JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

Utilization of Crushed stone Dust as a Stablizer for Sub Grade Soil: A Case Study In Jimma Town

A Thesis Submitted To The School of Graduate Studies Of Jimma University In Partial Fulfillment Of The Requirements For The Degree Of Masters Of Science In Highway Engineering

By: Abubekir Jemal Hussen

January, 2019
Jimma, Ethiopia

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Advisor: Elmer C.Agon

Co-Advisor: Engr.AntenehGeremew

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DECLARATION

I, the undersigned, declare that the work in this thesis entitled **"Utilization of Crushed Stone Dust as a Stabilizerfor Sub Grade Soil: A Case Study In Jimma Town"** has been performed by me in School of Civil and Environmental Engineering, Department of Civil Engineering, under the supervision of main-advisor Professor. Elmer C.Agon and co-advisor Engr. AntenehGeremew. The information derived from literature has been duly acknowledged in the text and list of reference provided.No part of this thesis was previously presented for another degree at any university.

Name:Abubekir Jemal Hussen Signature__________________

Date ____/_____/_______

This thesis has been submitted for examination with my approval as university supervisor.

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ABSTRACT

Aggregate crushing industry is candle of all construction projects. Crushed stone Dust is material obtained from aggregate crushing industries. Use of such stone dust materials creates lots of problems in environment and public due to excess storage and dust nusance. Considering this aspect an experimental study was conducted on expansive soil by mixing it with locally available crushed stone dust. This paper reflects the visionary light on the suitability of crusher dust as soil stabilizer for use in pavement construction. The role of crusher dust in improving the characteristics of expansive sub grade material is analysed. The amount of cost savings for a pavement when it is stabilized with crusher dust is also studied.

In order to realize the desiered objective a purposive sampling techniques wich is non probability method was adopted. Inorder to collect disturbed soil sample at depth of 1.5m at Ginjo kebele around Honey land hotel and a crushed stone dust from aggregate production area for the preparation of different lab tests.

The lab work involves sieve analysis along the consistence test to classify the soil samples. The prelemenary inveastigation of the soil shows that it belongs to A-7-5 class of soil in AASHTO & CH in USCS. Soil under this class was poor for sub grade constraction. Atterberg limit,compuction and CBR test were used to evaluate the properties of stabilized soil. The soil stablezed with the crushed stone dust in stepped constraction of 0,5,10,15,20,25,30,35,40,45 & 50 % by dry weight of soil. The analysis of the result shows the addition of crushered stone dust improve the geotechnical properties of soil. The addition of crushed stone dust reduces PI,Swelling and the optimum moisture content with an increase in MDD& CBR with an increase of crushed stone dust.A considerable amount of cost savings is also possible when the expansive clay soil is stabilized with crusher dust.

***Key Words:* Expansive Sub-Grade Soil, Crushed Stone Dust, Stabilization, CBR, Swelling Index, Cost**

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ACRONOMY

AADT	Annual Average Daily Traffic
AASHTO	American Association Of Highway And Transportation Officials
ASTM	American Society For Testing And Materials
CBR	California Bearing Ratio
CD	Curusher dust
C_C	Coefficient of curvature
C_U	Coefficient of uniformity
D_{10}	Grain diameter at 10% passing
D_{30}	Grain diameter at 30% passing
D_{60}	Grain diameter at 60% passing
ERA	Ethiopian Roads Authority
LL	Liquid Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
PI	Plastic Index
PL	Plastic Limit
SP	Swelling Pressure
USCS	Unified Soil Classification System
USA	United States Of America

CHAPTER ONE

INTRODUCTION

1.1. Background

Scarcity of buildable land with high bearing capacity, enforcing high way agency to construct road on weak sub grade that multiplying performance of road by zero. Performance of flexible pavement depends on cumulative performance of its layers. Road foundation with problematic soil not only reduces the life of pavement, also it creates problems during construction and maintenance operation which firing the bullet of design to the target of uneconomical design. Therefore to reduce the pavement failure it is important to hamper volume change under variation of moisture so that improving engineering properties since all properties of soil are affected by moisture content. Volume change due to variation moisture occurs as result of plastic fine in soil mass. Therefore, improving plastic characteristics of soil mean improving soil strength which would be increase pavement life.

Expansive soil is one of the most abundant soils in Ethiopia, which mostly are creates problem on built structure. Over the past 13 years, 40% of the total road sector development expenditure in Ethiopia was allocated to rehabilitation and upgrading of trunk roads with additional 11% utilized to maintenance works alone [1]. This problem urges the need for wider application of cost effective and environmentally friendly technologies of improving soil properties, such as chemical stabilization, to be customized and adopted to the current road construction trend in the country.

Durability of a structure requires good foundation or foundation material to transfer load smoothly without causing any undue deformations. To meet the demands of the population lot of civil engineering structures like building, roads, embankments and others required for the need of the people. Structures constructed on poor grounds are subjected to failures due to settlements, which result in increase the maintenance cost. To increase the bearing strength of the ground as a foundation material and to reduce the plastic deformation due to presence of fines in the natural soils as fill materials, alternative materials like fly ash, pond ash, crusher dust etc., have been gaining importance now-a-days. Availability of these wastes in large quantities encourages the geotechnical engineers for their bulk utilization in construction activities in place of natural soils.

In the present investigation an attempt is made to study the performance of crusher dust as geo technical material in construction activities. Crusher dust has wider applications in the areas of infrastructural facilities as a retaining material without reinforcement, fill material in highway construction and others. In north coastal districts of Andhra Pradesh abundant quantities of crusher dust has been produced and its production is nearly 2.4 miliion tones per annum [2].

The availability of buildable land is fast drifting away each day due to scarcity of lands with good natural bearing capacity. This leads to construction of buildings on poor soils which eventually lead to structural foundation failures. It has become very imperative to improve soils or the quality of grounds by the adoption of suitable improvement methods depending on the materials available. However, during soil or ground improvement, cost effectiveness is one of the major factors considered cardinal. Consequent, there is a permanent need to adopt the use of admixtures during cement/soil improvement or stabilization. This necessitated the review on a very important admixture in geotechnical engineering and in cement stabilization of soils during pavement construction. However, crusher dust which is a waste product from aggregate production could replace some proportions of sand/soil. This admixture not only replaces some proportions of soil for cost effective soil improvement but according to researches carried over the years on this waste product, improves the geophysical properties of the joint mixture; cement/soil/quarry dust. Since the introduction of crusher dust improves the engineering behaviour of soils, this review work exposes those qualities and applications that make quarry dust a good replacement or admixture during soil improvement and for a more economic approach [3].

Aggregate crusher units produce enormous quantities of quarry dust, a waste product, produced during crushing of rubble. Stacking or disposal of such large quantities of this waste is a serious environmental problem and health hazard to both plants and animals. Thus there is an urgent need to explore the possibility for an effective utilization of this waste material. Due to the increasing cost of high quality materials needed for different geotechnical projects, engineers try to improve the physical properties of local soils through different methods and techniques. The word improvement means to increase the shear strength, reducing settlements, resists harsh environment conditions and decreases or eliminates all problems associated with weak soils**[4]**.

The addition of chruser dust not only improve the swelling nature but also increases the CBR value which in turn reduces the thickness of pavement. The total pavement thickness can be reduced by replacement of clayey soil with chruser dust [5].

1.2. Statement of the Problem

The swell and shrinkage distinctiveness of expansive soil causes significant damage to structures such as buildings and pavements. This damage can be attributed to moisture fluctuations caused by seasonal variations. Volumetric changes weaken the subgrade by inducing cracking which metes out damage to the overlying structures. A vast majority of the expansive soils are montmorillonite-rich clays, over consolidated clays and shale's [6].

Engineering problems related to expansive soils have been reported in many countries of the world as 3% of the world land area but are generally most series in arid and semi-arid regions. As a result, highly reactive soil undergo substantial volume changes associated with shrinkage and swelling process. Consequently, many engineering structures suffer severe distress and damage. Cracked foundations, pavement, floors and basement wall are topically types of damage done by swelling soils. Every year they cause billions of dollars in damage. Expansive soil are not as dramatic as hurricane or wide areas rather than being constructed in a small locality [7,8].

The above problems are extensively occurring in Ethiopia. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million hectar[9]. They are widely spread in the central part of Ethiopia following the major truck roads like Addis-Ambo, Addis-Wolliso, Addis–Debrebirhan, Addis-Gohatsion, and Addis-Modjo are covered by expansive soils. Also, areas like Mekele and Gambella are covered by expansive soil. Soil stabilization is the alteration of one or more soil properties, by mechanical or chemical means, to create an improved soil material possessing the desired engineering properties. The process may include the blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder for cementation of the soil [10].

Performance of Flexible Pavement depends on the functions of the component layers especially Sub grade. Sub grade is compacted layer of soil provide the lateral support to the pavement. Frequently natural soils composed of high amount of fines which causes plasticity characteristics with adsorption of moisture under heavy loads and repeated traffic. Excess deformation leading several failure which require huge investment of money for their repairs. To reduce the excess deformation of the soils and to increase the life period of the pavement there is a need to arrest their plastic characteristics and stabilization is one such technique to improve the natural soils by addition of industrial wastes. Accordingly roads in Jimma zone experienced many types of failures such as cracks, large surface deformation and structural deformation of pavement layers and the sub grade. Therefore, to prevent the problems, it is essential for engineers to stabilize the existing weak soils before commencing the construction activities. Thus, one method to ensure that existing natural soil improved and suitable for construction is by mixing it with crusher dust as a cost effective stabilizer and locally available material.

1.3. Research questions

- What are the engineering properties of expansive soil and the crushed stone dust of the study area?
- How much percentage of crusher dust added to improve soil strength?
- How much percentage of cost saved for sub grade formation using stabilizer such as crusher dust?
- What was the laboratory test result compared with standard specification?

1.4. Objectives

1.4.1. General Objective

- The main objective of this study was utilization of crushed stone dust as astabilizer for sub-grade soil.

1.4.2.Specific Objectives

- To addentify the engineering properties of expansive soil and the crushed stone dust of the study area
- To determine the optimum crusher dust percentage to be added.
- To quantify the amount of cost savings for sub grade formation using crusher dust stabilizer.
- To compare the laboratory test result with standard spesification

1.5 Significance of the study

For sub-grade and foundation preparation, particularly in the road construction sector, expansive soil couses an increase in initial cost of construction due to the need of improving strength of expansive soil to use as a foundation for pavement structure. Cement and lime stabilization method is commen methods to improve properties of expansive soil. This research will serve as areference guide for practicing crusher dust as stabilizer. This is useful in the sense that, it will reduce initial costs of road construction.

1.6 Scope of the study

The Jimma town which is known to abundance of soft soil, experiencing many types of failures such as cracks, large surface deformation and structural deformation of pavement layers and the sub grade.To reduce the excess deformation of pavement layers and to increase the life period of the pavement there is a need to arrest plastic characteristics of soils. Lime stabilization is one such technique to improve the soft soils but it is expensive. Therefore, this research provides insight in to crusher dust which is a problem of aggregate crushing industry due to cost of disposal and impact on environment. To reduce those problem of aggregate crushing industry and to improve performance of sub grade soil with low cost it’s important to utilize crusher dust as stabilizer of high plastic soil.

CHAPTER TWO
LITERATURE REVIEW

2.1. Introduction

Pavement design is based on the premise that specified levels of quality will be achieved for each soil layer in the pavement system. So that each layer must resist shearing within the layer, avoid excessive elastic deflections, and prevent excessive permanent deformation through densification. The quality of the material in each layers pavement should meet the specific requirement. However, In many instances fulfilling the requirements is challenging due to; The absence of quality material in the project vicinity, The higher cost of transporting quality materials ,The need of using the local material which are weak or reclaimed materials that cannot be dumped due to environmental and other reasons. Hence, improving the property of the available material will become mandatory or economical.

2.2. Pavement Performance

Performance is a general term for how pavements change their condition or serve their intended function with accumulating use. Performance is defined by the distress, loss of serviceability index and skid resistance, loss of overall condition, and by the damage that is done by the expected traffic. The deterioration accumulates with the passage of time and results in failure of the pavement structure. There are two types of pavement distress or failure [11].

The first is a structural failure, in which a collapse of the entire structure or a breakdown of one or more of the pavement components renders the pavement incapable of sustaining the loads imposed on its surface. The second type of failure is a functional failure; it occurs when the pavement, due to its roughness, is unable to carry out its intended function without causing discomfort to drivers or passengers or imposing high stresses on vehicles. The cause of these failure conditions may be due to inadequate maintenance, excessive loads, climatic and environmental conditions, poor drainage leading to poor subgrade conditions, and disintegration of the component materials.

2.2.1. Swelling and Shrinkage in Road Performance

The mechanism of swelling in expansive soil is complex and is influenced by a number of factors. Expansion is the result of changes in the soil, water system that disturbs the internal force equilibrium. There must be a potential gradient, which can cause water migration and a continuous passage throughwhich water transfer can take place to cause volumetric expansion. Fractures and fissures, shrinkage cracks, capillary rise, vapor transfer, thermal gradients, etc. are some of the sources that cause moisture migration and swelling of expansive soils [8].In General, the movement of expansive soil occurs in uneven pattern and the resulting expansion is a magnitude that cannot be predicted by the classical elastic plastic theory [12].However,theswellingbehavior can be basically related to the combined effect of interacting factors that can be grouped into:

- Engineering properties
- Local Geology
- Local Environment of deposition.

The main geological factors include the rock type and age as related to the type and amount of clay minerals, type and amount of cementing materials and the soil particle arrangement. The engineering factors include the moisture content, Atterberg limits, and the dry density. The environmental factors include the confining pressure, type and degree of weathering as related to the amount of clay fraction, initial water content and water. Thickness andlocation of potentially expansive layers into a profile considerably influence potential movement. The movements are higher around the ground surface and decrease with depth. Less movement will occur if the expansive soil is overlain by non-expansive material or have got shallow depths[12].

2.3. Soil properties

To have an understanding of the soil action, the engineer must be familiar with certain basic properties of the soil. It must be remembered that the properties of any given soil depend not only on its general type but also on its condition at the time when it is being examined.

Atterberg Limits

The Atterberg limits are a basic measure of the nature of a fine-grained soil. They are used to distinguish between silt and clay, and they can distinguish between different types of silts and clays. They cover a range of soil tests relating to reactivity to moisture (water), better known as Plasticity. The amount of water a soil sample can absorb before changing from a solid to semisolid, a plastic and then to a liquid state is a very important indication of 1) whether the soil is mainly silt or clay, and 2) if it is clay, the characteristics of that particular clay minerals in the sample. In each state the consistency and behavior of a soil is different and thus so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soils behaviour [12].

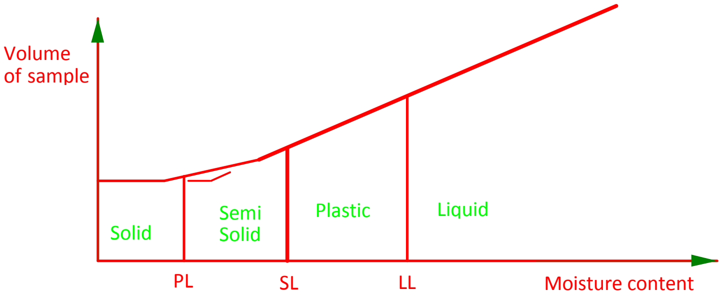


Figure 2.1Consistance limit of soil[12]

Shrinkage Limit:

This limit is achieved when further loss of water from the soil does not reduce the volume of the soil. It can be more accurately defined as the lowest water content at which the soil can still be completely saturated.

Plastic Limit:

This limit lies between the plastic and semi-solid state of the soil. It is determined by rolling out a thread of the soil on a flat surface which is non-porous. It is the minimum water content at which the soil just begins to crumble while rolling into a thread of approximately 3mm diameter.

Liquid Limit:

It is the water content of the soil between the liquid state and plastic state of the soil. It can be defined as the minimum water content at which the soil, though in liquid state, shows small shearing strength against flowing. It is measured by the Casagrande's apparatus

Particle Size Distribution

Soil at any place is composed of particles of a variety of sizes and shapes, sizes ranging from a few microns to a few centimeters are present sometimes in the same soil sample. The distribution of particles of different sizes determines many physical properties of the soil such as its strength, permeability, density etc.

Particle size distribution is found out by two methods, first is sieve analysis which is done for coarse grained soils only and the other method is sedimentation analysis used for fine grained soil sample. Both are followed by plotting the results on a semi-log graph. The percentage finer N as the ordinate and the particle diameter i.e. sieve size as the abscissa on a logarithmic scale. The curve generated from the result gives us an idea of the type and gradation of the soil. If the curve is higher up or is more towards the left, it means that the soil has more representation from the finer particles; if it is towards the right, we can deduce that the soil has more of the coarse grained particles[12].

The soil may be of two types- well graded or poorly graded (uniformly graded). Well graded soils have particles from all the size ranges in a good amount. On the other hand, it is said to be poorly or uniformly graded if it has particles of some sizes in excess and deficiency of particles of other sizes. Sometimes the curve has a flat portion also which means there is an absence of particles of intermediate size, these soils are also known as gap graded or skip graded. For analysis of the particle distribution, we sometimes use D_{10}, D_{30}, and D_{60} etc. terms which represents a size in mm such that 10%, 30% and 60% of particles respectively are finer than that size. The size of D_{10} also called the effective size or diameter is a very useful data. There is a term called uniformity coefficient C_u which comes from the ratio of D_{60} and D_{10}, it gives a measure of the range of the particle size of the soil sample[12].

Specific gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words we can define it as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than equal volume of water. Different types of soil have different specific gravities, general range for specific gravity of soils.

Table 2.1 Specific gravities of soils [13]

Sand	2.63-2.67
Silt	2.65-2.7
Clay and Silty clay	2.67-2.9
Organic soil	<2.0

Strength bearing capacity

Bearing capacity is the capacity of soil to support the loads applied to the ground. The bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil.

A pavement, like any other engineering structure, is designed to withstand certain loads. In this case the primary load that needs to be considered in the design is the traffic spectrum that will be carried by the road. The bearing capacity of a structurally well balanced pavement structure increases evenly with increasing depth (cover) over the subgrade material.

Compaction

Compaction is the process whereby the volume of air in the soil is reduced. The compaction is normally achieved through the use of compaction equipment. During this process solid particles become more closely spaced. This reduction of air volume in a mixture produces a corresponding Increase in material unit weight, or density. Compaction is the greatest determining factor in dense graded pavement performance. Inadequate compaction results in a pavement with decreased stiffness, reduced fatigue life, accelerated aging / decreased durability, rutting, and moisture damage. The extent of compaction depends on the moisture content of the soil and the comp active effort used.

Permeability

The ability of water to flow through a soil (under action of gravity or applied force) is referred to as the soil's permeability. Different soil types have varying degrees of permeability. It is generally the pore sizes and their connectivity that determines whether a soil has high or low permeability. Water will flow easily through soil with large pores with good connectivity between them. Small pores with the same degree of connectivity would have lower permeability, because water would flow through the soil more slowly. The permeability of gravel is higher than that of clay.

Swell

A soil increases in volume when it is excavated because soil grains are loosened during excavation and air fills the void spaces created. As a result, a unit volume of soil in the bank condition will occupy more than one unit volume after excavation. This phenomenon is called and must be taken into account when assessing the amount of transport required, for costing and construction purposes.

Shrinkage

Shrinkage is the term used to describe reduction of the volume of a material that has been excavated when it is used as fill in an embankment. A small proportion of this loss may be attributed to spillage during transport from the cut to the fill, but the main loss occurs because the bank volume of the material is greater when in its natural state before being excavated than the compacted volume of the same material after it has been used to form an embankment. This shrinkage factor must be determined for the material concerned and included in the calculations of the earthworks cost estimate and claims for payment. The factor is usually not applicable to rock but significant for most soils.

Table 2.2 Shrinkage factor of soils [13]

Soil type	Condition Representing 1 m^3	Altered condition (m^3)		
		Bank	Loose	Compacted
Sand	Natural state	1	1,11	0 , 95
	Loose	0,9	1	0 , 86
	Compacted	1,05	1,27	1
Average soil	Natural state	1	1,35	0 , 81
	Loose	0,8	1	0 , 72
	Compacted	1,22	1,29	1
Clay	Natural state	1	1,43	0 , 9
	Loose	0,7	1	0 , 63
	Compacted	1,11	1,59	1

2.4. Evaluation of the swelling potential of Expansive soil

Swelling pressure is defined as the pressure required for preventing volume expansion in soil in contact with water [14]. The swell potential of a soil is a measure of the ability and the degree to which a soil might swell if its environment were to be changed in some definite way. Moisture content alone is not a good indicator of shrink - swell potential. Instead, the moisture content relative to limiting moisture contents such as the plastic limit and shrinkage limit must be known. Water content changes below the shrinkage limit produce little or no change in volume. The availability of water to an expansive soil profile is influenced by many environmental factors and man-made factors. Generally the upper few meters of the soil profiles are subject to the widest ranges of potential moisture variation and also overburden stress is low and the soil is not restrained against movement at shallow depth. The swell potential depends on the following factors which influence the volume change:

- Mineral type and Amount
- Density
- Surcharge loads
- Soil structure, time and water content

The differential free swell may also be expressed by the term ’free swell index’. The ‘potentialexpansivity’ PE, or the “degree of expansion” and consequent damage to structures with light loading are qualitatively judged from the Atterberg limit and free swell tests.

2.4.1. Swelling potential based on plasticity index and liquid limit

The plasticity Index and Liquid limit are useful indices for determining the swelling characteristics of most clays, since the liquid limits and the swelling of clays both depends on the amount of water a clay tries to absorb [14,15]. A soil sample with liquid limit exceeding 70% and plasticity index greater than 35% is judged to have a very high potential swell. The swelling potential of a soil can be estimated from linear shrinkage in combination with shrinkage limits [12]. They propose values given below to classify the given soil swell potential.

Table 2.3 Potential swell based on plasticity Linear [13]

Classification ofpotential swell	Liquid limit (LL), %	Plasticity index (PI), %	Shrinkage limit (SL), %
Low	20-35	< 18	>15
Medium	35 - 50	15 – 28	10 - 15
High	50 - 70	25 – 41	7 - 12
Very high	>70	> 35	< 11

2.5. Practical Problems of Highway Construction on Expansive clay Soil.

2.5.1. General

Black cotton soils are inorganic clays of medium to high compressibility. They are characterized by high shrinkage and swelling properties. Because of its high swelling and shrinkage characteristics, the Black cotton soils have been a challenge to the highway engineers. The Black cotton soils are very hard when dry, but loses its strength completely when in wet condition.It is observed that on drying, the black cotton soil develops cracks of varying depth. As a result of wetting and drying process, vertical movement takes place in the soil mass. All these movements lead to failure of pavement, in the form of settlement, heavy depression, cracking and unevenness. This article covers highway construction in Black cotton soils and also describes a case history of highway construction in highway construction in Black cotton soils[16].

2.5.2. Black Cotton Soil Peculiar Characteristics

Black cotton soil is a highly clayey soil. It is so hard that the clods cannot be easily pulverized for treatment for its use in road construction. This poses serious problems as regards to subsequent performance of the road. Moreover, the softened sub grade has a tendency to up heave into the upper layers of the pavement, especially when the sub-base consists of stone soling with lot of voids. Gradual intrusion of wet Black cotton soil invariably leads to failure of the road.

The roads laid on Black cotton soil bases develop undulations at the road surface due to loss of strength of the sub grade through softening during monsoon. The black color in Black cotton soil is due to the presence of titanium oxide in small concentration. The Black cotton soil has a high percentage of clay, which is predominantly montmorillonite in structure and black or blackish grey in color. The physical properties of Black cotton soil vary from place to place. Due to its peculiar characteristics, it forms a very poor foundation material for road construction. Soaked laboratory CBR values of Black Cotton soils are generally found in the range of 2 to 4%. Due to very low CBR values of Black cotton soil, excessive pavement thickness is required for designing for flexible pavement. research& revelopment efforts have been made to improve the strength characteristics of Black cotton soil) with new technologies[16].

2.5.3. Problems of Highway Construction in Expansive Soil Areas

Problems Arising out of Water Saturation

It is a well-known fact that water is the worst enemy of road pavement, particularly in expansive soil areas. Water penetrates into the road pavement from three sides viz. top surface, side berms and from sub grade due to capillary action. Therefore, road specifications in expansive soil areas must take these factors into consideration. The road surfacing must be impervious, side berms paved and sub grade well treated to check capillary rise of water. It has been found during handling of various road investigation project assignments for assessing causes of road failures that water has got easy access into the pavement. It saturates the sub grade soil and thus lowers its bearing capacity, ultimately resulting in heavy depressions and settlement. In the base course layers comprising of Water Bound Macadam, water lubricates the binding material and makes the mechanical interlock unstable. In the top bituminous surfacing, raveling, stripping and cracking develop due to water stagnation and its seepage into these layers[17].

Design Problems in Black cotton soil

CBR method developed in USA is generally used for the design of crust thickness. This method stipulates that while determining the CBR values in the laboratory and in the field, a surcharge weight of 15 kg and 5 kg per 62 mm and 25 mm thickness respectively should be used to counteract the swelling pressure of Black cotton soils. BC soils produce swelling pressure in the range of 20-80 tons/m2 and swelling in the range of 10-20%.Therefore, CBR values obtained are not rational and scientific modification is required for determining CBR values of expansive soil[17].

Having heavy-duty traffic of 4500 commercial vehicles per day and msa 150 as generally found on our National Highways and taking CBR value of 2%, total crust thickness of flexible pavement works out to 830 mm which is practically an impossible preposition. It is felt that CBR design curves require modification for expansive soils.Assuming heavy traffic intensity of 4500 commercial vehicles per day and msa 150, crust thickness of rigid pavement works out approximately 300-320 mm, which is about one third of thickness needed for flexible pavement. Therefore, it sounds reasonable to adopt cement concrete pavement in Black cotton soil areas. This type of pavement may save the engineers from day to day maintenance problems also. Another approach to the problem can be in having semi rigid sub-bases. It is suggested that the CBR value of the BC soil be improved by giving a suitable treatment with the appropriate technology and then work out the crust thickness. This will substantially reduce the required crust thickness. Uncompacted berms without any treatment cannot withstand the traffic stresses. It is a common sight and experience that heavy vehicles get stuck up while overtaking and sometimes results in serious accidents. Development of separate specifications for berms need to be evolved[17].

2.6. Construction Practices on Expansive clay soil

The construction of roadways often requires traversing areas that contain materials that are unsuitable for the sub grade soils that lie beneath the pavement. These materials can be expansive, highly plastic, soft, wet, and/or weak. The exact nature of potential construction problems depends on whether or not the natural grade is to be excavated or if an embankment is to be constructed. The supporting soils may be susceptible to Excessive consolidation, shrinking and/or swelling with changes in moisture conditions or heave - induced volume changes due to the excavation of overlying soils, i.e. a cut section [6]. When poor sub grade soils are encountered, four approaches are taken, individually or in combination. These approaches are:

- ✓ Remove and replace of weak sub grade soil
- ✓ Apply mechanical and chemical stabilization
- ✓ Employ Reinforcement Geosynthetics
- ✓ Install subsurface drainage using vertical or horizontal drainage elements.

2.7. Soil Stabilization

2.7.1 Definition

Stabilization is defined as the process of altering the properties of sub-grade and pavement materials either by blending and improving particle gradation (mechanical stabilization) or by using stabilizing additives to meet the specified engineering properties (Chemical Stabilization). Soils are generally stabilized to increase their strength and durability or to prevent erosion and dust formation in soils. The main aim is the creation of a soil material or system that will hold under the design use conditions and for the designed life of the engineering project. The properties of soil vary a great deal at different places or in certain cases even at one place; the success of soil stabilization depends on soil testing. Various methods are employed to stabilize soil and the method should be verified in the lab with the soil material before applying it on the field[1].

Principles ofSoil Stabilization:

- Evaluating the soil properties of the area under consideration.
- Deciding the property of soil which needs to be altered to get the design value and choose the effective and economical method for stabilization.
- Designing the Stabilized soil mix sample and testing it in the lab for intended stability and durability values.

2.7.2 Needs & Advantages

Soil properties vary a great deal and construction of structures depends a lot on the bearing capacity of the soil, hence, we need to stabilize the soil which makes it easier to predict the load bearing capacity of the soil and even improve the load bearing capacity. The gradation of the soil is also a very important property to keep in mind while working with soils. The soils may be well-graded which is desirable as it has less number of voids or uniformly graded which though sounds stable but has more voids. Thus, it is better to mix different types of soils together to improve the soil strength properties. It is very expensive to replace the inferior soil entirely soil and hence, soil stabilization is the thing to look for in these cases [1].

- It improves the strength of the soil, thus, increasing the soil bearing capacity.
- It is more economical both in terms of cost and energy to increase the bearing capacity of the soil rather than going for deep foundation or raft foundation.

- It is also used to provide more stability to the soil in slopes or other such places.
- Sometimes soil stabilization is also used to prevent soil erosion or formation of dust, which is very useful especially in dry and arid weather.
- Stabilization is also done for soil water-proofing; this prevents water from entering into the soil and hence helps the soil from losing its strength.
- It helps in reducing the soil volume change due to change in temperature or moisture content.
- Stabilization improves the workability and the durability of the soil.

2.7.3 Methods of Stablization

Mechanical method of Stabilization

In this procedure, soils of different gradations are mixed together to obtain the desired property in the soil. This may be done at the site or at some other place from where it can be transported easily. The final mixture is then compacted by the usual methods to get the required density [18].

Additive method of stabilization

It refers to the addition of manufactured products into the soil, which in proper quantities enhances the quality of the soil. Materials such as cement, lime, bitumen, fly ash etc. are used as chemical additives. Sometimes different fibers are also used as reinforcements in the soil [18]. The addition of these fibers takes place by two methods;

1. Oriented fiber reinforcement-

The fibers are arranged in some order and all the fibers are placed in the same orientation. The fibers are laid layer by layer in this type of orientation. Continuous fibers in the form of sheets, strips or bars etc. are used systematically in this type of arrangemen*t.*

2. Random fiber reinforcement-

This arrangement has discrete fibers distributed randomly in the soil mass. The mixing is done until the soil and the reinforcement form a more or less homogeneous mixture. Materials used in this type of reinforcements are generally derived from paper, nylon, metals or other materials having varied physical properties. Randomly distributed fibers have some advantages over the systematically distributed fibers. Somehow this way of reinforcement is similar to addition of admixtures such as cement, lime etc. Besides being easy to add and mix, this method also offers strength isotropy, decreases chance of potential weak planes which occur in the other case and provides ductility to the soil.

2.8. Stabilization of pavement materials

The stabilization of pavement materials is a widely used practice in road construction. Soil stabilization is a process by which a soil material is improved and made more stable [19].Soil stabilization as the treatment of natural soil to improve its engineering properties [20]. In general, soil stabilization is the process of creating or improving certain desired properties in a soil material so as to render it stable and useful for a specific purpose. Since the inception of this process of stabilization, most soil materials which have been thought not useful have found application in many areas of engineering[21]. The improvements in engineering properties caused by stabilization can include the following: increases in soil strength (shearing resistance), stiffness (resistance to deformation) and durability (wear resistance), reductions in swelling potential of wet clay soils and other desirable characteristics, such as dust proofing and water proofing unsealed roads[22]. Stabilization of soil is employed when it is more economical to overcome a deficiency in a readily available material than to bring in one that fully complies with the requirements of specification for the soil [23]. It has been regarded as a best option for upgrading marginal materials where no economic alternative is available. There are many techniques for soil stabilization, including compaction, dewatering and by adding material to the soil. Mechanical or granular stabilization is accomplished by mixing or blending soils to obtain a material meeting the required specifications[24].The soil blending may take place at construction site, or a borrow area. The blended material is then spread and compacted to required density by conventional means. This is the simplest method of stabilization. In general, if a soil is coarse grained (i.e. sandy gravel) requisite quantity of fine grained soil (i.e. cohesive soils) is added to adjust the proportion. Similarly, if the soil is fine grained then coarse grained is added [24]. Chemical stabilization has traditionally relied on Portland cement, lime and bitumen[19]. He reported that cement and bitumen are best suited for granular and non-plastic soils, while lime performs better in cohesive soils.

For this thesis emphasis will be given to mechanical stabilization of balack cotton for the selected study area around Jimma town. Black cotton soils are those that exhibit particularly large volumetric changes, both shrinkage and swell, due to variations in their moisture content. They exhibit poor bearing capacity and also some stability problems. When the subgrade is a particular black cotton soil, it may be necessary to stablize the expansive material with locally available stabilizing.

2.8.1. Mechanical stabilization

Mechanical Stabilization Is an improvement of an available material by blending it with one or more materials to improve the gradation and plasticity characteristics of the material. Materials produced by blending (mechanically stabilized) are still unbound granular materials their characterizations and testing are similar to the conventional granular materials. The mix design of mechanically stabilized material is based on;

- Gradation requirement
- Plasticity property requirements
- strength test requirements (CBR)

For safety of constructing it is necessary to improve the quality of ground by adoption of some suitable ground improvement materials and techniques. The method of ground improvement technique adopted depends on the soil to be treated and availability of materials required for improving the soil and also on the cost effectiveness. The use of quarry dust in soil stabilization is to improve engineering properties of soil. Quarry dusts are considered as one of the well-accepted as well as cost effective ground improvement for the stabilization of weak soil deposits. 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, sharp and angular particles and as such causes a gain in strength due to better interlocking [3].

2.9. Crusher dust stabilization

2.9.1. Over view Crusher dust / Stone dust

Crusher dust is a common by-product of mining and quarrying. Rather than being discarded as a waste material however, recycled crusher dust has many practical applications around the home and in construction. Using crusher dust in lieu of other materials can have resounding environmental and economical benefits. With fine particles like soft sand, crusher dust can be used as a cost-effective filling and packing material around water tanks; blended with natural sands to improve concrete shrinkage and water demand; and as a material to back-fill trenches with. It can also be used as a concrete aggregate used to create distinctive textures and as a substitute for concrete when creating pathways and driveways.

The production costs of crusher dust are relatively low compared to other building materials. Crusher dusts use less water than other alternatives and have excellent load bearing capabilities and durability. Crusher dust is fire and heat resistant; non-plastic; and alkaline when exposed to moisture, making it an ideal material to use in construction[7].

Characteristics of crusher dust

Every day, quarries move large amounts of stones and aggregate In the process of removing these materials from the earth and moving them, quarries create a large amount of dust that is made from very small stone particles, known as crusher dust. Crusher dust is also created when metals such as iron ore are separated from iron ore and the resulting slag is crushed into fine particles. Crusher dust looks much like sand but is made up of angular particles with a rough surface [25].

Some of the characteristics are:

- Consistent chemistry
- Excellent load bearing capacity
- Non-plastic
- Resistant to heat and fire
- Alkaline in presence of moisture
- Effective utilization of an industrial by-product conserving natural resources

2.10. Stabilization of sub grade by using Crusher dust

The comprehensive review of literature shows the related works done on expansive and murrum soil in and around the world. the properties of black cotton soil by replacement of quarry dust. The test results revealed that his compaction parameters and CBR values of the soil are improved substantially with the addition of the granite dust[26]. The combined effects of two industrial wastes flyash and quarry dust on, compaction characteristics, unconfined compressive strength, California bearing ratio (CBR) shear strength parameters and swelling pressure of an expansive soil have been discussed[27].Utilization of solid wastes like quarry dust not only protects the environment from degradation but also improves the engineering properties of the expansive soil. The disposal of which creates a lot of geo environmental problems. India and optimal percentage of crusher dust was found to be 40%.the effect of lime on some geotechnical properties of an expansive soil stabilized with optimum percentage of quarry dust has been described in the paper[28].The quarry dust/ crusher dust is obtained as solid wastes, during crushing of stones to obtain aggregates. The annual production of quarry dust is roughly around 200 million tons[29].

Out of the different quarry wastes, quarry dust is one, which is produced in abundance. Bulk utilization of this waste material is possible through geotechnical applications like embankments, back-fill material and sub-base material. Swelling and strength properties of expansive soil by using quarry dust and fly ash studied. From the experimental study he observed the combination of 20% stone dust and 25% fly ash addition at the optimum moisture content to the expansive soil is found to be a suitable measure to reduce the swelling and increase the strength of the two expansive soils tested[30]. Mixing of FDCS enhances the soaked CBR value, unconfined compressive strength and split-tensile strength [32].

In another work the index properties and unconfined strength of expansive soil when treated with fly ash and stone dust studied[33]. it results showed that when soil was treated with an optimum percentage of 20% -30% of admixture, the swelling of expansive clay could be controlled and also there is marked improvement in other properties of soil. the effect of lime on some geotechnical properties of an expansive soil stabilized with optimum percentage of quarry dust studded. It is concluded that addition of quarry dust decreases liquid limit, plastic limit and plasticity index but increases shrinkage limit of expansive soil. It also decreases the OMC but increases the MDD of the expansive soil. The addition of quarry dust to expansive soil decreases the cohesion and increases the angle of internal friction [34]. When crusher dust is added with expansive soil, it is expected that it will make it more porous, less durable, reduce cohesion., and also quarry dust has rough, sharpened angular particles and as such causes a gain in strength due to better interlocking[3]. the combined effect of fly ash and quarry dust on compaction characteristics, unconfined compressive strength, California bearing ratio(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% [35]. Crusher dust is mixed with high plastic gravels to reduce the excess deformation of the gravel soils and increase the life period of pavement. Addition of crusher dust reduced the plastic characteristics and improved the CBR value [36].

CBR value of black cotton soil – fly ash mixture increase up to an optimum fly ash content beyond which CBR value decreases [37]. The effect of quarry dust on CBR and angle of shearing resistance values steadily increases with increase in percentage of quarry dust[29].A study is conducted to know whether normal sand can be substituted by stone powder from stone crushing units in concrete and mortar. It is revealed from laboratory experiments that concrete made of stone powder and stone chip gained about 15% higher strength than that of the concrete made of normal sand and brick chip. It also shows that better mortar can be prepared by the stone powder [38]. Stone crusher dust has been used as a substitute for other construction activities. It is used as fine aggregate in concrete for paving blocks [39].In brick masonry, sand in cement mortar is substituted by crusher dust and investigation indicates that the crusher dust can replace natural sand completely in masonry construction with higher strength and cheaper cost [40].

The values at 30% stone dust are also full fill the requirement of granular sub base material but when we are getting our suitability of admixes on lower % of stone dust i. e. 25% then seeing to economy of construction the 25% stone dust is recommended as additive. As the percentage of stone dust additive increases from 10% to 25% the plasticity of the murrum stone dust mixture decreases from 23.2% to 20.14%. As the percentage of stone dust additive increases from 10% to 30% the MDD values of the murrum stone dust mixture increases from 1.70 gm/cc to 2.07 gm/cc and the corresponding OMC values decreases from 11% to 7.95%. As the percentage of stone dust additive increases the CBR values of the murrum stone dust mixture increases from 14.37% to 28.74%. As the percentage of stone dust additive increases from 10% to 30% the Gradation of the murrum stone dust mixture moves towards upper limits of gradation value and at maximum of 30% stone dust it just touches the upper limit of gradation. It is observed that the mixture of the murrum mixed with 25% of stone dust full fill the requirement for granular sub base [41].

The addition of the Quarry dust to the soil reduces the clay content and thus increases in the percentage of coarser particles, reduces the Liquid limit by 26.86% and plasticity index by 28.48% of unmodified soil. Optimum moisture content of soil is decreased by 36.71%, with increase in Percentages of Quarry dust. Maximum dry density of soil is increased by 5.88% by addition of (40%) Quarry dust. It is also identified that addition of (40%) Quarry dust yield high CBR value [42].

The total pavement thickness can be reduced from 615 mm to 540 mm by replacement of clayey soil with 30 % Quarry Dust. Minimum of 10 % replacement in clayey soil with quarry dust is required to arrest the swelling nature of the soil. As a whole the quantum of replacement of quarry dust is found to be in the range of 40% to 50 % in laying road pavements for the in-situ korattur clayey soil which is marginally higher. For economic considerations and for laying local pavements inside streets and villages 30% replacement of clayey soil can be sorted [5].

The effect of stone dust on geotechnical properties of poor soil and concluded that the CBR and MDD of poor soils can be improved by mixing stone dust. They also indicated that the liquid limit, plastic limit, plasticity index and optimum moisture content decrease by adding stone dust which in turn increases usefulness of soil as highway sub-grade material[43].

The decrease in optimum moisture contents are due to replacement of Silt and Clay particles by Crusher Dust particles which reduces the intake of moisture compared to Crusher Dust particles and increase in dry densities are due to occupation of more solids with respect to interaction of Crusher Dust and fines of gravel particles. Hence the optimum dosage of Crusher Dust for these types of Gravel soils is 10-20%.as the percentage of Crusher dust is increasing CBR values are increasing up to 15% for Anakapalli soils, 25% for Vizianagaram soils and 5-10% for Visakhapatnam Gravel soils. Attainment of maximum values are due to more solids occupied in the given volume due to the effective interaction between the Crusher Dust particles and Fine and coarser particles of Gravel soil, offers more shearing resistance against compression[44].

The plasticity, compaction and strength tests on gravel soil with various percentages of stone dust and found that by addition of stone dust plasticity characteristics were reduced and CBR of the mixes improved. Addition of 25-35% of stone dust makes the gravel soil meet the specification of morth as sub-base material. From the test results it is identified that as the percentage of crusher dust is increasing the optimum moisture content values are continuously decreasing, whereas the Maximum Dry Density values are continuously increasingupto 30-35% and then decreasing. The decrease in optimum Moisture Contents are due to replacement of Silt and Clay particles by Crusher Dust particles which reduces the intake of Moisture compared to Crusher Dust particles and increase in dry densities are due to occupation of more solids with respect to interaction of Crusher Dust and fines of gravel particles upto 30-35%. As the percentage of Crusher dust is increasing up to CBR values are increasing upto 35% and then decreasing. Attainment of maximum values at 30-35% doses are due to more solids occupied in the given volume due to the effective interaction between the Crusher Dust particles and Fine and coarser particles of Gravel soil, offers more shearing resistance against compression [27]. Consistency limit, standard compaction test, unconfined compressive test and CBR test and concluded that there is remarkable influence on strength and CBR value at 1% lime + 6% waste stone powder for CBR and 7% lime + 6% waste stone powder for U.C.S which are optimum percentage. Sabat (2012) conducted series of tests and concluded that addition of quarry dust decreases Liquid limit, Plastic limit, Plasticity index, Optimum moisture content, Cohesion and increases shrinkage limit, Maximum dry density, Angle of internal friction of expansive soil[45].

Conducted tests for Atterberg Limits, Compaction characteristics (Modified Proctor), Shear Strength parameters using lime with expansive soil stabilized with Optimum percentage of quarry dust (40%). Increase in percentage of addition of lime, decreased Liquid limit, Plasticity Index, Maximum Dry Density whereas Plastic Limit, Shrinkage Limit, Cohesion and Angle of internal friction, Optimum Moisture Content of the soil –quarry dust mixes increased. Addition of lime had made the soil –quarry dust mixes durable. Curing had positive effects on shear strength parameters and maximum values were reported at 5% addition of lime and 28 days of curing [34].

Presented the results of an experimental programmed undertaken to investigate the effect of stone dust and fly ash mixing in different percentages on expansive soil. They observed that at optimum percentages, i.e., 20 to 30% of admixture, the swelling of expansive clay is almost controlled and there is a marked improvement in other properties of the soil as well. It is concluded by them that the combination of equal proportion of stone dust and fly ash is more effective than the addition of stone dust/fly ash alone to the expansive soil in controlling the swelling nature [46]

CHAPTER THREE

MATERIALS AND METHODOLOGY

3.1 Study Area

The soil sample used for this study is collected from Ginjo kebele around Honey Land hotel, Jimma town. The soil sample collected from site would be transported to laboratory for testing. The crusher dust brought from local Aggregate crushing industry near JIT Bosa kitto kebele,Jimma town. Jimma town is located at about 354 Kms in Southwest of Addis Ababa [59]. The Geographical condition of the town approximately 7°41'N Latitude and 36°50'E Longitude [59]. The town has a temperature of 20-30°C with an average annual rainfall 800-2500mm.The town is found in an area of the altitude of 1718-2000m above sea level [59]. It lies in the climatic zone locally known as Woynā Dagā which is considered ideal for agriculture as well as human settlement [61].

According to the Central Statistical Agency (CSA), the total projected population of the town from 2007 is 130,254. The main Geological formation of Jimma town is the Cenozoic tertiary volcanic rock of Nazareth series and Jimma volcanic that were formed by lava and debris ejected from fissure eruptions. Basalts, Trachyte, Rhyolite, and Ignimbrite are the major rock types that belong to the trap series formation [60]. Tropical Residual fine-grained soils, like clays and silt-clays, developed mainly on basaltic bedrock represent the soils found in Jimma town [61].

Figure 3.1 Location of sampling area

3.2 Study Design

This research was designed to answer the research questions and meet its objectives based on experimental findings. The first step in the research work was sample collection. The next step was laboratory tests on the treated and untreated expansive soil using crusher dust as stabilizer. The laboratory test data was analyzed and interpreted so that the effects ofexpansive clay soil and its performances on additives requirement was addressed. Finally, the research findings and recommendations was expressed based on the laboratory test results.

The tasks of this study delighted through the list of the following tests were conducted in evaluating the properties of expansive soil and crusher dust.

- Free swell test to evaluate the swelling index of the soil sample.
- Liquid limit test, plastic limit test to evaluate the liquid limit, plasticity index of the soil sample.
- Sieve analysis for grain size distribution and determination of type of soil.
- Specific gravity test of expansive soil and crusher dust.
- Standard proctor test to evaluate the optimum moisture content and dry densities of various mix proportions.
- California bearing ratio test for evaluating the suitability of the expansive soil and crusher dust mix to be used in sub-grade course of a pavement embankment.

3.3 Sample collection

The Expansive soil sample used for this research work is collected from Jimma town, Ginjo kebele around Honey Land hotel from one test pit. The soil is grayish black in color highly plastic clay. Disturbed and undisturbed sample were collected from the test pit at a depth of 1.5m. Soil sampling from the test pit is shown in Figure 3.2.

Figure 3.2 Sample Collection (Natural Soil ans Crusher Dust)

The crusher dust brought from local Aggregate crushing industry near Bosa kitto kebele,Jimma town. The collected crusher dust separated from non parental materials and subjected to geotechnical characterization to now its natural performance as per standard specification such as AASHTO,ASTM and the like.

3.4 Study Variables

3.4.1 Dependent Variable:

- ✓ Strength of crusher dust stabilized expansive clay soil.

3.4.2 Independent Variable:

- ✓ Engineering properties of untreated and treated expansive soil (MDD, OMC, Particle Size Distribution, Free Swell Index, Atterberg Limits, Specific Gravity and CBR)
- ✓ Dosage of Crusher dust
- ✓ Cost

3.5 Sources of data

Both primary data and secondary data sources were used. Primary data for this study were a laboratory experiment output. Secondary data needed for this research was collected from different journals, book, website and manuals.

3.6 Sampling Techniques

The sampling technique used for this research was a purposive sampling which is non probability method. This sampling technique was proposed based on the information that to determine the strength of the expansive clay soil.

3.7 Methods for Preparing Specimens

- ✓ The sample was collated from the site.
- ✓ In order to prevent moisture the natural soil were placed inside the thick-gauge plastic bags.
- ✓ If the natural moisture content of the sample was higher than desired for mixing, the samples was air-dry to moisture content just below the target value.
- ✓ Soil sample and crusher dust dried in either oven or air separately.
- ✓ Dried soil sample and crusher dust in different percentage of crusher dust (0%,5%, 10%, 15%, 20%, 25%,30%,35%,40%,45% and 50%) are mixed together in proportion of by weight to form various mixes.
- ✓ The formed dry mixes would be blend together with water in order to get homogeneous blends.
- ✓ The formed mixes would be kept as side for 24hrs and then dried.
- ✓ These oven dried mixes are now ready for laboratory testing and considered as sample
- ✓ Testing of different geotechnical properties of the natural expansive soil, crusher dust and treated soil was according to applicable standards like AASHTO, ASTM, ERA and the like.

3.8 Laboratory tests

The samples were collected from different source subjected to various Geotechnical characterizations. The basic test such as sieve analysis, Atteberg limit, natural moisture content, compaction, Atteberg limit and CBR of materials investigated separately in order to know the natural properties of materials as per relevant code of standard. The crusher Dust which is passing through 4.75mm sieve was collected and mixed with the expansive soil from 0% to 50% at an increment of 5%.Totally 12 samples were prepared. Oven dried ingredients would be taken for sample preparation in order to keep accuracy of weight proportioning..

3.8.1 Moisture Content (AASHTO T-80)

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

3.8.2 Grain Size Analysis (AASHTO T-88)

This test is performed to determine the percentage of different grain sizes contained within a soil. The mechanical or sieve analysis is performed to determine the distribution of the coarser, larger-sized particles, and hydrometer method is used to determine the distribution of finer particles. For this study both wet sieve analysis and hydrometer analysis was done according to ASTM D422-63.lastly the analysis was combined particle size distribution curve was plotted as figure 4.1

3.8.3 Specific Gravity (ASTM D 854-00)

Values for specific gravity of the natural soil and crusher dust 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.

3.8.4 Atterberg Limits (ASTM D424 or AASHTO T90)

Representative samples of each soil were subjected to Atterberg limits testing to determine the consistency of the soils. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 475 µm (No. 40) sieve. The liquid limit of each soil had been determined by using casagrande apparatus. The plastic limit of each soil was determined by using soil passing through a 475 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.

3.8.5 Soil Classification (AASHTO M-145)

Soil was classified using the AASHTO Soil Classification System using particle size distribution and Atterberg limits.

Soil classification is the arrangement of soils into different group in order that the soils in a particular group would have similar behavior. The method of classification used in this study was the AASHTO M-145 System. The AASHTO Classification system is useful for classifying soils for high way. According to laboratory test result the soil ander study classified as A-7-5.

Table 3.1 AASHTO soil classification system (AASHTO standard M-145)

General classification	Granular materials							Silt clay materials (more than 35% passing 75µm (No.200))			
	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis,% passing:											
2.00mm (No.10)	50 max.										
0.425mm (No.40)	30 max.	50 max.	51 min.								
75µm (No.200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	37 min.	38 min.	39 min.
Characteristics of fraction passing 0.425mm(No.40)											
Liquid Limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.
General type of significant constitute materials	Stone fragments gravel and sand		Fine sand	Silt or clayey gravel and sand				Silty soil			Clayey soil
General rating as sub-grade	Excellent to good					Fair to poor					

3.8.6 Procter compaction test (AASHTO T-180)

This test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the material. It was done on the natural soil and then various percentages of crusher dust added on the Expansive clay soil and MDD and OMC were determined

3.8.7 California Bearing Ratio (CBR) (AASHTO T-193 and AASHTO T-180)

CBR test conducted to determine the strength of a given material, and how it was behave under loading. This had been determined by measuring the relationship between force and penetration when a cylindrical plunger of cross sectional area 1935mm^2 is made to penetrate the soil at given rate. At any penetration value the ratio of the force to a standard force is defined as the California Bearing Ratio. The CBR test consisted of the following procedures as key point to arrive the result of the strength value deserved.

A. Compacting a sample at its optimum moisture content.
B. Applying a surcharge to the sample to represent the estimated thickness of pavement over the sub base and sub grade materials.
C. Soaking the sample for four days.
D. Forcing a 19.4cm^2 (3in^2) plungers into the sample.

3.9. Detail theory and equation involved in the experiments

3.9.1 Moisture Content

Weight of water contained in a given soil mass compared with the oven dried weight of the soil, expressed as percentage.

$$\mathrm{MC}(\%) = \frac{\text{Wet weight} - \text{Drybweight}}{\text{Dryvweight}} * 100$$

$$\mathrm{MC}\ (\%) = \frac{\text{Weight of water}}{\text{Dry Weight}} * 100$$

3.9.2 Specific Gravity of the Soil

The specific gravity of soil is the ratio between the weight of the soil solids and weight of equal volume of water. It is measured by the help of a volumetric flask in a very simple experimental setup where the volume of the soil is found out and its weight is divided by the weight of equal volume of water. Determined by;

$$\text{Specific Gravity(G)} = \frac{\text{W2} - \text{W1}}{(\text{W4} - \text{W1}) - (\text{W3} - \text{W2})}$$

Where: W_1- Weight of bottle in gms

W_2- Weight of bottle + Dry soil in gms

W_3- Weight of bottle + Soil + Water

W_4- Weight of bottle + Water

3.9.3 Liquid Limit

The Casagrande tool cuts a groove of size 2mm wide at the bottom and 11 mm wide at the top and 8 mm high. The number of blows used for the two soil samples to come in contact is noted down. Graph is plotted taking number of blows on a logarithmic scale on the abscissa and water content on the ordinate. Liquid limit corresponds to 25 blows from the graph

3.9.4 Plastic Limit

This is determined by rolling out soil till its diameter reaches approximately 3 mm and measuring water content for the soil which crumbles on reaching this diameter. Plasticity index (Ip) was also calculated with the help of liquid limit and plastic limit;

3.9.5. Particle Size Distribution

The results from sieve analysis of the soil when plotted on a semi-log graph with particle diameter or the sieve size as the abscissa with logarithmic axis and the percentage passing as the ordinate gives a clear idea about the particle size distribution. The results from sieve analysis of the soil when plotted on a semi-log graph with particle diameter or the sieve size as the abscissa with logarithmic axis and the percentage passing as the ordinate gives a clear idea about the particle size distribution. From the help of this curve, D_{10} and D_{60} are determined. This D_{10} is the diameter of the soil below which 10% of the soil particles lie. The ratio of, D_{10} and D_{60} gives the uniformity coefficient (C_u) which in turn is a measure of the particle size range.

3.9.6. Proctor Compaction Test

This experiment gives a clear relationship between the dry density of the soil and the moisture content of the soil. The experimental setup consists of (i) cylindrical metal mould (internal diameter- 10.15 cm and internal height-11.7 cm), (ii) detachable base plate, (iii) collar (5 cm effective height), (iv) rammer (2.5 kg). Compaction process helps in Increasing the bulk density by driving out the air from the voids. The theory used in the experiment is that for any compactive effort, the dry density depends upon the moisture content in the soil. The maximum dry density (MDD) is achieved when the soil is compacted at relatively high moisture content and almost all the air is driven out, this moisture content is called optimum moisture content (OMC). After plotting the data from the experiment with water content as the abscissa and dry density as the ordinate, we can obtain the OMC and MDD. The equations used in this experiment are as follows:

$$\mathrm{MC} = \frac{\mathrm{Ww}}{\mathrm{Ws}} \times 100$$

Where: Ww- weight of water

Ws- weight of solid

Wet density = weight of wet soil in mouldgms /volume of mould cc

$$\text{Dry density}(\gamma\text{d}) = \frac{\text{wet density}}{1 + \frac{\text{moisture content}}{100}}$$

3.9.7. AASHTO Classification System

Classifies soils into 7-groups based on laboratory determination of particle size distribution, liquid limit (LL), and Plasticity Index (PI).Evaluation of soils within each group is made by means of group index.

$$G = (F-35)[0.2 + 0.005\,(LL - 40)] + [(0.01)\,(F - 15)\,(PI - 10)]$$

Where: F: % passing sieve #200 (whole number).

LL: Liquid Limit.

PI: Plasticity Index (nearest whole number).

If G is negative …… Use G = 0.0

For A-2-6 & A-2-7 subgroups, only the PI portion of the formula should be used.

Inverse ratio of G indicate supporting value of sub grade (i.e. G = 0 good & G = 20 very poor)

3.9.8 CBR Test

It is a penetration test wherein a standardized piston, having an end diameter of 49.53mm (1.95in), is caused to penetrate the soil at a standard rate of 1.27mm/min (0.05in/min). The CBR value is calculated as the ratio of the load or stress at 2.54mm (0.1in) penetration to a standard load or stress. Although the CBR test is an empirical test, but it's widely used in:

- ✓ Used in evaluating the strength of the compacted soil.
- ✓ Used in pavement design for both roads and airfields

CBR Test Procedure

- ✓ The selected sample of subgrade soil (pass Sieve ¾") is compacted in a mold that is 152 mm (6 in) in diameter and 152 to 178 mm (6 to 7 in) high.
- ✓ The moisture content, density, and compactive effort used in molding the sample are selected to correspond to expected field conditions (i.e. standard or modified Proctor).
- ✓ After the sample has been compacted (three molds with 10, 25, and 55 blows /layer), a surcharge weight equivalent to the estimated weight of pavement, base, and subbase layers is placed on the sample, and the entire assembly is immersed-in water for 4 days.

At the completion of this soaking period the sample is removed from the water and allowed to drain for a period of 15 min. The sample, with the same surcharge imposed on it, is immediately subjected to penetration by a piston 49.53 mm (1.95 in) in diameter (cross section area = 3 square inches) moving at a speed of 1.27 mm/min (0.05 in/min). The total loads corresponding to penetrations of 2.5, 5.0, 7.5, 10.0, and 12.5 mm (0.1, 0.2, 0.3, 0.4, and 0.5 in) are recorded.

A load-penetration curve is then drawn, any necessary corrections made, and the corrected value of the unit load corresponding to 2.5 mm (0.1 in) penetration determined. This value is then compared with a value of 6.9 MPa (1000 lb/in^2) required to produce the same penetration in standard crushed rock.

$$CBR(\%) = \frac{\text{unit load at 2.5 mm penetration (MPa)}}{6.9\,\text{MPa}} \times 100$$

$$CBR(\%) = \frac{\text{unit load at 5.0 mm penetration (MPa)}}{10.3\,\text{MPa}} \times 100$$

3.10. Design of Flexible Pavement

The first step in pavement design was to estimate the cumulative number of standard axles to be catered for the design. For this study to evaluate the pavement layer difference between untrated expansive soil and treated expansive soil using crusher dust a traffic class of T8 from ERA Pavement Design Manual Volume 1, 2002 with a total of 20 million ESAs where considered. Traffic class of T8 was selected to consider maximum traffic loading in of design of pavement layer on untrated expansive soil and treated expansive soil using crusher dust foundation.For the thickness of pavement design ERA Pavement Design, Manual Volume 1, 2002, where used. The procedure used for pavement design was as follows

- ✓ Traffic class was selected (T8)
- ✓ Subgrade class for both untreated expansive soil and treated expansive soil using crusher dust where selected
- ✓ Based on selected traffic class and subgrade class pavement layer alternatives was selected from ERA Pavement Design, Manual Volume 1, 2002 pavement design catalogue for both untreated expansive soil and treated expansive soil using crusher dust
- ✓ For both untrated expansive soil and treated expansive soil using crusher dust pavement layer alternative cost evaluation based on current rate of material where evaluated and one cost effective alternative pavement layer was selected for both untrated expansive soil and treated expansive soil using crusher dust
- ✓ Using selected cost effective pavement layer on both untreated expansive soil and treated expansive soil using crusher dust foundatons cost estimation for pavement layer on untreated expansive soil and pavement layer on untreated expansive soil using crusher dust was under taken.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Engineering properties of natural Soil

To determine the quality of the materials, laboratory tests were carried out. The tests involved to identify the properties of the natural soil such as its physical and mechanical properties.

The tests carried out on the natural soil and crusher dust include sieve analysis, Atterberg limit test, compaction test, California bearing ratio and specific gravity.

Table 4. 1 Engineering Properties of natural soil

Properties	Observed Values
Grain size distribution	
Gravel (%)	0.09
Sand (%)	4
Fines (%)	
Silt	32.33
Clay	62.56
Consistency characteristic	
Liquid limit (%)	80.09
Plastic limit (%)	35.27
Plastic index (%)	44.81
AASHTO Classification	A-7-5
USCS Classification	CH
Free swell index (%)	90
Specific gravity	2.69
Compaction characteristics:	
OMC (%)	30.91%
MDD (%)	1.323 g/cm3
Strength characteristics	
CBR (%)	1.817

4.1.1 Particle size distribution

Sieve analysis was carried out to determine the grain size distribution of sub-grade soil and used in the classification of the soil. Accordingly, the wet sieve analysis was employed to determine the grain size distribution of sub-grade soil samples in accordance with AASHTO T-88 Test Method for Particle-size analysis of Soils. The grain size distribution of the soil samples is presented in figure 4.1below. While the test result attached on Appendix A.

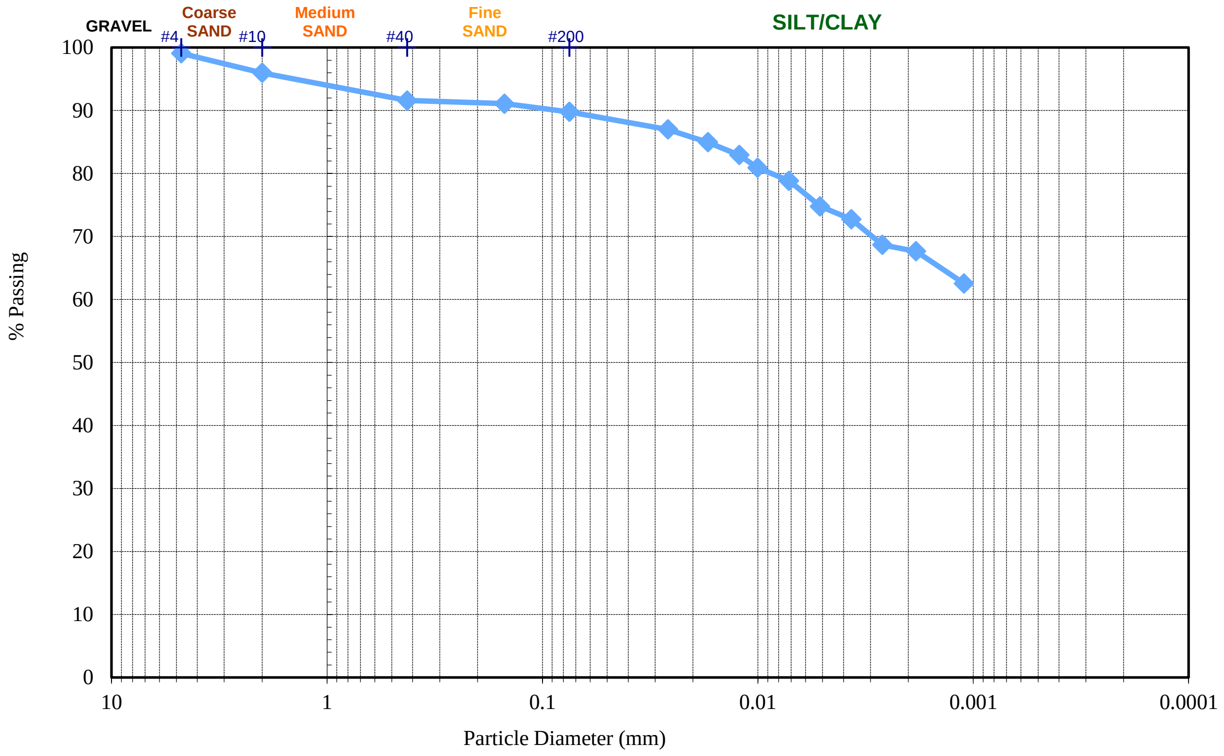


Figure 4.1 Particle size distribution curve of expansive soil.

The soil is Light gray, and almost 89.834 % of the soil is passing through No.200 sieve as shown in figure 4.1 Almost the given soil sample were a fine clay soil. This assist to know its grain size distribution of the selected area. Mechanical analysis was used for coarse sized soil by using set of sieve and whereas hydrometer analysis was used for fined grained soils. The sodium hexametphosphate is used as a dispersing agent. For soils comprising coarser and finer sizes, both mechanical and hydrometer testing methods are performed.

4.1.2.Atterberg's Limits

The Atterberg limits are a basic measure of the nature of fine grained soil. Depending upon the water content of the soil, it may appear in four states namely Solid, Semi solid, Plastic, Liquid. In each state the consistency and behavior of the soil is different and thus so are its engineering properties. Thus, the boundary between each state can be defined based on changes in the soil's behavior. The liquid limit test is conducted as per AASHTO T 89 whereas the plastic limit test is conducted as per AASHTO T 90. The laboratory data analysis is attached in Appendix (A).

Table 4. 2 Atterberg's Limit test result for natural soil

Atterberg's Limit's	Percentage. %	ERA(2002) requirement of PI for subgrade	Status for ERA Specifications
liquid limit, LL	80.08	PI≤30%	Fail for subgrade
Plastic limit. PL	35.27		
Plasticity Index, PI	44.81		

Table 4.2 show that the soil sample changed from liquid state to plastic state and got an average liquid limit of 80%. As a result, at this stage all the soils possess certain small shear strength. This arbitrary chosen shear strength is probably the smallest value that is feasible to measure in standardized procedure. The given soil sample translate from plastic state to semisolid state and got an average plastic limit of 35.27%. At this state the soil rolled into threads. Further decrease of water contents of the same will lead finally to the point where the sample can decrease in volume no further.at this point the sample begins to dry at the surface, saturation is no longer complete, and further decrease in water in the voids occurs without change in the void volume. The difference between the liquid lime and plastic limit is called Plastic Index. The soil sample also has called Plastic Index of 54%.Generally Liquid limit less than 35% is low plasticity, between 35% and 50% intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity.Therefore, the representative sample of natural soil was very high plastic clay that makes the sub grade shrink and swell easily and does not satisfy standard specification of ERA. Therefore, it needs improvement to use for road construction as sub-grade material.

Figure 4.2 Atterberg limit determination

4.1.3 Soil Classification

The soils classification according to AASHTO system and USCS plasticity chart is as Follows.

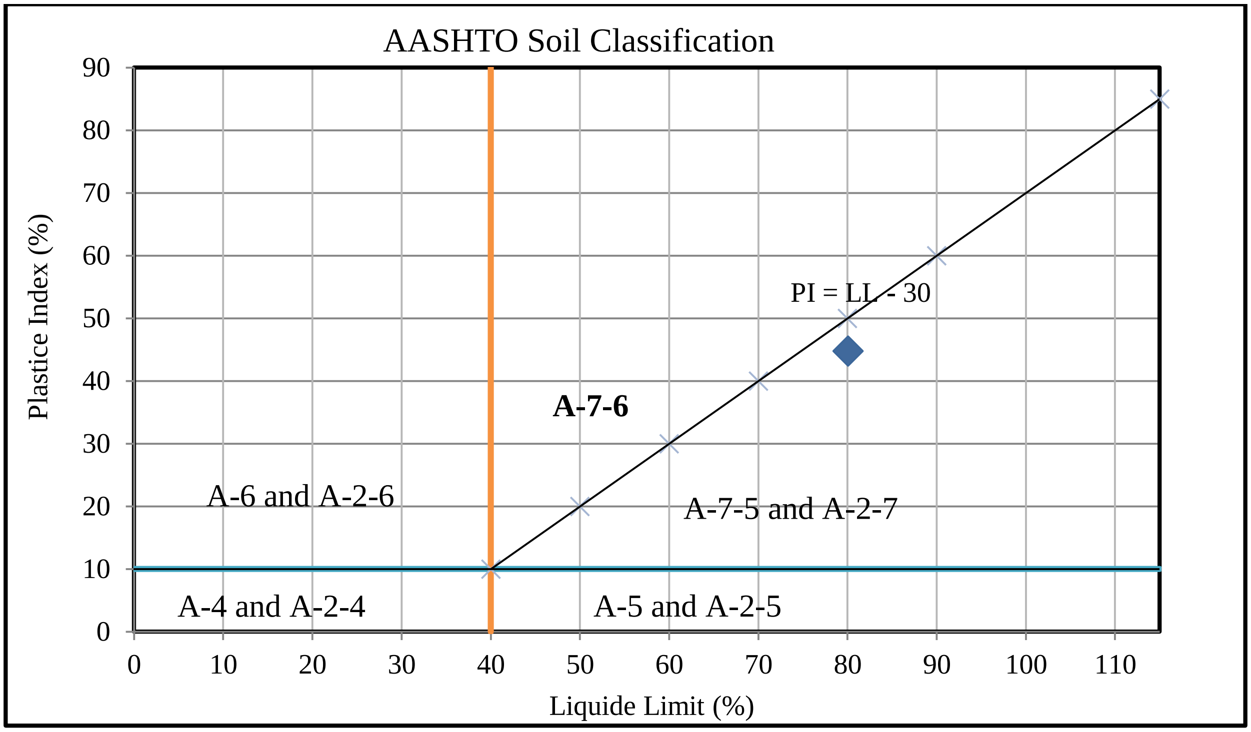


Figure 4.3 Soil classification according to AASHTO system

With the required data in mind, proceed from left to right in the chart. The correct group will be found by a process of elimination. The first group from the left consistent with the test data is the correct classification. As a result of LL and PI the soil sample classified under group A-7-5. So that the usual types of significant constituent materials are clayey soils with fair to poor general rating of subgrade.

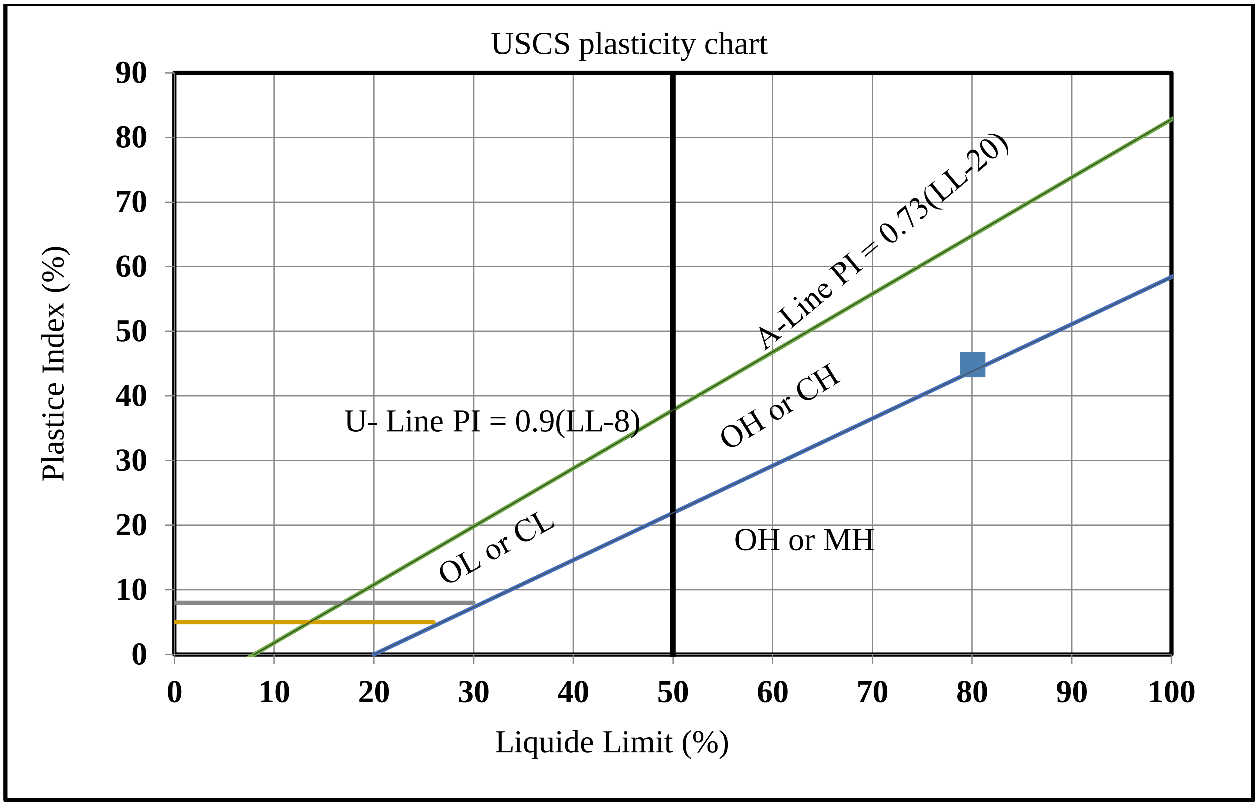


Figure 4.4 USCS plasticity chart of the studied soil

If the Liquid limit are greater or equal to 50% the soil can be clay, silt, or organic depends on whether the soil coordinates plot above or below the A line. Since soil sample has Liquid limit more than 50% and above A-Line, so they are classified under high to very high CH.

4.1.4 Free swell index

Table 4. 3 Free swell index test result of Expansive Soil Sample

Additive content	Expansive soil		
Readings on the Glass Jar	S1	S2	S3
V_w = volume of soil specimen read from the graduated cylinder containing distilled water.	19	18.5	19.5
V_k = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= $[V_w - V_k] / V_k$ x 100%	90	85	95
Average Free Swell index	90%		

This result indicated that the soils is highly Expansive Soils. Soils are called highly expansive when the free swell index exceeds 50%, and such soils undergo volumetric changes leading to pavement distortion, cracking and general unevenness due to seasonal wetting and drying.

4.1.5 Compaction Test

Proctor compaction test wasconducted for the expansive soil under consideration to determine the maximum dry density and optimum moisture content of the soils. The value of laboratory data analysis is attached in Appendix (A).

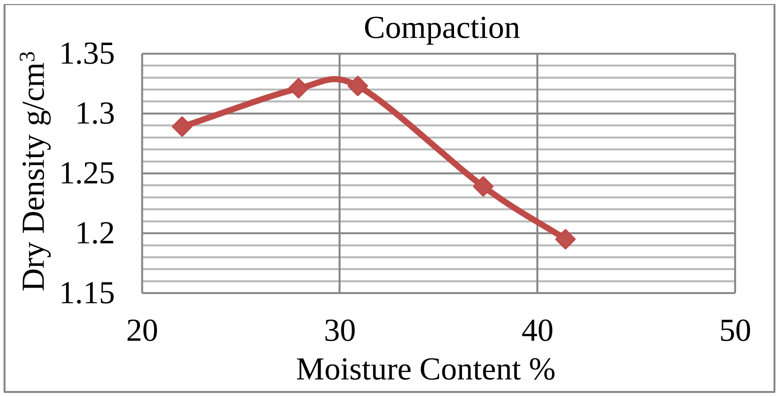


Figure 4.5 Shows a graph of moisture content and maximum dry density

The purpose of drawing the compaction curves shown in Figure 4.5 is to show the peak moisture-density relationship and to extract MDD and OMC values from the curve. The soil sample has a maximum dry density of 1.323 g/cm^3 and the optimum moisture content of 30.91 %. The maximum dry density and optimum moisture content obtained are used to determine the strength to be attained during construction of a road especially sub grade layer. During road construction the CBR value is obtained using the compaction test result. And these CBR results used to determine the thickness of the sub-grade layer of a road construction.

Figure 4.6 Compaction test preparation.

4.1.6. Soaked CBR and CBR Swell of soil sample

According to OMC and MDD of the Expansive soil sample the Soaked CBR value is 1.817%.

Table 4. 4 CBR test result of the Expansive soil sample

Compaction Data		OMC	30.913%	MDD	1.323g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.45	0.24	0.36	1.817	1.815	3.181
MDD		1.323g/cm3				
CBR at MDD		1.817%				

Table 4.4 showed CBR test was determined at 2.54 and 5.08 penetration at the given maximum dry density and optimum moisture content of Original soil sample. The soil sample has 1.817% soaked CBR value at maximum dry density. Therefore, based on the ERA requirement, the soil was lower CBR value and it is not suitable for sub grade in road construction. From test result of Table 4.4, the soil sample was expansive soil so it required additives to be stabilized. To achieve the objective of this study the soil sample should stabilize mechanically using Crusher dust.

4.1.7. Specific Gravity (ASTM D854-98)

The specific gravity of a substance is the ratio of the unit weight of that substance to the unit weight of water at varies degree centigrade. The specific gravity of a soil depends on the mineralogy of the soilgrains. Most soils are a blend of several basic minerals. The subgrade soil under study is expansiveblack cotton soil composed of different minerals. The average specific gravity of the soil under studywas 2.69. The summary of the test result is tabulated while the laboratory test analysis and plots are given in Appendix A.

4.2. Engineering properties of Crusher dust

The crusher dust brought from local Aggregate crushing industry near Bosa kitto kebele,Jimma town. The collected crusher dust separated from non-parental material and subjected to geotechnical characterization to now its natural performance as per standard specification such as AASHTO, ASTM and the like. The basic test such as sieve analysis, atteberglimit, compaction, Atteberg limit and CBR of crusher dust investigated. The summary of test result shown in Table 4.5 below.

Table 4. 5 Engineering Properties of crusher dust

Properties	Observed Values
Grain size distribution	
Gravel (%)	7.6
Sand (%)	92.4
Fines (%)	0
Coefficient of Uniformity	4.7
Coefficient of Curvature	0.772
Consistency characteristic	
Liquid limit(%)=W_L	NP
Plastic limit(%)=W_P	NP
AASHTO Classification	A-1-a
Specific gravity	2.75
Compaction characteristics:	
OMC (%)	10.3%
MDD (%)	2.01g/cm3
Strength characteristics	
CBR (%)	11.8

4.3. The effect Crusher dust on Expansive soil

4.3.1 The effect of Crusher dust on Atterberg's limit

Plasticity Characteristics and their deformation can be explained with Index Properties like Liquid Limit, Plastic Limit, and Plasticity Index. To know the results of Expansive clay Crusher Dust mixes, the Material passing through 425 μm of Crusher dust clay soil mixes have taken at various percentages of crusher dust have been subjected to consistency limits such as Liquid Limit, Plastic Limit and Plasticity Indexand the results are shown in Figure 4.7.It is found that as the percentage of crusher dust increases the liquid limit and plastic limit decreases. Consequently the plasticity index also decreased followed with increase in crusher dust content.

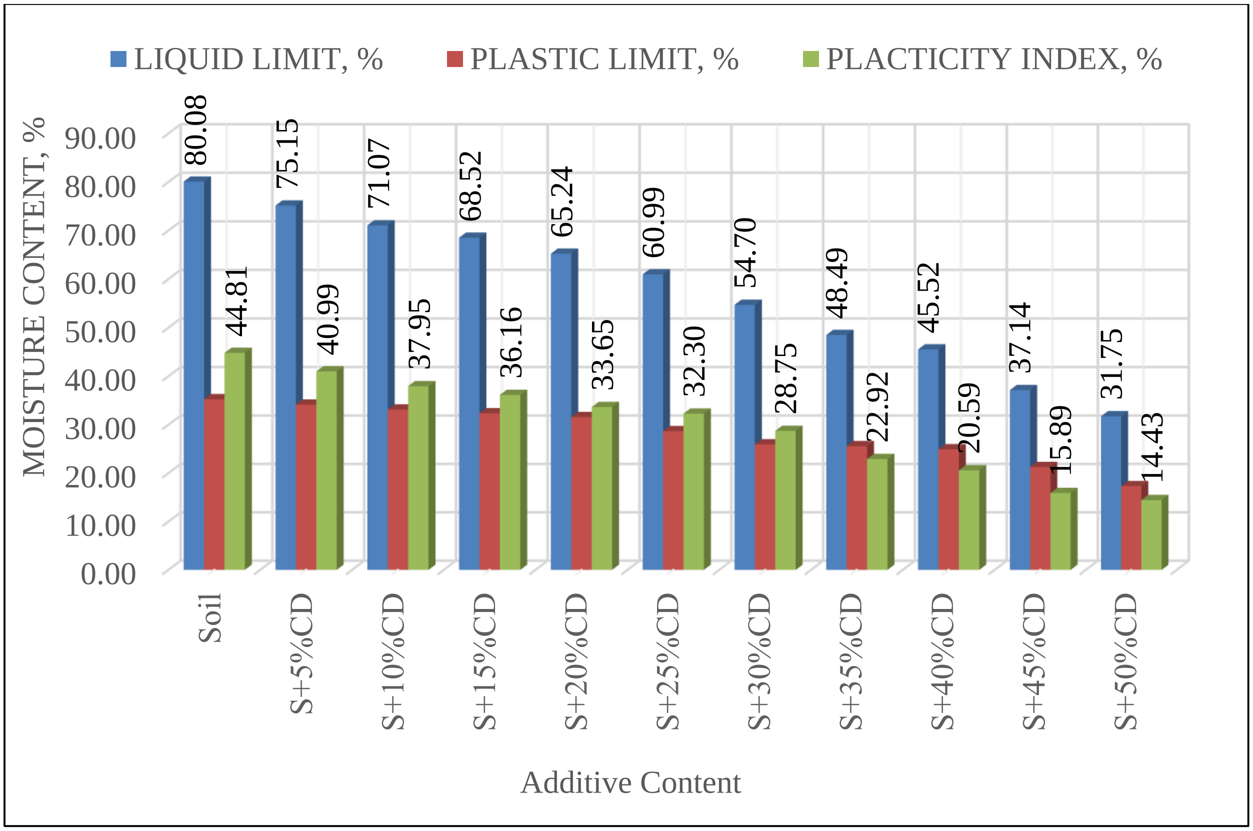


Figure 4.7 Effectof Crusher dust on Atterberg’s limits

The expansive soil has been modified by addition of crusher dust in the range of 5% to 50% of original soil. The liquid limits, plastic limit, plasticity index of original soil without modification are 80.08%, 35.27% and 44.81% respectively. From the test data it is observed that addition of crusher dust decreases Liquid Limit, Plastic Limit and Plasticity Index values. After modification PI reduced from a value of 44.81% to a value of 14.43% after an improvement with 50% crusher dust. Hence crusher dust has great impact in reduction of PI.The probable reason for reduction of liquid limit of modified soil may be due to mechanical stabilization and addition of non-plastic material.

Blending expansive soilwith crusher dust was satisfying ERA standard specification for Sub-grade construction. Blending expansive soilwith 30% crushed stone dust and above was satisfying ERA standard specification of for sub grade construction.

4.3.2 The effect of addition of Crusher dust on Free swell index

The free swell index of expansive soil decrease when the ration of Crusher dust increases. The free swell index result of stabilized soil is presented in Table 4.6 below and are illustrated in figure 4.8.

Table 4. 6 Free swell test result of stabilized expansive clay soil

Additive Content	FSI (%)	IS 2720 (part XL) requirement	Compare Result
SOIL	90	FSI < 50%	Control
S+5%CD	81		Slight reduction
S+10%CD	68		Slight reduction
S+15%CD	60		Slight reduction
S+20%CD	51		Slight reduction
S+25%CD	39		In range
S+30%CD	28		In range
S+35%CD	24		In range
S+40%CD	18		In range
S+45%CD	13		In range
S+50%CD	11		In range

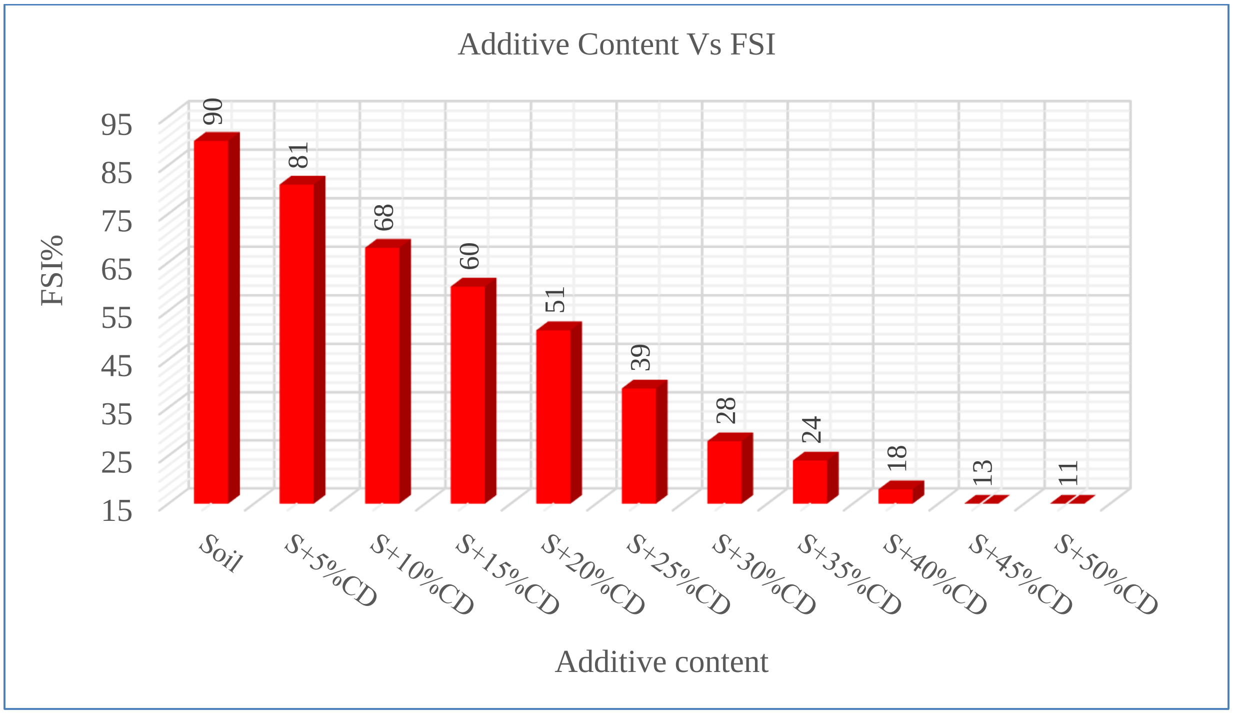


Figure 4.8 Free swell index of Expansive soil sample at different stabilizer ratio

As shown in Figure 4.8 above, the free swell of the samples has decreased with increase in Crusher dust ratio. But slight reduction is observed with higher ratio of Crusher dust added. Except 5%, 10%, 15% and 20% of Crusher dust soil mix all ratio were under the specification. This reduction in free swell index indicated that removing potentially expansive soil is important especially to the sub grade soil to stay for long period of time without failure.

As more percent of crusher dust added to the soil the swell and shrink properties of the affected soil lower. Beside, more crusher dust content slightly reduce the expansiveness of the soil. As a whole the quantum of replacement of quarry dust is found to be in the range of 40% to 50 % in laying road pavements for the in-situ expansive clay soil which is marginally higher. For economic considerations and for laying local pavements inside streets and villages 30% replacement of clayey soil can be sorted.

4.3.3 The effect of addition of Crusher dust on Compaction Characteristics

The results of standard Proctor tests on expansive soil treated with different percentages of Crusher dust are shown in Table 4.7 and Figure 4.9 through 4.10 shows the variation Maximum Dry Density (MDD) with percentage of Crusher dust. The summary of the test result is tabulated while the laboratory test analysis and plots are given in Appendix (C).

Table 4. 7 Effect of Crusher dust on Maximum Dry Density

Additive Content	Symbol	MDD , g/cm3	OMC, %
Natural Soil	Soil	1.323	30.913
Soil + 5% Crusher Dust	S+5%CD	1.349	29.130
Soil + 10% Crusher Dust	S+10%CD	1.395	28.739
Soil + 15% Crusher Dust	S+15%CD	1.413	27.096
Soil + 20% Crusher Dust	S+20%CD	1.437	26.092
Soil + 25% Crusher Dust	S+25%CD	1.513	25.325
Soil + 30% Crusher Dust	S+30%CD	1.555	24.310
Soil + 35% Crusher Dust	S+35%CD	1.596	22.130
Soil + 40% Crusher Dust	S+40%CD	1.631	21.460
Soil + 45% Crusher Dust	S+45%CD	1.669	20.254
Soil + 50% Crusher Dust	S+50%CD	1.735	18.158

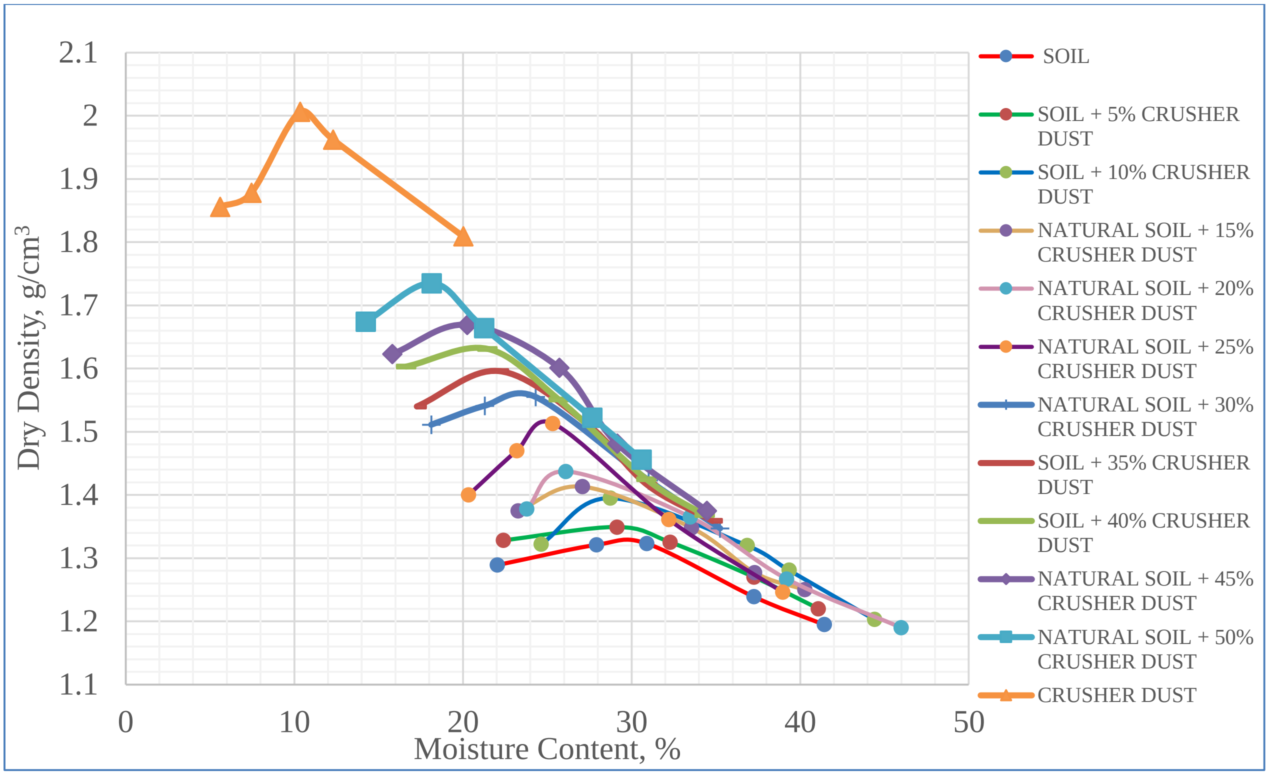


Figure 4.9 Effect of Crusher dust on Dry Density and Moisture Content

The values for the maximum dry densities were noted to significantly increase with the addition of crusher dust from a value of 1.323 g/cm^3 to a maximum value of 1.735 g/cm^3 attained in the blend 50% crusher dust.

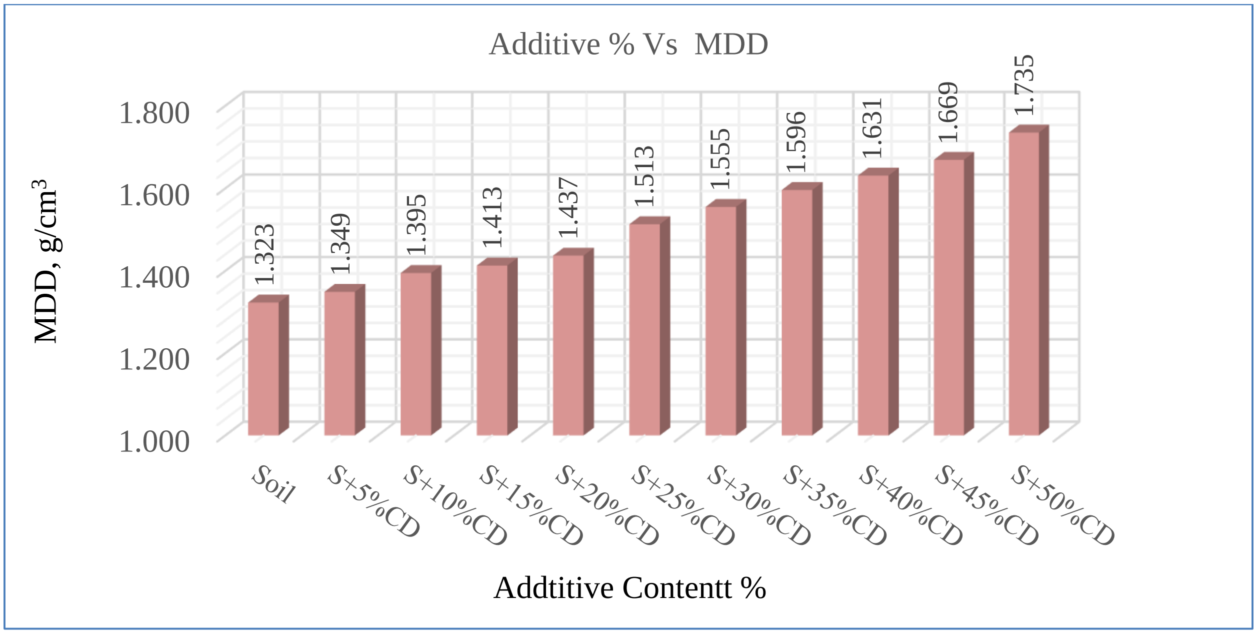


Figure 4.10 Effect of Crusher dust on Maximum Dry Density

Figure 4.10 gives the values of maximum dry density of original and modified soil. From table and figure, it is found that by addition of crusher dust in proportion of 5%,10%,15%, 20%, 25%, 30%, 35%, 40%, 45% and 50% the percentage increase maximum dry density, is found to be 2.0%, 5.40%, 6.8%, 8.6%, 14.4% ,17.5%, 20.6%, 23.3% ,26.2% and 31.1.% respectively. Thus as percentage of stone dust increases maximum dry density increases.Whereas, the optimum moisture content values are continuously decreasing. The OptimumMoisture Content (OMC) decreases from 30.91% to 18.16 % when Crusher dust is increased from 0 to 50%

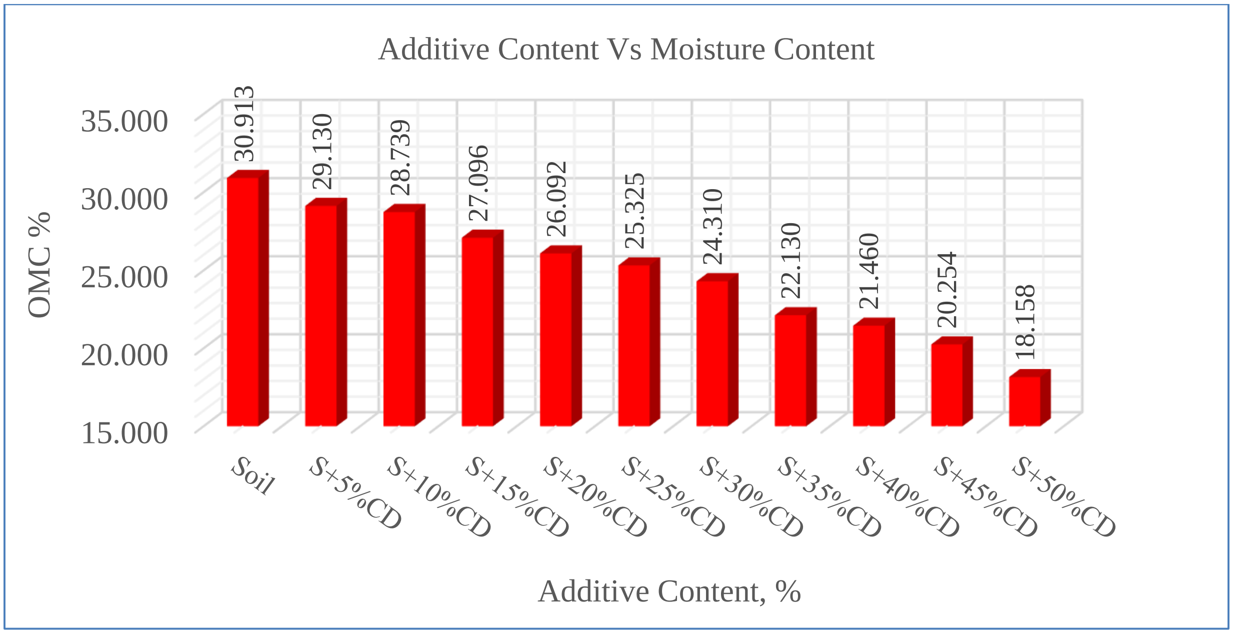


Figure 4.11 Variation OMC with percentage of Crusher dust

Figure 4.11 shows the variation Optimum Moisture Content (OMC) with percentage of Crusher dust. it is found that by addition of crusher dust in proportion of 5%,10%,15%, 20%, 25%, 30%, 35%, 40%, 45% and 50% the percentagedecreaseOptimum Moisture Content (OMC) , is found to be 5.8%, 7.0%, 12.3%, 15.6%, 18.1% ,21.4%, 28.4%, 30.6% ,34.5% and 41.3% respectively.Thus as percentage of crusher dust increases OMC decreases. The decrease in optimum Moisture Contents are due to replacement of Silt and Clay particles by Crusher Dust particles which reduces the intake of Moisture.

The probable reason for increase in maximum dry density of soil by addition of crusher dust is due to proper rearrangement of soil particles and addition of non-plastic material which improves the binding capacity further increasing the dosage of crusher dust the majority of the finesclayarrestedby the crusher dust particles and attaining the behavior of crusher dust.It is not practicable to add crusher dust beyond 30% and 35% since there were slightly increasing in MDD. In general, dense soil mass is considered to besuitable to act as a good sub grade.

As the replacement of crusher dust is found to be in the range of 40%to 50 % in laying road pavements for the in-situ expansive soil which is marginally higher. For economic considerations and for laying local pavements inside streets and villages replacement of 30% crusher dust ispractically feasible.

4.3.4. Effect of crusher 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 1.817%. The Expansive soil was modified by addition of Crusher dust in the proportion of 5%,10%,15%,20%, 25%, 30%, 35%, 40%, 45% and 50% which are designed and symbolized as shown in table below. The increase in percentage of CBR value for stabilized Expansive clay for 5%,10%,15%,20%, 25%, 30%, 35%, 40%, 45% and 50%crusher dust were found to be 33.902%, 51.350%, 83.434%, 102.7%, 133.9% ,179.75%, 207.26%,165.99, 143.09% and 129.33.% respectively.The summary of test shown on Table while the test result attached on Appendices(C).

Table 4. 8 Effect of Crusher dust on CBR

Additive Content	OMC, %	MDD, g/cm^3	CBR,%	ERA Requirement	Compare Result
SOIL	30.913	1.323	1.817	CBR > 3%	Control
S+5%CD	29.130	1.349	2.433		Slight Increase
S+10%CD	28.739	1.395	2.750		Slight Increase
S+15%CD	27.096	1.413	3.333		Slight Increase
S+20%CD	26.092	1.437	3.683		Slight Increase
S+25%CD	25.325	1.513	4.250		In range
S+30%CD	24.310	1.555	5.083		In range
S+35%CD	22.130	1.596	5.583		In range
S+40%CD	21.460	1.631	4.833		In range
S+45%CD	20.254	1.669	4.417		In range
S+50%CD	18.158	1.735	4.167		In range

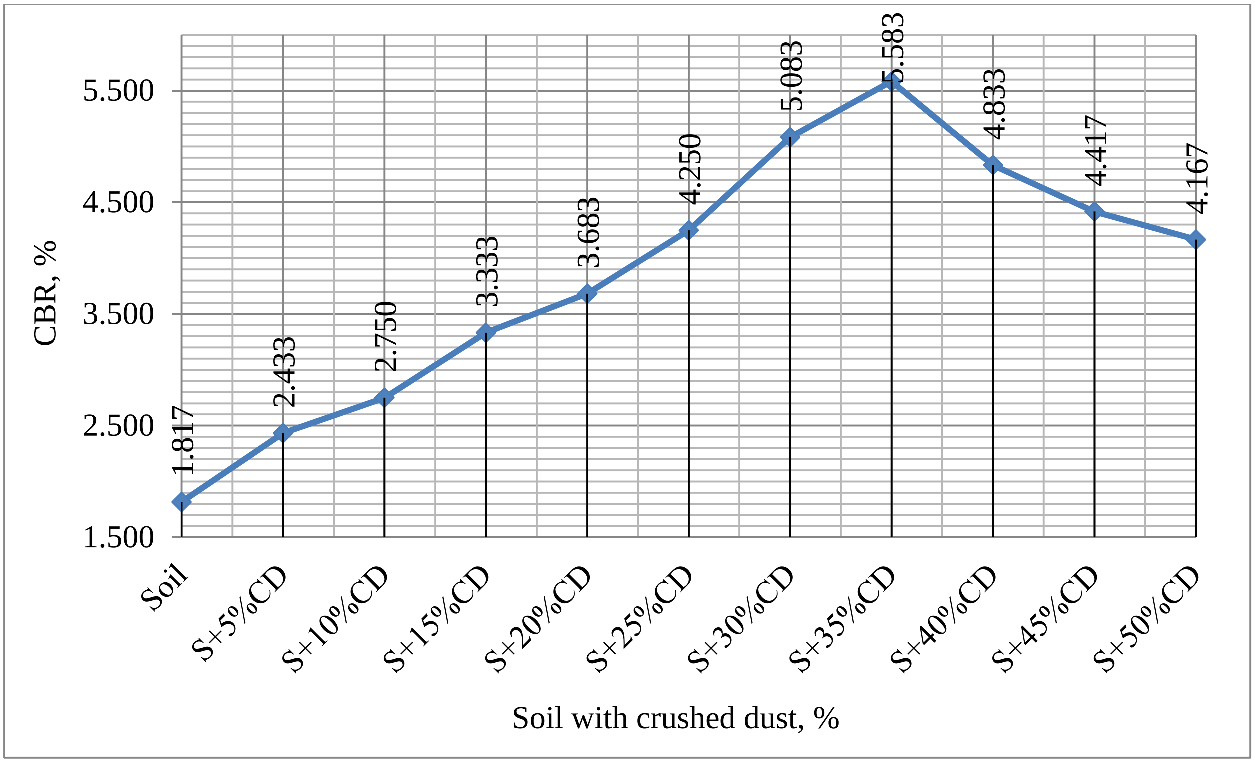


Figure 4.12 Shows the variation in soaked CBR value with Crusher dust

From Table 4.8 and Figure 4.12 it is found that as percentage of Crusher dust increases soaked CBR value increases. The CBR of soil first increases to 5.585 from 1.817 with the increase in percentage of stone dust from 0% to 35% and subsequently it decreases to 4.167 on further increasing the stone dust content to 50%. From practical consideration the addition of Crusher dust about 30% of total weight of modified soil mass is feasible and economical. The probable reason for increase in CBR value of soil is by addition of stone dust in comparison with original soil may be due to increase in density of modified soil mass having more strength.

4.3.5. Effect of crusher dust on CBR Swell

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

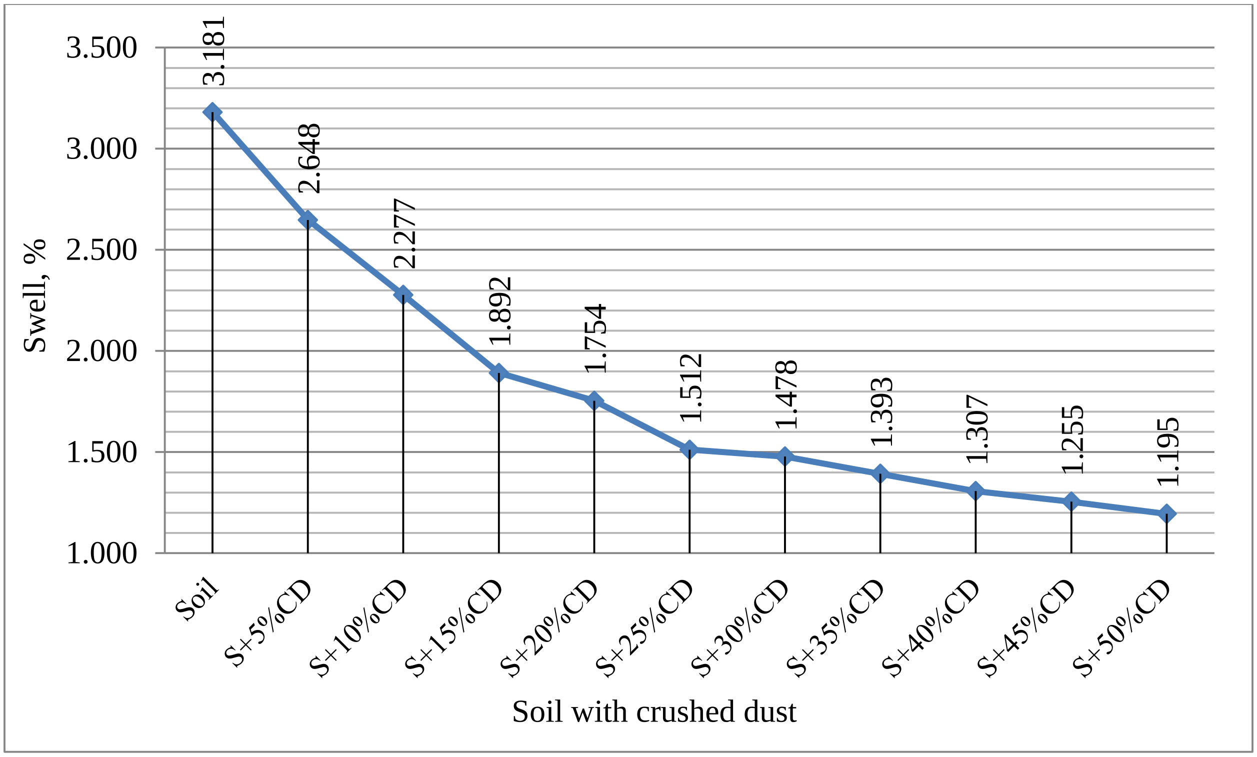


Figure 4.13 CBR Swell test result of stabilized and natural Expansive soil

The Figure 4.13 shows natural soil have the properties of swelling and potentially expansive soil. However, when crusher dust mix with different ratio the CBR swellreduce.The decrease in CBR Swell of expansive soil is due to replacement of crusher dust in place of fine clay in soil. This is also due to replacement of some the volume that is previously occupied by expansive clay minerals by crusher dust.

Soil sample had 3.181% value of CBR swell but when 30% crusher dust added it reduce to 1.478%. This indicate highly reduction in CBR swell. When it mix with crusher dust beyond 30% it improve the expansive soil strongly but there is slightly reduction was observed. Therefore using crusher dust stabilizers improve the stability and strength of the subgrade soils. The strength of subgrade is the principle factor in determining the thickness of the pavement, but deterioration due to frost action must also be taken into account. The strength of subgrade is associated on CBR scale.

4.4. Design of Pavement structure

From conducted laboratory test the untreated soil has 1.817% of CBR, for the minimum CBR value of 2%, the subgrade strength class to be assigned to this project is therefore S2 As per ERA Pavement Design, Manual Volume 1,2001.The following preliminary information has been derived from material investigations

- ✓ The materials which may be considered for cement- or lime-stabilization have relatively low percentages of fines and low plasticity, thus making cement-stabilization more promising.
- ✓ Granular sub base materials are available in sufficient quantities and cement stabilization of the sub base is uneconomical when compared to bank-run materials. Stabilization of sub base materials will not be further considered.
- ✓ All other materials entering the composition of the possible pavement structures are available in various quantities and associated transport/construction costs.

Based on the above, and with the T8/S2 and T8/S3 combination of traffic and subgrade strength classes, the design charts 4 through 7 indicate the possible alternate pavement structures given in Table 4.9 and Table 4.10

Table 4. 9 Possible Pavement Structure before stabilization

Design Chart No.		4	5	6	7
Pavement Components and Selected Fill	Possible Alternate Pavement Structures	Alternate Structure No. 1	Alternate Structure No. 2	Alternate Structure No. 3	Alternate Structure No. 4
Surfacing (asphalt concrete) (1)		5 cm AC	15cm AC	15cm AC	5 cm AC
Roadbase: · Crushed Stone · Cement stabilized (e.g. 4 Mpa) · Cement stabilized (e.g. 2.5 Mpa) · Bituminous stabilized		15 cm 15 cm 15 cm —	25 cm — — —	15 cm 12.5cm 12.5 cm —	— — — 20 cm
Granular subbase		—	25 cm	—	25 cm (2)
Selected fill		20 cm	20cm	20cm	20cm (2)
Buffer layer		60cm	60cm	60cm	60cm

Table 4. 10 Possible Pavement Structure after stabilization

Design Chart No.		4	5	6	7
Pavement Components and Selected Fill	Possible Alternate Pavement Structures	Alternate Structure No. 1	Alternate Structure No. 2	Alternate Structure No. 3	Alternate Structure No. 4
Surfacing (asphalt concrete) (1)		5 cm AC	15cm AC	15cm AC	5 cm AC
Roadbase:					
· Crushed Stone		15 cm	25 cm	15 cm	—
· Cement stabilized (e.g. 4 Mpa)		15 cm	—	—	—
· Cement stabilized (e.g. 2.5 Mpa)		12.5 cm	—	22.5 cm	—
· Bituminous stabilized		—	—	—	20 cm
Granular subbase		—	27.5 cm	—	27.5 cm (2)
Selected fill		15 cm	—	—	— (2)

The alternate structures including cement stabilized layers appear prohibitive, and the alternate number two including only crushed stone road base and sub base also appear at a disadvantage. Since Granular sub base materials are available in sufficient quantities and cement stabilization of the sub base is uneconomical when compared to bank-run materials. Stabilization of sub base materials will not be further considered. Therefore the alternative 2 is best Alternate Pavement Structure. With these Alternative The total pavement thickness is 850mm and 675mm for untreated and threated sub grade respectively. The recommended pavement structure is given in Figure 4.14 and Figure 4.15.

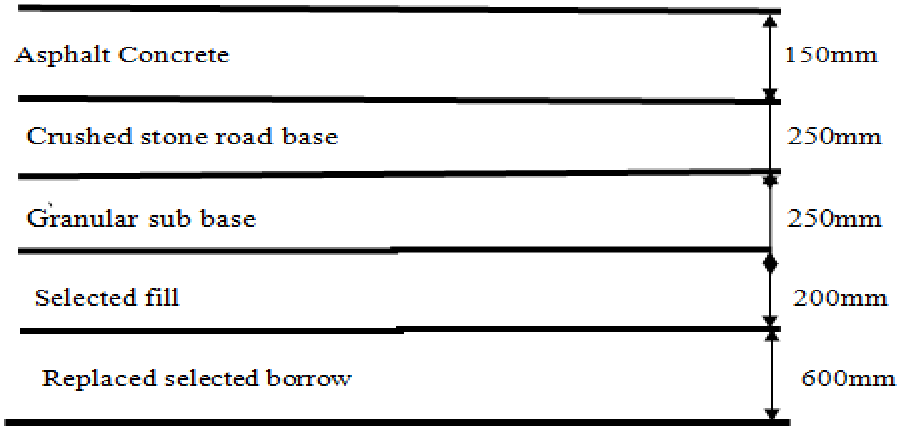


Figure 4.14 Pavement structure before stabilization

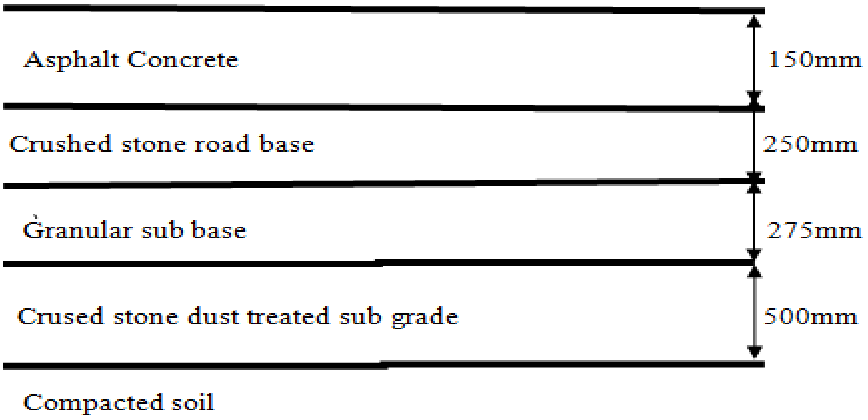


Figure 4.15 Pavement structure after stabilization

4.5. Cost Estimation

The quantitative cost of pavement for untreated and treated sub grade are given in tables 4.11 and 4.12 trough table 4.13 respectively.

Table 4. 11 Quantitative cost for untreated Expansive soil(Constractionethiopia.com,2018)

Item No	Item description	Unit	Rate	Length (m)	Width (m)	Depth (m)	Amount
1	Sub Grade						
	Site clearing	m^2	15.49	1000	3.5		54215
	Balk excavation in expansive soil not exceeding 1.5m	m^3	99.58	1000	3.5	0.6	209118
	Disposal of excavated material (5KM hauling distance)	m^3	126.66	1000	3.5	0.6	265986
	Road bed preparation compaction to 93% MMD	m^2	58	1000	3.5		203000
	Selected material (5km)	m^3	145	1000	3.5	0.6	304500
	Placing and compacted selected material to 95% MDD	m^2	78.24	1000	3.5		273840
	Sub Total	1310659					
2	Capping layer/selected material	m^3	145	1000	3.5	0.2	101500
3	SUB BASE						0
	Gravel sub base 97%, MDD (MAT. From 5KM)	m^3	170.9	1000	3.5	0.25	149502.5
4	Base course						0
5	Crushed stone road base	m^3	469.19	1000	3.5	0.25	410541.3
6	15cm Asphalt Concrete	m^2	1500	1000	3.5		5250000
	Sub Total	5911544					
	Total Cost of Constraction	7222203					

Table 4. 12 Quantitative cost for crusher dust stabilized Expansive soil

I. Road Section	Unit	Unit price
Clearing and Grubbing within Road Prism	m^2	15.49
Purchase Cost of Stabilizer including transport		
Purchase Cost of Stabilizer from local crusher	m^3	456.25
For 1m^3 of Expansive soil, 0.39m^3 of crusher dust required(by using 30% CD wich is optimum)	m^3	177.94
Purchase Cost of Stabilizer of crusher dust	m^2	106.76
III. Placing of Stabilizer		
Hauling of Stabilizer	m^2	48.05
Mixing of Stabilizer	m^2	71.94
Placing of Stabilizer	m^2	54.19
Total Quantitative Cost	m^2	296.43

Table 4. 13 Quantitative cost of pavement after stabilizing

Item No	Item description	Unit	Rate	Length (m)	Width (m)	Depth (m)	Amount
1	Stablized Sub Grade	m^2	296.43	1000	3.5		1037505
2	Gravel Sub Base 97% MDD (MAT. From 5KM)	m^3	170.9	1000	3.5	0.275	164452.8
3	Crushed Stone Road Base	m^3	469.19	1000	3.5	0.25	410541.3
4	15cm Asphalt Surfacing	m^2	1500	1000	3.5		5250000
Total Cost		6862499					

The comparisons of the cost benefits were made from Tables 4.11 and 4.13. As shown in the tables, the total quantitative cost of crusher dust stabilized subgrade was estimated as 1,037,505 Birr/km against the cost of 1,310,659 Birr / km for replacing selective borrow material from a 5km distance. The saving in cost for crusher dust stabilization thus estimated to be 20.84% of construction cost of sub grade wich is 5% of total constraction cost.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

The study on stabilizing the locally available clayey soil in Jimma by crusher dust with the support of series of laboratory investigations in specific arrived at the following conclusions.

- ✓ crusher dust a product from crusher unit consists of mainly sand size particles having high specific gravity and the soaked CBR value for standard compaction. This indicates that crusher dust can be used as an embankment material, backfill material for the lower layer of sub base. Also reuse of this waste material is economically advantages and does not bring any environmental hazards
- ✓ The test result showed that the subgrade soil considered for this study were A-7-5 as per AASHTO soil classification system and CH as USCS. The plastic index for soil sample is 44% and the laboratory result showed that MDD is 1.323 g/cm3. The soaked CBR value of soil sample is 1.86%. The engineering properties of the natural soil sample were expansive clay soil. The soil sample have high plasticity index, very low load-bearing capacity with high swelling potential which make the subgrade unsuitable without additives and stabilizers.
- ✓ The addition of the crusher dust to the soil reduces the clay content and thus increases in the percentage of coarser particles, reduces the Liquid limit by 60.89% and plasticity index by 67.62%. With 30% of crusher dust the liquid limit of Expansive soil become 28.75% which makes it suitable for subgrade as per ERA specification.
- ✓ There is a demonstrable effect on maximum dry density of soils on mixing crusher dust. Adding percentage of crusher dust increases its maximum dry density. The study also reveals the fact that with increase in percentage of crusher dust in soil, the optimum moisture content decreases which is helps in decreasing water quantity required during compaction. The information based on the studies carried out will be useful for the design and construction of sub grade, embankment and structural fills for utilization of crusher dust as a stabilizing agent.
- ✓ The swelling characteristics of the samples has decreased with increase in Crusher dust ratio. But slight reduction is observed with higher ratio of Crusher dust added. Except

5%, 10%, 15% and 20% of Crusher dust soil mix all ratio were under the specification. This reduction in free swell index indicated that removing potentially expansive soil is important especially to the sub grade soil to stay for long period of time without failure.

- ✓ The Expansive soil was modified by addition of Crusher dust in the proportion of 5%,10%,15%, 20%, 25%, 30%, 35%, 40%, 45% and 50%.The increase in percentage of CBR value for stabilized Expansive were found to be 33.902%, 51.350%, 83.434%, 102.7%, 133.9% ,179.75%, 207.26%, 165.99%, 143.09% and 129.33.% respectively. Therefore CBR of Expansive clay soil increases from 1.86% to 5.561% with increasing percentage of crusher dust from 0% to 35%, further increasing the content of crusher dust the CBR of treated soil slightly decreased as the CBR swell continuously decreasing with increasing of crusher dust from 0% to 50%. It is also identified that addition of 30% and 35% crusher dust yield high CBR value. This finding is very useful in decreasing pavement thickness design.
- ✓ It is observed that the mixture of the expansive clay mixed with 30% of crusher dust full fill the requirement for sub grade material recommended by ERA manual. The values at 35% crusher dust are also full fill the requirement of sub grade material .As a whole the quantum of replacement of Crusher Dust. is found to be in the range of 35% to 50 % in laying road pavements for the in-situ Jimma clayey soil which is marginally higher. For economic considerations and for laying local pavements inside streets and villages 30% replacement of clayey soil can be sorted.
- ✓ The study also reveal that the total pavement thickness can be reduced from 850 mm to 675mm by replacement of Expansive clayey soil with 30 % Crusher Dust. The reduction of about 225 or 20.59% in pavement thickness will save substantial amount of money in construction.Reduction of expansive nature of subgrade eliminates buffer layer while increase of CBR value reduces the overall thickness of pavement. Therefore this Elimination of buffer layer and reduction of overall thickness of pavement offsets the construction cost of road sub grade by 20.84% which estimated nearly 5% of overall construction cost of pavement structure.

5.2 Recommendations

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

- ✓ Additional curing time effect on all geotechnical laboratory tests should be performed.
- ✓ This study was taken only one high expansive soil sample. It is recommended to take a large number of soil sample which characterizes the whole study area.
- ✓ The present study was conducted by taking limited parameter such as atterberg limit, free swell index, moisture density relation, CBR and CBR swell potential on stabilization by crusher dust. It is recommended to test additional parameter like unconfined compressive strength and mineralogical tests should also be performed to have more realistic test results.
- ✓ The similar nature of investigation are also recommended for finding out use of existing plastic soil for other road construction material like, sub base, base and hard shoulder by adding suitable good engineering property material.
- ✓ Next researcher can also stabilize expansive soil using crusher dust with cement or crusher dust lime.The similar nature of investigation are also recommended for the garbage obtained from demolishing of absolute existing road pavement or damaged layer of pavement, which can be used as sub grade or base course by adding crusher dust/sand as additive or more than one additive like crusher dust with clay and cement or crusher dust with clay and lime.

REFERENCE

1. ERA, “Road Sector Development Program (RSDP) 13 years Performance and Phase IV”. Addis Ababa, Ethiopia. January, 2011.
2. Satyanarayana P.V.V, PremTeja R, Harshanandan T, Lewis Chandra K, “A Study On The Use Of Crushed Stone Aggregate And Crusher Dust Mixes In Flexible Pavements,” in IJSER, Vol. 4, Issue 11, (2013), ISSN:2229-5518, PP 1126-1136.
3. Onyelowe Ken C, Okafor F.O and Nwachukwu D G (2010), “Geophysicsluse of Quarry dust (as admixture) asapplied to soil stabilization and modification – A Review”, ARPN Journalof Earth Sciences, Vol. 1, No. 1, pp. 6-8.
4. Ghausuddin Syed Quadri and Koranne Sunil Shubhada,(2011), “Evaluation of soil quarry dust mixtures reinforced with polypropylene fibres”, Electronic Journal of Geotechnical Engineering, Vol. 16, pp. 1007-1017.
5. Jayapal J.(2014) “ Stabilization of Korattur Clay with Quarry Dust for Effective Utilization in Flexible Pavement” , IN IJERT, Vol. 3 Issue 9, ISSN: 2278-0181,PP.462-468
6. Nelson, D.J and Debora, J.m.(1992), expansive soils problems and practice in foundation and pavement engineering, John Wiley and Sons, Inc. , New York
7. Jemal J., "In-depth investigation into Engineering characteristics of Jimma soils," MSc Thesis, Addis Ababa University, Ethiopia, 2014.
8. Sachim N., Bhavsar,Ankit J., "effect of waste material on swelling and shirinkage properties of clayeye soil," vol. 3 (11), Patel, international Journal of application or innovation in Engineering & Management (IJAIEM), 2014, pp. 200-2006.
9. Lyon Assoiciates, Inc, , "Laterite and Lateritic soils and other problems soils of Africa,", Baltimore, 1971.
10. V.N.SMurthy (2007), advanced foundation engineering,CBSPublisher,Bangolare, India
11. Daniel Teklu (2003), examining the swelling pressure of Addis Ababa expansive soil, Msc thesis, Addis Ababa university
12. Martin Rogers (2003), highway engineering, Dublin institute of technology, black well publishing, Ireland
13. Chen F.H. (1975).Foundation on expansive soils. Elsevier. Amsterdam.

14. Soils and geology procedures for foundation design of buildings and other structuresdepartment of army and air force, October 1993,
15. BereketYohannis, Kimberly hill and lev.Khazanovich,(2009) mechanistic modeling of unbound granular materials, university of Minnesota
16. Website:http://www.nbmcw.com/roads-pavements/307-practical-problems-of-highway-construction-in-black-cotton-soil-area.html
17. Satyanarayana, P.V.V.; Raghu, P.; kumar, R.A. and Pradeep, N. (2013). Performance of Crusher Dust in High Plastic Gravel soils as road construction material. IOSR Journal of mechanical and civil engineering, vol.10, issue 3, 01-05.
18. Methods of soil stabilization, December 24, 2010 [online] Available at: < http://www.engineeringtraining.tpub.com/14070/css/14070_424.htm >
19. B. Thagesen,(1996)"Tropical rocks and soils, Highway And Traffic Engineering In Developing Countries", Chapman and Hall, London.
20. N.J. Garber and L.A. Hoel,(2000) "Traffic and Highway Engineering, Second edition Brooks/Cole Publishing Company", London, 481- 492, 927- 930, 2000.
21. O.O. Amu, O.F. Bamisaye and I.A. Komolafe,(2011) "The Suitability and Lime Stabilization Requirement of Some Lateritic Soil Samples as Pavement", International Journal Pure Application Science Technology. 2(1), ISSN 2229-6107, pp. 29 – 46.
22. G.H. McNally,(1998) "Soil and Rock Construction Materials, Routledge, London, 276-282, 330-341.
23. S.A. Ola,(1975)"Stabilization of Nigerian Lateritic Soils With Cement, Bitumen, and Lime", Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering.
24. S. Hashim, "Mechanical Treatment of Granular Base Course Materials", M.Sc. Thesis, Sudan University of Science and Technology, Center For Engineering And development
25. Narayana Reddy S. G. &Satyanarayana C. N. V Addition of Crusher Dust for Stabilization of Expansive Soil, Imperial Journal of Interdisciplinary Research (IJIR) ISSN: 2454-1362,
26. Jaganmohan Mishra, R. K. Yadav and A. K. Singhai (2014) "Effect of granite dust on engineering properties of lime stabilized black cotton soil", International Journal of Engineering Research and Technology, Vol 3 Jan

27. Akshaya Kumar Sabat, Bidula Bose (2013) “Improvement in Geotechnical Properties of an Expansive Soil using Fly Ash - Quarry Dust Mixes”, Electronic Journal of Geotechnical Engineering, Vol. 18
28. Akshaya Kumar Sabat (2012) “Statistical Models for Prediction of Swelling Pressure of a Stabilized Expansive Soil”, Electronic Journal of Geotechnical Engineering,Vol. 17 p p: 837 – 846
29. Sridharan A, Soosan T, Jose G and Abraham B M (2005), “Utilization of crusher dust to improve the geotechnical properties of soil in highway construction”,Geotechnical Testing Journal , Vol. 28,No. 4, pp. 391-400.
30. Quardi Syed Ghausuddin and Shubhada Sunil Koranne (2011) “Evaluation of soil – Quarry Dust Mixtures Reinforced with Polypropylene Fibres”, Electronic Journal of Geotechnical Engineering, Vol.16, Bund.I, pp.1007-1016
31. Quardi Syed Ghausuddin and Shubhada Sunil Koranne (2011) “Evaluation of soil – Quarry Dust Mixtures Reinforced with Polypropylene Fibres”, Electronic Journal of Geotechnical Engineering, Vol.16, Bund.I, pp.1007-1016
32. Ranisnchung R N, Praveen Kumar, AdityaKumarAnup and Pooja Sharma (2013);“Evaluation of Efficacy of Fines Obtainedfrom Demolished Concrete Slabs as aSoilStabilizer”,HighwayResearchJournal, Vol. 6, No. 1, pp. 35-42.
33. Ali M S and Korrane SS (2011), “Performance analysis of Expansive soil treated with stone dust and fly ash”, Electronic Journal of Geotechnical Engineering, Vol. 16, pp. 973-982,
34. Sabat A K (2012), “A study on somegeotechnical properties of Limestabilized expansive soil-Quarry DustMixes”, International Journal ofEmerging trends in Engineering andDevelopment, Vol. 1, No. 2, pp. 42-49.
35. Sabat A K and Bose B (2013),“Improvement in Geotechnical propertiesof an Expansive soil using Fly ash-Quarrydust Mixes”, Electronic Journal ofGeotechnical Engineering, V18, pp.3487-3500.
36. Satyanarayana P V V, Raghu P, Ashok Kumar R and Pradeep N (2013),“Performance of Crusher Dust in High Plastic Gravel Soils As Road Construction Material”, IOSR Journal of Mechanical and Civil Engineering(IOSR-JMCE) Vol. 10, No. 3, pp. 1-5.

37. Pandian N S, Krishnan K C and Sridharan A (2011), "California Ratio Behaviour of Soil/FlyashMixtures",Journal of Testing and Evaluation,ASTM, Vol. 29, No. 2, pp. 220-226.
38. Mahzuz H M A, Ahmed A A M and YusufM A (2011), "Use of stone powder inconcrete and mortar as an alternative ofsand", African Journal of EnvironmentalScience Technology, Vol. 5, No. 5,pp.381-388.
39. Radhikesh P N, Amiya K D andMoharana N C (2010), "Stone crusherdust as a fine aggregate in concrete forpaving blocks", International Journal ofCivil and Structural Engineering, Vol. 1,No. 3, pp. 613-619.
40. Appukutty P and Murugesan R (2009), "Substitution of quarry dust to sand for mortar in brick masonry works",International Journal on Design andManufacturing Technologies, Vol. 3, No.1, pp. 59-63
41. Rohit Mahent and Rajesh Joshi,(2015) "Improvement of Soil Index Properties by Adding Stone Dust Mix," in IJSTE, Vol. 2, Issue 02,ISSN: 2349-784,PP 61-68.
42. Kumar Arun U and BiradarKiran B,(2014) " SOFT SUBGRADE STABILIZATION WITH QUARRY DUST-AN INDUSTRIAL WASTE ",in IJRET,VOL.3,ISSUE 8, eISSN: 2319-1163, pISSN: 2321-7308,PP.409-412.
43. Bshara, A.S.; Bind, Y.K. and Sinha, P.K. (2014). Effect of Stone Dust on Geotechnical properties of Poor soil. Int. Journal of Civil Engineering and Technology (IJCIET), vol. 5, issue 4, 37-47.
44. SatyaNarayana P V V, Raghu P, AshokKumar R and Pradeep N "Utilization of Crusher Dust Stabilized Gravels as Sub-Base Materialsl",in IOSR-JMCE, Vol. 2,issue 11, (2013), ISSN: 2278-0181, pp. 4034-4046.
45. Roobhakhshan, A. and Kalantari, B. (2013). Stabilization of Clayey Soil with Lime and Waste Stone Powder. Int. Journal of Scientific Research in Knowledge, vol. 1, issue 12, 547-556.
46. Ali M S and Korrane SS (2011), "Performance analysis of Expansive soil treated with stone dust and fly ash", Electronic Journal of Geotechnical Engineering, Vol. 16, pp. 973-982,
47. AASHTO (1993), Standard Specification for Transportation materials and methods ofsampling and testing (Twenty first edition 2001 (Part 1).

48. AASHTO, Guide for Design of Pavement Structures.(1993), American Association of StateHighway and Transportation Officials: Washington, D.C., USA.
49. Bowles J.E. (1996), foundation of analysis and design, fifth edition, the McGraw-Hill companies Inc.
50. Braja Das (2007), principle of foundation engineering, 7th edition, Global engineering, Dallas,Texas
51. Depaa. Ra. B,(2013), “Stabilization Of Pavement Material Using Waste Brick Kiln Dust ” International Journal of Engineering Research & Technology (IJERT), Vol. 2 Issue 4 ISSN: 2278-0181,PP. 1684-1691
52. E.J. YODER and M.W. WIKZAK, Principle of pavement design, second edition, john wileyand sons, Inc. New York.
53. Indiramma P. and Sudharani CH.(2016) “Use of Quarry Dust for Stabilising Expansive Soil” , IN IJERT, Vol. 5, Issue 1, ISSN:2319-8753,PP.1151-1157.
54. Kumar Sabat. January (2012) “A Study on Some Geotechnical Properties of Lime Stabilised Expansive Soil” Quarry Dust Mixes Issue 2, vol.1.1.
55. Ramadas T.L., Kumar N. Darga, Aparna G.(2010) “Swelling and strength characteristics of expansive soil treated with stone dust and fly ash”, Indian geotechnical conference
56. Satyanarayana P.V.V, PremTeja R, Harshanandan T, Lewis Chandra K, “A Study On The Use Of Crushed Stone Aggregate And Crusher Dust Mixes In Flexible Pavements,” in IJSER, Vol. 4, Issue 11, (2013), ISSN:2229-5518, PP 1126-1136.
57. Prof. Krishna Reddy, UIC, 2008, Engineering Properties of Soils Based on
58. CBR, Centre de recherchesroutièresBelgique’ (2004a), “Code de Bonne Pratique Pour le Traitement des Solsà la Chaux et/ouCiment- Recommandations Pour le Recyclage des Terres”, CRR, Belgique..
59. Lyon Assoiciates, Inc, , "Laterite and Lateritic soils and other problems soils of Africa,", Baltimore, 1971.
60. Mengesha, et al., "Geological Map of Ethiopia," Addis Ababa, Ethiopia, 1996.
61. Blight, G.E., "Mechanics of Residual soils," in A.A Balkema, the Netherlands, 1997.

APPENDIX

Appendix A: Laboratory Test Result of Natural Soil sample

1. Natural Moisture Content

Sample No.	Tare Mass (g)	Tare + Wet Soil Mass (g)	Tare + Dry Soil Mass (g)	Dry Soil Mass (g)	Water Mass (g)	Moisture Content (%)
T1	18	95	74	56	21	37.5
T2	17	96	76	59	20	33.89
T3	18	99	77	59	22	37.28
					Average	36.22

2. wet sieve analysis

Sieve Number	Diameter (mm)	Soil Retained (g)	Soil Retained (%)	Cumulative retained (%)	Persentege of fine(%)
4	4.75	4.6	0.9	0.9	99.1
10	2.00	15.44	3.1	4.0	96.0
20	0.85	13.22	2.6	6.6	93.4
40	0.43	9.02	1.8	8.4	91.6
60	0.30	2.1	0.4	8.9	91.1
140	0.150	0.24	0.0	8.9	91.1
200	0.075	6.31	1.3	10.2	89.8
Pan		449.07	89.8	100.0	0.0
			TOTAL	**500**	**100.0**

3. Hydrometer Analysis

Time	□T (min)	Temp (°C)	R_a	$R_{a,corr}$	L	K	D (mm)	C_T	a	R_c	% Finer
					Table 2, D422	Table 3, D422		Table 4, Lab Manual	Table 1, D422		
	□										
3:10	1	23	49	49	8.3	0.013	0.0365	0.7	1.019	43.7	89.1
3:12	2	23	48	48	8.4	0.013	0.0262	0.7	1.019	42.7	87.0
3:15	5	23	47	47	8.6	0.013	0.017	0.7	1.019	41.7	85.0
3:20	10	23	46	46	8.8	0.013	0.0122	0.7	1.019	40.7	82.9
3:25	15	23	45	45	8.9	0.013	0.01	0.7	1.019	39.7	80.9
3:40	30	23	44	44	9.1	0.013	0.0072	0.7	1.019	38.7	78.9
4:10	60	23	42	42	9.4	0.013	0.0051	0.7	1.019	36.7	74.8
5:10	120	23	41	41	9.6	0.013	0.0037	0.7	1.019	35.7	72.8
7:10	240	23	39	39	9.9	0.013	0.0026	0.7	1.019	33.7	68.7
11:10	480	25	38	38	10.1	0.013	0.0018	1.3	1.016	33.3	67.7
3:10	1440	23	36	36	10.4	0.013	0.0011	0.7	1.019	30.7	62.6

4. Combined sieve analyses

Diameter (mm)	Passing (%)	Combined passing (%)
9.50	100	100.000
4.75	99.1	99.100
2.00	96.012	96.012
0.85	93.368	93.368
0.43	91.564	91.564
0.30	91.144	91.144
0.150	91.096	91.096
0.075	89.834	89.834
0.0365	89.061	89.061
0.0262	87.023	87.023
0.0170	84.985	84.985
0.0122	82.947	82.947
0.0100	80.909	80.909
0.0072	78.871	78.871
0.0051	74.795	74.795
0.0037	72.757	72.757
0.0026	68.681	68.681
0.0018	67.666	67.666
0.0011	62.567	62.567

5. Specific gravity

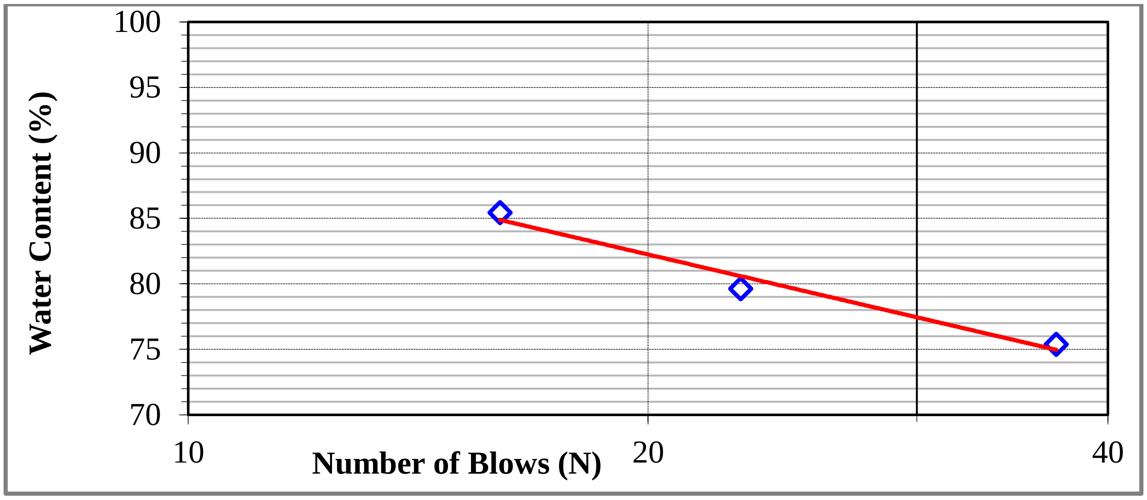
ADDITIVE CONTENT	EXPANSIVE SOIL		
sample number	T11	T23	T33
mass of empty bottle (M1) in gms.	112.45	118.67	115.27
mass of bottle+ dry soil (M2) in gms.	162.45	168.67	165.27
mass of bottle + dry soil + water (M3) in gms.	390.65	396.9	398.72
mass of bottle + water (M4) in gms.	359.448	365.378	367.377
Observed temperature	22	23	22
K Temperature correction	1.007	1.0005	1.007
specific gravity	2.678476	2.707274	2.69871898
Avg. specific gravity	**2.694823**		

6. Atterberg Test Result

Additive Content		SOIL						
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				37	23	16
Can Number	---	---	14	23	54	E13	C45	C31
Mass of Empty Can	M_C	(g)	5.54	5.86	6.47	17.96	17.24	18.49
Mass Can & Soil (Wet)	M_{CMS}	(g)	12.97	12.28	14.88	42.18	43.54	49.92
Mass Can & Soil (Dry)	M_{CDS}	(g)	11.06	10.61	12.65	31.77	31.88	35.44
Mass of Soil	M_S	(g)	5.52	4.75	6.18	13.81	14.64	16.95
Mass of Water	M_W	(g)	1.91	1.67	2.23	10.41	11.66	14.48
Water Content	w	(%)	34.66	35.07	36.08	75.38	79.64	85.43

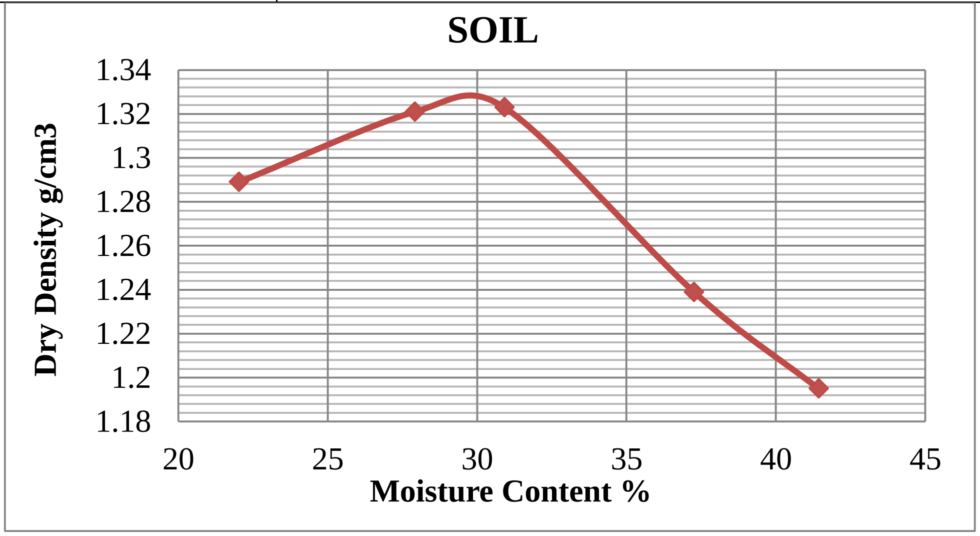
Liquid Limit (LL or w_L) (%):	80.08
Plastic Limit (PL or w_P) (%):	35.27
Plasticity Index (PI) (%):	44.81

PI at "A" Line [25 No. of Blow] =
80.08



7. Compaction laboratory result

<table>
<tr><td colspan="2">ADDITIVE CONTENT</td><td colspan="9">SOIL</td></tr>
<tr><td colspan="11">DENSITY DETERMINATION</td></tr>
<tr><td>Trial No</td><td colspan="2">1</td><td colspan="2">2</td><td colspan="2">3</td><td colspan="2">4</td><td colspan="2">5</td></tr>
<tr><td>Wgt. of Mould +Wet soil (gm) A</td><td colspan="2">4485</td><td colspan="2">4595</td><td colspan="2">4635</td><td colspan="2">4605</td><td colspan="2">4595</td></tr>
<tr><td>Wgt. of Mould (gm) B</td><td colspan="2">3000</td><td colspan="2">3000</td><td colspan="2">3000</td><td colspan="2">3000</td><td colspan="2">3000</td></tr>
<tr><td>Wgt. of wet soil (gm) A-B=C</td><td colspan="2">1485</td><td colspan="2">1595</td><td colspan="2">1635</td><td colspan="2">1605</td><td colspan="2">1595</td></tr>
<tr><td>Volume of mould (cm3) D</td><td colspan="2">944</td><td colspan="2">944</td><td colspan="2">944</td><td colspan="2">944</td><td colspan="2">944</td></tr>
<tr><td>Wet Density(gm/cm3) C/D=E</td><td colspan="2">1.573</td><td colspan="2">1.690</td><td colspan="2">1.732</td><td colspan="2">1.700</td><td colspan="2">1.690</td></tr>
<tr><td colspan="11">MOISTURE CONTET DETERMINATION</td></tr>
<tr><td>Container Code .</td><td>T34</td><td>D12</td><td>E</td><td>G53</td><td>P33</td><td>H2</td><td>D51</td><td>45</td><td>CV</td><td>SE</td></tr>
<tr><td>Mass of Wet soil+Container(gm)(F)</td><td>74.12</td><td>67.949</td><td>86.314</td><td>77.949</td><td>81.467</td><td>82.343</td><td>90.319</td><td>107.83</td><td>110.339</td><td>88.378</td></tr>
<tr><td>Mass of dry soil+container(gm)(G)</td><td>63.655</td><td>59.12</td><td>71.655</td><td>64.52</td><td>66.037</td><td>67.24</td><td>71.058</td><td>83.053</td><td>82.91</td><td>67.766</td></tr>
<tr><td>Mass of container(gm)(H)</td><td>17.371</td><td>17.957</td><td>17.371</td><td>17.957</td><td>17.051</td><td>17.44</td><td>18.562</td><td>17.548</td><td>17.566</td><td>17.355</td></tr>
<tr><td>Mass of moisture(gm)F-G=(I)</td><td>10.465</td><td>8.829</td><td>14.659</td><td>13.429</td><td>15.43</td><td>15.103</td><td>19.261</td><td>24.777</td><td>27.429</td><td>20.612</td></tr>
<tr><td>Mass of Dry soil(gm)G-H=(J)</td><td>46.284</td><td>41.163</td><td>54.284</td><td>46.563</td><td>48.986</td><td>49.8</td><td>52.496</td><td>65.505</td><td>65.344</td><td>50.411</td></tr>
<tr><td>Moisture content % (I/J)*100=K</td><td>22.61</td><td>21.45</td><td>27.00</td><td>28.84</td><td>31.50</td><td>30.33</td><td>36.69</td><td>37.82</td><td>41.98</td><td>40.89</td></tr>
<tr><td>Avg. Moisture Content % (L)</td><td colspan="2">22.030</td><td colspan="2">27.922</td><td colspan="2">30.913</td><td colspan="2">37.258</td><td colspan="2">41.432</td></tr>
<tr><td>Dry Density gm/cm^3 E/(100+L)*100</td><td colspan="2">1.289</td><td colspan="2">1.321</td><td colspan="2">1.323</td><td colspan="2">1.239</td><td colspan="2">1.195</td></tr>
<tr><td>OMC</td><td colspan="10">30.91%</td></tr>
<tr><td>MDD</td><td colspan="10">1.323g/cm^3</td></tr>
</table>



8. Free swelling index

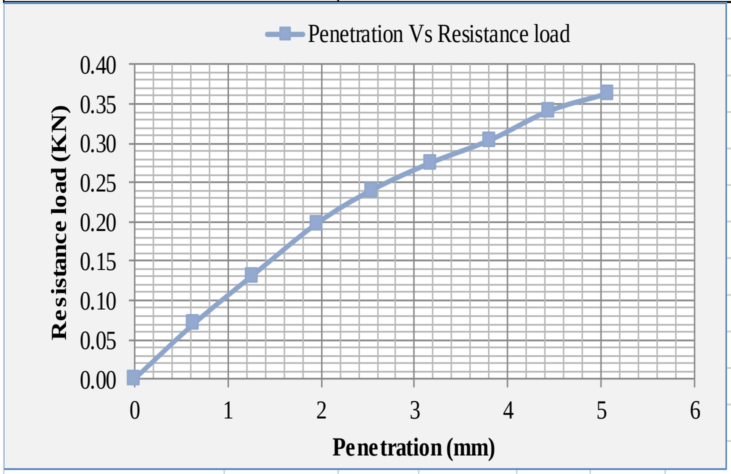
Additive content	Expansive soil		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	19	18.5	19.5
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	90	85	95
Average Free Swell index	90%		

9. CBR and CBR Swelling

CBR test result for Natural soil

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	3.2	6.0	9.0	10.9	12.5	13.8	15.5	16.5
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.07	0.13	0.20	0.24	0.28	0.30	0.34	0.36

Compaction Data		OMC	30.913%	MDD	1.323g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.45	0.24	0.36	1.817	1.815	3.181
MDD		**1.323g/cm3**				
CBR at MDD		**1.817%**				

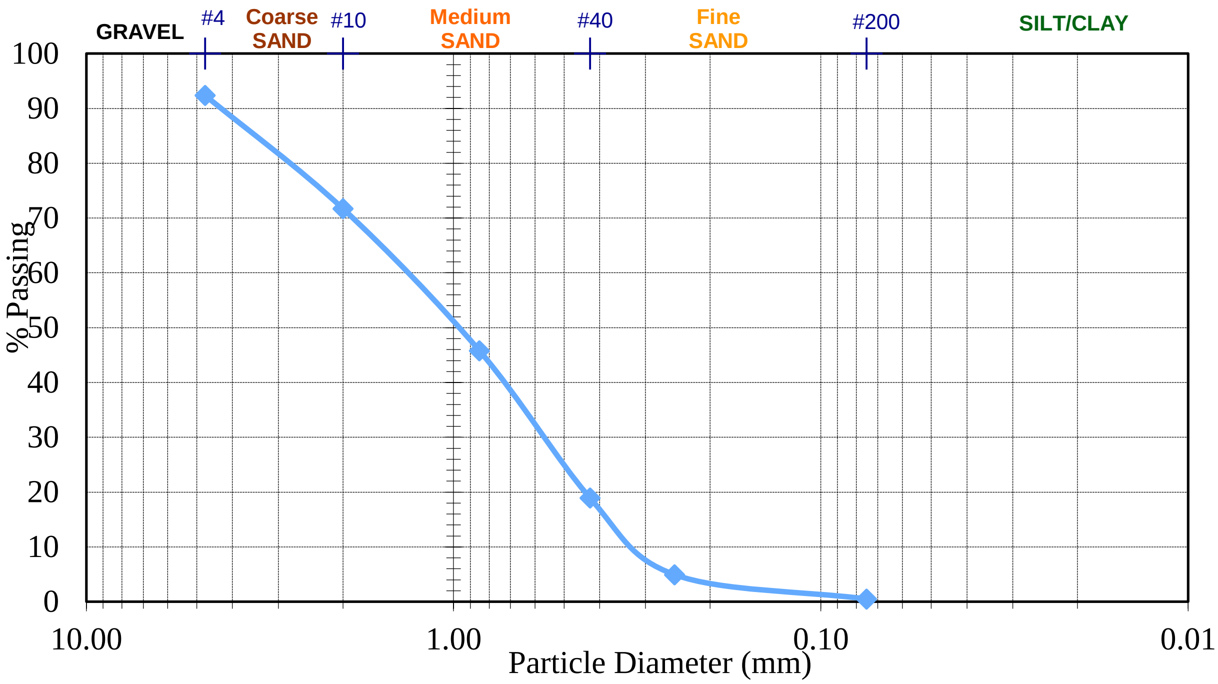


Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	10226.1	11000.5
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	3726.1	4500.5
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	1.76	2.12
Moisture content (%)	21.3	43.19
Dry density of soil (g/cm3)	1.45	1.48
Dial gage reading of Height H1	45.19	
Dial gage reading of Height H2		48.89

Appendix B: Laboratory Test Result of crusher dust

1. Sieve analysis

Sieve Number	Diameter (mm)	Soil Retained (g)	Soil Retained (%)	Soil Passing (%)
#4	4.75	75.9	7.6	92.4
#10	2.00	206.3	20.7	71.7
#20	0.85	258.4	25.9	45.8
#40	0.43	267.9	26.9	18.9
#60	0.25	139.7	14.0	4.9
#200	0.075	44.2	4.4	0.5
Pan		4.9	0.5	0.0
	TOTAL:	**997.2**	**100.0**	



2. Atterberg Test Result

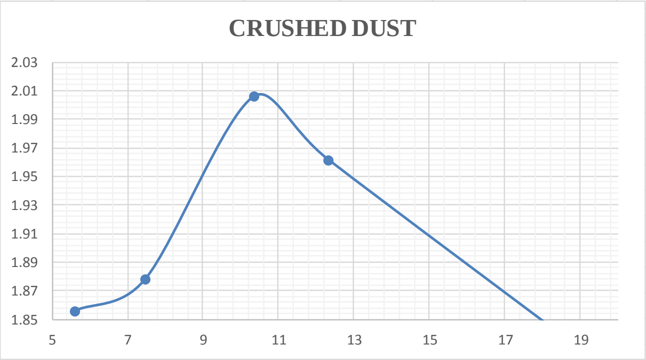
Additive Content			CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	Blows	LESS THAN 25 AT ANY WATER CONTENT					
Can Number	---	---						
Mass of Empty Can	M_C	(g)	CRUSHER DUST NON PLASTIC					
Mass Can & Soil (Wet)	M_{CMS}	(g)						
Mass Can & Soil (Dry)	M_{CDS}	(g)						
Mass of Soil	M_S	(g)						
Mass of Water	M_W	(g)						
Water Content	*w*	(%)						

Liquid Limit (LL or w_L) (%):	NP
Plastic Limit (PL or w_P) (%):	NP
Plasticity Index (PI) (%):	NP

3. Compaction laboratory result

ADDITIVE CONTENT	CRUSHER DUST									
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4835		4890		5075		5065		5035	
Wgt. of Mould (gm) B	2985		2985		2985		2985		2985	
Wgt. of wet soil (gm) A-B=C	1850		1905		2090		2080		2050	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.960		2.018		2.214		2.203		2.172	
MOISTURE CONTET DETERMINATION										
Container Code .	T5C1	G84	G4211	P112	ATR14	G53	ATR13	SPP1	HC22	OS2
Mass of Wet soil+Container(gm)(F)	47.86	60.824	65.267	63.505	76.057	87.684	96.538	86.61	113.864	69.373
Mass of dry soil+container(gm)(G)	46.293	58.501	61.946	60.24	70.65	80.977	87.8	79.032	97.995	60.742
Mass of container(gm)(H)	17.589	18.121	17.327	16.651	17.694	16.985	17.477	16.929	18.5098	17.754
Mass of moisture(gm)F-G=(I)	1.567	2.323	3.321	3.265	5.407	6.707	8.738	7.578	15.869	8.631
Mass of Dry soil(gm)G-H=(J)	28.704	40.38	44.619	43.589	52.956	63.992	70.323	62.103	79.4852	42.988
Moisture content % (I/J)*100=K	5.46	5.75	7.44	7.49	10.21	10.48	12.43	12.20	19.96	20.08
Avg. Moisture Content % (L)	5.606		7.467		10.346		12.314		20.021	
Dry Density gm/cm^3 E/(100+L)*100	1.856		1.878		2.006		1.962		1.809	

OMC	10.346%
MDD	2.006g/cm3



4. Specific Gravity

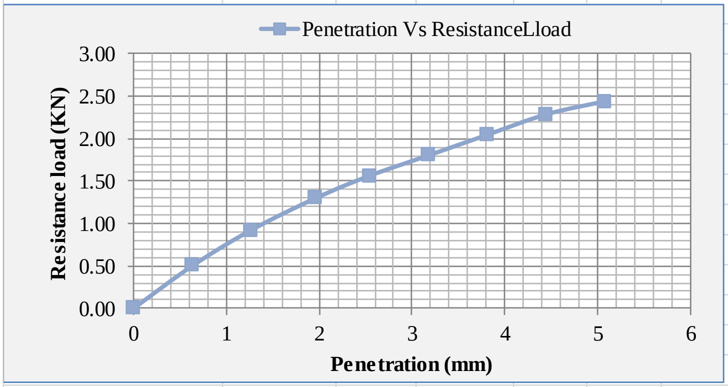
Additive content	CRUSHER DUST		
sample number	T16	T42	T43
mass of empty bottle (M1) in gms.	122.16	128.41	122.16
mass of bottle+ dry soil (M2) in gms.	172.16	178.41	172.16
mass of bottle + dry soil + water (M3) in gms.	399.03	401.48	399.03
mass of bottle + water (M4) in gms.	367.355	369.67	367.355
Observed temperature	21	23	22
K Temperature correction	1.009	1.0005	1.007
specific gravity	2.7530696	2.7501374	2.747612551
Avg. specific gravity	**2.7502732**		

5. CBR and CBR Swelling

CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	23.0	42.0	59.0	71.0	82.0	93.0	104.0	111.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.51	0.92	1.30	1.56	1.80	2.05	2.29	2.44

Compaction Data		OMC	10.346%	MDD	2.006g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.65	1.56	2.44	11.833	12.210	0.120
MDD		2.006g/cm3				
CBR at MDD		11.833%				



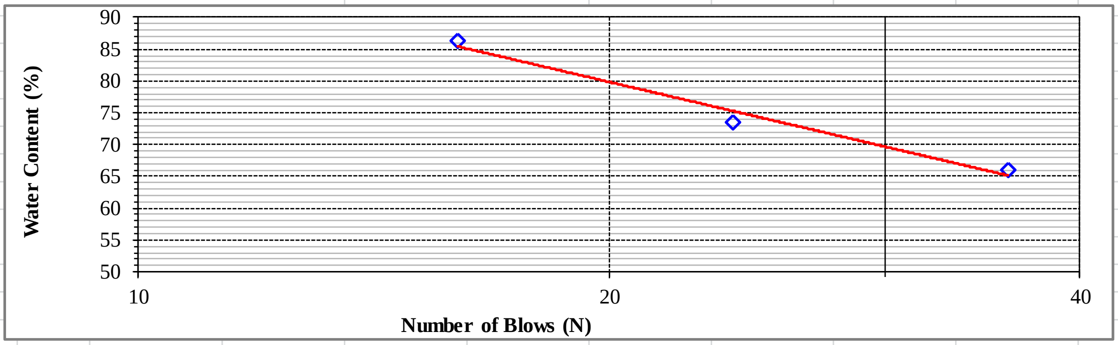
Soaking condition	56 Blows	
	Before	After
Mold number	T1	
Weght of soil+Mold (gm)	10808.2	11653.2
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4308.2	5153.2
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.03	2.43
Moisture content (%)	22.97	39.83
Dry density of soil (g/cm3)	1.65	1.74
Dial gage reading of Height H1	45.23	
Dial gage reading of Height H2		45.37

Appendix C: Laboratory Test Result of Crusher dust stabilized Expansive soil

1.Atterberg Limits

Additive Content			SOIL + 5% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				36	24	16
Can Number	---	---	14.00	23.00	54.00	E13	C45	C31
Mass of Empty Can	M_C	(g)	5.48	5.56	6.42	17.50	17.30	18.03
Mass Can & Soil (Wet)	M_{CMS}	(g)	12.95	12.27	14.88	42.35	46.73	47.05
Mass Can & Soil (Dry)	M_{CDS}	(g)	11.04	10.60	12.69	32.47	34.26	33.60
Mass of Soil	M_S	(g)	5.56	5.04	6.28	14.97	16.96	15.57
Mass of Water	M_W	(g)	1.91	1.67	2.19	9.88	12.47	13.44
Water Content	w	(%)	34.38	33.19	34.91	66.02	73.51	86.33

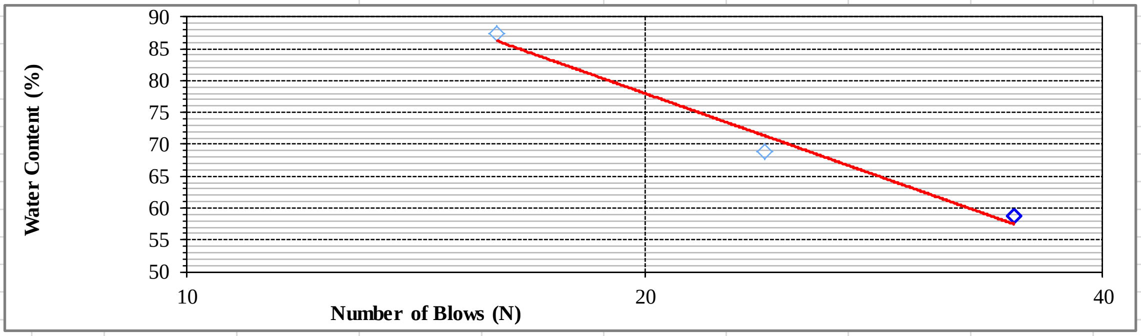
Liquid Limit (LL or w_L) (%):	75.15	PI at "A" Line [25 No. of Blow] =
Plastic Limit (PL or w_P) (%):	34.16	75.15
Plasticity Index (PI) (%):	40.99	



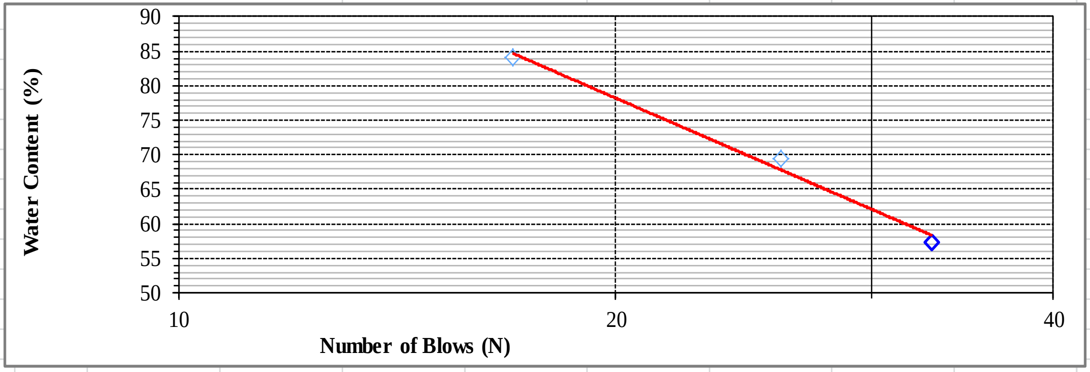
Atterberg Limits Data Sheet

Additive Content			SOIL + 10% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				35	24	16
Can Number	---	---	12.00	6.00	13.00	E	45.00	C3
Mass of Empty Can	M_C	(g)	5.42	5.26	6.36	17.05	17.35	17.58
Mass Can & Soil (Wet)	M_{CMS}	(g)	12.93	12.26	14.88	43.33	49.92	44.18
Mass Can & Soil (Dry)	M_{CDS}	(g)	11.02	10.58	12.73	33.60	36.64	31.77
Mass of Soil	M_S	(g)	5.60	5.33	6.37	16.55	19.29	14.19
Mass of Water	M_W	(g)	1.91	1.68	2.15	9.73	13.28	12.41
Water Content	w	(%)	34.11	31.51	33.76	58.75	68.86	87.42

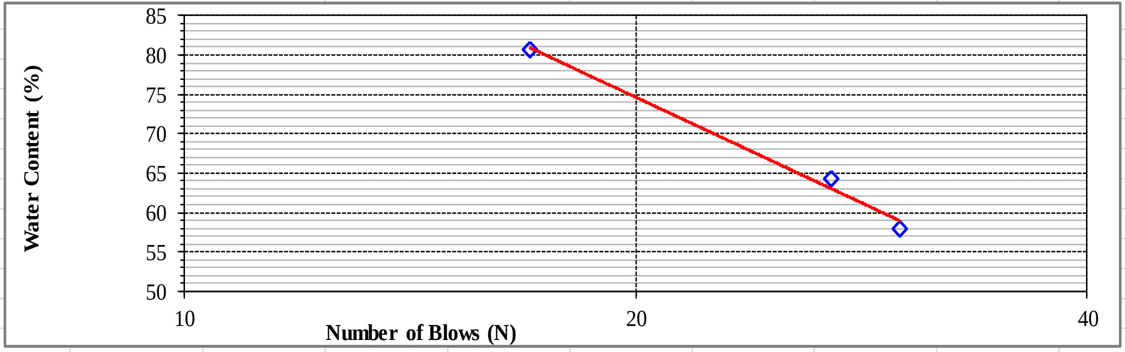
Liquid Limit (LL or w_L) (%):	71.07	PI at "A" Line [25 No. of Blow] =
Plastic Limit (PL or w_P) (%):	33.13	71.07
Plasticity Index (PI) (%):	37.95	



Additive Content			SOIL + 15% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				33	26	17
Can Number	---	---	12	6	13	E	45	C3
Mass of Empty Can	M_C	(g)	5.81	5.44	6.35	17.25	17.48	17.66
Mass Can & Soil (Wet)	M_{CMS}	(g)	13.48	14.52	14.86	42.43	46.96	43.49
Mass Can & Soil (Dry)	M_{CDS}	(g)	11.59	12.32	12.76	33.26	34.89	31.69
Mass of Soil	M_S	(g)	5.79	6.88	6.41	16.01	17.41	14.03
Mass of Water	M_W	(g)	1.88	2.19	2.10	9.18	12.07	11.80
Water Content	w	(%)	32.50	31.86	32.72	57.32	69.35	84.09
Liquid Limit (LL or w_L) (%):			68.52		PI at "A" Line [25 No. of Blow] =			
Plastic Limit (PL or w_P) (%):			32.36			68.52		
Plasticity Index (PI) (%):			36.16					

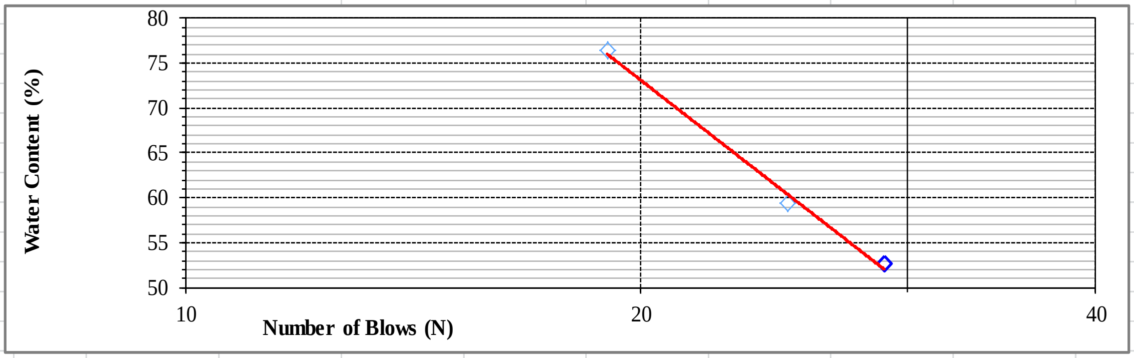


Additive Content			SOIL + 20% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				30	27	17
Can Number	---	---	J	D2	LL3	D53	K-9	T2C1
Mass of Empty Can	M_C	(g)	6.19	5.62	6.34	17.44	17.61	17.74
Mass Can & Soil (Wet)	M_{CMS}	(g)	14.02	16.77	14.83	42.54	43.12	42.80
Mass Can & Soil (Dry)	M_{CDS}	(g)	12.17	14.06	12.79	33.33	33.14	31.61
Mass of Soil	M_S	(g)	5.98	8.44	6.45	15.89	15.53	13.87
Mass of Water	M_W	(g)	1.85	2.71	2.04	9.21	9.98	11.19
Water Content	w	(%)	31.00	32.09	31.68	57.93	64.29	80.69
Liquid Limit (LL or w_L) (%):			65.24		PI at "A" Line [25 No. of Blow] =			
Plastic Limit (PL or w_P) (%):			31.59			65.24		
Plasticity Index (PI) (%):			33.65					



Additive Content			SOIL + 25% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				29	25	19
Can Number	---	---	A2	A3	PB2	P33	AFE-3	T3C2
Mass of Empty Can	M_C	(g)	6.10	5.81	5.92	18.00	17.47	17.52
Mass Can & Soil (Wet)	M_{CMS}	(g)	14.71	15.90	15.72	42.36	42.54	44.03
Mass Can & Soil (Dry)	M_{CDS}	(g)	12.80	13.67	13.51	33.96	33.20	32.55
Mass of Soil	M_S	(g)	6.69	7.86	7.59	15.96	15.73	15.03
Mass of Water	M_W	(g)	1.92	2.23	2.21	8.40	9.34	11.48
Water Content	w	(%)	28.65	28.32	29.08	52.66	59.39	76.38

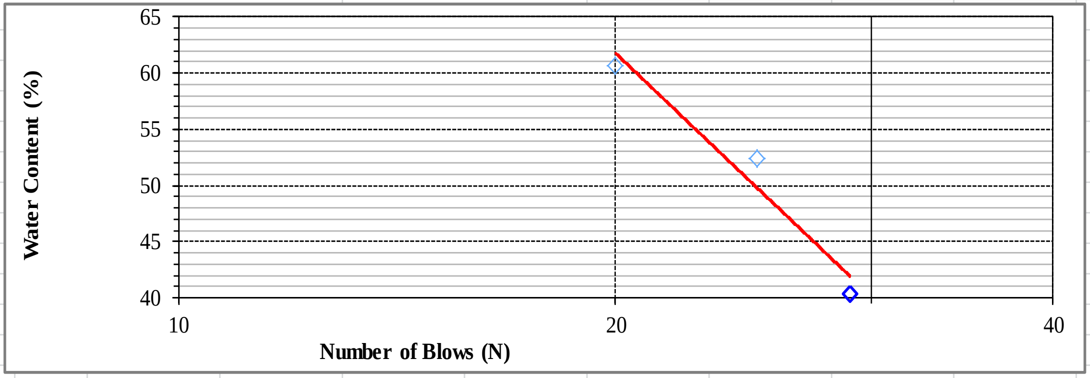
Liquid Limit (LL or w_L) (%):	60.99	PI at "A" Line [25 No. of Blow] =
Plastic Limit (PL or w_P) (%):	28.68	60.99
Plasticity Index (PI) (%):	32.30	



Additive Content			SOIL + 30% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				29	24	21
Can Number	---	---	A2	A3	PB2	P33	AFE-3	T3C2
Mass of Empty Can	M_C	(g)	6.02	6.01	5.51	18.56	17.34	17.30
Mass Can & Soil (Wet)	M_{CMS}	(g)	15.40	15.03	16.61	43.18	39.08	44.25
Mass Can & Soil (Dry)	M_{CDS}	(g)	13.42	13.28	14.24	35.58	31.26	33.49
Mass of Soil	M_S	(g)	7.41	7.28	8.73	17.02	13.92	16.19
Mass of Water	M_W	(g)	1.98	1.74	2.37	7.60	7.82	10.77
Water Content	w	(%)	26.76	23.95	27.16	44.65	56.13	66.51

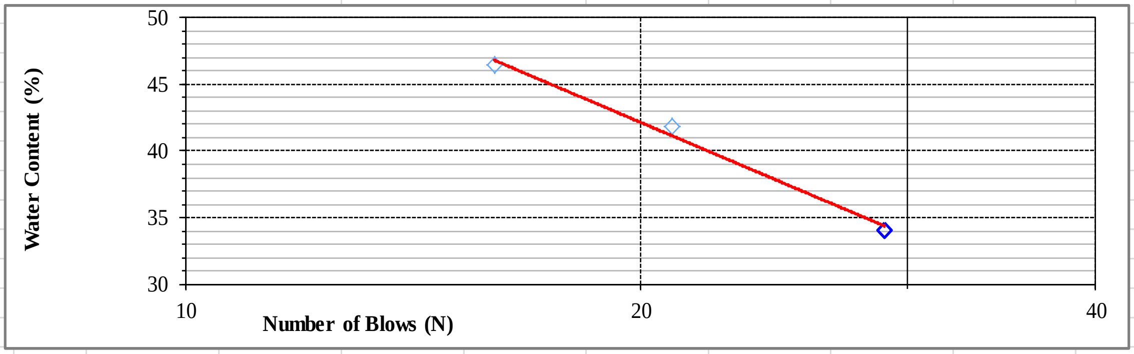
Liquid Limit (LL or w_L) (%):	54.70	PI at "A" Line [25 No. of Blow] =
Plastic Limit (PL or w_P) (%):	25.96	54.70
Plasticity Index (PI) (%):	28.75	

Additive Content			SOIL + 35% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				29	25	20
Can Number	---	---	A2	A3	PB2	P33	AFE-3	T3C2
Mass of Empty Can	M_C	(g)	6.23	5.99	5.84	18.05	17.64	17.63
Mass Can & Soil (Wet)	M_{CMS}	(g)	16.48	15.69	15.01	42.76	41.55	44.98
Mass Can & Soil (Dry)	M_{CDS}	(g)	14.33	13.78	13.14	35.65	33.32	34.65
Mass of Soil	M_S	(g)	8.10	7.79	7.30	17.60	15.68	17.02
Mass of Water	M_W	(g)	2.15	1.91	1.87	7.10	8.23	10.34
Water Content	w	(%)	26.55	24.54	25.63	40.35	52.47	60.72
Liquid Limit (LL or w_L) (%):	48.49		PI at "A" Line [25 No. of Blow] =					
Plastic Limit (PL or w_P) (%):	25.57			48.49				
Plasticity Index (PI) (%):	22.92							

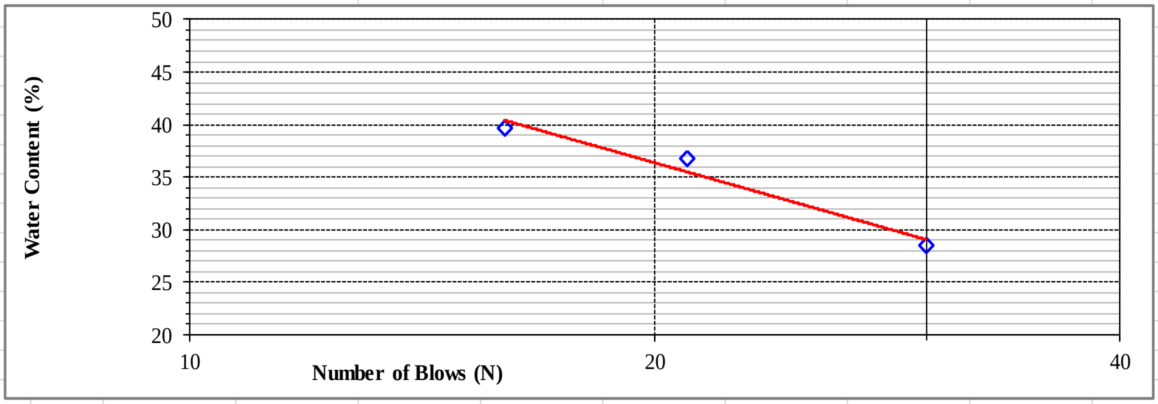


Additive Content			SOIL + 40% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				28	26	19
Can Number	---	---	DB1	B1	PL1	H-2	T34	D12
Mass of Empty Can	M_C	(g)	6.45	5.98	6.17	17.55	17.95	17.96
Mass Can & Soil (Wet)	M_{CMS}	(g)	17.56	16.36	13.41	43.33	43.03	45.71
Mass Can & Soil (Dry)	M_{CDS}	(g)	15.24	14.28	12.04	35.78	35.39	35.81
Mass of Soil	M_S	(g)	8.80	8.30	5.87	18.23	17.44	17.85
Mass of Water	M_W	(g)	2.32	2.08	1.37	7.55	7.64	9.91
Water Content	w	(%)	26.37	25.06	23.35	41.40	43.81	55.48
Liquid Limit (LL or w_L) (%):	45.52		PI at "A" Line [25 No. of Blow] =					
Plastic Limit (PL or w_P) (%):	24.93			45.52				
Plasticity Index (PI) (%):	20.59							

Additive Content			SOIL + 45% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				29	21	16
Can Number	---	---	DB1	B1	PL1	NC13	P12	LC31
Mass of Empty Can	M_C	(g)	6.40	6.15	6.53	18.49	18.31	17.37
Mass Can & Soil (Wet)	M_{CMS}	(g)	15.49	17.43	18.26	46.95	49.64	47.93
Mass Can & Soil (Dry)	M_{CDS}	(g)	14.00	15.30	16.24	39.72	40.40	38.24
Mass of Soil	M_S	(g)	7.60	9.15	9.71	21.23	22.09	20.87
Mass of Water	M_W	(g)	1.50	2.12	2.02	7.23	9.24	9.69
Water Content	w	(%)	19.70	23.21	20.84	34.05	41.82	46.44
Liquid Limit (LL or w_L) (%):		37.14		PI at "A" Line [25 No. of Blow] =				
Plastic Limit (PL or w_P) (%):		21.25			37.14			
Plasticity Index (PI) (%):		15.89						



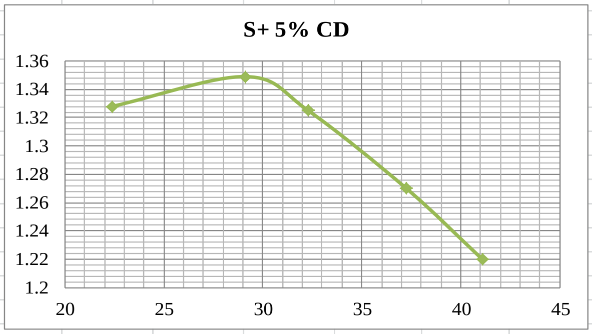
Additive Content			SOIL + 50% CRUSHER DUST					
Test			Plastic Limit			Liquid Limit		
Variable	NO		1	2	3	1	2	3
	Var.	Units						
Number of Blows	N	blows				30	21	16
Can Number	---	---	DB1	B1	PL1	NC13	P12	LC31
Mass of Empty Can	M_C	(g)	6.35	6.32	6.89	19.43	18.68	16.78
Mass Can & Soil (Wet)	M_{CMS}	(g)	13.43	18.49	23.11	50.57	55.26	50.14
Mass Can & Soil (Dry)	M_{CDS}	(g)	12.75	16.32	20.44	43.66	45.42	40.67
Mass of Soil	M_S	(g)	6.40	10.01	13.54	24.23	26.74	23.89
Mass of Water	M_W	(g)	0.68	2.17	2.68	6.91	9.84	9.48
Water Content	w	(%)	10.54	21.67	19.75	28.51	36.78	39.68
Liquid Limit (LL or w_L) (%):		31.75		PI at "A" Line [25 No. of Blow] =				
Plastic Limit (PL or w_P) (%):		17.32			31.75			
Plasticity Index (PI) (%):		14.43						



2.Compaction test result

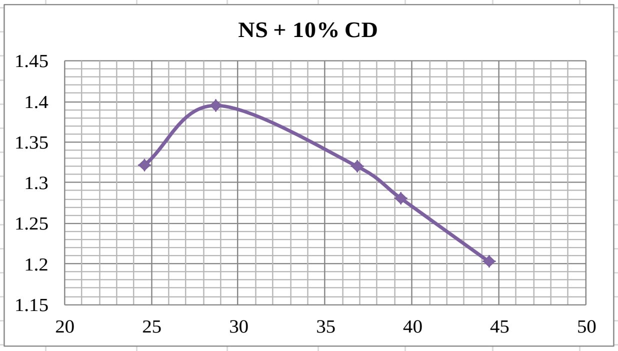
ADDITIVE CONTENT		SOIL + 5% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4535		4645		4655		4645		4625	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1535		1645		1655		1645		1625	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.626		1.743		1.753		1.743		1.721	
MOISTURE CONTET DETERMINATION										
Container Code .	D31	N73	H2	606	T1C1	K9	T2C1	D4	T2C	D
Mass of Wet soil+Container(gm)(F)	66.98	71.25	66.289	92.867	85.618	91.436	104.771	92.809	101.789	78.677
Mass of dry soil+container(gm)(G)	58.36	61.02	55.021	76.126	68.929	73.479	80.912	72.578	77.495	60.98
Mass of container(gm)(H)	18.25	17.16	17.211	17.299	17.694	17.39	17.589	17.635	17.735	18.345
Mass of moisture(gm)F-G=(I)	8.62	10.23	11.268	16.741	16.689	17.957	23.859	20.231	24.294	17.697
Mass of Dry soil(gm)G-H=(J)	40.11	43.86	37.81	58.827	51.235	56.089	63.323	54.943	59.76	42.635
Moisture content % (I/J)*100=K	21.49	23.32	29.80	28.46	32.57	32.02	37.68	36.82	40.65	41.51
Avg. Moisture Content % (L)	22.408		29.130		32.294		37.250		41.080	
Dry Density gm/cm^3 E/(100+L)*100	1.328		1.349		1.325		1.270		1.220	

OMC	29.130%
MDD	1.349g/cm3



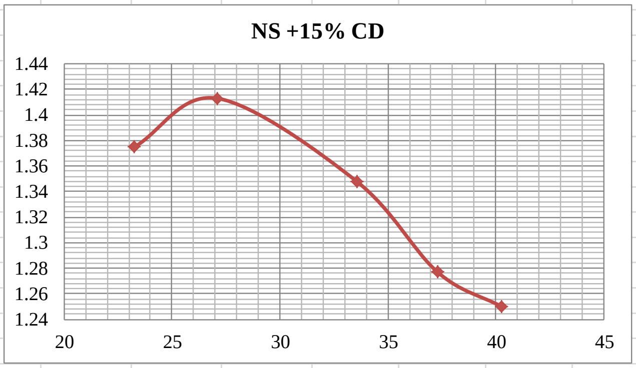
ADDITIVE CONTENT		SOIL + 10% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4555		4695		4705		4685		4640	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1555		1695		1705		1685		1640	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.647		1.796		1.806		1.785		1.737	
MOISTURE CONTET DETERMINATION										
Container Code .	X	D32	9	NC42	24	LC42	LC12	A3	46	CA
Mass of Wet soil+Container(gm)(F)	100.2	95.86	57.448	71.476	87.77	68.408	82.901	79.174	89	79.401
Mass of dry soil+container(gm)(G)	84.23	80.15	48.297	59.838	68.906	54.726	64.418	62.003	67.219	60.443
Mass of container(gm)(H)	17.97	17.7	17.567	17.82	17.14	18	17.67	18.14	18.01	17.888
Mass of moisture(gm)F-G=(I)	15.97	15.71	9.151	11.638	18.864	13.682	18.483	17.171	21.781	18.958
Mass of Dry soil(gm)G-H=(J)	66.26	62.45	30.73	42.018	51.766	36.726	46.748	43.863	49.209	42.555
Moisture content % (I/J)*100=K	24.10	25.16	29.78	27.70	36.44	37.25	39.54	39.15	44.26	44.55
Avg. Moisture Content % (L)	24.629		28.738		36.848		39.342		44.406	
Dry Density gm/cm^3 E/(100+L)*100	1.322		1.395		1.320		1.281		1.203	

OMC	28.738%
MDD	1.395g/cm3



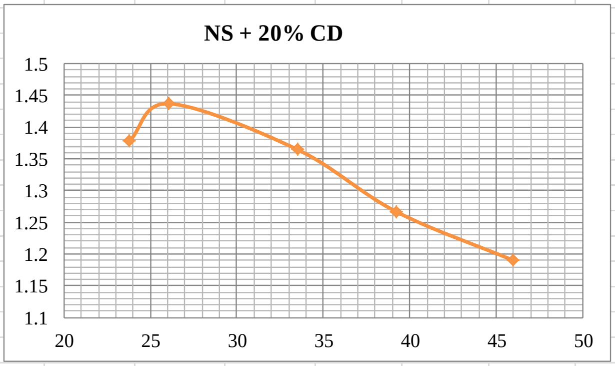
ADDITIVE CONTENT	NATURAL SOIL + 15% CRUSHER DUST									
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4600		4695		4700		4655		4655	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1600		1695		1700		1655		1655	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.695		1.796		1.801		1.753		1.753	
MOISTURE CONTET DETERMINATION										
Container Code .	D31	9	NC422	24	LC42	LC12	36	14	N1	14
Mass of Wet soil+Container(gm)(F)	64.024	69.700	87.567	76.396	81.335	74.372	82.996	82.816	82.996	82.816
Mass of dry soil+container(gm)(G)	55.312	59.806	72.853	63.728	65.763	59.778	65.192	65.351	64.192	64.351
Mass of container(gm)(H)	17.541	17.665	17.108	18.155	17.664	17.771	17.611	18.389	17.611	18.389
Mass of moisture(gm)F-G=(I)	8.712	9.894	14.714	12.668	15.5725	14.594	17.804	17.465	18.804	18.465
Mass of Dry soil(gm)G-H=(J)	37.771	42.141	55.745	45.573	48.099	42.0075	47.5815	46.9615	46.5815	45.9615
Moisture content % (I/J)*100=K	23.07	23.48	26.40	27.80	32.38	34.74	37.42	37.19	40.37	40.17
Avg. Moisture Content % (L)	23.272		27.096		33.559		37.304		40.271	
Dry Density gm/cm^3 E/(100+L)*100	1.375		1.413		1.348		1.277		1.250	

OMC	27.096%
MDD	1.413g/cm3



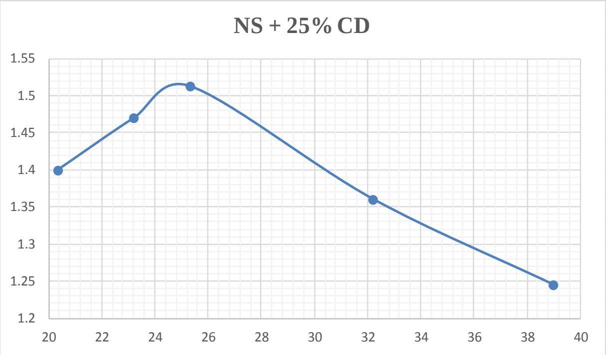
ADDITIVE CONTENT	NATURAL SOIL + 20% CRUSHER DUST									
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4610		4710		4720		4665		4640	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1610		1710		1720		1665		1640	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.706		1.811		1.822		1.764		1.737	
MOISTURE CONTET DETERMINATION										
Container Code .	T1C1	GS3	G10	NC21	G73	G-6-3	B3	82	GBP1	G-10-5
Mass of Wet soil+Container(gm)(F)	70.599	67.924	87.364	84.383	79.769	69.57	76.992	86.23	78.26	82.84
Mass of dry soil+container(gm)(G)	59.845	58.774	72.8	70.729	64.107	56.553	60.165	67.258	62.706	59.119
Mass of container(gm)(H)	17.514	17.51	17.076	18.309	17.657	17.401	17.211	18.89	17.446	17.941
Mass of moisture(gm)F-G=(I)	10.754	9.15	14.564	13.654	15.662	13.017	16.827	18.972	15.554	23.721
Mass of Dry soil(gm)G-H=(J)	42.331	41.264	55.724	52.42	46.45	39.152	42.954	48.368	45.26	41.178
Moisture content % (I/J)*100=K	25.40	22.17	26.14	26.05	33.72	33.25	39.17	39.22	34.37	57.61
Avg. Moisture Content % (L)	23.789		26.092		33.483		39.199		45.986	
Dry Density gm/cm^3 E/(100+L)*100	1.378		1.437		1.365		1.267		1.190	

OMC	26.092%
MDD	1.437g/cm3



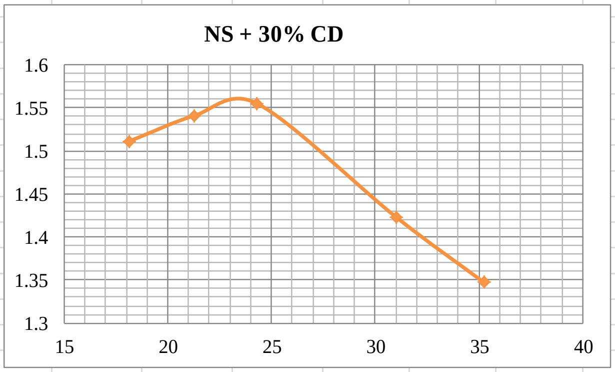
ADDITIVE CONTENT		NATURAL SOIL + 25% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4575		4695		4775		4684		4620	
Wgt. of Mould (gm) B	2985		2985		2985		2985		2985	
Wgt. of wet soil (gm) A-B=C	1590		1710		1790		1699		1635	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.684		1.811		1.896		1.800		1.732	
MOISTURE CONTET DETERMINATION										
Container Code .	G1	PL.1.1	NC22	LC42	T1	30	KLC11	1.5.2	C5	T3
Mass of Wet soil+Container(gm)(F)	76.360	68.950	66.821	72.438	92.264	91.165	83.011	83.840	76.992	96.838
Mass of dry soil+container(gm)(G)	66.590	60.230	57.514	62.249	77.572	75.892	66.980	67.750	59.165	76.507
Mass of container(gm)(H)	17.368	18.36	17.368	18.36	17.601	17.4885	17.6595	17.3095	17.363	18.8915
Mass of moisture(gm)F-G=(I)	9.77	8.72	9.30705	10.189	14.6915	15.2735	16.031	16.09	17.827	20.331
Mass of Dry soil(gm)G-H=(J)	49.222	41.87	40.1455	43.889	59.971	58.403	49.32	50.4405	41.802	57.615
Moisture content % (I/J)*100=K	19.85	20.83	23.18	23.22	24.50	26.15	32.50	31.90	42.65	35.29
Avg. Moisture Content % (L)	20.338		23.199		25.325		32.202		38.967	
Dry Density gm/cm^3 E/(100+L)*100	1.400		1.470		1.513		1.361		1.246	

OMC	25.325%
MDD	1.513g/cm3



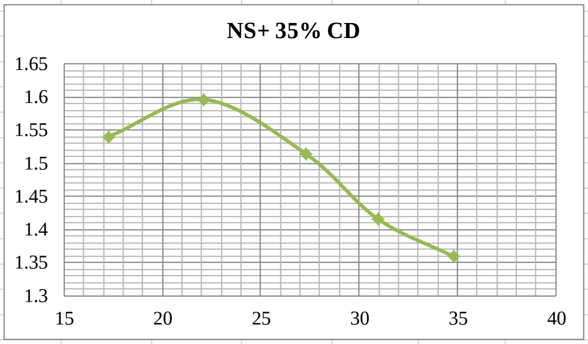
ADDITIVE CONTENT		NATURAL SOIL + 30% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4685		4765		4825		4760		4720	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1685		1765		1825		1760		1720	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.785		1.870		1.933		1.864		1.822	
MOISTURE CONTET DETERMINATION										
Container Code .	KL	GH	T34	D12	E	G53	P33	H2	D51	45
Mass of Wet soil+Container(gm)(F)	63.042	76.952	63.042	76.952	97.163	97.947	97.639	107.445	96.202	99.192
Mass of dry soil+container(gm)(G)	56.182	67.724	54.230	67.724	81.250	82.530	78.428	86.755	75.505	78.382
Mass of container(gm)(H)	17.875	17.456	17.875	17.456	18.126	16.668	17.515	18.893	17.952	18.144
Mass of moisture(gm)F-G=(I)	6.8601	9.228	8.8121	9.228	15.913	15.417	19.211	20.69	20.697	20.81
Mass of Dry soil(gm)G-H=(J)	38.307	50.268	36.355	50.268	63.124	65.862	60.913	67.862	57.553	60.238
Moisture content % (I/J)*100=K	17.91	18.36	24.24	18.36	25.21	23.41	31.54	30.49	35.96	34.55
Avg. Moisture Content % (L)	18.13		21.30		24.31		31.01		35.25	
Dry Density gm/cm^3 E/(100+L)*100	1.511		1.541		1.555		1.423		1.347	

OMC	24.310%
MDD	1.555g/cm3



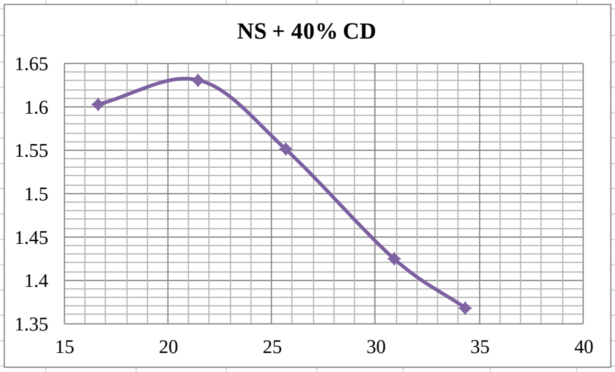
ADDITIVE CONTENT		SOIL + 35% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4705		4840		4820		4750		4730	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1705		1840		1820		1750		1730	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.806		1.949		1.928		1.854		1.833	
MOISTURE CONTET DETERMINATION										
Container Code .	B	CS	D31	N73	H2	606	T1C1	K9	T2C1	D4
Mass of Wet soil+Container(gm)(F)	76.56	79.90	92.16	99.58	91.75	91.72	98.78	104.59	91.77	94.10
Mass of dry soil+container(gm)(G)	67.8425	70.756	78.677	84.65	76	75.672	79.5335	84.274	72.4675	74.6385
Mass of container(gm)(H)	17.639	17.4875	17.893	17.0365	17.824	17.405	17.671	18.23	17.5365	18.2325
Mass of moisture(gm)F-G=(I)	8.71605	9.14	13.484	14.927	15.7465	16.0435	19.2445	20.3205	19.298	19.4575
Mass of Dry soil(gm)G-H=(J)	50.2035	53.2685	60.784	67.6135	58.176	58.267	61.8625	66.044	54.931	56.406
Moisture content % (I/J)*100=K	17.36	17.16	22.18	22.08	27.07	27.53	31.11	30.77	35.13	34.50
Avg. Moisture Content % (L)	17.26		22.13		27.30		30.94		34.81	
Dry Density gm/cm^3 E/(100+L)*100	1.540		1.596		1.514		1.416		1.359	

OMC	22.130%
MDD	1.596g/cm3



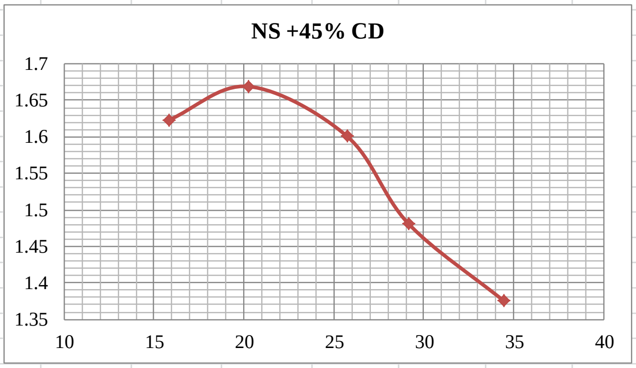
ADDITIVE CONTENT		SOIL + 40% CRUSHER DUST								
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4765		4870		4840		4760		4735	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1765		1870		1840		1760		1735	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.870		1.981		1.949		1.864		1.838	
MOISTURE CONTET DETERMINATION										
Container Code .	G1	PL.1.1	D32	9	NC42	24	LC42	LC12	A3	46
Mass of Wet soil+Container(gm)(F)	90.075	82.84	87.159	101.207	97.241	85.321	99.917	101.744	87.329	89
Mass of dry soil+container(gm)(G)	79.503	73.788	75.01	86.246	81.148	71.397	80.639	81.793	69.43	70.895
Mass of container(gm)(H)	17.997	17.519	17.66	17.405	17.986	17.592	17.827	17.567	17.121	18.321
Mass of moisture(gm)F-G=(I)	10.572	9.052	12.149	14.961	16.093	13.924	19.278	19.951	17.899	18.105
Mass of Dry soil(gm)G-H=(J)	61.506	56.269	57.35	68.841	63.162	53.805	62.812	64.226	52.309	52.574
Moisture content % (I/J)*100=K	17.19	16.09	21.18	21.73	25.48	25.88	30.69	31.06	34.22	34.44
Avg. Moisture Content % (L)	16.64		21.46		25.68		30.88		34.33	
Dry Density gm/cm^3 E/(100+L)*100	1.603		1.631		1.551		1.425		1.368	

OMC	21.460%
MDD	1.631g/cm3



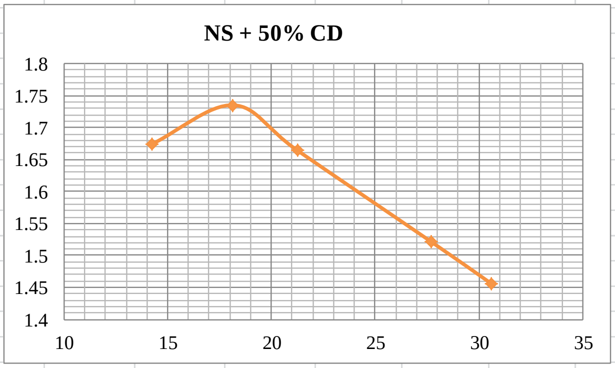
ADDITIVE CONTENT			NATURAL SOIL + 45% CRUSHER DUST							
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4775		4895		4900		4805		4745	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1775		1895		1900		1805		1745	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.880		2.007		2.013		1.912		1.849	
MOISTURE CONTET DETERMINATION										
Container Code .	G1	PL.1.1	D31	9	NC422	24	LC42	LC12	36	14
Mass of Wet soil+Container(gm)(F)	72.52	63.23	64.0235	69.7	87.567	76.3955	81.335	74.372	82.996	82.8155
Mass of dry soil+container(gm)(G)	64.23	57.62	56.3115	60.806	72.853	64.7275	66.7625	61.778	66.192	66.3505
Mass of container(gm)(H)	17.14	17.72	17.5405	17.665	17.108	18.1545	17.6635	17.7705	17.6105	18.389
Mass of moisture(gm)F-G=(I)	8.29	5.61	7.712	8.894	14.714	11.668	14.5725	12.594	16.804	16.465
Mass of Dry soil(gm)G-H=(J)	47.09	39.9	38.771	43.141	55.745	46.573	49.099	44.0075	48.5815	47.9615
Moisture content % (I/J)*100=K	17.60	14.06	19.89	20.62	26.40	25.05	29.68	28.62	34.59	34.33
Avg. Moisture Content % (L)	15.832		20.254		25.724		29.149		34.459	
Dry Density gm/cm^3 E/(100+L)*100	1.623		1.669		1.601		1.481		1.375	

OMC	20.254%
MDD	1.669g/cm3



ADDITIVE CONTENT			NATURAL SOIL + 50% CRUSHER DUST							
DENSITY DETERMINATION										
Trial No	1		2		3		4		5	
Wgt. of Mould +Wet soil (gm) A	4805		4935		4905		4835		4795	
Wgt. of Mould (gm) B	3000		3000		3000		3000		3000	
Wgt. of wet soil (gm) A-B=C	1805		1935		1905		1835		1795	
Volume of mould (cm3) D	944		944		944		944		944	
Wet Density(gm/cm3) C/D=E	1.912		2.050		2.018		1.944		1.901	
MOISTURE CONTET DETERMINATION										
Container Code .	T1C1	GS3	G10	NC21	G73	G-6-3	B3	82	GBP1	G-10-5
Mass of Wet soil+Container(gm)(F)	70.599	67.924	87.364	84.383	78.769	68.57	76.992	86.23	78.26	82.84
Mass of dry soil+container(gm)(G)	63.845	61.774	76.985	73.729	68.107	59.553	64.165	71.258	67.706	64.119
Mass of container(gm)(H)	17.514	17.514	17.214	17.514	17.514	17.514	17.514	17.514	17.514	17.514
Mass of moisture(gm)F-G=(I)	6.754	6.15	10.379	10.654	10.662	9.017	12.827	14.972	10.554	18.721
Mass of Dry soil(gm)G-H=(J)	46.331	44.26	59.771	56.215	50.593	42.039	46.651	53.744	50.192	46.605
Moisture content % (I/J)*100=K	14.58	13.90	17.36	18.95	21.07	21.45	27.50	27.86	21.03	40.17
Avg. Moisture Content % (L)	14.236		18.158		21.262		27.677		30.598	
Dry Density gm/cm^3 E/(100+L)*100	1.674		1.735		1.664		1.522		1.456	

OMC	18.16%
MDD	1.735g/cm3



3. Free swelling Indexes

Additive content	5% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	18	18.2	18.2
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	80	82	82
Average Free Swell index	81%		

Additive content	10% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	16.5	16.4	17.5
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	65	64	75
Average Free Swell index	68%		

Additive content	15% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	15.6	15.8	16.5
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	56	58	65
Average Free Swell index	60%		

Additive content	20% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	15.0	14.8	15.4
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	50	48	54
Average Free Swell index	51%		

Additive content	25% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	13.5	13.8	14.4
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	35	38	44.48
Average Free Swell index	39%		

Additive content	30% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	12.5	12.8	13.0
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	25	28	30
Average Free Swell index	28%		

Additive content	35% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	12.0	12.5	12.7
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	20	25	26.582
Average Free Swell index	24%		

Additive content	40% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	11.5	11.5	12.4
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	15	15	23.946
Average Free Swell index	18%		

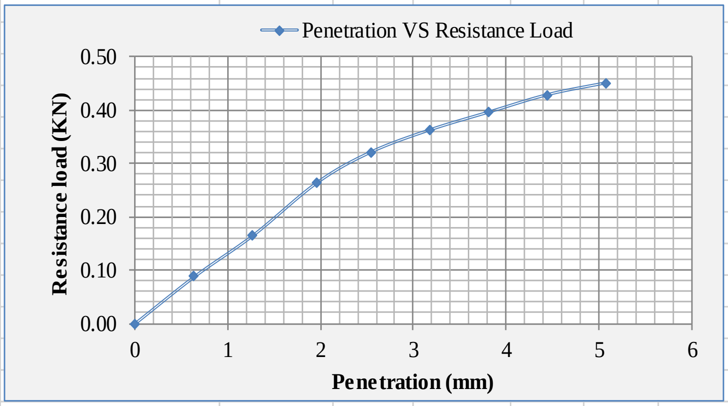
Additive content	45% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	11.5	11.0	11.4
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	15	10	14.091

Additive content	50% Crusher dust		
Readings on the Glass Jar	S1	S2	S3
Vw = volume of soil specimen read from the graduated cylinder containing distilled water.	10.5	10.7	12.0
Vk = volume of soil specimen read from the graduated cylinder containing kerosene	10	10	10
Free swell index= [Vd - Vk] / Vk x 100%	5	7	19.781
Average Free Swell index	11%		

4. CBR Test& CBR Swell

SOIL + 5% CRUSHER DUST									
Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	4.0	7.5	12.0	14.6	16.5	18.0	19.5	20.5
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.09	0.17	0.26	0.32	0.36	0.40	0.43	0.45

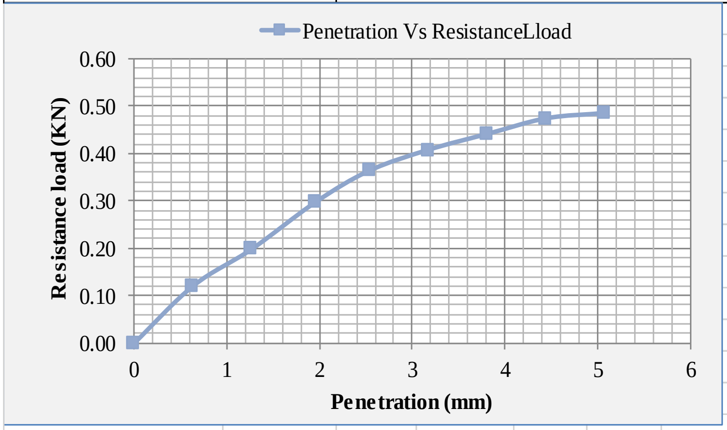
Compaction Data		OMC	29.130%	MDD	1.349g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.51	0.32	0.45	2.433	2.255	2.648
MDD		**1.349g/cm3**				
CBR at MDD		**2.433%**				



Soaking condition	56 Blows	
	Before	After
Mold number	CM1	
Weght of soil+Mold (gm)	10492.5	11150.5
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	3992.5	4650.5
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	1.88	2.19
Moisture content (%)	24.23	42.19
Dry density of soil (g/cm3)	1.51	1.54
Dial gage reading of Height H1	46.01	
Dial gage reading of Height H2		49.09

SOIL + 10% CRUSHER DUST									
Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	5.5	9.0	13.5	16.5	18.5	20.0	21.5	22.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.12	0.20	0.30	0.36	0.41	0.44	0.47	0.48

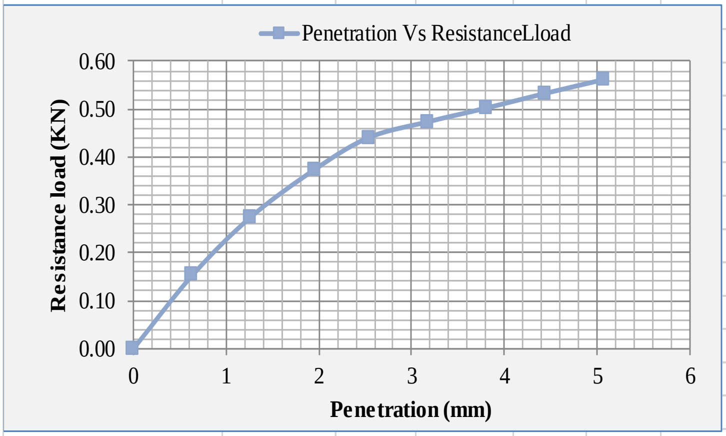
Compaction Data		OMC	28.739%	MDD	1.395g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.62	0.36	0.48	2.750	2.420	2.277
MDD		**1.395g/cm3**				
CBR at MDD		**2.750%**				



Soaking condition	56 Blows	
	Before	After
Mold number	BT	
Weght of soil+Mold (gm)	10789.6	12063.2
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4289.6	5563.2
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.02	2.62
Moisture content (%)	24.9	45.9
Dry density of soil (g/cm3)	1.62	1.80
Dial gage reading of Height H1	45.81	
Dial gage reading of Height H2		48.46

SOIL + 15% CRUSHER DUST									
Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	7.0	12.5	17.0	20.0	21.5	22.8	24.2	25.5
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.15	0.28	0.37	0.44	0.47	0.50	0.53	0.56

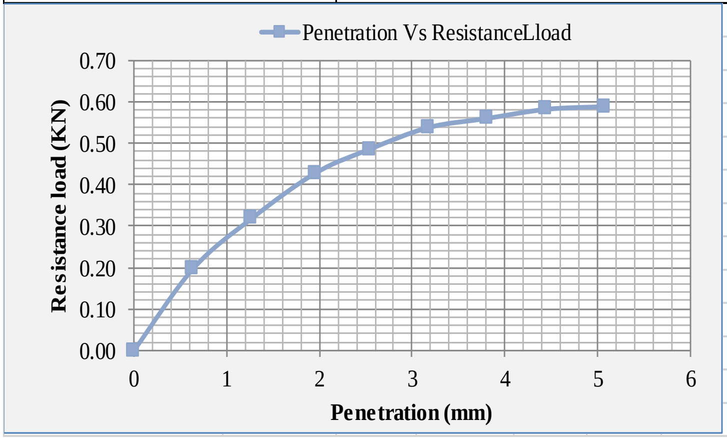
Compaction Data		OMC	27.096%	MDD	1.413g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.77	0.44	0.56	3.333	2.805	1.892
MDD		**1.413g/cm3**				
CBR at MDD		**3.333%**				



Soaking condition	56 Blows	
	Before	After
Mold number	BY	
Weght of soil+Mold (gm)	11176.8	12196.5
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4676.8	5696.5
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.20	2.68
Moisture content (%)	24.51	46.78
Dry density of soil (g/cm3)	1.77	1.83
Dial gage reading of Height H1	45.18	
Dial gage reading of Height H2		47.38

SOIL + 20% CRUSHER DUST									
Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	9.0	14.5	19.5	22.1	24.5	25.5	26.5	26.8
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.20	0.32	0.43	0.49	0.54	0.56	0.58	0.59

Compaction Data		OMC	26.092%	MDD	1.437g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.84	0.49	0.59	3.683	2.948	1.754
MDD		**1.437g/cm3**				
CBR at MDD		**3.683%**				

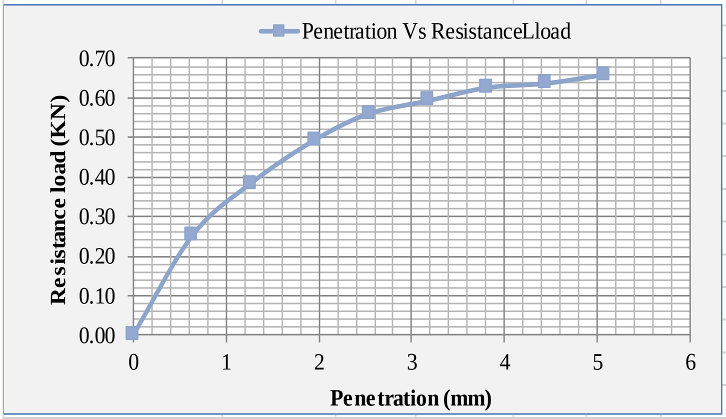


Soaking condition	56 Blows	
	Before	After
Mold number	T1	
Weght of soil+Mold (gm)	11378.3	12369.3
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4878.3	5869.3
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.30	2.76
Moisture content (%)	24.98	48.01
Dry density of soil (g/cm3)	1.84	1.87
Dial gage reading of Height H1	45.32	
Dial gage reading of Height H2		47.36

SOIL + 25% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	11.5	17.5	22.5	25.5	27.0	28.5	29.0	30.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.25	0.39	0.50	0.56	0.59	0.63	0.64	0.66

Compaction Data		OMC	25.33%	MDD	1.513g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.75	0.56	0.66	4.250	3.300	1.512
MDD		**1.513g/cm3**				
CBR at MDD		**4.250%**				

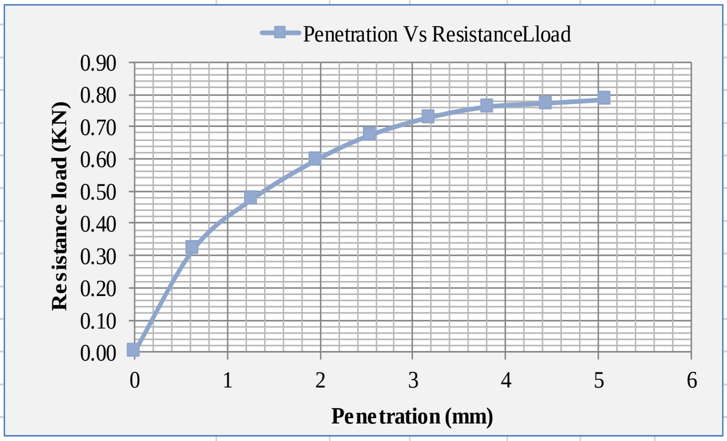


Soaking condition	56 Blows	
	Before	After
Mold number	C1	
Weght of soil+Mold (gm)	11169.3	12018.1
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4669.3	5518.1
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.20	2.60
Moisture content (%)	25.98	47.01
Dry density of soil (g/cm3)	1.75	1.77
Dial gage reading of Height H1	46.29	
Dial gage reading of Height H2		48.05

SOIL + 30% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	14.5	21.5	27.0	30.5	33.0	34.5	35.0	35.5
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.32	0.47	0.59	0.67	0.73	0.76	0.77	0.78

Compaction Data		OMC	24.310%	MDD	1.555g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.62	0.67	0.78	5.083	3.905	1.478
MDD		**1.555g/cm3**				
CBR at MDD		**5.083%**				

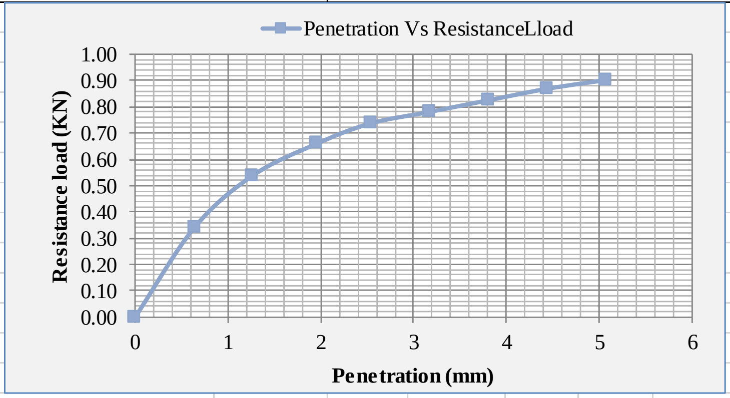


Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	10756.1	11859.2
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4256.1	5359.2
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.00	2.52
Moisture content (%)	23.8	44.32
Dry density of soil (g/cm3)	1.62	1.75
Dial gage reading of Height H1	45.97	
Dial gage reading of Height H2		47.69

SOIL + 35% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	15.5	24.5	30.0	33.5	35.5	37.5	39.5	41.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.34	0.54	0.66	0.74	0.78	0.83	0.87	0.90

Compaction Data		OMC	22.13%	MDD	1.596g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.75	0.74	0.90	5.583	4.510	1.393
MDD		**1.596g/cm3**				
CBR at MDD		**5.583%**				

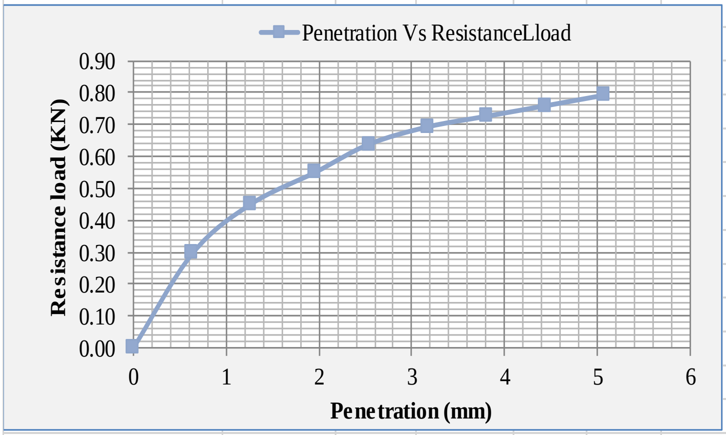


Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	11167.3	12596.6
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	4667.3	6096.6
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.20	2.87
Moisture content (%)	25.8	46.2
Dry density of soil (g/cm3)	1.75	1.96
Dial gage reading of Height H1	45.37	
Dial gage reading of Height H2		46.99

SOIL + 40% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	13.5	20.5	25.0	29.0	31.5	33.0	34.5	36.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.30	0.45	0.55	0.64	0.69	0.73	0.76	0.79

Compaction Data		OMC	21.46%	MDD	1.631g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.92	0.64	0.79	4.833	3.960	1.307
MDD		**1.631g/cm3**				
CBR at MDD		**4.767%**				

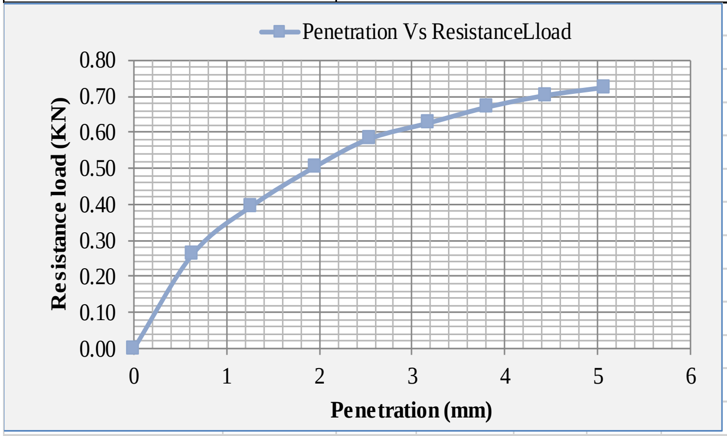


Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	11706.3	13002.8
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	5206.3	6502.8
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.45	3.06
Moisture content (%)	27.5	48.2
Dry density of soil (g/cm3)	1.92	2.07
Dial gage reading of Height H1	46.11	
Dial gage reading of Height H2		47.63

SOIL + 45% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	12.0	18.0	23.0	26.5	28.5	30.5	32.0	33.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.26	0.40	0.51	0.58	0.63	0.67	0.70	0.73

Compaction Data		OMC	20.254%	MDD	1.669g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.82	0.58	0.73	4.417	3.630	1.255
MDD		**1.669g/cm3**				
CBR at MDD		**4.417%**				

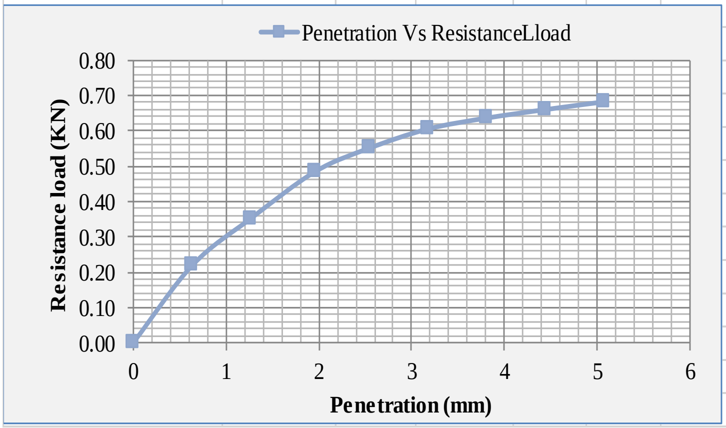


Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	11506.1	12896.1
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	5006.1	6396.1
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.36	3.01
Moisture content (%)	29.4	53.6
Dry density of soil (g/cm3)	1.82	1.96
Dial gage reading of Height H1	44.99	
Dial gage reading of Height H2		46.45

SOIL + 50% CRUSHER DUST

Penetration (mm)	0	0.64	1.27	1.96	2.54	3.18	3.81	4.45	5.08
Dial RDG	0.0	10.0	16.0	22.0	25.0	27.5	28.9	30.0	31.0
Ring factor (KN/div)	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022	0.022
Load (KN)	0.00	0.22	0.35	0.48	0.55	0.61	0.64	0.66	0.68

Compaction Data		OMC	18.158%	MDD	1.735g/cm3	
Blow	Dry density (g/cc)	Load(KN)		CBR (%)		Swell (%)
		2.54	5.08	2.54	5.08	
56	1.77	0.55	0.68	4.167	3.410	1.195
MDD		**1.735g/cm3**				
CBR at MDD		**4.250%**				



Soaking condition	56 Blows	
	Before	After
Mold number	A1-3	
Weght of soil+Mold (gm)	11508.2	13523.1
Weght of Mold (gm)	6500	6500
Weight of soil (gm)	5008.2	7023.1
Volume of mold (cm3)	2123	2123
Wet density of soil (g/cm3)	2.36	3.31
Moisture content (%)	33.6	55.8
Dry density of soil (g/cm3)	1.77	2.12
Dial gage reading of Height H1	45.97	
Dial gage reading of Height H2		47.36