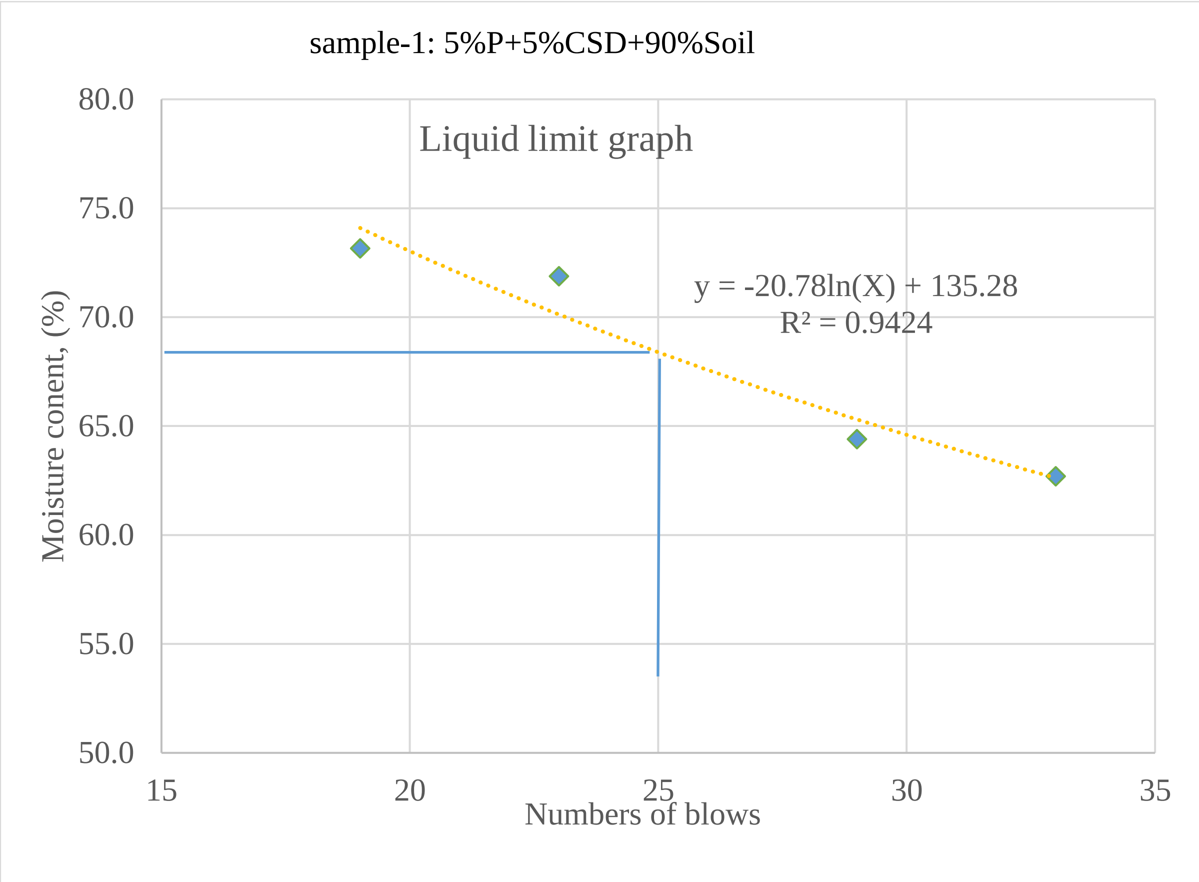
Sample location: Chida -Jimma road segment	Natural soil of sample -1					
Determination	Liquid limit				Plastic limit	
Number of blows	33	24	23	19		
Sample trial number	1	2	3	4	1	2
Container number	L1	C4	L3	BB	A	X9
Mass of container + wet soil (g) = (w1)	14.46	15.7	15.87	18.04	15.15	20.1
Mass of container + dry soil (g) = (w2)	19.17	11.27	11.5	12.4	12.94	18.25
Mass of container (g) = (w3)	6.33	6.06	6.57	6.19	6.35	12.91
Mass of moisture (g) = (w1 - w2) = a	3.82	4.43	4.37	5.64	2.2	1.85
Mass of dry soil (g) = (w2 - w3) = b	4.6	5.21	4.93	6.21	6.59	5.34
Moisture content (%) = (a / b) x 100	83	85	88.6	90.8	33.5	34.6
Liquid limit (LL) (%):	87	Av. Plastic. Limit.			34	
Plastic limit (PL) (%):	34					
Plasticity index (PI) (%): LL - PL	53					



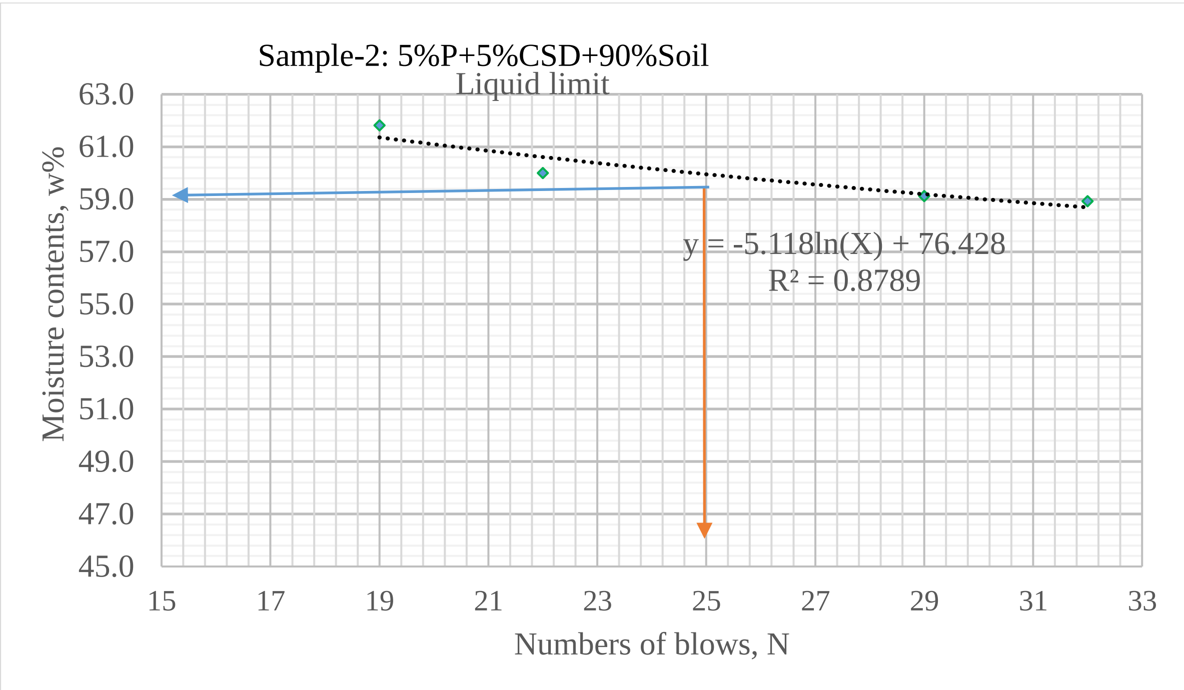
Sample Location: Chida-Jimma road segment	Sample-2 natural soil					
Determination	Liquid Limit			Plastic Limit		
Number of blows	30	28	24	18		
Sample Trial Number	1	2	3	4	1	2
Container Number	2	35	4	P	B1	Z
Mass of Container + Wet Soil (g) = (W1)	28.5	16.1	16	19.36	14.1	12.14
Mass of Container + Dry Soil (g) = (W2)	25.19	12.41	12.33	14.03	12.63	11.1
Mass of Container (g) = (W3)	18.94	6.25	6.42	6.88	5.47	6.41
Mass of Moisture (g) = (W1 - W2) = A	3.31	3.69	3.67	5.33	1.5	1.04
Mass of Dry Soil (g) = (W2 - W3) = B	6.25	6.16	5.91	7.15	7.16	4.69
Moisture Content (%) = (A / B) x 100	59.2	61.5	62.1	64.5	20.5	22.2
Liquid Limit (LL) (%):	62	AV. Plastic. Lim.			21	
Plastic Limit (PL) (%):	21					
Plasticity Index (PI) (%): LL - PL	41					



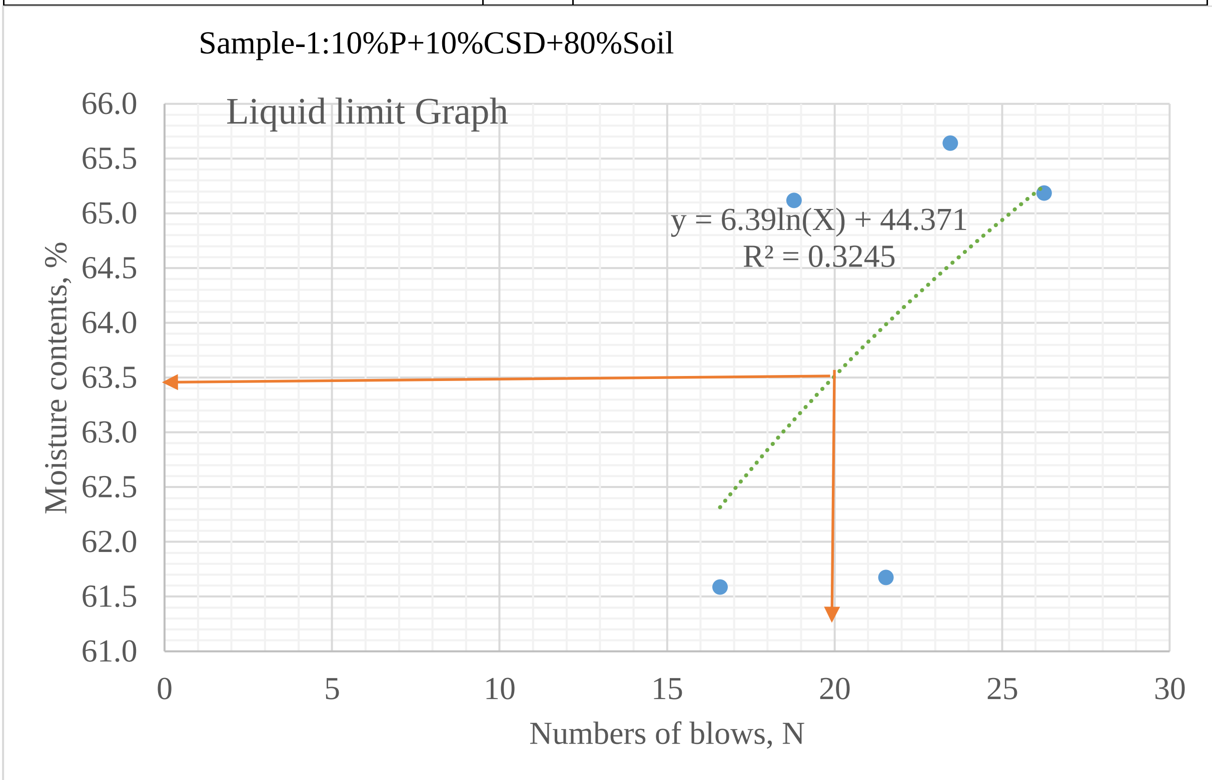
Sample Location: Chida	Sample-1: 5%P + 5%CSD +90%soil					
Determination	Liquid Limit				Plastic Limit	
Number of blows	33	29	23	19		
Sample Trial Number	1	2	3	4	1	2
Container Number	J	D	S	A	C4	B1
Mass of Container + Wet Soil (g) = (W1)	21.3	17.34	19.79	19.58	14.74	12.35
Mass of Container + Dry Soil (g) = (W2)	15.45	13.25	14.04	14.02	12.88	10.81
Mass of Container (g) = (W3)	6.12	6.9	6.04	6.42	6.09	5.47
Mass of Moisture (g) = (W1 - W2) = A	5.85	4.09	5.75	5.56	1.9	1.54
Mass of Dry Soil (g) = (W2 - W3) = B	9.33	6.35	71.9	7.6	6.79	5.34
Moisture Content (%) = (A / B) x 100	62.7	64.4	71.9	73.2	27.4	28.8
Liquid Limit (LL) (%):	69	AV. Plastic. Limit.				29
Plastic Limit (PL) (%):	29					
Plasticity Index (PI) (%): LL - PL	40					



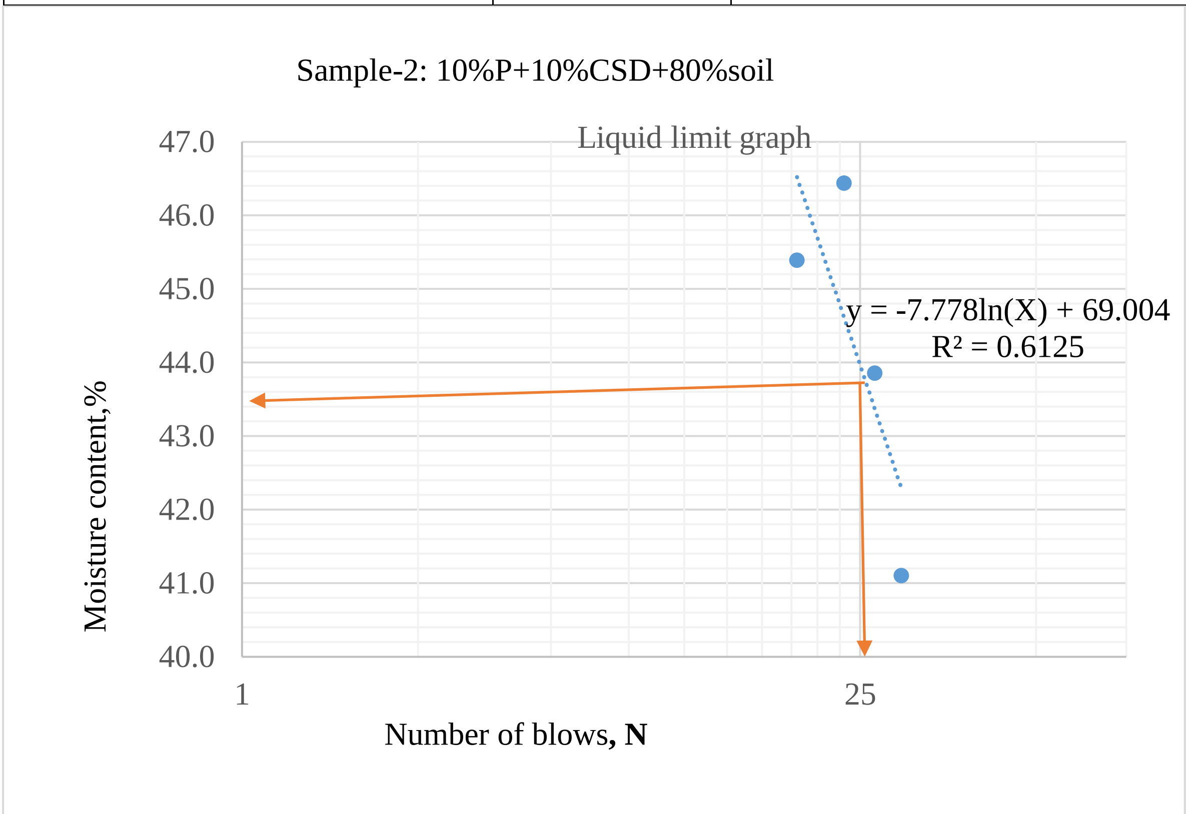
Sample Location: Chida – Jimma road segment	Sample-2: 5%P + 5%CSD +90% soil					
Determination	Liquid Limit				Plastic Limit	
Number of blows	32	29	22	19		
Sample Trial Number	1	2	3	4	1	2
Container Number	B9	A17	A7	3,.3	A7	3
Mass of Container + Wet Soil (g) = (W1)	19.6	21	28.9	29.2	28.4	17.3
Mass of Container + Dry Soil (g) = (W2)	14.5	15.6	24.45	24.62	26.25	15.06
Mass of Container (g) = (W3)	6.01	6.28	17	17.2	17.5	6.28
Mass of Moisture (g) = (W1 - W2) = A	5.02	5.51	4.47	4.58	2.2	2.28
Mass of Dry Soil (g) = (W2 - W3) = B	8.52	9.32	7.45	7.41	8.75	8.78
Moisture Content (%) = (A / B) x 100	59	59	60	61.8	25.2	26.1
Liquid Limit (LL) (%):	60	AV. Plastic. Lim.			26	
Plastic Limit (PL) (%):	26					
Plasticity Index (PI) (%): LL - PL	34					



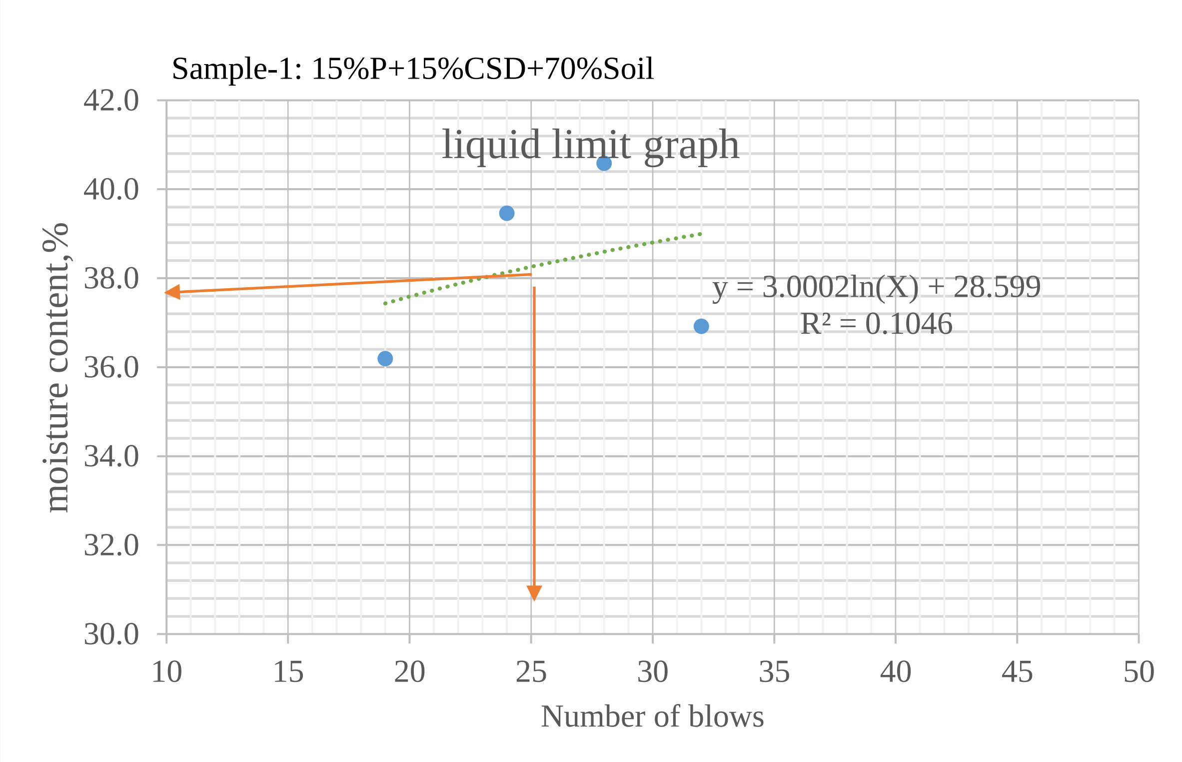
Sample Location: Chida	Sample-1: 10%P + 10%CSD +80% soil							
Determination	Liquid Limit					Plastic Limit		
Number of blows	26.25	23.45	21.53	18.79	16.58			
Sample Trial Number	1	2	3	4	5	1	2	3
Container Number	A7	C9	3-,3	C4	A	A7	BB	B9
Mass of Container + Wet Soil (g) = (W1)	36.33	32.6	39.22	16.54	23.54	36.2	31.68	35.08
Mass of Container + Dry Soil (g) = (W2)	28.29	26.04	32.91	12.09	16.4	32.71	28.57	30.75
Mass of Container (g) = (W3)	17.49	16.96	24.3	6.07	6.43	21.95	18.7	18.7
Mass of Moisture (g) = (W1 - W2) = A	8.04	6.56	6.31	4.45	7.14	3.49	3.11	4.33
Mass of Dry Soil (g) = (W2 - W3) = B	10.8	9.08	8.61	6.02	9.97	10.76	9.87	12.05
Moisture Content (%) = (A / B) x 100	74	72	73.3	73.9	71.6	32.4	31.5	35.9
Liquid Limit (LL) (%):	65	AV. Plastic. Limit.				33		
Plastic Limit (PL) (%):	33							
Plasticity Index (PI) (%): LL - PL	32							



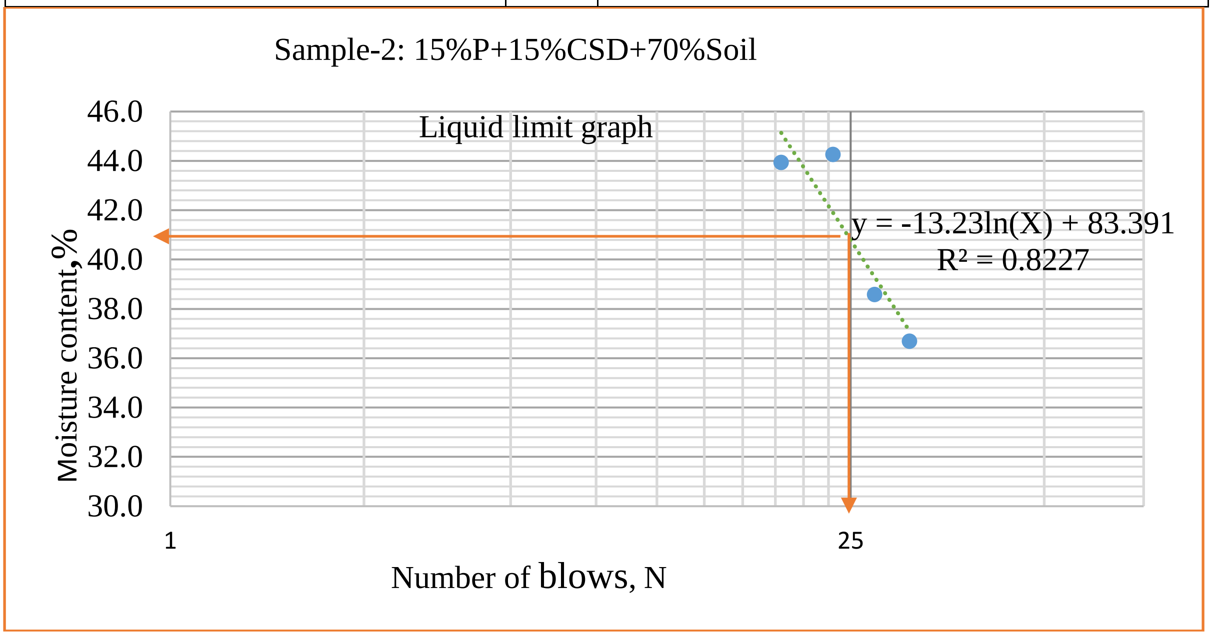
Sample Location: Chida	Sample-2: 10% P + 10%CSD +80% soil					
Determination	Liquid Limit			Plastic Limit		
Number of blows	31	27	23	18	Plastic Limit	
Sample Trial Number	1	2	3	4	18	
Container Number	C9	B1	3%	C4	4	2
Mass of Container + Wet Soil (g) = (W1)	18.92	17.5	22.95	21.52	S	B1
Mass of Container + Dry Soil (g) = (W2)	15.1	13.78	17.86	16.69	16.59	16.75
Mass of Container (g) = (W3)	6.05	6.21	6.9	6.05	14.78	14.73
Mass of Moisture (g) = (W1 - W2) = A	3.82	3.72	5.09	4.83	6.01	5.48
Mass of Dry Soil (g) = (W2 - W3) = B	9.05	7.57	10.96	10.64	1.81	2.02
Moisture Content (%) = (A / B) x 100	42.2	49.141347	46.441606	45.394737	8.77	9.25
Liquid Limit (LL) (%):	45.5				20.6	21.8
Plastic Limit (PL) (%):	21	AV. Plastic. Limit.			21	
Plasticity Index (PI) (%): LL - PL	24					



Sample Location: Chida to Jimma road segment	Sample -1 Additive Content: 15%P+ 15% CSD+70% soil					
Determination	Liquid Limit				Plastic Limit	
Number of blows	32	28	24	19		
Sample Trial Number	1	2	3		1	2
Container Number	T6	C1	2	3,	A	3
Mass of Container + Wet Soil (g) = (W1)	16.41	38.14	29.28	25.51	13.4	12.7
Mass of Container + Dry Soil (g) = (W2)	13.7	33.16	26.36	23.23	12.11	11.67
Mass of Container (g) = (W3)	6.36	20.89	18.96	16.93	5.67	6.24
Mass of Moisture (g) = (W1 - W2) = A	2.71	4.98	2.92	2.28	1.29	1.03
Mass of Dry Soil (g) = (W2 - W3) = B	7.34	12.27	7.4	6.3	6.44	5.43
Moisture Content (%) = (A / B) x 100	37	40.6	39.45	36	20	19
Liquid Limit (LL) (%):	38	AV. Plastic. Limit.			19	
Plastic Limit (PL) (%):	19					
Plasticity Index (PI) (%): LL - PL	19					

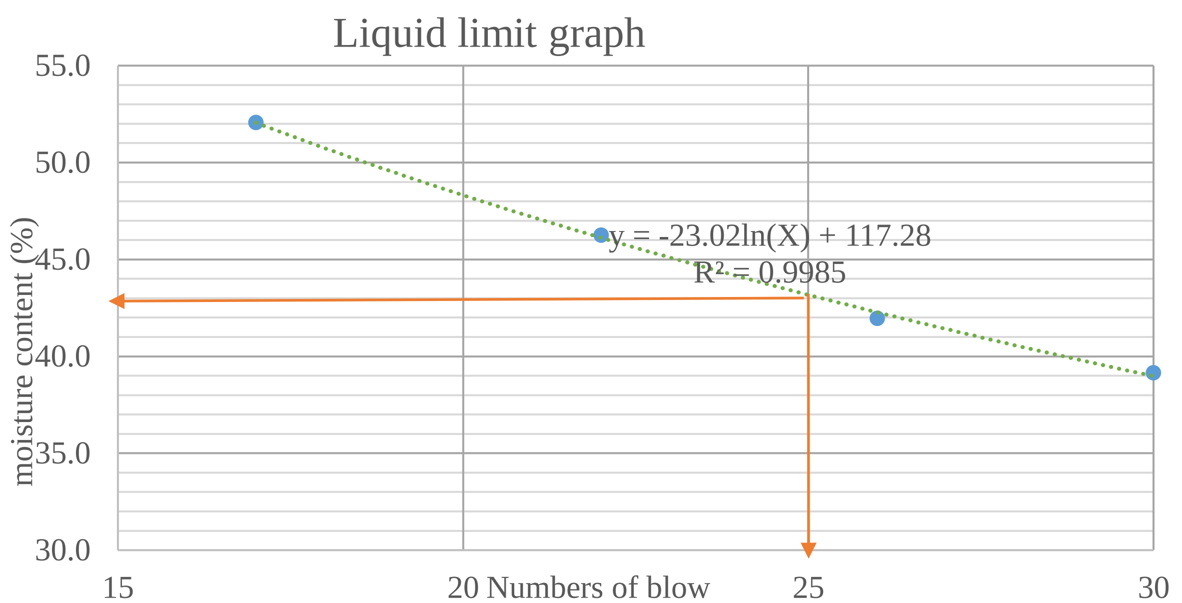


Sample Location: Chida to Jimma road segment	Sample-2 Additive Content: 15% P + 15% CSD 70% soil					
Determination	Liquid Limit				Plastic Limit	
Number of blows	33	28	23	18		
Sample Trial Number	1	2	3	4	1	2
Container Number	T4	BB	D	C4	B1	S
Mass of Container + Wet Soil (g) = (W1)	18.72	17	22.71	21.22	13.504	14.03
Mass of Container + Dry Soil (g) = (W2)	15.1	13.78	17.86	16.69	12.44	12.49
Mass of Container (g) = (W3)	6.05	6.21	6.9	6.05	5.49	6.02
Mass of Moisture (g) = (W1 - W2) = A	3.6	3.22	4.85	4.53	1.06	1.54
Mass of Dry Soil (g) = (W2 - W3) = B	9.05	7.57	10.96	10.64	6.95	6.47
Moisture Content (%) = (A / B) x 100	40	42.5363276	44.2518248	42.575188	15.3	23.8
Liquid Limit (LL) (%):	42.5	AV. Plastic. Limit.			20	
Plastic Limit (PL) (%):	20					
Plasticity Index (PI) (%): LL - PL	19					

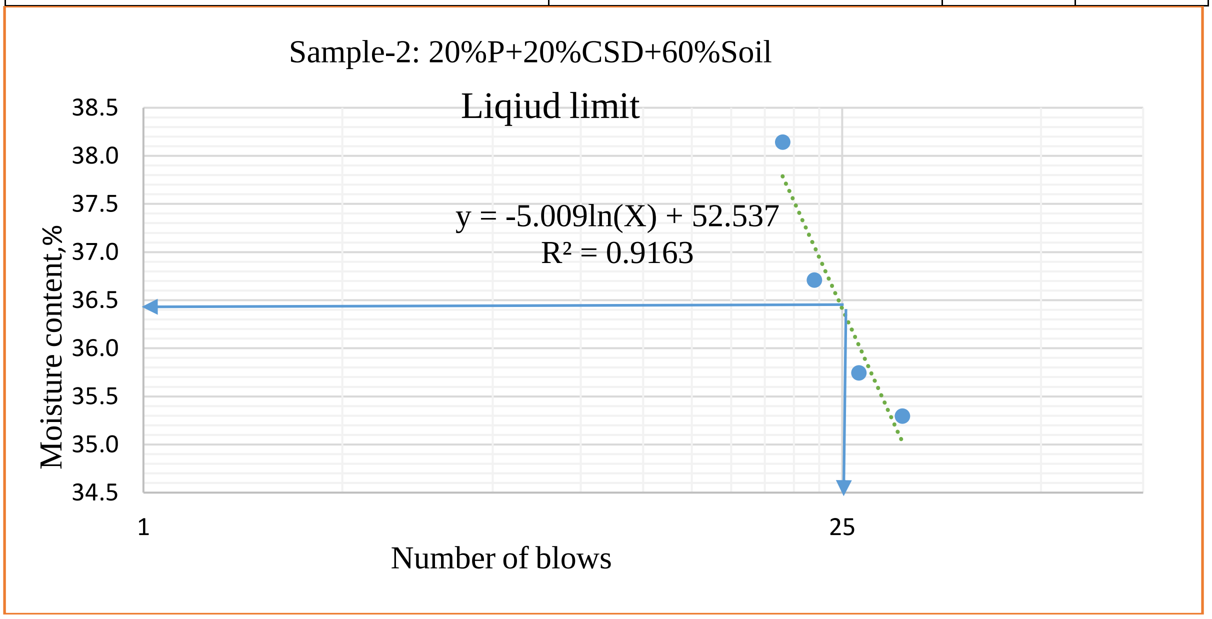


Sample Location: Chida to Jimma road segment	Sample -1 Additive Content: 20% P + 20%CSD +60% soil					
Determination	Liquid Limit				Plastic Limit	
Number of blows	30	26	22	17		
Sample Trial Number	1	2	3	4	1	2
Container Number	PLL	S	A17	D	BB	A
Mass of Container + Wet Soil (g) = (W1)	21.23	23.89	16	25.34	17.36	16.45
Mass of Container + Dry Soil (g) = (W2)	16.44	18.05	24.52	19.03	14.49	13.92
Mass of Container (g) = (W3)	6.07	6.04	19.22	6.91	6.19	6.41
Mass of Moisture (g) = (W1 - W2) = A	4.79	5.84	7.76	6.31	2.87	2.53
Mass of Dry Soil (g) = (W2 - W3) = B	10.37	12.01	5.3	12.12	8.3	7.51
Moisture Content (%) = (A / B) x 100	46.2	48.6	46.2	52.1	35.8	35.6
Liquid Limit (LL) (%):	46.3	AV. Plastic. Limit.			36	
Plastic Limit (PL) (%):	36					
Plasticity Index (PI) (%): LL - PL	10					

Sample-1: 20% P+20%CSD+60%Soil

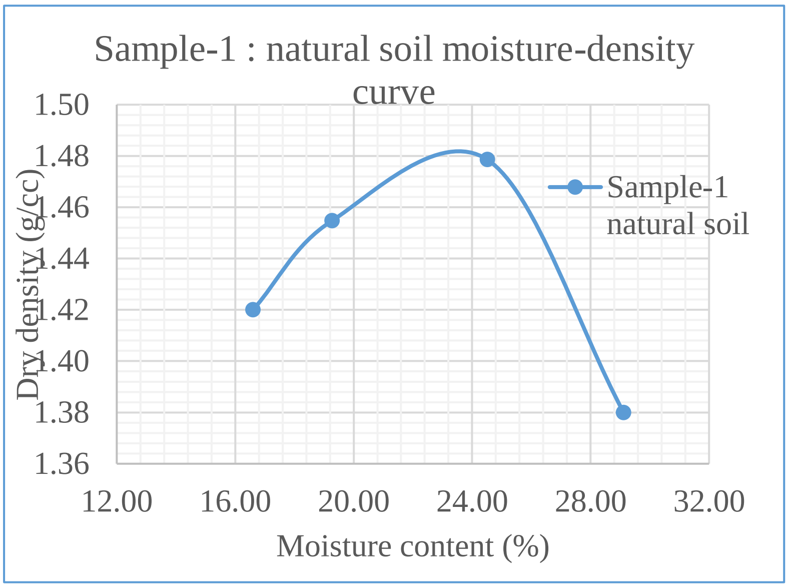


Sample Location: Chida to Jimma road segment	Sample -2 Additive Content: 20%P + 20%CSD% +60%soil					
Determination	Liquid Limit			Plastic Limit		
Number of blows	33	27	22	19		
Sample Trial Number	1	2	3	4	1	2
Container Number	A	T6	T4	3.-	B1	C4
Mass of Container + Wet Soil (g) = (W1)	20.19	21.56	16.8	17	16.72	16.92
Mass of Container + Dry Soil (g) = (W2)	16.41	17.56	13.9	14.14	14.51	14.73
Mass of Container (g) = (W3)	5.7	6.37	6	6.28	5.46	6.16
Mass of Moisture (g) = (W1 - W2) = A	3.78	4	2.9	2.86	2.21	2.19
Mass of Dry Soil (g) = (W2 - W3) = B	10.71	11.19	7.9	7.86	9.05	8.57
Moisture Content (%) = (A / B) x 100	35.3	35.7	36.7	36.4	24.4	25.55
Liquid Limit (LL) (%):	36	AV. Plastic. Limit.			25	
Plastic Limit (PL) (%):	25					
Plasticity Index (PI) (%): LL - PL		11				



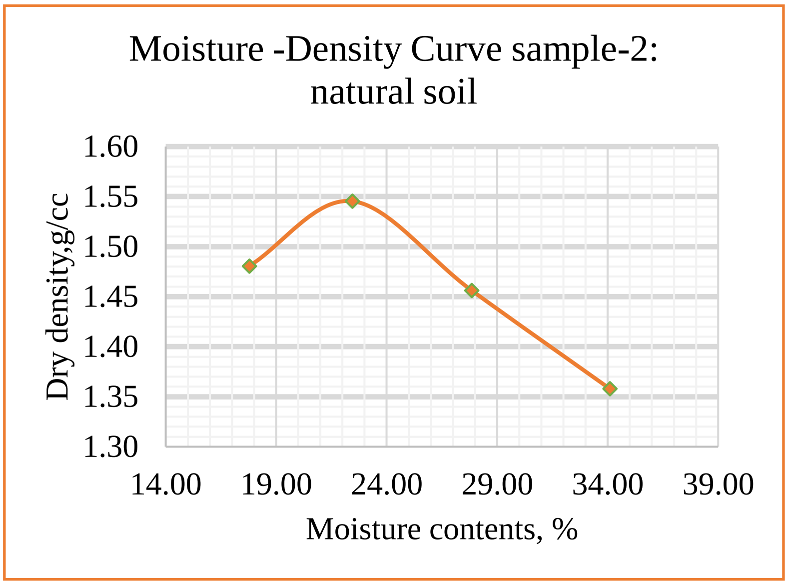
Appendix: F Compaction Test Analysis Data

Sample -1: natural soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	190	350	510	670	
Mass of Mold+Wet soil(gm)(A)	6255.6	6450.2	6615.1	6499.3	
Mass of Mold(gm)(B)	2714.8	2714.8	2714.8	2714.8	
Mass of Wet Soil(gm) A-B=C	3540.8	3735.4	3900.3	3784.5	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.66	1.76	1.84	1.78	
Moisture Content Determination					
Container Code.	P3	9	C15	T5C1	P1(NMC)
Mass of Wet soil+ Container (gm)(F)	132.5	147.7	193.9	149.6	131.5
Mass of dry soil+ container(gm)(G)	117.87	129.07	163.73	123.42	124.72
Mass of container(gm)(H)	32.6	32.4	37.8	33.5	17.7
Mass of moisture(gm)F-G=(I)	14.63	18.63	30.17	26.18	6.78
Mass of Dry soil(gm)G-H=(J)	85.27	96.67	125.93	89.92	107.02
Moisture content % (I/J) *100=K	17.1572652	19.27	24.51	29.11	6.34
Dry Density gm/cm^3 E/(100+K) *100	1.42	1.47	1.48	1.38	

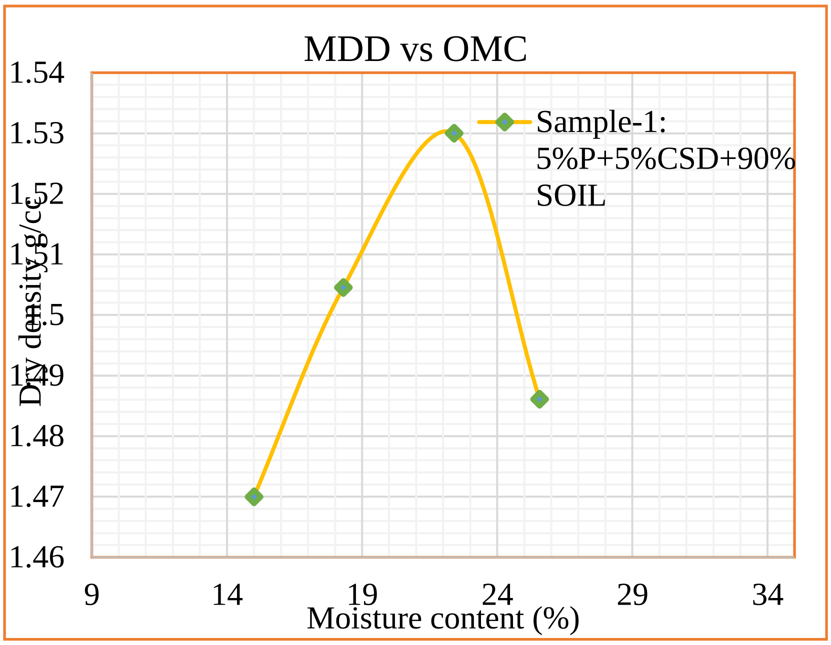


From the compaction curve	
MDD	1.48
OMC	24.1

Sample 2 - NS				
Moisture Density Determination				
Test No.	1	2	3	4
Mass of sample (gm)	4000	4000	4000	4000
Water Added(cc)	320	480	640	800
Mass of Mold+Wet soil(gm)(A)	6420	6733.6	6670.5	6584.2
Mass of Mold(gm)(B)	2716.4	2714.6	2716.4	2716.4
Mass of Wet Soil(gm)A-B=C	3703.6	4019	3954.1	3867.8
Volume of Mold cm^3(D)	2124	2124	2124	2124
Bulk Density gm/cm^3 C/D=(E)	1.74	1.89	1.86	1.82
Moisture Content Determination				
Container Code.	G19	ZE	P3	G34
Mass of Wet soil+Container (gm)(F)	168.5	145.5	152.7	198.25
Mass of dry soil+container (gm)(G)	148.5	124.9	127.3	156.7
Mass of container(gm)(H)	36	33.1	36.1	34.83
Mass of moisture(gm)F-G=(I)	20	20.6	25.4	41.55
Mass of Dry soil(gm)G-H=(J)	112.5	91.8	91.2	121.87
Moisture content % (I/J) *100=K	17.78	22.44	27.85	34.09
Dry Density gm/cm^3 E/(100+K) *100	1.48	1.55	1.46	1.36

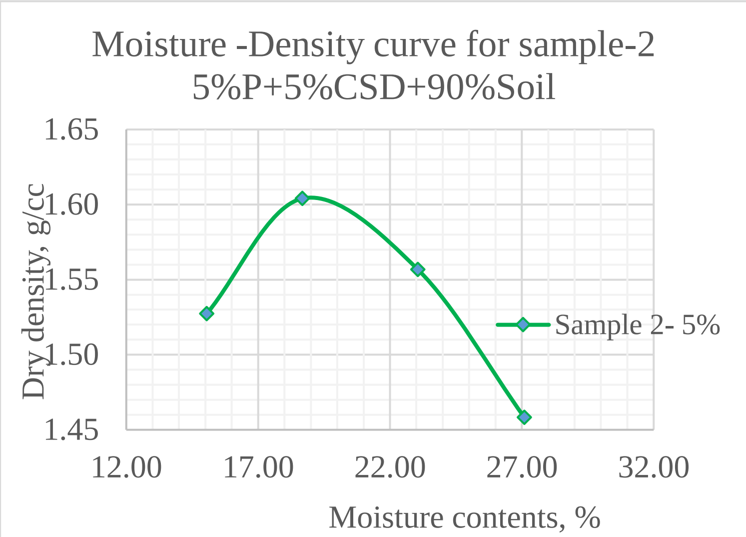


From the compaction curve	
MDD	1.55
OMC	22.44



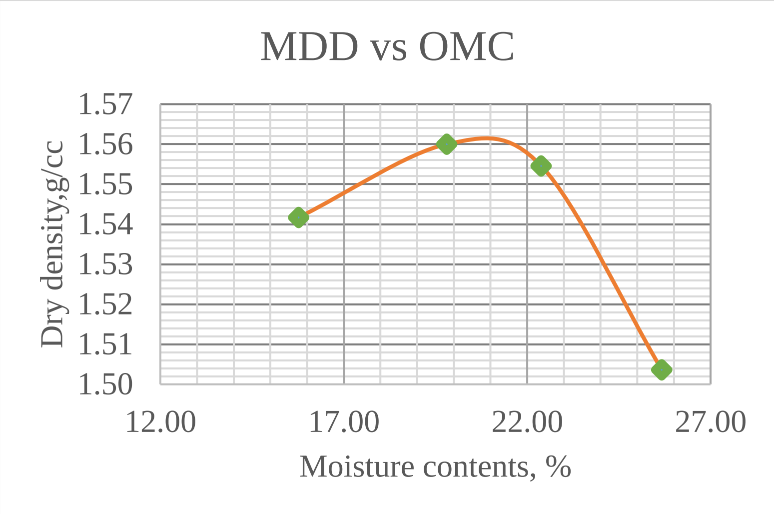
From the compaction curve	
MDD	1.53
OMC	20.57

Sample 2- 5%P+5%CSD+90%Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	300	460	620	780	
Mass of Mold + Wet soil(gm)(A)	6452	6763.5	6789	6656.5	
Mass of Mold(gm)(B)	2720	2720	2720	2720	
Mass of Wet Soil(gm)A-B=C	3732	4043.5	4069	3936.5	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.76	1.90	1.92	1.85	
Moisture Content Determination					
Container Code.	G3T2	9	J41	G	T1(NMC)
Mass of Wet soil + Container (gm)(F)	163	170	181	131.86	170.5
Mass of dry soil+ container (gm)(G)	146.23	148.36	153.22	107.52	161.47
Mass of container(gm)(H)	34.8	32.53	32.71	17.7	37.61
Mass of moisture(gm)F-G=(I)	16.77	21.64	27.78	24.34	9.03
Mass of Dry soil(gm)G-H=(J)	111.43	115.83	120.51	89.82	123.86
Moisture content % (I/J) *100=K	15.05	18.68	23.05	27.10	7.29
Dry Density gm/cm^3 E/(100+K) *100	1.53	1.60	1.56	1.46	



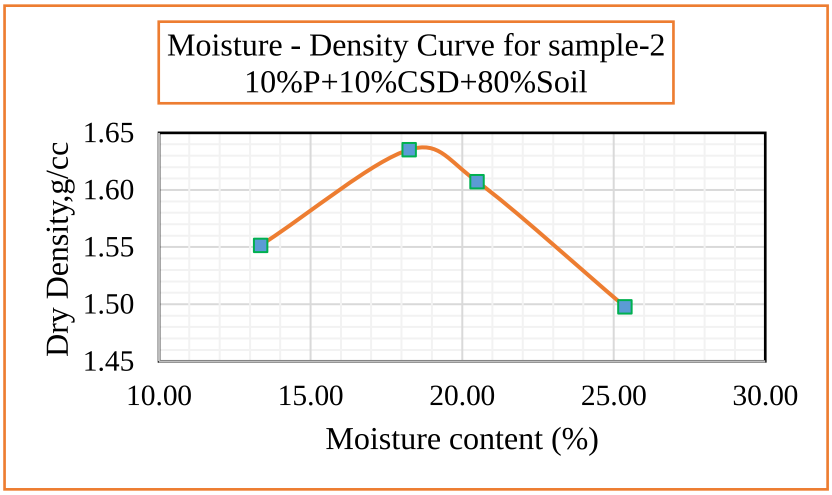
From the compaction curve	
MDD	1.6
OMC	18.68

sample 1-10%P+10%+CSD +80%Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	180	340	500	660	
Mass of Mold+Wet soil (gm)(A)	6451	6761	6728.5	6641	
Mass of Mold(gm)(B)	2717	2707.5	2707.5	2707.5	
Mass of Wet Soil(gm) A-B=C	3734	4053.5	4021	3933.5	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.76	1.91	1.89	1.85	
Moisture Content Determination					
Container Code.	9	C15	T5C1	T2	P1(NMC)
Mass of Wet soil +Container (gm)(F)	124.25	122.4	112.4	139.5	149.86
Mass of dry soil + container (gm)(G)	109.16	103.04	92.5	113	136.92
Mass of container(gm)(H)	32.5	25.5	18	17.56	18.8
Mass of moisture(gm)F-G=(I)	15.09	19.36	19.9	26.5	12.94
Mass of Dry soil(gm)G-H=(J)	76.66	77.54	74.5	95.44	118.12
Moisture content % (I/J) *100=K	19.68	19.97	26.71	27.77	10.95
Dry Density gm/cm^3 E/(100+K) *100	1.47	1.56	1.49	1.45	



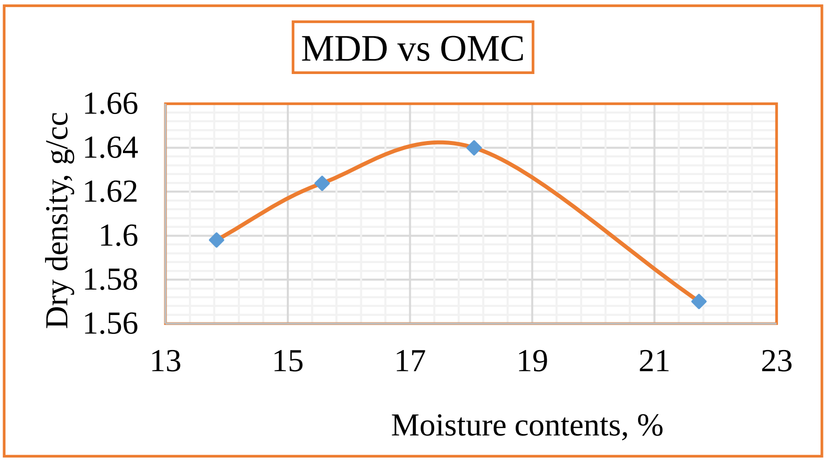
From the compaction curve	
MDD	1.56
OMC	19.97

Sample: - 2-10%P+10%CSD+80% Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	240	400	560	720	
Mass of Mold+Wet soil(gm)(A)	6454.7	6826.5	6832.8	6707.3	
Mass of Mold(gm)(B)	2719	2719	2719	2719	
Mass of Wet Soil(gm)A-B=C	3735.7	4107.5	4113.8	3988.3	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.76	1.93	1.94	1.88	
Moisture Content Determination					
Container Code.	A	A-16	P15	P65	P1(NMC)
Mass of Wet soil+Container(gm)(F)	192	168.8	129	134	131.5
Mass of dry soil+container(gm)(G)	173.74	147.82	110.25	114.51	122.62
Mass of container(gm)(H)	37	32.9	18.75	37.7	17.5
Mass of moisture(gm)F-G=(I)	18.26	20.98	18.75	19.49	8.88
Mass of Dry soil(gm)G-H=(J)	136.74	114.92	91.5	76.81	105.12
Moisture content % (I/J) *100=K	13.35	18.26	20.49	25.37	8.45
Dry Density gm/cm^3 E/(100+K) *100	1.55	1.64	1.61	1.50	



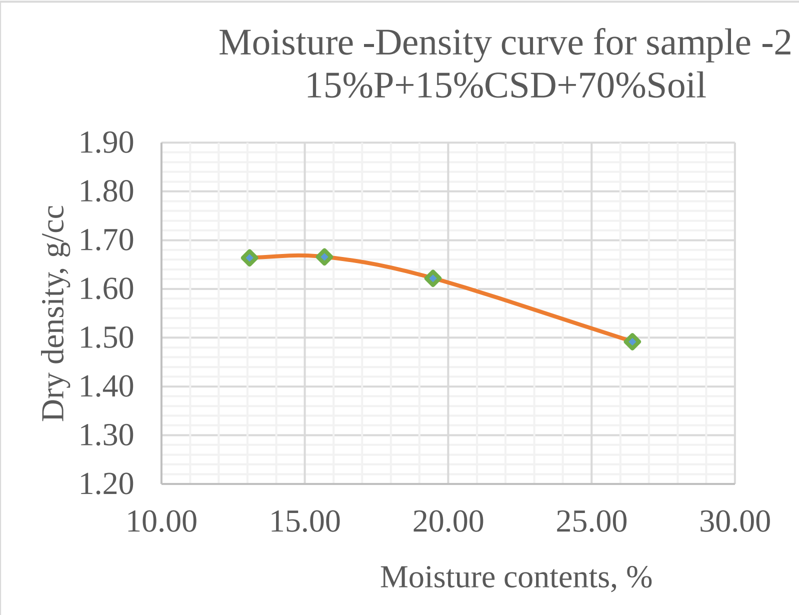
From the compaction curve	
MDD	1.56
OMC	19.97

Sample -1:15%P+15%CSD+70% Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	140	300	460	620	
Mass of Mold+Wet soil(gm)(A)	6622.2	6744.2	6837.5	6718.2	
Mass of Mold(gm)(B)	2718.5	2718.5	2715.5	2718.6	
Mass of Wet Soil(gm)A-B=C	3903.7	4025.7	4122	3999.6	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.84	1.90	1.94	1.88	
Moisture Content Determination					
Container Code.	9	G3T2	G3T3	P65	T5C2(NMC)
Mass of Wet soil+Container(gm)(F)	122.7	142.99	137.46	194.34	128.43
Mass of dry soil+container(gm)(G)	109.36	128.42	121.9	163.104	117.5
Mass of container(gm)(H)	32.5	34.79	37.9	37.76	17.97
Mass of moisture(gm)F-G=(I)	13.34	14.57	15.56	31.236	10.93
Mass of Dry soil(gm)G-H=(J)	90.2	93.63	84	125.344	99.53
Moisture content % (I/J) *100=K	14.8	15.56	18.52	24.92	10.99
Dry Density gm/cm^3 E/(100+K) *100	1.60	1.64	1.64	1.51	



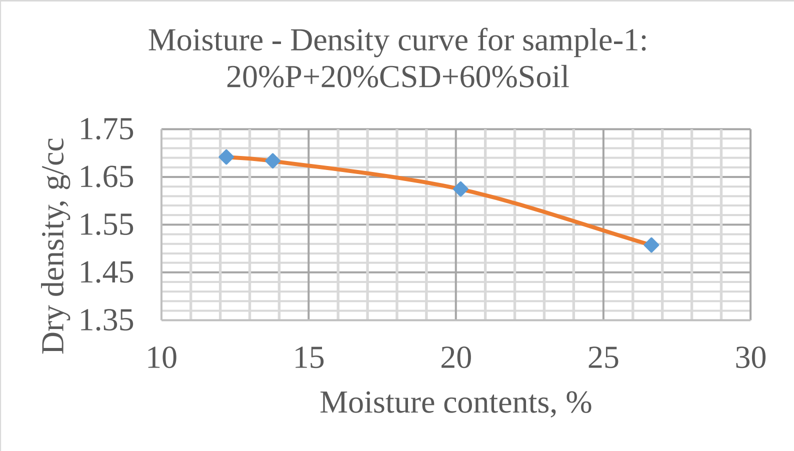
From the compaction curve	
MDD	1.5446
OMC	18.52

SAMPLE- 2:15%P+15%CSD+70%Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	300	460	620	780	
Mass of Mold+Wet soil(gm)(A)	6711.6	6811.6	6834.6	6725	
Mass of Mold(gm)(B)	2714.8	2718.5	2718.5	2718.5	
Mass of Wet Soil(gm)A-B=C	3996.8	4093.1	4116.1	4006.5	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.88	1.93	1.94	1.89	
Moisture Content Determination					
Container Code.	E	T1	A	P15	G19(NMC)
Mass of Wet soil +Container (gm)(F)	139.93	119.45	146.3	166.18	130.31
Mass of dry soil + container (gm)(G)	128.14	108.35	128.5	138.46	122.44
Mass of container(gm)(H)	37.94	37.57	37.06	33.57	17.41
Mass of moisture(gm)F-G=(I)	11.79	11.1	17.8	27.72	7.87
Mass of Dry soil(gm)G-H=(J)	90.2	70.78	91.44	104.89	105.03
Moisture content % (I/J) *100=K	13.07	15.68	19.47	26.43	7.49
Dry Density gm/cm^3 E/(100+K) *100	1.66	1.67	1.62	1.49	



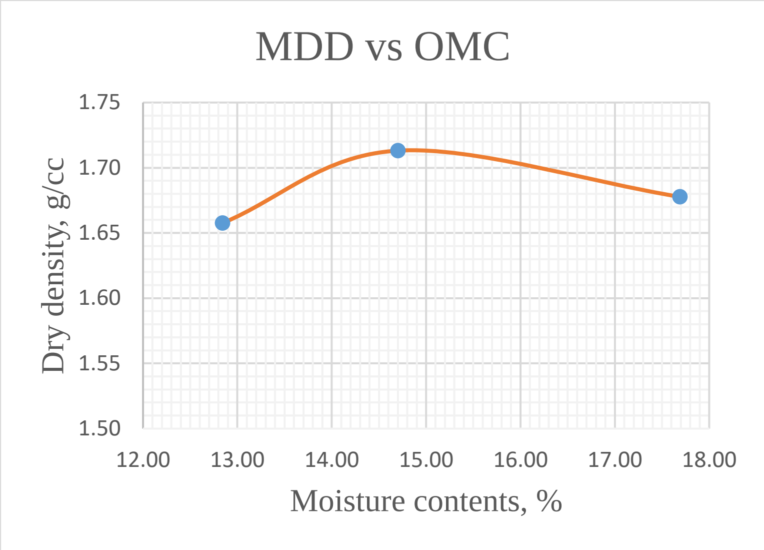
From the compaction curve	
MDD	1.67
OMC	15.68

Sample- 1: 20%P+20%CSD+60%Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	100	260	420	580	
Mass of Mold + Wet soil(gm)(A)	6750	6786.1	6852.3	6760.7	
Mass of Mold(gm)(B)	2718.5	2718.5	2707.1	2707.1	
Mass of Wet Soil(gm)A-B=C	4031.5	4067.6	4145.2	4053.6	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.90	1.9150659	1.95	1.91	
Moisture Content Determination					
Container Code.	A	G3T3	T2	9	G19(NMC)
Mass of Wet soil + Container (gm)(F)	123.1	161.7	130.37	188.25	131.9
Mass of dry soil + container (gm)(G)	113.8	146.7	111.45	155.5	121.16
Mass of container(gm)(H)	37.6	37.83	17.59	32.56	17.99
Mass of moisture(gm)F-G=(I)	9.3	15	18.92	32.75	10.74
Mass of Dry soil(gm)G-H=(J)	76.2	108.87	93.86	122.94	103.17
Moisture content % (I/J) *100=K	12.204724	13.78	20.16	26.64	10.41
Dry Density gm/cm^3 E/(100+K) *100	1.691613	1.68	1.62	1.51	



From the compaction curve	
MDD	1.68
OMC	15.68

Sample- 2: 20%P+20%CSD+60%Soil					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (gm)	4000	4000	4000	4000	
Water Added(cc)	240	400	560	720	
Mass of Mold + Wet soil(gm)(A)	6680	6880.8	6900.9	6680	
Mass of Mold(gm)(B)	2707.1	2707.1	2707.1	2707.1	
Mass of Wet Soil(gm)A-B=C	3972.9	4173.7	4193.8	3972.9	
Volume of Mold cm^3(D)	2124	2124	2124	2124	
Bulk Density gm/cm^3 C/D=(E)	1.8704802	1.97	1.97	1.87	
Moisture Content Determination					
Container Code.	C9	E	T1	P15	P3(NMC)
Mass of Wet soil+ Container (gm)(F)	165	170.6	229.9	205.1	205.3
Mass of dry soil + container (gm)(G)	149.5	153.59	200.38	172.96	193.47
Mass of container(gm)(H)	36.6	37.9	33.5	37.5	35.8
Mass of moisture(gm)F-G=(I)	15.5	17.01	29.52	32.14	11.83
Mass of Dry soil(gm)G-H=(J)	112.9	115.69	166.88	135.46	157.67
Moisture content % (I/J) *100=K	13.728964	14.70	17.69	23.73	7.50
Dry Density gm/cm^3 E/(100+K) *100	1.6446824	1.71	1.68	1.51	



From the compaction curve	
MDD	1.71
OMC	15.68

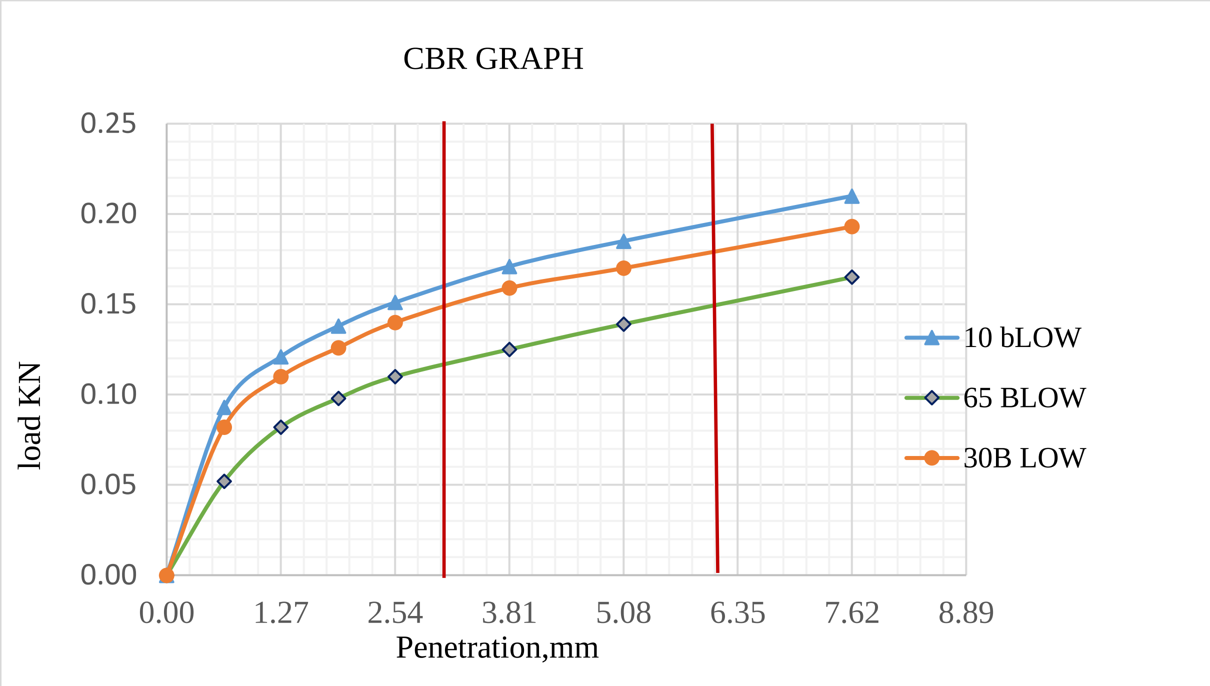
APPENDIX: F: Sample California Bearing Ratio (CBR) Test Analysis

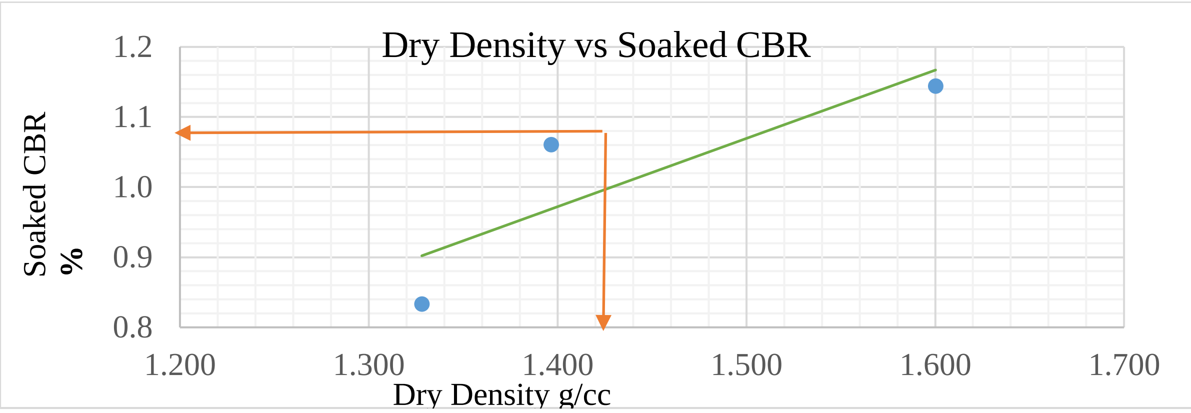
Natural soil sample for sample-1

	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.093		0.082		0.052	
1.27	0.121		0.11		0.082	
1.91	0.138		0.126		0.098	
2.54	0.151	1.14	0.14	1.061	0.11	0.83
3.81	0.171		0.159		0.125	
5.08	0.185	0.93	0.17	0.85	0.139	0.7
7.62	0.21		0.193		0.165	

OMC (%)					23.96
MDD					1.48
Dry Density at 95% of MDD:					1.406
No. of blows	MCBS %	DDBS g/cm3	Corrected CBR %		% Compaction
10	16.0	1.328	0.8		90
30	16.6	1.397	1.1		94
65	17.2	1.600	1.1		108
CBR (%) @ 95 % MDD			1.1	% Swell	5.60

Sample-2: natural soil

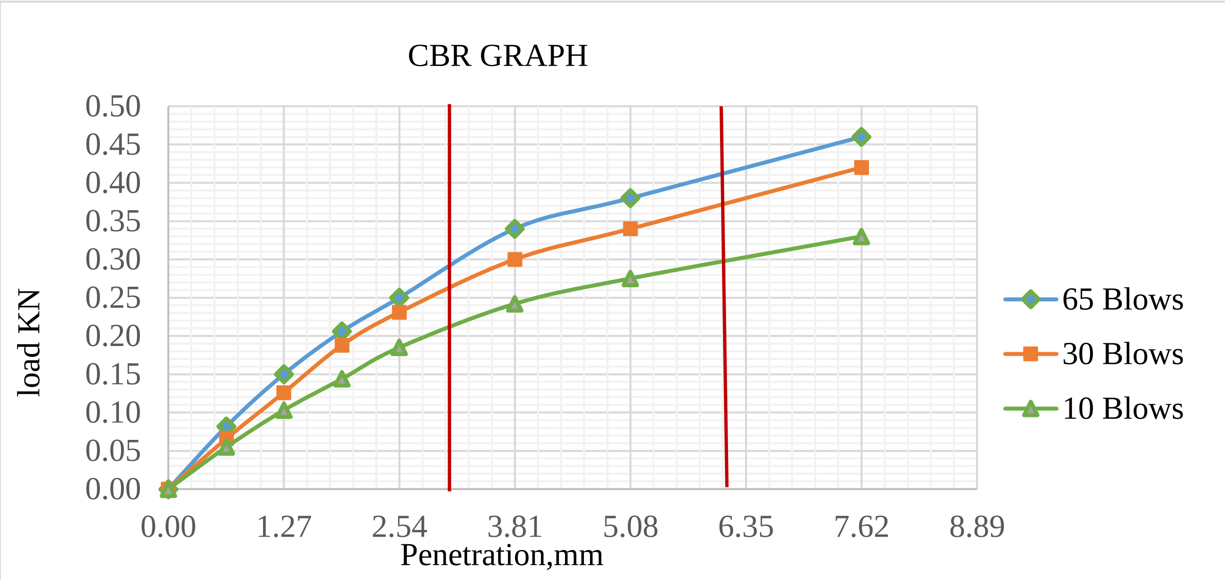


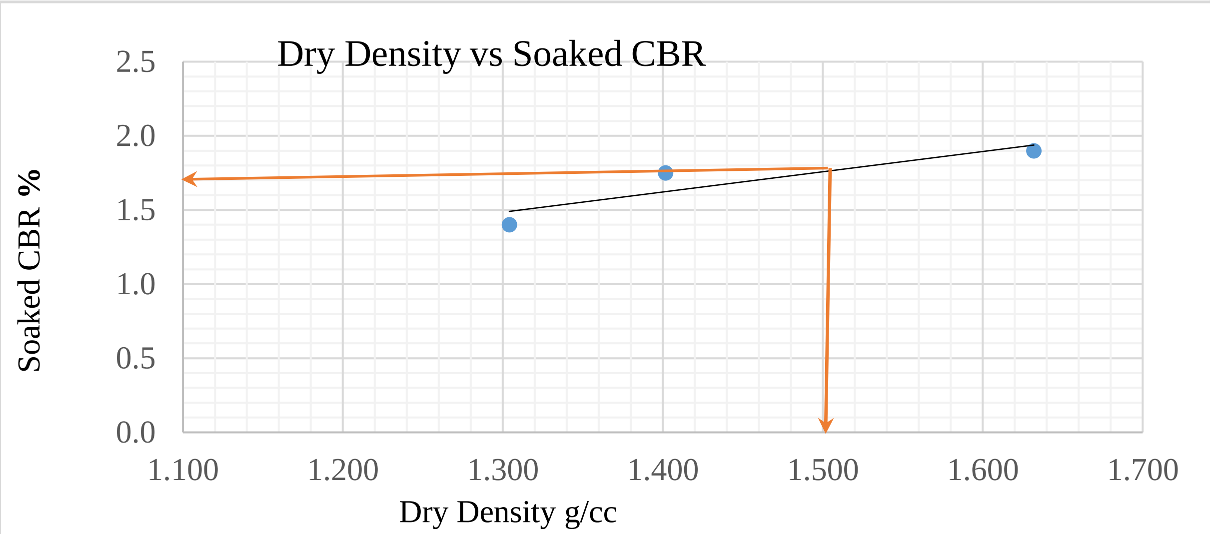


	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.082		0.066		0.055	
1.27	0.15		0.126		0.103	
1.91	0.206		0.188		0.144	
2.54	0.25	1.894	0.231	1.75	0.185	1.401515
3.81	0.34		0.3		0.242	
5.08	0.38	1.9	0.34	1.7	0.2751	1.38
7.62	0.46		0.42		0.33	

Modified Max.Dry Density g/cc			1.55		OMC %		22.44	
Swell Determination								
	10 Blows				30 Blows		65 Blows	
	Gauge rdg mm		Swell in %	Gauge rdg mm	Swell in %		Gauge rdg mm	Swell in %
	Initial	0.30	5.84	0.3	5.96		0.30	8.05
	Final	7.10		7.24			9.67	

No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %		% Compaction
10	18.9	1.304	1.4	5.8	84
30	18.0	1.402	1.8	6.0	90
65	20.4	1.632	1.9	8.0	105
CBR (%) @ 95 % MDD			1.7	% Swell	6.81





Sample-1:5%P+5%CSD+90% Soil

	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.16		0.12		0.072	
1.27	0.204		0.164		0.12	
1.91	0.227		0.19		0.142	
2.54	0.246	1.86	0.2064	1.56	0.16	1.21
3.81	0.276		0.24		0.19	
5.08	0.293	1.47	0.264	1.32	0.204	1.02
7.62	0.316		0.283		0.215	

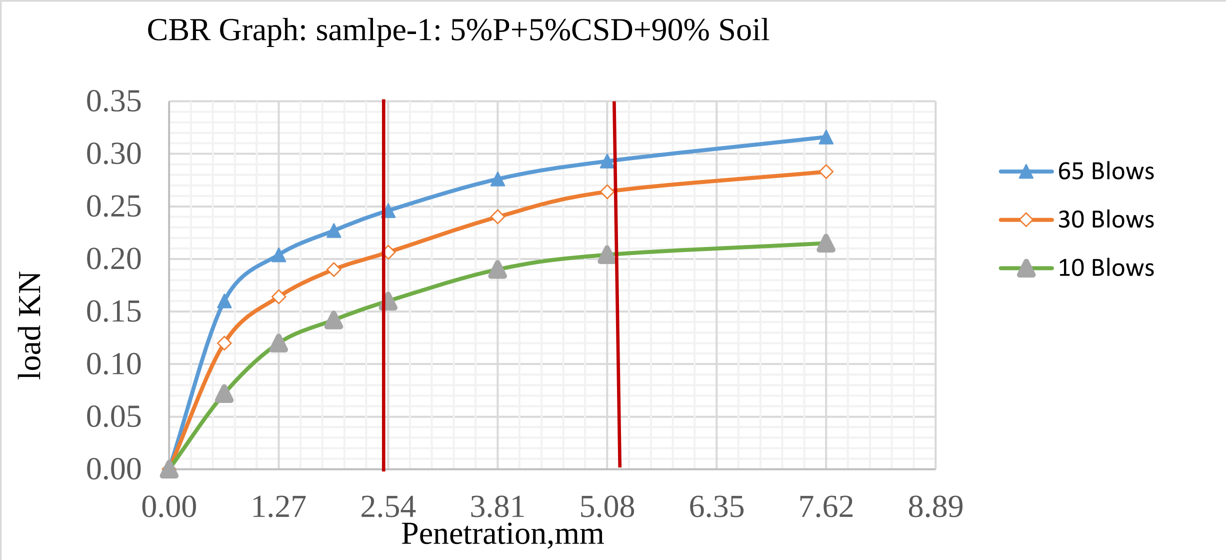
Modified Max.Dry Density g/cc	1.56	OMC %	20.57

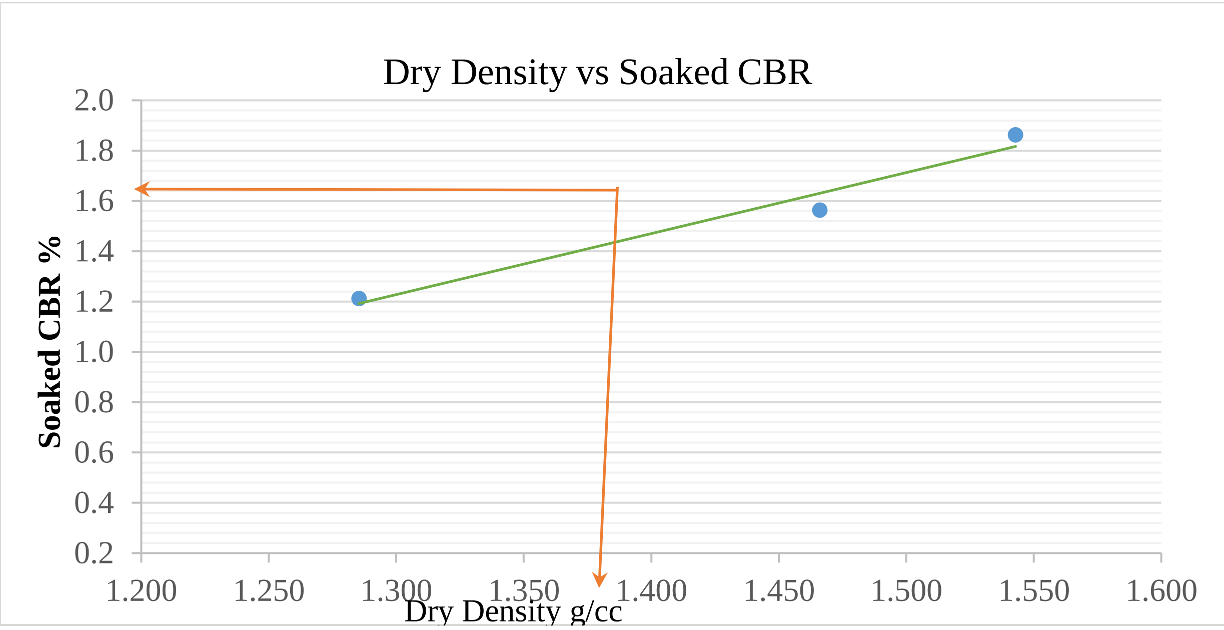
Swell Determination

Date		10 Blows Gauge rdg mm	Swell in %	30 Blows Gauge rdg mm	Swell in %	65 Blows Gauge rdg mm	Swell in %
15/2/2021	Initial	0.33	11.40	0.4	8.93	0.19	9.28
19/2/2021	Final	13.60		10.80		10.99	

Dry Density at 95% of MDD: 1.482

No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	swelling	% Compaction
10	28.8	1.285	1.2	11.4	82
30	24.7	1.466	1.6	8.9	94
65	21.5	1.543	1.9	9.3	99
CBR (%) @ 95 % MDD			1.7	% Swell	6.81



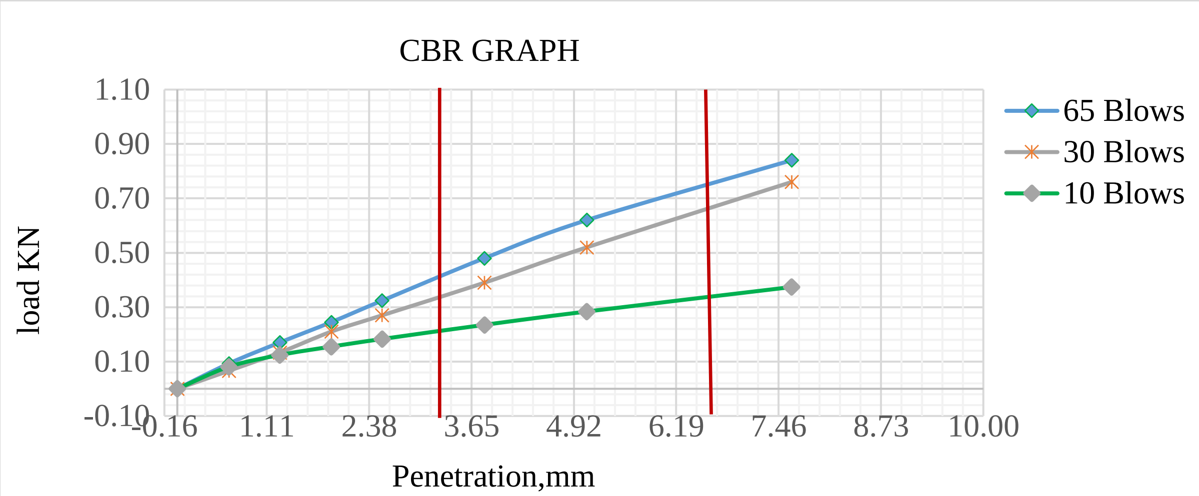


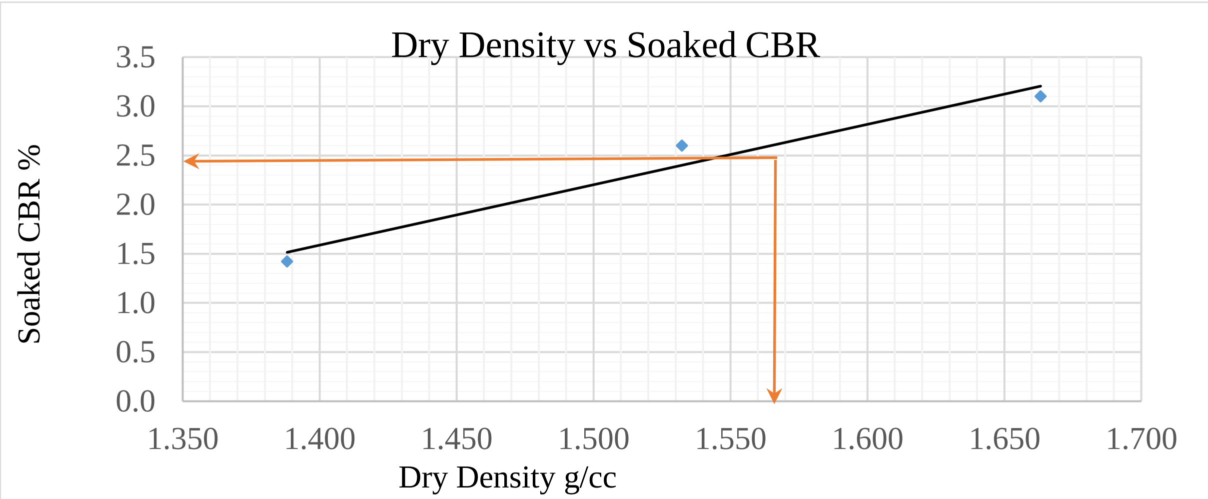
Sample-2:5%P+5%CSD+90% Soil

	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.093		0.066		0.081	
1.27	0.17		0.133		0.124	
1.91	0.245		0.21		0.155	
2.54	0.324	2.454	0.27	2.0455	0.183	1.386
3.81	0.48		0.39		0.235	
5.08	0.62	3.1	0.52	2.6	0.284	1.42
7.62	0.84		0.76		0.374	

Modified Max.Dry Density g/cc			1.56		OMC %		22.44	
Swell Determination								
Date	Gauge rdg mm		10 Blows		30 Blows		65 Blows	
			Swell in %		Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
15/2/2021	Initial	0.40	5.76		0.37	6.81	0.56	6.49
19/2/2021	Final	7.10			8.30		8.60	

No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %		% Compaction
10	17.7	1.388	1.4		89
30	17.9	1.532	2.6		98
65	15.0	1.663	3.1		107
CBR (%) @ 95 % MDD			2.2	% Swell	6.49

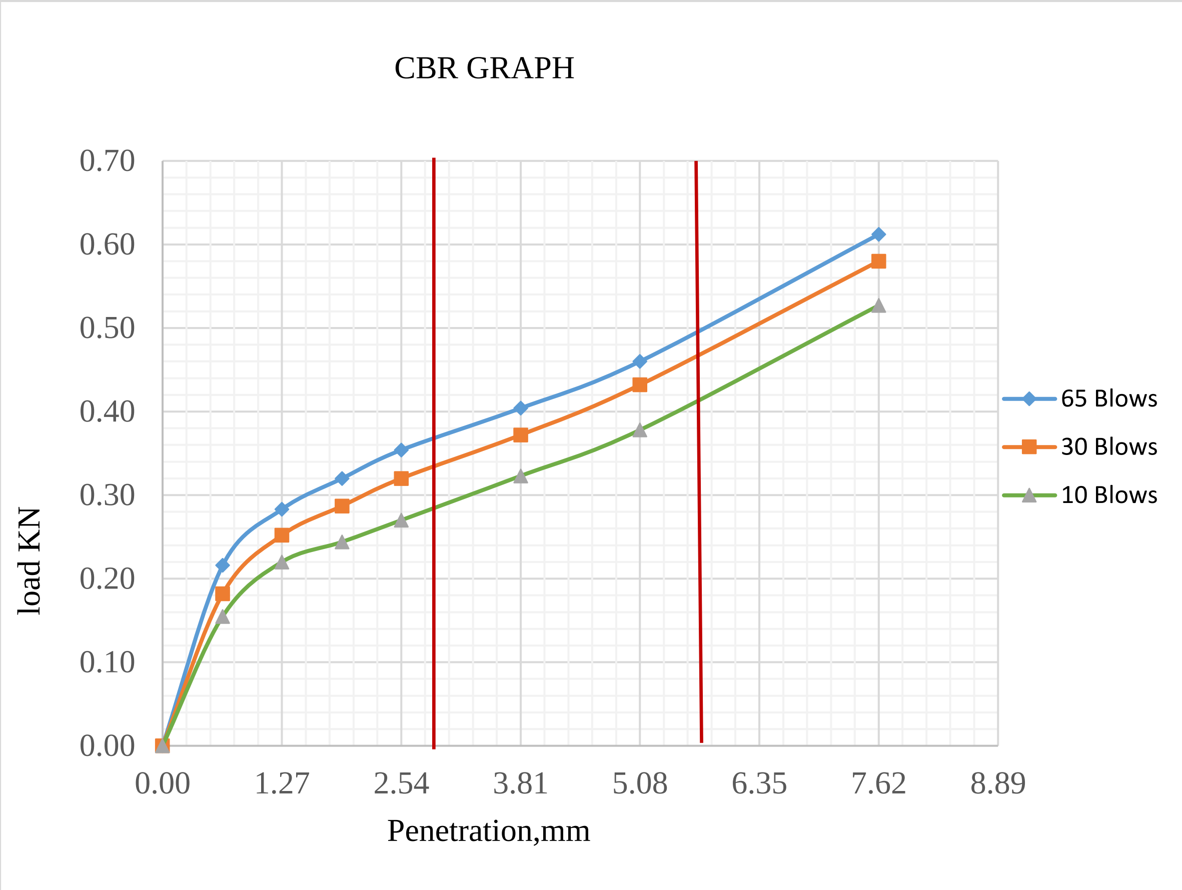


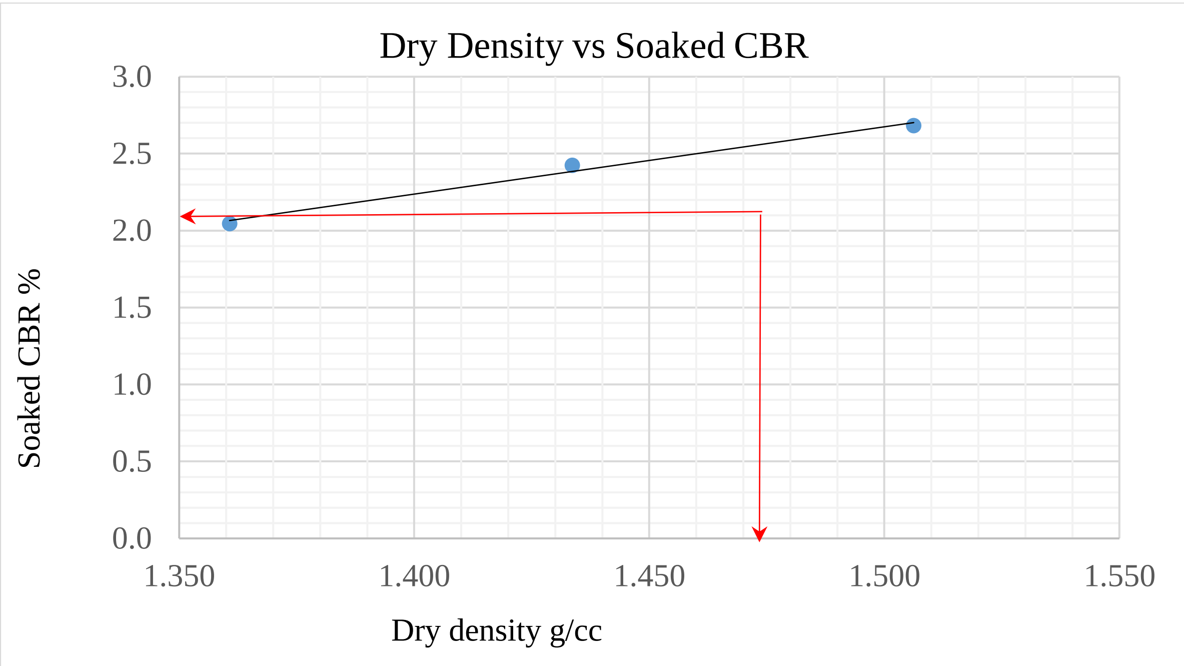


CBR penetration determination for Sample-1:10%P+10%CSD+80% Soil						
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.216		0.182		0.155	
1.27	0.283		0.252		0.22	
1.91	0.32		0.287		0.244	
2.54	0.354	2.682	0.32	2.42	0.27	2.045
3.81	0.404		0.372		0.323	
5.08	0.46	2.3	0.432	2.16	0.378	1.89
7.62	0.612		0.58		0.527	

Modified Max. Dry Density g/cc		1.56		OMC %	19.27	
Swell Determination						
	10 Blows			30 Blows		65 Blows	
Date	Gauge rdg mm		Swell in %	Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
	Initial	0.50	16.67	0.4	17.31	0.30	5.69
	Final	19.90		20.55		16.41	

Dry Density at 95% of MDD:					1.482
No. of blows	MCBS %	DDBS g/cm3	Corrected CBR %		% Compaction
10	24.1	1.361	2.0		87
30	25.7	1.434	2.4		92
65	25.0	1.506	2.7		97
CBR (%) @ 95 % MDD			2.7	% Swell	6.69

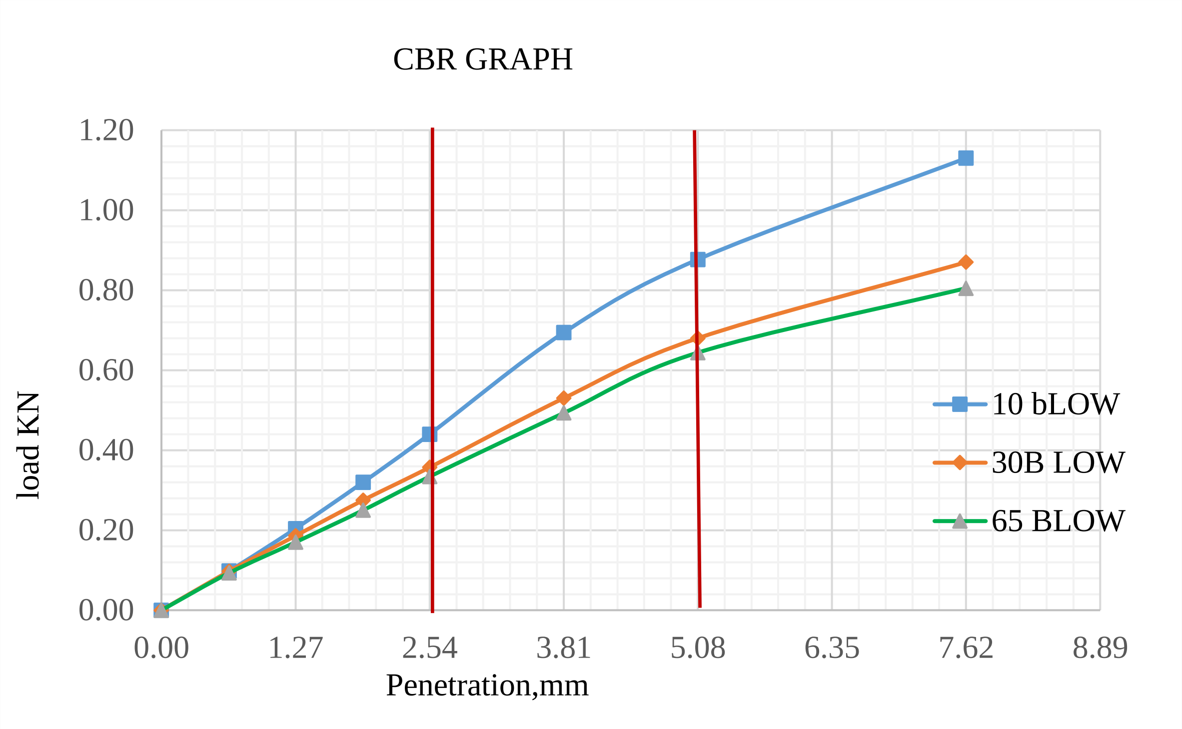

CBR GRAPH
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
load KN
0.00
1.27
2.54
3.81
5.08
6.35
7.62
8.89
Penetration,mm
65 Blows
30 Blows
10 Blows

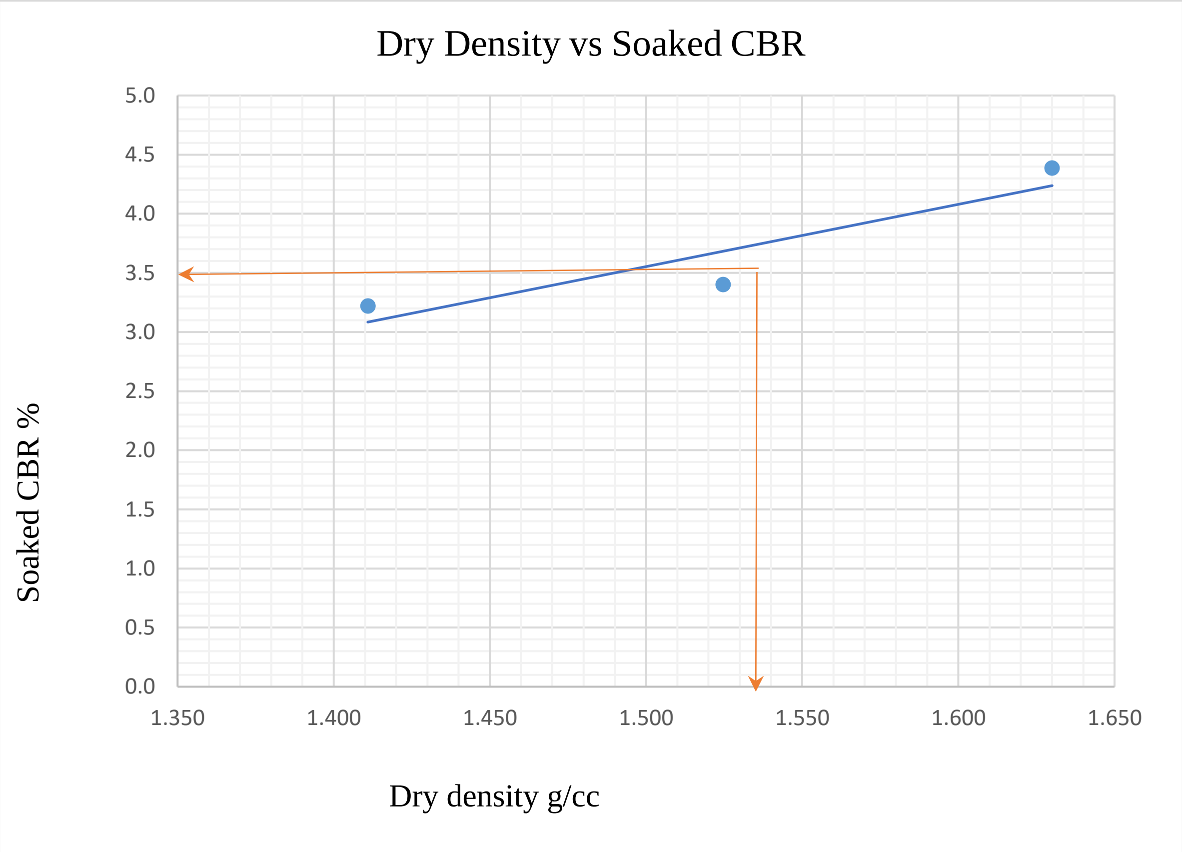

Dry Density vs Soaked CBR
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Soaked CBR %
1.350
1.400
1.450
1.500
1.550
Dry density g/cc

CBR penetration determination for Sample-2:10%P+10%CSD+80% Soil						
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.098		0.0972		0.093	
1.27	0.204		0.186		0.17	
1.91	0.32		0.275		0.25	
2.54	0.44	3.33	0.3571	2.71	0.334	2.53
3.81	0.514		0.53		0.463	
5.08	0.66	3.3	0.54	2.7	0.504	2.52
7.62	1.13		0.87		0.805	

Modified Max.Dry Density g/cc	1.56		OMC %		19.27		
Swell Determination							
	10 Blows			30 Blows		65 Blows	
	Gauge rdg mm		Swell in %	Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
	Initial	0.50	16.67	0.4	17.31	0.30	3.72
	Final	19.90		20.55		16.41	

Dry Density at 95% of MDD:				1.520
a	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction
10	14.4	1.411	3.2	88
30	15.3	1.525	3.4	95
65	16.6	1.630	4.4	102
CBR (%) @ 95 % MDD		3.4	% Swell	3.72


CBR GRAPH
1.20
1.00
0.80
0.60
0.40
0.20
0.00
load KN
0.00
1.27
2.54
3.81
5.08
6.35
7.62
8.89
Penetration,mm
10 bLOW
30B LOW
65 BLOW


Dry Density vs Soaked CBR
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
Soaked CBR %
1.350
1.400
1.450
1.500
1.550
1.600
1.650
Dry density g/cc

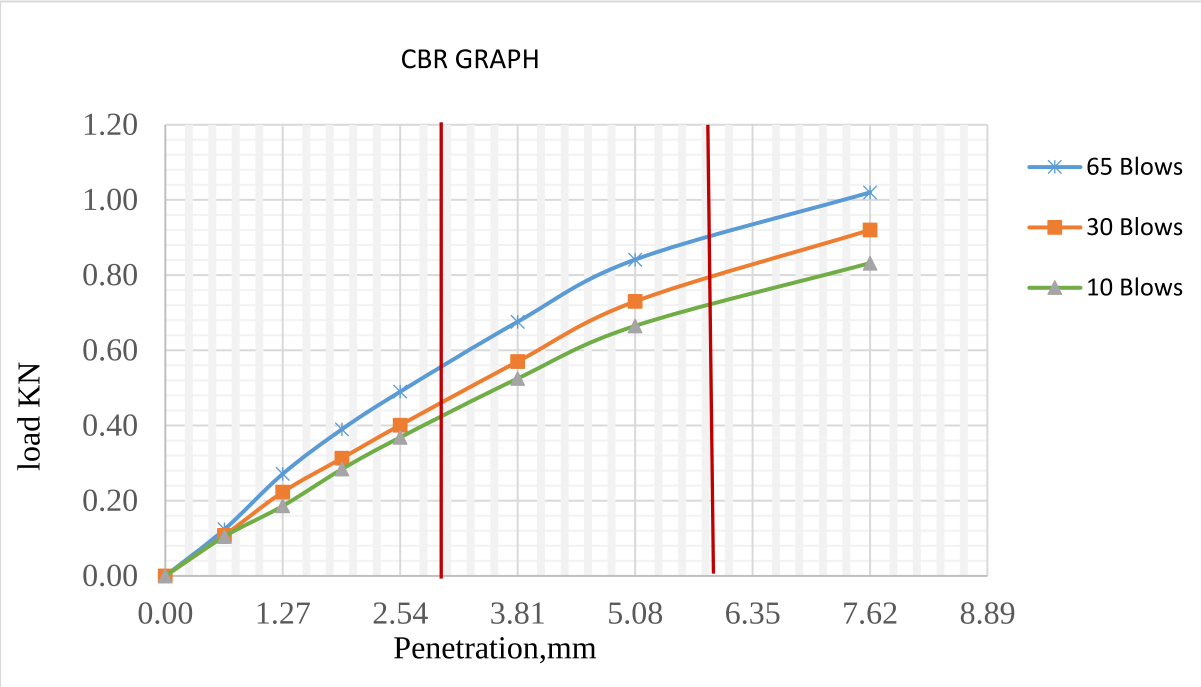
	Sample-1:15%P+15%CSD+70% Soil					
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.124		0.108		0.105	
1.27	0.2714		0.223		0.186	
1.91	0.39		0.313		0.284	
2.54	0.49	3.71	0.401	3.04	0.3685	2.79
3.81	0.676		0.57		0.525	
5.08	0.8413	4.21	0.73	3.65	0.665	3.325
7.62	1.02		0.92		0.832	

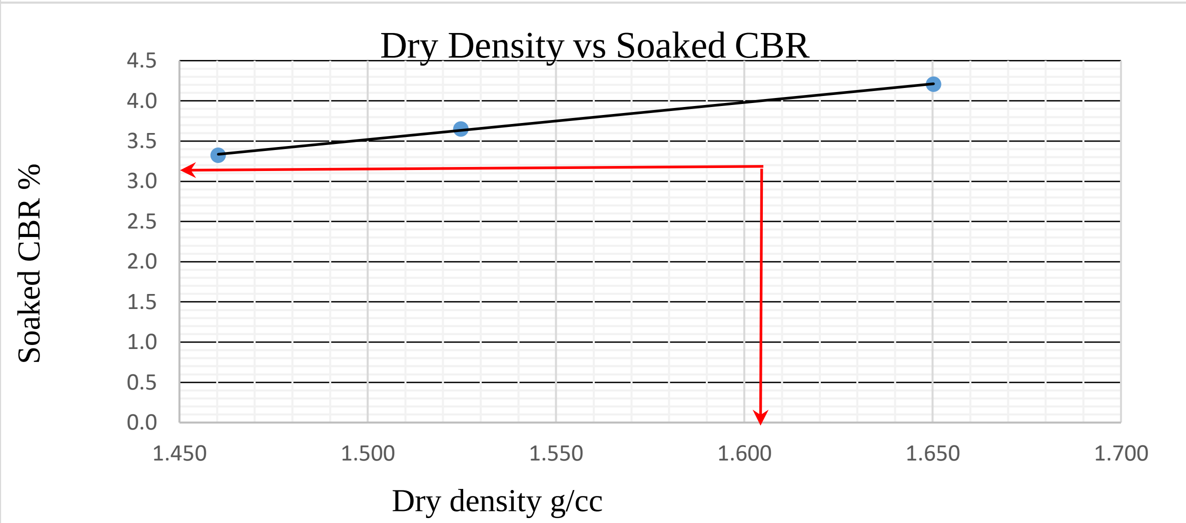
Modified Max.Dry Density g/cc	1.64	OMC %	15.56

Swell Determination

	10 Blows			30 Blows		65 Blows	
	Gauge rdg mm		Swell in %	Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
	Initial	0.30	13.92	0.4	20.94	0.30	26.29
	Final	16.50		24.77		30.90	

Dry Density at 95% of MDD:				1.558	
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	14.4	1.460	3.3	89	
30	15.3	1.525	3.7	93	
65	16.6	1.650	4.2	101	
CBR (%) @ 95 % MDD			3.8	% Swell	3.59

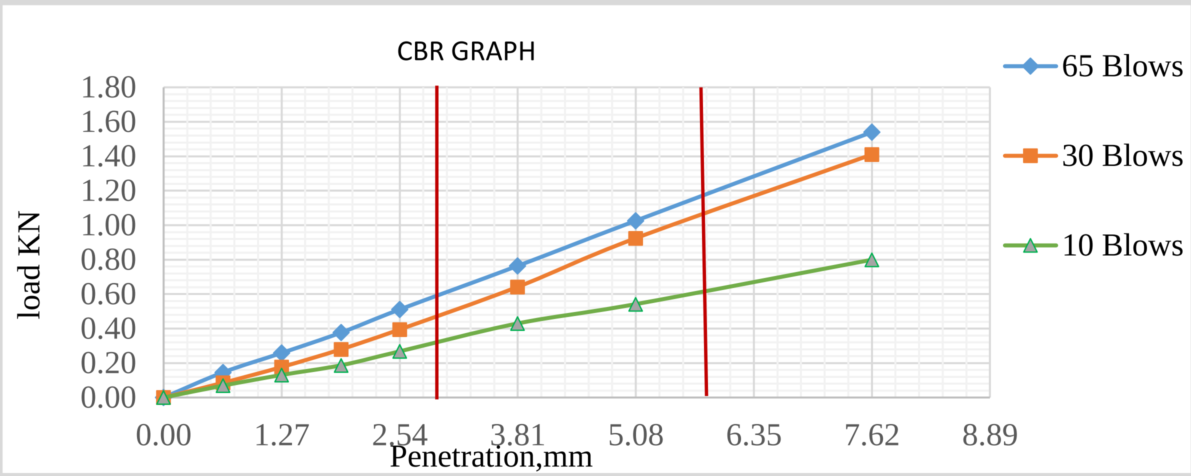


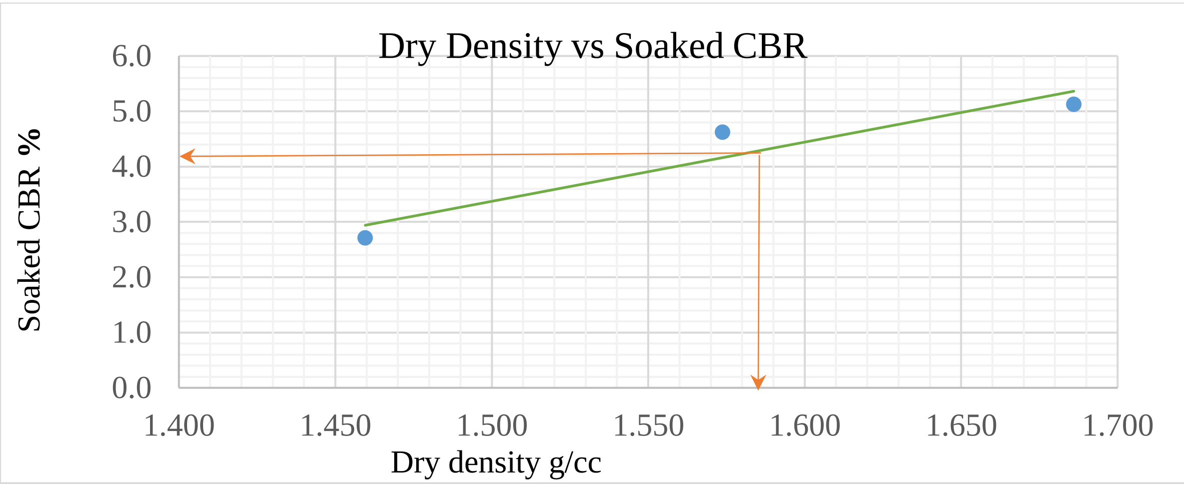


CBR penetration determination for sample-2:15%P+15%CSD+70% Soil						
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.146		0.0855		0.0693	
1.27	0.2586		0.177		0.131	
1.91	0.377		0.28		0.1872	
2.54	0.511	3.87	0.395	2.99	0.268	2.03
3.81	0.764		0.642		0.43	
5.08	1.026	5.13	0.9246	4.6	0.542	2.71
7.62	1.54		1.41		0.7995	

Modified Max.Dry Density g/cc			1.66		OMC %	14.70	
Swell Determination							
Date	10 Blows Gauge rdg mm		Swell in %	30 Blows Gauge rdg mm	Swell in %	65 Blows Gauge rdg mm	Swell in %
15/2/2021	Initial	0.35	4.30	0.4	5.69	0.35	6.79
19/2/2021	Final	5.36		7.02		8.25	

Dry Density at 95% of MDD:					
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	13.6	1.460	2.7	88	
30	13.6	1.574	4.6	95	
65	12.5	1.686	5.1	102	
CBR (%) @ 95 % MDD			4.7	% Swell	3.5





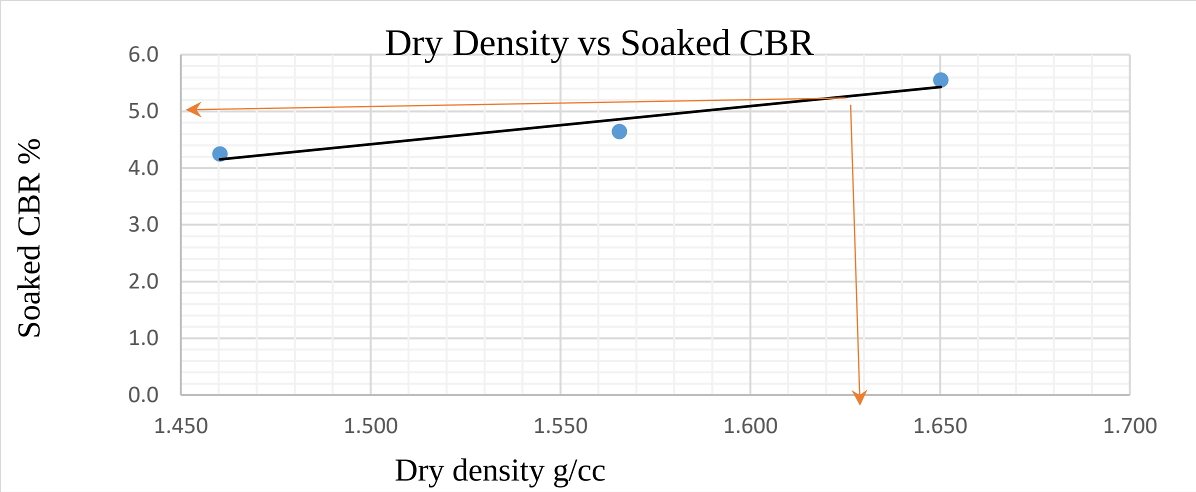
CBR penetration determination for Sample-1:20%P+20%CSD+60% Soil						
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.142		0.111		0.126	
1.27	0.32		0.255		0.22	
1.91	0.47		0.372		0.334	
2.54	0.61	4.621	0.496	3.76	0.441	3.341
3.81	0.878		0.723		0.65	
5.08	1.11	5.55	0.929	4.645	0.85	4.25
7.62	1.382		1.2		1.06	

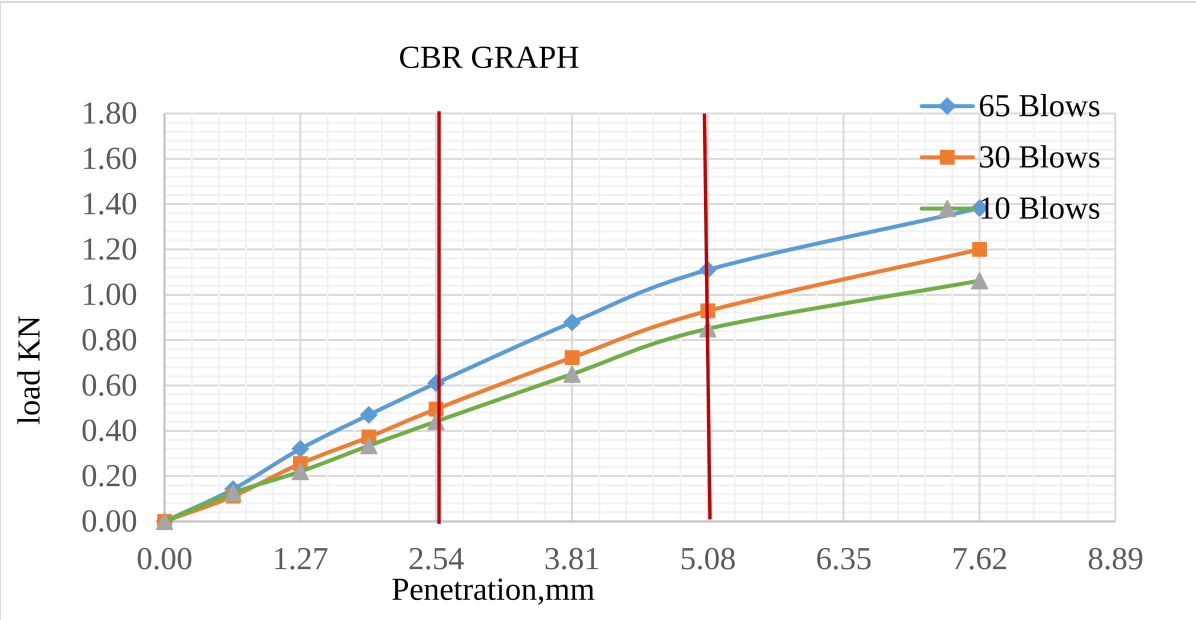
Modified Max.Dry Density g/cc	1.71	OMC %	13.37

Swell Determination

	10 Blows			30 Blows		65 Blows	
	Gauge rdg mm		Swell in %	Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
	Initial	0.20	15.03	0.28	16.96	0.31	24.82
	Final	17.70		20.02		29.20	

Dry Density at 95% of MDD:				1.622
No. of blows	MCBS%	DDBS g/cm3	Corrected CBR %	% Compaction
10	14.4	1.460	4.3	86
30	15.3	1.566	4.6	92
65	16.6	1.650	5.6	97
CBR (%) @ 95 % MDD		5	% Swell	1.92

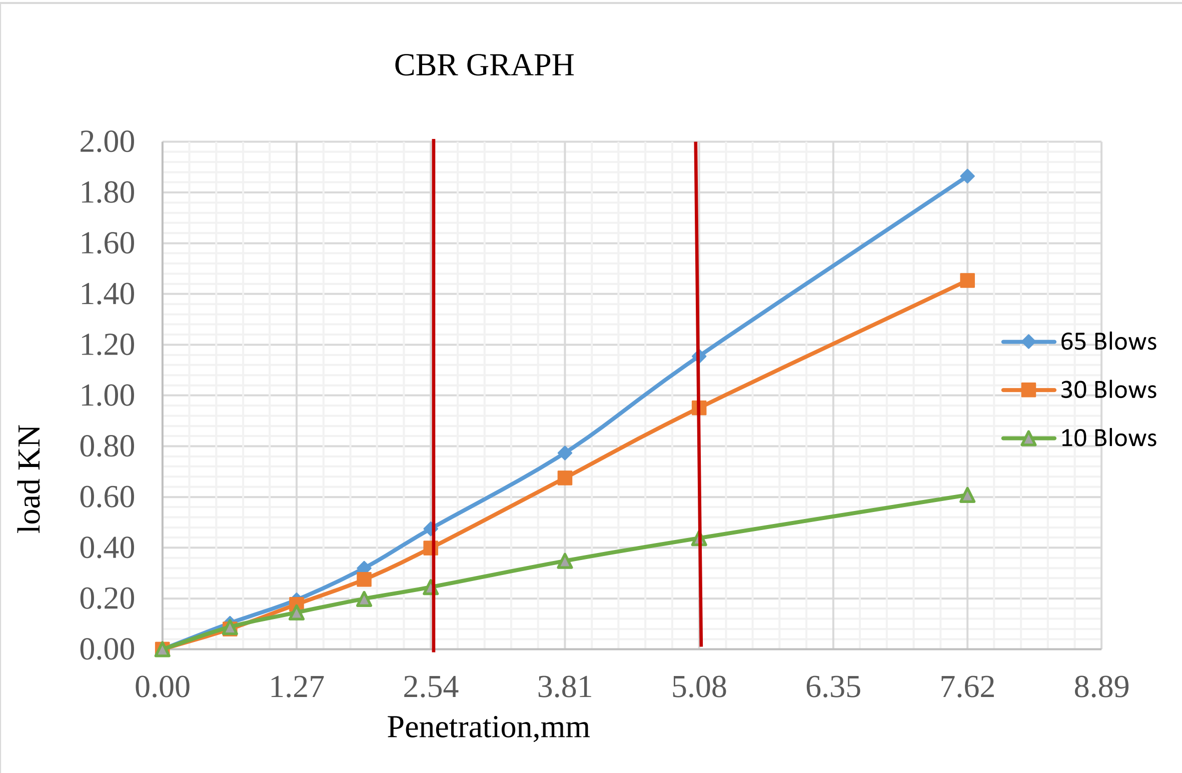


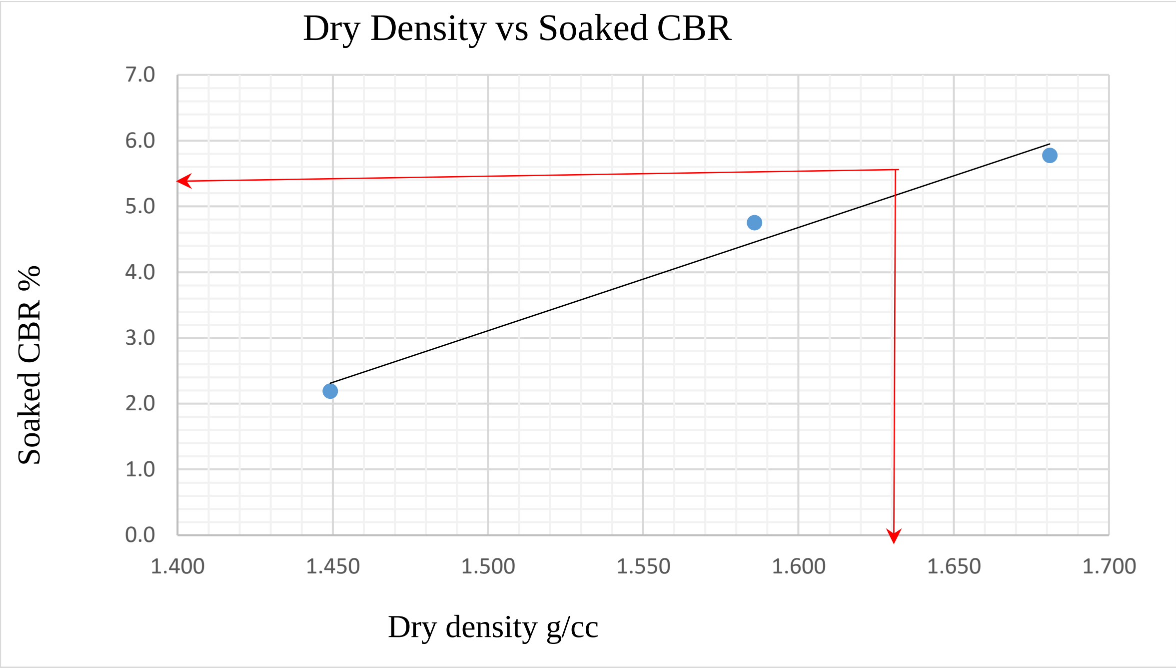


CBR penetration determination for sample-2:20%P+20%CSD+60% Soil						
	65 blows		30 blows		10 blows	
Penetration	Load (KN)	CBR	Load	CBR	Load	CBR
0	0		0		0	
0.64	0.1029		0.08		0.088	
1.27	0.195		0.177		0.145	
1.91	0.32		0.275		0.199	
2.54	0.4751	3.6	0.399	3.023	0.245	1.856
3.81	0.774		0.675		0.348	
5.08	1.155	5.775	0.951	4.755	0.438	2.19
7.62	1.864		1.453		0.608	

Modified Max. Dry Density g/cc			1.71		OMC %		13.07	
Swell Determination								
10 Blows					30 Blows		65 Blows	
Gauge rdg mm				Swell in %	Gauge rdg mm	Swell in %	Gauge rdg mm	Swell in %
	Initial	0.35		4.25	0.35	5.25	0.35	5.49
	Final	50.30			6.46		6.74	

Dry Density at 95% of MDD:					1.625
No. of blows	MCBS %	DDBS g/cm3	Corrected CBR %		% Compaction
10	12.2	1.449	2.2		85
30	11.8	1.586	4.8		93
65	12.1	1.681	5.8		98
CBR (%) @ 95 % MDD			5.5	% Swell	1.86




Dry Density vs Soaked CBR
Soaked CBR %
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
1.400
1.450
1.500
1.550
1.600
1.650
1.700
Dry density g/cc