JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

HIGHWAY ENGINEERING STREAM

Evaluating The Effects of Non-Woven Geo-textiles and Geo-membranes Reinforcement on Performance Improvement and Drainage Barrier Efficiency on Expansive Subgrade Soil.

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

By

Abuye Boja

January 28, 2021

Jimma, Ethiopia

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DECLARATION

I hereby declare that this thesis is my original work and has not been presented for a degree in any other university.

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ABSTRACT

Subgrade is a very important components of pavement structures in which the ultimate wheel loads rests on it. The strength of subgrade materials could sometimes fall under doubt due to the presence of moisture problem, highly plastic soil and high clay mineral contents. This doubt leads to extreme deterioration and distress which in turns consumes much of budget for maintenance work. One mechanism of minimizing doubt of strength and/or performance and drainage problem is stabilization of weak expansive subgrade soil with geo-synthetic materials. These particular studies used non-woven geo-textiles and geo-membrane for strength and drainage problems respectively. Purposive sampling was used to get samples for laboratory tests which were about 230 Kg by weight. Generally, the study is designed to evaluate the effects of non-woven geo-textiles and geo-membranes reinforcement on performance improvement and drainage barrier efficiency on expansive subgrade soil.

Reinforcement was made at middle of the mould and at the middle and at one-third of the mould for single and double layer reinforcement respectively. The depth of reinforcements was selected by reviewing the previous investigations with which good effectiveness have shown. The study has examined Natural Moisture Content, Unconfined Compression Strength Test, Particle Size Distribution, Atterberg's Limit, Specific Gravity, Free Swell Index, Compaction and California Bearing Ratio tests for natural soil whereas Compaction and California Bearing Ratio test for reinforced soil by using ASTM manuals for all the tests. After analyzing laboratories results the soils under investigations were classified as A-7-6(21) as per AASHTO and CH as per USCS respectively which was fat clay soil with montmorillonite clay mineral contents, high swell potential and highly plastic clay soil in which its specific gravity and group index (GI) was 2.72 and 21 respectively showing that the soil is poor subgrade soil in its natural state. The mineral contents, swelling potential and plasticity of the soil were determined from the combined results of free swell ratio and Atterberg's limit test.

For single layer and double layer reinforcements OMC and MDD was slightly decreased and increased respectively for each layers. The CBR value of the natural soil was 2.38. After Reinforcement, single reinforcement has shown good performances with CBR value of 12.24%. The percentage of performance improvement in terms of CBR value for single reinforcement was 414.3%. Double layer reinforcement with expansive subgrade soil was showed CBR value of 6.82% and the percentage of performance improvement in terms of CBR value was 186.5%. The CBR swell of natural soil was 6.25 which is above minimum requirements as per ERA manual (2%).As a result of reinforcements CBR swell was reduced to 1.59% and 0.88% for single and double layer reinforcement respectively. The percentage of swelling decreases was about 74.56% and 85.92% for each layer respectively. The aim of controlling moisture migration within expansive subgrade soil was achieved with double reinforcement.

Keywords: **Expansive Subgrade soil, Performance Improvement, Geo-textiles, Geo-membranes and Drainage**

Table of Contents

DECLARATION i

ACKNOWLEDGEMENT ii

ABSTRACT iii

Lists of Tables vii

Lists of Figures viii

ACRONYMS ix

UNITS x

CHAPTER ONE 1

INTRODUCTION 1

1.1 Background 1

1.2. Statements of Problem 2

1.3. Research Questions 4

1.4. Objectives 4

1.4.1. General Objectives 4

1.4.2. Specific Objectives 4

1.5. Scope of the Study 4

1.6. Significance of the study 5

1.7. Justification of the Study 5

CHAPTER TWO 6

REVIEW OF RELATED LITERATURES 6

2.1. General Characteristics of Expansive Soils 6

2.2. Soil Stabilization Mechanism 7

2.2.1. Chemical Stabilization Mechanism 9

2.2.2. Mechanical Soil Stabilization 9

2.3. Effects of Geo-synthetic Materials on Soil Properties 10

CHAPTER THREE 16

MATERIALS AND RESEARCH METHODOLOGY 16

3.1. Study Area 16

3.2. Study Materials 17

3.2.1. Weak Expansive Subgrade Soil and its Locations 17

3.2.2. Geo-synthetic Material 17

3.3. Study Period 18

3.4. Study Design ... 18
3.4.1. Field Survey ... 19
3.4.2. Sample Size and Sampling Techniques ... 19
3.4.3. Sample Preparations ... 20
3.5. Laboratory Tests ... 21
3.5.1. Moisture Content ... 21
3.5.2. Field Density ... 21
3.5.3. Unconfined Compressive Strength Test ... 22
3.5.4. Particle Size Distribution ... 22
3.5.5. Atterberg Limits ... 23
3.5.6. Soil Classification ... 24
3.5.7. Specific Gravity ... 24
3.5.8. Free Swell Index (FSI) ... 25
3.5.9. Compaction Test ... 26
3.5.10. California Bearing Ratio (CBR) ... 27
3.5.11. CBR Swell of Soil ... 27
CHAPTER FOUR ... 29
RESULTS AND DISCUSSIONS ... 29
4.1 General Properties of Geo-synthetic Materials Used in The Study ... 29
4.2. Engineering Properties of Subgrade Expansive Soil in Study Area ... 32
4.2.1. Natural Moisture Content ... 33
4.2.2. Field Density Determination ... 33
4.2.3. Unconfined Compressive Strength Test ... 34
4.2.4. Sieve Analysis ... 35
4.2.5. Atterberg's Limit Test ... 36
4.2.6. Soil Classifications ... 37
4.2.7. Specific Gravity ... 37
4.2.8. Free Swell Index Test ... 38
4.2.9. Compaction Test for Natural and Stabilized Soil ... 39
4.2.10. California Bearing Ratio (CBR) ... 42
4.2.11. CBR Swell Value ... 47
4.3. Effects of Geo-synthetic Materials on Drainage Barrier Efficiency ... 49

4.4. Percentage of Performance Improvement of Subgrade Soil in terms of CBR 53
CHAPTER FIVE 55
CONCLUSIONS AND RECOMMENDATIONS 55
5.1. Conclusions 55
5.2. Recommendations 56
5.3. Future Research Ideas 57
REFERENCES 58
Appendix A: - Details of Natural Soil Laboratory Results 60
Appendix B: - Details of One layer Reinforced Soil Laboratory Results 86
Appendix C: - Details of Two layer Reinforced Soil Laboratory Results 98
Appendix D:-Photo Taken During Investigation 111

Lists of Tables

Table 2. 1 Types of Stabilization [9] ... 8

Table 3. 1 Coordinates Locations of each sample ... 20
Table 3. 2 Consistency and Unconfined Compression Strength of Clay [30] ... 22
Table 3. 3 Expansive Soil Classification based on Liquid Limit [31] ... 23
Table 3. 4 Expansive Soil Classification based on Plastic Index [31] ... 23
Table 3. 5 Expansive Soil Classification System of the basis of Atterberg's limt test [39] ... 24
Table 3. 6 Expansive soil classification based on shrinkage limit [31] ... 24
Table 3. 7 Specific Gravities of Some Soil [39] ... 24
Table 3. 8 Classification of Soils on Free Swell Ratio [31] ... 26

Table 4. 1 General Properties of Geo-textile used during Study as Determined Mattex Geo-synthetic [33] ... 29
Table 4. 2 Durability Properties Geo-textile used during Study as Determined Mattex Geo- synthetic[33] ... 30
Table 4. 3 General Properties of Geo-membrane used during investigation as determined by GIW [33] ... 31
Table 4. 4 Natural Moisture Content of the Soil ... 33
Table 4. 5 Field Density Determination by Sand Replacement Method ... 34
Table 4. 6 The Value of Unconfined Compressive Strength Test of Study Area ... 34
Table 4. 7 The Value of Atterberg's Limit Test ... 36
Table 4. 8 Linear Shrinkage and Shrinkage Product ... 37
Table 4. 9 Soil Classification on the basis of USCS and AASHTO ... 37
Table 4. 10 Specific Gravity Test Value ... 37
Table 4. 11 The Value of Free Swell Ratio ... 38
Table 4. 12 Proctor Compaction Tests for Natural Soil ... 39
Table 4. 13 Compaction of Geo-reinforced Soil ... 41
Table 4. 14 CBR Penetration for Natural Soil ... 44
Table 4. 15 CBR Value of One Layer Reinforced Soil ... 45
Table 4. 16 CBR Value of Two Layer Reinforced Soil ... 46
Table 4. 17 Swelling Values of the Soil under Study ... 48
Table 4. 18 Summary of Geotechnical Properties of Natural Soil ... 48
Table 4. 19 Moisture Variation of Single Layer Reinforced with Geo-membrane ... 50
Table 4. 20 Moisture Variation of Double Layer Reinforced with Geo-membrane ... 51
Table 4. 21 CBR Value Improvement in Percentage ... 53

Lists of Figures

Figure 2. 1 Soil subgrade moisture-proofing (relief of pore-water may be necessary) 13
Figure 2. 2 Control of Expansive Soils (Horizontal Infiltration) 14

Figure 3. 1 The maps of the Study Area [Sources: https://www.google.com/maps/jimma+ JUIT)] 16
Figure 3. 2 Photo of Geo-textile during purchasing from AB-HAM Enterprise [Captured by researcher on Nov 5, 2020] 17
Figure 3. 3 Photo of Geo-membrane during purchasing from GIW [Captured by researcher on Nov 5, 2020] 18
Figure 3. 4 General Method ology and Study Design of the Research [Drawn by the Researcher] 19

Figure 4. 1 Grain Size Distribution Curve 36
Figure 4. 2 Compaction of Natural Soil for Trial One 41
Figure 4. 3 Natural Soil Compaction for Trial Two 41
Figure 4. 4 Natural Soil Compaction for Trial Three 41
Figure 4. 5 Compaction of one Layer Reinforcement 42
Figure 4. 6 Compaction of Two Layer Reinforcement 42
Figure 4. 7 soil properties and Classification categories [9] 44
Figure 4. 8 Potential Use of Horizontal Geo-composite Drainage Layers [41] 50
Figure 4. 9 Moisture Migration within Expansive Subgrade Soil and Function of Geo-membrane [Drawn by Researcher] 53

ACRONYMS

AASHTO	American State Association State Highway and Transportation Organization
ASTM	American Society for Testing and Material
CBR	California Bearing Capacity
CCL_4	Carbon Tetra Chloride
C_u	Unconfined Shearing Strength
ERA	Ethiopian Road Authority
FSI	Free Swell Index
FSR	Free Swell Ratio
GIW	Geo-synthetic Industrial Work
H_2O	Water
HDPE	High Density Polyethylene
HP	High Performance
JIT	Jimma Institute of Technology
JUIT	Jimma University Institute of Technology
LL	Liquid Limit
MDD	Maximum Dry Density
MFSR	Modified Free Swell Ratio
MSL	Mechanically Stabilized Layer
NP	Needle Punched
OMC	Optimum Moisture Content
PEC	Polyethylene Chloride
PGI	Partial Group Index
pH	Power of Hydrogen
PI	Plastic Index
PLC	Private Limited Company
PL	Plastic Limit
PP	Polypropylene polymer
q_u	Unconfined Compression Strength
SL	Shrinkage Limit
SP	Shrinkage Product
TS	Tensile Strength
UCST	Unconfined Compression Strength Test
USCS	Unified Soil Classification System
UV	Ultraviolet

UNITS

%	Percentage
‘	Minute
µm	Micro meter
cm^3/cc	Cubic Centimeter
E	East
ft	Feet
g/ml	Gram per mililiter
G	Gram
hr	Hour
Kg	Kilogram
Kpa	Kilo Pascal
lp/ft^2	Pound per Squared Feet
ml	Mililiter
mm	milimeter
m	Meter
N	Newton
N	North
^{o}	Degree
^{o}F	Degree Fahrenheit
^{o}C	Degree Celsius

CHAPTER ONE

INTRODUCTION

1.1 Background

Weak subgrade soil is a common problem in road construction. Whether it is a temporary access road or a permanent road built over a weak subgrade, a large deformation of the subgrade can lead to deterioration of the paved or unpaved surface. The use of cementitious materials to treat/stabilize the poor subgrade is a conventionally accepted practice by many state highway agencies. However, geo-synthetics offer an environmental friendly and potentially economical alternative solution for reinforcing/stabilizing roads built over weak soil. Many experimental, numerical, and analytical studies have thus been performed to evaluate the benefits of using geo-synthetics in pavement application [1].

Construction of pavement on weak or soft soil is highly risky because such soil is susceptible to differential settlements, poor shear strength, and high compressibility. In developing countries like Ethiopia transportation facilities are very important for sustainable development. However, a better performance of the agricultural sector in particular as Ethiopia's economic growth is highly dependent on it, and the sustainable economic growth of the country at large would be achieved through an improvement of the basic infrastructure. Consequently, the road network has been identified as a serious bottleneck for the economic development of the country. An appreciable part of Ethiopia is covered by expansive soil. Most of the roads constructed and proposed as well as substantial amount of the newly planned railway routes in the country pass through in the heart of expansive soils. The roads on this type of soils fail before their expected design life, in some cases after few months of completion. It has been reported in 2004 that Addis Ababa City Roads Authority had annual expenditure of around 300 million Ethiopian Birr for road construction and maintenance out of which more than 30 million Ethiopian Birr was expended for routine maintenances which is too big and require special attention [2].

Subgrade soil is a natural soil layer assumed to receive the load of pavement materials placed over it. The loads on the pavement are ultimately received by the soil subgrade for ensuring the benefits of distributing loads to the earth. The performance of pavements depends to a large extent on the strength and stiffness of the sub-grades. Sub-grade soil bearing capacity plays very important role for the design and durability of highway structure. So it is very important to mitigate the strength of these materials. The use of geo-synthetics in roadways has involved multiple functions. While geotextiles have been used to provide separation, filtration, and lateral drainage, geo-grids have often been used when the primary function is reinforcement. The mechanisms by which geo-synthetics provide reinforcement include Lateral Restraint, Tensile Membrane Support and Increased Bearing Capacity [3].

The inclusion of a single layer and double layers of geo-synthetic reinforcements at varying depths in soil enhances the strength of the subgrade soil in terms of CBR value The CBR value of the soil increases by 5–60% when a single layer of reinforcement is placed within the subgrade soil and strength increases by 112–325% when it is reinforced with double layers of reinforcement. The amount of improvement depends upon the position of reinforcement layer and type of reinforcement. Placing the geo-synthetic reinforcement in the double layers yields the largest improvement regardless of the type of geo-synthetic. The optimum benefit of reinforcement is evident if it is placed at middle height of the CBR mould and for better improvement in strength the reinforcement layer should be placed between the upper one-third layer and middle layer [4].

The inclusion of geo-synthetic reinforcement in expansive subgrade soil can increase the strength subgrade soil which mitigates the life span of pavement by reducing deterioration and distress.
Moreover, geo-synthetic reinforcement reduces the budget for maintenance. The contribution of geo-synthetic material in pavement in such away inspired me to come up with the research so called evaluating effects of non-woven geo-textiles and geo-membranes reinforcement on performance improvements and drainage barrier efficiency on expansive subgrade soil.

1.2. Statements of Problem

Every year, expansive soils cause billions of dollars in damages. These problems are extensively occurring in Ethiopia. The aerial coverage of expansive soils in Ethiopia is estimated to be 24.7 million acres. It is widely spread in the central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Wolliso, Addis– Debrebirhan, Addis-Gohatsion, and Addis-Modjo. Also, areas like Mekele and Gambella are covered by expansive soils. Soil stabilization is the alteration of one or more soil parameters property by mechanical or chemical treatments, to create an improved soil material possessing the desired engineering properties. The expansive soils within Jimma Town contain a high Plasticity index and low CBR value; causing unstable subgrade soil which affects the upper pavement layers. In this study, the researchers established an approach to identify the chemical and physical properties of pumice and properties of expansive clay soil in Jimma, as well as strength determination of stabilized expansive soil with pumice and lime [1].

Expansive soil is one of the most abundant problematic soils in Ethiopia. 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 [5].

The current maintenance and rehabilitation practice also depends more on visual observation and functional evaluations such as surface roughness and visual survey at network level rather than detail pavement evaluation at project level. Major trunk and lower class universal rural access roads failed with in liability period where subgrade soil is black clay soil, but researches show that various treatments such as mechanical, lime and chemical stabilization has been implemented. Moreover, based on the pavement survey, the CBR criteria couldn’t result in reliable solution in case of pavement on expansive subgrade. Thus additional Stability Criteria should be adopted to resist the heaving condition [3].

Subgrade is a very important components of pavement structures in which the ultimate wheel loads rests on it. The strength of these subgrade materials could sometimes fall under doubt due to presence of moisture problem, highly plastic soil and high clay mineral contents within subgrade soil. As a result of weak subgrade soil; especially expansive soil pavement surface is exposed to extreme deterioration and distress which in turns consumes much of budget for maintenance work. One mechanism of minimizing doubt of strength and/or stiffness and drainage problem is stabilization of weak expansive subgrade soil with geo-synthetic materials. These particular studies used non-woven geo-textiles and geo-membrane for minimizing strength and drainage problems respectively.

1.3. Research Questions

The research questions that this study will go to explains; are as follows:

1. What are the general properties of non-woven geo-textiles and geo-membranes materials?
2. What are the engineering properties of expansive subgrade soil of study area?
3. What is the effect of using geo-membrane material on drainage barrier efficiency of expansive soil in pavement?
4. How much is performance improvement of stabilized expansive subgrade soil in terms of CBR?

1.4. Objectives

1.4.1. General Objectives

The main objective of this research is to evaluate the effects of non-woven geo-textiles and geo-membranes reinforcement on performance improvement and drainage barrier efficiency on expansive subgrade soil.

1.4.2. Specific Objectives

- ❖ To describe the general properties of non-woven geo-textiles and geo-membranes materials.
- ❖ To determine the engineering properties of expansive subgrade soil of study area.
- ❖ To determine the effects of using geo- membrane material on drainage efficiency of expansive soil in pavement.
- ❖ To determine the performance improvement of stabilized expansive subgrade soil in terms of CBR.

1.5. Scope of the Study

This particular study focused on the selected and representative expansive subgrade soil in Jimma town (i.e. Jimma Institute of Technology around new stadium under construction). The studies selected non-woven geo-textiles and geo-membranes materials to evaluate their effects on performance improvements and drainage barrier efficiency with expansive subgrade soil.

1.6. Significance of the study

This study evaluates the effects of geo-synthetic material on structural stability and drainage efficiency of expansive subgrade soil of Jimma town in pavement structure which can provide helpful information to various stake holders.

The study serves as a source of information and foundation for highway that can help to improve and control standard and specifications. Owners, contractors and consultants will benefit from the study as a source of information for pavement structure construction over problematic soil. The study will provide lessons that will helps the concerned body to come up with appropriate measures to address problems resulting from those types of soil. Other researchers will use the findings as a reference for further research by incorporating many more parameters.

1.7. Justification of the Study

The rationale for conducting this study was providing the bench marks under which the performance improvement and drainage performance of pavement structure over weak subgrade is improved. Facts showed that; in Jimma town there are expansive soils which can affect the pavement performance as a whole. In Jimma town the place called Kitto in Jimma University, Jimma Institute of Technology around new stadium under construction, was area with expansive soil [5], [6].

CHAPTER TWO

REVIEW OF RELATED LITERATURES

2.1. General Characteristics of Expansive Soils

Expansive soil refers to a soil that has the potential for swelling and shrinking due to changing moisture condition. Expansive soils cause more damage to structures particularly pavements and light buildings than any other natural hazard, including earthquakes and floods. It has been reported that the damage caused by these soils contribute significantly to the burden that the natural hazard pose on the economy of countries where the occurrence of these soils is significant [7]. Expansive Soil is a kind of high plastic clay. Because it has strong hydrophilic mineral composition, its engineering prosperities embodies that its shape contracts under dehydrating, Inflation and softening under the influence of water and the strength attenuates. This is very difficult to construct in the region of expansive soil. In the region of seasonal frozen, as capillary water rising height is larger; it is prone to phenomenon of frost boil or thawing settlement. It has important meaning to improve hydrophilic and physical and mechanical properties of expansive soil for Slope stability of embankment and cutting of highway engineering and reducing the cost of investment [8].

Soils which exhibit significant volume changes in the presence of water are termed as expansive soils. These soils exhibit behavior opposite to consolidation and compression. Expansive soils generally owe their expansive character to their constituent clay minerals, past and present loading history, and to their natural and imposed environments. Swelling may also be due to chemical processes acting on certain non-clay minerals which result in the formation of new minerals of lesser density. Typical damage to roads on expansive soils includes longitudinal unevenness and bumpiness, differential movement near culverts, and longitudinal cracking. The volume changes exhibited by expansive soils are related to the interactions of various intrinsic and external factors. The intrinsic factors are soil composition and thickness, dry density, soil fabric and moisture content while the external factors include climate and time. Laboratory related variables which influence the measurement of volume change are initial moisture content, initial dry density, soil fabric, surcharge load, solution characteristics, time allowed for swell, stress history (loading sequence), sample size and shape and temperature [9].

Desirable properties

The desirable properties of sub grade soil as a highway materials are:-

- Stability
- Incompressibility
- Permanency of strength
- Minimum changes in volume and stability under adverse conditions of weather and ground water
- Good drainage, and
- Ease of compaction [10]

2.2. Soil Stabilization Mechanism

Soil stabilization is a general term that involves the use of mechanical or chemical modifiers to enhance the strength of soils and reduce the change in moisture. The process is often called soil modification when the purpose is to change the physical properties and thereby improve the quality of the subgrade soil. Soil stabilization is usually performed for the following reasons:

- As a construction platform to dry very wet soils and facilitate compaction of the upper layer. For this case, the stabilized soil is usually not considered as a structural layer in the pavement design process.
- To strengthen a weak soil and restrict the volume change potential of a highly plastic (expansive) or compressible soil. For this case, the modified soil is usually given some structural value in the pavement design process [9].

The thickness, depth or zone of the subgrade that may be selected for soil improvement depends upon a number of factors. Among these are the anticipated traffic loads, the importance of the transportation network, constructability, the drainage characteristics, the geometric design, and the purpose of stabilization. When only a thin zone or short roadway length is subject to improvement, removal and replacement can usually be the preferred alternative, unless a suitable replacement soil is not economically available. The thickness of the subgrade to be treated is based primarily on the project economics and the objective of stabilization. Mechanical stabilization using thick gravel layers, in conjunction with geo-textiles or geo-grids is an effective technique for improving roadway support over soft, wet subgrades [9].

Table 2. 1 Types of Stabilization [9]

Stabilization Method and Materials Used		**Soil Type**	**Improvement**	**Remarks**
Mechanical	More Gravel	Silts and Clays	None	Reduce dynamic stress level
	Blending	Moderately Plastic	None	Too difficult to mix
		Others	Improved gradation, reduced plasticity, reduced breakage	
	Geo-synthetics	Silts and Clays	Strength gain through minimum disturbance and consolidation	Fast, plus provides long-term separation
Water Proofers	Asphalt	Plastic and Collapsible	Reduce change in moisture	Long-term moisture migration problem
	Geo-membranes	Plastic and Collapsible	Reduce change in moisture	Long-term moisture migration problem

Road and civil engineering projects demand high-performance and high-quality soils. Soil stabilization involves the alteration of one or more of the soil properties collectively using techniques of modification to enhance both the physical and mechanical properties of the soil for specific applications through controlled modification of soil texture, plasticity, structure and durability. The suitability of the soil for construction purposes is measured in terms of the size and distribution of its particles and as such is described as well-graded or poorly graded. The main purpose of undertaking the process is to prepare the land and build a strong foundation that can support the design loading. The Processes of compaction and stabilization increases soil strength and durability as well as suppress dust formation and prevent soil erosion. The advantages of soil stabilization include significant improvement of durability and strength and hence bearing capacity and eliminate the need for expensive surface treatments. Dust control, waterproofing and promotion of the use of waste geo-materials in construction are amongst many other reasons. However, the cost of raw materials can be increased with limitations due to location and expenses incurred due to the transportation of raw materials in cases where marginal materials cannot be utilized. The methods used to improve the engineering properties of soil are broadly classified into two categories [11].

- Chemical stabilization
- Mechanical stabilization

2.2.1. Chemical Stabilization Mechanism

Chemical stabilization alters the chemical properties of the soil through the use of admixtures. The limitations of chemical soil stabilization include disparity between simulated laboratory and field conditions could render in-situ application impossible, the risk of groundwater contamination is very high as a result of release of toxic compounds from some of the traditional agents as the leachate of toxic chemicals can affect the environment and human life in general, the balance between cost of chemical soil stabilizer and quantity required to achieve effective stabilization can be a challenge and in prevalent unsuitable conditions the effect of chemical stabilization can result in further detrimental conditions of the soil for instance in the soil-lime-sulphate reactions and stabilization induced cracking [11].

2.2.2. Mechanical Soil Stabilization

The objective of mechanical stabilization is to blend available soils so that, when properly compacted, they give the desired stability. In certain areas, for example, the natural soil at a selected location may have low load-bearing strength because of an excess of clay, silt, or fine sand. Within a reasonable distance, suitable granular materials may occur that may be blended with the existing soils to markedly improve the soil at a much lower cost in manpower and materials than is involved in applying imported surfacing [12].

Mechanical stabilization involves the use of physical processes. It is the modification of soil porosity and inter-particle friction or interlock for example by compaction. Unlike chemical stabilization, it changes only the physical properties of soil through compaction, soil blending (adding fibrous and non-biodegradable reinforcement) or placing a barrier on the soil.

The mechanism to mechanical stabilization involves addition of different grades of materials to achieve a dense packed material and addition of small amount of fine materials as binders for non-cohesive soils to enhance strength of the material. Sands and gravels with strong angularity impart internal friction and incompressibility to the mix, which renders stabilization with addition of a suitable binder loading. Mechanical stabilization also promotes use of locally available materials in a fit for purpose approach and utilization of mine tailings, coral, shell, clinker, slags and construction waste just to mention but a few [11].

2.3. Effects of Geo-synthetic Materials on Soil Properties

Geo-synthetic reinforcement layers are often used to improve the performance of pavement structures. The performance of an unpaved road is routinely measured in terms of the California bearing ratio (CBR), which is an index of strength of subgrade soil of unpaved road. In the present study, an experimental investigation was carried out to evaluate the performance of the subgrade soil by placing a single layer and double layers of geo-synthetic reinforcements (Glass-grid, Tenax 3D grid and Tenax multimat) horizontally at varying depths from the top surface of subgrade soil. Through a series of CBR tests in the laboratory, an attempt was made to determine the optimum depth of the reinforcement layer. The single layer of reinforcement has been placed at the middle, one-third and one-fourth of the height of the CBR specimen from the top surface of the soil in the CBR mould. The double layers of reinforcement were placed at one-fourth of the specimen height from the top surface and the bottom surface. The results show the significant contribution in terms of increased CBR value of the soil, resulting in reduced design thickness of the pavement layers above the subgrade soil. The reason for the improvement in the strength of the subgrade soil reinforced with single and double layers of reinforcement is that, the geo-synthetics used in the study has good interlocking and frictional capability which can provide tensile resistance to any lateral movements. Thus it improves the strength of the soil with low CBR. Another reason for the improvement in the strength of the subgrade with low CBR is that, through the inclusion of geo-synthetic reinforcement layers the maximum vertical stress on the subgrade is reduced. The vertical stress on the subgrade is more uniformly distributed than on the absence of a geo-synthetic. Therefore, reinforcement helps to improve the bearing capacity of the soil with low CBR. Also, the combining action of geo-synthetic tension and geo-synthetic improved load distribution results in vertical restraint of the subgrade. Generally, the improvement in the strength of the subgrade soil reinforced with single and double layers of reinforcement varies from 6%-324% [4].

Soil alone is strong enough in compression but comparatively weak in tension. Reinforcing soil is the technique where tensile elements are placed in the soil to improve stability and control deformation. The geotextiles are used as reinforcement; their prime role is to provide tensile strength to soil at strain level which is compatible with the performance of the soil structure. Textiles are used as reinforcement in the form of fibers, fabric form like woven, knitted, non-woven. Geo-synthetics are used as reinforcement in paved roads, in railway tracks, embankment of shallow weak soils, earth retaining walls, mining subsidence protection etc. Load on the soil produces expansion. Thus, under load at the interface between the soil and reinforcement (assuming no slippage occurs, i.e. there is sufficient shear strength at the soil/fabric interface). These two materials must experience the same extension, producing a tensile load in each of the reinforcing elements that in turn is redistributed in the soil as an internal confining stress. Thus the reinforcement acts to prevent lateral movement because of the lateral shear stress developed. Hence, there is an inbuilt additional lateral confining stress that prevents displacement. This method of reinforcing the soil can be extended to slopes and embankment stabilization. Strength created by the introduction of geotextile into the soil & developed primarily through the following three mechanisms-

1. Lateral restraint through interfacial friction between geotextile and soil/aggregate.
2. Forcing the potential bearing surface failure plane to develop at alternate higher shear strength surface.
3. Membrane type of support of the wheel load

The structural stability of the soil is greatly improved by the tensile strength of the geo-synthetic material. This concept is similar to that of reinforcing steel to the concrete. Since concrete is weak in strength & tension, reinforcing steel is used to strengthen it. Geotextile materials function in a similar manner as the reinforcing steel by providing strength that helps to hold the soil in place. Reinforcement provided by the geotextiles and geo-grids allow embankment & roads to be built over very weak soils & allows for steeper embankments to be built [13].

Geo-synthetic stabilization and reinforcement are mechanical processes. The geo-synthetic is placed on the subgrade or sub-base, under or within aggregate layers and works with the soils and aggregate to create a reinforced composite section. This is achieved through the separation, filtration & drainage, and confinement functions provided by the geo-synthetic. Soil chemistry is not usually involved. Geo-synthetics are delivered to the project site in ready to use rolls and can be installed quickly and easily. The material is deployed evenly over the subgrade simply by unrolling and then aggregate fill is placed, spread and compacted over top. Unlike chemical stabilization, specialized equipment is not required, a specialty contractor is not required (any earthwork contractor can install geo-synthetics utilized for roadway stabilization / reinforcement applications) and there is no curing time or extended waiting period before the stabilized area can be trafficked. Construction can continue immediately after installation. Commonly used roadway geo-synthetic stabilization is applicable regardless of soil type, soil mineralogy, presence or absence of sulfates, water content or pH. A geo-synthetic reinforced mechanical stabilization application is designed based on the in-situ soil strength. This can be accomplished via several standards of practice design methodologies, or even through a small test section conducted on site. In addition, there are no environmental concerns or safety concerns such as those associated with chemicals or airborne particles. Furthermore, geo-synthetics can be installed in all weather conditions, including wind or cold and in populated area and near active traffic zones. Because the geo-synthetic reinforced MSL is a composite section comprised of geo-synthetic and aggregate, it is not degraded or otherwise adversely affected by freeze-thaw cycles, as are lime and cement stabilization applications [14].

One of the functions geo-synthetics possess is that they can act as a means to prevent moisture from infiltrating into the pavement structure, and such waterproofing action may limit base and subgrade movement due to freeze-thaw action or expansive soils consequently, delaying the deterioration of the pavement structure. Moisture is frequently the main source of pavement damage and roughness. Asphalt-impregnated fabrics control infiltration of surface water into a pavement. Fabrics may remain intact after the asphalt overlay has cracked and provide a moisture barrier. The fabric must be saturated with sufficient asphalt to provide a continuous moisture barrier; insufficient tack will diminish this waterproofing effect. If a moisture barrier is justified fabrics and composites offer this added benefit but grids cannot [15].

Deterioration on asphalt pavements is affected mainly by traffic and moisture. The traffic on road pavements is responsible by the occurrence of cracks that promotes the pavement deterioration. The moisture decreases the driving comfort and reduces the asphalt-aggregate bond leading to the degradation of the asphalt materials. Additionally, moisture, and especially the moisture inside the asphalt layers, is more difficult to assess than the cracks (visible). Moreover, the diagnosis is difficulted by traffic and the nature of most common investigating techniques [16].

Many parts of the world contain expansive, fine-grained soils that swell to alarming proportions when water is absorbed. To eliminate moisture from moving downward in the roadway cross section, a geo-membrane has been used as shown in Figure below. A geotextile is necessary (as a cushion) above and, depending on the quality of the subgrade, perhaps below the geo-membrane. Moisture entering expansive soils beneath pavements can occur horizontally as well as vertically. To seal off the potentially affected zone, vertical barriers can be deployed as shown in Figure below [17].

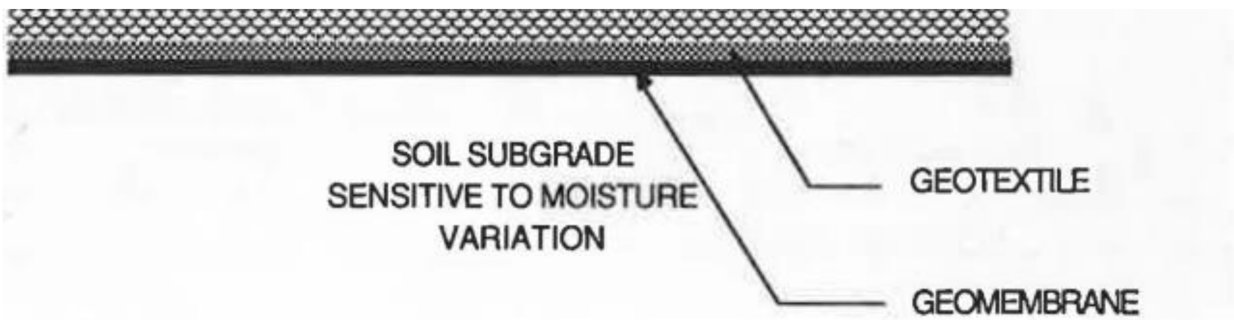

Figure 2. 1 Soil subgrade moisture-proofing (relief of pore-water may be necessary)

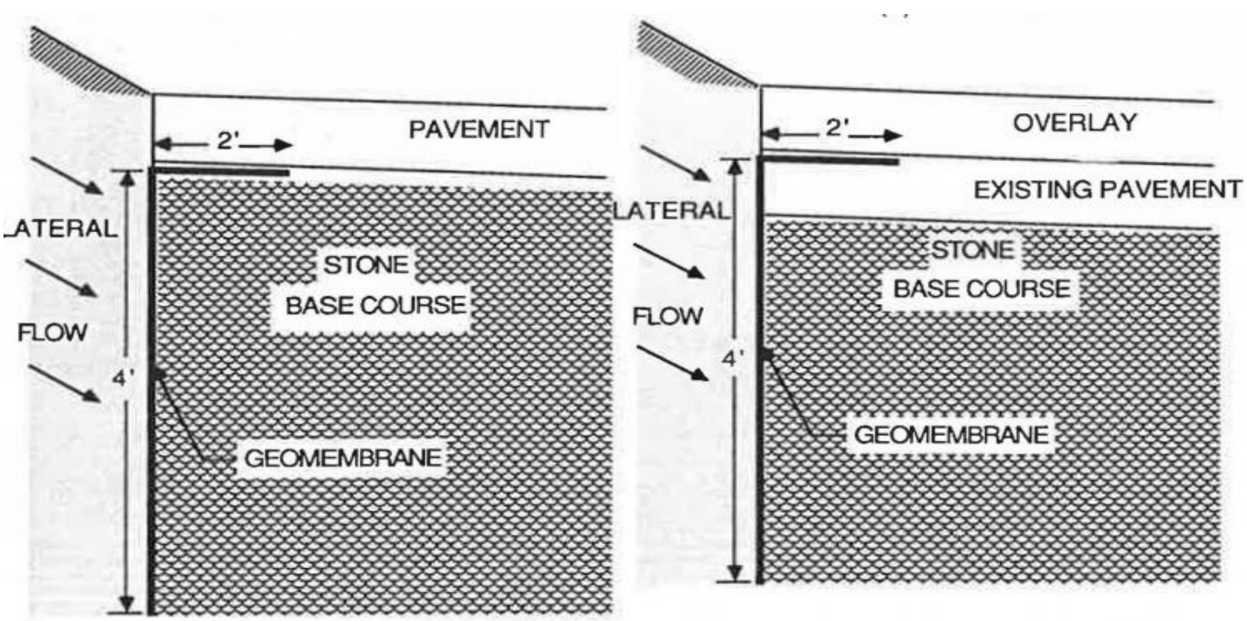

Figure 2. 2 Control of Expansive Soils (Horizontal Infiltration)

The behavior of the road surface depends upon the strength of the fill material and subgrade below it. Road construction on soft subgrade soil is a major issue affecting cost and scheduling of highway road projects. The strength of subgrade is most often expressed in terms of California Bearing Ratio (CBR) which is the ratio of test load to standard load at specified penetration by a standard plunger. In India the design of flexible pavement is primarily on basis of subgrade CBR. Many techniques have been evolved to strengthen the highway soil subgrade. Among them stabilization techniques are most common. In the Present study stabilization of soft subgrade soil is achieved by using geotextiles. The objective of this study is to find out an increase in strength mobilization in terms of CBR values, by conducting CBR tests on the subgrade soil when reinforced with geotextiles placed at different positions. Silty expansive clay soil and three varieties of geotextile (HP-370, PEC, and TS-50) are used in the study. CBR tests are conducted with the geotextile placed at different depths from top surface of soil in the CBR mould. The reinforced soil is a composite material of soil and reinforcement (which improves the resistance of soil by increasing its friction in the direction of greatest stress). By means of friction, soil transfer forces to the reinforcement built in the soil mass. The reinforcement thus develops tension when the soil mass is subjected to shear stress along the reinforcement. In the present work the strength improvement of expansive soils, which are used as subgrade for roads were studied by introducing the geo-synthetic materials. Geotextile materials (woven & nonwoven) such as HP-370, PEC and TS-50 (Ten-cate Geo-synthetics, 2015) were used to reinforce the expansive soils to test the CBR value. This geotextile reinforcement was placed at different positions and in different layers on the soil specimen. The CBR values of the un-stabilized and stabilized soils were tabulated and as well as depicted in graphs for comparison purpose [18].

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1. Study Area

The study was conducted at Jimma town, southwestern Ethiopia which is located at about 352 km by road southwest of Addis Ababa. Its geographical coordinates are between 7° 13'- 8° 56N latitude and 35°49'-38°38'E longitude with an estimated area of 19,506.24. The town is found in an area of average altitude, of about 5400 ft. (1780 m) above sea level. It lies in the climatic zone locally known as Woyna Dega which is considered ideal for agriculture as well as human settlement.

Figure 3. 1 The maps of the Study Area [Sources: https://www.google.com/maps/jimma+ JUIT)]

3.2. Study Materials

3.2.1. Weak Expansive Subgrade Soil and its Locations

The weak expansive subgrade soil for this particular study was collected from Jimma town which is locally called Kitto in JIT compound around new stadium under construction. The Soil was blackish brown in colour which was highly plastic clay soil. The expansiveness and plasticity of the soil was known by conducting free swell ratio and Atterberg's limit tests refer section 4.2.5 and 4.2.8. The sampling was done as per ERA manual at depth below 1.5m to avoid involvements of organic soil.

3.2.2. Geo-synthetic Material

Geo-textile and Geo-membrane were purchased from AB-HAM enterprise PLC in Addis Abeba and from GIW in Akeki respectively.

Figure 3. 2 Photo of Geo-textile during purchasing from AB-HAM Enterprise [Captured by researcher on Nov 5, 2020]

Figure 3. 3 Photo of Geo-membrane during purchasing from GIW [Captured by researcher on Nov 5, 2020]

3.3. Study Period

This research study was carried out within the prescribed time frame as per attached work schedule/plan from **July to January 2020.**

3.4. Study Design

This section provides details about the study design that was followed during the progress of the study which include both experimental and analytical method. The effectiveness of geo-synthetic materials on performance improvement and drainage barrier efficiency of expansive subgrade soil was shown by embedding geo-synthetic materials at various depths in expansive soils. In such a manner the effects of geo-synthetic reinforcement was examined.

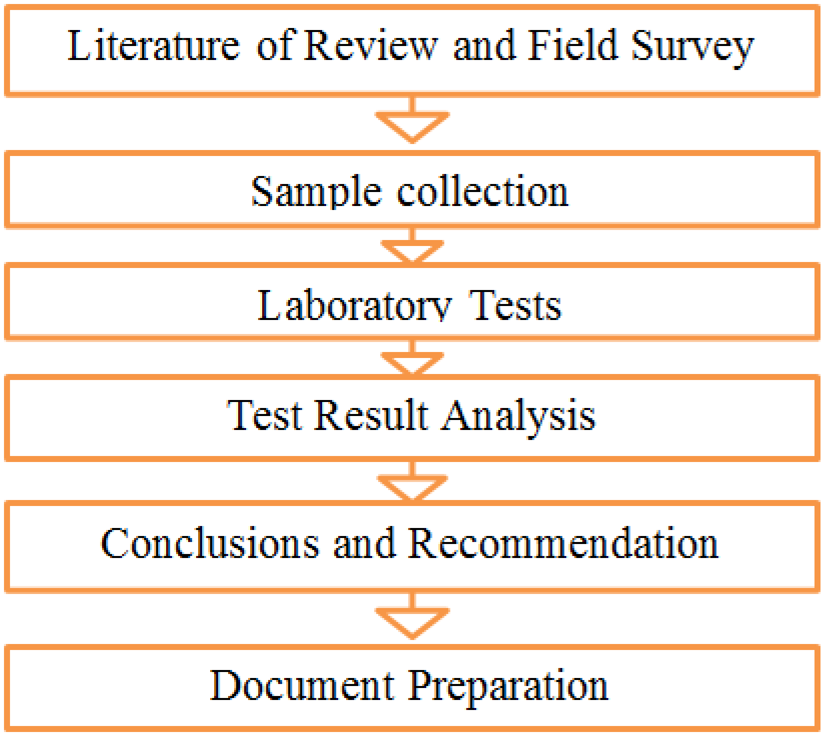

Figure 3. 4 General Methodology and Study Design of the Research [Drawn by the Researcher]

3.4.1. Field Survey

The survey was used to establish representative sample locations. Representative sample was taken from JIT compound.

3.4.2. Sample Size and Sampling Techniques

Final Representative Sample locations were selected using:-

- ❖ Visual inspections
- ❖ Applying experience
- ❖ Engineering judgments

For the purpose of sampling, pits should be dug to at least 50cm below the expected sub-grade level. In the case of a new alignment, the depth from the natural ground surface should be not less than 1.5m unless a rock stratum is encountered. At least one sample should be taken per test pit. Six (6) representative sample locations were selected for field and laboratory tests from Jimma town, Jimma Institute of Technology around new stadium under construction in JIT compound. About 230 Kg of expansive subgrade soil was brought from the site to JIT Geotechnical Laboratories and Jimma University Colleges of Agriculture and Veterinary Medicine soil laboratory.

- Undisturbed samples for density and moisture content tests. Samples should be identified and sealed against moisture loss using either a sample tube sealing device or layers of molten wax as soon as the sample is recovered from the hole.
- Disturbed samples will be taken from the area for compaction, CBR and other laboratory tests.

The place of sampling corresponds to the following coordinates (i.e. in terms of latitude and longitude)

Table 3. 1 Coordinates Locations of each sample

Samples	Coordinates in Degrees	
	Latitude	**Longitude**
1	7.68955	36.81702
2	7.68176	36.82288
3	7.67967	36.83091

3.4.3. Sample Preparations

Once representative sample location was selected and the required amounts of samples were taken; the next steps should be sample preparations. Soil samples were prepared on the basis of method described in [19].

- Expose the soil sample as received from the field to the air at room temperature until dried thoroughly.
- Break up the aggregations thoroughly in the mortar with a rubber-covered pestle.
- Select a representative sample of the amount required to perform the desired tests by the method of quartering or by the use of a sampler.

- Then, geo-textiles and geo-membranes were prepared in circular shape with diameter slightly less than that of mould for the purposes of reinforcements.

3.5. Laboratory Tests

For the selected samples the following laboratories were performed:-

1. Natural Moisture Content Determination [20]
2. Field Density by Sand Replacement Method [21]
3. Unconfined Compressive Strength [22]
4. Grain Size Analysis [23]
   - Sieve Analysis and
   - Hydrometer Test
5. Atterberg's Limit Test [24]
6. Specific Gravity [25]
7. Free Swell Index Test [26]
8. Modified Compaction Test [27]
9. California Bearing Ratio [28]

Characteristics of the soils before and after being treated with geo-synthetic in the laboratory were investigated. Each observed characteristic are described below.

3.5.1. Moisture Content

The main purposes of this test is to determine the moisture contents of expansive subgrade soil material using oven-drying method so that the results of the test will be used together with other laboratory results for soil classification purposes.

3.5.2. Field Density

These test methods used for determination of the in-place density and unit weight of soil and rock by sand replacement method. The dry density is an important factor in determining the magnitude of volume change. The swell or swelling pressure of an expansive soil increases with increasing dry density for constant moisture content. The reason is that higher densities result in closer particle spacing, therefore causing greater particle interaction. This particle interaction, or more precisely, double-layer water interaction, results in higher osmotic repulsive forces and a greater volume change [14].

3.5.3. Unconfined Compressive Strength Test

The unconfined compression test is a special case of a tri-axial compression test in which the all-round pressure $\delta_3 = 0$. The tests are carried out only on saturated samples which can stand without any lateral support. The test is, therefore, applicable to cohesive soils only. The test is an un-drained test and is based on the assumption that there is no moisture loss during the test. The unconfined compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils. The vertical stress at any stage of loading is obtained by dividing the total vertical load by the cross-sectional area. The cross-sectional area of the sample increases with the increase in compression. The cross-sectional area A at any stage of loading of the sample may be computed on the basic assumption that the total volume of the sample remains the same [29].

Table 3. 2 Consistency and Unconfined Compression Strength of Clay [30]

Consistency	qu(KN/m^2)	qu(lb/ft^2)
Very Soft	0-25	0-500
Soft	25-50	500-1000
Medium	50-100	1000-2000
Stiff	100-200	2000-4000
Very Stiff	200-400	4000-8000
Hard	>400	>8000

3.5.4. Particle Size Distribution

A) Sieve Analysis

This test is performed to determine the distribution of the coarser, larger-sized particles by using mechanical or sieve analysis method. The distribution of particle sizes larger than 75 µm (retained on the No. 200 sieve) is determined by sieving.

B) Hydrometer Tests

Hydrometer test is used to determine the distribution of the finer particles smaller than 75 µm using sedimentation process.

Many researchers have proposed criteria based on percentage clay size fraction (i.e. < 0.002mm) or colloid content (i.e. content of particles of size less than 0.001mm) to predict the swelling of fine grained soils[31].

3.5.5. Atterberg Limits

The main uses of this test to determine the plasticity of the soils Atterberg limits testing. An Atterberg limits device was used to determine the liquid limit of each soil using the material passing through a 475μm (No. 40) sieve. The plastic limit of each soil was determined by using soil passing through a 475μm sieve and rolling 3-mm diameter threads of soil until they began to crack. The plasticity index was then computed for each soil based on the liquid and plastic limit obtained. The liquid limit and plasticity index were then used to classify each soil.

Table 3. 3 Expansive Soil Classification based on Liquid Limit [31]

Swelling Potential	Liquid Limit (%)		
	Chen(1965)	Snethan et al.(1977)	IS:1498 (1970)
Low	< 30	< 50	20-35
Medium/Marginal	30-40	50-60	35-50
High	40-60	> 60	50-70
Very High	>60	-	70-90

Table 3. 4 Expansive Soil Classification based on Plastic Index [31]

Swelling Potential	Plastic Index (%)		
	Holtz and Gibbs(1956)	Chen(1988)	IS:1498 (1970)
Low	< 18	0-15	< 12
Medium	15-28	10-35	12-23
High	25-41	20-55	23-32
Very High	>35	>35	> 32

Table 3. 5 Expansive Soil Classification System of the basis of Atterberg's limt test [39]

Liquid Limit	Plasticity Index	Potential Swell (%)	Potential Swell Classification
<50	<25	<0.5	Low
50-60	25-35	0.5-1.5	Marginal
>60	>35	>1.5	High

Table 3. 6 Expansive soil classification based on shrinkage limit [31]

Swell Potential	**Shrinkage Limit**
Low	>15
Medium	10-16
High	7-12
Very High	<11

3.5.6. Soil Classification

The soil in the study area was classified using the Unified Soil Classification System (USCS) and AASHTO. Using the particle size distribution and the Atterberg limits, the USCS designates a two letter symbol and a group name for each soil.

3.5.7. Specific Gravity

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

Table 3. 7 Specific Gravities of Some Soil [39]

Types of Soil	G_s
Quartz Sand	2.64-2.66
Silt	2.67-2.73
Clay	2.70-2.90
Chalk	2.60-2.75
Loess	2.65-2.73
Peat	1.30-1.90
Mine tailings	2.80-4.50

3.5.8. Free Swell Index (FSI)

FSI is one of the most commonly used simple tests to estimate the swelling potential of expansive clay soil together with Atterberg's limit test. The calculation free swell index is as follow:-

$$Free\ Swell\ Index = \frac{Final\ volume\ of\ soil\ in\ water - Final\ volume\ of\ soil\ in\ kerosene}{Final\ volume\ of\ soil\ in\ kerosene}$$

Or

$$Free\ Swell\ Index(FSI) = \frac{Vd\ (H2O) - Vk\ (CCL4)}{Vk(CCL4)}$$ ………………… Equation 3.1

Where

- ✓ V_d-equilibrium Sediment volume of 10g oven-dried soil passing a 425 µm sieve mixed thoroughly with distilled water
- ✓ V_k- equilibrium Sediment volume of 10g oven-dried soil passing a 425 µm sieve in kerosene or ccl_4

The method based on FSI has a shortcoming in that it gives negative free swell indices for kaolinite-rich soils. To counter this problem the researchers provided

1) Modified free swell index (MFSI) is proposed which is defined as the ratio of equilibrium sediment volume of 10g oven-dried soil in distilled water (i.e. V_d) to the dry weight of the soil.

   $$\text{Modified Free Swell Index (MFSI)} = \frac{Vd}{10}$$ ……………….Equation 3.2

2) Free Swell Ratio which is defined as the ratio of equilibrium Sediment volume of 10g oven-dried soil passing a 425 µm sieve in distilled water (Vd) to that in carbon tetra chloride or kerosene

   $$\text{Free Swell Ratio (FSR)} = \frac{Vd}{Vk}$$ ………………..Equation 3.3

FSR in addition to predicting the degree of soil expansivity more realistically gives additional information about the clay mineralogy of the soils in the absence of sophisticated instrumentation such as X-ray diffractometer, differential thermal analysis apparatus and electron microscope[31].

Table 3. 8 Classification of Soils on Free Swell Ratio [31]

Free Swell Ratio	Clay Type	Potential Swell Classification	Dominant Clay Mineral
≤1.0	Non-swelling	Negligible	Kaolinite
1.0-1.5	Mixture of Swelling and Non-swelling	Low	Kaolinite and Montmorillonite
1.5-2.0	Swelling	Moderate	Montmorillonite
2.0-4.0	Swelling	High	Montmorillonite
>4.0	Swelling	Very High	Montmorillonite

3.5.9. Compaction Test

Compaction is applied to soil which result in expulsion of air, increase in bulk density and resistance to penetration. Increase in compaction increase moisture content which increases stability of soils. Compaction improves strength and workability of soil. Compaction is affected by water content in the soil. Compaction of a soil improves the engineering properties, i.e. it increases the shear strength of the soil and hence, the bearing capacity. It increases the stiffness and thus, reduces future settlement, void ratio and permeability. At lower water content than the optimum the soil is rather stiff and has a lot of void spaces and hence, the dry density is low. On the other hand, at water content more than the optimum the additional water reduces the dry density as it occupies the space that might have been occupied by solid particles. The laboratory standard proctor and modified proctor tests were performed as per [32] and [27] respectively. The tests were performed on disturbed samples of soil particles passing sieve sizes 4.75mm or 19mm mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples were compacted in three or five layers at 25 blows per layer in accordance with the specified nominal compaction energy of standard or modified proctor test respectively. Dry density is determined based on the moisture content and the unit weight of compacted soil. The corresponding water content at which the maximum dry density occurs is termed as the optimum moisture content.

3.5.10. California Bearing Ratio (CBR)

This test method covers the determination of the CBR (California Bearing Ratio) of pavement sub-grade, sub-base, and base/course materials from laboratory compacted specimens. After allowing specimens to take on water by soaking, or other specified treatment such as curing, each specimen is subjected to penetration by a cylindrical rod. At the end of the soaking period the penetration test is carried out at a rate of 1.27mm/min and the force or load required to cause the penetration will be recorded with respect to the standard penetration depths at each 0.5mm penetration, including the load value at 2.54 mm and 5.08 mm until the total penetration is 12.7mm. The penetration resistance load is then plotted against the penetration depth and correction is made for the load-penetration curve (if any). Using the corrected value taken from the load-penetration curve for 2.54mm and 5.08mm penetration, the bearing ratio is calculated by dividing the corrected load by the corresponding standard load, multiplied by 100. Its value ranges from 0 (worst) to 100 (best). If the bearing ratio at 2.54 mm is greater than that of 5. 08 mm, the bearing ratio that should be reported for the soil is normally the one at 2.54 mm penetration. When the ratio at 5.08 mm penetration is greater, the test is entirely repeated on a fresh specimen. If the repeated result of 5.08 mm is again greater, the design bearing ratio will be that of 5.08 mm or else, if the bearing ratio of 2.54 mm is greater the design bearing ratio will be that of 2.54 mm penetration. Results of stress (load) versus penetration depth are plotted to determine the CBR for each specimen. The CBR at the specified density is determined from a graph of CBR versus dry unit weight.

$$\text{CBR}(\%) = \frac{\text{Material Resistance on the Piston Pressure for 2.54 mm or 5.08 mm}}{\text{Corresponding Standard Pressure for well graded Crushed Stone}} * 100$$

Or

$$\text{CBR}(\%) = \frac{\text{Test Load for 2.54 mm or 5.08 mm}}{\text{Corresponding Standard Load}} * 100$$ ………………………….. Equation 3.4

3.5.11. CBR Swell of Soil

Take initial measurements for swell and allow the specimen to soak for 96 h, maintain a constant water level during this period. A shorter immersion period is permissible for fine grained soils or granular soils that take up moisture readily, if tests show that the shorter period does not affect the results. At the end of 96 h, take final swell measurements and calculate the swell as a percentage of the initial height of the specimen.

$$CBR\ Swell = \frac{Change\ in\ Length\ (mm) During\ Soaking}{116.4}$$

$$CBR\ Swell = \frac{Final\ Height - Initial\ Height}{116.4}$$ ……………………… Equation 3.5

CHAPTER FOUR

RESULTS AND DISCUSSIONS

This section presents and discusses the laboratory results which are very important to know the effect of Geo-synthetic reinforcement in subgrade soil concerning Performance improvement and drainage problems by comparing reinforced and unreinforced expansive subgrade soil samples.

4.1 General Properties of Geo-synthetic Materials Used in The Study

A) Non-Woven Geo-textiles

Mattex Geotextiles are manufactured from 100% virgin polypropylene (PP) polymer and are UV stabilized. These products can be successfully used in transportation, environmental, hydraulic and civil engineering applications and perform major functions including separation, filtration, drainage, reinforcement and membrane protection. Nonwoven geotextiles have the highest coefficient of friction against soil or stone, and have the ability to locally elongate to relieve stress concentrations to avoid construction damage and the nonwovens conform well to irregular ground surfaces. Nonwovens have higher permittivity and water flow, yet have a tighter pore size to be a finer filter [33] [34] .

The general properties of geo-synthetic materials are Mechanical property, Hydraulic property, Physical property and Durability. Generally, for this particular study the researcher used non-woven geo-textile of NP12. The detail properties the materials used are described in detail as follows:

Table 4. 1 General Properties of Geo-textile used during Study as Determined Mattex Geo-synthetic [33]

General Properties		Test	Units	NP12
nic al Pr	Grab Tensile Strength-MD	ASTM D-4632	N	365

	Grab Tensile Strength-XD	ASTM D-4632	N	365
	Elongation-MD	ASTM D-4632	%	65
	Elongation-XD	ASTM D-4632	%	75
	CBR Puncture Strength	ASTM D-6241	N	1100
Durability	UV Protection	ASTM D 4355	Retained Strength of ≥ 70% after 500 hours UV exposure	
	Trapezoid Tear Strength-MD	ASTM D-4533	N	160
	Trapezoid Tear Strength-CD	ASTM D-4533	N	160
	Index Puncture Strength	ASTM D-4833	N	250
Hydraulic Properties	Permeability	ASTM D-4491	$m/s.10^{-3}$	110
	Water flow to the Plane	ASTM D-4491	l/m2.s	110
	Permittivity at Water Temp 20°C	ASTM D-4491	S^{-1}	2.2
	Apparent Opening Size(AOS)	ASTM D-4751	Microns	175
Physical Properties	Thickness Under 2kpa	ASTM D-5199	mm	1.4
	Mass per Area	ASTM D-5261	g/m^2	100
	Roll Width		cm	590
	Roll Length		m	100
	40' Container Load (±10%)		m^2	65490
	Full Track Load (±10%)		m^2	59000
	Roll Diameter (±10%)		cm	33

Table 4. 2 Durability Properties Geo-textile used during Study as Determined Mattex Geo-synthetic[33]

			To be covered within 1 month after installation. Predicted to be durable for more than 25 years in natural soils with 4<ph<9 and soil temperature <25^{o}c
	Oxidation Resistance	EN ISO 13438	Retained Strength >90% at (110±1)^{o}C after 14 days exposure
	Chemical Resistance	EN 14030	Excellent
	Microbiological Resistance	EN 12225	Excellent

B) Geo-membranes

As mentioned in section 3.2.2 the geo-membrane material for the study was obtained from Geo-synthetic Industrial Work Private Limited Company, which was established in 2004 with the latest technology to manufacture products of geo-synthetics. GIW PLC offers high density Polyethylene (HDPE) available in smooth with various thicknesses varying 0.3mm-2mm, 6.5m wide. Geo-synthetic products are manufactured using first quality, high molecular weight resins, designed specifically for containment in hydraulic structures. Geo-synthetics HDPE Geo-liner provides excellent yield strength and seam strength and is ideal for applications requiring high chemical resistance, with low permeability and high ultraviolet resistance. This factory provides HDPE Geo-membrane products available in black color. The products are manufactured by the technology of co extrusion-blown triple layer with excellent physical and mechanical performance, high tearing resistance, good deformation adaptability, high puncture resistance, high aging resistance, high UV resistance, anti-acid and alkali, excellent low high temperature resistance, innocuous, long life span, perfect water proof performance, seepage and humidity resistance. The thickness of geo-membrane used for this particular study is 0.5mm.

Table 4. 3 General Properties of Geo-membrane used during investigation as determined by GIW [33]

Property	Test Method	Frequency	HDS 05
Thickness(nominal) (m)			0.5
Thickness(mm)	D5199	Per roll	0.5

Tensile Properties	D6693 Type IV	6000 Kg	
Yield Strength(KN/m)	(50mm/min)		7
Break Strength(KN/m)			13
Yield Elongation (%)	(33mm gage)		13
Break Elongation (%)	(50mm gage)		700
Tear Resistance(N)	D1004	12000 Kg	63
Puncture Resistance(N)	D4833	12000 Kg	160
Carbon Black Content (Range) (%)	D1603/D4218	6000 Kg	3-Feb
Carbon Black Dispersion	D5596	12000 Kg	Note 2
Density(g/cm^3)	D1505/D792	Resin Batch	0.94
Stress Crack Resistance(mm)(hr)	D5397	Resin Batch	300
Dimensional Stability (%)	D1204	Resin Batch	±1
Oxidative Induction Time(minutes)	D3895	Resin Batch	> 100
Note 2= Carbon Black Dispersion for 10 different views :all in categories 1 or 2			

Membrane encapsulation creates/installs a membrane to prevent migration of water. The approach has been primarily used to control volume change in expansive soils or soils susceptible to frost heaving. Essentially, this provides a means to maintain consistent water content, thereby reducing the potential for volume change. Shrink/Swell in soil is a result of change in moisture levels. As long as the moisture is consistently maintained, a soil will neither shrink, nor swell. Geo-synthetic membranes may be used for this purposes[35].

4.2. Engineering Properties of Subgrade Expansive Soil in Study Area.

The general summaries of properties of the natural soil before applying Geo-synthetic reinforcement are shown in Table 4.25. The soil is blackish brown in color. The particle size distribution curve indicates that about 98.05% of the soil is passing through No. 200 sieve; it possessed a liquid limit of 67.04%, a plastic limit of 31.76% and plasticity index of 35.28% with high swelling potentials. Therefore, those values indicate that the soil is highly plastic clay. Accordingly the soil falls under the CH and A-7-6(21) on the basis of USCS and AASHTO soil classification system respectively. The soil has optimum moisture content of 25.5% and a maximum dry density of 1.484 g/cm3, soaked CBR value of 2.38%. The CBR swell value of natural soil is 6.25%. Generally, this Soil possesses poor engineering property and therefore cannot used as a sub-grade material in its natural state unless some modification has been done.

Each properties of natural soil is described one by one as follow

4.2.1. Natural Moisture Content

The natural water content also called the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage.

Table 4. 4 Natural Moisture Content of the Soil

Natural Moisture Content of JiT Test Pit	**Number of Trial**			
Moisture content determination	**1**	**2**	**3**	**4**
Container No	G-53	1A	A	D
Mass of wet soil + container (A)	66.80	76.1	77.9	99.7
Mass of dry soil + container (B)	52.20	58.8	59.7	78.8
Mass of container (C)	23.70	27.1	19.4	34.5
Mass of moisture = A-B = (D)	**14.60**	**17.30**	**18.20**	**20.90**
Mass of dry soil = B-C =(E)	**28.50**	**31.70**	**40.30**	**44.30**
Natural Moisture content (%) = D/E*100 =(F)	**51.23**	**54.57**	**45.16**	**47.18**
Average Moisture Content	**49.54**			

If the natural moisture contents of a given soil is near the plastic limit or less are more likely to swell [30].

4.2.2. Field Density Determination

These test methods cover the determination of the in-place density and unit weight of soil and rock using a pouring device and calibrated sand to determine the volume of a test pit. The dry density is an important factor in determining the magnitude of volume change. The swell or swelling pressure of an expansive soil increases with increasing dry density for constant moisture content [14].

Table 4. 5 Field Density Determination by Sand Replacement Method

Number of Trials	1	2	3
Mass of wet soil (g) = W_w	4101.1	3950	3839.3
Mass of sand + jar before pouring (g) = W1	7880	7296	8360
Mass of sand + jar after pouring (g) = W2	1780	1600	2348
Mass sand in cone (g) = W3	2022.7	1898.5	2572.5
Mass of sand in hole (g) = w1-w2-w3	4077.3	3797.5	3439.5
Density of Sand (g/cm3)	1.485	1.535	1.467
Volume of Hole (cm3)	2745.66	2473.94	2344.58
Bulk Density of Soil (g/cm3)	1.49	1.60	1.64
Natural Moisture Content (%)	46.63	46.08	45.87
Field Dry Density of Soil (g/cm3)	1.02	1.09	1.12
Average Field Density	1.08		

4.2.3. Unconfined Compressive Strength Test

The tests are carried out only on saturated samples which can stand without any lateral support and therefore, applicable to cohesive soils only. The results of the test are used to know the consistency and shearing strength of the soil under investigation.

Table 4. 6 The Value of Unconfined Compressive Strength Test of Study Area

Trail for pit 1	1	2	3	Average
Height of sample (mm)	80.0	74.3	72.7	75.7
Diameter of sample (mm)	37.0	37.0	38.0	37.3
Mass of specimen, (g)	154.6	150.6	137.8	147.7
Unit weight (g/cm^3)	1.8	1.9	1.7	1.8
Moisture (%)	42.0	41.5	43.0	42.2
Dry Unit weight (g/cm^3)	1.3	1.3	1.2	1.3
Unconfined compressive strength (qu) (Kpa)	30.6	21.3	20.3	24.1
Shearing Strength (c_u) (Kpa)	15.3	10.6	10.2	12.0

The average values unconfined compressive strength of the soil of JIT is 24.1 kpa. If the value of unconfined compressive strength lies in the range from 0 - 25 kpa, then the soil is categorized as very soft concerning consistency refer section3.5.3 [30].

4.2.4. Sieve Analysis

Percent passing #. 200 (75μm) is greater than 35% as shown in Figure 4.1. Hence, the soil is categorized as fine grained soil (Silt-Clay material) according to[36]. The percent passing of each test is not only used to categorize soil as coarse-grained and fine grained but also it helps to determine the soil class together with the Atterberg limits. The group index is a function of the liquid limit, the plasticity index and the amount of material passing the 0.075mm sieve. Under average conditions of good drainage and thorough compaction, the supporting value of a material may be assumed as an inverse ratio to its group index, i.e. a group index of 0 indicates a "good" subgrade material and a group index of 20 or more indicates a poor subgrade material [37]. The detail procedure is as follow:-

Percent passing #. 200 (75μm) is 98.05% which is greater than the minimum requirement (36%).The Liquid Limit and Plastic Index of the soil 67.04% and 35.28% respectively which is again greater than the minimum value 41% and 11% respectively. Hence, the soil is classified as clayey soils. To determine the group index of the soil A-7-5 and A-7-6, we use Partial Group Index (PGI)

$$Partial\ Group\ Index(PGI) = 0.01(Percent\ Passing\ Sieve\ \#.200 - 15)(Plastic\ Index - 10)$$ ….Equation 4.1

$$Partial\ Group\ Index(PGI) = 0.01(98.05 - 15)(35.28 - 10) = 20.995 \approx 21$$

Therefore, the soil is Classified as **A-7-6 (21) indicating poor subgrade material as** per AASHTO Soil Classification.

Percent passing #. 200 (75μm) is 98.05% which is greater than the minimum requirement (50%).As a result the soil is classified as fine –grained soil. The liquid Limit of the soil is 67.04% which is greater than 50%. If the liquid limit is ≥ 50% whether a soil is Clay(C), Silt (M) or Organic depending on whether the soil coordinates plot above or below the A-line.

$$A - Line\ Equation, PI = 0.73(LL - 20) = .73 * (67.04 - 20) = 34.34$$ ….Equation 4.2

This is above A-line. Therefore, the Soil is classified as **CH (Fat Clay)** as per USCS soil classification.

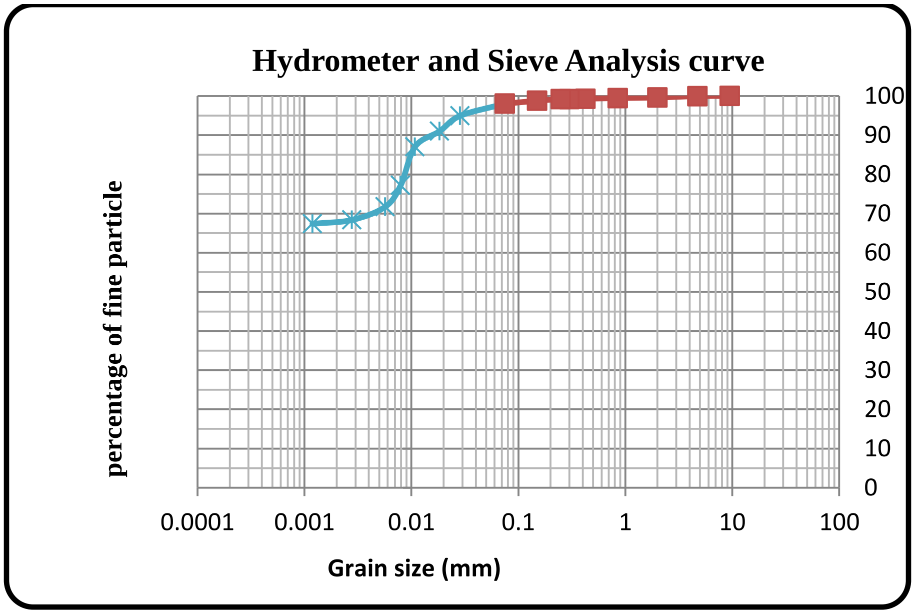

Figure 4. 1 Grain Size Distribution Curve

From grain size distribution the percentage of clay particles can be known (i.e. the particle <0.002) which is about more than 60% showing that the soil is fine grained soil of clay group [30].

4.2.5. Atterberg's Limit Test

Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils. Such tests are easy to perform and should be used as routine tests in the investigation of building sites in those areas having expansive soil. Among those soil properties; Atterberg's Limit Tests, Linear Shrinkage Tests can be mentioned as an example [38].

Table 4. 7 The Value of Atterberg's Limit Test

LL (%)	67.04	Percentage of Course Soil	0.02	Soil Classification (USCS)	Soil Classification (AASHTO)
PL (%)	31.76	Percentage of Sandy Soil	1.93		
PI (%)	35.28	Percentage of Fine Soil	98.05	CH	A-7-6 (21)

Table 4. 8 Linear Shrinkage and Shrinkage Product

LINEAR SHRINKAGE AND SHRINKAGE PRODUCT		
Trial 1	1	2
Initial Length L0	140.00	140.00
Oven dried Length LD	133.75	131.84
Linear shrinkage,Ls	**0.04**	**0.06**
Shrinkage Product, SP=Ls×%<425μm	**4.43**	**5.79**
Average Shrinkage Product, Sp	**5.11**	

The Liquid Limit, Plastic Index and Shrinkage limit of the soil under investigation is 67.04%, 35.28% and 5.11% respectively. Hence, based on the previous study the soil under investigation is classified as high swelling potential [31], [39] .

4.2.6. Soil Classifications

The soil is classified as **A-7-6 (21) indicating poor subgrade material as** per AASHTO Soil Classification. Therefore, the Soil is classified as **CH (Fat Clay)** as per USCS soil classification.

Table 4. 9 Soil Classification on the basis of USCS and AASHTO

LL(%)	PL(%)	PI(%)	**Soil Classification (USCS)**	**Soil Classification (AASHTO)**
67.04	**31.76**	**35.28**	**CH**	**A-7-6 (21)**

4.2.7. Specific Gravity

The specific gravity of a soil is used in calculating the phase relationships of soils (that is, the relative volumes of solids to water and air in a given volume of soil).

Table 4. 10 Specific Gravity Test Value

Trial No.	1	2	3
Mass of empty Pycnometer (g)	25.4	26.9	26.4
Mass of empty Pycnometer+dry soil (g)	49	50.8	50.4
Mass of empty Pycnometer+dry soil+water at temperature Tx℃ (g)	133.6	137.5	139.3

Test temperature(Tx), ℃	20	20	20
Density of water at Tx,g/ml	0.9982343	0.9982343	0.9982343
Mass of empty Pycnometer+water at temperature Ti^o c(20℃) (g)	118.6	122.4	124.2
Density of water at Ti,g/ml at20℃	0.9982343	0.9982343	0.9982343
Correction factor,k	1	1	1
Specific gravity G at Tx℃	2.74	2.72	2.70
Average specific gracity at Tx℃	2.72		

The obtained value of specific gravity indicates that the soil is clay soil [39].

4.2.8. Free Swell Index Test

This test has not yet been standardized by AASHTO and ASTM. The method was suggested by Holtz and Gibbs, (1956) to measure the expansive potential of cohesive soils. The free swell test gives a fair approximation of the degree of expansiveness of the soil sample. Finally, free swell ratio is calculated using equation 3.3 from section 3.5.8.

Table 4. 11 The Value of Free Swell Ratio

Trials	Initial Volume Vk (ml)	Final volume Vd (ml)	Free Swell Ratio
Trial 1	11.00	22.00	2.00
Trial 2	11.00	22.50	2.05
Trial 3	10.00	21.00	2.10
	Average		**2.05**

The observed laboratory value of FSR indicates that the soil is high swelling clay with montmorillonite mineral type [31].

4.2.9. Compaction Test for Natural and Stabilized Soil

Pavement performance also depends on subgrade density. Usually, the depth of influence for wheel loading varies between 1.5 and 3.0 m below the pavement surface. One of the factors which determine the depth of influence for wheel loading is the inherent characteristic of the subgrade and its natural density. If the density varies, then an adequate subgrade compaction is necessary for obtaining a high-quality travel surface. In-situ soils used as subgrades for the construction of road pavements are invariably compacted to improve their density. The purpose of compaction is generally to enhance the strength of a soil by increasing density. The evaluation of density reached as a result of the compactive efforts of compaction equipment is the most common quality-control measurement made at construction sites. Compaction also increases stiffness, decreases the sensitivity of the subgrade soil to changes in moisture content, minimizes long term settlement, and reduces the swelling potential of expansive soils [9].

Table 4. 12 Proctor Compaction Tests for Natural Soil

Number of Trials	Maximum Dry Density (g/cm^3)	Optimum Moisture Content (%)
1	1.49	28.5
2	1.48	24
3	1.484	24
Average Value	1.485	25.5

In order to know the effects of Geo-synthetic material (i.e. Geo-textile) on compaction; compaction of geo-reinforced soil both for single and two layers were done. The compaction test results were analyzed from the pure soil between the reinforcement layers without including the thickness of embedded reinforcement layers. The test results indicate that the density of reinforced clay increased with the number of geotextile layers without significant changes in the optimum moisture content (OMC) [19].

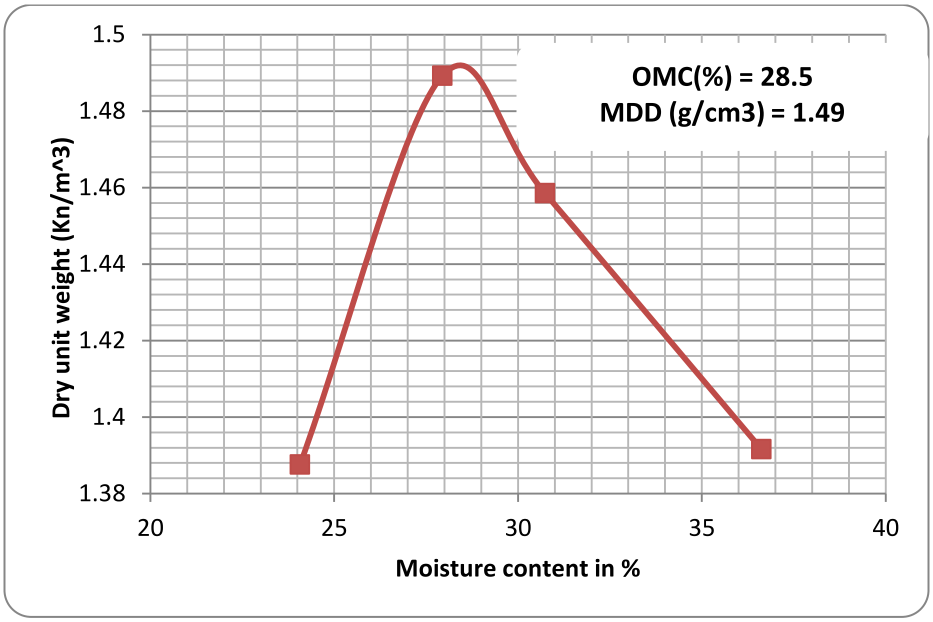
OMC(%) = 28.5
MDD (g/cm3) = 1.49
Dry unit weight (Kn/m^3)
1.5
1.48
1.46
1.44
1.42
1.4
1.38
20
25
30
35
40
Moisture content in %

Figure 4. 2 Compaction of Natural Soil for Trial One

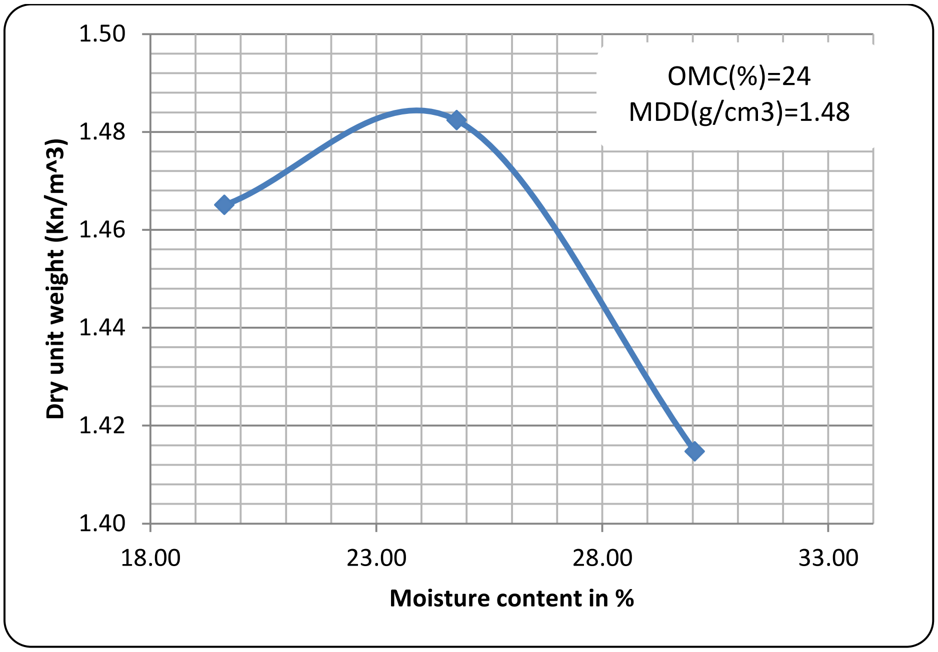

Figure 4. 3 Natural Soil Compaction for Trial Two

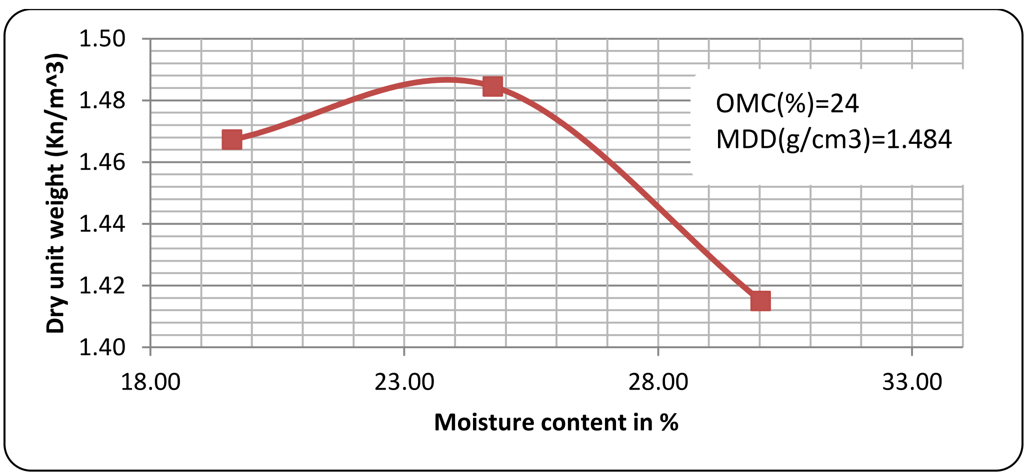

Figure 4. 4 Natural Soil Compaction for Trial Three

Table 4. 13 Compaction of Geo-reinforced Soil

Layers	Maximum Dry Density (g/cm^3)	Optimum Moisture Content (%)
One Layer Reinforcement	1.38	28.26
Two Layer Reinforcement	1.42	28.12

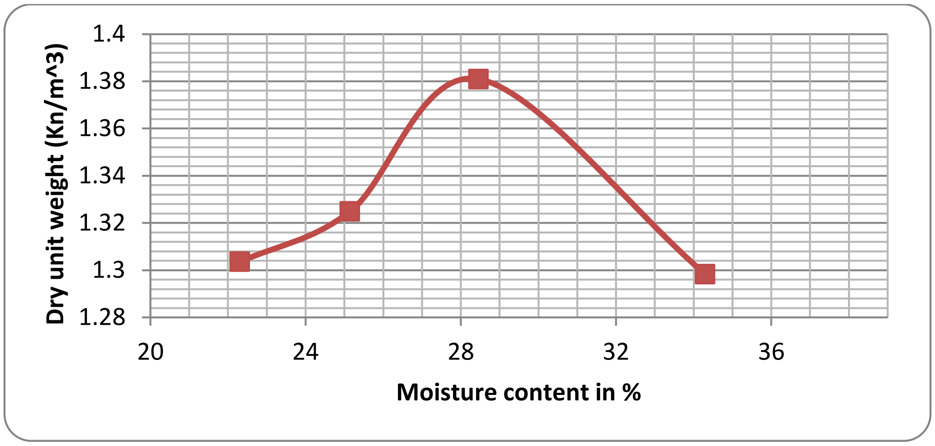

Figure 4. 5 Compaction of one Layer Reinforcement

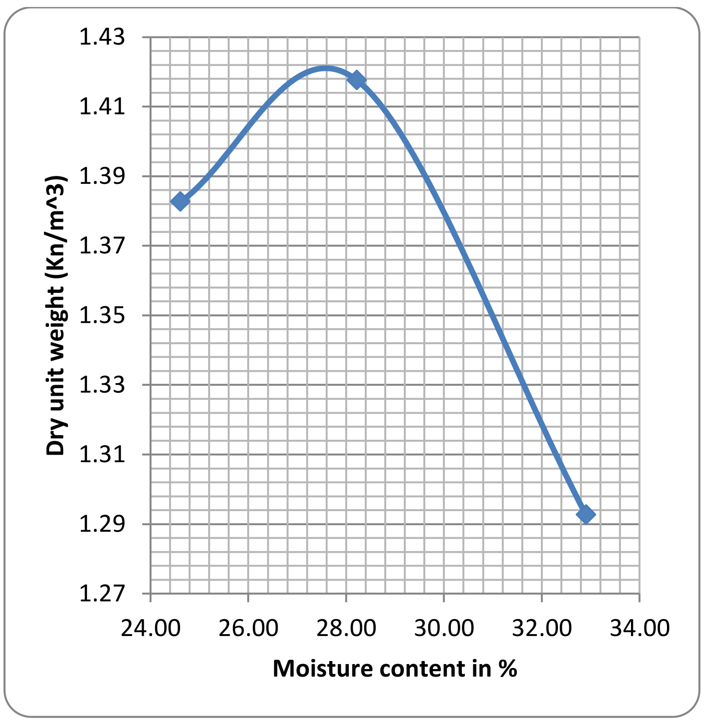

Figure 4. 6 Compaction of Two Layer Reinforcement

4.2.10. California Bearing Ratio (CBR)

A) **For Natural Soil**

The CBR tests were conducted to illustrate the effects of geo-textile on CBR value. Totally nine CBR trials (27 moulds) were performed three trials for natural soil whereas the remaining trials were for single and double layer reinforcement so that the average value will be taken to increase accuracy. The California Bearing Ratio (CBR) is an indirect measure of the strength of the subgrade. It is also the most widely used method for designing pavement structures. The higher the CBR value of a subgrade, the more strength it has to support the pavement. This means that a thinner pavement structure could be designed on a subgrade with higher CBR compared to a lower CBR value. However, it should be noted that although the CBR value is directly correlated with strength, the change in pavement thickness needed to carry a given traffic load is not directly proportional to the change in CBR value of the subgrade soil [9].

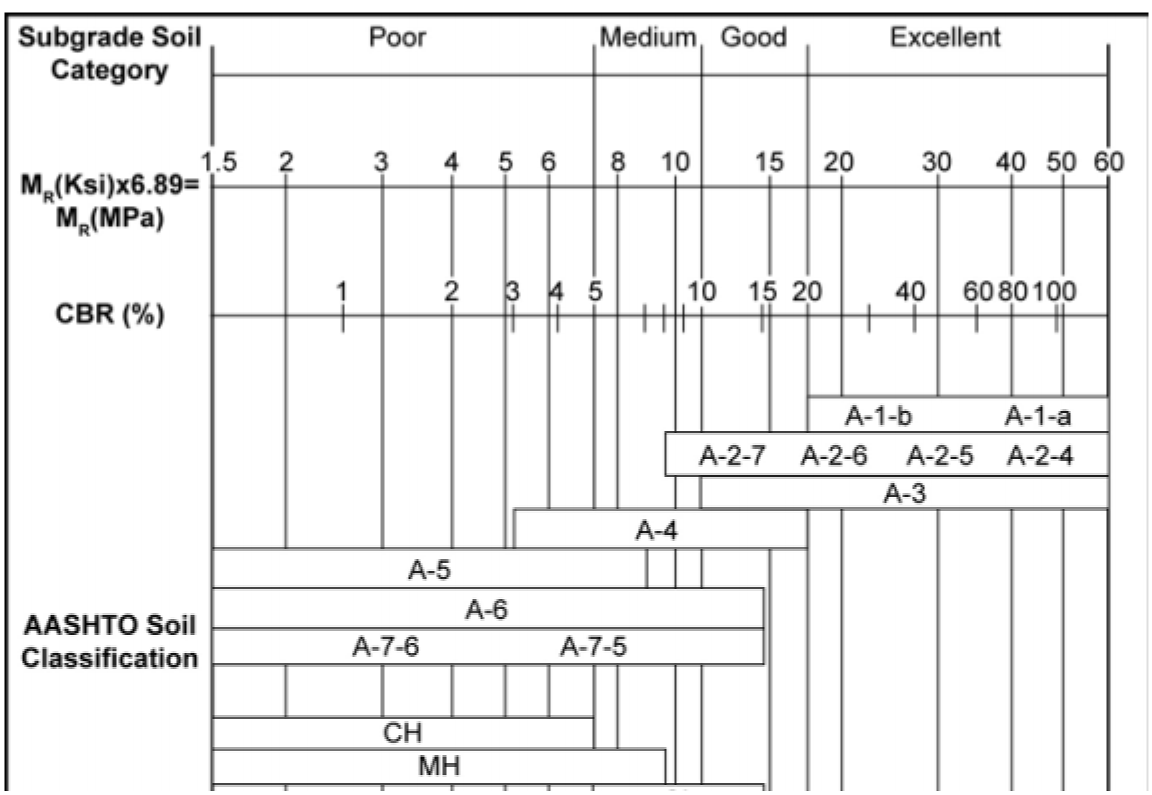

Figure 4. 7 soil properties and Classification categories [9]

By combining soil classification results from section 4.2.6 with the above table the soil under investigation fall under poor subgrade soil. Because the CBR ranges of soil is within specified interval (1 -5 %) [37].

Table 4. 14 CBR Penetration for Natural Soil

No of Trials	Blows	Penetration(mm)	Test Load (KN)	Standard Load (KN)	CBR Value (%)	CBR Value @ 95% MDD	Average CBR %
1	65	2.54	0.381	13.34	2.89	2.8	**2.38**
		5.08	0.564	20	2.82		
	30	2.54	0.216	13.34	1.64		
		5.08	0.32	20	1.6		
	10	2.54	0.119	13.34	0.90		
		5.08	0.178	20	0.89		
2	65	2.54	0.338	13.34	2.56	2	
		5.08	0.507	20	2.535		
	30	2.54	0.175	13.34	1.33		
		5.08	0.256	20	1.28		
	10	2.54	0.109	13.34	0.83		
		5.08	0.151	20	0.755		
3	65	2.54	0.326	13.34	2.47	2.35	
		5.08	0.485	20	2.425		
	30	2.54	0.139	13.34	1.05		
		5.08	0.195	20	0.975		
	10	2.54	0.034	13.34	0.26		

		5.08	0.043	20	0.215		

The average CBR value of natural soil is 2.38% which shows that the soil is poor for subgrade material [37].

B) For Reinforced Soil

The location of single and double layers of reinforcement within the subgrade were selected based on the literature because most of the researchers believe that maximum benefit of reinforcement is obtained when the reinforcement layer is placed in the upper half portion of the CBR mould. Half of the height of CBR mould was taken to place the reinforcement layers to get the maximum benefit of reinforcement. It is clear that there is a considerable amount of increase in the CBR value of a soil reinforced with different types of geo-synthetics at various depths. The geo-synthetic reinforcement was cut in the form of a circular disk of diameter slightly less than the diameter of CBR mould [4].

On the basis of the above literature for single layer of reinforcement position of reinforcement was at half height of CBR mould whereas for two layer reinforcements position of reinforcement was at half and one-third height of CBR mould.

Table 4. 15 CBR Value of One Layer Reinforced Soil

No of Trials	Blows	Penetration(mm)	Test Load (KN)	Standard Load (KN)	CBR Value (%)	CBR Value @ 95% MDD	Average CBR %
1	65	2.54	2.86	13.34	21.67	13.6	**12.24**
		5.08	3.41	20	17.05		
	30	2.54	0.836	13.34	6.33		
		5.08	1.232	20	6.16		
	10	2.54	0.191	13.34	1.45		
		5.08	0.276	20	1.38		
2	65	2.54	1.887	13.34	14.30	9.91	
		5.08	2.587	20	12.935		
	30	2.54	1.326	13.34	10.05		
		5.08	1.891	20	9.455		
	10	2.54	0.143	13.34	1.08		
		5.08	0.203	20	1.015		
3	65	2.54	1.972	13.34	14.94	13.21	

		5.08	2.885	20	14.425		
		2.54	1.425	13.34	10.80		
	30	5.08	2.127	20	10.635		
		2.54	0.354	13.34	2.68		
	10	5.08	0.396	20	1.98		

The average CBR value at 95% MDD for single layer reinforcement is about 12.24%. The results obtained on single layer reinforcement shows that maximum depth of reinforcement is at the middle of the mould from the top. Generally, for these particular studies stabilization of expansive subgrade soil with non-woven geo-textile enables to attain modified CBR value. Hence, percentage of performance improvement in terms of CBR is about 414.3% as shown below.

$$\%age\ of\ CBR\ Increases\ = \left(\frac{Modified\ CBR\ Value - Natural\ CBR\ Value}{Natural\ CBR\ Value}\right) * 100 \ \text{......Equation 4.3}$$

$$\%age\ of\ CBR\ Increases\ for\ Sl = \left(\frac{12.24 - 2.38}{2.38}\right) * 100 = 414.3\%$$

Table 4. 16 CBR Value of Two Layer Reinforced Soil

No of Trials	Blows	Penetration(mm)	Test Load (KN)	Standard Load (KN)	CBR Value (%)	CBR Value @ 95% MDD	Average CBR %
1	65	2.54	1.399	13.34	10.60	6.85	**6.82**
		5.08	2.06	20	10.3		
	30	2.54	0.667	13.34	5.05		
		5.08	0.95	20	4.75		
	10	2.54	0.14	13.34	1.06		
		5.08	0.206	20	1.03		
2	65	2.54	2.13	13.34	16.14	6	
		5.08	3.187	20	15.935		
	30	2.54	1.125	13.34	8.52		
		5.08	1.667	20	8.335		
	10	2.54	0.233	13.34	1.77		
		5.08	0.273	20	1.365		
3	65	2.54	1.37	13.34	10.38	7.62	
		5.08	1.785	20	8.925		
	30	2.54	1.145	13.34	8.67		
		5.08	1.688	20	8.44		

		2.54	0.131	13.34	0.99		
	10	5.08	0.152	20	0.76		

The average CBR value at 95% MDD for double layer reinforcement is about 6.82%. Hence, percentage of performance improvement in terms of CBR is about 186.5% using equation 4.3.

The attainment of maximum CBR value with single reinforcement brings the benefit saving expenses of purchasing Geo-textile and hence showed us no requirements of further reinforcements than single reinforcement. The percentage of effectiveness obtained from single layer reinforcement can be known simply by taking the differences of the two numbers (i.e. 414.3 and 186.5) which is 227.8%. Generally, the improvement in the strength of the subgrade soil reinforced with single and double layers of reinforcement varies from 6%-324% on the basis of Singh investigations.

4.2.11. CBR Swell Value

Expansive soils occurring in arid and semi-arid climate regions of the world cause serious problems on civil engineering structures. Such soils swell when given an access to water and shrink when they dry out. Several attempts are being made to control the swell-shrink behavior of these soils. The Swelling potential of the expansive soil mainly depends upon the properties of soil and environmental factors and Stress Conditions. Each year, expansive soils cause in damage to houses, other buildings, roads, pipelines, and other structures. This is more than twice the damage from floods, hurricanes, tornadoes, and earthquakes combined. Control the shrink-swell behavior through the following alternatives:

- ❖ Replace existing expansive soil with non-expansive soil.
- ❖ Maintain constant moisture content.
- ❖ Improve the expansive soils by stabilization [40].

Among methods of controlling swell-shrinkage of expansive subgrade soil; one is stabilization of the soil with geo-synthetic materials. Geo-synthetic materials have the effects of reducing the Swelling potentials of weak soils.

Table 4. 17 Swelling Values of the Soil under Study

Nature of Soil	Trials	CBR % @ 95% MDD	Corresponding Swelling Values	Averages
Natural Soil	1	2.8	4.87	6.25
	2	2	7.06	
	3	2.35	6.83	
Single Layer Reinforcement	1	13.6	1.04	1.59
	2	9.91	2.45	
	3	13.21	1.27	
Double Layer Reinforcement	1	6.8	0.46	0.88
	2	6	1.28	
	3	7.22	0.9	

As we can observes from the results of the tests natural soil swelling values corresponding to 95% MDD are deviated from minimum requirement which is 2% [37] whereas the reinforced soil are within the minimum requirement. Therefore, using geo-textile as reinforcing/stabilizing materials together with weak soil can minimize the swellings of the soil as shown above in the table. The percentage of swelling decreases relative to natural soil is as follow:

- For Single Reinforcement percentage of swelling decreases is about 74.56%
- For Double Reinforcement percentage of swelling decreases is about 85.92%

Hence, double reinforcement is effective in reducing the swelling of expansive subgrade soil. CBR Swell value of the soil under study decreased because of the effects of Geo-synthetic materials on degree of compaction. As a result, the volumes occupied with expansive clay minerals are avoided during compaction.

Table 4. 18 Summary of Geotechnical Properties of Natural Soil

Parameters	Test Results
Natural Moisture Content , %	49.54
Field Density , g/cm^3	1.08
Percentage of Passing Sieve # 200, %	98.05
Liquid Limit , %	67.04
Plastic Limit , %	31.76
Plastic Index , %	35.28
Shrinkage Limit , %	5.11
AASHTO Soil Classification	A-7-6 (21)

USCS	CH
Unconfined Compressive Strength, Kpa	24.08
Specific gravity	2.72
Free Swell Ratio	2.05
Clay Mineral Content	Montmorillonite
Average MDD , g/cm^3	1.484
Average OMC , %	25.5
Soaked CBR Value , %	2.38
CBR Swell , %	6.25
Color	Blackish Brown

4.3. Effects of Geo-synthetic Materials on Drainage Barrier Efficiency

One of the functions geo-synthetics possess is that they can act as a means to prevent moisture from infiltrating into the pavement structure, and such waterproofing action may limit base and subgrade movement due to freeze-thaw action or expansive soils, consequently, delaying the deterioration of the pavement structure. For example, if geotextiles are impregnated with bitumen, they are able to protect the underlying layers from degradation. Given that the fabric is saturated with sufficient asphalt to provide a continuous moisture barrier, fabrics may remain intact even after the asphalt overlay has cracked and provide a moisture barrier. However, if a moisture barrier is justified, fabrics and composites offer this added benefit but grids cannot. As a result, geo-synthetics may be used to limit base and subgrade movements by preventing surface water intrusion[15].

Water in pavement systems is one of the principal causes of pavement distress. It is well known that improved roadway drainage extends the life of a roadway system.in the 19th century, Mac-Adam recognized that it was necessary to have good drainage if adequate support was to be maintained and the road was to last. Adequate drainage is predicted to extend the life of a pavement system up to 2 to 3 times over that of un-drained pavement sections[41].

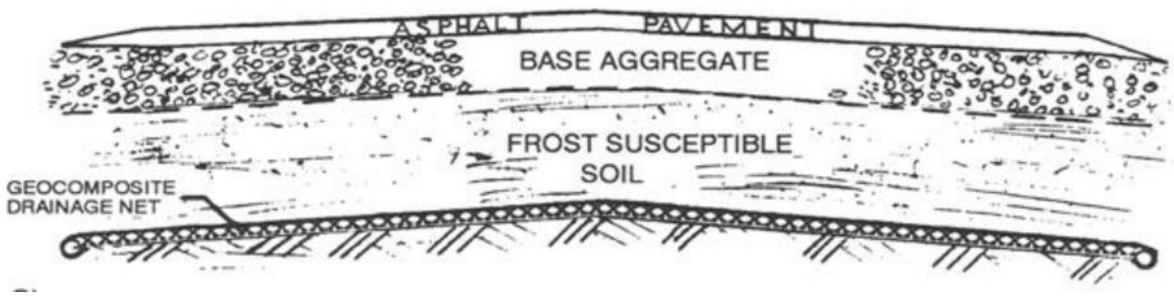

Figure 4. 8 Potential Use of Horizontal Geo-composite Drainage Layers [41]

After reviewing different literatures on using geo-membranes for minimizing moisture variation of expansive subgrade soil within the pavement; the researcher impregnated geo-membrane at different layers and checked moisture variation by simply taking sample for moisture content determination. The laboratory results of soil reinforced with geo-membrane both for single layer reinforcement and double layer reinforcement to determine moisture variation is shown as follows:-

The bottom and Top surfaces of the compacted and geo-reinforced expansive subgrade soil in CBR mould was exposed to excess moisture in water tanker so as to know the effectiveness of Geo-membrane in prohibiting moisture migration from one layer of soil to another soil layer. For single layer reinforcement the effectiveness of Geo-membrane is not this much visible relative to double layer reinforcement.

Table 4. 19 Moisture Variation of Single Layer Reinforced with Geo-membrane

Number of Blows	**65**			
Layer	**Top**		**Bottom**	
Can	A-13	ZE	B-3	G-10
Mass of Can g	36.3	32.8	17.4	34.1
Mass of Can +wet Soil g	197	195.5	121.3	190.5
Mass of Can + dry Soil g	167.7	165.8	100.2	163.7
Moisture Content in %	22.30	22.33	25.48	20.68
Average	22.31		23.08	
Number of Blows	**30**			
Layer	**Top**		**Bottom**	

Can	P-15	12	P-3	G
Mass of Can g	33.3	41.1	35.7	17.9
Mass of Can +wet Soil g	169.8	193	168.7	105.5
Mass of Can + dry Soil g	141.1	160.1	143.2	85.2
Moisture Content in %	26.62	27.65	23.72	30.16
Average	27.14		26.94	
Number of Blows	**10**			
Layer	**Top**		**Bottom**	
Can	SSB	K-4	GT-3	T-2D
Mass of Can g	17.5	17.7	17.6	17.6
Mass of Can +wet Soil g	191.5	143.6	246.4	153.4
Mass of Can + dry Soil g	136.1	102.9	179.9	110.3
Moisture Content in %	46.71	47.77	40.97	46.49
Average	47.24		43.73	

As it can be inferred from the results of single layer reinforcement shown above moisture variation observed has no significant influences on the performances of expansive subgrade soil due to small variations.

For double layer reinforcement we can easily shows the effectiveness of Geo-membrane in preventing moisture migration within the layer by comparing top, middle and bottom layers.

Table 4. 20 Moisture Variation of Double Layer Reinforced with Geo-membrane

Number of Blows	**65**					
Layer	**Top Layer**		**Middle Layer**		**Bottom Layer**	
Can	P-15	G	F	B-12	C	A-7
Mass of Can g	33.5	9.7	36.3	16.6	32.8	17.5
Mass of Can +wet Soil g	183.3	107.8	152.5	86.4	162.8	80.3
Mass of Can + dry Soil g	144.5	82.5	123.7	69.3	129.2	63.2
Moisture Content in %	34.95	34.75	32.95	32.45	34.85	37.42
Average	34.85		**32.70**		36.14	
Number of Blows	**30**					
Layer	**Top Layer**		**Middle Layer**		**Bottom Layer**	
Can	K-4	G-19	A-13	9	G3T3	A-1
Mass of Can g	17.7	35.9	36.5	32.4	34.7	37
Mass of Can +wet Soil g	92.4	154.6	188.6	189.7	193.8	189.2
Mass of Can + dry Soil g	71.6	117.7	145.3	146.6	151	142
Moisture Content in %	38.59	45.11	39.80	37.74	36.80	44.95
Average	41.85		**38.77**		40.88	

Number of Blows	**30**					
Layer	**Top Layer**		**Middle Layer**		**Bottom Layer**	
Can	SSB	6	C-11	T4C2	#A	II
Mass of Can g	17.4	27.6	27.7	16.6	27.5	17.5
Mass of Can +wet Soil g	105.8	184.3	179.1	80.6	174.7	142.3
Mass of Can + dry Soil g	77.3	133.5	143.5	60.4	130	105.1
Moisture Content in %	47.58	47.97	30.74	46.12	43.61	42.47
Average	47.77		**38.43**		43.04	

As mentioned under single layer reinforcement both bottom and top surfaces of the soil was exposed to excess water. Unlike single layer reinforcement it is not difficult to know its effectiveness. Since the soil on the Top and Bottom are exposed to excess moisture and migration of moisture towards to middle was prevented as a result of geo-membrane reinforcement. Hence, moisture of the middle layer should be less than the two layers. In practical world if there is doubt both from surface (i.e. moisture infiltration or Percolation) and subsurface below subgrade; the percolating and uplifting moisture migration can be controlled and minimized by geo-membrane reinforcement.

Therefore, from the observed cases we can conclude that as number of geo-membrane reinforcement increased the degree of effectiveness also increases (i.e. Moisture migration can be controlled than showed for the two cases).

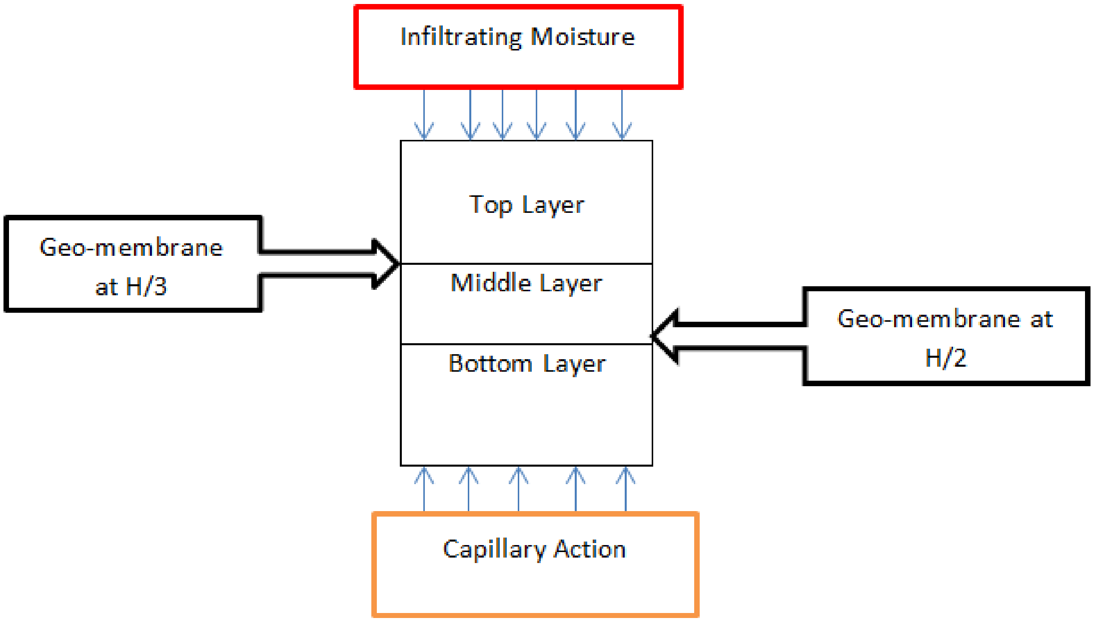

Figure 4. 9 Moisture Migration within Expansive Subgrade Soil and Function of Geo-membrane [Drawn by Researcher]

4.4. Percentage of Performance Improvement of Subgrade Soil in terms of CBR

It is observed that, as the placement of the geotextiles is deeper in the CBR mould, their performance is becoming poorer. In another sense, placing the geotextiles nearer to the surface of CBR mould could give best CBR values compare to other depths [18].

Table 4. 21 CBR Value Improvement in Percentage

	CBR Values

No of Layer	Geo-synthetic Types used	Natural Soil CBR	After Reinforcement	% increases of CBR
Single	Geo-textile	2.38	12.24	414.29
Double	Geo-textile	2.38	6.82	186.55

Unlike Geo-membrane; the increment of reinforcement layer showed the decreases in CBR values which shows the effective depth of attaining maximum strength was at middle of the mould for single layer reinforcement.

Generally, the uses of Geo-synthetic materials in pavement works over problematic soil like expansive subgrade material seem beneficial.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

By combining reviewed literatures with laboratories results the following conclusions have been drawn:-

5.1. Conclusions

1. The soils under investigation (i.e. JIT Compounds around the Stadium under construction) were classified as CH (on the basis of USCS) and A-7-6(21) (on the basis of AASHTO Soil Classification) possessing the following properties:-

- The Natural Moisture Contents of the soil was nearby the plastic. Hence, the soil of JIT is more likely to swell.
- A Fat Clay Soil with clay mineral to be montmorillonite. The mineral content of the soil was determined by free swell ratio.
- High Swell Potential.
- High plastic clay with specific gravity G_s=2.72.
- Poor Subgrade soil in which it's CBR and Group Index (GI) was 2.38 and 21.

2. Reinforcing Expansive subgrade soil with non-woven geo-textile materials increased the degree of compaction which in turns increases the density of soil due to increased bonding capacity even if the percentage of increment was small.

- For single Layer reinforcement OMC=28.2% and MDD=1.38 g/cm^3
- For Double Layer Reinforcement OMC=28.12% and MDD=1.42g/cm^3

3. The CBR value of soils under study was increased relative to natural CBR. The Natural CBR was 2.38% whereas CBR values after reinforcement with non-woven geo-textiles was 12.24% and 6.82% for single layer and double layer reinforcement respectively showing the negative effects of increasing number of reinforcement layer on CBR values. The percentages of CBR increases relative to natural soil CBR was about 414% and 186% for single and double reinforcement respectively. Hence, Single layer reinforcement was the optimum depth of

reinforcement and has shown good performances on performance improvements of expansive subgrade soil.

4. The effectiveness of geo-textile in reducing swelling value was observed clearly. The natural CBR swell value was 6.25% whereas CBR swell values after reinforcement with non-woven geo-textiles was 1.59% and 0.88% for single layer and double layer reinforcement respectively. Double reinforcement showed minimum values of swelling than single reinforcement. Therefore, using non-woven geo-textile as reinforcing/stabilizing materials together with expansive subgrade soil minimize the swelling potential of the soil. The percentage of swelling decreases relative natural soil is as follow:

- ❖ For Single Reinforcement percentage of swelling decreases is about 74.56%
- ❖ For Double Reinforcement percentage of swelling decreases is about 85.92%

Hence, double reinforcement is effective in reducing the swelling of expansive subgrade soil.

5. Geo-membrane Reinforcement for preventing moisture variation was clearly observed on double layer reinforcement. Moisture of top and bottom layer of compacted and reinforced expansive subgrade soil exposed to excess moisture was greater than that of middle layer for all cases showing that moisture migration was controlled as much as possible.

- ✓ Average Moisture Content of Top layer 47.77%
- ✓ Average Moisture Content of Middle layer **38.43%**
- ✓ Average Moisture Content of Bottom layer 43.08%

5.2. Recommendations

Based on the findings of this research, the following recommendations are forwarded: The results the findings obtained during this investigation were discussed in the previous sections.

1. Geo-synthetic materials (Geo-textile and Geo-membrane) as investigated in this particular study can be used as a powerful soil modifying material to increase the strength and/or stiffness of the soil (CBR) and prevent moisture content movement within expansive soil, which is sensitive to moisture, of pavement layers being economical and environmentally friendly without causing damages or pollution to under lying living things and water surfaces like chemical stabilization. Therefore, concerned bodies of

different sectors and government bodies being together should have to perform promotion works so as to create awareness about the importance of geo-synthetic materials in soil stabilization and promote its standardized manufacturing and utilization.

2. Higher education institution, any governmental and non-governmental organization participating in academic affairs have to stand and work in collaboration with Geo-synthetic manufacturing industry to set out a research team so that further investigation of using Geo-synthetic material in pavements would be known than before.
3. While using Geo-synthetic material for weak soil stabilization the proper and appropriate geo-synthetic materials should have to be selected.

5.3. Future Research Ideas

1. Comparing the better performances of different geo-synthetic materials (geo-grid, geo-textile and geo-composite) in Stabilization of weak subgrade.
2. Stabilization of weak subgrade soil with geo-textile and geo-membrane by using different thickness.
3. Comparative studies on effectiveness of asphalt and geo-membrane reinforcement in controlling moisture variation (Waterproofing).
4. Stabilization of Weak Subgrade Soil with Geo-synthetic Materials Including the Issue of Durability.

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Appendix A: - Details of Natural Soil Laboratory Results

Natural Moisture Content of JiT Test Pit	**Number of Trial**					
Moisture content determination	1	2	3	4	4	5
Container No	C	10G	P-1	P-15	A	D
Mass of wet soil + container (A)	89.00	68	81.4	108.7	77.9	99.7
Mass of dry soil + container (B)	66.2	52.1	61.4	85.5	59.7	78.8
Mass of container (C)	17.30	17.6	17.8	33.2	19.4	34.5
Mass of moisture = A-B = (D)	22.80	15.90	20.00	23.20	18.20	20.90
Mass of dry soil = B-C =(E)	48.90	34.50	43.60	52.30	40.30	44.30
Natural Moisture content (%) = D/E*100 =(F)	46.63	46.09	45.87	44.36	45.16	47.18
Average Moisture Content	**45.88**					

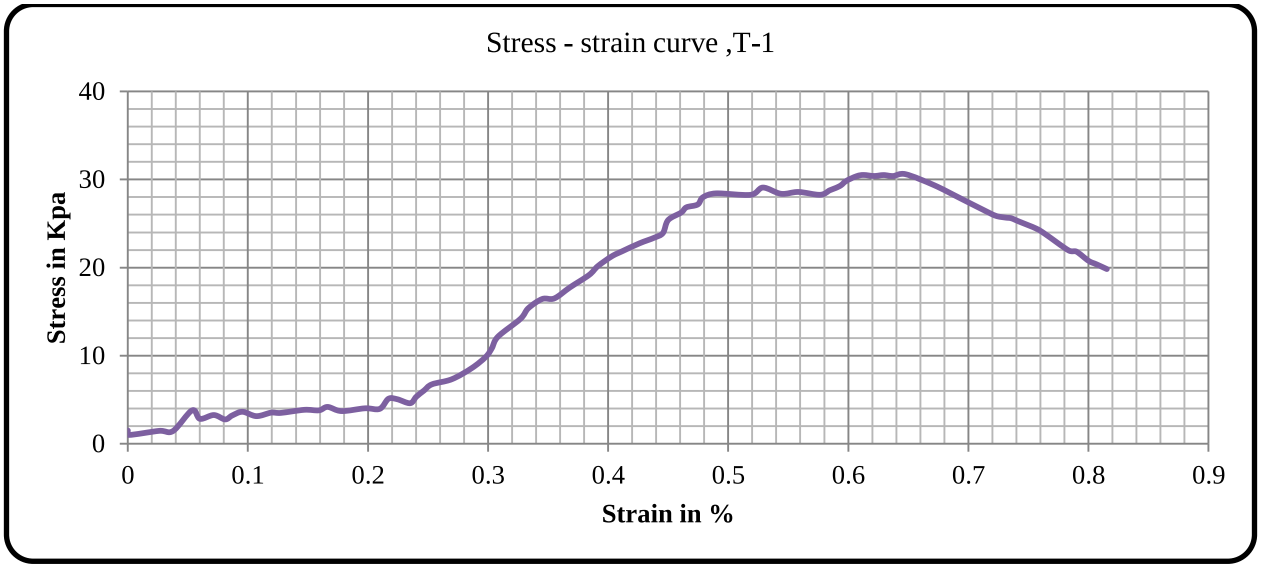
Field Density by Sand Replacement Method

Number of Trials	1	2	3
Mass of wet soil (g) = W_w	4101.1	3950	3839.3
Mass of sand + jar before pouring (g) = W1	7880	7296	8360
Mass of sand + jar after pouring (g) = W2	1780	1600	2348
Mass sand in cone (g) = W3	2022.7	1898.5	2572.5
Mass of sand in hole (g) = w1-w2-w3	4077.3	3797.5	3439.5
Density of Sand (g/cm3)	1.485	1.535	1.467
Volume of Hole (cm3)	2745.66	2473.94	2344.58
Bulk Density of Soil (g/cm3)	1.49	1.60	1.64
Natural Moisture Contnet (%)	46.63	46.08	45.87
Field Dry Density of Soil (g/cm3)	1.02	1.09	1.12
Average Field Density	1.08		

Unconfined Compression Test Trial One

∆L(mm)	Є(%)	Ac(mm2)	Load(N)	Stress(Kpa)
5.8775E-37	7.347E-39	1075.2	1.6189963	1.505762893
0.0579834	0.0007248	1075.9799	1.0793209	1.003105119
2.08358765	0.0260448	1103.9523	1.6189963	1.466545531
3.00979614	0.0376225	1117.233	1.6189963	1.449112401
4.28314209	0.0535393	1136.0218	4.3173134	3.800379089
4.80422974	0.0600529	1143.8941	3.2379925	2.830675014
5.72967529	0.0716209	1158.1476	3.777653	3.261806099
6.48269653	0.0810337	1170.0103	3.2379925	2.767490689
6.94580078	0.0868225	1177.4272	3.777653	3.208396267
7.64007568	0.0955009	1188.7243	4.3173134	3.631887942
8.56628418	0.1070786	1204.1373	3.777653	3.137227834
9.54971313	0.1193714	1220.9461	4.3173134	3.536039456
10.1867676	0.1273346	1232.0873	4.3173134	3.504064431
11.7492676	0.1468658	1260.2942	4.8569888	3.853853261
12.7334595	0.1591682	1278.7338	4.8569888	3.798279775
13.3117676	0.1663971	1289.8228	5.3966492	4.18402389
14.2379761	0.1779747	1307.9889	4.8569888	3.713325577
15.8004761	0.197506	1339.823	5.3966492	4.027882163
16.784668	0.2098083	1360.6826	5.3966492	3.966133901
17.3629761	0.2170372	1373.2453	7.0156455	5.108807144
17.9420471	0.2242756	1386.0593	7.0156455	5.061576894
18.8102722	0.2351284	1405.7261	6.4759851	4.606861076
19.2153931	0.2401924	1415.0951	7.555306	5.339079972
19.7364807	0.246706	1427.3312	8.6346418	6.049501263
20.2568054	0.2532101	1439.7623	9.7139776	6.746931413
21.5881348	0.2698517	1472.5775	10.793298	7.329528186
22.9194641	0.2864933	1506.9235	12.95197	8.594975285
23.8456726	0.2980709	1531.7787	15.110627	9.864758712
24.2507935	0.3031349	1542.9099	16.729623	10.84290379
24.6559143	0.3081989	1554.204	18.88828	12.15302474
26.1604309	0.3270054	1597.6354	22.665948	14.18718443
26.6815186	0.333519	1613.2492	24.824604	15.3879535
27.6077271	0.3450966	1641.7688	26.983261	16.43548155
28.4179688	0.3552246	1667.5574	27.522936	16.50494058
29.4593811	0.3682423	1701.9182	30.221254	17.75717149
30.7327271	0.3841591	1745.9054	33.459246	19.1644091
31.3117981	0.3913975	1766.6703	35.617918	20.16104413
32.2380066	0.4029751	1800.9299	38.316235	21.27580637

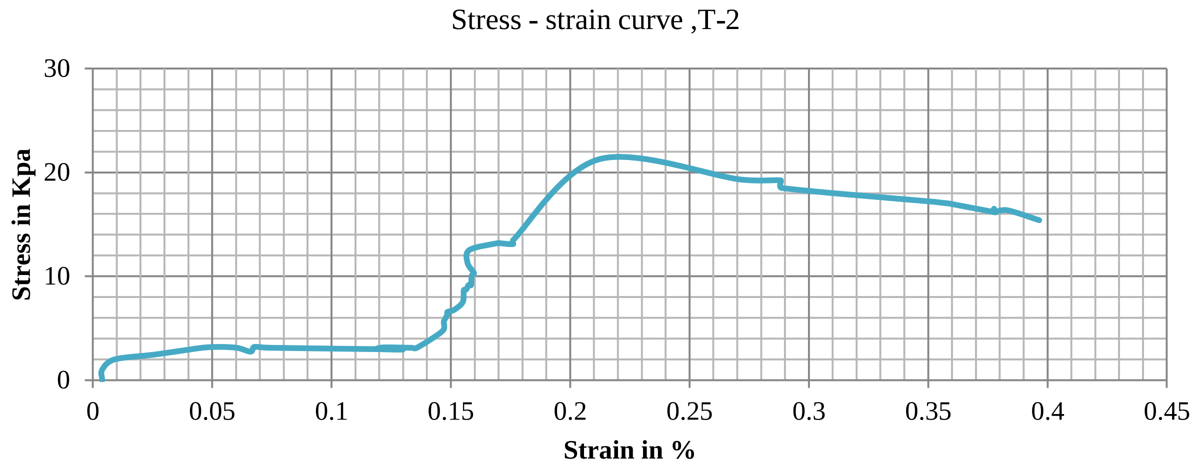
32.9322815	0.4116535	1827.4946	39.935231	21.85244863
34.0896606	0.4261208	1873.5649	42.633563	22.75531713
35.0738525	0.4384232	1914.6089	44.79222	23.39497164
35.6521606	0.445652	1939.5759	46.411216	23.92853845
36.000061	0.4500008	1954.9118	49.649209	25.39716048
36.8675232	0.460844	1994.2282	52.347541	26.24952441
37.2154236	0.4651928	2010.4441	53.966537	26.84309235
37.9676819	0.474596	2046.4253	55.585533	27.16225844
38.3148193	0.4789352	2063.4671	57.74419	27.98406106
39.1830444	0.4897881	2107.3595	59.902847	28.42554684
41.4978027	0.5187225	2234.0543	63.140839	28.26289355
42.3080444	0.5288506	2282.0785	66.378832	29.08700685
43.523407	0.5440426	2358.115	66.918507	28.37796635
44.6815491	0.5585194	2435.4409	69.616824	28.58489579
46.1280823	0.576601	2539.4488	71.775496	28.26420312
46.7651367	0.5845642	2588.1256	74.473821	28.77519585
47.1702576	0.5896282	2620.0632	76.092802	29.04235363
47.517395	0.5939674	2648.0635	77.711813	29.34665793
47.8645325	0.5983067	2676.6687	79.870485	29.8395109
48.5588074	0.6069851	2735.7741	83.108477	30.37841382
49.0219116	0.6127739	2776.6723	84.727459	30.51402881
49.7169495	0.6214619	2840.4008	86.34647	30.39939673
50.353241	0.6294155	2901.3627	88.505141	30.50468054
50.9895325	0.6373692	2964.9988	90.124123	30.396007
51.8005371	0.6475067	3050.2709	93.362115	30.60781141
53.7101746	0.6713772	3271.8361	96.060447	29.3597981
55.4466248	0.6930828	3503.2251	97.679429	27.88271563
57.0091248	0.7126141	3741.3104	99.29844	26.54108589
57.8193665	0.7227421	3877.9776	100.37776	25.88404865
58.4564209	0.7307053	3992.6513	102.53643	25.68128887
58.8615417	0.7357693	4069.1709	104.15541	25.59622473
59.3818665	0.7422733	4171.8616	105.23476	25.22489337
60.5392456	0.7567406	4419.9725	107.9331	24.41940434
61.1183167	0.763979	4555.526	108.47276	23.81124733
62.6808167	0.7835102	4966.5159	109.01242	21.94947505
63.2019043	0.7900238	5120.5804	111.71075	21.81603263
64.012146	0.8001518	5380.0842	111.71075	20.76375505
64.5332336	0.8066654	5561.3435	113.32973	20.37812116
65.2275085	0.8153439	5822.7145	115.4884	19.83411737



Unconfined Compression Test Trial Two

ΔL(mm)	Є(%)	Ac(mm2)	Load(N)	Stress(Kpa)
0.289692	0.003900	1079.409698	0.000000	0.000000
0.290230	0.003907	1079.417540	1.079321	0.999910
0.669623	0.009015	1084.980944	2.158657	1.989580
1.885222	0.025380	1103.199133	2.698317	2.445902
2.680394	0.036085	1115.451060	3.118996	2.796175
3.596950	0.048424	1129.915252	3.579321	3.167778
4.435628	0.059715	1143.483059	3.579321	3.130191
4.911282	0.066118	1151.323794	3.158657	2.743500
5.032949	0.067756	1153.346674	3.698317	3.206596
5.519039	0.074300	1161.499996	3.618996	3.115795
9.555577	0.128643	1233.936932	3.648996	2.957198
8.843146	0.119052	1220.502681	3.658657	2.997664
9.050307	0.121840	1224.378841	3.858657	3.151522
9.907980	0.133387	1240.692084	3.861900	3.112698
10.082802	0.135740	1244.070743	3.867900	3.109067
10.885625	0.146549	1259.825597	5.969832	4.738618
10.922867	0.147050	1260.566133	6.998317	5.551726
10.936195	0.147229	1260.831358	7.258657	5.757040
11.015684	0.148299	1262.415538	7.798317	6.177298
11.027850	0.148463	1262.658367	8.018996	6.350884
11.041758	0.148650	1262.936066	8.301900	6.573492

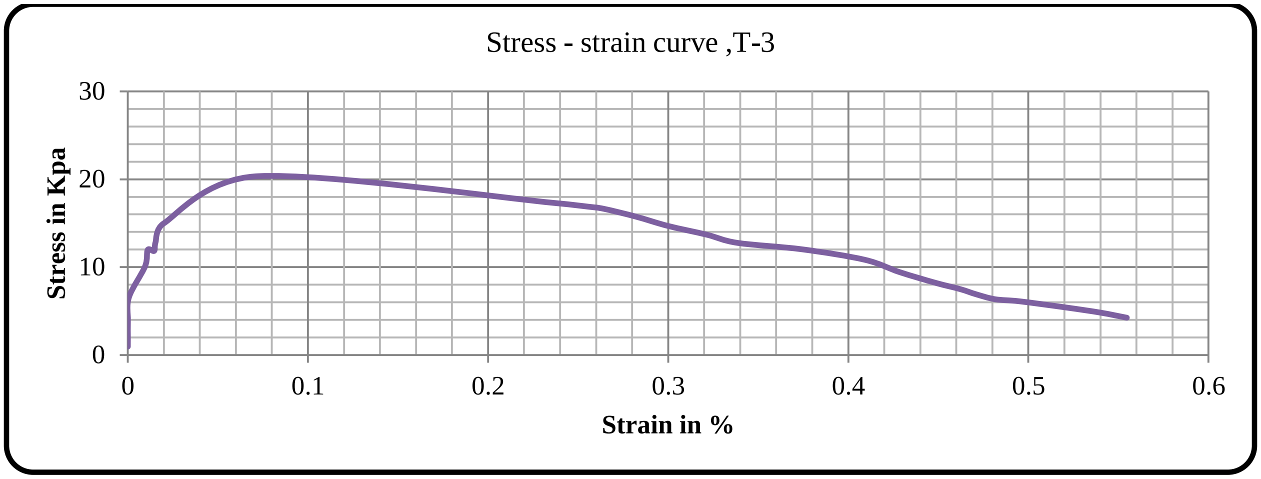
11.237810	0.151290	1266.863598	8.558657	6.755784
11.512455	0.154987	1272.406879	9.558657	7.512264
11.544797	0.155423	1273.062844	11.018996	8.655501
11.621111	0.156450	1274.613351	11.118996	8.723427
11.696144	0.157460	1276.141511	11.698317	9.166944
11.771515	0.158475	1277.680233	11.698317	9.155904
11.792490	0.158757	1278.109113	12.858657	10.060688
11.860430	0.159672	1279.500252	13.198317	10.315213
11.661048	0.156988	1275.426259	14.398317	11.289024
11.726957	0.157875	1276.770117	16.017313	12.545182
12.615936	0.169843	1295.176662	17.096649	13.200245
12.623909	0.169950	1295.344139	17.096649	13.198538
13.084280	0.176148	1305.088926	17.096649	13.099988
13.090658	0.176234	1305.224948	17.636310	13.512084
15.769930	0.212304	1364.993353	29.046951	21.279922
20.138739	0.271119	1475.138443	28.507276	19.325153
21.412694	0.288270	1510.685177	29.046951	19.227667
21.437891	0.288609	1511.405545	27.967616	18.504375
26.188005	0.352558	1660.689170	28.507276	17.165931
26.234289	0.353181	1662.288966	28.507276	17.149411
26.431339	0.355834	1669.134613	28.507276	17.079075
26.558811	0.357550	1673.593172	28.507276	17.033576
26.657260	0.358875	1677.052936	28.507276	16.998435
28.013170	0.377129	1726.201156	27.967616	16.201829
28.117595	0.378535	1730.106040	27.967616	16.165261
28.034608	0.377418	1727.001402	27.967616	16.194321
28.049670	0.377621	1727.564042	28.507276	16.501429
28.053614	0.377674	1727.711425	27.967616	16.187666
28.470204	0.383282	1743.423100	28.507276	16.351324
29.454694	0.396536	1781.713554	27.427955	15.394144



Unconfined Compression Test Trial Three

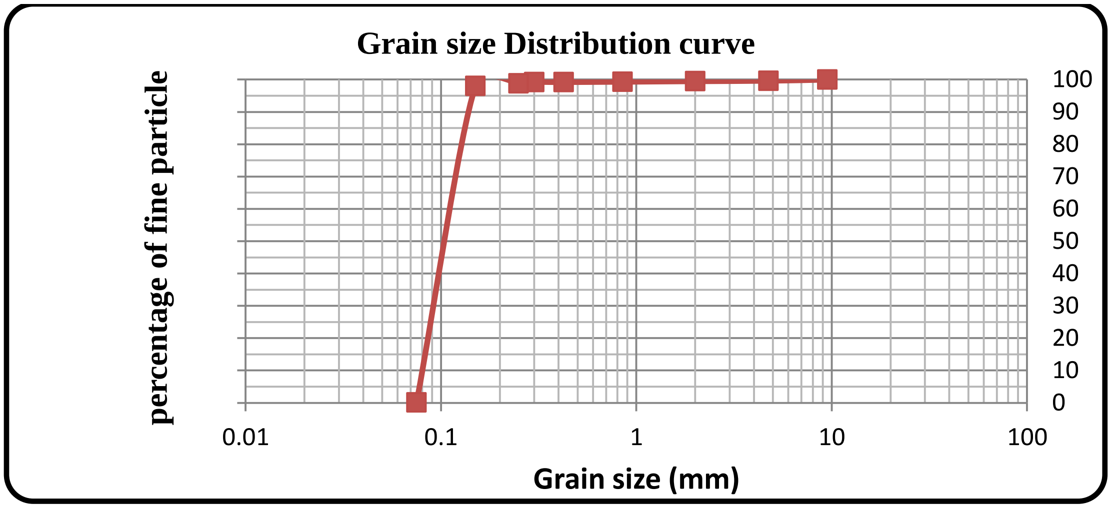
ΔL(mm)	€(%)	Ac(mm2)	Load(N)	Stress(Kpa)
6.06E-37	8.1632E-39	1134.1	1.0793507	0.95172446
0.002031	2.7339E-05	1134.131	2.2379925	1.973310415
0.003344	4.5024E-05	1134.1511	4.4759851	3.94655103
0.047769	0.00064309	1134.8298	7.4759851	6.587758827
0.710593	0.00956641	1145.0541	11.555336	10.09151984
0.818032	0.01101282	1146.7287	13.713978	11.95921702
1.086748	0.01463042	1150.9387	13.634657	11.84655309
1.144587	0.01540909	1151.8489	14.793328	12.84311481
1.48683	0.02001656	1157.2645	17.34623	14.98899398
5.365001	0.07222672	1222.3892	24.872649	20.34757009
17.67431	0.23794174	1488.2064	25.713978	17.27850186
18.98798	0.2556271	1523.5643	25.713978	16.87751345
19.64521	0.26447508	1541.892	25.634657	16.62545523
20.95809	0.28214984	1579.8562	24.872649	15.74361616
22.27176	0.29983521	1619.7615	23.793328	14.68940201
23.88453	0.32154719	1671.5975	22.872649	13.68310829
25.13837	0.33842723	1714.2483	21.872649	12.75932388
27.82474	0.37459268	1813.3782	21.793328	12.01808231
30.39226	0.40915807	1919.4643	20.793328	10.83288087
31.82557	0.42845408	1984.2675	18.713978	9.43117671
33.3187	0.44855542	2056.5983	16.872649	8.20415396
34.33327	0.46221415	2108.832	15.793328	7.489135389
34.9905	0.47106213	2144.1082	14.793328	6.899525216

35.76657	0.48151016	2187.3138	13.872649	6.342322218
36.78193	0.49517949	2246.5411	13.713978	6.10448558
38.69222	0.52089691	2367.1315	12.793328	5.404570225
40.12553	0.54019292	2466.4692	11.872649	4.813621492
41.19992	0.55465698	2546.5764	10.793328	4.238368211



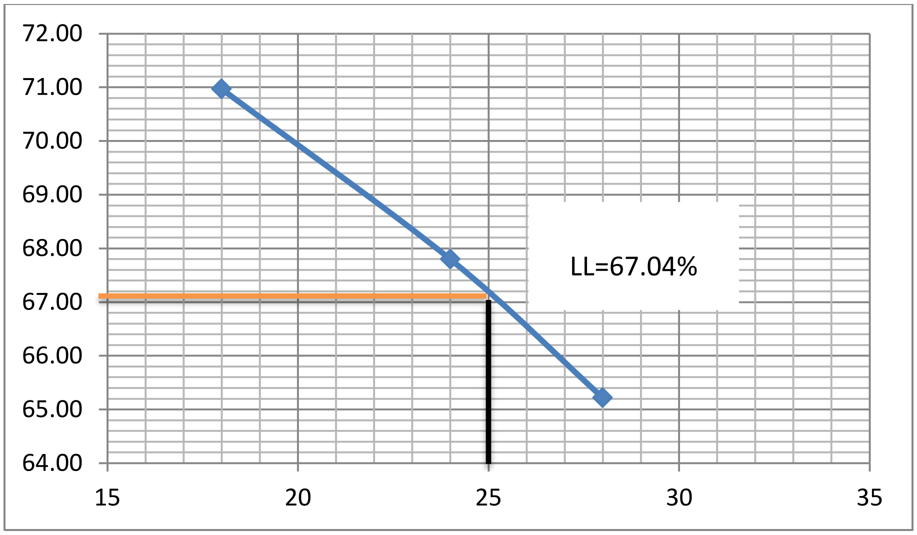
Particle Size Distribution

Sieve size (mm)	Mass of Retain on Each Sieve (g)	Percentage of Retained Soil	Percentage of cumulative Retained Soil	Percentage of Passing Soil Particle
9.500	0.000	0.00	0.00	100.00
4.750	0.200	0.02	0.02	99.98
2.000	3.500	0.35	0.37	99.63
0.850	1.500	0.15	0.52	99.48
0.425	1.600	0.16	0.68	99.32
0.300	1.000	0.10	0.78	99.22
0.250	0.800	0.08	0.76	99.24
0.150	4.500	0.45	1.21	98.79
0.075	7.400	0.74	1.95	98.05
pan	979.500	97.95	99.90	0.10
sum	1000.000			



Atterberg's Limit Test

Material location:	**JiT Compound**				
Location	**selective material**				
Determination	***Liquid Limit (D-4318)***			***Plastic Limit (D-4318)***	
Number of blows	28	24	18		
Test No	II	ZE	D	G-14	A-17
Wt. of Container, (g)	17.90	33.1	29.70	20.30	21.90
Wt. of container + wet soil, (g)	21.70	43	35.00	26.00	27.40
Wt. of container + dry soil, (g)	20.20	39	32.80	24.60	26.10
Wt. of water, (g)	1.50	4.00	2.20	1.40	1.30
Wt. of dry soil, (g)	2.30	5.90	3.10	4.30	4.20
Moisture container, (%)	65.22	67.80	70.97	32.56	30.95
Average	67.99			31.76	



Determination of (PI) (LL - PL)	LL (%)	67.04	Sieve Analysis Result	Percentage of Course Soil	0.02	Soil Classification (USCS)	Soil Classification (AASHTO)
	PL (%)	31.76		Percentage of Sandy Soil	1.93		
	PI (%)	35.28		Percentage of Fine Soil	98.05	**CH**	**A-7-6(21)**

Linear Shrinkage and Shrinkage Product

LINEAR SHRINKAGE AND SHRINKAGE PRODUCT		
Trial 1	1	2
Initial Length L0	140.00	140.00
Oven dried Length LD	133.75	131.84
Linear shrinkage,Ls	**0.04**	**0.06**
Shrinkage Product, SP=Ls×%<425µm	**4.43**	**5.79**
Average Shrinkage Product, Sp	**5.11**	

Specific gravity of soil at test Temprature,G at Tx^o c (ASTM D-854)

Location:	JiT Compound		
Trial No.	1	2	3
Mass of empty Pycnometer (g)	25.4	26.9	26.4
Mass of empty Pycnometer+dry soil (g)	49	50.8	50.4
Mass of empty Pycnometer+dry soil+water at temperature Tx^o c (g)	133.6	137.5	139.3
Test temperature(Tx),oc	20	20	20
Density of water at Tx,g/ml	0.9982343	0.9982343	0.9982343
Mass of empty Pycnometer+water at temperature Ti^o c(20^oc) (g)	118.6	122.4	124.2
Density of water at Ti,g/ml at20^0c	0.9982343	0.9982343	0.9982343
Correction factor,k	1	1	1
Specific gravity G at Tx^o c	2.74	2.72	2.70
Average specific gracity at Tx^o c	2.72		

Free Swell Test Result

Trials	Initial Volume(ml)	Final volume (ml)	Free Swell in %
Trial 1	11.00	17.00	54.55
Trial 2	12.00	17.50	45.83
Trial 3	10.00	14.50	45.00
	Average		**48.46**

Natural Soil Compaction Trial One

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm³)	Moisture content Determination				Average Moisture content,%	Dry unit weight (g/cm³)
			Mass of can, g	Mass of wet soil + can, g	Mass of dry soil + can, g	Moisture content,%		
1	10292.2	1.60	32.8	124.6	106.9	23.89	24.07	1.29
			33.1	120.7	103.6	24.26		
2	10682.6	1.77	35.5	169.5	140.3	27.86	27.94	1.38
			17.3	111.4	90.8	28.03		
3	10685.5	1.77	17.8	130.8	104.3	30.64	30.74	1.36
			17.6	115.6	92.5	30.84		
4	10673.7	1.77	17.5	60.4	48.9	36.62	36.62	1.29
			17.5	60.4	48.9	36.62		
Mass of mould=6635.5g								

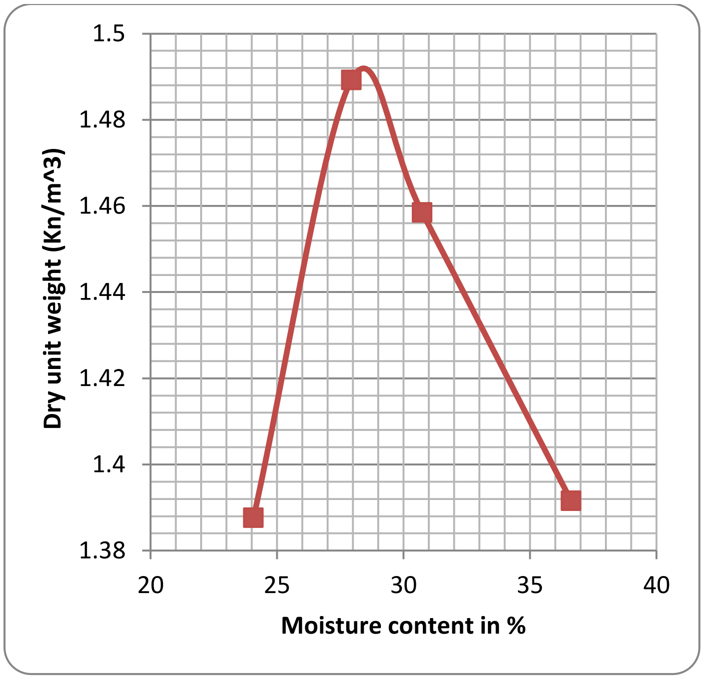
Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm³)	Average Moisture content,%	Dry unit weight (g/cm³)
1	10292.2	1.60	24.07	1.29
2	10682.6	1.77	27.94	1.38
3	10685.5	1.77	30.74	1.36
4	10673.7	1.77	36.62	1.29
		OMC (%)		**28.5**
		MDD (g/cm³)		**1.49**

Natural Moisture during Compaction

Can	A-53	P-67	P-3
Mass of can(g)	36.5	35.5	36
mass of can + Soil (g)	60.5	78.9	71.8
Mass of can + dry Soil (g)	58.1	74.6	68.1
Moisture Content in (%)	10.00	9.91	10.34
Average	10.08		

Optimum Moisture Content for CBR

OMC of During Compaction	Natural Moisture Content	OMC For CBR	Amount of H_2O
28.5	10.08	16.73	836.61



Natural Soil Compaction Trial Two

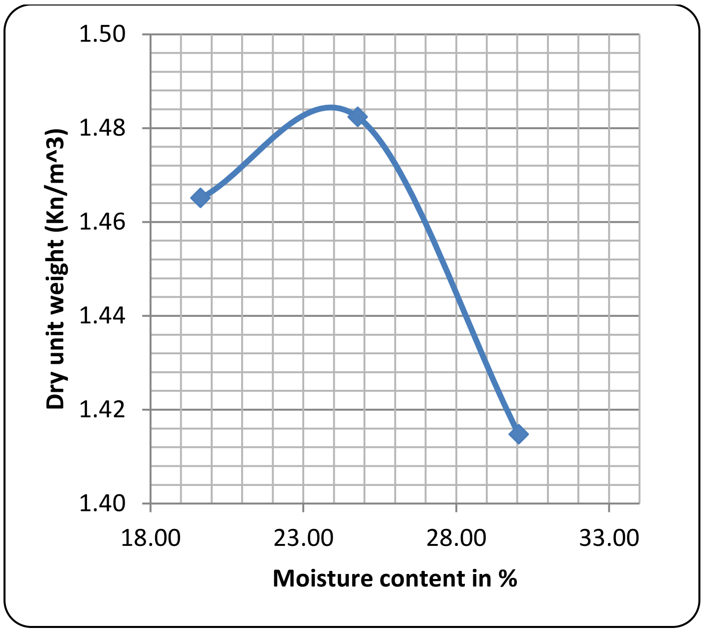
Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Moisture content Determination				Average Moisture content,%	Dry unit weight (g/cm^3)
			Mass of can, g	Mass of wet soil + can, g	Mass of dry soil + can, g	Moisture content,%		
1	10570.6	1.75	16.7	128.8	110.1	20.02	19.64	1.47
			17.1	139.1	119.4	19.26		
2	10792	1.85	18.4	111.7	93.5	24.23	24.78	1.48
			17.6	114.6	95	25.32		
3	10777.6	1.84	19.4	110.9	88.6	32.23	30.05	1.41
			19.6	114.1	93.5	27.88		
Mass of mould=6565.5g								

OMC and MDD Determination for Sample Two

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Average Moisture content,%	Dry unit weight (g/cm^3)
1	10570.6	1.75	19.64	1.47
2	10792	1.85	24.78	1.48
3	10777.6	1.84	30.05	1.41
		OMC (%)		**24**
		MDD(g/cm^3)		**1.48**

Natural Moisture Content During Compaction		
Can	B-11	AA
Mass of can(g)	17	19.3
mass of can + Soil (g)	59.4	72
Mass of can + dry Soil (g)	54.7	66.2
Moisture Content in (%)	11.08	11.01
Average	11.05	

OMC	NMC	New OMC	Amount of H_2O
24	11.05	11.67	583.31



Natural Soil Compaction Trial Three

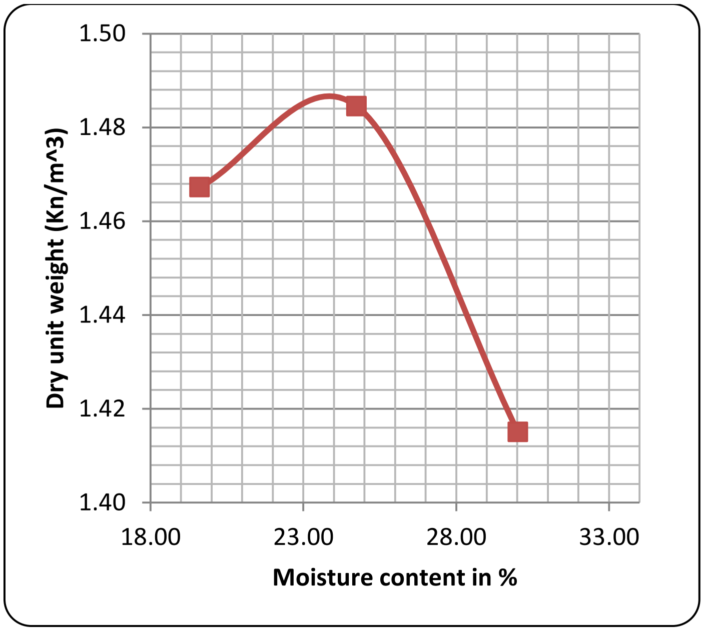
Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Moisture content Determination				Average Moisture content,%	Dry unit weight (g/cm^3)
			Mass of can, g	Mass of wet soil + can, g	Mass of dry soil + can, g	Moisture content,%		
1	10570.6	1.75	16.7	128.65	110	19.99	19.61	1.47
			17.1	138.95	119.3	19.23		
2	10792	1.85	18.4	111.55	93.4	24.20	24.75	1.48
			17.6	114.45	94.9	25.29		
3	10777.6	1.84	19.4	110.75	88.5	32.20	30.02	1.42
			19.6	113.95	93.4	27.85		

OMC and MDD Determination

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Average Moisture content,%	Dry unit weight (g/cm^3)
1	10570.6	1.75	19.61	1.47
2	10792	1.85	24.75	1.48
3	10777.6	1.84	30.02	1.42
		OMC (%)		**24**
		MDD(g/cm^3)		**1.484**

Natural Moisture Content During Compaction		
Can	B-11	AA
Mass of can(g)	17	19.3
mass of can + Soil (g)	59.4	72
Mass of can + dry Soil (g)	54.7	66.2
Moisture Content in (%)	11.08	11.01
Average	11.05	

OMC	NMC	New OMC	Amount of H_2O
24	11.05	11.67	583.31

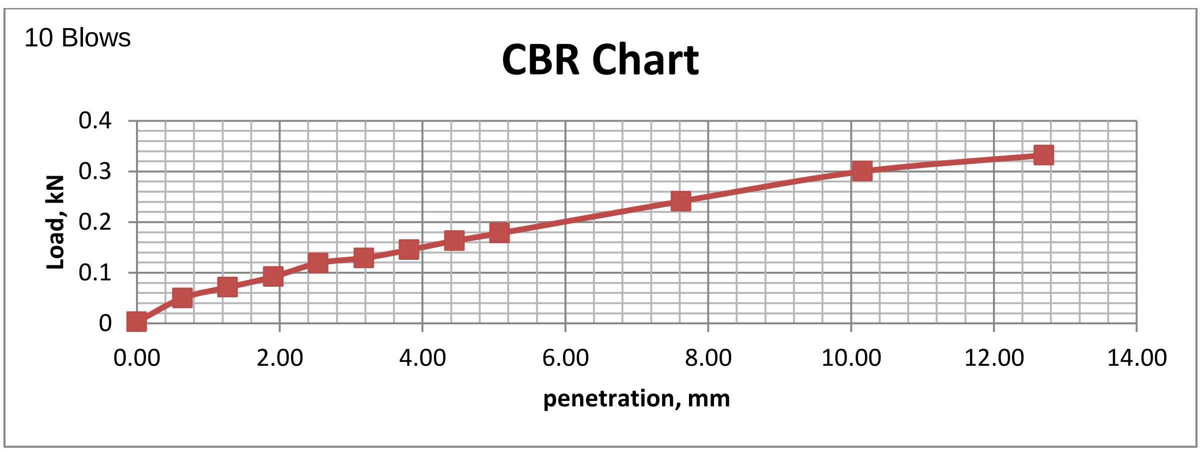

1.50
1.48
1.46
1.44
1.42
1.40
Dry unit weight (Kn/m^3)
18.00
23.00
28.00
33.00
Moisture content in %

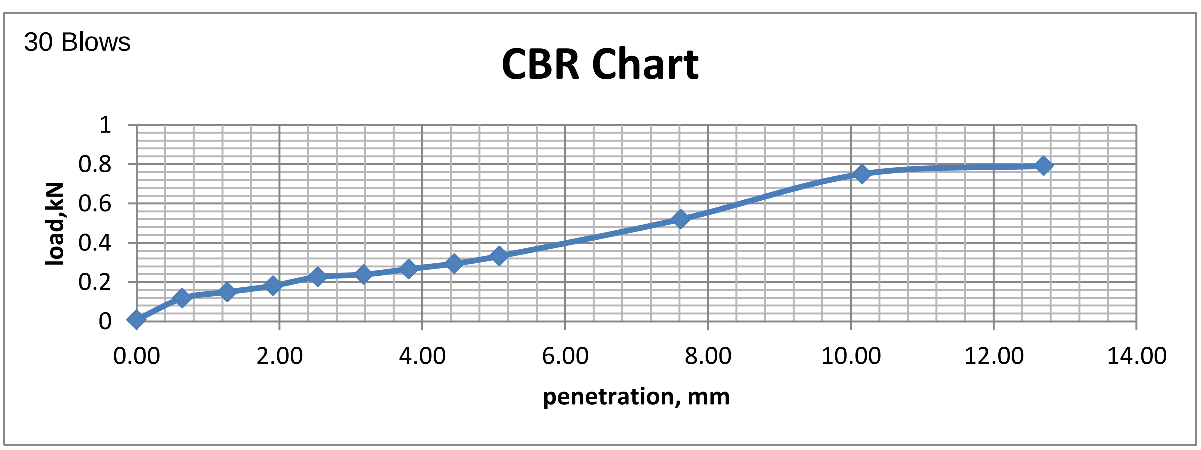
CBR Data of Natural Soil Trial One

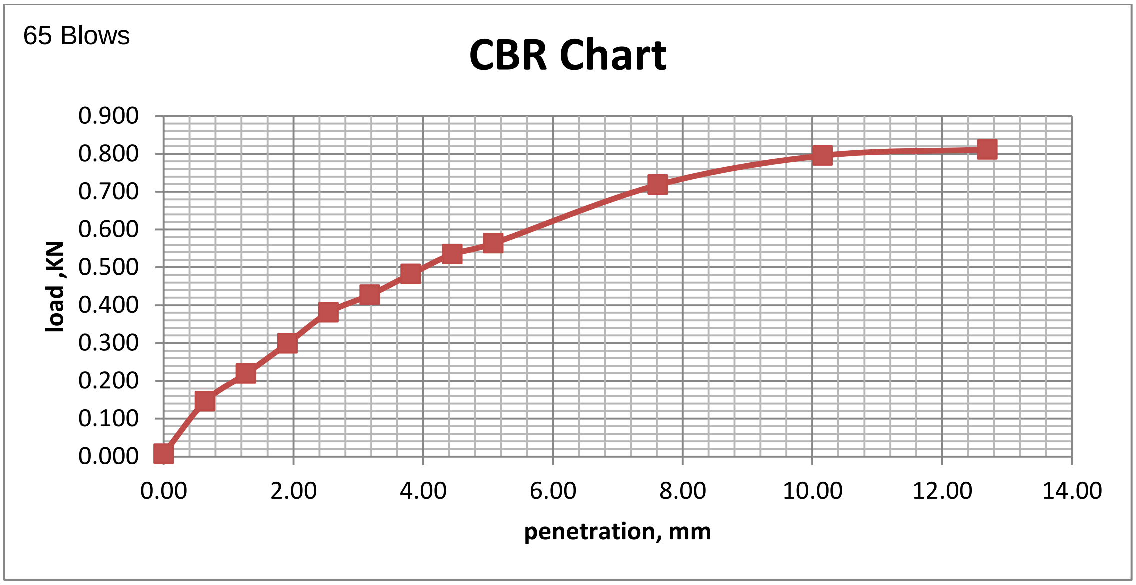
COMPACTION DATA		65 Blows		30 Blows		10 Blows	
		Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.		1	1	2	2	3	3
Mass of soil + Mould	g	13325.2	13740.5	12811.5	13331.4	12259.4	12911.78
Mass Mould	g	9285.1	9285.1	9412.3	9412.3	9282.9	9282.9
Mass of Soil	g	4040.1	4455.4	3399.2	3919.1	2976.5	3628.88
Volume of Mould	g	2285	2285	2285	2285	2285	2285
Wet density of soil	g/cc	1.768	1.950	1.488	1.715	1.303	1.588
Dry density of soil	g/cc	1.422	1.431	1.178	1.206	1.032	1.093

MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		After soak		Before soak		After soak		Before soak		After soak	
Container no.	A	T-1	C-2	NCP-1	Q-2	P-67	K-4	P-2	G-90	A-1	A-13	T-2D
Mass of wet soil + Container	164.0	175.8	80.6	89.9	141.7	143.5	96.0	73.8	109.2	74.7	153.0	91.6
Mass of dry soil + Container	139.3	148.6	63.7	70.8	117.1	122.1	73.5	56.6	93.8	62.7	116.3	68.8
Mass of container	37.0	37.6	17.6	17.4	28.7	35.5	17.8	17.6	34.1	17.6	36.4	17.7
Mass of water	24.7	27.2	16.9	19.1	24.6	21.4	22.5	17.2	15.4	12.0	36.7	22.8
Mass of drysoil	102.3	111.0	46.1	53.4	88.4	86.6	55.7	39.0	59.7	45.1	79.9	51.1
Moisture content	24.1	24.5	36.7	35.8	27.8	24.7	40.4	44.1	25.8	26.6	45.9	44.6
Average moisture content	24.3		36.2		26.3		42.2		26.2		45.3	

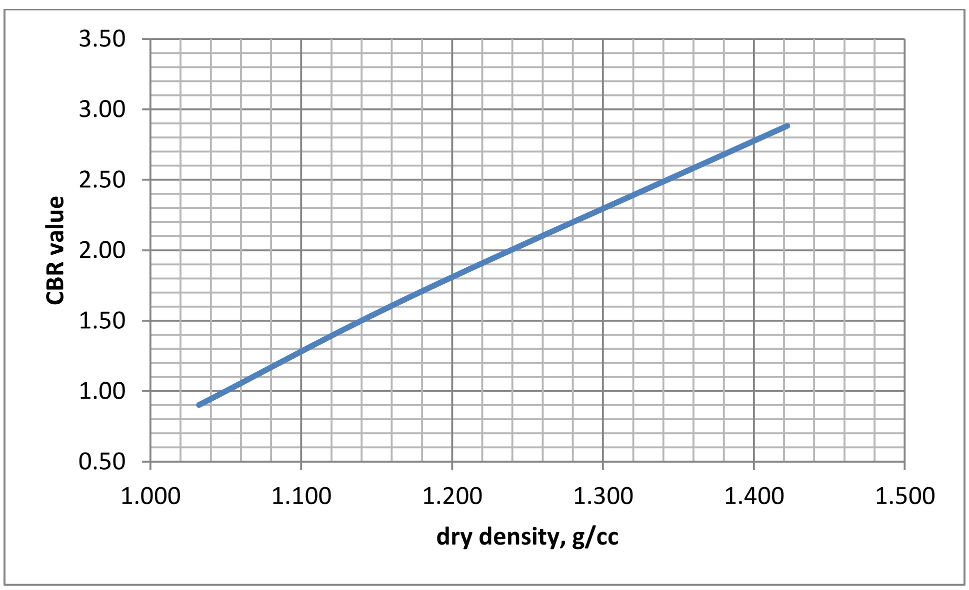
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG				
10 Blows			30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.003		0.00	0.008		0.00	0.007	
0.64	0.05		0.64	0.117		0.64	0.146	
1.27	0.071		1.27	0.149		1.27	0.219	
1.91	0.092		1.91	0.181		1.91	0.299	
2.54	0.119	0.90	2.54	0.226	1.71	2.54	0.381	2.88
3.18	0.129		3.18	0.238		3.18	0.427	
3.81	0.145		3.81	0.266		3.81	0.482	
4.45	0.163		4.45	0.294		4.45	0.535	
5.08	0.178	0.89	5.08	0.332	1.66	5.08	0.564	2.82
7.62	0.241		7.62	0.52		7.62	0.718	
10.16	0.3		10.16	0.75		10.16	0.796	
12.70	0.332		12.70	0.791		12.70	0.812	



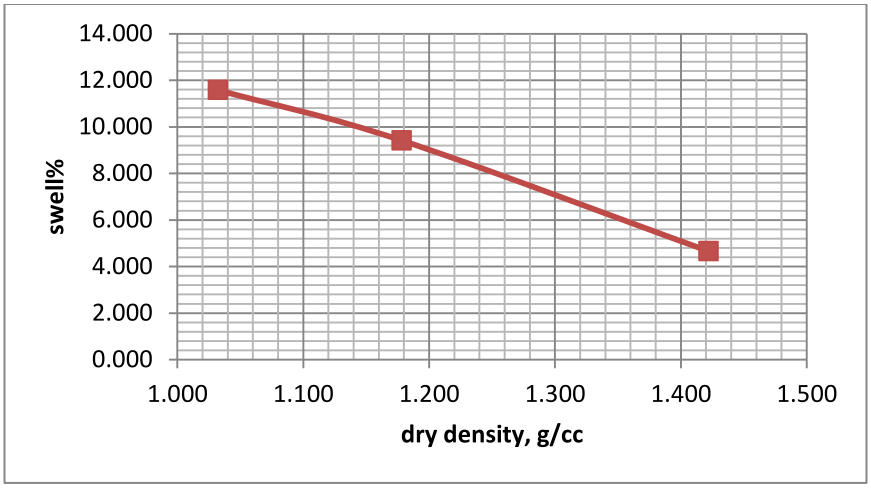




No.of blows	MCBS %	DDBS g/cm3	swell %	Corrected CBR %	% OF Compaction
65	24.3	1.422	4.656	2.88	95
30	26.3	1.178	9.416	1.70	79
10	26.2	1.032	11.581	0.90	69



CBR % at 95 % MDD	2.80	Swell %	4.87



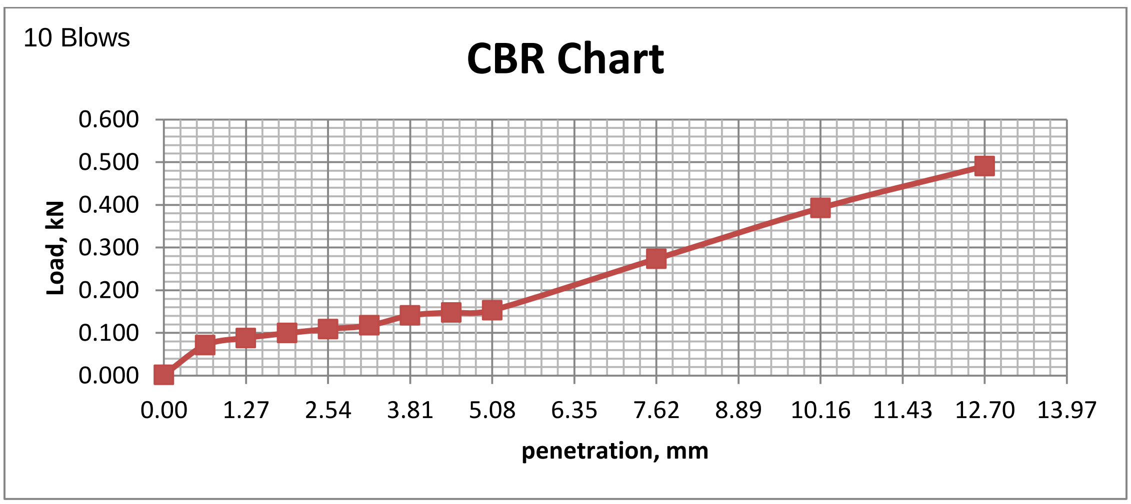
CBR Data of Natural Soil Trial Two

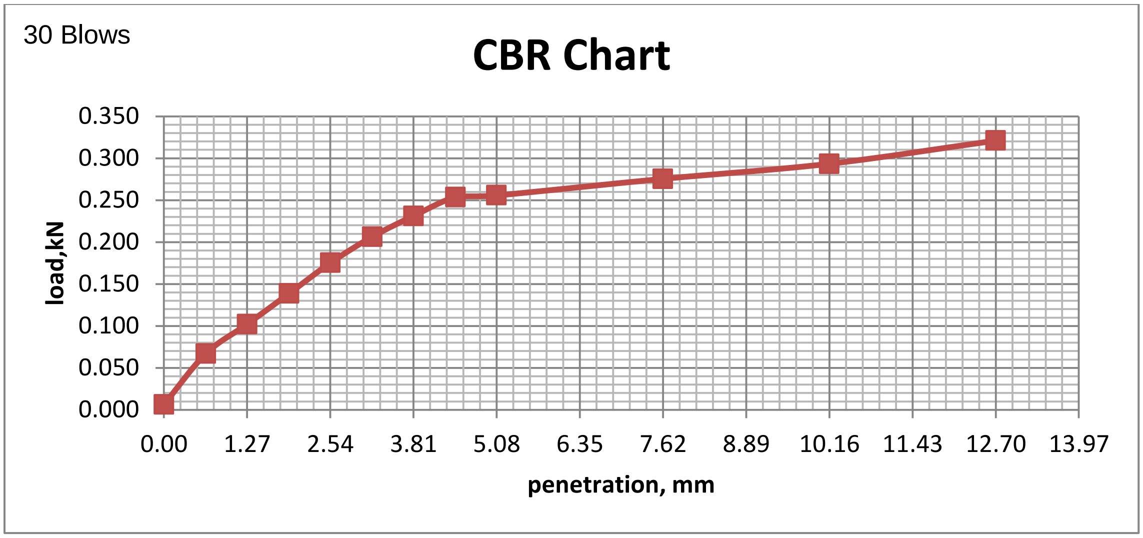
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould	13511.3	14080.6	13253.6	13947	12709.4	13715
Mass Mould	9330.5	9330.5	9444.7	9444.7	9313.6	9313.6
Mass of Soil	4180.8	4750.1	3808.9	4502.3	3395.8	4401.4
Volume of Mould	2285	2285	2285	2285	2285	2285
Wet density of soil	1.830	2.079	1.667	1.970	1.486	1.926
Dry density of soil	1.463	1.598	1.329	1.471	1.200	1.271

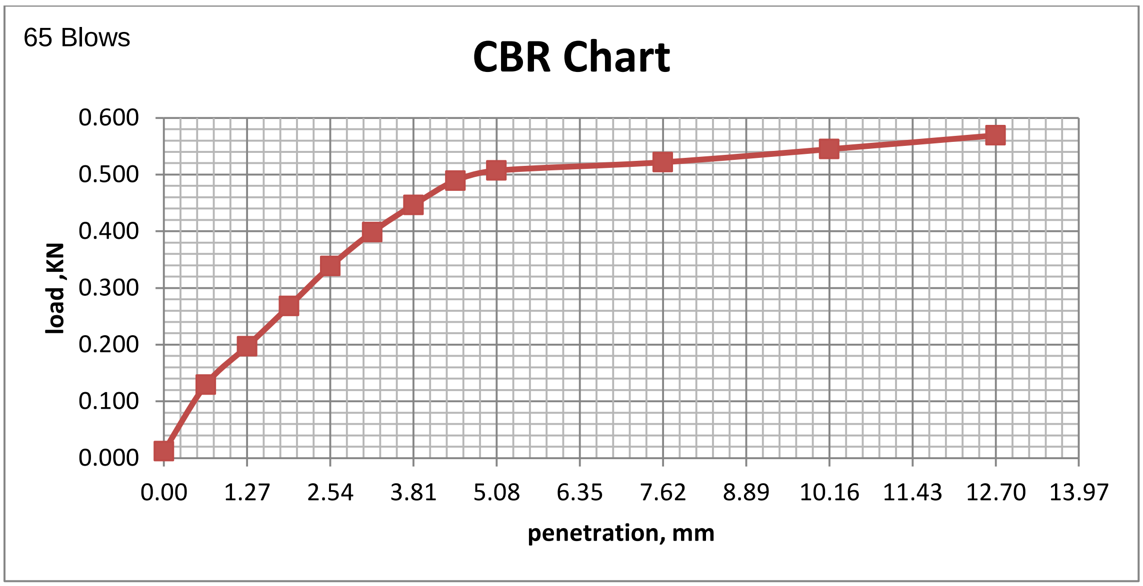
MOISTURE CONTENT DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	NM	GT-3	T-2D	K-4	10	2
Mass of wet soil + Container	138.6	137.8	134.5	141.4	166.1	171.9
Mass of dry soil + Container	114.1	110.0	110.8	110.1	139.4	119.6
Mass of container	16.8	17.5	17.7	17.8	27.4	18.2
Mass of water	24.4	27.8	23.7	31.3	26.7	52.3
Mass of dry soil	97.4	92.5	93.1	92.3	112.0	101.4
Moisture content	25.1	30.1	25.4	33.9	23.9	51.6

Average moisture content	25.1	30.1	25.4	33.9	23.9	51.6

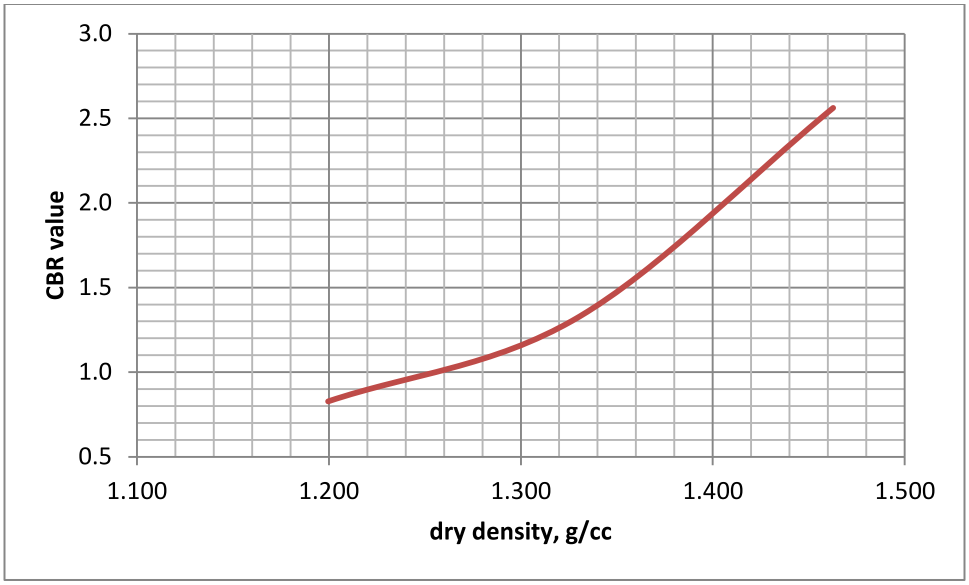
Penetration after 96 hrs Soaking Period			**Surcharge Weight:-4.55 KG**					
10 Blows			**30 Blows**			**65 Blows**		
Pen.mm	**Load, KN**	**CBR %**	**Pen.mm**	**Load, KN**	**CBR %**	**Pen.mm**	**Load, KN**	**CBR %**
0.00	0.002		0.00	0.006		0.00	0.012	
0.64	0.071		0.64	0.067		0.64	0.129	
1.27	0.088		1.27	0.102		1.27	0.197	
1.91	0.100		1.91	0.139		1.91	0.268	
2.54	0.109	0.83	2.54	0.175	1.33	2.54	0.338	2.56
3.18	0.118		3.18	0.206		3.18	0.398	
3.81	0.141		3.81	0.231		3.81	0.446	
4.45	0.148		4.45	0.253		4.45	0.489	
5.08	0.153	0.77	5.08	0.256	1.28	5.08	0.507	2.54
7.62	0.274		7.62	0.275		7.62	0.522	
10.16	0.393		10.16	0.293		10.16	0.545	
12.70	0.491		12.70	0.321		12.70	0.569	



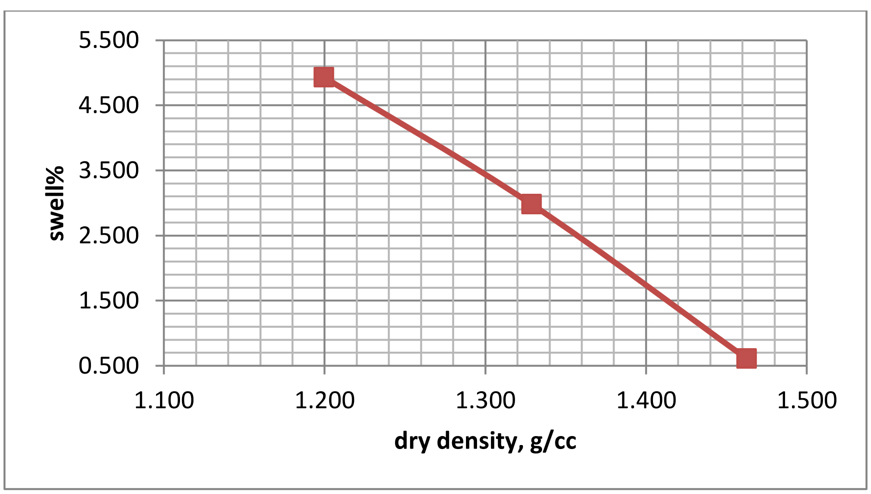




No.of blows	MCBS %	DDBS g/cm3	swell %	Corrected CBR %	% OF Compaction
65	25.1	1.463	4.931	2.6	99
30	25.4	1.329	9.854	1.3	90
10	23.9	1.200	15.601	0.8	81



CBR % at 95 % MDD	**2.00**	**Swell %**	**7.06**

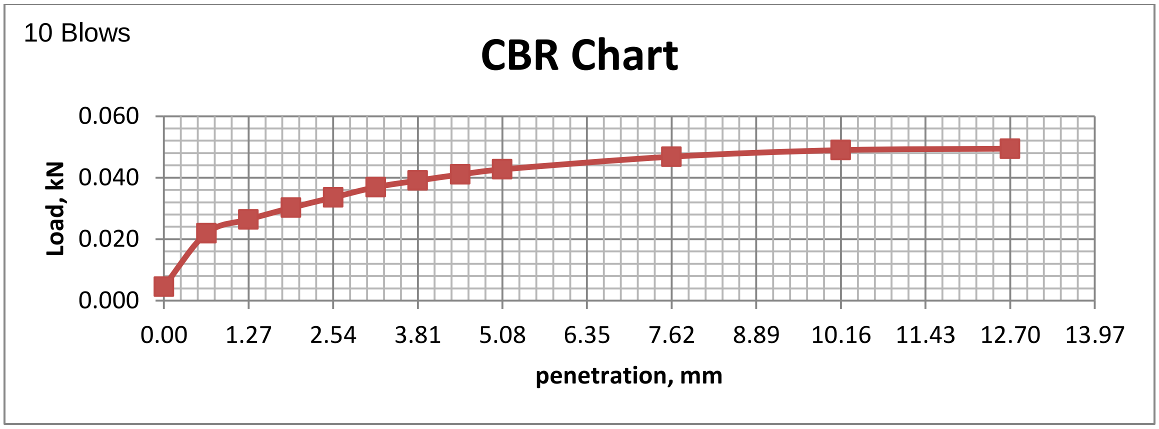


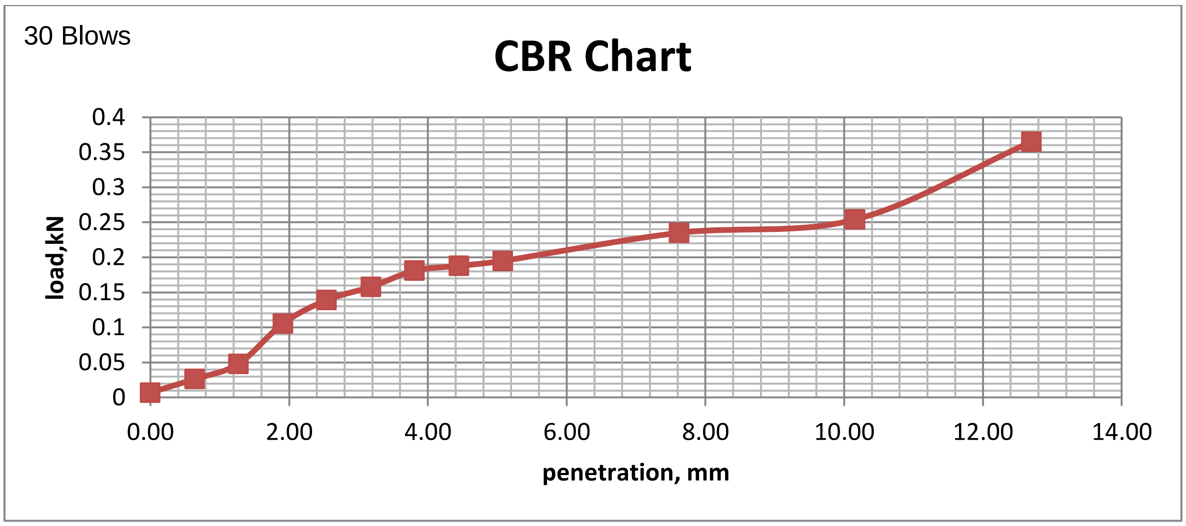
CBR Data of Natural Soil Trial Three

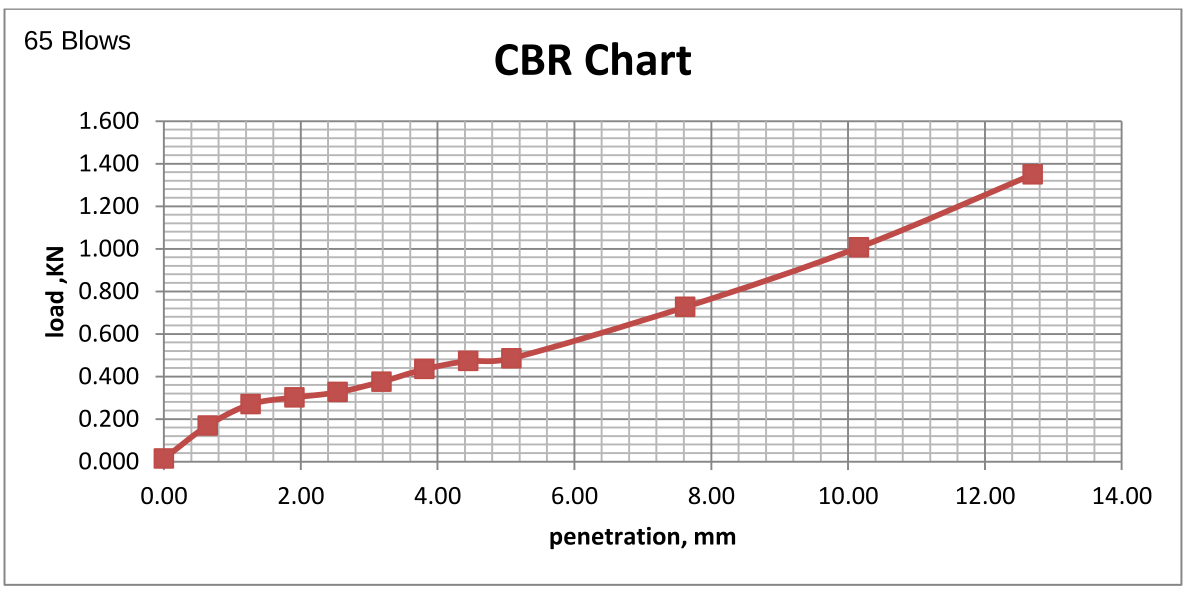
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould	13288.3	13910.1	13059.6	13716.4	12779.4	13752
Mass Mould	9159.9	9159.9	9471.5	9471.5	9338.5	9338.5
Mass of Soil	4128.4	4750.2	3588.1	4244.9	3440.9	4413.5
Volume of Mould	2285	2285	2285	2285	2285	2285
Wet density of soil	1.807	2.079	1.570	1.858	1.506	1.932
Dry density of soil	1.421	1.436	1.241	1.244	1.185	1.282

MOISTURE CONTENT DATA	65 Blows		30 Blows		10 Blows	
	Before soak	After soak	Before soak	After soak	Before soak	After soak
Container no.	H	D	3B	G-10	4B	SSB
Mass of wet soil + Container	115.1	193.2	122.1	155.9	142.5	178.5
Mass of dry soil + Container	94.5	142.7	100.3	110.1	117.7	124.2
Mass of container	18.7	29.7	18.2	17.2	26.2	17.2
Mass of water	20.6	50.6	21.8	45.8	24.8	54.3
Mass of dry soil	75.8	113.0	82.1	92.9	91.5	107.0
Moisture content	27.2	44.8	26.6	49.3	27.1	50.7
Average moisture content	27.2	44.8	26.6	49.3	27.1	50.7

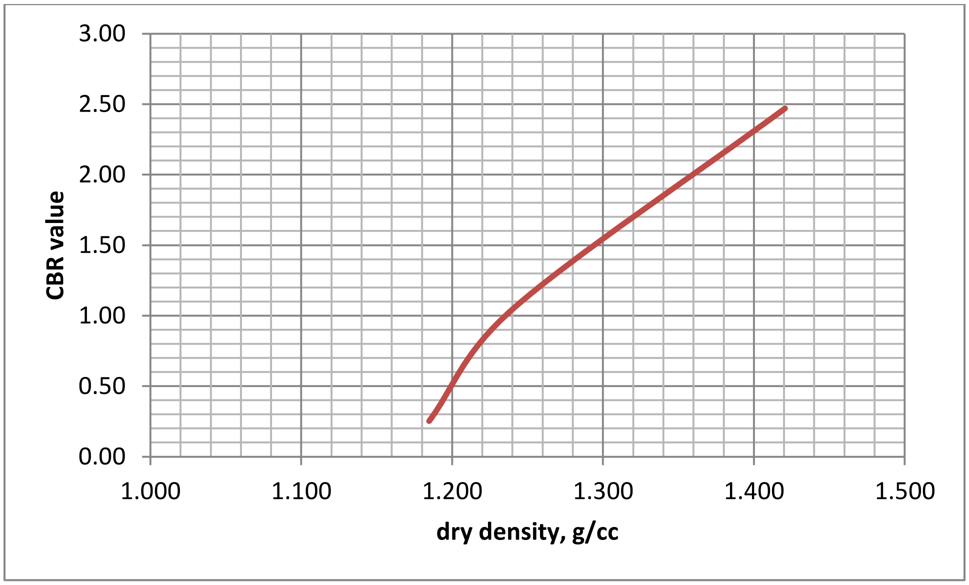
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG				
10 Blows			30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.005		0.00	0.007		0.00	0.014	
0.64	0.022		0.64	0.0265		0.64	0.169	
1.27	0.026		1.27	0.048		1.27	0.269	
1.91	0.030		1.91	0.106		1.91	0.301	
2.54	0.034	0.25	2.54	0.139	1.05	2.54	0.326	2.47
3.18	0.037		3.18	0.158		3.18	0.375	
3.81	0.039		3.81	0.181		3.81	0.434	
4.45	0.041		4.45	0.188		4.45	0.473	
5.08	0.043	0.21	5.08	0.195	0.98	5.08	0.485	2.43
7.62	0.047		7.62	0.235		7.62	0.726	
10.16	0.049		10.16	0.254		10.16	1.007	
12.70	0.049		12.70	0.365		12.70	1.349	


10 Blows
CBR Chart
Load, kN
0.060
0.040
0.020
0.000
0.00 1.27 2.54 3.81 5.08 6.35 7.62 8.89 10.16 11.43 12.70 13.97
penetration, mm

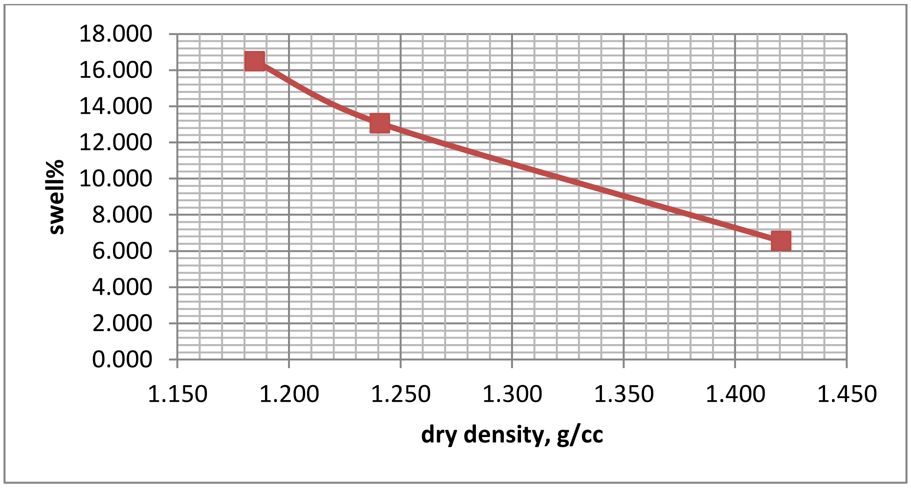

30 Blows
CBR Chart
load,kN
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0.00 2.00 4.00 6.00 8.00 10.00 12.00 14.00
penetration, mm


65 Blows
CBR Chart
load ,KN
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
0.00 2.00 4.00 6.00 8.00 10.00 12.00 14.00
penetration, mm

No.of blows	MCBS %	DDBS g/cm3	swell %	Corrected CBR %	% OF Compaction
65	27.2	1.421	6.546	2.47	96
30	26.6	1.241	13.058	1.05	84
10	27.1	1.185	16.478	0.25	80



CBR % at 95 % MDD	2.35	Swell %	6.83



Appendix B: - Details of One layer Reinforced Soil Laboratory Results

Compaction of One Layer Reinforced Soil

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Moisture content Determination				Average Moisture content,%	Dry unit weight (g/cm^3)
			Mass of can, g	Mass of wet soil + can, g	Mass of dry soil + can, g	Moisture content,%		
1	10239.6	1.59	25.3	233.2	195.2	22.37	22.31	1.30
			37.7	203.1	173	22.25		
2	10385.1	1.66	18	109.1	90.7	25.31	25.14	1.32
			25.5	163.1	135.6	24.98		
3	10650	1.77	17.9	110.2	90.5	27.13	28.46	1.38
			17.7	122.3	98.3	29.78		
4	10580.7	1.74	17.6	113.8	89.2	34.36	34.30	1.30
			17.8	106.4	83.8	34.24		
Mass of mould=6596.7g								

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm^3)	Average Moisture content,%	Dry unit weight (g/cm^3)
1	10239.6	1.59	22.31	1.30
2	10385.1	1.66	25.14	1.32
3	10650	1.77	28.46	1.38
4	10580.7	1.74	34.30	1.30
		OMC (%)		**28.2**
		MDD (g/cm^3)		**1.38**

Natural Moisture Content During Compaction			
Can	P-67	P-3	F
Mass of can(g)	35.5	36	36.4
mass of can + Soil (g)	192.7	206.8	210.2
Mass of can + dry Soil (g)	181.5	194.6	197.9
Moisture Content in (%)	7.12	7.14	7.08
Average	7.11		

New OMC for CBR

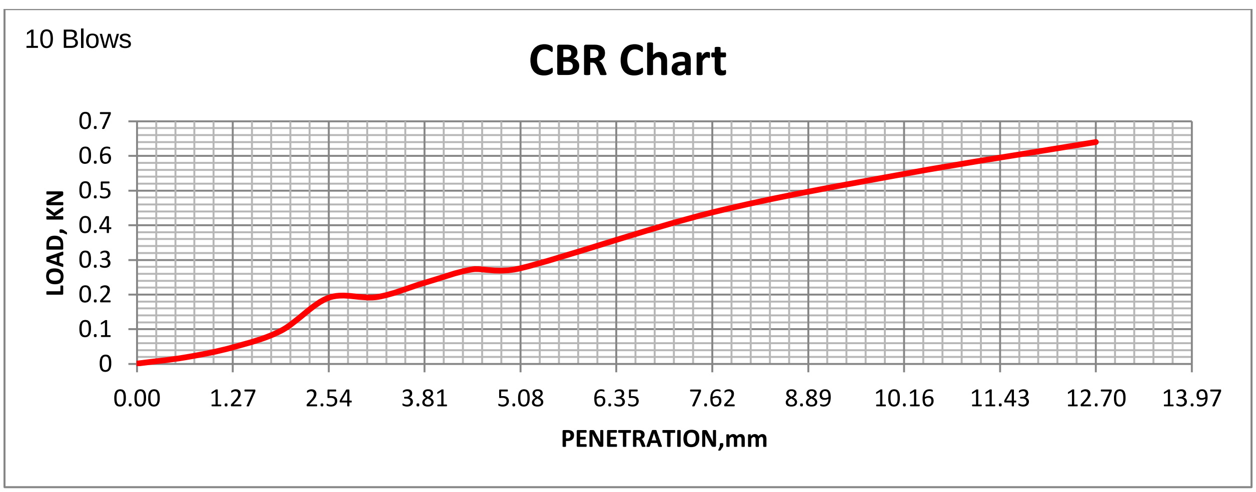
OMC	NMC	New OMC	Amount of H_2O
28.26	7.11	19.74	987.03

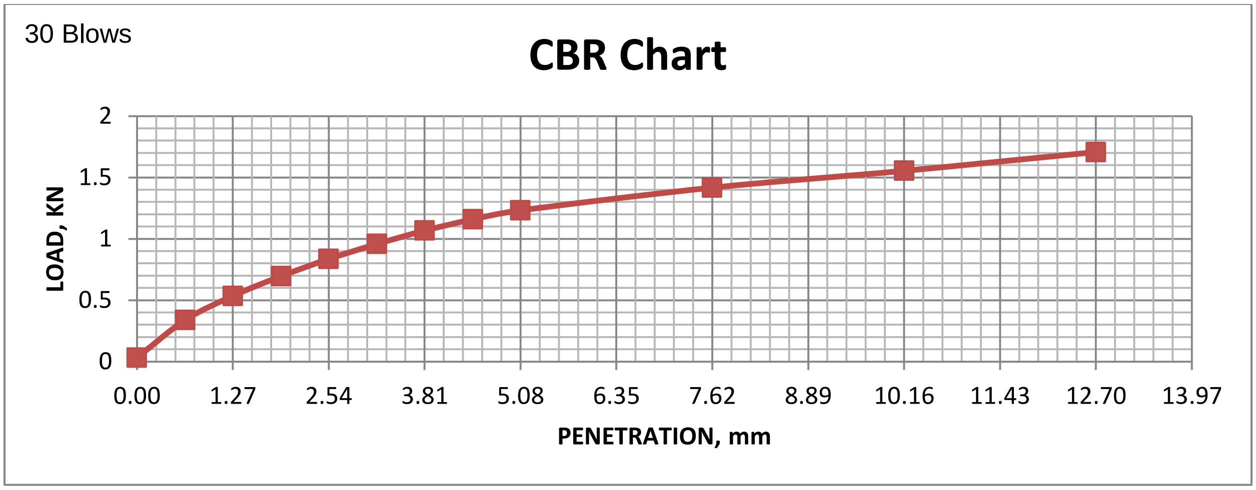
CBR Data of One Layer Reinforced Soil Trial One

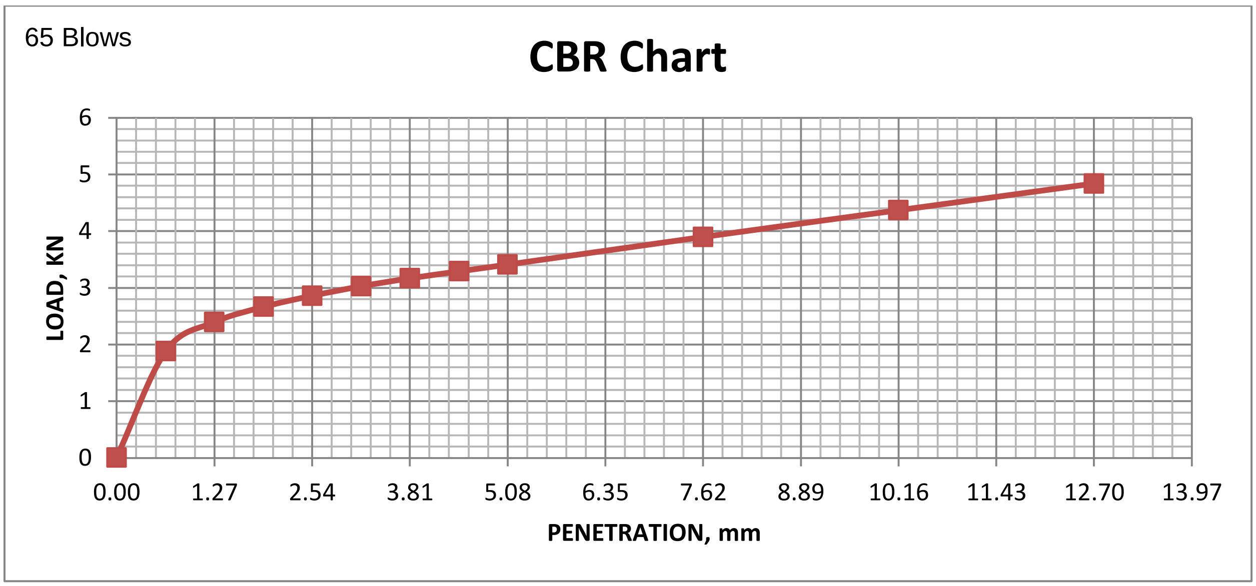
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate, g	13333.1	13538.4	13048.5	13376.9	12705.9	13194.3
Mass of Mould + Base Plate, g	9287.8	9287.8	9349.7	9349.7	9514.9	9514.9
Mass of Soil, g	4045.3	4250.6	3698.8	4027.2	3191	3679.4
Volume of Mould, cm3	2285	2285	2285	2285	2285	2285
Wet density of soil, g/cm3	1.770	1.860	1.619	1.762	1.396	1.610
Dry density of soil, g/cm3	1.366	1.501	1.247	1.408	1.090	1.119

MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak		**After soak**		**Before soak**		**After soak**		**Before soak**			**After soak**
Container no.	F	T-1	A-13	B-3	P-3	P-67	P-15	P-3	G-9	P-15	SSB	GT-3
Mass of wet soil + Container, g	202.0	171.7	197.0	121.3	155.4	166.1	169.8	168.7	179.6	175.3	191.5	246.4
Mass of dry soil + Container, g	163.9	141.3	167.7	100.2	128.3	135.8	141.1	143.2	144.4	143.8	136.1	179.9
Mass of container, g	36.2	37.7	36.3	17.4	35.9	35.5	33.3	35.7	25.2	25.4	17.5	17.6
Mass of water, g	38.1	30.4	29.3	21.1	27.1	30.3	28.7	25.5	35.2	31.5	55.4	66.5
Mass of dry soil, g	127.7	103.6	131.4	82.8	92.4	100.3	107.8	107.5	119.2	118.4	118.6	162.3
Moisture content, %	29.8	29.3	22.3	25.5	29.3	30.2	26.6	23.7	29.5	26.6	46.7	41.0
Average moisture content, %	29.6		23.9		29.8		25.2		28.1		43.8	

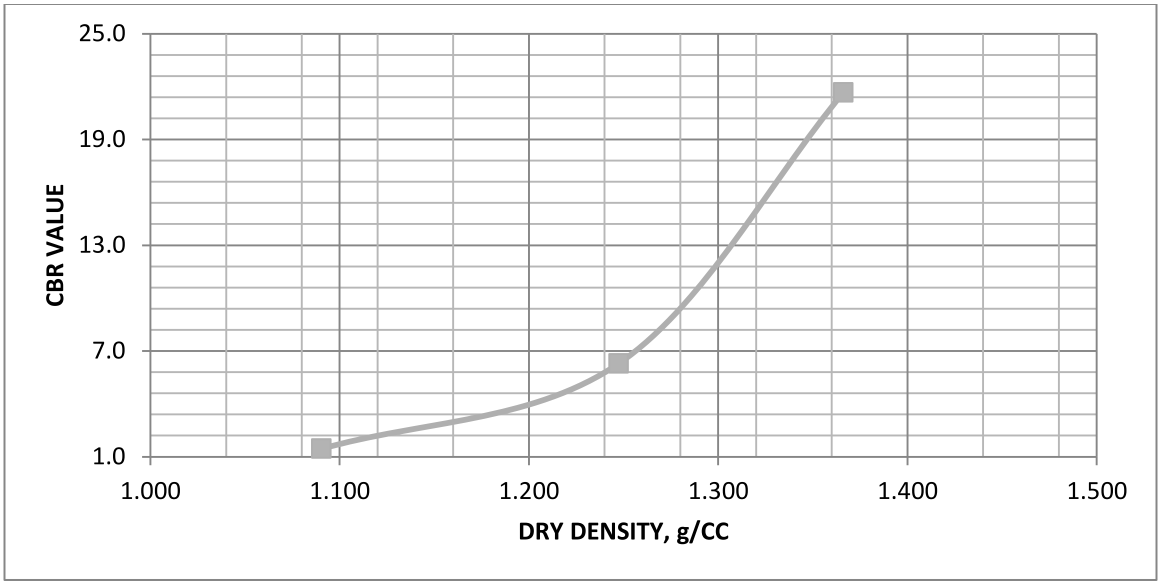
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG				
10 Blows			30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.001		0.00	0.032		0.00	0.008	
0.64	0.019		0.64	0.339		0.64	1.886	
1.27	0.048		1.27	0.535		1.27	2.395	
1.91	0.096		1.91	0.696		1.91	2.665	
2.54	0.191	1.45	2.54	0.836	6.33	2.54	2.86	21.67
3.18	0.193		3.18	0.958		3.18	3.028	
3.81	0.234		3.81	1.067		3.81	3.168	
4.45	0.273		4.45	1.159		4.45	3.291	
5.08	0.276	1.38	5.08	1.232	6.16	5.08	3.41	17.05
7.62	0.437		7.62	1.417		7.62	3.897	
10.16	0.548		10.16	1.554		10.16	4.369	
12.70	0.64		12.70	1.707		12.70	4.84	
Modified Max.Dry Density g/cc			1.380		OMC %		28.26	



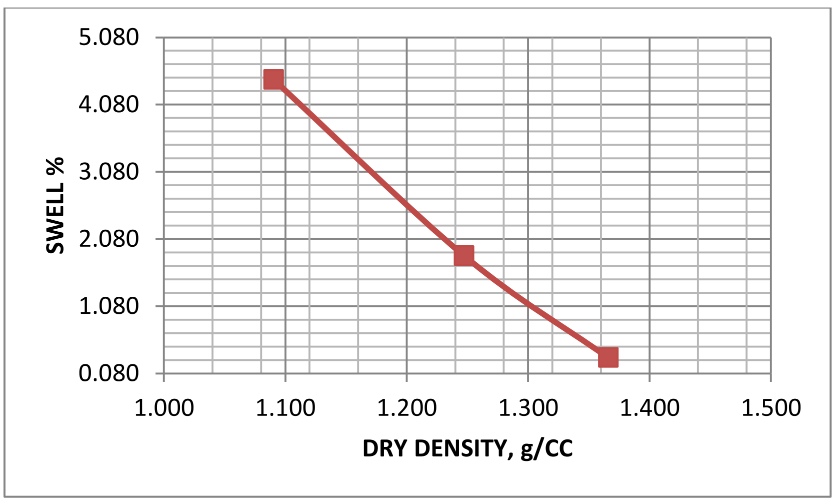




No.of blows	MCBS %	DDBS g/cm3	SWELL %	Correcrt CBR %	% OF Compaction
65	**29.6**	**1.366**	**0.312**	**21.7**	**99**
30	**29.8**	**1.247**	**1.830**	**6.3**	**90**
10	**28.1**	**1.090**	**4.450**	**1.4**	**79**



CBR % at 95 % MDD	13.60	Swell %	1.04

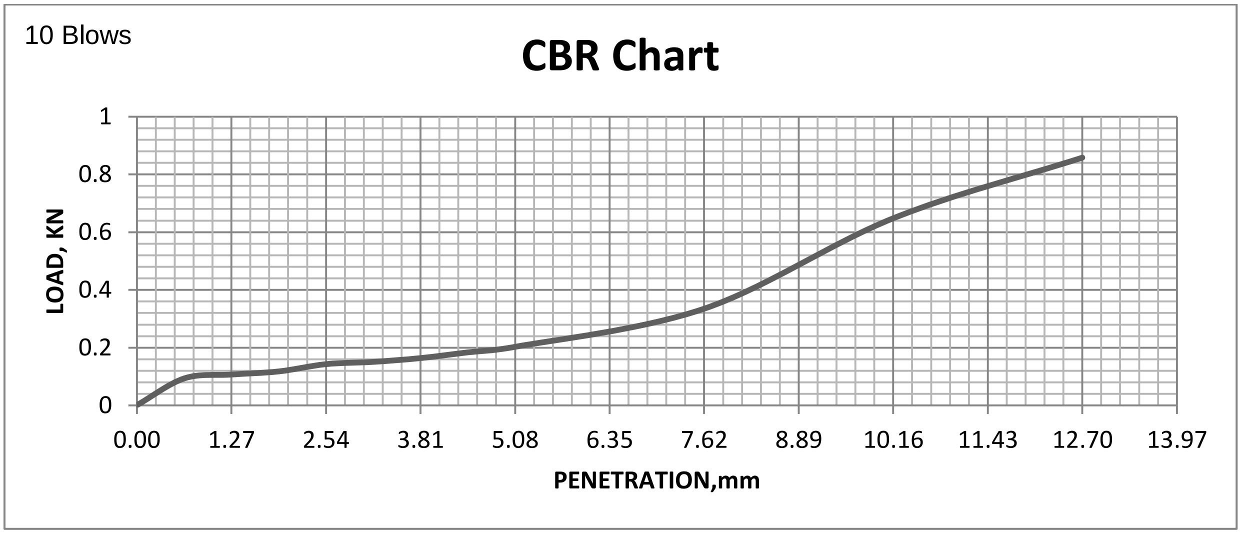


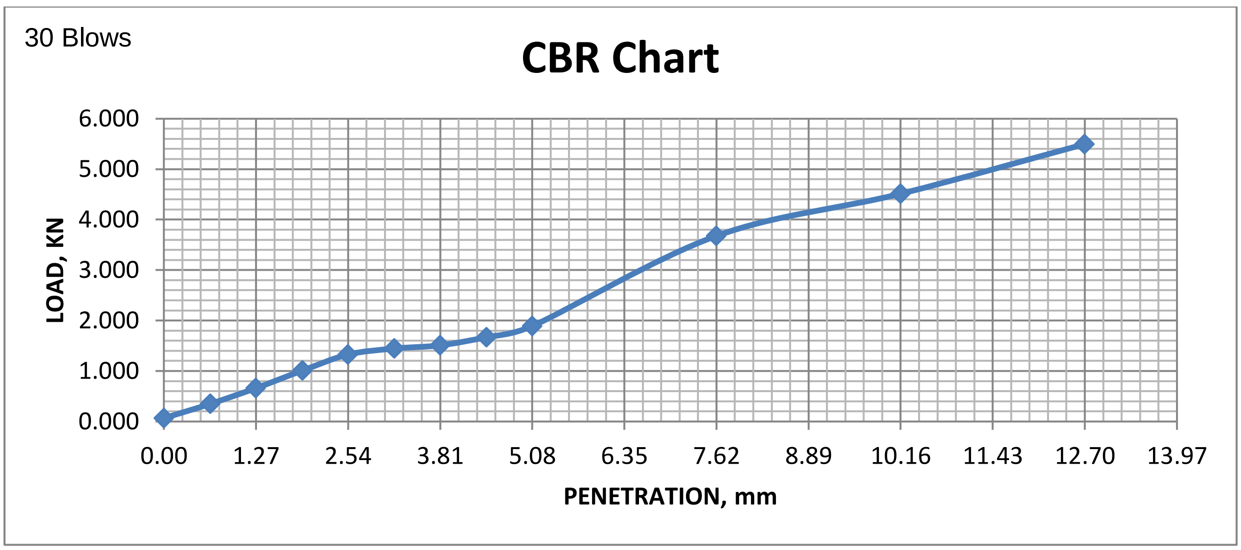
CBR Data of One Layer Reinforced Soil Trial Two

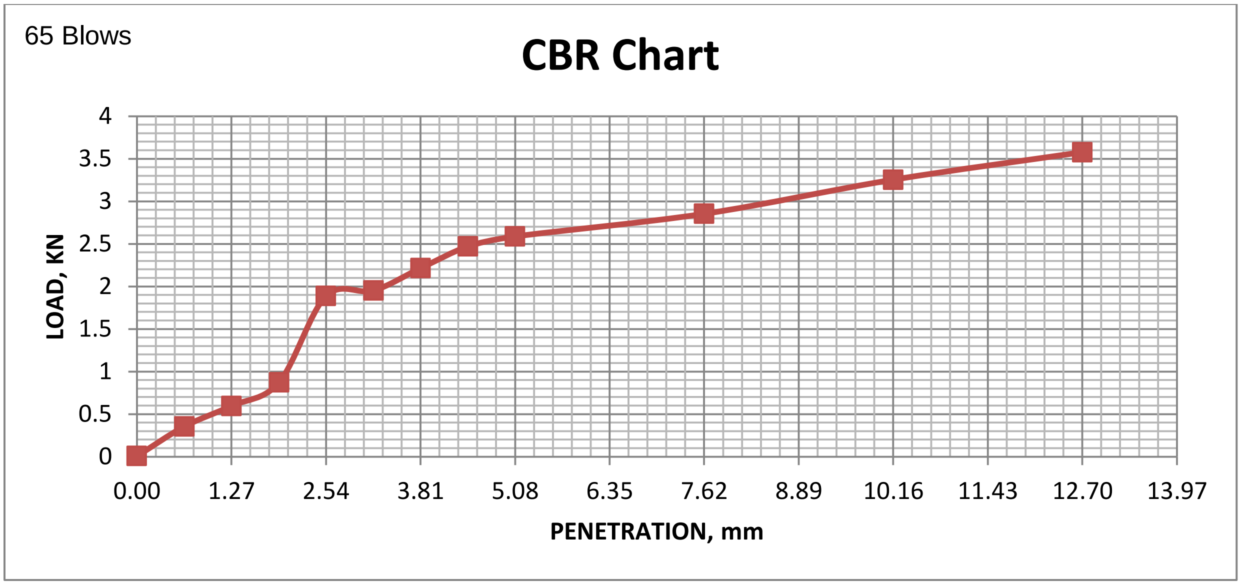
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before Soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate, g	13337.7	13863.1	13166.9	13810.3	12705.9	13194.3
Mass of Mould + Base Plate, g	9202.6	9202.6	9425.6	9425.6	9495.1	9495.1
Mass of Soil, g	4135.1	4660.5	3741.3	4384.7	3210.8	3699.2
Volume of Mould, cm3	2285	2285	2285	2285	2285	2285
Wet density of soil, g/cm3	1.810	2.040	1.637	1.919	1.405	1.619
Dry density of soil, g/cm3	1.418	1.365	1.325	1.276	1.106	1.035

MOISTURE CONTENT DATA	65 Blows			30 Blows			10 Blows		
	Before soak	After soak		Before soak	After soak		Before soak	After soak	
Container no.	4A	2	K-4	5S	T4C2	T1C1	3A	3A	B3
Mass of wet soil + Container, g	138.6	137.0	142.0	107.2	170.5	129.1	111.0	180.6	188.5
Mass of dry soil + Container, g	112.4	98.5	100.3	90.2	112.1	97.5	91.2	125.0	123.7
Mass of container, g	17.4	18.5	17.9	18.1	16.8	17.6	18.1	17.5	17.6
Mass of water, g	26.2	38.5	41.7	17.0	58.4	31.6	19.8	55.6	64.8
Mass of dry soil, g	95.0	80.0	82.4	72.1	95.3	79.9	73.1	107.5	106.1
Moisture content, %	27.6	48.2	50.6	23.6	61.2	39.6	27.1	51.7	61.1
Average moisture content, %	27.6	49.4		23.6	50.4		27.1	56.4	

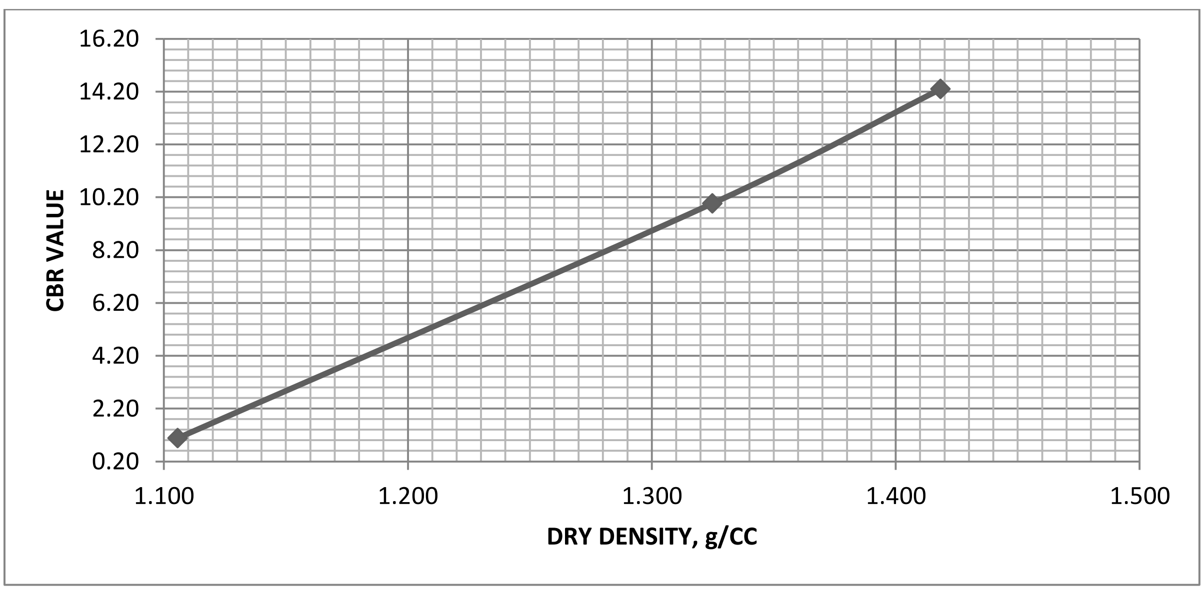
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG				
10 Blows			30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.001		0.00	0.066		0.00	0.006	
0.64	0.094		0.64	0.349		0.64	0.355	
1.27	0.107		1.27	0.661		1.27	0.595	
1.91	0.118		1.91	1.009		1.91	0.873	
2.54	0.143	1.08	2.54	1.326	10.04	2.54	1.887	14.30
3.18	0.151		3.18	1.445		3.18	1.952	
3.81	0.164		3.81	1.509		3.81	2.214	
4.45	0.184		4.45	1.671		4.45	2.471	
5.08	0.203	1.02	5.08	1.891	9.46	5.08	2.587	12.94
7.62	0.335		7.62	3.676		7.62	2.853	
10.16	0.648		10.16	4.511		10.16	3.254	
12.70	0.858		12.70	5.495		12.70	3.577	
Modified Max.Dry Density g/cc			1.380		OMC %		28.26	


10 Blows
CBR Chart
LOAD, KN
1
0.8
0.6
0.4
0.2
0
0.00
1.27
2.54
3.81
5.08
6.35
7.62
8.89
10.16
11.43
12.70
13.97
PENETRATION,mm

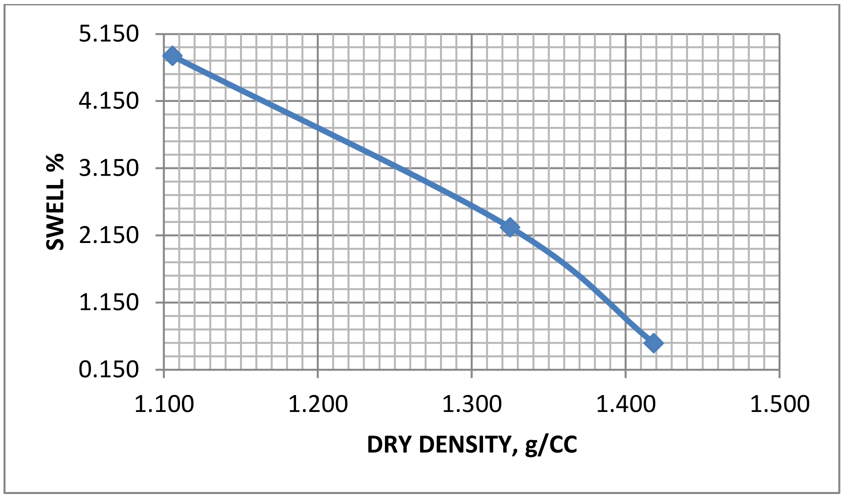

30 Blows
CBR Chart
LOAD, KN
6.000
5.000
4.000
3.000
2.000
1.000
0.000
0.00
1.27
2.54
3.81
5.08
6.35
7.62
8.89
10.16
11.43
12.70
13.97
PENETRATION, mm



No.of blows	MCBS %	DDBS g/cm3	SWELL %	Correcrt CBR %	% OF Compaction
65	**27.6**	**1.418**	**0.541**	**14.30**	**103**
30	**50.4**	**1.325**	**2.268**	**9.97**	**96**
10	**0.0**	**1.106**	**4.820**	**1.08**	**80**



CBR % at 95 % MDD	**9.91**	**Swell %**	**2.45**

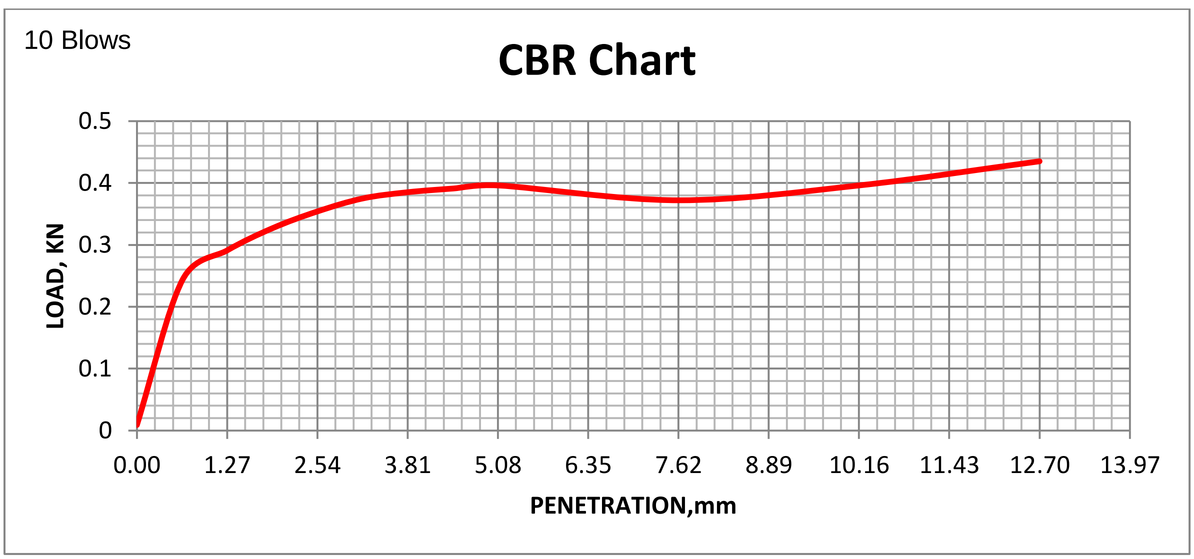


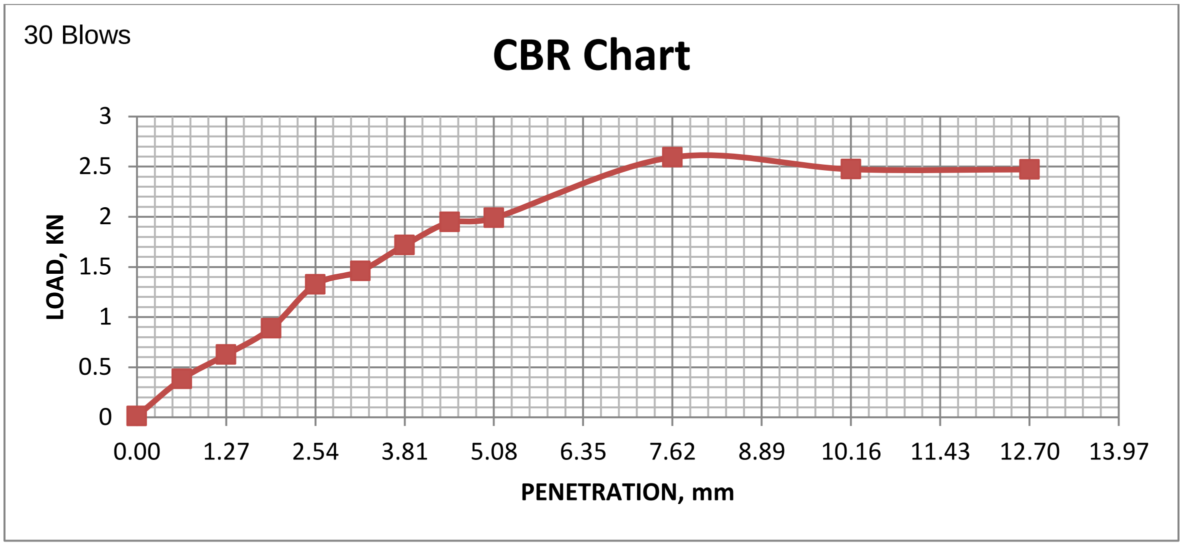
CBR Data of One Layer Reinforced Soil Trial Three

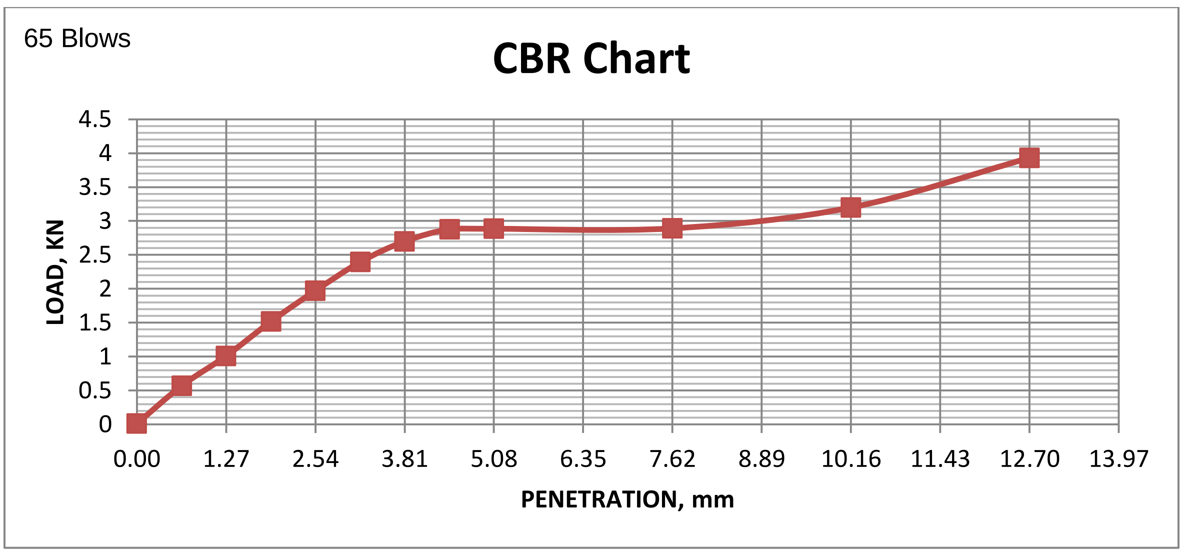
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before Soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate, g	13505.4	14052.3	13434.9	14119.6	12848.8	13619.3
Mass of Mould + Base Plate, g	9371.7	9371.7	9548.5	9548.5	9468.6	9468.6
Mass of Soil, g	4133.7	4680.6	3886.4	4571.1	3380.2	4150.7
Volume of Mould, cm3	2285	2285	2285	2285	2285	2285
Wet density of soil, g/cm3	1.809	2.048	1.701	2.000	1.479	1.816
Dry density of soil, g/cm3	1.440	1.454	1.343	1.427	1.165	1.226

MOISTURE CONTENT DATA	65 Blows			30 Blows			10 Blows		
	Before soak	After soak		Before soak	After soak		Before soak	After soak	
Container no.	2B	A3	A-14	A	F	C-10	G-10	B	DH
Mass of wet soil + Container, g	155.8	264.5	185.6	121.0	193.0	241.3	120.4	220.8	179.1
Mass of dry soil + Container, g	129.7	185.1	151.5	99.4	158.3	170.9	98.5	158.7	126.9
Mass of container, g	27.8	38.0	28.9	18.4	36.6	35.5	17.2	31.3	16.9
Mass of water, g	26.1	79.5	34.1	21.6	34.7	70.4	21.9	62.1	52.3
Mass of dry soil, g	101.9	147.0	122.6	81.0	121.7	135.4	81.3	127.4	110.0
Moisture content, %	25.6	54.0	27.8	26.7	28.5	52.0	26.9	48.8	47.6
Average moisture content, %	25.6	40.9		26.7	40.2		26.9	48.2	

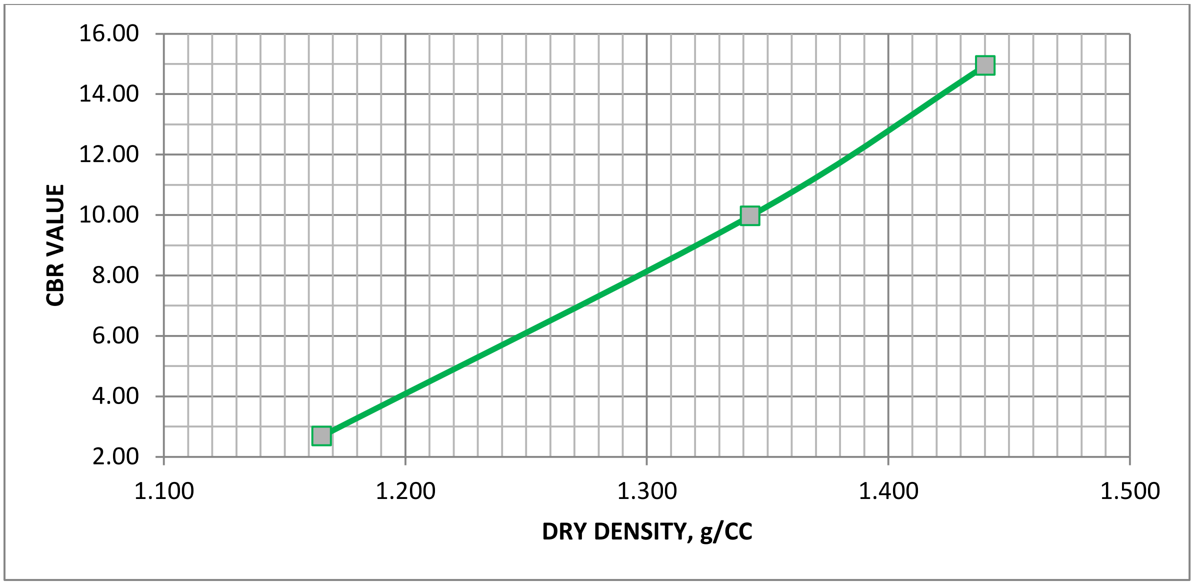
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG				
10 Blows			30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.009		0.00	0.013		0.00	0.011	
0.64	0.243		0.64	0.382		0.64	0.571	
1.27	0.291		1.27	0.624		1.27	1.009	
1.91	0.327		1.91	0.888		1.91	1.52	
2.54	0.354	2.68	2.54	1.325	10.04	2.54	1.972	14.94
3.18	0.375		3.18	1.459		3.18	2.396	
3.81	0.385		3.81	1.717		3.81	2.699	
4.45	0.391		4.45	1.946		4.45	2.877	
5.08	0.396	1.98	5.08	1.987	9.94	5.08	2.885	14.43
7.62	0.372		7.62	2.593		7.62	2.891	
10.16	0.396		10.16	2.473		10.16	3.199	
12.70	0.435		12.70	2.471		12.70	3.93	
Modified Max.Dry Density g/cc			1.480		OMC %		24	



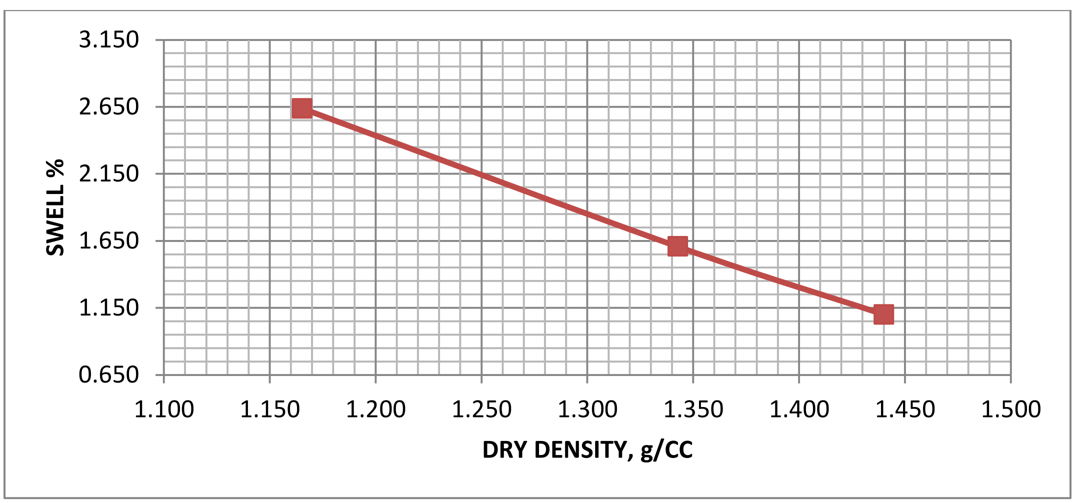




No.of blows	MCBS %	DDBS g/cm3	SWELL %	Correcrt CBR %	% OF Compaction
65	25.6	1.440	1.100	14.94	97
30	40.2	1.343	1.607	9.96	91
10	0.0	1.165	2.637	2.68	79



CBR % at 95 % MDD	**13.21**	**Swell %**	**1.27**



Appendix C: - Details of Two layer Reinforced Soil Laboratory Results

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm³)	Moisture content Determination				Average Moisture content,%	Dry unit weight (g/cm³)
			Mass of can, g	Mass of wet soil + can, g	Mass of dry soil + can, g	Moisture content,%		
1	10579.5	1.72	41.2	174.6	147.7	25.26	24.61	1.38
			36.5	174.6	147.9	23.97		
2	10795.8	1.82	35.9	162.8	135.7	27.15	28.22	1.42
			37.6	180.2	147.9	29.28		
3	10568.1	1.72	25.3	155	123.1	32.62	32.90	1.29
			34.6	160.2	128.9	33.19		
Mass of mould=6642.5g								

Trial No.	Mass of compacted soil + mold, g	Wet unit weight (g/cm³)	Average Moisture content,%	Dry unit weight (g/cm³)
1	10579.5	1.72	24.61	1.38
2	10795.8	1.82	28.22	1.42
3	10568.1	1.72	32.90	1.29
		OMC (%)		**28.12**
		MDD (g/cm³)		**1.42**

Natural Moisture Content During Compaction			
Can	J-41	SSB-1	K-4
Mass of can(g)	32.7	17.4	17.8
mass of can + Soil (g)	205.7	128	118.5
Mass of can + dry Soil (g)	196.1	121.9	113.1
Moisture Content in (%)	5.55	5.52	5.36
Average	5.48		

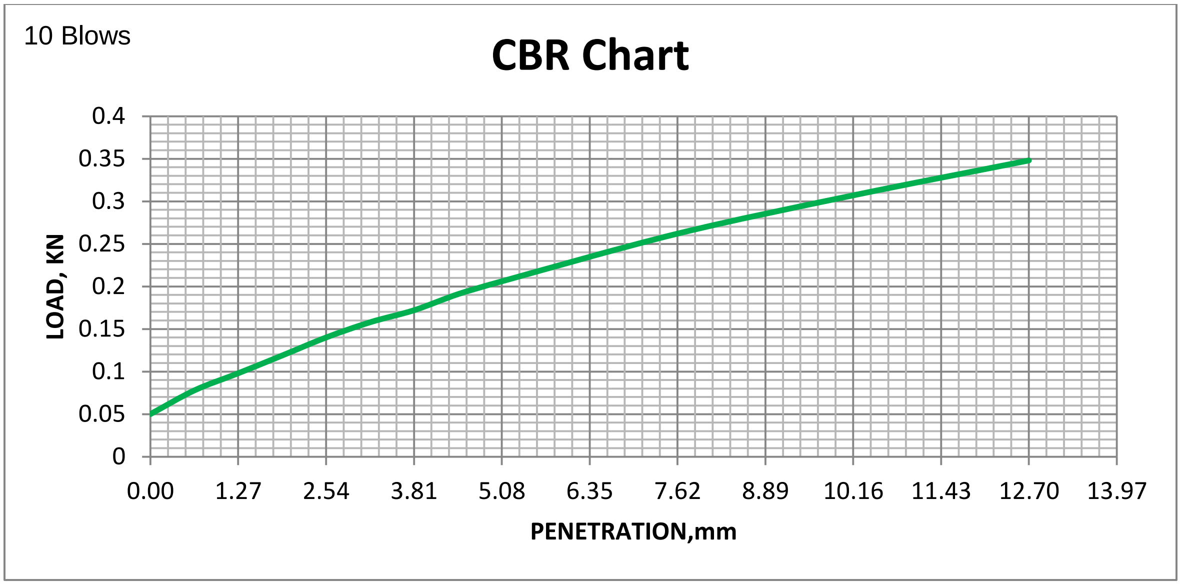
New OMC for CBR

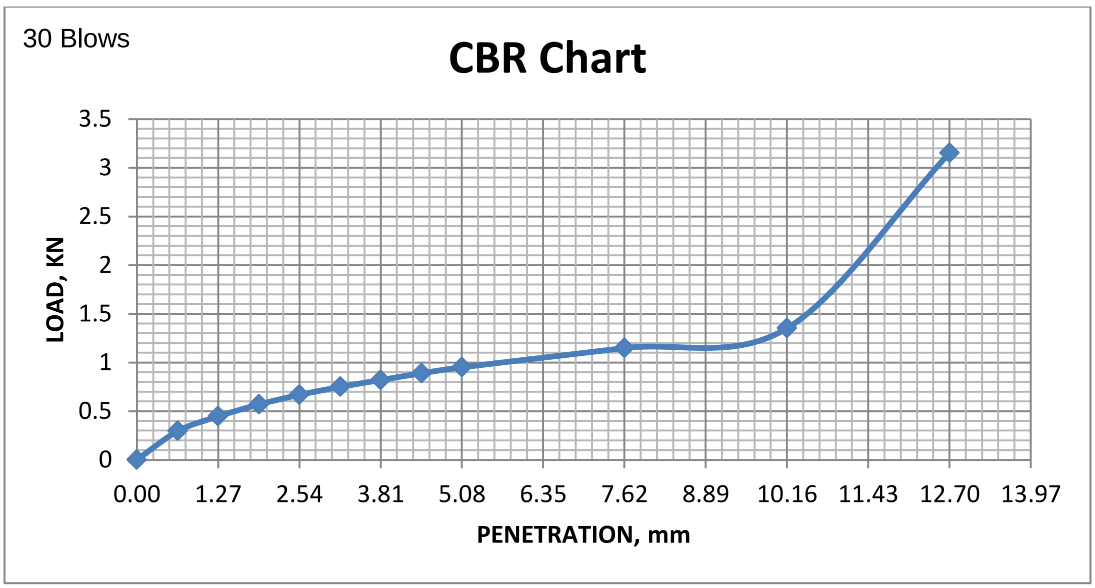
OMC	NMC	New OMC	Amount of H_2O
28.12	5.48	21.47	1073.44

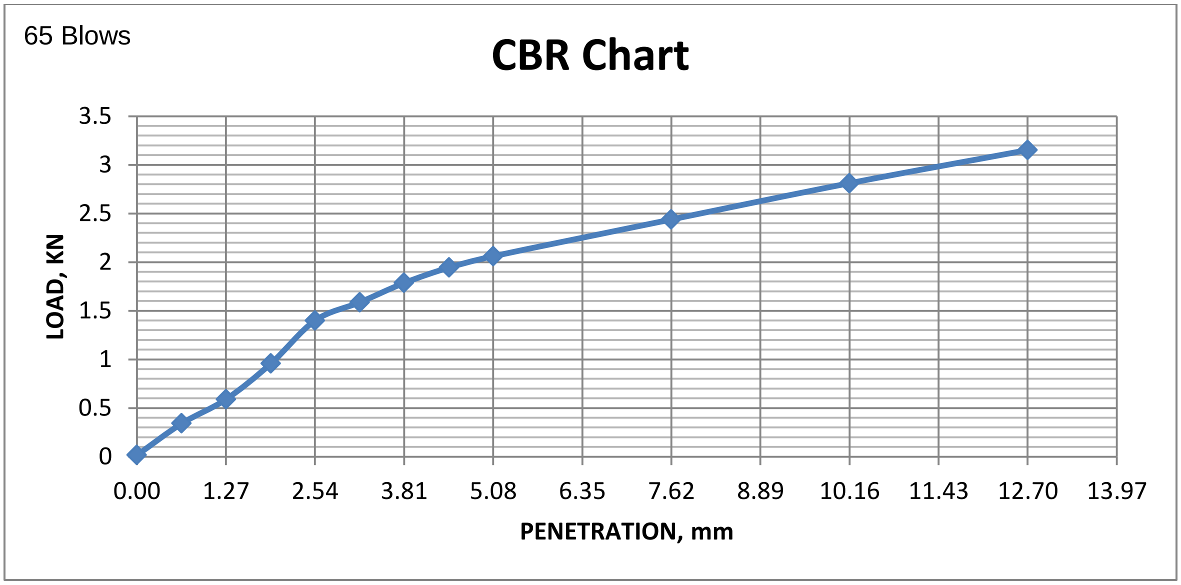
CBR Data of Two Layer Reinforced Soil Trial One

COMPACTION DATA	**65 Blows**		**30 Blows**		**10 Blows**	
	Before Soak	**After soak**	**Before soak**	**After soak**	**Before soak**	**After soak**
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate, g	13508.7	13845.9	12952.5	13401	12466.9	13093.3
Mass of Mould + Base Plate, g	9482	9482	9321.2	9321.2	9187.7	9187.7
Mass of Soil, g	4026.7	4363.9	3631.3	4079.8	3279.2	3905.6
Volume of Mould, cm3	2285	2285	2285	2285	2285	2285
Wet density of soil,g/cm3	1.762	1.910	1.589	1.785	1.435	1.709
Dry density of soil, g/cm3	1.370	1.413	1.284	1.261	1.130	1.166

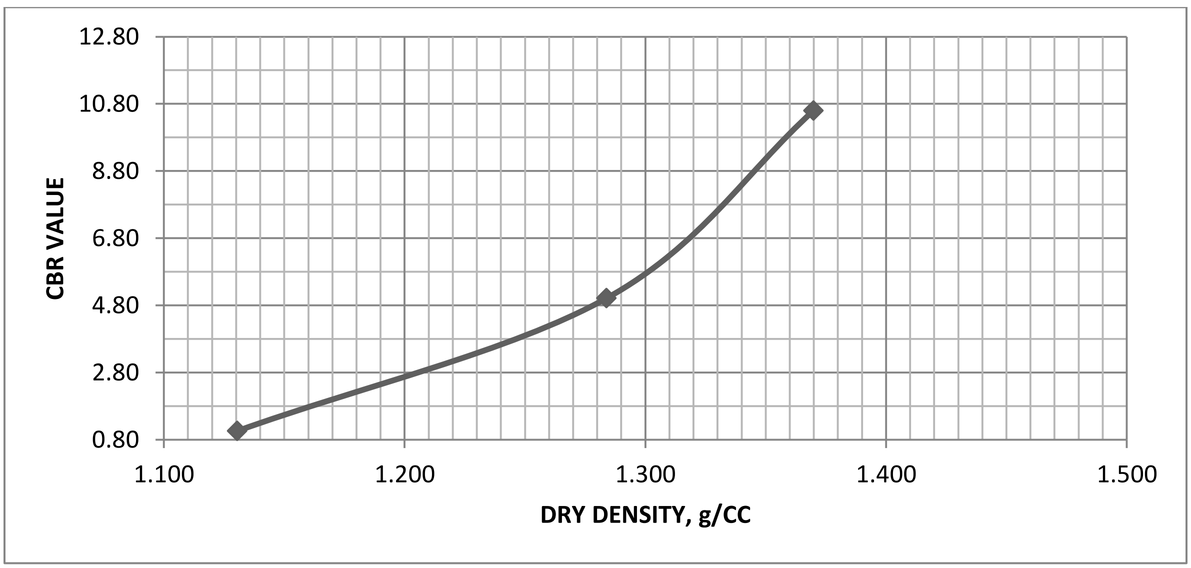
MOISTURE CONTENT DATA	**65 Blows**				**30 Blows**				**10 Blows**			
	Before soak	**After soak**			**Before soak**	**After soak**			**Before soak**	**After soak**		
Container no.	P-65	G	F	C	Z	G-19	9	GT-3	G-90	6	C-10	#A
Mass of wet soil + Container	194.9	107.8	154.7	162.8	193.4	154.6	195.7	193.8	188.9	184.3	199.1	174.7
Mass of dry soil + Container	159.9	82.5	123.7	129.2	162.9	117.7	146.6	151.0	156.0	133.5	143.5	130.0
Mass of container	37.7	9.7	36.3	32.8	34.6	35.9	32.4	34.7	33.9	27.6	27.7	27.5
Mass of water	35.0	25.3	31.0	33.6	30.5	36.9	49.1	42.8	32.9	50.8	55.6	44.7
Mass of dry soil	122.2	72.8	87.4	96.4	128.3	81.8	114.2	116.3	122.1	105.9	115.8	102.5
Moisture content	28.6	34.8	35.5	34.9	23.8	45.1	43.0	36.8	26.9	48.0	48.0	43.6
Average moisture content	28.6		35.2		23.8		41.6		26.9		46.5	


10 Blows
CBR Chart
LOAD, KN
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0.00 1.27 2.54 3.81 5.08 6.35 7.62 8.89 10.16 11.43 12.70 13.97
PENETRATION,mm

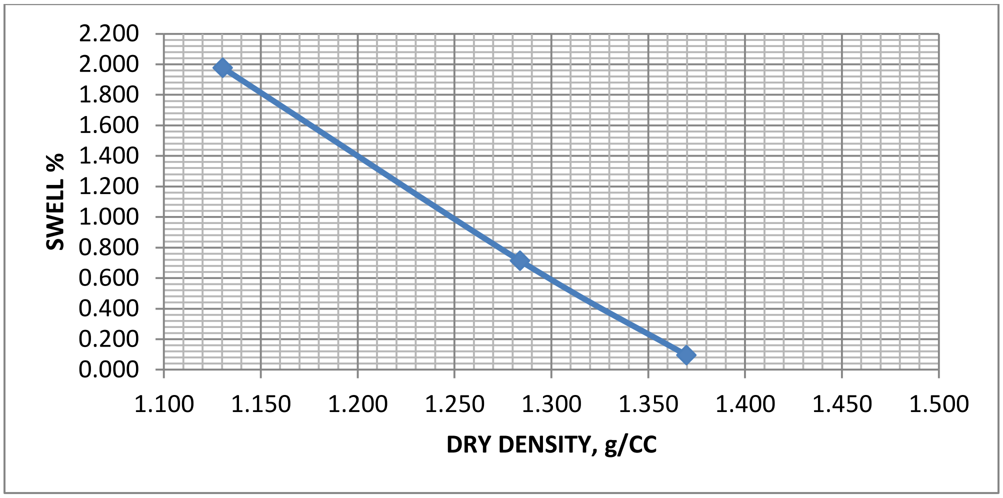

30 Blows
CBR Chart
LOAD, KN
3.5
3
2.5
2
1.5
1
0.5
0
0.00 1.27 2.54 3.81 5.08 6.35 7.62 8.89 10.16 11.43 12.70 13.97
PENETRATION, mm



No.of blows	MCBS %	DDBS g/cm3	SWELL %	Corrected CBR %	% OF Compaction
65	28.6	1.370	0.095	10.60	99
30	23.8	1.284	0.713	5.02	92
10	26.9	1.130	1.976	1.06	81



CBR % at 95 % MDD	6.80	Swell %	0.46

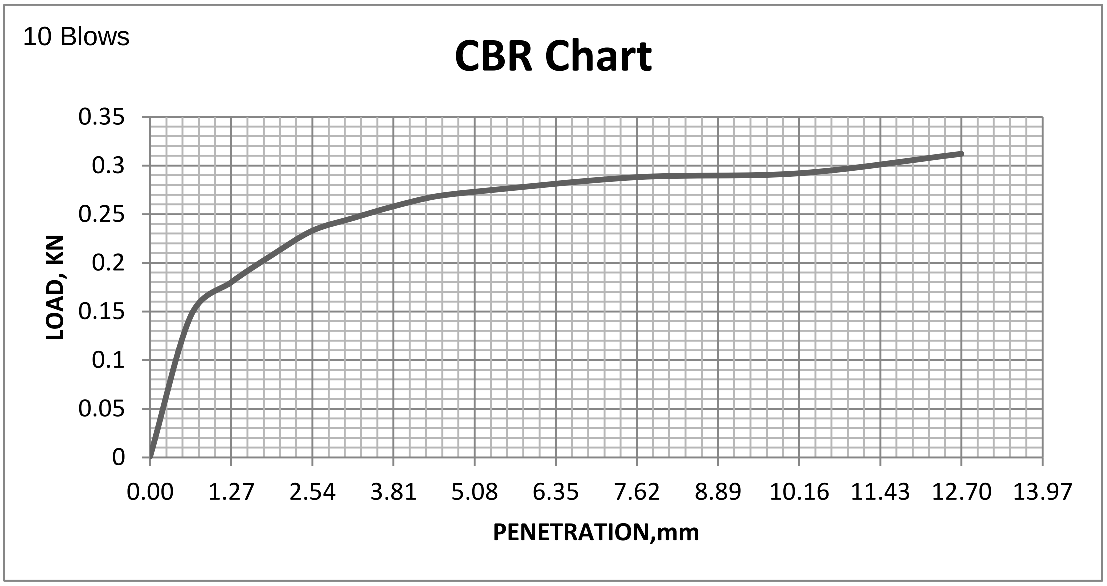


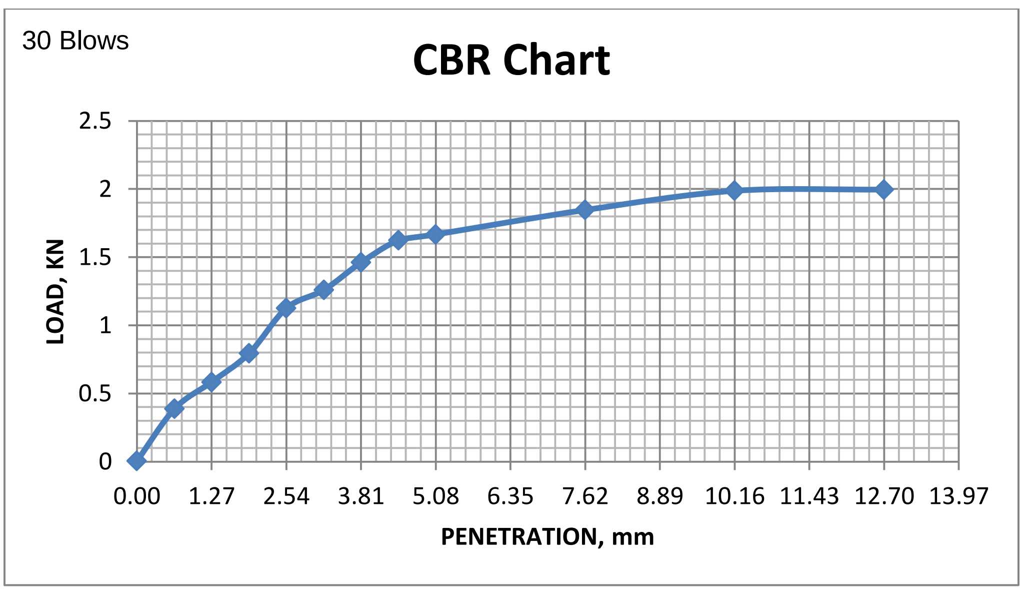
CBR Data of Two Layer Reinforced Soil Trial Two

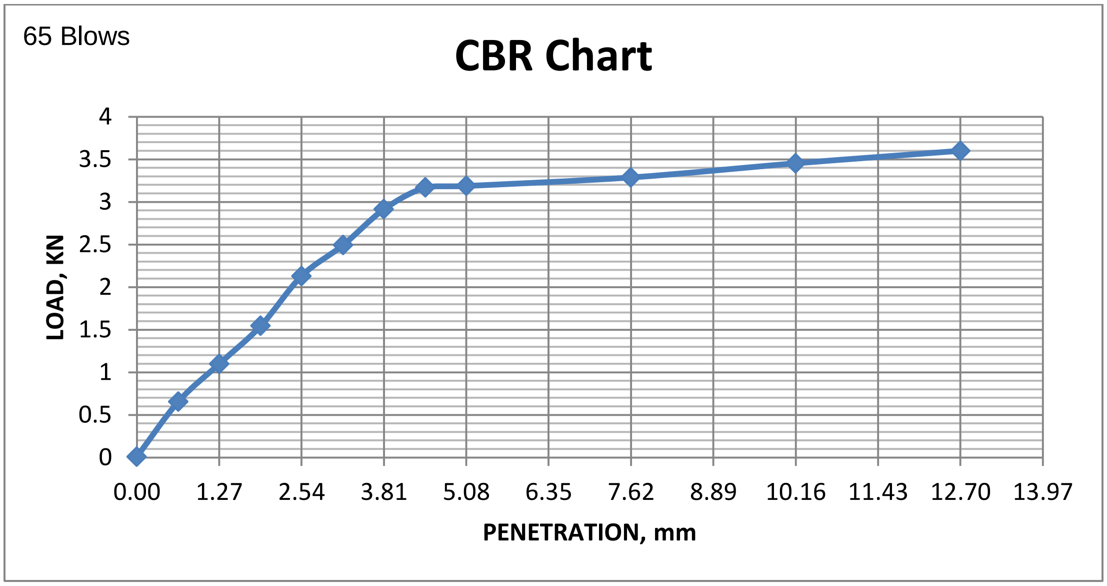
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before Soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate, g	13505.1	14076.3	13302.5	13950.1	12720.5	13679
Mass of Mould + Base Plate, g	9413.4	9413.4	9353.6	9353.6	9347.6	9347.6
Mass of Soil, g	4091.7	4662.9	3948.9	4596.5	3372.9	4331.4
Volume of Mould, cm3	2285	2285	2285	2285	2285	2285
Wet density of soil, g/cm3	1.791	2.041	1.728	2.012	1.476	1.896
Dry density of soil, g/cm3	1.445	1.485	1.389	1.459	1.188	1.258

MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak	After soak			Before soak	After soak			Before soak	After soak		
Container no.	F	D	A3	A-14	A-14	2	K-4	SSB	A-3	F	C-10	T4C2
Mass of wet soil + Container	202.2	197.9	187.4	165.2	182.5	154.0	143.4	160.7	184.1	197.7	229.4	163.3
Mass of dry soil + Container	170.2	156.3	155.0	118.1	152.3	120.9	117.3	109.7	154.5	145.4	165.6	111.3
Mass of container	36.5	29.7	33.1	28.8	28.5	18.5	17.9	17.2	32.7	36.5	35.5	16.8
Mass of water	32.0	41.7	32.4	47.2	30.2	33.1	26.1	50.9	29.6	52.2	63.7	52.1
Mass of dry soil	133.7	126.6	121.9	89.3	123.8	102.4	99.4	92.6	121.8	108.9	130.1	94.5
Moisture content	23.9	32.9	26.5	52.8	24.4	32.3	26.3	55.0	24.3	48.0	49.0	55.1
Average moisture content	23.9	37.4			24.4	37.9			24.3	50.7		

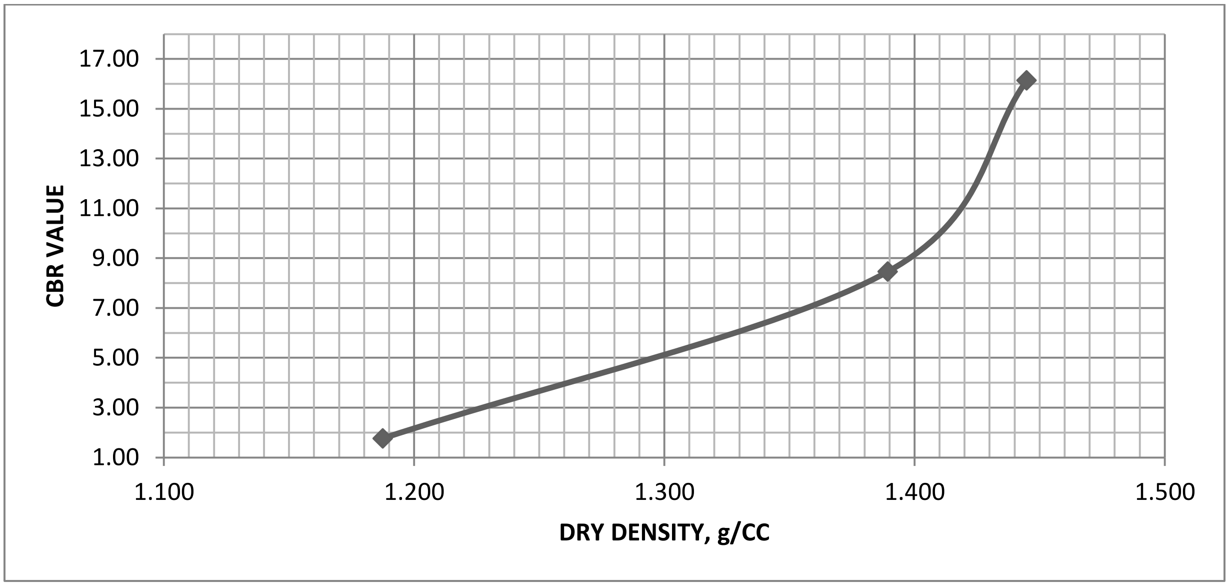
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG					
10 Blows				30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %		Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.001			0.00	0.004		0.00	0.009	
0.64	0.146			0.64	0.387		0.64	0.656	
1.27	0.18			1.27	0.584		1.27	1.098	
1.91	0.208			1.91	0.795		1.91	1.544	
2.54	0.233	1.77		2.54	1.125	8.52	2.54	2.13	16.14
3.18	0.246			3.18	1.26		3.18	2.493	
3.81	0.258			3.81	1.461		3.81	2.913	
4.45	0.268			4.45	1.623		4.45	3.168	
5.08	0.273	1.37		5.08	1.667	8.34	5.08	3.187	15.94
7.62	0.288			7.62	1.846		7.62	3.287	
10.16	0.292			10.16	1.987		10.16	3.453	
12.70	0.312			12.70	1.996		12.70	3.6	
Modified Max.Dry Density g/cc			**1.390**				**OMC %**		**28.12**


10 Blows
CBR Chart
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
LOAD, KN
0.00 1.27 2.54 3.81 5.08 6.35 7.62 8.89 10.16 11.43 12.70 13.97
PENETRATION,mm

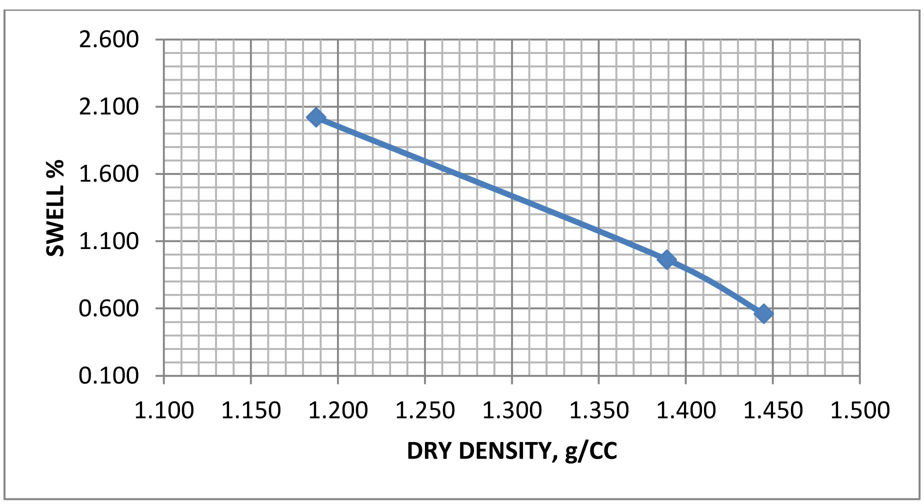

30 Blows
CBR Chart
2.5
2
1.5
1
0.5
0
LOAD, KN
0.00 1.27 2.54 3.81 5.08 6.35 7.62 8.89 10.16 11.43 12.70 13.97
PENETRATION, mm



No.of blows	MCBS %	DDBS g/cm3	SWELL %	Corrected CBR %	% OF Compaction
65	23.9	1.445	0.558	16.14	104
30	37.9	1.389	0.962	8.46	100
10	24.302	1.188	2.019	1.77	85



CBR % at 95 % MDD	**6.00**	**Swell %**	**1.28**

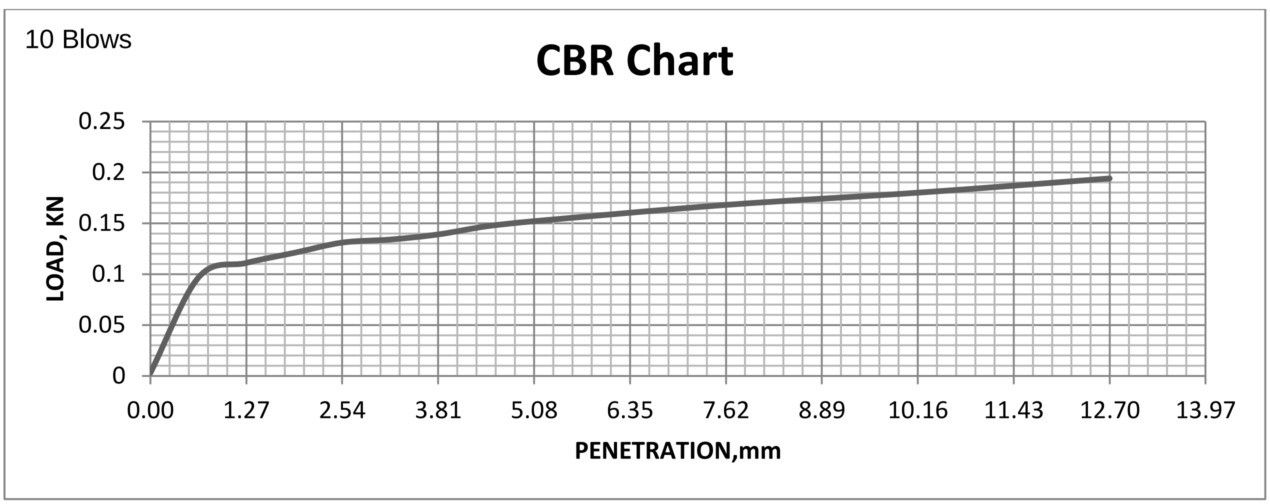


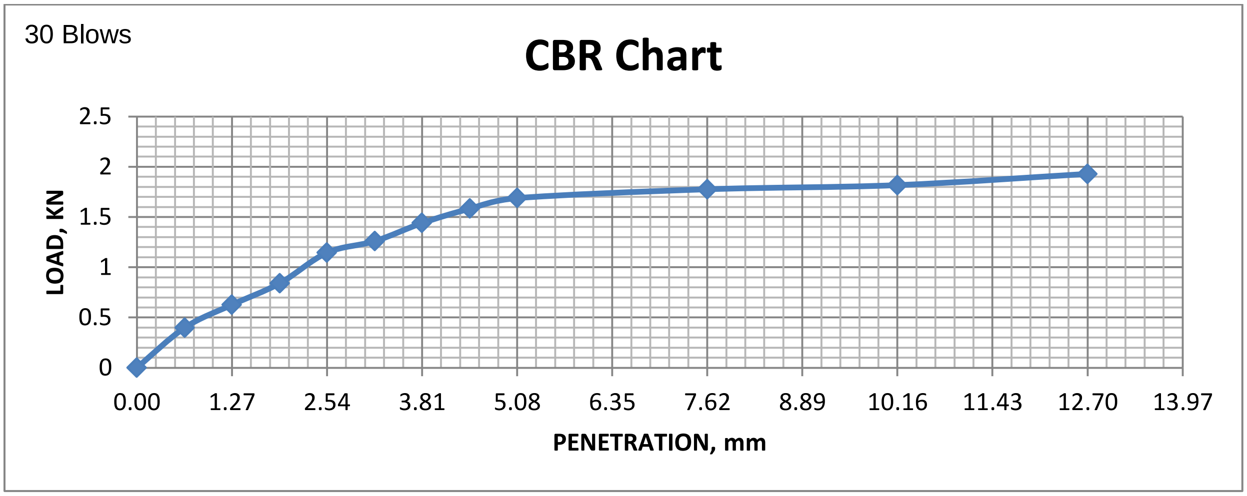
CBR Data of Two Layer Reinforced Soil Trial Three

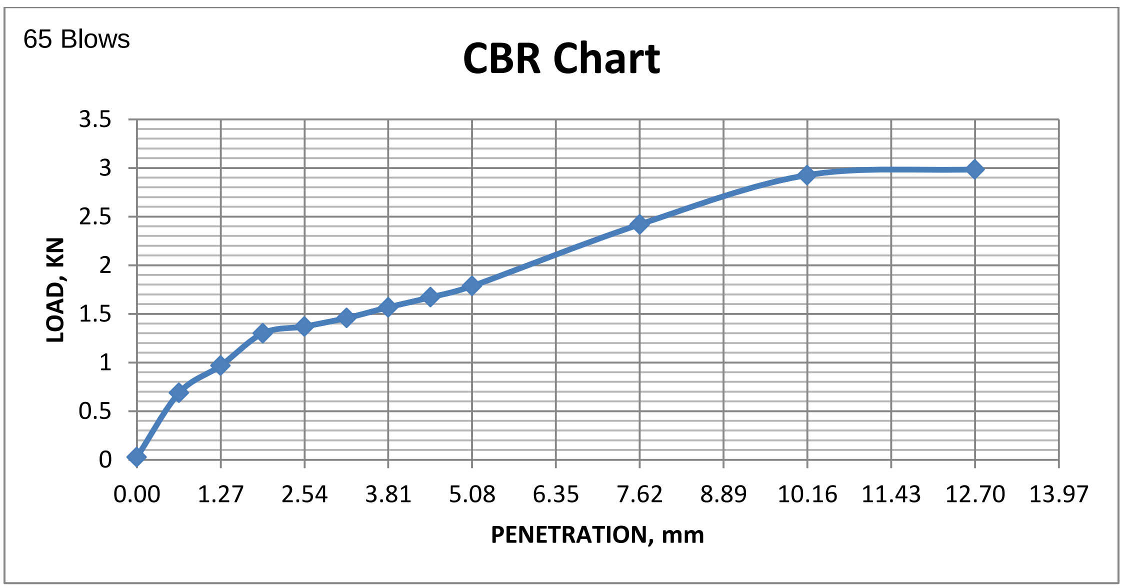
COMPACTION DATA	65 Blows		30 Blows		10 Blows	
	Before Soak	After soak	Before soak	After soak	Before soak	After soak
Mould No.	1	1	2	2	3	3
Mass of soil + Mould +Base Plate	13349.5	13694.5	13177.3	13860.7	12908.6	13958.5
Mass of Mould + Base Plate	9288.5	9288.5	9346.3	9346.3	9521.6	9521.6
Mass of Soil	4061	4406	3831	4514.4	3387	4436.9
Volume of Mould	2285	2285	2285	2285	2285	2285
Wet density of soil	1.777	1.928	1.677	1.976	1.482	1.942
Dry density of soil	1.430	1.449	1.337	1.438	1.185	1.263

MOISTURE CONTENT DATA	65 Blows				30 Blows				10 Blows			
	Before soak	After soak			Before soak	After soak			Before soak	After soak		
Container no.	GT-3	2B	4	13	K-4	C	A	2A	2	B	DH	4B
Mass of wet soil + Container	130.6	179.0	129.4	143.1	129.3	175.6	123.6	178.0	128.5	196.3	168.7	185.8
Mass of dry soil + Container	108.5	138.9	106.9	106.8	106.7	137.5	102.1	126.7	106.4	139.2	117.4	128.0
Mass of container	17.5	27.9	17.4	18.2	17.6	27.9	18.4	28.0	18.4	31.5	17.1	26.2
Mass of water	22.1	40.1	22.5	36.3	22.6	38.1	21.5	51.3	22.1	57.2	51.3	57.8
Mass of dry soil	91.0	111.1	89.5	88.6	89.1	109.7	83.8	98.7	88.0	107.7	100.3	101.8
Moisture content	24.3	36.1	25.2	41.0	25.4	34.7	25.6	52.0	25.1	53.1	51.2	56.8
Average moisture content	24.3		33.1		25.4		37.4		25.1		53.7	

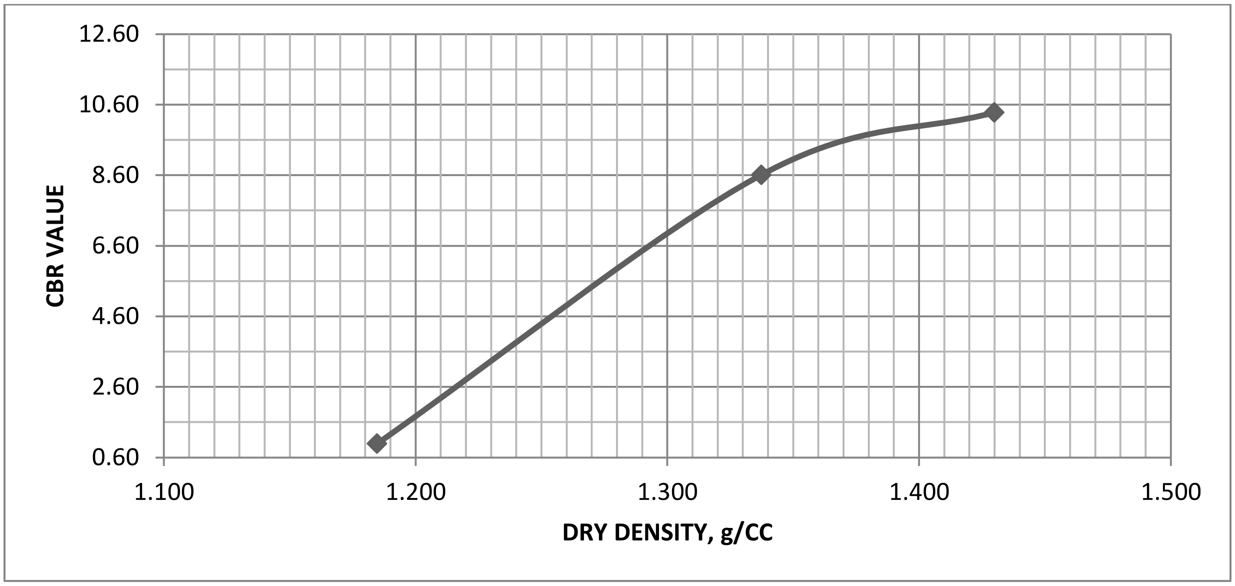
Penetration after 96 hrs Soaking Period				Surcharge Weight:-4.55 KG					
10 Blows				30 Blows			65 Blows		
Pen.mm	Load, KN	CBR %		Pen.mm	Load, KN	CBR %	Pen.mm	Load, KN	CBR %
0.00	0.003			0.00	0.001		0.00	0.026	
0.64	0.097			0.64	0.399		0.64	0.687	
1.27	0.111			1.27	0.627		1.27	0.967	
1.91	0.121			1.91	0.841		1.91	1.3	
2.54	0.131	0.99		2.54	1.145	8.67	2.54	1.37	10.38
3.18	0.134			3.18	1.26		3.18	1.457	
3.81	0.139			3.81	1.44		3.81	1.567	
4.45	0.147			4.45	1.583		4.45	1.67	
5.08	0.152	0.76		5.08	1.688	8.44	5.08	1.785	8.93
7.62	0.168			7.62	1.776		7.62	2.419	
10.16	0.18			10.16	1.817		10.16	2.926	
12.70	0.194			12.70	1.929		12.70	2.985	
Modified Max.Dry Density g/cc			**1.390**			**OMC %**		**28.12**	



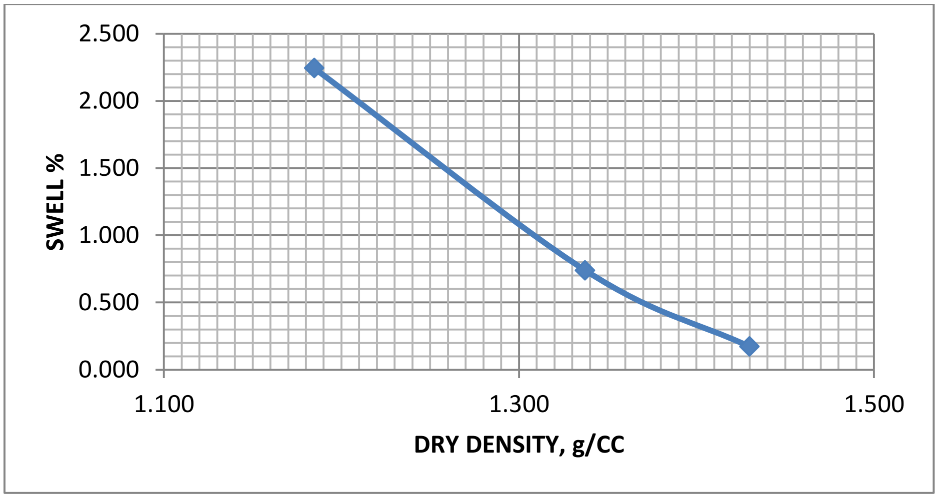




No.of blows	MCBS %	DDBS g/cm3	SWELL %	Corrected CBR %	% OF Compaction
65	24.3	1.430	0.172	10.38	103
30	37.4	1.337	0.739	8.61	96
10	25.1	1.185	2.242	0.99	85



CBR % at 95 % MDD	**7.22**	**Swell %**	**0.90**



Appendix D:-Photo Taken During Investigation


CAS
5000.2
COUNT
0
PCS
UNIT WEIGHT
0