JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HIGHWAY ENGINEE0RING STREAM

IMPROVEMENT OF THE EXPANSIVE SOIL SUBGRADE STRENGTH USING QUARRY DUST, SAW DUST AND QUARRY DUST –SAW DUST MIXES

BY: GETU REGEA ARERU

THIS THESIS IS SUBMITTED TO THE SCHOOL OF GRADUATE STUDIES OF JIMMA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN CIVIL ENGINEERING

Jul, 2022

Jimma, Ethiopia

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Jul, 2022

Jimma, Ethiopia

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engineering

Highway engineering stream

Improvement of the expansive soil subgrade strength using quarry dust, saw dust and quarry dust –saw dust mixes

By

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DECLARATION

I, the undersigned person, declare that this Thesis is my original work and has not been presented for a degree award in any other universities.

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The undersigned certify that the thesis entitled: **Improvement of the expansive soil subgrade strength using quarry dust, saw dust and quarry dust–saw dust mixes**, and hereby recommend for acceptance by Jimma University in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Highway Engineering Stream).

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The Thesis has been summited for the examination with my approval as University advisor.

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ABSTRACT

Expansive soils are active clays soil which its volume changes due to variation in moisture content that pose problem to civil engineering structures. Therefore the aim of this study is to improving the strength of subgrade expansive soil using local material or easily available material of quarry dust and saw dust mixes. The expansive soil sample from Jimma Town around teknik sefer, with addition of quarry dust alone, saw dust alone, and combination of quarry dust and saw dust by varying content of stabilizers in steeped concentration of 0%, 10%, 10%, 20%, 20%, 10% + 10% and 20% + 20% each by dry weight of the soil, was used to treat the soil. The laboratory tests are index tests, strength testes and swelling tests. The preliminary investigation of the soil shows that it belongs to A-7-5 class of soil. Soil under this class was poor for sub grade construction. The quarry dust and saw dust addition to expansive soil subgrade soil improves plasticity index and reduce plastic deformations. The soil bearing ratio (CBR) continuously increases from 1.36 to 5.6% with quarry dust and saw dust dosages of 0-20%. And 20% quarry dust and 20% saw dust selected as optimum additive to improve and make the Jimma Town expansive soil suitable for subgrade soil stabilization. The liquid limit of the natural soil initially 105% and when in combination with 10% quarry dust decrease to 96% and at 10% saw dust 98%. When mixing both agents 10% QD + 10% SD and 20% QD + 20% SD with natural soil was further decreased from 98% to 93%. The plasticity index of the natural soil continuously decreases from 34% to 29% with addition of 20% QD + 20% SD. The soaked CBR values of mechanically modified soil sample blended with 0%, 10%, 10%+10%, 20% and 20% + 20% of quarry dust and saw dust are 1.36%, 1.58%, 1.87%, 1.71%, 2.26% and 4.77% and 5.6% respectively. Hence, 20% of quarry dust and saw dust was optimum content that makes the expansive subgrade soil strength stabilized suitable for subgrade construction. The strength of the natural soil is increase from 1.36 to 5.6 (S1 to S3) and the soil class is also turn from poor to fair which is applicable for subgrade material. Further investigations of flied study will be necessary for performance and economic comparative of quarry dust and saw dust with other agents.

KEYWORDS: *Expansive soil stabilization, quarry dust, saw dust, test engineering properties, CBR.*

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Acronyms

AASHTO	American Association of Highway and Transportation Officials
AACRA	Addis Ababa City Road Authority
AADT	Average Annual Traffic
ANSS	Anyway Natural Soil Stabilizer
ASTM	American Society for testing and Materials
CBR	California Bearing Ratio
CS	Clay sand
ERA	Ethiopian Road Authority
GI	Group Index of AASHTO soil classification
GRT	Global Road Technologies
LGC	Light grey clay
LL	Liquid Limit
OMC	Optimum Moisture Content
MDD	Maximum Dry Density
NS	Natural Soil
PI	Plastic Index
PL	Plastic Limit
QD	Quarry dust
SD	Saw Dust
USCS	Unified Soil Classification System

CHAPTER ONE

1. INTRODUCTION

1.1. General Background

The problems caused by issues related to expansive soil in civil engineering structures were first identified in the late [1]. Since then many countries have been reporting negative consequences of expansive soil. Expansive clay soils have a world-wide distribution; their occurrence is not climatic specific though they are particularly widespread in arid to semi-arid climate and are problematic to engineering structures because of their tendency to heave during wet season and shrink during dry season. Although the extent and range of distribution of this problematic soil have not been studied thoroughly, expansive soil is known to be widely spread in Ethiopia [2].

Expansive soil covers large portion of Ethiopia, covering nearly 40% surface area of the Country [3]. Especially Jimma zone are mainly covered with expansive and black cotton soils which has surface and subsurface water which mostly encloses the flat area. For this reason, constructions could be sensitive for structural failure as a result of excessive consolidation settlement [4].

Expansive soils because more damage to structures, particularly lighter buildings and pavements, than any other natural hazard, including earthquakes and floods [5]. In Ethiopia, there are several roads, whose premature failures attributed to the volumetric changes of expansive clay soil; Modjo-Ejerie-Areti Road and Addis-Jimma Road could be examples of such failures. Different methods have been conducted to enhance and treat the geotechnical properties of the Expansive soils (such as strength and the stiffness) by treating it in situ.

Jimma City soils are problematic soils that need treatment. In order to manage this problem, this research was done to evaluate the strength of Jimma City Expansive soil which is stabilize with Quarry dust and Saw dust as admixture. The use of Quarry dust and Saw dust stabilizers increases the strength of expansive soil by filling void space of soil particles and reducing plasticity index. In this research the proper stabilizer type and the amount of stabilizer ratio to be used in future construction on Jimma City expansive clay soil were analyzed.

The laboratory results as well as the statistical analysis from this study will be expected to be useful in designing better sub-grade of road pavements. Soil stabilization is carried out by various methods and one of them is mixing with the Stabilizing Additives such as Quarry dust and Saw dust. Exploration of coal in Ethiopia was started in the year 1935 at Nejo and Wuchale; however, it is not put into commercial application [6].

The present worldwide energy crisis and increase in chemical fertilizer prices initiated the country to explore its own natural resources and promising reserves of coal have been explored in different parts of the country and now the government and many private investors are trying to use indigenous coal for different applications [7].

The major Applications which have got greater attention are the utilization of coal as a raw material for the production of fertilizer as source of energy for power and heat generation, especially in cement industries and other chemical industries. Widespread and concentrated studies have been carried out at various scales from inspection to detailed mapping and preliminary to detailed drilling networks to determine the amount of coal deposit in Ethiopia.

Saw dust or wood dust is a by-product of cutting, grinding, drilling, sanding, or otherwise pulverizing wood or any other material with a saw or other tool; it is composed of fine particles of wood. It is also the by-product of certain animals, birds and insects which live in wood, such as the woodpecker and carpenter ant. It can present a hazard in manufacturing industries, especially in terms of its flammability. Billions of dollars in damages are attributed to expansive soils in many countries each year. Geotechnical design and analyses in/on/with expansive soils may involve additional complications that otherwise would not have to be dealt with if expansive soils were not present. Traditional methods for chemical stabilization of expansive soils include the addition of lime, class-C or class-F fly ash, Portland cement, or other industrial byproducts such as cement kiln dust, steel or copper slag. These methods include stabilization with chemical additives, rewetting, soil replacement, compaction control, moisture control, surcharge loading, and thermal methods. All these methods may have the disadvantages of being ineffective and expensive. [8]

Taking these into consideration, the aim of this research was to establish the effects of alone Quarry dust, alone Saw dust and Quarry dust-Saw dust combination on expansive clay soils to reduce the cost of road construction as well as reducing the environmental hazard molasses causes.

1.2 Statement of the problem

Engineering problems related to expansive soils have been reported in many countries of the world. Expansive soil problems are the result of its changing in volume upon wetting and drying and present in its natural state at construction site may not have the suitable strength. It may have swell and shrinkage distinctiveness and causes significant damage to pavement structures.

This damage can be attributed to moisture fluctuations caused by seasonal variations. Volumetric changes weaken the subgrade by inducing -cracking which meets out damage to the overlying structures. For imparting high amount of strength and stability soil thus needs to be stabilized. [9].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. In the United States alone, expansive clays have been estimated to produce at least two billion dollars of damage annually. the above problems are extensively occurring in Ethiopia.

The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres [10].

As a result, Pavement failure in Ethiopia is becoming a common problem and great challenge, consuming a lot of money. According to [10] Expansive clay soil is available in different parts of Ethiopia. Since most soils which is found in Jimma City have high plastic index and low CBR value. These soils are a consequence for expansive and unstable subgrade soil. As a result, they make pavement structure failure.

Expansive soils, which swell on absorption of water and shrink on removal, because severe damage to structures if no measures are adopted beforehand. the aim of this study was utilization of Quarry dust and saw dust as stabilizer to improve the properties of expansive soils for subgrade. One method to ensure that existing natural soil improved and suitable for construction is by mixing it with QD and SD dust as a cost effective stabilizer

1.3 Significance of the Study

This study is important to solve the problems and prospects of expansive soil subgrade. It is pertinent to the role of subgrade soil for all construction type. It is expected that the study would investigate the problem of expansive soil in highway construction process .It is also hoped that the study would be able to solve the factors influencing in the subgrade soil use additive material quarry dust and saw dust. Finally, to improve the engineering properties of expansive soils, including compaction behavior, bearing capacity, CBR, Atterberg limits, Grain size analysis, Strength.

1.4 Research Questions

The research questions are:

1. What are the engineering properties of existing sub grade soil in the study area?
2. What are the effects of the stabilizer on the expansive soil and optimum percentage of quarry dust and saw dust?
3. How the results compared and recommended its applicability?

1.5 Objective

1.5.1 General Objective

The purpose of the study is to improve the expansive soil subgrade strength in Jimma city using quarry dust, saw dust and quarry dust –saw dust mixes.

1.5.2 Specified Objectives

The specific objectives of this Study are

- to examine the engineering properties of sub grade soil in the study area;
- to determine the optimum amount of stabilizing agent using different proportions of quarry dust and saw dust;
- to compare and recommend the result applicability.

1.6 Scope and Limitation of the Study

The scope of the research was to improvement laboratory strength of expansive soilfrom Jimma City with the addition of quarry dust alone from Ana O/Nada, saw dust alone from Jimma City around Woha Fasash and quarry dust and saw dust combination of various content of the additives.The results are also specific to the type of additive /quarry dust, saw dust and quarry dust-saw dust mixes /used and test procedures that have been adopted in the experimental work. Therefore, findings should be considered the effective of those additive on the strength of sub-grade soil engineering properties (Atterberg limit,Compaction,specific gravity,Linear shrinkage,Free swell and Soaked CBR).

However, the findings of the research are limited to one soil sample considered in this research. Finally mechanism of soil stabilization with quarry dust and saw dust combination is relatively new concept and literatures are scanty in the area. Therefore, it is strongly recommended that the results and findings of the present study must be considered as a complete only for quarry dust and saw dust.

1.7 Organization of the Thesis

The presentation of this thesis work is organized in five Chapters. The first Chapter gives a brief description of the thesis background, objectives, scope and methodology employed. Chapter two presents literature review on expansive soils, the material of Quarry dust and Saw dust additive respectively. Important details from previous studies are also included in this Chapter. Chapter three, briefly discuses about the materials for study and study area. The fourth Chapter deals with the characterization of samples used for the study, laboratory testing procedures sequenced and the test results obtained; analysis of results and discussions of results with respect to the theoretical background and findings of previous studies. Chapter five presents conclusions and recommendations drawn from this research.

CHAPTER TWO

2. LITERATURES REVIEW

2.1 Background

Expansive soil as road subgrade is considered one of the most common causes of pavement distresses. Longitudinal cracking results from the volumetric change of the expansive subgrade, is one of the most common distresses form in low volume roads. This type of cracking is initiated from the drying highly plastic subgrade (PI>35) through the pavement structure during the summer [11].

Expansive soils are residual deposits formed from basaltic and sedimentary rocks. Expansive soils of different origin and location have common characteristics such as high clay content with appreciable plasticity, dark or gray color and tendency to expand as result of moisture increase and shrink due to loss of moisture [12] .

According to [13] the constituents of the parent material during the early and intermediate stages of the weathering process determine the type of the clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long period of time. The parent materials that can be associated with expansive soils are categorized into two groups. The first group comprises the basic igneous rocks and the second group comprises sedimentary rocks that contain montmorillonite as a constituent, which breaks down physically to form expansive soils. Similarly regional states such as Mekelle, Gambella, and the most southern, southwestern and south eastern parts of Addis Ababa are found to be covered with expansive soils.

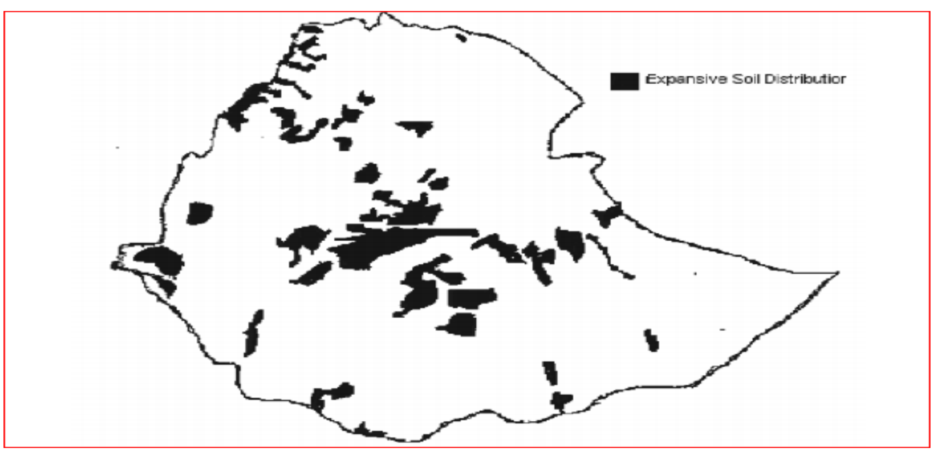
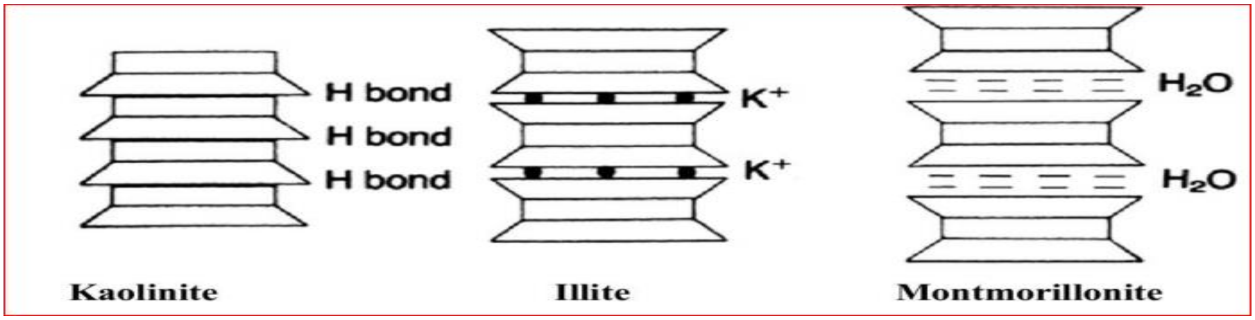


Fig. 2.1 Expansive soil distribution in Ethiopia [14].

2.2 Mineralogy of Expansive Soils

Expansiveness of soils is due to the presence of clay minerals. Clay particles have sizes of 0.002 mm or less. However, according to [15] the grain size alone does not determine clay minerals and the most important property of fine grained soils is their mineralogical composition. Clay minerals are crystalline hydrous alumina-silicates derived from parent rock by weathering. The basic building blocks of clay minerals are the silica tetrahedron and the alumina octahedron and combine into tetrahedral and octahedral sheets to form the various types of clays.



Source: Schematic representation of clay minerals [16].

Kaolinite is a typical two layered mineral having a tetrahedral and an octahedral sheet joined to form 1 to 1 layer structure held by a relatively strong hydrogen bond. Kaolinite does not absorb water and hence does not expand when it comes in contact with water. The montmorillonite groups of clay minerals have 2 to 1 layer structure formed by an octahedron sandwiches between two tetra herons [13].

As Nelson concluded these clay groups have significant amount of magnesium and iron sandwiched into the octahedral layers. The most important aspect of the montmorillonite clay mineralogy group is the ability for water molecules to be absorbed between the layers, causing the volume of the minerals to increase when they come in contact with water.The Elite clay minerals have a structure similar to that of kaolinite, but are typically deficient in alkalis, with less aluminum substitution for silicon, magnesium and calcium can also sometimes substitute for potassium and elites are non-expanding type of clay minerals [16].

2.3 Identification of Expansive Soils

2.3.1 Field identification

In the field expansive soils can be identified by applying several identification techniques. Some of the important field identification methods used to indicate potential of expansiveness of soils when wet sticky and difficult to clean the soil from hand and when dry very hard like rock.

They usually have black or grey color. Cracks are observed on nearby light weight structures such as houses and fences. The typical features and common information of expansive soils identified during field reconnaissance surveys:

Table 2.1: Typical features of expansive soils [17]

Descriptions	Typical features of expansive soil
Soil type	More clay soils are likely to be expansive
Consistency when slightly most to dry	Stiff to very stiff
Consistency when wet	Soft to firm and sticky
Structure	Typical cracking surface, slick-sided fissures
Color	Only a reliable indicator when combined with local

2.3.2 Laboratory identification

In the laboratory several methods are developed to identify expansive soils such as mineralogical Identification, direct and indirect methods as described here under [15]. Expansiveness of a soil is governed by the type and proportion of clay minerals it contains. Knowing the type and proportion of the clay mineral in a soil gives a clue on the swelling Potential. The swell shrink behavior of expansive soils depends on the type of minerals present within the clay. The clay mineralogy of expansive soils can be identified in the laboratory by Applying tests such as X-ray diffraction, electron microscope and differential thermal analysis.

The swelling pressure and volume changes of soils are measured directly using representative undisturbed samples. The swelling pressure is determined by measuring the pressure needed to prevent heaving of sample under the given condition of moisture, density and confinement. Swelling tests provide complete swelling but due to varying initial conditions of moisture, Density, etc. it is difficult to assess the swelling expected in the field. The methods provide Quantitative information, which are very useful for design engineers. These are simple and more practical methods to identify expansive soils.

The indirect tests conducted include the Atterberg limits and grain size distribution which help determining the activity of clay (the ratio of plasticity index) to the percentage of clay fractions finer than 2µm sieve size) present in the sample and degree of expansiveness. According to the [18], all Greyish and/or brownish clays in Ethiopia with plasticity indices > 25% can be Identified as expansive soils. The classification or rating from low potential to high heaven Potential usually depends on the clay content and plasticity.

2.4 Characteristics of Expansive Soils

According to [19], These soils are easily compressible when wet and possesses a tendency to heave during wet condition and shrink in volume and develop cracks during dry seasons of a year and they show extreme hardness and cracks when they are in dry condition. The seasonal change in volume of expansive soils is manifested by both horizontal and vertical movements, the horizontal movement leads to fissure opening during dry seasons and closing during wet seasons whereas the vertical movement leads to cyclic changes in levels.

The magnitude of these movements decrease with depth where there is no seasonal moisture changes. According to [20], about 40 to 60% of expansive soils have grain sizes less than 0.001mm. These soils generally have higher liquid limit and plasticity index and extremely low CBR values. At their liquid limit, the volume change is of the order of 200 to 300% and results in swelling pressure as high as 8 kg/cm2 to 10 kg/cm2. Soaked laboratory CBR values of expansive soils are generally found to be in the range of 2 to 4%. Due to very low CBR values, highly exaggerated pavement thickness is required for designing flexible pavement which leads to extremely high project cost estimates.

2.5 Classification of Expansive Soils

Expansive soils are classified by measuring their swelling potential which can be measured directly in the laboratory or indirectly by correlating with other test results of swell test data. According to Bureau of reclamation method, based on direct correlation of observed volume change with colloid content, plastic index and shrinkage limit, expansive soils can be grouped according to their degree of expansiveness as shown in the Table below.

Table 2.2: Classification of expansive soils based on Bureau of reclamation method [21]

Colloid content (%)	PI, (%)	SL, (%)	Probable expansion, (%)	Degree of expansion
<15	<18	>15	<10	Low
13-23	15-18	10-16	10-20	Medium
20-31	25-41	7-12	20-30	High
>28	>35	<11	>30	Very high

According to ERA low volume pavement manual specification it is not allowed to use CBR values less than 3%, because from both a technical and economic perspective it would normally be in appropriate to lay a pavement on soils of such bearing capacity.According to AACRA design manual suitability of sub-grade soils based on CBR values are :

Table 2.3: Suitability sub-grade materials based on CBR values Adapted from [52].

Soil class based on USCS	Typical design CBR (%)	Suistability
GW	40-80	Good to excellaent
GP	30-60	Good to excellaent
GM	40-60	Good
GC	20-30	Good
SW	20-40	Good
SP	10-40	Faire to poor
SM	15-40	Faire to poor
SC	4-20	Poor to faire
ML	$\leq$ 15	Poor to faire
CL	$\leq$ 15	Poor to faire
OL	$\leq$ 5	Poor
MH	$\leq$ 15	Poor
CH	$\leq$ 15	Poor to faire
CH	$\leq$ 5	Poor

Based on the suitability of sub-grade soils the studied soils were classified under Poor and not allowed to use for sub-grade materials. But, the stabilized soils at optimum ratio were classified under fair to good for sub-grade materials.

Small specimens were taken from the closing groove and placed in empty cans and kept in the oven for 24 hours at 110 ±5 Cº to calculate the water content.

Water content % = $(\frac{Mass\ of\ water}{Mass\ of\ dry\ soil})$ … (1)

Plastic limit (PL) = 100× $(\frac{Mass\ of\ water}{Mass\ of\ dry\ soil})$ … … … … … … … … … … … … … … … ….................….. (2)

Plasticity index (PI) = $(LL - PL)$………………..……………………….…………………… (3)

The Atterberg limit tests consist of Liquid limit Plastic limit and Plasticity index. The test procedures are outlined in AASHTO materials testing Manual. The liquid limit values are determined in accordance with (AASHTO T089-94, T090-96) similarly the plastic limit and plasticity index of the soil samples are determined in accordance with (AASHTO T089-94, T090-96).

Liquid limit**:** The boundary between the liquid and plastic states; The water content at which the soil has such small shear strength that it flows to close a groove of standard width when jarred in a specified manner. The container was covered with plastic wrap to avoid moisture loss. After moisture conditioning, researchers performed the liquid limit test as *per ASTM D4318.*

Plastic limit: The boundary between the plastic and semi-solid states; the water content at which the soil begins to crumble when rolled into threads of specified size. Soil sample from the liquid limit test was used for the plastic limit. The test was performed as per ASTM D4318.

Shrinkage limit: The Linear Shrinkage Limit test was performed with approximately one hundred and fifty grams of soil. A third of the soil was placed in a greased brass mold approximately 140 mm long and 25mm in diameter.

The soil was placed in the mold in three layers and tapped against a flat surface in between the layering to remove air bubbles from the soil. The sample was allowed to air dry for four hours. Then the soil sample was placed in an oven at 105˚C for 18 hours. After the soil was dry, the mold was removed from the oven and allowed to cool. to calculate linear shrinkage using the equation:

$LS = (1 - \frac{Lave}{Lo})$×100……………………………………………………………………… (3)

Linear shrinkage = [1- (Length of oven dried Specimen / Initial length of specimen)] × 100

Where, LS = Linear Shrinkage (%); L_{avg} = Average Length (mm); and Lo = Original Length of Brass mold (mm).

Standard test for Specific Gravity of Soil Solids by water Pycnometer.

$$Specific\ Gravity\ , Gs = W_0 + \left(\frac{W_0}{\mathrm{WA-WB}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (4)$$

Where, W0= weight of sample of oven-dry soil; W_A = weight of pycnometer filled with water and W_B= weight of pycnometer filled with water and soil.

$$Free\ swell\ index\ percen = \left(\frac{\mathrm{Vd - Vk}}{Vk * 100}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (5)$$

Where, Vd = The volume of soil specimen read from the graduated cylinder containing distilled water and Vk = the volume of soil specimen read from the graduated cylinder containing kerosene.

2.6 Problems Associated with Expansive Soils

According to [22], most of the problems associated with expansive soils arise mostly from the nature of the soil itself and drainage facilities provided. As a result of their low CBR and strength, expansive soils fail to support the loads transmitted from the pavement structure and cause excessive deformation beyond permissible limits. The common problems associated with expansive soils are described below.

2.6.1 Volume change

Expansive soils have tendency to heave during wet condition, shrink and develop cracks during dry seasons which makes expansive soils a problem to road pavements. The cracks developed during dry seasons allow water to penetrate deep in to the soil during rainy seasons, hence causing considerable heave and expansion. This results in deformation of road surface constructed on expansive soils as the expansion and the subsequent heave are never uniform. Furthermore, the shrink-swell behavior of expansive soils may lead to lateral displacements/ creep of the pavement layer on expansive soils, if the side slopes are not gentle enough.

During seasonal change (rainy and dry seasons), the road edges get wet at faster rate than the surfacing of the road. This results in differential movements over the road cross section and associated crack development, first occurring in the shoulder areas and developing to carriageways.

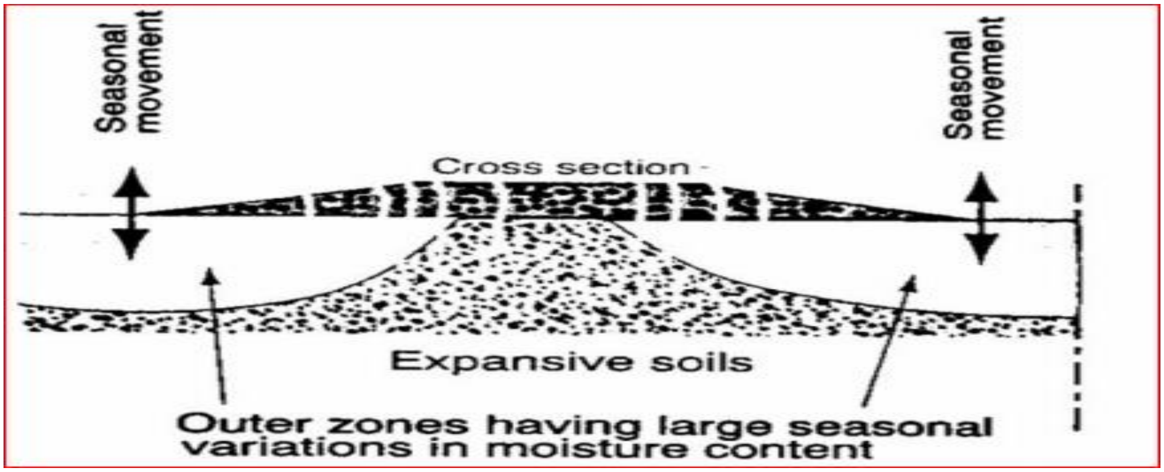


Fig. 2.2 Moisture content in expansive soils [23]

2.6.2 Bearing capacity

When the moisture content increases, expansive soils swell and become loose and hence their bearing capacity reduces dramatically. If the soil becomes fully saturated, the CBR value reduces to a value under 2% which makes such soils unsuitable to be used as road subgrade material.

2.6.3 Erosion

In dry state expansive soils exhibit sand like structure where they are prone to erosion to a much greater extent than that normally anticipated from their plasticity and clay content.

2.7 Design and Construction Considerations in Expansive Soils

In road projects if expansive soils are encountered, the measures proposed to deal with them should be economically reasonable and proportionate to the risks of potential pavement damages and increased maintenance costs.

2.8 Mitigation Measures on Expansive Soils

Expansive soils do not meet the specification requirements of many standards including the Ethiopian Roads Authority Standard Technical Specifications [24]. According to ERA Site Investigation Manual -200 (Special Investigation), whenever expansive soils are encountered during the design or construction phase of a road project, the following mitigation measures are recommended.

2.8.1 Realignment

Realignment is recommended and possible if the areas covered with expansive soils is of limited extent. When the coverage of the expansive soil is of limited extent rather than going for treatment or removal of the problematic section, realignment can be effective and economical.

2.8.2 Excavation and Replacement

This is mostly recommended measure as the problematic soils are completely removed and replaced by selected suitable material. However, these measures are only economically viable if the selected borrow material is found in the project vicinity. The back fill materials should have CBR values similar to that of the over laying embankment materials (CBR > 5%, i.e. subgrade strength class S3) and should not pervious in order not act as drain.

Table 2.4: Suggested treatment depths below the normal subgrade level of high swelling potential soils [25]

Highly Trafficked (Primary Roads)		Lighty Trafficked (Secondary Roads)	
Plasticity Index	Depth of Treatment (m)	Plasticity Index	Depth of Treatment (m)
10-20	0.6	10-30	0.6
20-30	0.9	30-50	0.9
30-40	1.2	* > 50	1.2
40-50	1.5		
* > 50	1.8		

*Excavate and waste, replace with selected impermeable material

2.8.3 Soil Treatment/Modification

The problematic nature of expansive soils can be improved by applying several treatment measures. Some of the treatment methods developed and being applied in road construction projects include stabilizing by using stabilizing agents such as lime, cement, bitumen and chemicals. Similarly, in road projects with expansive soils and scarce suitable borrow materials, the expansive soil itself can be used as fill material by providing protective ‘’blankets’’ and applying proper drainage as which is practiced in Adura-Akobo road project under administration of the Ethiopian roads Authority (ERA). As per specification requirements of several standards, separate treatment methods are applied on expansive subgrade soils for different road classes depending on their AADT.

Accordingly, for AADT design greater than 50 of higher traffic class roads, the following treatment measures can be applied [26].A) Removal of the expansive soil i. Where the finished road level is designed to be less than 2m above ground level, remove the expansive soil to a minimum depth of 0.6m over the full width of the road, or

ii. Where the finished road level is designed to be greater than 2m above ground Level, remove the expansive soil to a depth of 0.6m below the ground level under the un-surfaced area of the road structure, or iii. Where the expansive soil does not exceed 1m in depth, remove it to its full depth) Stockpile the excavated material on the either side of the excavation for subsequent spreading on the fill slopes so as to produce a flat slope as possible. C) The excavation formed as directed in paragraph (A) should be backfilled with a plastic Non-expansive soil of CBR value of 3-4 or better, and compacted to a density of 95% of Modified AASHTO density. D) After the excavated material has been replaced with non-expansive material, in 150mm Limits to 95% of modified AASHTO density, bring the road to finished level in approved Materials, with a side slope of 1:2, and ensure that pavement criteria are compiled with the previously stockpiled expansive soil excavated as directed under (A) should then be spread over the slope.

E) Do not construct side drains unless they are absolutely essential to stop ponding; where side drains are necessary, they should be as shallow as possible and located as far from the toe of the fill as possible. F) Ideally, construction over expansive soils should be done when the in-situ moisture content s at its highest (i.e. at the end of rainy season) Similarly, for light traffic road classes of AADT design less than 50, [27] recommends the following treatment methods to be applied on expansive soil sections of a subgrade.

A) Remove 150mm of the expansive top soil and stockpile conveniently for subsequent use on shoulder slopes. B) Shape road bed and compact to 90% of modified AASHTO density. C) The excavation formed as directed in (A) should be backfilled with a plastic no expansive soil of CBR value of 3-4 or better, and compacted to a density of 95% of Modified AASHTO density in each 150mm layer; the subgrade material may be plastic but non expansive.

The CBR was determined by a load penetration test in the laboratory and is mostly used for the design of roadway pavements. The strength of natural ground soil is determined using the CBR test and assigned an index on a scale from 0 to 100 relative to a designated top-quality granular based material. The three-point CBR tests at 10, 30, and 65 blows were conducted according to (AASHTO T 193-93). In the soaked state, and the CBR at 95% MDD was determined. Soaking of CBR specimen prior to penetration is used to simulate the field condition of the worst condition because of high rain fall and complete saturation of the entire soil mass due to presence of near ground water table.

However, in our country Ethiopia in most part of the country, the ground water table is much lower than the annual moisture fluctuation depth (commonly 2-3m) and the annual rain fall intensity is not as high as the worst condition considered for CBR test. Thus, in this study the un-soaked conditions of CBR test are conducted on the natural subgrade soil and the marble dust blended samples to compare the result with the soaked case of the same.

Table 2.5: Rating of materials based on their CBR value [41]

CBR (%)	General rating	To be used as:
0-3	Very poor	Not recommended
3-7	Poor to fair	Sub-grade
7-20	Fair	Sub-base
20-50	Good	Base coarse/sub-base
>50	Excellent	Base coarse

Lime treatment is costly as substantial thickness of soil (at least 30cm compacted thickness) has to be prepared. This treatment can be considered advantageous only if when it is difficult to get selected backfill material near the project and when pavement savings can be made by taking advantage of the enhanced strength of the treated clay.

iv. Minimizing Moisture Changes and Consequent Movements. If there is shortage of selected borrow material in the project vicinity and if Lime treatment is found to be costly, the existing expansive soil can be used for fill and subgrade. To apply this option, the moisture change has to be controlled by employing mechanisms such as; a) Confining expansive soils under impervious subgrades and protective blankets Selected materials placed over weak subgrades reduces sub-base thickness and hence pavement thickness. Selected materials placed over expansive soils protect them from moisture changes.

It is recommended that selected materials should be at least 30cm of thickness and should be relatively impervious. Expansive soils can be used to form shallow embankments (up to about 3m), provided that a protective blanket of thickness at least 30cm is provided on the slopes.

The blanketing material should be at least conducive to a subgrade strength class of S3 quality and be impermeable and Resistant to erosion) Surcharging Expansive Soils:-When non-swelling materials are placed over them expansive soils reduce in heave. The minimum thickness required depends on the expansion pressure of the swelling soil, but it is commonly recommended to use 1-3m to have significant swell reduction. It is therefore possible to use expansive soil to form the lower part of an embankment.

The total thickness of pavement plus improved subgrade should be at least 60cm thick, irrespective of the other protective measures taken. c) Limiting the compaction of expansive clays:-Expansion pressure and potential volume change increase significantly with the dry density of swelling soils. High degree of compaction may therefore be detrimental and should be avoided. It is recommended that the dry density of expansive soils in no case exceeds 95% MDD.

d) Placing expansive soils at equilibrium moisture content:-If possible, the equilibrium moisture content should be measured under existing roads of the project area. Otherwise, it can be assumed that the equilibrium moisture content is near the plastic limit. This applies in areas where the mean annual rainfall exceeds 500mm and the water table is non-existent or deep (grater that 5-6m).

In arid areas or in case of a water table close to the ground level, a special study may be required to determine the equilibrium moisture content. e) Preventing moisture change under the pavement:-To control moisture change in swelling soils caused whether by external water or internal variation, the following measures can be adopted;

3.7.6. Potential Swell

The volume change/swell-shrink of expansive soils as result of moisture change is one of the significant identification features. The potential swell of expansive soils is important parameter to classify subgrade soils based on their expansiveness. Potential swell can be measured directly or indirectly. The swells of marble dust blended expansive soil samples are measured and determined from the CBR mold before and after soaking.

i. Direct measurement of swell

The direct measurement of swell can be done by laboratory free swell method (Odometer test) and constant volume swell test. Swell from CBR mold for the soaked case of CBR test, the volume change of the compacted specimen is measured before and after soaking using dial gauge. The calculated swell value is showed in the table below; as per several specification requirements such as [47], a subgrade soil should have a maximum swell of 2%. Thus, the natural soil of the subject study area has high swelling potential and cannot be used as subgrade material.

ii. Indirect measurement of swell

The indirect measurements involve the use of soil properties and classification schemes to estimate swelling potential. Some of indirect measurement methods of swell potential are described below;

a. Potential Swell based on Plasticity

The plot of plasticity index against liquid limit helps to detect the swelling potential of soils by looking at where the soil samples fall in the chart. The swelling potential for any given plasticity index and liquid limit is indicated.

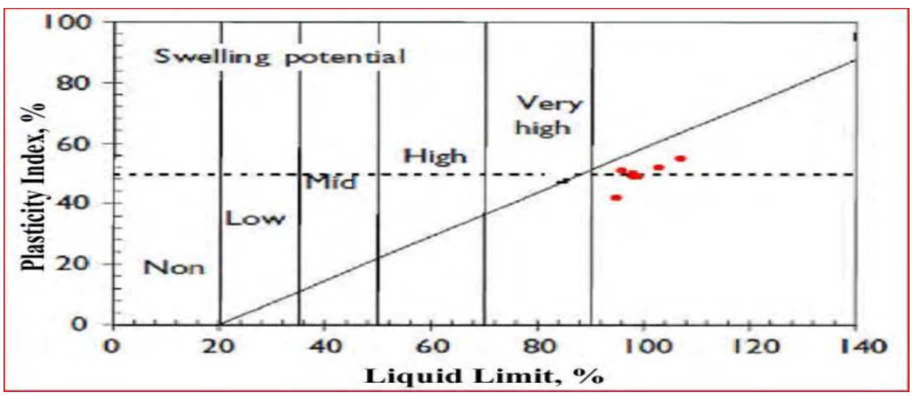


Fig. 3.10 Potential Swell of soil sample based on plasticity chart [48].

b. Relation between swelling potential and plasticity index (PI)

Table 2.6: Relationship between PI and swelling potential [42]

Plasticity Index(PI)	Swelling Potential
0-15	Low
10-35	Medium
20-55	High
>55	Very high

2.9 Soil Stabilization

Soil stabilization method is widely used to improve soil strength and decrease its compressibility through bonding the soil particles together. Additives or grout are mixed with soil to bring about the stabilizing action required. In road projects, all the naturally available material cannot be utilized as construction material as there exists some problematic soils (such as expansive soils) and soils with limitations to meet specifications and design standards.

The problematic nature and limitations of such soils can be improved by application of stabilizing agents. With weak subgrades, it is common practice to provide capping layers between the subgrade and the sub-base. The capping layer is of granular material of less quality of the specification requirement for sub-base material.

As alternative to provision of capping layer of imported granular material, subgrade soil stabilization using different stabilizers such as lime, cement and quarry dust has comparative advantage with respect to environmental protection and economic advantage in areas where the granular materials are scarce. Starting from early human history, stabilizing agents such as lime and cement were used for construction of pyramids, bridges, dames and roads being mixed with soil. Early in the 20th Century, the Americans introduced the scientific way of stabilizing soils in road projects to improve their engineering properties by application of stabilizing agents. The application of stabilizing agents can improve [28].

2.9.1 Techniques of Stabilization

Most of researches investigated the effect of using different soil improvement techniques on increasing soil bearing capacity and /or decreasing the expected settlement while, a few researches considered the cost of foundation works as one of the governing factors when selecting between different soil improvement techniques.

Stabilization by Compaction

Loose materials can be made more stable simply by application of compaction. Though, compaction cannot be considered as stabilization process, it plays a fundamental role in the properties of stabilized materials.

Mechanical Stabilization

Mechanical stabilization is a process by which the gradation of soil is improved by the incorporation of another material which affects only the physical properties of the soil. In the case of mechanical stabilization, unlike other stabilizing agents, the proportion of the stabilizing material exceeds 10% and may be as high as 50%.

Stabilization using stabilizing agents

Application of stabilizing agents such as lime, cement and chemical stabilizers in low amount causes significant improvement in engineering properties of expansive soils. Stabilizing expansive soil by adding lime is an ancient art and an age old practice, which has been followed all over the world. When lime is added to clayey soils provides an abundance of calcium magnesium ions, also sodium and potassium present on clay mineral plates. Lime stabilization has proved to be one of the most efficient techniques used to mitigate swell potential. The chemical reaction occurring between lime and soil is quite complex.

Chemical Stabilization

Chemical stabilization of soil is mixing of soil with one or a combination of admixtures of powder, slurry, or liquid for the general objective of improving or controlling its volume stability, strength and stress-strain behavior, permanently and durability [29].

2.9.2 Mode of Stabilization [30]

Stabilization by Physical Reaction

It is the simplest procedure by adding water, bitumen or other bonding materials with the soil. The bitumen is added to the soil in the form of a liquid of low viscosity, which is suitably converted in to a highly viscous semi solid state by reduction in temperature or by evaporation of the solvent. As bitumen is hydrophobic it also works as water proofing agent as well.

Stabilization by Reaction between Two or More Chemicals

Most of the stabilizing agents in this category are bonding agents. They form the solid by the addition of two or more chemicals, which in themselves are not stabilizing agents but react by precipitation or polymerization is the formation of calcium silicate in a soil by the reaction of sodium silicate and calcium chloride.

Stabilization by Chemical Reaction between the Soil and Stabilizer

Hydrated lime is the best example of a bonding agent in this group; the strength being derived from the reaction between lime and the clay fraction of the soil. Once attached, they are difficult to displace and since each particle is in effect surrounded by hydrophobic cations, the soil then is made water proof.

2.10 Previous similar works

2.10.1 Stabilizations of Expansive Soils by using quarry dust

Quarry dust is a filler material from the quarrying industry as the result of crashing activities and is mainly composed of calcium carbonate $CaCO_3$. However, it is defined as "the inherent fraction of aggregates passing 0.063 mm (63 microns)", and the production process of quarry dust is described as the secondary result from blasting, processing, handling, and transportation of aggregates in quarries, and it is figured that the majority if quarry dust is produced during crushing, milling, and screening of the quarried rocks [31]. Stabilized expansive soil using quarry dust and lime for strengthening the subgrade of a rural road for low volume traffic. The properties tested were compaction (standard proctor), UCS, soaked CBR and Ps. The stabilizer strengthened road was found to be cost effective for low volume traffic [32].

Investigated the effect of lime on Atterberg's limit, compaction (modified proctor), shear strength parameters and durability of an expansive soil stabilized with optimum percentage of quarry dust (40%).The lime added were 2 to 7 % at an increment of 1%. The effect of 7 and 28 days of curing were also studied on shear strength parameters.

had developed statistical models for prediction of Ps of expansive soil (Bentonite) stabilized with quarry dust and lime by correlating the percentage of stabilizers, MDD, OMC, curing period and activity. Models were also developed to predict the Ps of stabilized expansive soil cured at 7 and 28 days from the Ps of the expansive soil cured at 0 day and the Ps of expansive soil cured at 28 days from the Ps of the expansive soil cured at 7 days. [33]

[34] Concluded that the strength behavior of the stone dust mixed with any types of soil has improved substantially. The mixed soil is better compactable and lesser compressible in comparison to Non-plastic clay and grading of the soil also improved by addition of 25% stone dust which is presented. For coarse graded granular sub-base material. So the soil mixed with 25% of stone dust can be used for granular sub-base material in road construction work.

[40] conducted studies on those qualities and applications that make quarry dust a good replacement or admixture during soil improvement. When quarry dust is added with expansive soil, it is expected that it will make it more porous, less durable, reduce cohesion, etc., and also quarry dust has rough, sharpened angular particles and as such causes a gain in strength due to better interlocking.

[41]Studied the combined effect of fly ash and quarry dust on compaction characteristics, CBR, shear strength parameters and swelling pressure of an expansive soil. It is seen that maximum dry density, California bearing ratio and angle of internal friction increases and cohesion and optimum moisture content decreases with addition of increased percentage of fly ash – quarry dust mix. The maximum value of unconfined compressive strength is achieved when the fly ash – quarry dust mix is 45%. Crusher dust is mixed with high plastic gravels to reduce the excess deformation of the gravel soils and increase the life period of pavement.

2.10.2 Stabilizations of expansive soils by using saw dust

Research has already been carried out on the use of SDA as partial replacement in concrete No doubt; it has been found out that it can act as a significant pozzolan in concrete. Further, the use of sawdust ash as highway pavement material has been tested and has shown an increase in particle size distribution of lateritic soil and maximum which falls under A-2-7 as per AASHTO classification [35].

Differential free swell also reduce considerably. The index of differential free swell has come down from the range of high to a range of moderate hence the ill effects due to the volume change of the expansive soil will be less.

Soaked CBR value for 15% addition of saw dust increases by 135% of raw soil. Bricks made by mixing saw dust with black cotton soil in certain proportion have shown satisfactory result. From his study it is very clearly that the addition of light weight waste materials like saw dust can improve the behavior of Expansive soil.

[36] Had done the work for improving the properties of black cotton soil mixed with saw dust. The saw dust mixed black cotton soil has minimum increase in liquid limit at 15% of saw dust mix, and has minimum value of plasticity index due to increase in plastic limit. This makes the blended mix less plastic in nature. Sawdust composites have been applied in construction for a long time. For example it has been used to produce sawdust concrete for more than 40 years [37].

Apart from its use in concrete, literature indicates that other sawdust composites used in the construction industry include particleboards, floor panels, partitioning, cladding, ceiling, formwork and concrete blocks and bricks. Sawdust or wood dust is a by-product of cutting, grinding, drilling, sanding, or otherwise pulverizing wood or any other material with a saw or other tool; it is composed of fine particles of wood. It is also the by-product of certain animals, birds and insects which live in wood, such as the woodpecker and carpenter ant.

It can present a hazard in manufacturing industries, especially in terms of its flammability. Sawdust is the main component of particleboard [8].

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Sawdust, a waste production the processing of wood, is an important source of energy for biomass gasifier. The use of sawdust ash as highway pavement material has been tested and has shown an increase in particle size distribution of lateritic soil and maximum which falls under A-2-7 as per AASHTO classification [35].

Geotechnical properties of south-western Nigerian Soil was again tested by [38] who has proved that sawdust ash is an effective soil stabilizer for lateritic soil and road quality can enhanced by its addition to the soil. Sawdust or wood dust is a by-product of cutting, grinding, drilling, sanding, or otherwise pulverizing wood or any other material with a saw or other tool; it is composed of fine particles of wood. It is also the by-product of certain animals, birds and insects which live in wood, such as the woodpecker and carpenter antic can present a hazard in manufacturing industries, especially in terms of its flammability.

Sawdust is the main component of particleboard [40].

Table 2.7: Properties of Saw Dust Chemical Composition

Chemical element present	Percentage Composition
SiO_2	65.472
Al_2O_3	5.69
Fe_2O_3	2.16
CaO	9.82
MgO	4.23
SO_3	1.06
Na_2O	0.04
K2O	2.38
$CaCO_3$	7.89

Source:“Improvement of Mechanical Properties by Waste Saw Dust Addition into Soil” [41]

CHAPTER THREE

3. RESEARCH METHODS AND MATERIALS

3.1 Study Area

The study area of this research was conducted in Jimma zone, Southwestern Ethiopia, which is located 335 km by road southwest of Addis Ababa. Its geographical coordinates are between 7° 13’- 8° 56N latitude and 35°49’-38°38’E longitude with an estimated area of 19,506.24. The town is found in an area of average altitude, of about 1780 m above sea level. It lies in the climatic zone locally known as Woynā Dagā which is considered ideal for agriculture as well as human settlement. The samples use in this study were Expansive soil, quarry dust, saw dust and water. All samples are local available. The experimental test was conducted at Jimma University Highway Material Laboratory.

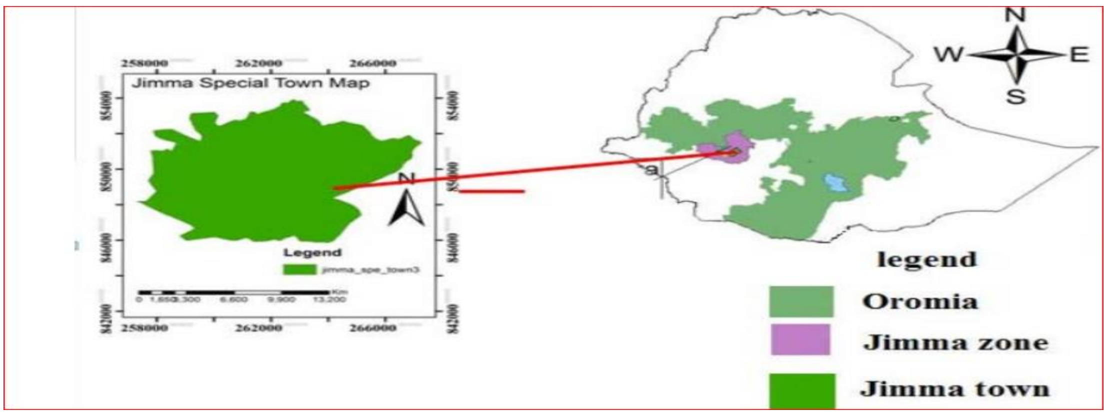

Fig 3.1 Map of the study area. “Source” Geological survey of Ethiopia (GSE, 2012) report

3.2 Study Design and Period

The study was experimental and carried out on strength of expansive soil with and without additives. A laboratory experiment program was designed to conduct all of the fundamental engineering properties of the expansive soil to investigate the engineering properties of the soil. Those engineering properties of the soil were Atterberg Limits, Shrinkage limit and linear shrinkage, Maximum Dry Density (MDD) and Optimum Moisture Content (OMC),California Bearing Ratio (CBR) and CBR Swell, Specific Gravity, Free swell and also result analysis up to conclusion and recommendation were conduct.

In this study both empirical and theoretical research methodologies were employed to attain the objectives of the research. The research has taken eight months and it was started on Aug 2021 and it was ended on May 2022, which was including from data collection up to the final paper submission.

This chapter was presents and describes the approaches and techniques the researcher used to collect data and investigate the research problem and also leads methodology to accomplish the research objectives .The first activity in this research was review literatures related to the research from different sources and references like: text books, research papers, journals, magazine, and web Internet. Then, the material used for stabilization of expansive soil produced should collect from available source and laboratory experimentations have been carried out.

So, in order to get the final results, material like expansive soil, quarry dust and saw dust chemical test of stabilizing expansive soil first making data collection and testing have been performed in a sequence ways.

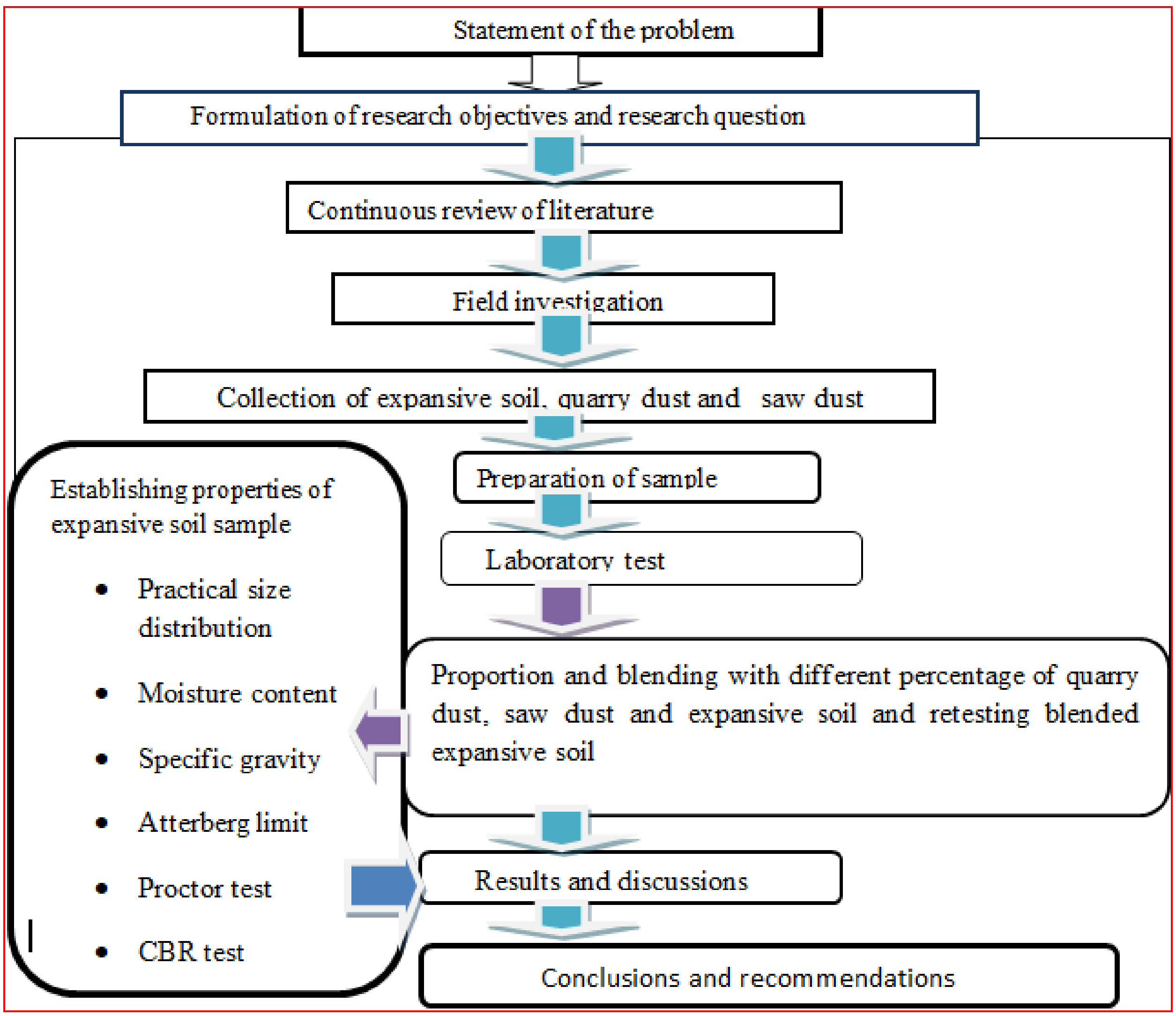

Fig 3.2 Study Layout design

3.3. Study Population

The population of this experimental study was stabilizing the expansive soil with quarry dust, saw dust and quarry dust-saw dust and without those additives.

3.4 Sample Techniques

The sampling technique applied for the collection of sample in the laboratory was quartering method. The samples to use in this research were collected from different area and at different time also in the different techniques. The weights of the samples were 150 kg expansive soil, 100 kg quarry dust and 100 kg saw dust.

3.4.1 Expansive soil

Expansive soil was collected from Jimma City Bacho Bore keble around Technique TVET borrow pit at depth of 2.5m. The samples were excavated manually using picks and spades and the sample soils were transported to laboratories for conducting tests.


Fig. 3.3 Expansive soil and Fig. 3.4 Measurement the depth of Pit

Fig 3.5 Expansive Soil Prepared for Lab Test

I. Initial Moisture Content of the expansive Soil (AASHTO T-265)

The oven-drying method was used to determine the moisture contents of the samples. 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 weighed, 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.

II. Grain Size Distribution (ASTM D422-63)

The sieve and hydrometer analysis tests were conducted to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method was used to determine the distribution of the finer particles. The sample was then washed through a series of sieves (No.4 (4.75 mm) sieve at top and No. 200 sieve at bottom) with progressively smaller screen sizes to determine the percentage of sand-sized particles in the specimens. Approximately 50 grams of dry soil, which was the fine soil from the bottom of the pan of sieve test was treated with a 125 mls dispersing agent (Sodium hex metaphosphate (40gm/L) solution) for 18 hours. A hydrometer analysis was then performed using 152H Hydrometer to measure the amount of silt and clay size particles.

Fig.3.6 Sieve analysis

a. Wet Sieve Analysis

To determine the distribution of coarser particles, 1630g of the natural subgrade soil is taken and washed on sieve size of 75µm and the results shown as Table 3.6 obtained.

Table 3.1: Sieve analyses for the natural soil

Description		NS	
Wt. before wash (g)		1630	
Sieve No.(mm)	Wt. retained (g)	% retained	% pass
4.75	3.30	0.33	96.67
2	3.80	0.38	97.63
0.425	5.00	0.05	93.75
0.075	9.20	0.92	90.75

b. Hydrometer Analysis

To determine the distribution of fine particles (silt and clay) 10 g of air dried soil sample passing sieve 75 µm is used. The soil sample is socked in chemical solution (Sodium hexa-meta phosphate) for 24 hours.

Table 3.2: Hydrometer analysis results for the natural soil

Date of test	28/8/2021 G.C
Hydrometer No.	152H
Specific gravity of soil	2.67
Wt. of soil sample	50 g
Zero correction	+ 4
Meniscus correction	+ 1

III. Atterberg Limits (AASHTO T089-94, T090-96)

Representative samples of each soil were subjected to Atterberg limits testing to determine the plasticity of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 425 µm (No. 40) sieve. The liquid limit was determined as the water content, at which a pat of soil in a standard cup and cut by a groove of standard dimensions flowed together at the base of the groove for a distance of 13 mm (1/2 inch) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit of each soil was determined by using soil passing through a 425 µm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for each soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index were then used to classify each soil.

Three specimens were prepared and about 400 grams of each soil and 10%, and 20% of quarry dust and saw dust by the weight of the soil were prepared and each proportion was mixed with 150 grams of soil.

Fig. 3.7 Atterberg test

IV. Shrinkage Limits

The shrinkage limit, expressed as moisture content in percent, represents the amount of water required just to fill all of the voids of a given cohesive soil at its minimum void ratio obtained by oven drying and used to evaluate the shrinkage potential, crack development potential, and swell potential of cohesive soil. A representative sample of each soil using the material passing through a 425 μm (No. 40) sieve was obtained. Then the moisture-content loss to dry the soil to a constant volume is determined and subtracted from the initial moisture content to calculate the shrinkage limit. The volume of the dry soil pat is determined from its mass in air and its indicated mass when submerged in water. A coating of wax is used to prevent water absorption by the dry soil pat.

Fig.3.8 Shrinkage test

VI. Specific Gravity

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 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.

Fig. 3.9 Specific test

VII. Moisture Density Relations of the subgrade Soils

Modified Proctor Test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the natural soil according to AASHTO T180-95.Modified proctor test. A sufficient quantity of air dried soil were obtained in large mixing pan and pulverized the soil and run it through the No. 4 (4.75mm) sieve and prepare 5 representative samples each about 4000gm for a single Proctor Test using 6 inch mold. Compaction for each portion were done with 4.5 kg hammer falling a distance of 18 inches, and used five equal layers by giving each layer 56 blows. For each proctor test five runs were conducted by increasing water content. These series of determinations were continued until there was either a decrease or no change in the wet unit mass (g/cm3) of the compacted soil.

Fig. 3.10 Moisture test

VIII. Californian Bearing Ratio

One of the major laboratory tests to be conducted on subgrade and other pavement layers of a road is the CBR test. The CBR test results of a soil sample are used to evaluate the strength/ bearing capacity of a pavement layer and to rate the suitability of subgrade soils to carry the overall pavement load.

CBR test is intended for but not limited, evaluating the strength of cohesive materials having maximum particle size less than 19 mm (AASHTO T 193-93).Generally about 10, 30, and 65 blows per layer are suitable for compacting specimens 1, 2, and 3 respectively. More than 65 blows per layer are generally required to mold a CBR specimen to 100% of maximum dry density. Some laboratories may prefer to test only one specimen, which would be compacted to a maximum dry density at optimum moisture content (AASHTO T 193-93). For the present study one point (one specimen) CBR test was conducted with 65 blows for each five layers. Unless specified by the authorized agency, or unless it has no effect on test results for the material being tested, all specimens shall be soaked prior to penetration (AASHTO T 193-93).

The soil is stabilized by adding 10% QD, 10% SD, 10% QD + 10% SD, 20% QD, 20% SD and 20% QD + 20% SD by weight of the soil. The Optimum moisture content, maximum dry density and CBR values for the expansive soil, quarry dust and sawdust mixture was determined in the laboratory for each percentage modified proctor test AASHTO T180-95 and CBR testing Machine. The CBR value was calculated at penetration of 2.54 and 5.08 mm and the higher value was taken. A graph was drawn for penetrations against dry densities in mold and CBR value at 95% of MDD was taken as the design CBR value. The design CBR value is used as the index measurement of soil strength. Table 3.1 presents the general sub-grade strength classes corresponding to ranges of CBR Value. Generally S1 & S2 class sub-grade are rated poor sub-grade soil and sub-grade soil whose CBR swell percent greater than 2 is considered to expansive sub-grade soil according ERA design manual (ERA, 2013).

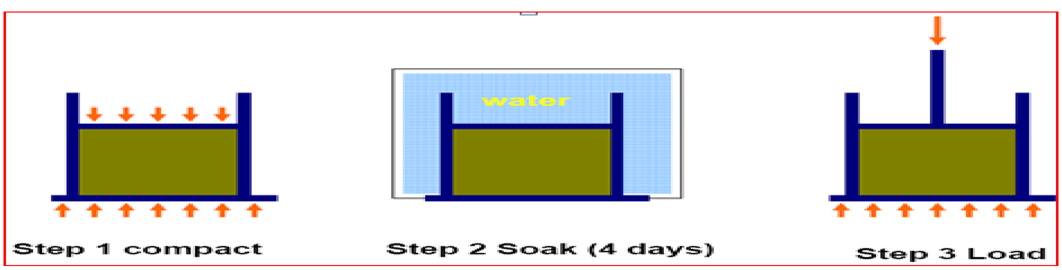

Fig 3.11 CBR Test Step

Fig 3.12 CBR Test Step

Table 3.3: Sub-grade Strength Classes (ERA, 2013)

Sub grade strength class	Range CBR (%)
S1	< 3
S2	3,4
S3	**5,6,7**
S4	8-14
S5	15-29
S6	30+

X. Free Swell

For one free swell test, two 10 gm ovens dry soil passing through a 425 μm (No. 40) sieve was taken. Each soil specimen was poured into each of two glass graduated cylinder of 100ml mark. One cylinder was filled with kerosene oil and the other with distilled water up to 100 ml mark. After removal of entrapped air (by gently shaking and stirring with a glass road), the soils in both the cylinders were allowed to settle for 24 hours. The final volume of the soils in each cylinder was read. The level of the soil in the kerosene graduated cylinder was read as the original volume of the soil. The level of the soil in the distilled water cylinder was read as the free swell level.

Fig. 3.13 Free swell

3.4.2 Saw Dust Additive

The sawdust was collected from wood manufacturing firm found in Jimma City around woha fesash. The wood manufacturing firm used wanz for manufacturing therefore the sawdust used was the byproduct of wanza and it is a hard wood. There are different size of sawdust, courser and finer but in the study sawdust used was finer saw dust.

Fig 3.14 Saw dust sample

3.4.3 Quarry Dust

Quarry dust for this study was collected from a local quarry Jimma Zone O/Nada Ana District.

It is a waste product of quarries readily available and cheap in Jimma zone and also used in very large quantity to increasing the strength of expansive soil as many researcher defined. So, in this study I used as one additive for improvement of expansive soil blended with SD, NS and alone at different percentages.

Fig 3.15 Quarry dust sample

3.5 Methods of data collection

The different activities that carry out in this study work can be classified into different main phases to collected data. Identifying the availability of the additives and site selection, place of Laboratory and functioning of laboratory. Use both primary and secondary data sources. Secondary data need for this research pre-fieldwork have been collecting journals, book, web site etc. and the primary sources fieldwork were collecting expansive soil, Quarry dust and saw dust from different sites and post-fieldwork laboratory experimental outputs, data processing and analyzing.

3.6 Study Variables

The study variables of this thesis are categorized into dependent and independent.

Dependent Variable (DV): - Atterberg's (PL LL PI), MDD, OMC, Specific Gravity and Soak CBR, Free swell.

Independent Variable (IV): -Dosage of Quarry dust (QD) and Saw Dust (SD).

3.7 Data Processing and analysis

Data processing and analyzing would conduct to finding the index and other important properties of the soils used during the study. In our country, Ethiopia there is no much studies and significant experience on expansive soil treatment/improvement/stabilization. Particularly the performance of quarry dust and saw dust mixing to improve the engineering properties of expansive soils is not much tried yet. This research discusses to improve the strength of expansive soil using quarry dust and saw dust mixed with different proportions to finding optimum percentages expansive soil stabilized and fulfill the requirement of specifications of standard. The test conducted were water content, specific gravity, liquid limit. Plastic limit, hydrometer analysis, MDD and CBR test for different proportion of quarry dust and saw dust.

3.8 Material Sources

The material sources of this study were both primary and secondary data sources would be used. Secondary data needed for this research has been collecting from different journals, book, web site etc. and the primary sources were laboratory experimental outputs.

3.9 Laboratory test

The laboratory proportion of the samples studies are carried out like of Soil alone, Soil + Saw Dust , Soil+ Quarry dust, and Soil + Saw Dust + Quarry dust, with different percentage of Additives. From different literature and research works it was noted that depending on the nature of the stabilizing agent and properties of the soil to be stabilized, the amount by present (weight or volume) of the stabilizing agent varies.

For Atterberg limit tests, soil sample passing sieve size 4.75 mm were again sieved on sieve size of 4.25 mm and for hydrometer analysis they were sieved on 19 mm sieve. Then each specimen was prepared by addition of quarry dust and saw dust with different percentage to have a sample with predetermined percentage of stabilizer varying from 10 to 20% by weight of the soil sample and well mixed by using mixing spoon.

Table 3.4 Maxing ratio of Additive

SI No	Trails	Mixing of expansive soil and Additive
1	Trail 1	100% Soil
2	Trail 2	90% NS + 10% QD
3	Trail 3	80% NS +20% QD
4	Trail 4	90% NS +10% SD
5	Trail 5	80% NS +20% SD
6	Trail 6	80% NS +10% SD +10% QD
7	Trail 7	60% NS +20% SD + 20% QD

Fig 3.16 Material prepared for Laboratory test

CHAPTER FOUR

4. RESULTS AND DISCUSSION

4.1 Introduction

In this chapter presents test results, discussion and analysis of all experimental that were performed on treated/stabilized soils with additives presented in the preceding parts are covered. The general procedure followed to determine whether quarry dust and saw dust stabilization has an effect on Atterberg limits, shrinkage limits, linear shrinkage, moisture density relations, California bearing ratios and unconfined compressive strength values were established by varying percent of stabilizers from 10% to 40% by 10% increment and compared with natural soil/untreated soil engineering property. With refined samples used in the laboratory, clear effects are identified. The figural, graphical and tabular interpretations were performed.

4.1.1 Geotechnical property of expansive soil

The value of engineering properties of expansive soil subgrade was shows as the table 4.1.

Table 4.1: Geotechnical Properties of Expansive Soil

Expansive Soil	
Parameters	Results
Sieve Analysis, % passing #200 sieve	90.75
Liquid Limit, %	105.00
Plastic Limit, %	42.00
Plastic Index, %	63.00
MDD, gm/cc	1.37
OMC, %	26.74
Specific Gravity, %	2.00
Linear Shrinkage, %	12.25
Free Swell Index, %	90.91
CBR Value, %	1.36
CBR Swell, %	20.06

From the table 4.1 the natural subgrade soil of the study area has the engineering properties of LL = 105%, PL = 42% and PI = 63% and More than 35% passing #200 sieve. This soil was very high degree of expansion because; PI value was > 35% as shown on table 2.2 and it affects workability, compactability, constructible pavement structures and layer strengths.

Consequently, the soil needs to be stabilized with an appropriate stabilizer that will increase its strength to a satisfactory level. In this study plasticity can be reduced by modifying or stabilizing the soil with quarry dust and saw dust. Before stabilizing the value of CBR was 1.36%. This was very poor and not recommended for subgrade material. As per AASHTO soil classification system, the subgrade soil falls in the category A-7-5/A-7- 6. Thus, the natural subgrade material was unsuitable to be used as subgrade material without employing improvement methods which was much less than the minimum requirements for a soil to be used as subgrade material.

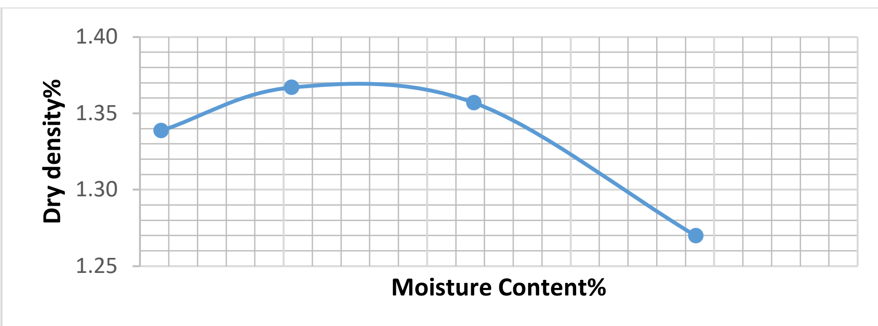

Fig.4.1 Dry density vs Moisture content

As the fig.4.1 shows the moisture content of the natural soil is increase the dry density of natural soil is decrease.

4.1.2 Property of quarry dust

Table 4.2: Properties of QD

Properties	Observed Values
Liquid limit	NP
Plastic limit	NP
Specific gravity	2.71
AASHTO Classification	A-1-a
OMC (%)	8.42
MDD (KN/m3)	1.96
CBR	10.9
Gravel size particles content % (20 to 4.75 mm)	6.6%
Sand size particles content % (4.75 to 0.075 mm)	91%
Fine size particles content % (< 0.075 mm)	2.4%

The results of expansive soil mixes with quarry dust and saw dust are shown in Table 4.2 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 quarry dust and saw dust increases the liquid limit and plastic limit decreases.

Consequently the plasticity index also decreased followed with increase in quarry dust and saw dust content.

Table 4.3: Properties of SD

Saw dust	
Properties	Observed Values
Specific gravity	2.13
OMC (%)	18.42
MDD (gm/cc)	1.46
CBR	6.9

4.2 Consistency Limits

Consistency limits were conducted for soils treated with quarry dust alone, saw dust alone and quarry dust and saw dust combination as described in chapter 3. The consistency limit is such as Liquid Limit, Plastic Limit and Plasticity Index. It was found that as the percentage of quarry dust and saw dust increases the liquid limit and plastic limit decreases. Consequently the plasticity index also decreased followed with increase in quarry dust and saw dust content.

4.2.1 Effect of quarry dust on Atterberg Limits

The natural subgrade soil was found to have a liquid limit of 105%, plastic limit of 42% and plasticity index of 63%. The blending of the subgrade soil with increasing percentage by weight of the quarry dust and saw dust have shown improvement on the values of Atterberg limits as shown Table 4.2 and Fig 4.1 belows. The effect of quarry dust addition in varying proportion with soil was studied and the variation in consistency limits for various mixes is presented in Table 4.2 and Figure 4.1. Soil was treated with various contents of quarry dust from 10% to 20% quarry dust.

Table 4.4: Atterberg limit values for soils treated with quarry dust

Limits	Natural Soil	Quarry dust	
	100%	10%	20%
LL	105	98	96
PL	42	40	34
PI	63	58	62

The LL of untreated soil was determined as 105%, whereas it varied from 98 to 96% after quarry dust 10% and 20% was added respectively.

The LL of the soil decreased with increase in quarry dust content up to 20% after that it increased with increasing in quarry dust content.

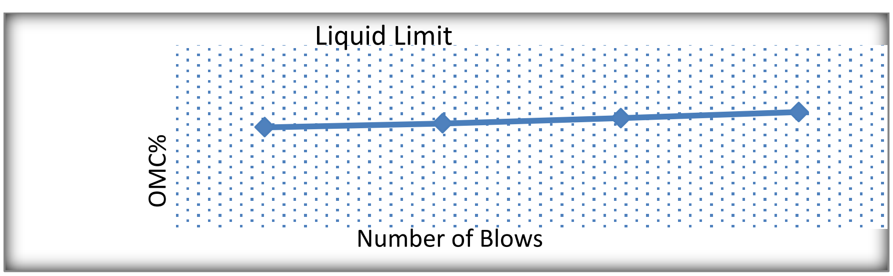

Fig. 4.2 Effect of addition of quarry dust on Atterberg's Limit for Soils

The plastic limit (PL) of untreated soil was determined as 42%, while it didn't change distinctly with an increase in quarry dust content. See Figure 4.1. Plasticity index (PI) values of soils treated with quarry dust decreased with increasing in quarry dust content up 20% content after then it decreased with increasing in quarry dust content. The PI varied from 63% to about 34% for soils treated with quarry dust.

4.2.2 Effect of saw dust on Atterberg Limits

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of soil treated with saw dust were determined (Table 4.2) and plotted (Figure 4.3). The LL values of the samples significantly decreased with increasing saw dust percentages from 10% to 20%, and then increased with the addition of saw dust up 20% contained. The addition of 10% saw dust, 20% saw dust reduced the LL of untreated soils by 94% and 93% respectively. The LL exhibits a minimum value of at an optimum saw dust content of about 20%. Unexpectedly, PL values of samples decreased with increasing stabilizer percentages up to 20%.

Table 4. 5: Atterberg limit values for soils treated with saw dust

Limits	Natural soil	Saw dust	
	100%	10%	20%
LL	105	94	93
	42	62	62
PI	63	32	31

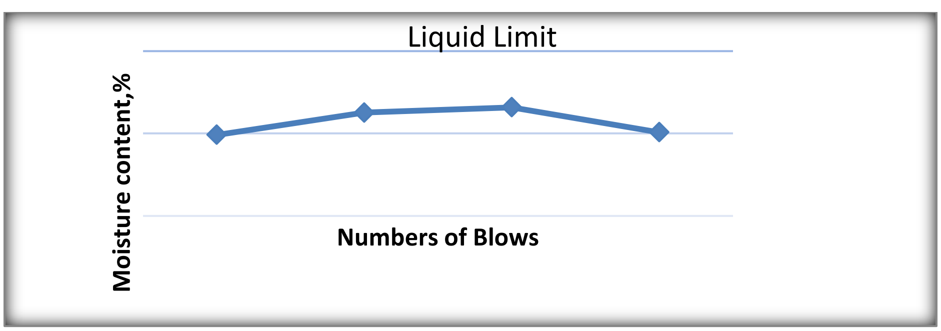

Figure 4.3: Effect of addition of saw dust on Atterberg's Limit for Soils

PI of the samples decreased significantly with increasing stabilizers percentages up 20% of saw dust content and then after increases with the addition of saw dust. The addition of 10% saw dust meaningfully reduced PI of the natural soil from 63% to 32%.

While, the addition of 20% saw dust produced very close result to 20% saw dust in terms of PI reduction with PI values of 31%.

4.2.3 Effect of quarry dust and saw dust on Atterberg Limits

The Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of soil treated with quarry dust and saw dust combination were determined and plotted against quarry dust and saw dust content. (Table 4.4 and Fig.4.3). The LL values of the samples decreased with increasing stabilizer percentages. The addition of 10% quarry dust + 10% saw dust and 20% quarry dust + 20% saw dust diminished the LL of untreated soils by 96% and 96% respectively. While, PL values were decrease 42% and 29% with addition stabilizer content.

Table 4.6: Atterberg Limit values for soils treated with Quarry dust and Saw dust

Limits	Natural soil	Quarry dust + Saw dust	
	100%	10% +10%	20% + 20%
LL	105	96	96
PL	42	54	67
PI	63	42	29

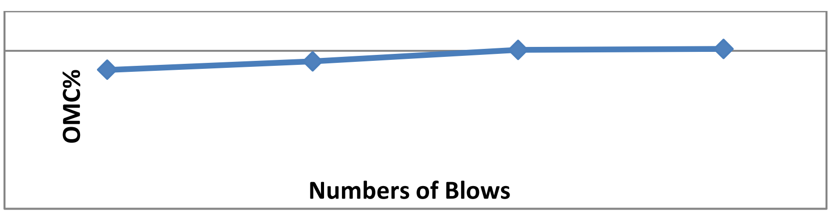

Figure 4.4 Effect of addition of quarry dust and saw dust on Atterberg's Limit for Soils

The result showed that there is a reduction in plastic index as increasing the ratio of stabilizer in both cases, QD and SD stabilizer. The highest reduction in plastic index occur when it was stabilized with maximum ratios and the minimum reduction occur at minimum ratios. In general for quarry dust and saw dust stabilization the following observation have been made.Liquid limit decreases with increasing quarry dust and saw dust proportios.Plastic limit increases with increasing quarry dust and saw dust proportios Plastic index decreases with increasing quarry dust and saw dust proportios.

4. 4 Compaction Characteristics

The compaction test is carried out on expansive soil blended with quarry dust and saw dust. The dry density-moisture content relationship for different proportions. It is observed that MDD and OMC value are increases from 1.2g/cm^3 to 1.51g/cm^3 and from 26.74% to 20.07% respectively with addition of 20% QD +20% SD.

4.4.1 Compaction Characteristics of Soils stabilized by Quarry dust

Sample soils treated with quarry dust for moisture density relationship characteristics with MDD value increased from 1.2 to 1.26 gm/cm3 with 10% and 20% quarry dust content by dry weight respectively see Figure 4.5.

Table 4.7: Effects of quarry dust on OMC vs MDD

% of Natural Soil	% of Additives	OMC (%)	MDD (gm/cc)
90	10 QD	26.74	1.2
80	20 QD	22.97	1.26

It is observed that with the increase in quarry dust content MDD values increases up to 20% quarry dust content and with 10% QD and 20% QD increase in 1.2 to 1.26 gm/cm^3 respectively.

4.4.2 Compaction Characteristics of Soils stabilized by Saw dust

Sample soils treated with saw dust for moisture density relationship characteristics with MDD Values increased from 1.22 to 1.41 gm/cm^3 with 10% and 20% saw dust content by dry weight respectively see Figure 4.6.

Table 4.8: Effects of saw dust on OMC vs MDD

% of Natural Soil	% of Additives	OMC (%)	MDD (gm/cc)
90	10 SD	24.45	1.22
80	20 SD	20.88	1.41

It is observed that with the increase in saw dust content MDD values increases up to 20% saw dust content and with 10% SD and 20% SD increase in 1.22 to 1.41 g/cm^3 respectively.

4.4.2 Compaction Characteristics of Soils stabilized by quarry dust and saw dust

Sample soils treated with quarry dust and saw dust for moisture density relationship characteristics with MDD values increased from 1.24 gm/cm3 to 1.51 gm/cm3 with 10% QD + 10% SD and 20% QD + 20% SD content by dry weight respectively see Table 4.9.

Table 4.9: Effects of quarry dust and saw dust on OMC vs MDD

% of Natural Soil	% of Additives	OMC (%)	MDD (gm/cc)
90	10 SD	24.37	1.24
80	20 SD	20.47	1.51

It is observed that with the increase in quarry dust content MDD values increases up to 20% quarry dust content and with 10% QD + 10% SD and 20% QD +20% SD increase in 1.24 to 1.51 gm/cm^3 respectively.

4.4.3 Comparison of Effect of Stabilizers on Compaction Characteristics of Soils

The summary of the modified Proctor test results of treated soils as compared with the natural soils is presented in Table 4.8 and Figure 4.7. OMC and MDD observed were treated by quarry dust and saw dust.

Table 4.10: Change in Compaction characteristics of soil with additives

Parameters	0%	10%	10%	20%	20%	10% +10%	20% +20%
OMC (%)	26.74	24.45	24.37	22.97	20.88	20.74	20.07
MDD (gm/cc)	1.2	1.22	1.24	1.31	1.37	1.41	1.51

Moreover, the maximum change in OMC was observed to be 26.74 % for soil treated by 10% quarry dust and 10% saw dust while the least change in OMC was noted to be 20.07 % of soil treated by 20% quarry dust +20% saw dust. The highest change in MDD was measured to be 1.51% for treated soil by 20% quarry dust + 20% saw dust as shows fig.4.7 below.

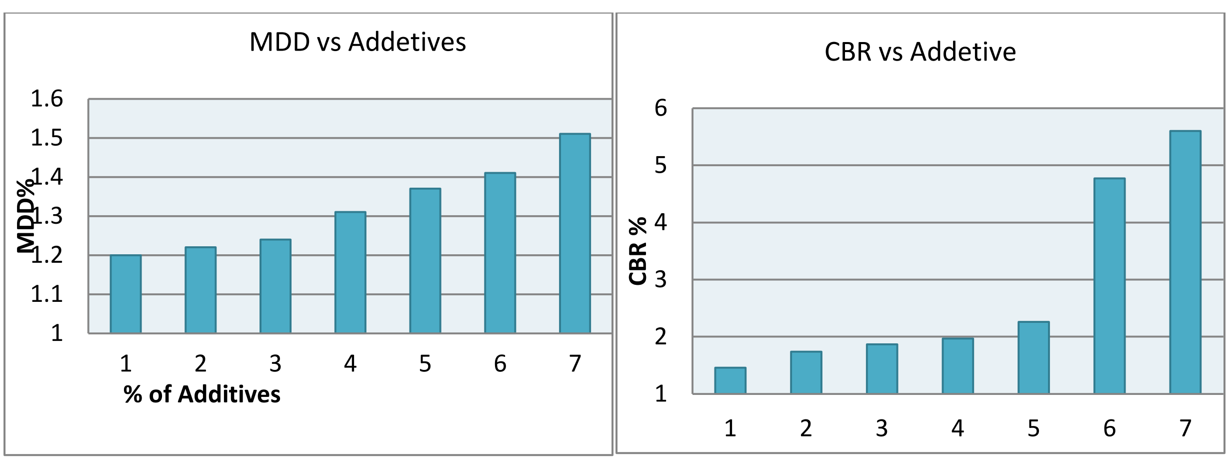

Fig.4.5 MDD and OMC vs Additives

The figure 4.7 shows the variation OMC value of expansive soil blended with different percentage of quarry dust and saw dust. It seen that the addition of quarry dust and saw dust 0%, 10%, 10%, 20%, 20% and 10% + 10% and 20% +20% the OMC of soil 26.74%, 24.45%, 24.37%, 22.97%, 20.88%, 20.74%, and 20.07% respectively. Further increasing of quarry dust and saw dust content increases the OMC. This increment was due to increasing additives content increases chemical reaction that needs more water to accomplish the task of chemical stabilization.

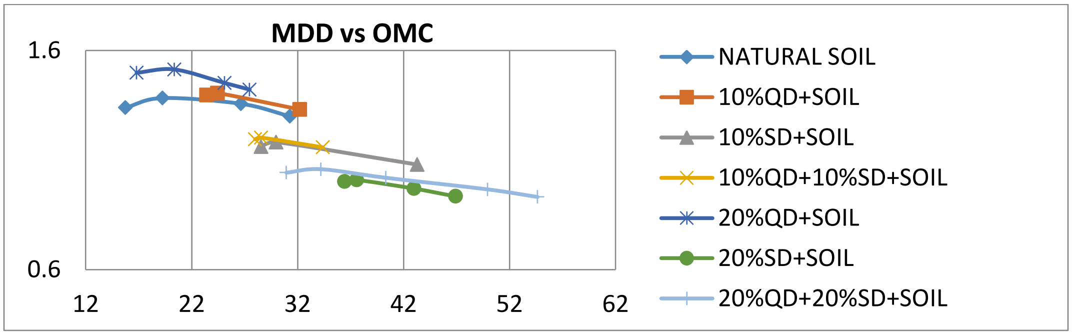

Figure 4.6 Compaction characteristics curve for stabilized soil with QD and SD.

The MDD of crusher dust stabilized laterite gravel were 1.2g/cm^3 ,1.22g/cm^3 , 1.24g/cm^3 , 1.31g/cm^3, 1.37g/cm, 1.41g/cm^3 and 1.51g/cm^3 with the additions 0% ,10% QD , 10% SD , 20% QD,10% SD, 20% QD ,20% ,SD 20% ,10% QD + 10% SD and 20% QD + 20% SD, respectively.

4.4.4 Effect of Quarry dust and Saw dust on Free Swell

Free swell tests result indicated the potential expansiveness of soil samples without being loaded was very high.The free swell of stabilized soil sample is shows as table 4.9.

Table 4.11: Free swell

Samples	V_o	V_f	FS (%)
NS	18.5	21	90.91
NS + 10% QD	18	20	81.82
NS + 10% SD	15	24	50
NS + 20% QD	27	27	28.57
NS + 20% QD	13	21	75
NS + 10% QD + 10% SD	12	23	64.29
NS + 20% QD + 20% SD	16	27	42.11

The natural soil free swell was 90.91, when adding the additives step by steps the free swell was decreased from 90.91 to 42.11%.As increasing the proportion of stabilizer reduces the swelling of soils. The result showed on table 4.9 both stabilizers were effective to reduce the swelling potential of expansive soils. When the swell decrease the strength of the natural soil was increase.

4.4.5 Effect of Quarry dust and Saw dust on CBR

The soil sample as it is without modification is tested for soaked CBR test and the CBR value is found to be 5.6% at 95% MDD. The Laterite soil was modified by addition of Crusher dust in the proportion of 0% up to 20% with 10% increment increased CBR from 1.36% to maximum value of 5.6%. Generally, As quarry dust and saw dust dosage increased CBR strength were increasing percentage of quarry dust and saw dust as shown below.

Table 4.12: CBR values of stabilized and expansive soil

Percentages of quarry dust and saw dust mixed with expansive soil	Soaked CBR value at	
	2.54 mm penetration	5.08 mm penetration
NS	1.36	1.46
10% QD	1.74	1.56
10% SD	1.70	1.87
20% QD	1.97	1.71
20% SD	1.82	2.26
10% QD + 10% SD	1.82	1.80
20% QD + 20% SD	4.7	5.6

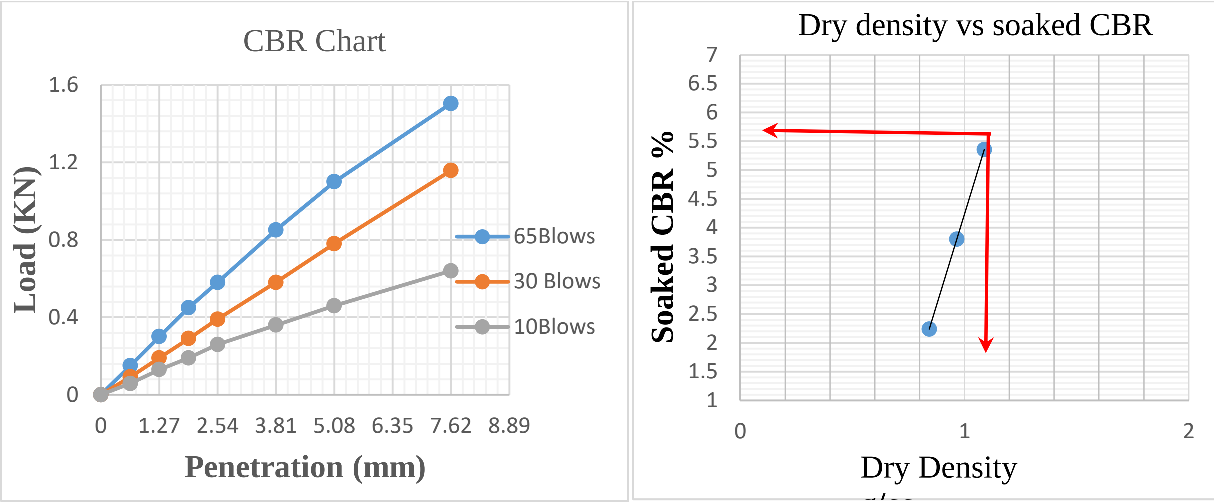

Fig 4.7 CBR vs MDD

The CBR values of natural sub grade expansive soils were not fulfills the requirement of sub-grade soil. According to ERA low volume pavement manual specification it is not allowed to use CBR values less than 3%, because from both a technical and economic perspective it would normally be in appropriate to lay a pavement on soils of such bearing capacity.The improved expansive soil using both quarry dust and saw dust was very suitable for increasing subgrade soils strength.Therefore, it need to improvement with mixing of QD and SD.

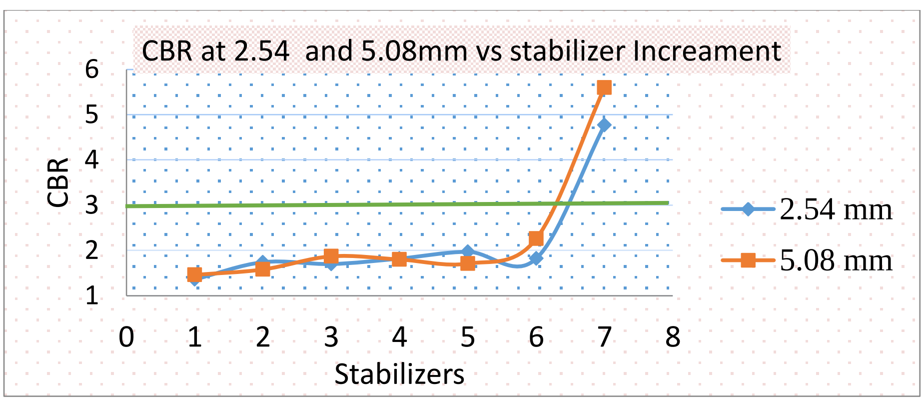

Fig 4.8 CBR @2.54 & 5.08

The improvement fulfills at optimum percentage of 60% NS + 20% QD + 20% SD ERA standard to use for sub-grade.becuase, the value of CBR at 2.54 mm increase from 1.36% to 4.7% and at 5.08 mm from 1.46 to 5.6%. So, 4.7 and 5.6% were > 3% of ERA manual spesification.

Poor to faire Soil class based on USCSas shows on table2.3 and 2.5. At 60% NS + 20% QD + 20% SD the strength class of natural soil was increase from **S1 to S3** and we can use as subgrade material. The result shows as the table 4.1.

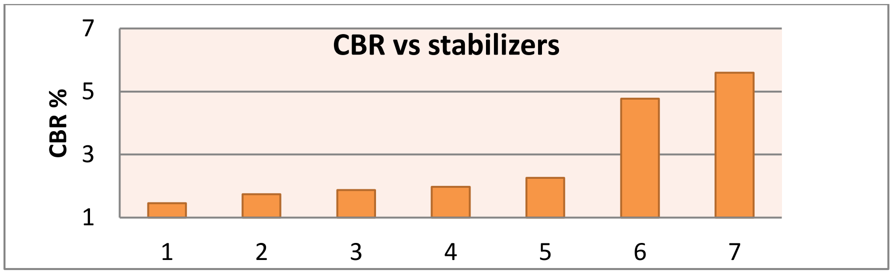

Fig 4.9 CBR vs stabilizers

As the result shows, the strength of the subgrade soil 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. As the additives increased the moisture content decrease the dry density increase and CBR increase. The additives which I used in this thesis for stabilizing expansive soil was increase the strength of the expansive soil from S1 to S3.

4.4.6 Effect of Quarry dust and Saw dust on CBR Swell

The quarry dust and saw dust soil mixtures compacted in CBR molds at optimum moisture content with maximum dry density gauged for swelling characteristics before and after soaking for four days to evaluate the percent of swell. The test result at different ratios was illustrated in figure 4.13 below.

Table 4.13: CBR Swell

Samples Name	Percentage of sample	CBR Swell (%)
NS	100	20.6
QD + NS	10 + 90	19.46
SQ + NS	10 + 90	9.68
QD + NS	20 + 80	18.95
SD + NS	20 + 80	9.46
QD + SD + NS	10 + 10 + 80	10.07
QD + SD + NS	20 + 20 + 60	6.35

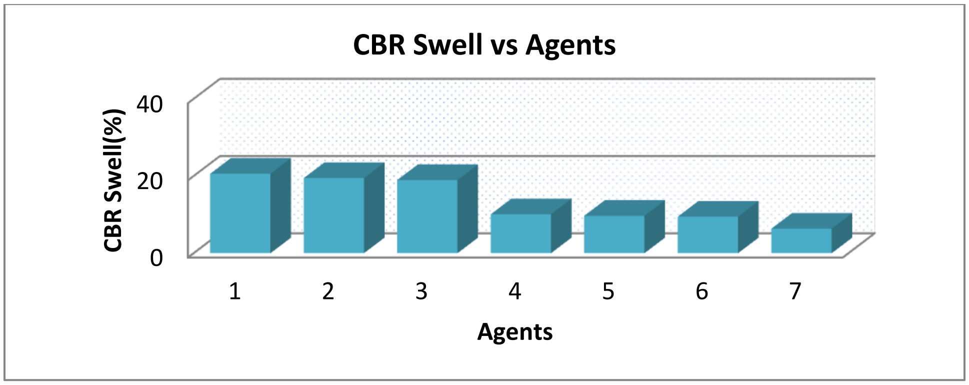

Figure 4.10 CBR Swell test result of stabilized and natural Expansive soil

The Figure 4.13 shows natural soil has the properties of swelling and high plastic. However, when quarry dust and saw dust mix with different ratio the CBR swell reduce. The decrease in CBR Swell is due to replacement of quarry dust and saw dust in place of plastic fine in soil.

4.5 Summary of parameters at 20% of agents

This research involved the investigation of the improvement of the strength sub-grade expansive soil using saw dust and quarry dust . The study investigated the index properties and strength measurement parameters. The parameters were grain size analysis, specific gravity, natural moisture content, Atterberg limits, compaction, free swell, Unconfined strength, CBR and CBR swelling potential of stabilized expansive soil. There was a variation of those parameters during stabilization. According to the laboratory test result of the natural soil samples obtained during the present study the sub grade soils were classified as A-7-5 as per AASHTO and CH as USCS system.

Table 4.14: Summary of the laboratory output at 60% NS + 20% QD + 20% SD

Parameter	LL	PL	PI	FS	SG	LS	CBR	CBR swell	OMC	DD
60%NS + 20% QD +20% SD	96	67	29	28.57	1.27	13.45	5.6	6.35	20.45	1.51

The result indicated that the two stabilizers namely saw dust and quarry dust were very effective in improving strength parameters.

Generally the changes on engineering properties of stabilized sub grade expansive soil for the present study were summarized as follows.

Increasing additive mixing proportion decreases liquid limit of the soil,Increasing additive mixing proportion increases plastic limit of the soil,Increasing additive mixing proportion decreases plastic index of the soils,Increasing additive mixing proportion decreases free swell of the soil,Increasing additive mixing proportion increases CBR of soils and Increasing additive mixing proportion decreases CBR swell potential of the soil

Table: 4.15: General summaries of laboratory results for sample

ID	Samples	Parameter							
		MDD	OMC	LL	PL	PI	CBR 2.54 mm	CBR 5.08 mm	CBR swell
1	Natural soil`	1.2	26.28	105	42	63	1.36	1.46	20.06
2	Soil+ 10% QD	1.22	24.45	98	40	58	1.74	1.58	19.46
3	Soil +10% SD	1.24	24.37	94	62	32	1.7	1.87	9.68
4	Soil + 20% QD	1.31	20.37	96	34	62	1.97	1.71	18.45
5	Soil + 20% SD	1.37	22.97	93	62	31	1.82	2.26	9.46
6	Soil + 10% QD + 10% SD	1.41	20.88	96	54	42	1.82	1.8	10.07
7	Soil + 20% QD + 20% SD	1.51	20.40	96	67	29	4.77	5.6	6.35

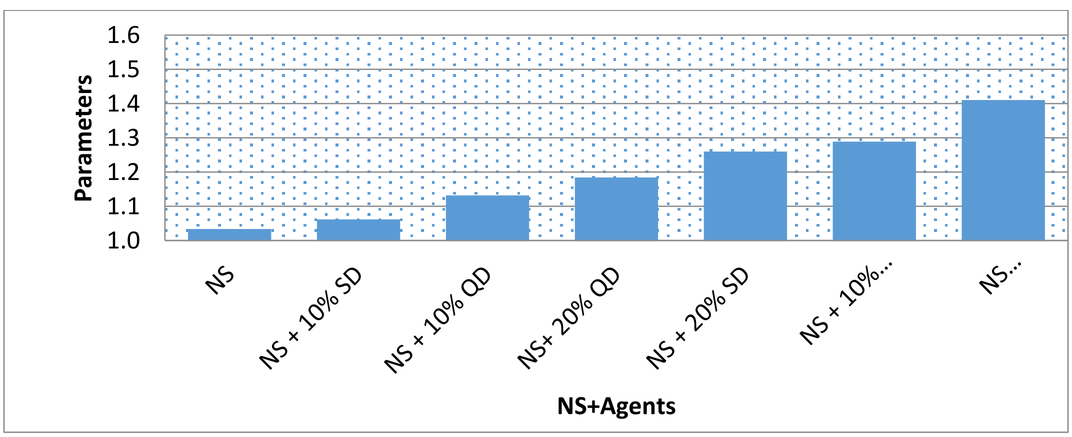

Fig. 4.11 Parameters vs. effects of additives on natural soil

Generally, as shows on the table 4.10 and fig 4.9 the strength of sub grade expansive soil is increase with increment values of MDD,CBR, and decreasing of OMC,FS,LL,PL,PI,CBR Swell with increments ratios of Saw dust and Quarry dust.

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study the following conclusions were drawn:

- Expansive soil is subject to swelling and shrinkage of the soil, varying in proportion to the amount of moisture presents in the soil and need to improve.
- Addition of quarry dust and saw dust stabilizers increases the strength of expansive soil of sub-grade from S1 to S3 strength class at 60% NS + 20% QD + 20% SD proportion.
- The effect of the quarry dust and saw dust was reduces the LL and PI. At 20% quarry dust and 20% saw dust mixing expansive soil strength was suitable for sub grade material as per ERA specification.
- The MDD were increase with the addition of quarry dust and saw dust from 1.2 to 1.51 g/cm^3, OMC decreased from 26.74 to 20.07%, CBR increased from 1.36 to 5.6% at 0% to 20% in the blending of natural soil with quarry dust and saw dust respectively.
- The improvement is fulfills at optimum percentage of 60% NS + 20% QD + 20% SD.

5.2. Recommendation

According to the findings of this research, the following recommendations are forwarded to next researcher:-

- Further study may have carried out on site to understand the effects of quarry dust and saw dust stabilize agents on expansive soil.
- Due to geological case and some other factors expansive soils may have different properties from place to place and location to location. Thus, it is recommended that the performance of quarry dust and saw dust as stabilizing additives should be studied on expansive soils of different origins.
- Investigation the comparative performance sustainability of natural soil with quarry dust and saw dust on flied and economical variation of quarry dust and saw dust with other agents.

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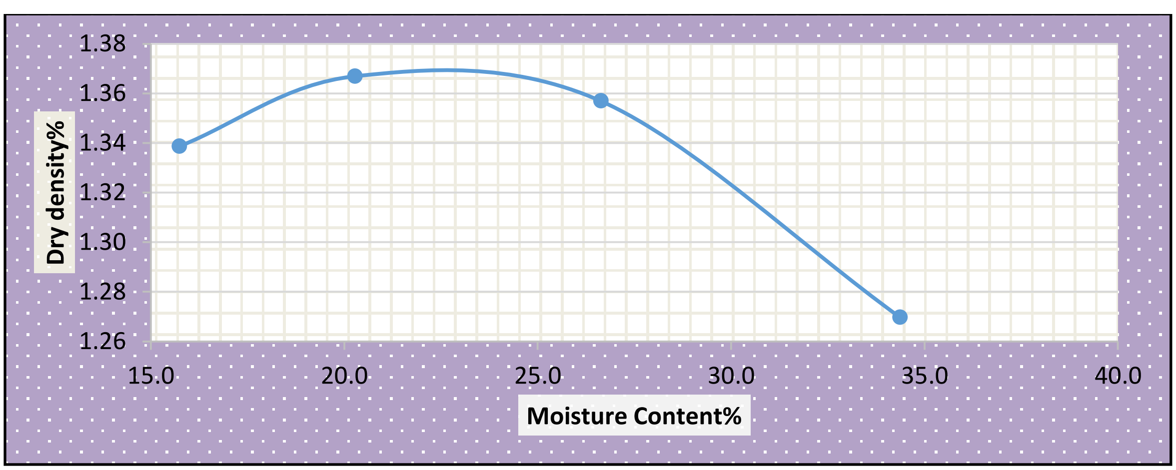
Appendices

Appendices-1 Compaction Test and results

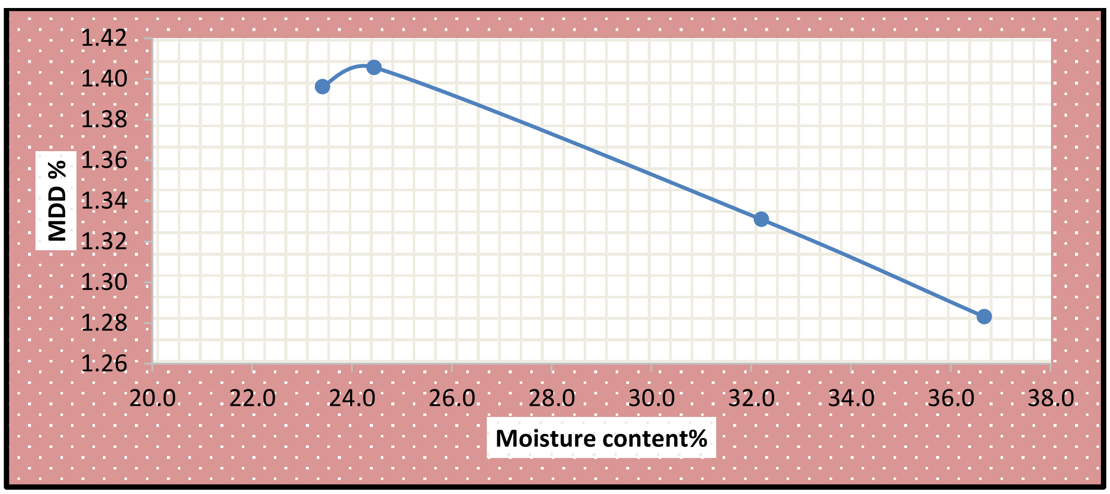
Fig 4.2 Compaction taken (15/10/2021 G.C)

Natural soil

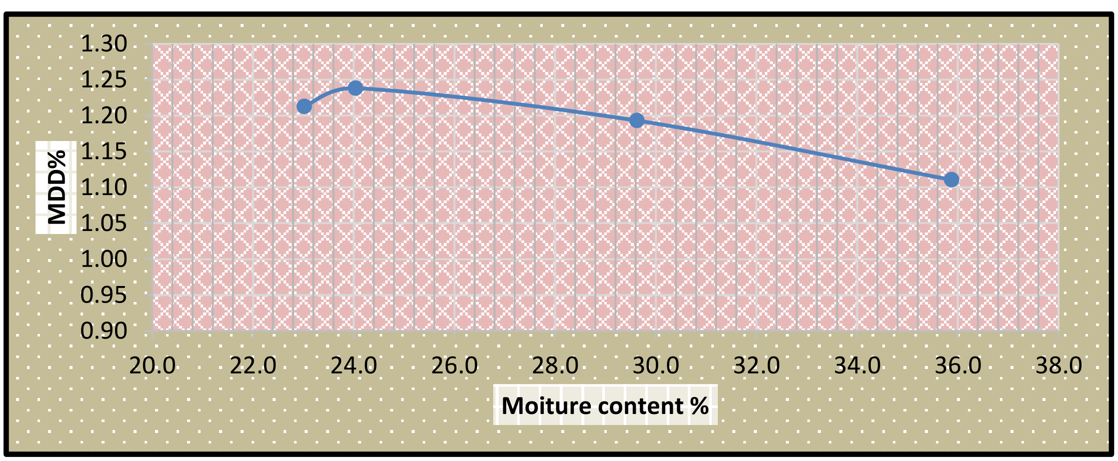
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	240	400	560	720	
Mass of mould +Wet soil (g),=m1	6005.5	6201.8	6364.5	6333.3	
Mass of mould (g)=m2	2714.8	2709.6	2714.8	2709.6	
Mass of Wet soil (g), m3=m1-m2	3290.7	3492.2	3649.7	3623.7	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.5	1.6	1.72	1.71	
Moisture content determination					
Container code	A2	E	G3T3	G3T2	NMC(P65)
Mass of wet soil + cont (g), m4	135.56	174.7	149.3	165.3	200.10
Mass of dry soil + cont (g), m5	121.6	151.62	125.82	131.92	186.59
Mass of container (g), m6	32.89	37.83	37.67	34.79	37.74
Mass of moisture (g), m7=m4-m5	14.0	23.1	23.48	33.38	13.51
Mass of dry soil (g), m8=m5-m6	88.71	113.79	88.15	97.13	148.85
Moisture content (%), w =(m7/m8)*100	15.7	20.3	26.64	34.37	9.08
Dry density, γd=(ρ/(100+w))*100	1.34	1.37	1.36	1.27	



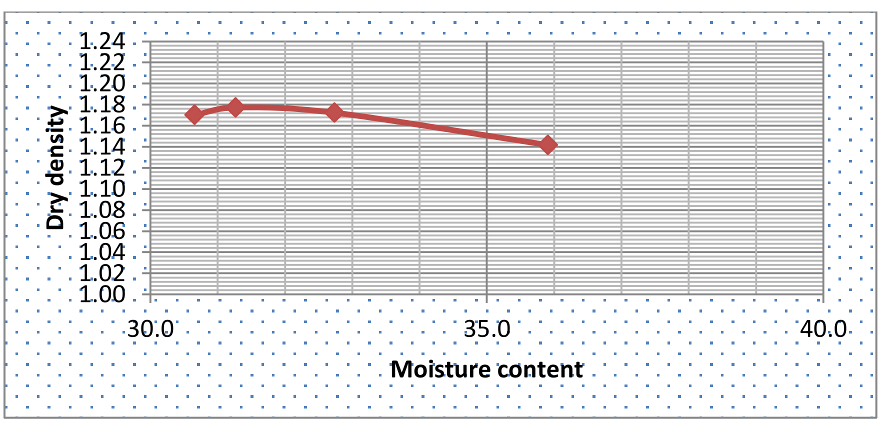
10% QD+ SOIL					
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	400	560	720	880	
Mass of mould +Wet soil (g),=m1	6380.3	6420.8	6458.1	6430	
Mass of mould (g)=m2	2721	2705.5	2721	2705.5	
Mass of Wet soil (g), m3=m1-m2	3659.3	3715.3	3737.1	3724.5	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.72	1.75	1.76	1.75	
Moisture content determination					
Container code	G3T2	G3T3	A16	T1	NMC(E)
Mass of wet soil +cont (g), m4	207.1	214.5	180.7	163.32	224.30
Mass of dry soil + cont (g), m5	174.41	179.77	144.71	129.59	207.94
Mass of container (g), m6	34.78	37.7	32.97	37.61	37.90
Mass of moisture (g), m7=m4-m5	32.7	34.7	35.99	33.73	16.36
Mass of dry soil (g), m8=m5-m6	139.63	142.07	111.74	91.98	170.04
Moisture content (%), w =(m7/m8)	23.4	24.4	32.21	36.67	9.62
Dry density, γd=(ρ/(100+w))*100	1.40	1.41	1.33	1.28	



10% SD+ SOIL					
Density Determination					
Test No.	1	2	3	4	10% SD+ SOIL
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	700	860	1020	1180	
Mass of mould +Wet soil (g),=m1	5889.1	5971.3	6004.8	5910.2	
Mass of mould (g)=m2	2720.4	2709.4	2720.3	2705.6	
Mass of Wet soil (g), m3=m1-m2	3168.7	3261.9	3284.5	3204.6	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.49	1.54	1.55	1.51	
Moisture content determination					
Container code	A3	P65	LC1	J41	NMC(E)
Mass of wet soil +cont (g), m4	188.38	160.1	167.8	160.11	224.30
Mass of dry soil + cont (g), m5	159.99	136.58	134.81	126.99	207.94
Mass of container (g), m6	36.68	38.74	23.48	34.68	37.90
Mass of moisture (g), m7=m4-m5	28.4	23.5	32.99	33.12	16.36
Mass of dry soil (g), m8=m5-m6	123.31	97.84	111.33	92.31	170.04
Moisture content (%), w =(m7/m8)*100	23.0	24.04	29.63	35.88	9.62
Dry density, γd=(ρ/(100+w))*100	1.21	1.24	1.19	1.11	

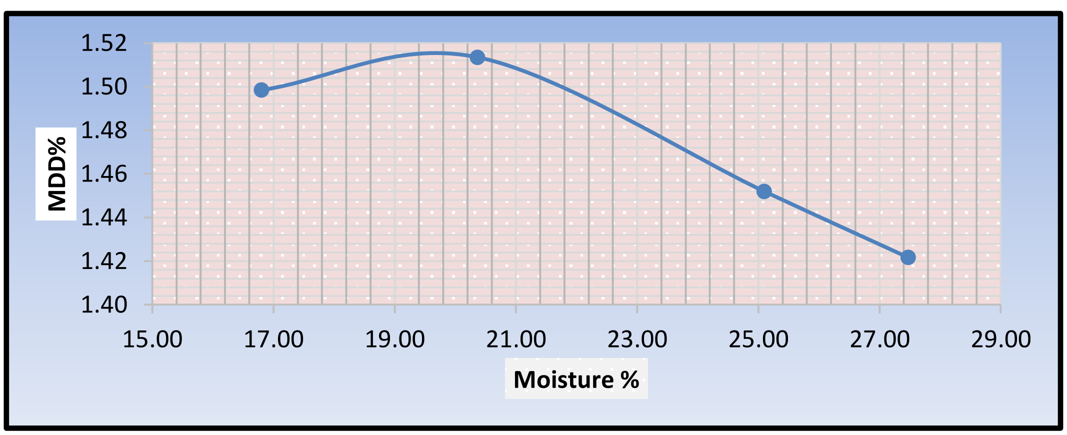


10%QD+10%SD+SOIL					
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	600	760	920	1080	
Mass of mould +Wet soil (g),=m1	5969.7	5988.0	6026.2	6016.7	
Mass of mould (g)=m2	2722.3	2705.7	2721	2721	
Mass of Wet soil (g), m3=m1-m2	3247.4	3282.3	3305.2	3295.7	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.5	1.5	1.56	1.55	
Moisture content determination					
Container code	A3	G3T3	C15	G63	NMC(G19)
Mass of wet soil +cont (g), m4	162.4	197.65	139.3	148.93	201.64
Mass of dry soil + cont (g), m5	136.22	162.12	110.68	116.92	185.20
Mass of container (g), m6	32.5	37.71	25.45	25.3	36.04
Mass of moisture (g), m7=m4-m5	31.8	38.9	27.9	32.9	16.44
Mass of dry soil (g), m8=m5-m6	103.72	124.41	85.23	91.62	149.16
Moisture content (%), w =(m7/m8)*100	30.7	31.3	32.73	35.91	11.02
Dry density, γd=(ρ/(100+w))*100	1.17	1.18	1.17	1.14	

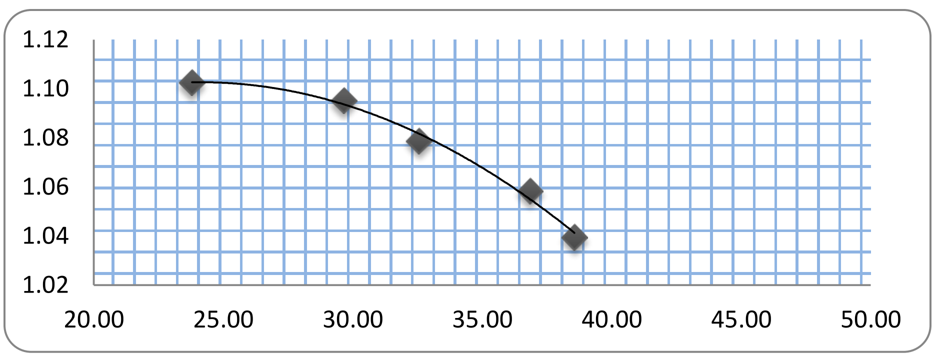


20%SD+SOIL					
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95					
Density Determination					
Test No.	1	2	3	4	
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	900	1060	1220	1380	
Mass of mould +Wet soil (g),=m1	5614	5652.0	5656.5	5635	
Mass of mould (g)=m2	2712	2704	2712	2721	
Mass of Wet soil (g), m3=m1-m2	2902.0	2948.0	2944.5	2914	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.4	1.4	1.39	1.37	
Moisture content determination					
Container code	G3T3	A3	G63	TR	NMC(J41)
Mass of wet soil +cont (g), m4	158.2	166.01	142.8	149.84	172.20
Mass of dry soil + cont (g), m5	135.96	145.98	127.9	129.89	155.58
Mass of container (g), m6	37.72	32.69	26.35	27.3	32.67
Mass of moisture (g), m7=m4-m5	22.2	20.0	14.9	19.95	16.62
Mass of dry soil (g), m8=m5-m6	98.24	113.29	101.55	102.59	122.91
Moisture content (%), w =(m7/m8)*100	22.64	17.68	14.67	19.45	13.52
Dry density, γd=(ρ/(100+w))*100	1.11	1.18	1.21	1.15	

20%QD+SOIL					
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95					
Density Determination					
Test No.	1	2	3	4	20%QD+SOIL
Mass of sample (g)	4000	4000	4000	4000	
Water Added (cc)	280	440	600	760	
Mass of mould +Wet soil (g),=m1	6430	6573.4	6562	6559.8	
Mass of mould (g)=m2	2712.5	2704	2704	2710.6	
Mass of Wet soil (g), m3=m1-m2	3717.5	3869.4	3858.0	3849.2	
Volume of Mould (cc), V	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.8	1.8	1.82	1.81	
Moisture content determination					
Container code	02,1	P65	G19	E	NMC(J41)
Mass of wet soil +cont (g), m4	236.3	198.86	163.11	177.72	172.20
Mass of dry soil + cont (g), m5	206.98	171.59	135.49	144.87	155.58
Mass of container (g), m6	32.5	37.71	25.45	25.3	32.67
Mass of moisture (g), m7=m4-m5	29.3	27.3	27.62	32.85	16.62
Mass of dry soil (g), m8=m5-m6	174.48	133.88	110.04	119.57	122.91
Moisture content (%), w =(m7/m8)*100	16.80	20.37	25.10	27.47	13.52
Dry density, γd=(ρ/(100+w))*100	1.50	1.51	1.45	1.42	



20%QD+20% QD +SOIL						
COMPACTION (MODIFIED PROCTOR TEST AASHTO T180-95						
Density Determination						
Test No.	1	2	3	4	5	
Mass of sample (g)	4000	4000	4000	4000	4000	
Water Added (cc)	800	960	1120	1280	1440	
Mass of mould +Wet soil (g),=m1	5604.7	5721.7	5743	5782	5765	
Mass of mould (g)=m2	2706	2706	2706	2706	2706	
Mass of Wet soil (g), m3=m1-m2	2898.7	3015.7	3037.0	3076.0	3059.0	
Volume of Mould (cc), V	2124	2124	2124	2124	2124	
Bulk Density (g/cc), ρ=m3/V	1.4	1.4	1.43	1.45	1.44	
Moisture content determination						
Container code	T1	0Z-1	G19	G63	A2	NMC(E)
Mass of wet soil +cont (g), m4	134.77	133.4	165.4	128.3	155.2	231.60
Mass of dry soil + cont (g), m5	112.81	106.55	128.76	98.44	116.77	210.54
Mass of container (g), m6	20.5	16	16.2	17.4	17.1	37.90
Mass of moisture (g), m7=m4-m5	22.0	26.9	36.64	29.86	38.43	21.06
Mass of dry soil (g), m8=m5-m6	92.31	90.55	112.56	81.04	99.67	172.64
Moisture content (%), w =(m7/m8)*100	23.79	29.65	32.55	36.85	38.56	12.20
Dry density, γd=(ρ/(100+w))*100	1.10	1.10	1.08	1.06	1.04	

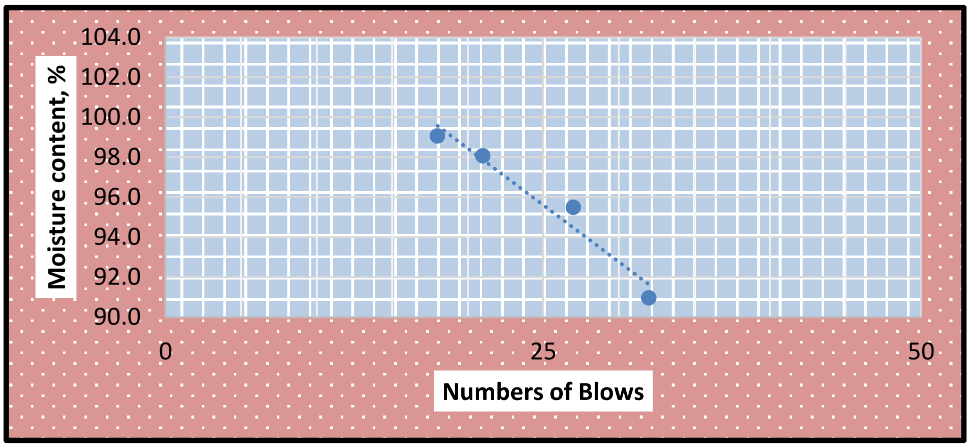


Appendices-2 Atterberg Limit Test and results

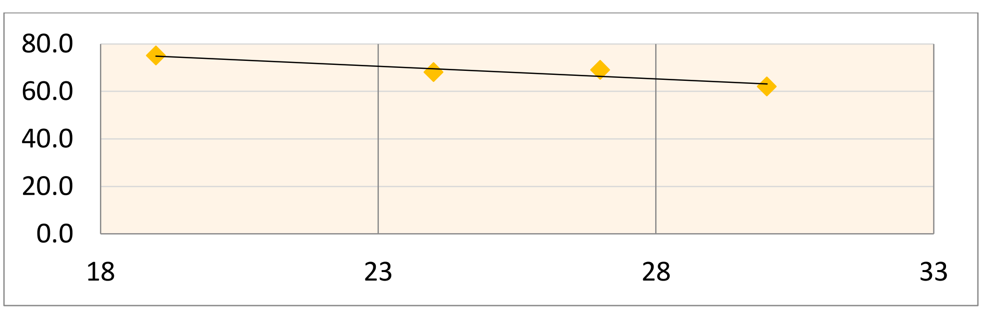
Figure 4.3 Liquid Limit (17/10/2021)

Natural soil				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**32**	**27**	**21**	**18**
Trial	1	2	3	4
Container	D	J1	C4	T1
Wt. of container + wet soil,g	16.01	19.53	19.38	23.32
Wt. of container + dry soil,g	11.69	12.97	12.79	14.92
Wt. of container,g	6.94	6.1	6.07	6.44
Wt. of water,g	4.32	6.56	6.59	8.4
Wt. of dry soil,g	4.75	6.87	6.72	8.48
Moisture content,%	**90.9**	**95.5**	**98.1**	**99.1**
Plastic Limit				
Trial	1	2		
Container Code	L3	B1		
Wt. of container + wet soil,g	17.2	13.7		
Wt. of container + dry soil,g	13.9	11.21		
Wt. of container,g	6.54	5.52		

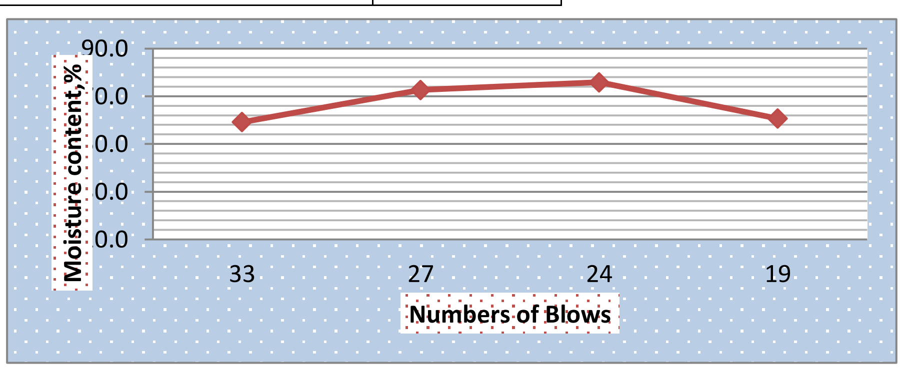
Wt. of water,g	3.3	2.49
Wt. of dry soil,g	7.36	5.69
Moisture content,%	44.8	43.8
Average moisture Content (%)	**44**	
Liquid limit	**96**	
Plastic limit	**44**	
Plasticity Index(PI)	**52**	



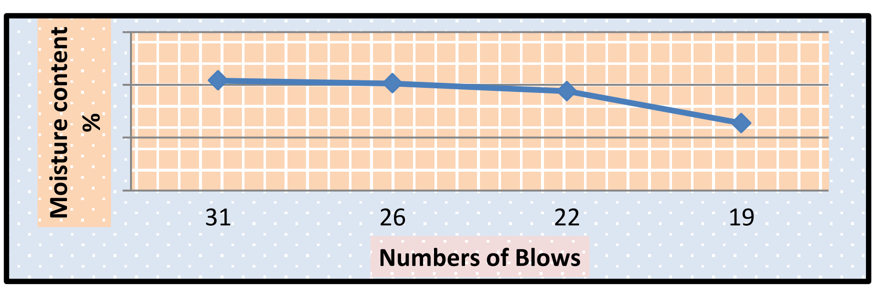
10%SD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**32**	**28**	**23**	**19**
Trial	1	2	3	4
Container	B9	4	14	C9
Wt. of container + wet soil,g	35.95	31.31	36.05	30.47
Wt. of container + dry soil,g	28.96	25.21	29.03	25
Wt. of container,g	19.66	17	19.48	16.99
Wt. of water,g	6.99	6.1	7.02	5.47
Wt. of dry soil,g	9.3	8.21	9.55	8.01
Moisture content,%	**75.2**	**74.3**	**73.5**	**68.3**
Plastic Limit				
Trial	1	2		
Container Code	A4	H23		
Wt. of container + wet soil,g	26.57	29.63		
Wt. of container + dry soil,g	24.84	25.45		
Wt. of container,g	16.94	18.61		
Wt. of water,g	1.7	4.18		
Wt. of dry soil,g	7.9	6.84		
Moisture content,%	21.9	61.1		
Average moisture content (%)	**42**			
Liquid limit	**105**			
Plastic limit	**42**			
Plasticity Index(PI)	**63**			



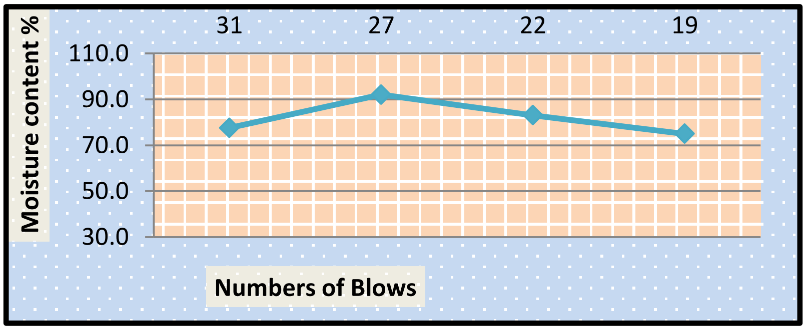
10%QD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**33**	**27**	**24**	**19**
Trial	1	2	3	4
Container	G8	E13	3L	13
Wt. of container + wet soil,g	36.12	31.45	34.81	35.14
Wt. of container + dry soil,g	29.66	25.58	28.25	29.08
Wt. of container,g	18.74	17.5	19.6	19.09
Wt. of water,g	6.46	5.87	6.56	6.06
Wt. of dry soil,g	10.92	8.08	8.65	9.99
Moisture content,%	**59.2**	**72.6**	**75.8**	**60.7**
Plastic Limit				
Trial	1	2		
Container Code	F5	A17		
Wt. of container + wet soil,g	26.75	30.22		
Wt. of container + dry soil,g	24.86	27.36		
Wt. of container,g	18.15	21.94		
Wt. of water,g	1.9	2.86		
Wt. of dry soil,g	6.71	5.42		
Moisture content,%	28.2	52.8		
Average moisture content (%)	**40**			
Liquid limit	**98**			
Plastic limit	**40**			
Plasticity Index(PI)	**58**			



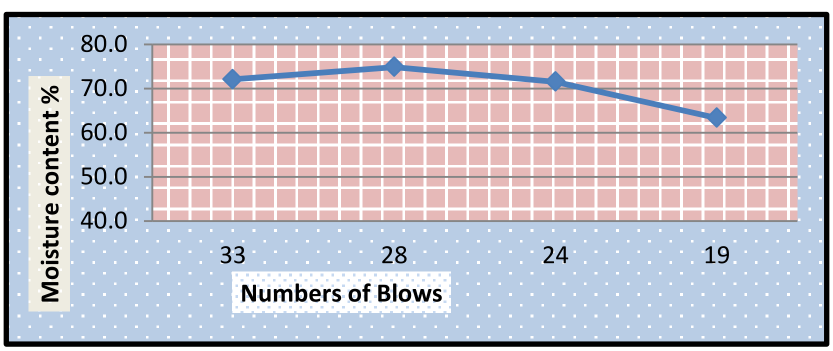
20%QD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**31**	**26**	**22**	**19**
Trial	1	2	3	4
Container	E13	A17	3L	G8
Wt. of container + wet soil,g	35.82	37.92	41.42	36.12
Wt. of container + dry soil,g	28.25	31.33	32.59	29.44
Wt. of container,g	17.56	21.95	19.75	18.79
Wt. of water,g	7.57	6.59	8.83	6.68
Wt. of dry soil,g	10.69	9.38	12.84	10.65
Moisture content,%	**70.8**	**70.3**	**68.8**	**62.7**
Plastic Limit				
Trial	1	2		
Container Code	2	C4		
Wt. of container + wet soil,g	27.34	14.55		
Wt. of container + dry soling	24.55	13.25		
Wt. of container	18.99	6.08		
Wt. of water	2.8	1.3		
Wt. of dry soling	5.56	7.17		
Moisture content,%	50.2	18.1		
Average moisture Content (%)	**34**			
Liquid limit	**96**			
Plastic limit	**34**			
Plasticity Index(PI)	**62**			



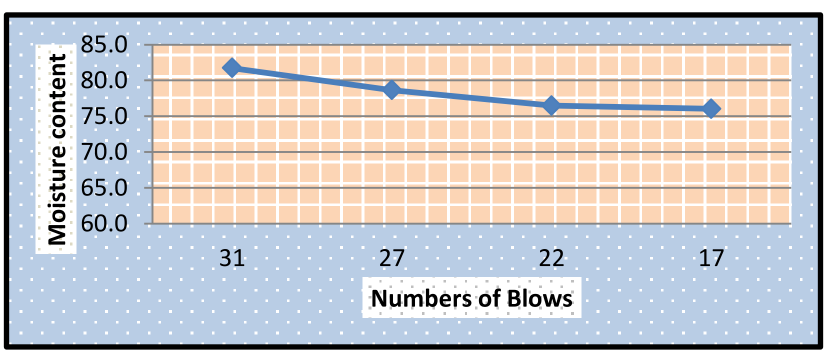
20%SD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liquid limit				
Number of blows	**31**	**27**	**22**	**19**
Trial	1	2	3	4
Container	D	C9	A4	13
Wt. of container + wet soling	33.1	32	34.5	32.3
Wt. of container + dry soling	27.16	24.57	26.56	26.6
Wt. of container	19.5	16.5	17	19
Wt. of water	5.94	7.43	7.94	5.7
Wt. of dry soil,g	7.66	8.07	9.56	7.6
Moisture content,%	**77.5**	**92.1**	**83.1**	**75.0**
Plastic Limit				
Trial	1	2		
Container Code	F5	H23		
Wt. of container + wet soil,g	24.6	26.5		
Wt. of container + dry soil,g	22.63	23.01		
Wt. of container,g	18.2	18.6		
Wt. of water,g	2.0	3.49		
Wt. of dry soil,g	4.43	4.41		
Moisture content,%	44.5	79.1		
Average moisture Content (%)	**62**			
Liquid limit	**96**			
Plastic limit	**62**			
Plasticity Index(PI)	**34**			



10%QD+10%SD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**33**	**28**	**24**	**19**
Trial	1	2	3	4
Container	1E	C9	14	A4
Wt. of container + wet soil,g	39.62	34.93	33.9	34.69
Wt. of container + dry soil,g	31.03	27.25	27.9	27.79
Wt. of container,g	19.11	16.99	19.51	16.89
Wt. of water,g	8.59	7.68	6	6.9
Wt. of dry soil,g	11.92	10.26	8.39	10.9
Moisture content,%	**72.1**	**74.9**	**71.5**	**63.3**
Plastic Limit				
Trial	1	2		
Container Code	H23	F5		
Wt. of container + wet soil,g	25.4	25.93		
Wt. of container + dry soil,g	23.02	23.21		
Wt. of container,g	18.71	18.15		
Wt. of water,g	2.4	2.72		
Wt. of dry soil,g	4.31	5.06		
Moisture content,%	55.2	53.8		
Average moisture Content (%)	**54**			
Liquid limit	**96**			
Plastic limit	**54**			
Plasticity Index(PI)	**42**			



20%QD+20%SD+SOIL				
ATTERBERG LIMIT (AASHTO T089-94,T090-96)				
Liqud limit				
Number of blows	**31**	**27**	**22**	**17**
Trial	1	2	3	4
Container	A17	2	3L	E13
Wt. of container + wet soil,g	38.2	34.4	33.7	31.3
Wt. of container + dry soil,g	30.69	27.62	27.59	25.34
Wt. of container,g	21.5	19	19.6	17.5
Wt. of water,g	7.51	6.78	6.11	5.96
Wt. of dry soil,g	9.19	8.62	7.99	7.84
Moisture content,%	**81.7**	**78.7**	**76.5**	**76.0**
Plastic Limit				
Trial	1	2		
Container Code	G8	B9		
Wt. of container + wet soil,g	27	28.5		
Wt. of container + dry soil,g	23.5	25.1		
Wt. of container,g	18.6	19.7		
Wt. of water,g	3.5	3.4		
Wt. of dry soil,g	4.9	5.4		
Moisture content,%	71.4	63.0		
Average moisture Content (%)	**67**			
Liquid limit	**96**			
Plastic limit	**67**			
Plasticity Index(PI)	**29**			



Appendices-3 Free Swell Test and results

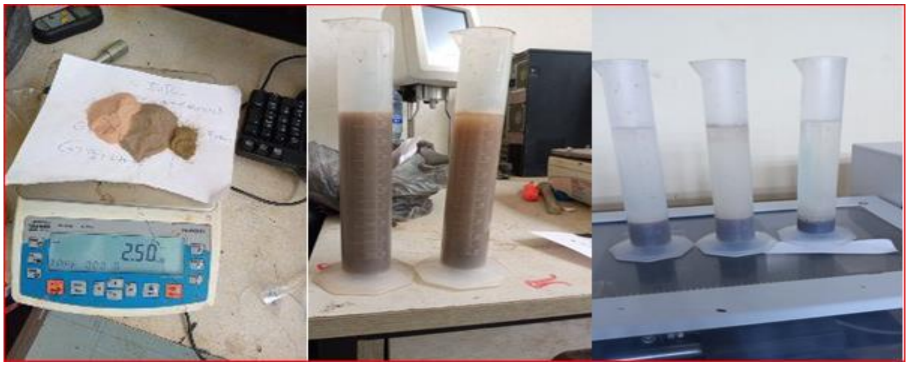


Figure 4.4 Free Swell Test procedures (18/10/2021)

	FREE SWELL INDEX				
Samples	Measuring Cylinder No.(ml)		Reading after 24 hrs(ml)		Free Swell
	Kerosene	Distilled water	Kerosene	Distilled water	Index, %
Natural Soil	11	18.5	11	21	90.91
Natural Soil+10% Quarry Dust	11	19	11	20	81.82
Natural soil +10% Saw Dust	16	15	16	24	50.00
Natural soil +10% QD+10%SD+Soil	12	12	14	23	64.29
Natural soil +20% Quarry Dust	12	13	12	21	75.00
Natural soil 20% Saw Dust	21	27	21	27	28.57
NS +20% QD+20%SD	15	16	19	27	42.11

Appendices-4 Linear Shrinkage Test and results

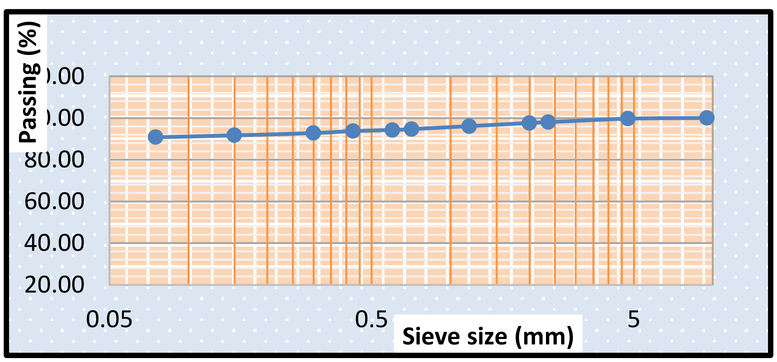
Figure 4.5 linear shrinkage (18/10/2021)

Linear Shrinkage				
Name of sample	Length of instr	Trail 1	Trail2	Avrage
Natural soil	14	12.5	12	12.25
90%NS+ 10%QD	14	12	12.5	12.25
90%NS+10%SD	14	12.5	13.5	13
10%QD+ 10%SD+80% Soil	14	12.8	12.2	12.5
20%QD+80 Soil	14	13.5	13.4	13.45
20%SD+80 Soil	14	13.6	13.5	13.55
20%QD+20%SD+60 Soil	14	13.4	13.5	13.45

Appendices-5 Gradation Test and result

Figure 4.6 Sieve Procedures (25/10/2021)

Sieve size (mm)	Mass of retain on each sieve(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	3.30	0.33	0.33	99.67
2.36	16.60	1.66	1.99	98.01
2	3.80	0.38	2.37	97.63
1.18	15.20	1.52	3.89	96.11
0.71	14.30	1.43	5.32	94.68
0.6	4.30	0.43	5.75	94.25
0.425	5.00	0.50	6.25	93.75
0.3	9.90	0.99	7.24	92.76
0.15	10.90	1.09	8.33	91.67
0.075	9.20	0.92	9.25	90.75
pan	907.5	90.75	100.00	0.00
Sum	1000.0			



Appendices-6 Specific Gravity Test and results

Figure 4.7 Specific gravity test procedure (22/10/2021)

Natural Soil								
Specific Gravity								
Determination Code					17		C	
Mass of dry, clean Calibrated pycnometer, Mp, in g					26.68		30.72	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					122.16		127	
C. Mass of Pycnometer + water + sample(gm)					134.7		139.5	
Observed temperature of water,Ti					26		26	
Water Temperature(^{O}C)								
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs			Gs=A*k/(A+B-C)		2.01		2.00	
Average Specific gravity at 20oc, Gs					**2.00**			

10% QD+90% Soil								
Specific Gravity								
Determination Code					17		C	
Mass of dry, clean Calibrated pycnometer, Mp, in g					26.96		30.94	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					122.57		127.16	
C. Mass of Pycnometer + water + sample(gm)					133.25		135.53	
Observed temperature of water,Ti					26		26	
Water Temperature(^{O}C)								
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs			Gs=A*k/(A+B-C)		1.75		1.50	
Average Specific gravity at 20oc, Gs					**1.62**			

10%SD+90% Soil								
Specific Gravity								
Determination Code					C		A3	
Mass of dry, clean Calibrated pycnometer, Mp, in g					26.42		27.78	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					77.25		78.3	
C. Mass of Pycnometer + water + sample(gm)					84.27		86.85	
Observed temperature of water,Ti					26		26	
Water Temperature(^{O}C)					22		22	
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs		Gs=A*k/(A+B-C)			1.39		1.52	
Average Specific gravity at 20oc, Gs					**1.45**			

10% QD+10%SD+80% Soil								
Specific Gravity								
Determination Code					0.3		A	
Mass of dry, clean Calibrated pycnometer, Mp, in g					30.54		22.24	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					124.81		77.47	
C. Mass of Pycnometer + water + sample(gm)					133.99		84.32	
Observed temperature of water,Ti					27		27	
Water Temperature(^{O}C)								
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs		Gs=A*k/(A+B-C)			1.58		1.38	
Average Specific gravity at 20oc, Gs					**1.48**			

20% QD+80% Soil								
Specific Gravity								
Determination Code					B		C	
Mass of dry, clean Calibrated pycnometer, Mp, in g					26.96		30.94	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					122.57		127.16	
C. Mass of Pycnometer + water + sample(gm)					133.21		135.5	
Observed temperature of water,Ti					22		22	
Water Temperature(^{O}C)						25	25	
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs			Gs=A*k/(A+B-C)		1.74		1.50	
Average Specific gravity at 20oc, Gs					**1.62**			

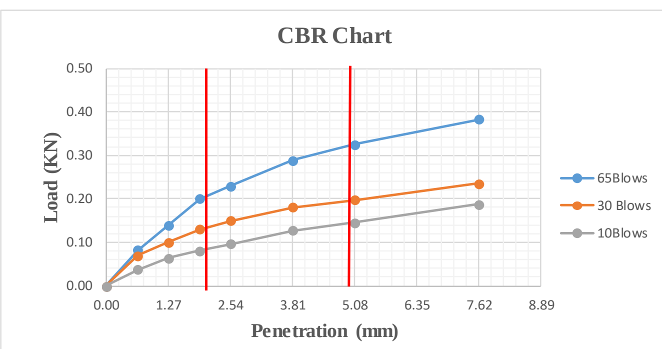
20%SD+80% Soil								
Specific Gravity								
Determination Code					3		C	
Mass of dry, clean Calibrated pycnometer, Mp, in g					30.5		30.8	
A. Mass of oven dry sample(gm)					25		25	
B. Mass of Pycnometer + water(gm)					113.8		111.4	
C. Mass of Pycnometer + water + sample(gm)					124.5		127.5	
Observed temperature of water,Ti					22		22	
Water Temperature(^{O}C)						25	25	
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs			Gs=A*k/(A+B-C)		1.75		2.81	
Average Specific gravity at 20oc, Gs					**2.28**			
20% QD+20% SD+60% Soil								
Specific Gravity								
Determination Code					17		A	
Mass of dry, clean Calibrated pycnometer, Mp, in g					26.77		22.2	
A. Mass of oven dry sample(gm)					51.7		46.6	
B. Mass of Pycnometer + water(gm)					108.2		71.2	
C. Mass of Pycnometer + water + sample(gm)					122.7		77.6	
Observed temperature of water,Ti					22		22	
Water Temperature(^{O}C)						21	21	
^{o}C	18	19	20	21	22	23	24	25
k	1.0016	1.0014	1.0012	1.0009	1.007	1.0005	1.0003	1.000
Temperature of contents of pycnometer when Mpsw was taken, Tx, in oc					26		26	
K for Tx					0.9997		0.9997	
Specific gravity at 20oc, Gs			Gs=A*k/(A+B-C)		1.39		1.16	
Average Specific gravity at 20oc, Gs					**1.27**			

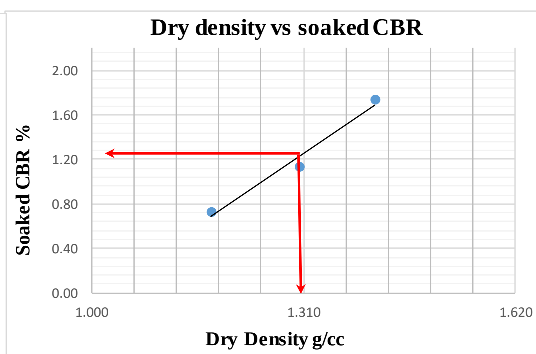
Appendices-6 CBR Test and results

Figure 4.8 CBR Compaction

Figure 4.9 Soaked CBR Procedures (16/10/2021 & 20/10/2021)

NATURAL SOIL						
CBR TEST (AASHTO T 193-93)						
Compaction Determination						
COMPACTION DATA	**65Blows**		**30Blows**		**10Blows**	
	Before soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Mould No.	N13	N13	30Blows	30Blows	N2	N2
Mass of wet soil + Mould, g	10538.8	11143.9	10356.1	11091.8	9928.8	10906.8
Mass Mould, g	6966	6966	7013	7013	6963.8	6963.8
Mass of Soil, g	3572.8	4177.9	3343.1	4078.8	2965	3943
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.68	1.97	1.574	1.920	1.396	1.856
Dry density of soil, g/cc	1.415	1.229	1.304	1.177	1.176	1.091
Moisture Determination						
MOISTURE CONTENT DATA	**65Blows**		**30Blows**		**10Blows**	
	Before soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Container no.	E	G63	G3T2	02,1	G3T3	G19
Mass of wet soil + Container, g	182.2	191	171.3	193.1	180.1	214.0
Mass of dry soil + Container, g	159.24	128.8	147.9	129.3	157.6	140.6
Mass of container, g	37.83	25.3	34.8	28.31	37.67	36.04
Mass of water, g	22.96	62.2	23.4	63.8	22.5	73.4
Mass of dry soil,g	121.41	103.5	113.1	101.0	119.9	104.6
Moisture content, %	18.9	60.1	20.7	63.1	18.8	70.1
CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		**Surcharge Weight:-4.55 KG**				
Pen.mm	**65 Blows**		**30 Blows**		**10Blows**	
	Load, KN	**CBR%**	**Load, KN**	**CBR%**	**Load, KN**	**CBR %**
0.00	0.00		0.00		0.00	
0.64	0.06		0.60		0.05	
1.27	0.10		0.09		0.06	
1.91	0.15		0.11		0.08	
2.54	0.18	1.36	0.13	1.01	0.10	0.76
3.81	0.29		0.17		0.13	
5.08	0.30	1.46	0.20	0.98	0.15	0.74
7.62	0.33		0.24		0.19	
Modified Max.Dry Density g/cc	**1.37**			**OMC %**	**20.28**	
Swell Determination						
Date	**65 Blows**		**30 Blow**		**10Blows**	
	Gauge reading (mm)	**Swell in %**	**Gauge reading(mm**	**Swell in %**	**Gauge reading(mm)**	**Swell in %**
06/01/2014 (Initial)	**0.5**	20.66	0.50	21.22	**0.28**	19.92
10/01/2014 (Final)	24.55		25.20		23.47	

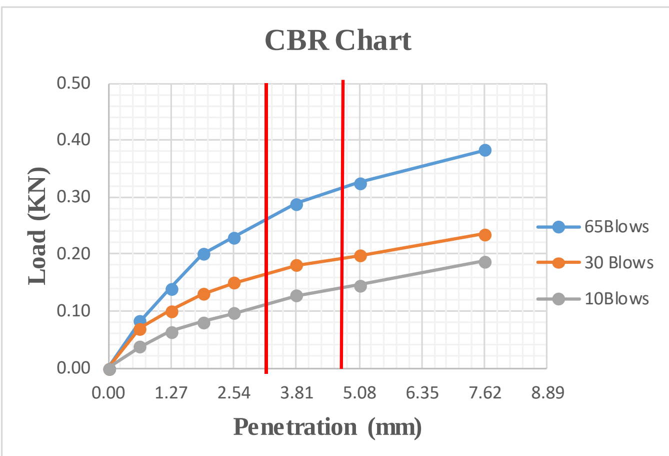


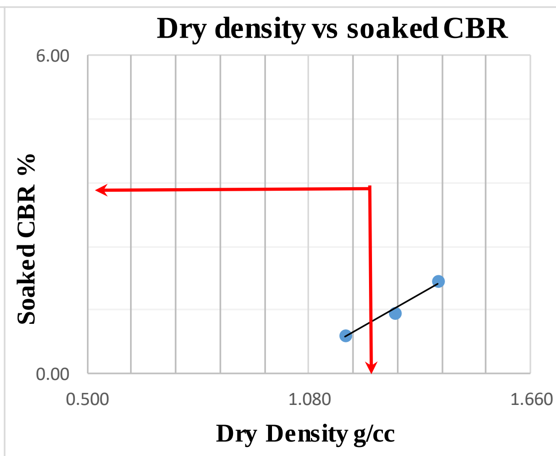


Dry Density at 95% of MDD:					**1.30**
No.of blows	**MCBS %**	**DDBS g/cm3**	Corrected CBR %	% Compaction	
10	18.8	1.176	0.76	86	
30	20.7	1.304	1.01	95	
65	18.9	1.415	1.46	103	
CBR (%) @ 95 % MDD			**1.0**	**% Swell**	**20.60**

10%QD+90%SOIL						
	CBR TEST (AASHTO T 193-93)					
Compaction Determination						
COMPACTION DATA	**65Blows**		**30Blows**		**10Blows**	
	Before soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Mould No.	N30	N30	N1	N1	M30	M30
Mass of wet soil + Mould, g	10790.7	11260.6	10609.6	11266.6	10391	11218.5
Mass Mold, g	6989.8	6989.8	6935.7	6935.7	6999.1	6999.1
Mass of Soil, g	3800.9	4270.8	3673.9	4330.9	3391.9	4219.4
Volume of Mold, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.79	2.01	1.730	2.039	1.597	1.987
Dry density of soil, g/cc	1.294	1.256	1.210	1.337	1.129	1.157
Moisture Determination						
MOISTURE CONTENT DATA	**65Blows**		**30Blows**		**10Blows**	
	Before soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Container no.	E	G63	G3T2	02,1	G3T3	G19
Mass of wet soil + Container, g	182.2	191	189.3	193.1	186.1	214.0
Mass of dry soil + Container, g	142.24	128.8	142.9	136.3	142.6	139.6
Mass of container, g	37.83	25.3	34.8	28.31	37.67	36.04
Mass of water, g	39.96	62.2	46.4	56.8	43.5	74.4
Mass of dry soling	104.41	103.5	108.1	108.0	104.9	103.6
Moisture content, %	38.3	60.1	42.9	52.6	41.4	71.8

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period			**Surcharge Weight:-4.55 KG**			
Pen.mm	**65 Blows**		**30Blows**		**10Blows**	
	Load, KN	**CBR%**	**Load, KN**	**CBR%**	**Load, KN**	**CBR %**
0.00	0.00		0.00		0.00	
0.64	0.08		0.07		0.06	
1.27	0.14		0.10		0.09	
1.91	0.19		0.14		0.12	
2.54	0.23	1.74	0.17	1.29	0.14	1.06
3.81	0.29		0.17		0.14	
5.08	0.33	1.58	0.20	0.96	0.16	0.79
7.62	0.38		0.24		0.19	
Modified Max.Dry Density g/cc	**1.28**			**OMC %**	**36.67**	
Swell Determination						
Date	**65 Blows**		**30 Blow**		**10Blows**	
	Gauge reading (mm)	**Swell in %**	**auge reading(mn**	**Swell in %**	**Gauge reading(mm)**	**Swell in %**
06/01/2014 (Initial)	**0.5**	18.08	0.50	19.50	**0.28**	20.78
10/01/2014 (Final)	21.55		23.20		24.47	

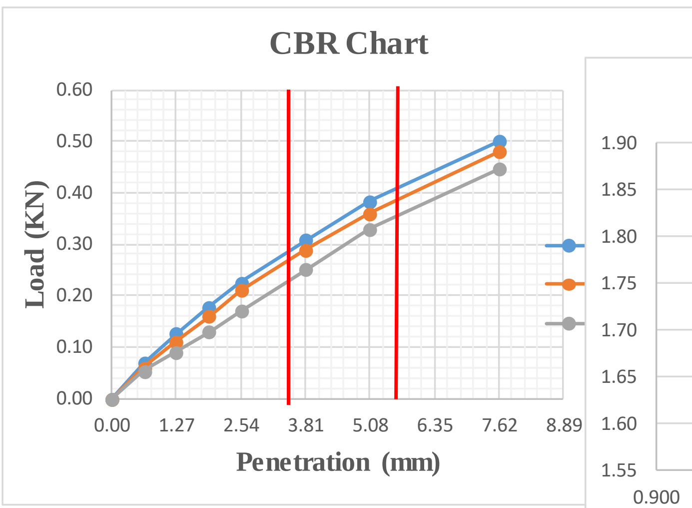


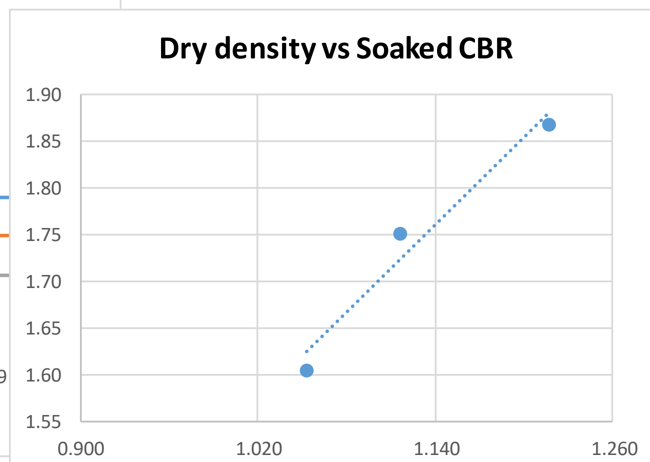


Dry Density at 95% of MDD:					**1.219**
No.of blows	**MCBS %**	**DDBS g/cm3**	Corrected CBR %	% Compaction	
10	41.4	1.129	1.06	88	
30	42.9	1.210	1.29	94	
65	38.3	1.294	1.74	101	
CBR (%) @ 95 % MDD			**1.3**	**% Swell**	**19.46**

10%SD+90%SOIL						
	CBR TEST (AASHTO T 193-93)					
Compaction Determination						
COMPACTION DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	N	N	M35	M35	N12	N12
Mass of wet soil + Mould, g	10253	10533.6	10132.2	10491.5	9889.1	10403.5
Mass Mould, g	6930.5	6930.5	6981.1	6981.1	6980.9	6980.9
Mass of Soil, g	3322.5	3603.1	3151.1	3510.4	2908.2	3422.6
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.56	1.70	1.484	1.653	1.369	1.611
Dry density of soil, g/cc	1.217	1.283	1.116	1.213	1.053	1.730
Moisture Determination						
MOISTURE CONTENT DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	E	E	P65	G19	G3T2	G3T3
Mass of wet soil + Container, g	169.89	158.13	177.2	163.1	184.5	133.6
Mass of dry soil + Container, g	140.56	128.8	142.7	129.3	149.9	140.6
Mass of container, g	37.9	37.9	37.7	36.10	34.74	37.70
Mass of water, g	29.33	29.33	34.6	33.8	34.6	-7.1
Mass of dry soil,g	102.66	90.9	104.9	93.2	115.2	102.9
Moisture content, %	28.6	32.3	32.9	36.2	30.1	-6.9

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		Surcharge Weight:-4.55 KG				
	65 Blows		30Blows		10Blows	
Pen.mm	Load, KN	CBR%	Load, KN	CBR%	Load, KN	CBR %
0.00	0.00		0.00		0.00	
0.64	0.07		0.06		0.05	
1.27	0.13		0.11		0.09	
1.91	0.18		0.16		0.13	
2.54	0.23	1.70	0.21	1.59	0.17	1.29
3.81	0.31		0.29		0.25	
5.08	0.38	1.87	0.36	1.75	0.33	1.61
7.62	0.50		0.48		0.45	
Modified Max.Dry Density g/cc	1.24			OMC %	24.04	
Swell Determination						
Date	65 Blows		30 Blow		10Blows	
	auge reading (mn	Swell in %	auge reading(mn	Swell in %	uge reading(m	Swell in %
06/01/2014 (Initial)	0.5	9.66	0.50	9.17	0.28	10.21
10/01/2014 (Final)	11.75		11.17		12.17	

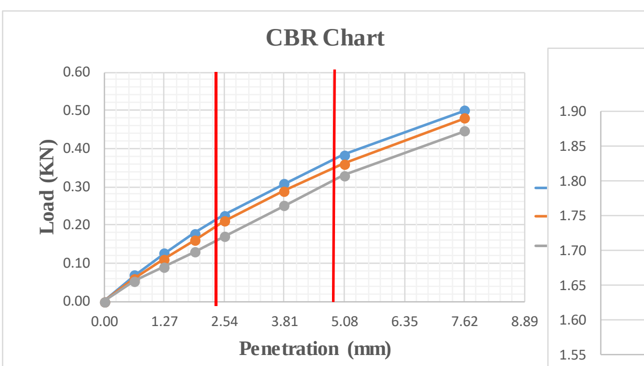


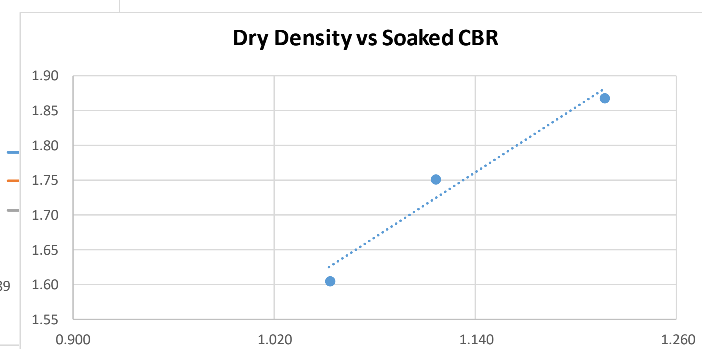


Dry Density at 95% of MDD:					1.18
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	30.1	1.053	1.61	85	
30	32.9	1.116	1.75	90	
65	28.6	1.217	1.87	98	
CBR (%) @ 95 % MDD			1.8	% Swell	9.68

20%SD+80%SOIL						
	CBR TEST (AASHTO T 193-93)					
Compaction Determination						
COMPACTION DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	N13	N13	N2	N2	CB1	CB1
Mass of wet soil + Mould, g	9957.8	10433.6	9679.5	10233.8	9333.3	10125.8
Mass Mould, g	6962.6	6962.6	6958.7	6958.7	6945.4	6945.4
Mass of Soil, g	2995.2	3471	2720.8	3275.1	2387.9	3180.4
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.41	1.63	1.281	1.542	1.124	1.497
Dry density of soil, g/cc	1.020	0.882	0.943	0.850	0.821	0.762
Moisture Determination						
MOISTURE CONTENT DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	G3T2	G3T2	0Z-1	A16	G65	C15
Mass of wet soil + Container, g	142.4	172.39	154.2	160.2	145.6	156.2
Mass of dry soil + Container, g	112.59	109.03	121.0	103.0	113.2	92.0
Mass of container, g	34.75	34.78	28.3	32.91	25.34	25.49
Mass of water, g	29.81	63.36	33.2	57.1	32.4	64.1
Mass of dry soil,g	77.84	74.25	92.7	70.1	87.9	66.5
Moisture content, %	38.3	85.3	35.8	81.5	36.9	96.4

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		Surcharge Weight:-4.55 KG				
	65 Blows		30Blows		10Blows	
Pen.mm	Load, KN	CBR%	Load, KN	CBR%	Load, KN	CBR %
0.00	0.00		0.00		0.00	
0.64	0.08		0.07		0.07	
1.27	0.15		0.12		0.12	
1.91	0.19		0.16		0.16	
2.54	0.24	1.82	0.21	1.62	0.20	1.49
3.81	0.32		0.33		0.28	
5.08	0.47	2.26	0.44	2.14	0.36	1.76
7.62	0.53		0.73		0.53	
Modified Max.Dry Density g/cc	1.11			OMC %	22.64	
Swell Determination						
Date	65 Blows		30 Blow		10Blows	
	Gauge reading (mm)	Swell in %	Gauge reading(mm)	Swell in %	Gauge reading(mm)	Swell in %
06/01/2014 (Initial)	0.65	9.71	0.30	10.21	0.4	8.47
10/01/2014 (Final)	11.95		12.19		10.26	

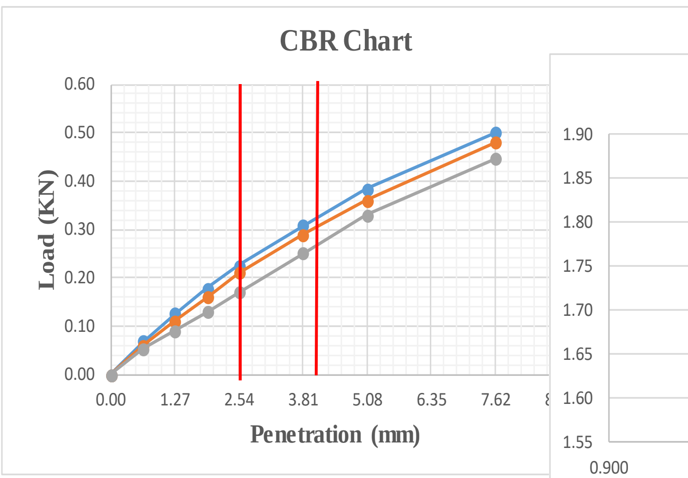


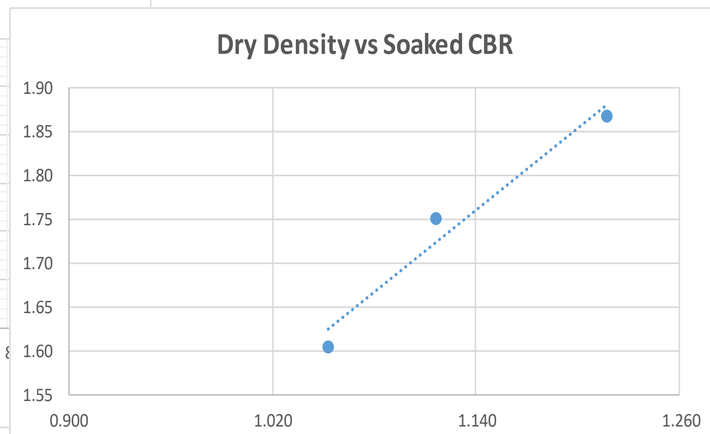


Dry Density at 95% of MDD:					1.06
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	36.9	0.821	1.76	74	
30	35.8	0.943	2.14	85	
65	38.3	1.020	2.26	92	
CBR (%) @ 95 % MDD			2.3	% Swell	9.46

20%QD+80%SOIL						
CBR TEST (AASHTO T 193-93)						
Compaction Determination						
COMPACTION DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	N4	N4	M10	M10	A1	A1
Mass of wet soil + Mould, g	10894.7	11845.3	10653.4	11592.8	10389.3	11341.6
Mass Mould, g	7029.5	7029.5	6948.1	6948.1	6996.3	6996.3
Mass of Soil, g	3865.2	4815.8	3705.3	4644.7	3393	4345.3
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.82	2.27	1.744	2.187	1.597	2.046
Dry density of soil, g/cc	1.575	1.438	1.512	1.445	1.378	1.346
Moisture Determination						
MOISTURE CONTENT DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	A16	P65	T1	A3	C15	J41
Mass of wet soil + Container, g	204.88	217.47	216.5	214.7	253.1	246.5
Mass of dry soil + Container, g	181.95	150.62	191.4	153.0	221.8	170.9
Mass of container, g	34.75	34.78	28.3	32.91	25.34	25.49
Mass of water, g	22.93	66.85	25.1	61.7	31.2	75.6
Mass of dry soil,g	147.2	115.84	163.1	120.1	196.5	145.4
Moisture content, %	15.6	57.7	15.4	51.4	15.9	52.0

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		Surcharge Weight:-4.55 KG				
	65 Blows		30Blows		10Blows	
Pen.mm	Load, KN	CBR%	Load, KN	CBR%	Load, KN	CBR %
0.00	0.00		0.00		0.00	
0.64	0.09		0.08		0.07	
1.27	0.10		0.10		0.09	
1.91	0.17		0.15		0.12	
2.54	0.26	1.97	0.24	1.82	0.15	1.14
3.81	0.33		0.29		0.15	
5.08	0.35	1.71	0.32	1.56	0.18	0.87
7.62	0.38		0.36		0.23	
Modified Max.Dry Density g/c	1.51			OMC %	20.37	
Swell Determination						
Date	65 Blows		30 Blow		10Blows	
	Gauge reading (mm)	Swell in %	auge reading(mr	Swell in %	uge reading(m	Swell in %
06/01/2014 (Initial)	**0.52**	30.66	0.25	10.26	**0.41**	21.92
10/01/2014 (Final)	36.21		12.19		25.92	

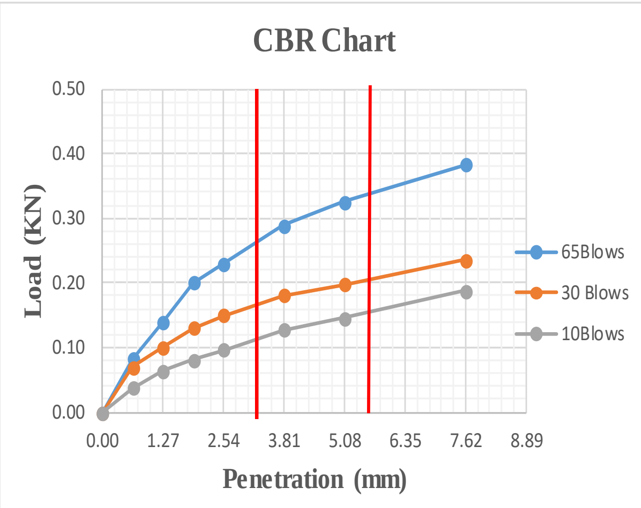


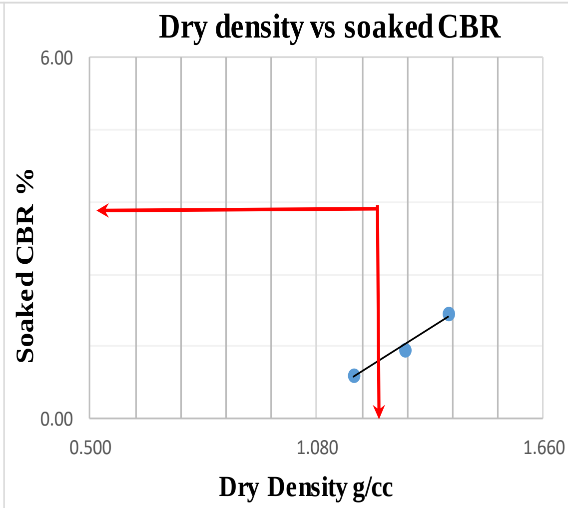


Dry Density at 95% of MDD:					**1.44**
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	15.9	1.378	1.14	91	
30	15.4	1.512	1.82	100	
65	15.6	1.575	1.97	104	
CBR (%) @ 95 % MDD			1.6	% Swell	20.95

10%SD+10%QD+SOIL						
CBR TEST (AASHTO T 193-93)						
Compaction Determination						
COMPACTION DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	N8	N8	30Blows	30Blows	I65	I65
Mass of wet soil + Mould, g	10236.6	10502.8	10223.3	10664.4	9906.1	10577
Mass Mould, g	6936.2	6936.2	6938.9	6938.9	6958.5	6958.5
Mass of Soil, g	3300.4	3566.6	3284.4	3725.5	2947.6	3618.5
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.55	1.68	1.546	1.754	1.388	1.704
Dry density of soil, g/cc	1.226	1.257	1.212	1.357	1.113	1.302
Moisture Determination						
MOISTURE CONTENT DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	G3T3	P65	G63	J41	A3	A3
Mass of wet soil + Container, g	206.51	186.29	147.7	199.3	200.5	203.0
Mass of dry soil + Container, g	178.84	138.44	126.6	139.8	169.6	142.5
Mass of container, g	37.83	25.3	34.8	28.31	37.67	36.04
Mass of water, g	37.71	37.95	25.3	32.6	32.5	32.8
Mass of dry soil,g	141.01	113.14	91.8	111.4	131.9	106.5
Moisture content, %	26.7	33.5	27.6	29.2	24.6	30.8

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		Surcharge Weight:-4.55 KG				
	65 Blows		30Blows		10Blows	
Pen.mm	Load, KN	CBR%	Load, KN	CBR%	Load, KN	CBR %
0.00	0.00		0.00		0.00	
0.64	0.10		0.07		0.05	
1.27	0.15		0.16		0.09	
1.91	0.92		0.18		0.12	
2.54	0.24	1.82	0.24	1.84	0.20	1.50
3.81	0.29		0.26		0.21	
5.08	0.37	1.80	0.35	1.70	0.27	1.33
7.62	0.58		0.48		0.39	
Modified Max.Dry Density g/cc	**1.20**			**OMC %**	**27.70**	
Swell Determination						
Date	**65 Blows**		**30 Blow**		**10Blows**	
	uge reading (n	**Swell in %**	**auge reading(m**	**Swell in %**	**ıge reading(ı**	**Swell in %**
06/01/2014 (Initial)	**0.42**	8.54	0.40	10.72	**0.3**	10.95
10/01/2014 (Final)	10.36		12.88		13.04	



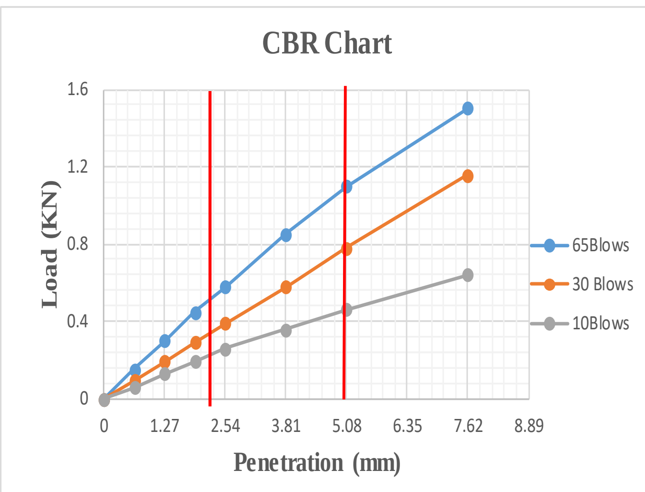


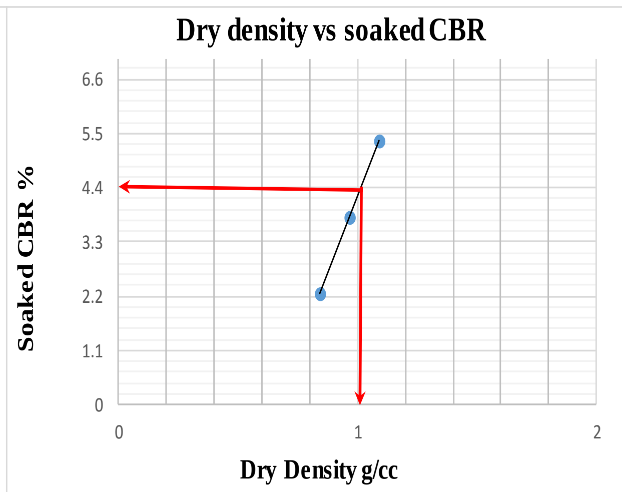
Dry Density at 95% of MDD:					1.137
No.of blows	**MCBS %**	**DDBS g/cm3**	Corrected CBR %	% Compaction	
10	24.6	1.113	1.50	93	
30	27.6	1.212	1.84	101	
65	26.7	1.226	1.82	102	
CBR (%) @ 95 % MDD			**2.0**	**% Swell**	**10.07**

20%SD+20%QD+SOIL						
CBR TEST (AASHTO T 193-93)						
Compaction Determination						
COMPACTION DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	CB1	CB1	M65	M65	30BLOWS	30BLOWS
Mass of wet soil + Mould, g	9779.9	10212	9739.1	10284.6	9340.4	10073.4
Mass Mould, g	6943.6	6943.6	6961.1	6961.1	6950	6950
Mass of Soil, g	2836.3	3268.4	2778	3323.5	2390.4	3123.4
Volume of Mould, cc	2124	2124	2124	2124	2124	2124
Wet density of soil, g/cc	1.34	1.54	1.308	1.565	1.125	1.471
Dry density of soil, g/cc	1.089	0.892	0.966	0.957	0.844	0.811
Moisture Determination						
MOISTURE CONTENT DATA	65Blows		30Blows		10Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	P65	A3	J41	P65	A3	J41
Mass of wet soil + Container, g	139.16	168.07	162.5	157.5	147.1	172.4
Mass of dry soil + Container, g	120.5	111.2	128.7	107.3	118.5	110.1
Mass of container, g	38.22	32.8	33.4	28.31	32.80	33.38
Mass of water, g	18.66	56.87	33.8	50.2	28.6	62.3
Mass of dry soil,g	82.28	78.4	95.3	79.0	85.7	76.7
Moisture content, %	22.7	72.5	35.4	63.5	33.4	81.2

CBR Penetration Determination						
Penetration after 96 hrs Soaking Period		Surcharge Weight:-4.55 KG				
	65 Blows		30Blows		10Blows	
Pen.mm	Load, KN	CBR%	Load, KN	CBR%	Load, KN	CBR %
0.00	0.00		0.00		0.00	
0.64	0.16		0.09		0.06	
1.27	0.31		0.19		0.13	
1.91	0.46		0.29		0.19	
2.54	0.63	4.77	0.39	2.95	0.26	1.97
3.81	0.87		0.59		0.37	
5.08	1.10	5.35	0.76	3.70	0.48	2.34
7.62	1.50		1.16		0.64	
Modified Max.Dry Density g/cc	**1.20**			**OMC %**	**18.63**	

Swell Determination						
Date	65 Blows		30 Blow		10Blows	
	Gauge reading (mm)	Swell in %	Gauge reading(mm)	Swell in %	uge reading(m	Swell in %
20/01/2014 (Initial)	**0.4**	4.73	0.60	6.22	**0.35**	8.09
24/01/2014 (Final)	5.9		7.84		9.77	





Dry Density at 95% of MDD:					1.14
No.of blows	MCBS %	DDBS g/cm3	Corrected CBR %	% Compaction	
10	33.4	0.844	2.34	70	
30	35.4	0.966	3.70	81	
65	22.7	1.089	5.35	91	
CBR (%) @ 95 % MDD			**6.0**	**% Swell**	**6.35**