Jimma University

School of Graduate Studies

Jimma Institute of Technology

School of Civil and Environmental Engineering

Highway Engineering Stream

Effects of Capillary rise on Properties of Subgrade Soils: Case study on Jimma to Bedele Road

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

By: Sinishaw Sahile

November, 2017

Jimma, Ethiopia

Jimma University

School of Graduate Studies

Jimma Institute of Technology

School of Civil and Environmental Engineering

Highway Engineering Stream

Effects of Capillary rise on Properties of Subgrade Soils: Case study on Jimma to Bedele Roads

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

By: Sinishaw Sahile

Main Advisor: Dr. Ing. Tamene Adugna

Co-Advisor: Mr. Jemal Jibril (PhD. Fellow)

November, 2017

Jimma, Ethiopia

DECLARATION

This thesis is my original work and has not been presented for any degree in any other University.

Sinishaw Sahile

Signature________________ Date_______________

I will submit this thesis for examination with my approval as University Supervisor

Adviser

Main adviser: Dr. Ing. Tamene Adugna

Signature______________ Date ________________

Co- adviser: Ing. Jemal Jibril

Signature ________________ Date____________________

ACKNOWLEDGMENT

First Praise and Glory be to Almighty GOD for best owing me strength with health and power to complete this work or researches.

Secondly, I would like to express my sincere thanks and appreciation to main Advisor Dr. Ing. Tamene Adugna (PhD) and Co-advisor Ing .-Jemal Jibril (M.Sc), for his advice, patience and guidance throughout the process of completing this research. I would like to all my deepest thanks to the Ethiopian Road Authority (ERA), for its kind help, support, and sponsorship of this study program for the M.Sc in Highway Engineering and to all my lecturers who have taught me in Jimma University and Jimma Institute of Technology, thank you for all the knowledge and guidance. Also, I would like to say thanks a lot to all my friends who shared their continuous unlimited help and kind support in preparing this research.

Finally, my special thanks go to my parents, brothers and sisters who are always been there in times of difficulties and giving me moral support to complete this research work.

ABSTRACT

Design of the pavement layer to be laid over the subgrade soil can start with the estimation of subgrade strength or capacity and the volume of traffic to be carried. Various pavement layers are very much dependent on the strength of subgrade soil over which they are going to be laid. The subgrade soil can be subjected to change in moisture or saturation level due to capillary rise/saturation (h_c). Change in moisture level in subgrade soil causes change in mechanical properties of subgrade material leading to pavement weakness and it can be quite essential for engineers to understand the effects of capillary rise on the variation of moisture in subgrade soil and effects on the strength properties. The hydraulic conductivity of subgrade soil can be strongly dependent on the texture (contents), grain size, densities and voids of the soil. Keeping this in mind this study has been carried out to evaluate the effects capillary rise/saturation on properties of subgrade soil. This research has also shown that measuring the height of capillary rise/saturation through, rate capillary rise through and effects of capillary saturation on the strength properties of subgrade soil.

The strength properties of subgrade soil is mostly expressed in terms of CBR and understanding the dependence of CBR strength of subgrade soil on water content (moisture variation) contribute better towards the design and maintenance practices. The strength of subgrade soil may vary largely on the amount of saturation in it. Hence in this research experimental investigation has been made are to determine hydraulic conductivity (rate of capillary rise) of the subgrade soil at different level of densities and height of capillary rise through subgrade soil and important geo-technical properties specific gravity, liquid limit, plastic limit, plasticity index, free swell index, Dry density and California Bearing Ratio at different level saturation from day (0) to day (4) through the purposive sampling for each station.

It observed that for each stations (type) subgrade soils, Hydraulic conductivity or rate capillary rise of subgrade soil within any densities and the strength properties reduced. For type one soil at station 25+200 subgrade soils, the CBR values reduced by 82% to the un soaked condition or no change in moisture and for second type at station 35+00 soils, the CBR values reduced by 77.3% to Unsoaked condition. This review Provide a subgrade soil with non-susceptible to hydraulic conductivity which can reduces flow of water through them which is providing the subgrade soil with a minimum diameter of grain size with corresponding to 10% finer in the distribution and further study should be carried out on the other factor that cause variation of moisture in a subgrade soil.

Key words*: subgrade soil, capillary rise/saturation, hydraulic conductivity, CBR*

TABLE OF CONTENT

Declaration I
Acknowledgment II
Abstract III
Table of Content IV
List of Tables VIII
List of Figures X
List of Equation XII
Acronyms XIII
Chapter One 1
Introduction 1
1.1. Background 1
1.2. Statement of the Problem 2
1.3. Research Questions 3
1.4. Objective 3
1.4.1. General Objective 3
1.4.2. Specific Objectives 3
1.5. Significance of the Study 3
1.6. Scope and Limitation of the Study 3
Chapter Two 5
Literature Review 5
2.1. Introduction 5
2.2. Subgrade 6
2.3. Subgrade Performance 6
2.3.1. Desirable Properties 7
2.4. California Bearing Ratio test 7
2.5. Hydraulic conductivity of soil 8
2.6. Laboratory determination of hydraulic conductivity 14
2.6.1. Constant-head test 14
2.6.2. Falling-head test 15

2.7. Effect of capillary rise on the moisture content of subgrade soil. 16
2.8. Effect of moisture variation on CBR.. 17
Chapter Three 20
Methodology 20
3.1. Study Area 20
3.2. Study Design 20
3.3. Study Population 21
3.4. Sampling frame 22
3.5. Sample Size and Sampling procedures 22
3.6. Study Variable 22
3.6.1. Dependent Variable 22
3.6.2. Independent Variable 22
3.7. Experimental Investigations 23
3.7.1. Attreberg Limits. 23
3.7.2. Differential Free Swell 24
3.7.3. Specific Gravity 24
3.7.4. Coefficient of permeability (Hydraulic conductivity) 25
3.7.5. Particle size distribution 26
3.7.6. Modified Proctor Test 27
3.7.7. California Bearing Ratio Test 28
Chapter Four 30
Results and Discussions 30
4.1. Type 1 Soil (Soil at station 25+200) 30
4.1.1. Index properties 30
4.1.2. Particle size distribution 30
4.1.3. Modified proctor compaction test 32
4.1.4. Falling head permeability test 32
4.1.5. California Bearing Ratio Test results 36
4.1.6. Variation of CBR with time of soaking 37
4.1.7. Moisture variation in a soil sample 38
4.1.8. Variation of CBR with respect to moisture 39

4.2. Types of soil -2 (station 35+000) 39
4.2.1. Index properties 39
4.2.2. Particle size distribution 40
4.2.3. Modified proctor compaction test........ 41
4.2.4. Falling head permeability test result........ 41
4.2.5. California Bearing Ratio Test results 44
4.2.6. Variation of CBR with time of soaking........ 44
4.2.7. Moisture variation in a soil sample........ 45
4.2.8. Variation of CBR with respect to moisture 46
4.3. Types of soil-3 (station 17+600) 47
4.3.1. Index Properties........ 47
4.3.2. Particle size distribution 47
4.3.3. Modified proctor compaction test........ 48
4.3.4. Falling head permeability test result........ 48
4.3.5. California Bearing Ratio Test results 51
4.3.6. Variation of CBR with time of soaking........ 51
4.3.7. Moisture variation in a soil sample........ 52
4.3.8. Variation of CBR with respect to moisture 52
4.4. Type of soil 4 (station 30+500) 53
4.4.1. Index Properties........ 53
4.4.2. Particle size distribution 54
4.4.3. Modified proctor compaction test........ 55
4.4.4. Falling head permeability test result........ 55
4.4.5. California Bearing Ratio Test results 57
4.4.6. Variation of CBR with time of soaking........ 58
4.4.7. Moisture variation in a soil sample........ 59
4.4.8. Variation of CBR with respect to moisture 59
4.5. Type of soil 5 (station 11+800) 60
4.5.1. Index Properties........ 60
4.5.2. Particle size distribution 61
4.5.3. Modified proctor compaction test........ 62

4.5.4. Falling head permeability test result. 62

4.5.5. California Bearing Ratio Test results 64

4.5.6. Variation of CBR with time of soaking 65

4.5.7. Moisture variation in a soil sample 66

4.5.8. Variation of CBR with moisture 66

Chapter Five 68

Conclusion and Recommendation 68

5.1. Conclusion 68

5.2. Recommendation 70

References 71

Appendix A: Test Outputs 73

LIST OF TABLES

Table 2.1: Typical range values of hydraulic conductivity for various soil type… 11

Table 4.1: Index properties of type-1 soil at station 25+200……………………… 30

Table 4.2: Specimen data for falling head test of type-1soil at 25+200………….. 33

Table 4.3: Rate of capillary rise at initial void ratio (e = 2.04) of type-1 soil at 25+200………………………………………………………………. 34

Table 4.4: Variation of CBR with time of soaking for type-1 soil at 25+200…….. 37

Table 4.5: Moisture variation in type-1 soil at 25+200 with depths & time of soaking……………………………………………………………... 39

Table 4.6: Index properties of type-2 soil at 35+000 …………………………… 40

Table 4.7: Specimen data for falling head test of type-2 soil at 35+000 ………… 42

Table 4.8: Rate capillary rise at initial void ratio (e = 1.52) of type-2 soil at 35+000….………………………………………………………….. 43

Table 4.9: Variation of CBR with time of soaking for type-2 soil at 35+000……. 45

Table 4.10: Moisture variation in type-2 soil at 35+000 with depths & time of soaking………………………………………………………………… 46

Table 4.11: Index properties of type-3 soil at 17+600……………………………. 47

Table 4.12: Specimen data for falling head test of type-3 soil at 17+600………… 49

Table 4.13: Rate of capillary rise at initial void ratio (e = 1.58) of type-3 soil at 17+600………………………………………………………………. 50

Table 4.14: Variation of CBR with time of soaking for type-3 soil at 17+600……. 51

Table 4.15: Moisture variation in type-3 soil at 17+600 with depths & time of soaking………………………………………………………………… 52

Table 4.16: Index properties of type -4 soil at 30+500 …………………………… 53

Table 4.17: Specimen data for falling head test of type -4 soil at 30+500 ………. 55

Table 4.18: Rate of capillary rise at initial void ratio of type-4 soil at 30+500 ….. 56

Table 4.19: Variation of CBR with time of soaking for type-4 soil at 30+500 ….. 58

Table 4.20: Moisture variation in type-4 soil at 30+500 with depth & time of soaking………………………………………………………………… 59

Table 4.21: Index properties of type-5 soil at 11+800…………………………….. 60

Table 4.22: Specimen data for falling head test of type-5 soil at 11+800………… 62

Table 4.23: Rate of capillary rise at initial void ratio of type-5 soil at 11+800…… 63

Table 4.24: Variation of CBR with time of soaking for type-5 soil at 11+800…….. 65

Table 4.25: Moisture variation in type-5 soil at 11+800 with depths & time of soaking……………………………………………………………… 66

LIST OF FIGURES

Figure 2.1: Conceptual model for capillary rise ……………………………………… 5
Figure 2.2: Definition of Darcy's law………………………………………………….. 9
Figure 2.3: Variation of in situ kv for clay soils……………………………………….. 13
Figure 2.4: Illustrative diagrams for constant head tests…………………………….. 14
Figure 2.5: Source and movement of water in subgrade from saturated zone……. 15
Figure 2.6: Illustrative diagrams for falling- head tests…………………………….... 17
Figure 3.1: Google map location of Jimma to Bedele roads……………………… 20
Figure 3.2: Flow chart of experimental investigation…………………………….. 29
Figure 4.1: Particle size distribution curve of type-1 soil at 25+200…………….. 31
Figure 4.2: Modified proctor compaction test result of type-1soil at 25+200…… 32
Figure 4.3: Falling head permeability (rate of capillary) test determination……… 35
Figure 4.4: Relationship b/n the densities& permeability of type-1soil at 25+200.. 36
Figure 4.5: Stress vs penetration graph for type-1 soil at 25+200 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition…………………………….... 37
Figure 4.6: Variation of CBR with time of soaking for type-1 soil at 25+200…… 38
Figure 4.7: Schematic diagrams for moisture variation in type-1 soil sample at 25+200………………………………………………………………… 38
Figure 4.8: Variation of CBR with moisture for type-1soil at 25+200…………… 39
Figure 4.9: Particle size distribution curve of type-2 soil at 35+000…………….. 40
Figure 4.10: Modified proctor compaction test result of type-2 soil at 35+000 ….. 41
Figure 4.11: Relationship b/n the densities & permeability of type-2 soil at 35+000……………………………………………………………….. 43
Figure 4.12: Stress vs penetration graph for type-2 soil at 35+000 soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition…………………………….. 44
Figure 4.13: Variation of CBR with time of soaking for type- 2 soil 35+000…….. 45
Figure 4.14: Variation of CBR with moisture for type-2 soil at 35+000………….. 46
Figure 4.15: Particle size distribution curve of type-3 soil at 17+600……………… 47
Figure 4.16: Modified proctor compaction test result of type-3 soil at 17+600…… 48
Figure 4.17: Relationship b/n the densities &permeability of type-3 soil at 17+600. 50

Figure 4.18: Stress vs penetration graph for type-3 soil at 17+600 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition…………………………….. 51
Figure 4.19: Variation of CBR with time of soaking for type-3 soil at 17+600…… 52
Figure 4.20: Variation of CBR with moisture for type-3 soil at 17+600…………... 53
Figure 4.21: Particle size distribution curve of type-4 soil at 30+500 ………….. 54
Figure 4.22: Modified proctor compaction test result of type-4 soil at 30+500 … 55
Figure 4.23: Relationship b/n the densities &permeability of type-4 soil at 30+500 57
Figure 4.24: Stress vs penetration graph for type-4 soil at 30+500 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition…………………………… 58
Figure 4.25: Variation of CBR with time of soaking for type-4 soil at 30+500...... 59
Figure 4.26: Variation of CBR with moisture for type-4 soil at 30+500……….. 60
Figure 4.27: Particle size distribution curve of type-5 soil at 11+800……………… 61
Figure 4.28: Modified proctor compaction test result of type-5 soil at 11+800…… 62
Figure 4.29: Relationship b/n the densities and permeability of type-5 soil at 11+800………………………………………………………........ 64
Figure 4.30: Stress vs penetration graph for type-5 soil at 11+800 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition……………………………... 65
Figure 4.31: Variation of CBR with time of soaking for type -5 soil at11+800……. 66
Figure 4.32: Variation of CBR with moisture for type-5 soil at 11+800…………... 67

LIST OF EQUATION

Equation 2.1:	California Bearing Ratio (CBR) …………………………..	8
Equation 2.2 -2.3:	Average flow velocity through soil………………………...	9
Equation 2.4:	Velocity through the void spaces of soil…………………...	9
Equation 2.5:	Volume Rate of flow through the soil……………………...	10
Equation 2.6 -2.8 :	Relationship between hydraulic conductivity with voids in granular soils……………………………………………….	10
Equation 2.9:	Hydraulic conductivity for fairly uniform sand……………	11
Equation 2.10- 2.12:	Rate of Capillary in one dimensional column soil…………...	12
Equation 2.13:	Hydraulic conductivity of clay soil…………………………..	12
Equation 2.14:	Hydraulic conductivity of clay soil in vertical and horizontal flow………………………………………………………..	13
Equation 2.15-2.16:	Temperature correction factor for viscosity of water……….	13
Equation 2.17-2.19:	Hydraulic conductivity of course- grained soils……………..	14
Equation 2.20-2.24:	Hydraulic conductivity of fine- grained soils………………..	15
Equation 3.1:	Free Swell index of soil………………………………………	24
Equation 3.2:	Specific gravity of soil……………………………………….	24
Equation 3.3:	Hydraulic conductivity of soil……………………………….	25
Equation 3.4:	Relationships between density, void ratio and hydraulic conductivity of soil……………………………………..	26
Equation 3.5:	Coefficient of uniformity and curvature……………………	27

ACRONYMS

AASHTO	American Association State Highway and Transportation
ASTM	American Society for Testing and Material
A	Area
CBR	California Bearing Ratio
e	Void ratio
H	Head
H_C	Height of Capillary
i	Hydraulic gradient
K	Coefficient of permeability (Hydraulic conductivity)
LL	Liquid Limit
L	Length
MDD	Maximum Dry Density
N	Number of Blows
n	Porosity
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
Q	Discharge
q	Discharge velocity
SWC	Subgrade Water Content
T	Temperature
t	Time
V	Velocity
Z	Depth
μ	Viscosity

CHAPTER ONE

INTRODUCTION

1.1. Background

The objective of pavement design is to provide a structural and economical combination of materials to carry traffic in a given climate over the existing soil conditions for a specified time interval. Soil mechanical properties represent key factor that affect pavement structural design. As noted by (Yoder and Witezek 1975), all pavements derive their ultimate support from the underlying subgrade: therefore, knowledge of basic soil mechanics is essential.

Soil is a gathering or deposit of earth material, derived naturally from the breakdown of rocks or decay of undergrowth that can be excavated readily with power equipment in the field or disintegrated by gentle reflex means in the laboratory. The supporting soil below pavement and its special under course is called sub grade. Without interruption soil beneath the pavement is called natural sub grade. Compacted sub grade is the soil compacted by inhibited movement of heavy compactors. Subgrade soils have desirable properties like, strength permanency, incompressibility, stability, bearing capacity, low change in volume during moisture content variation or low shrinking and/or swelling due to moisture contents, in order to with stands stresses due to traffic loads. Moisture contents of subgrade soil can be influenced by a number of things such as: poor drainage, ground water table elevation, capillary rise/saturation, infiltration or pavement porosity (cracks in pavements).

Capillary rises or saturations in soils are the movement of pore water from lower elevation to the higher elevation through the soil particles by the hydraulic gradients acting across the pore air/pore water interface. Capillary rise / saturation are highly dependent on the hydraulic conductivity of soil, type of soil (soil constituent), density and height of capillary. Therefore, Jimma to Bedele road (study area) there are many things that cause capillary rise/saturation. For instance poor drainage, weather condition, low land area, a lot of cutting section and marshland besides them and due to infiltration (penetration) of water into asphalt layer.

Based on the desirable properties of subgrade soils, identifying the effect of capillary rise on properties of subgrade is very essential. These identifications are important while designing the subgrade foundation for highway in area which capillary rise is sensitive, during the maintenance of roads capillary rise may one of cause of failures. So, identifying of the effects capillary rise with the effects of another factor are crucial impact on the highway contraction industry in terms of design, economies, maintenance, service and other. Considering the above, it has been proposed in this thesis to study the variation in strength properties of subgrade soil at particular area or feasible station on the study area due to variation of moisture contents (capillary rise/saturation) with respect to different density levels and conclude the general aspects of moisture conditions on determination of different strength parameters, so as to achieve the most viable and economical pavement design.

1.2. Statement of the Problem

A detailed and comprehensive geotechnical investigation is an essential requirement for the Design and construction of civil engineering projects. The proper design of civil engineering structures like foundation of buildings, retaining walls, highways, etc. requires adequate knowledge of sub surface conditions at the sites of the proposed structures. Many damages to buildings, roads and other structures founded on soils are mainly due to the lack of proper investigation of substructure condition (Fasil. A, 2003)
Investigations of the underground conditions at a site are pre-requisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies for a proposed project (Bowles 1996).

In roads the effects of capillary rise/ saturation on the properties of subgrade soil is increasing the soil water contents or saturation of the subgrade soil, in its ultimate height of capillary through the hydraulic conductivity (rate of capillary rise) of the soil. This can change the mechanical properties of subgrade material leading to pavement weakness. Decreasing subgrade strength, stiffness and capability. This incremental in subgrade moisture contents can loses the existing subgrade strength permanency, stability, incompressibility and stiffness etc. The capillary rise/saturation can affect subgrade soil water contents (SWC) at which the maximum dry density (MDD) was attained during the compaction. So that, this change in moisture content can reduce the CBR. The reduction in subgrade bearing capacity results decreasing the strength to withstands stresses due to traffic loads, longitudinal rutting in the wheel path and associated cracking and ultimate pavement failure.

1.3. Research Questions

The research questions that this study will attempt to clarify; are as follows**:**

1. What is the height of capillary rise in subgrade soil?
2. How to measure the rate of capillary rise (hydraulic conductivity or coefficient of permeability) of subgrade soil?
3. What are the effects of capillary rise (Saturation) on the strength of subgrade soil?

1.4. Objective

1.4.1. General Objective

To evaluate the effect of capillary rise (capillary saturation) on properties of subgrade soil

1.4.2. Specific Objectives

- ❖ To measure the height of capillary rise (saturation) in subgrade soil.
- ❖ To evaluate rate of capillary rise (hydraulic conductivity or coefficient of permeability) of a subgrade soil.
- ❖ To evaluate the effect of capillary rise (capillary saturation) on the strength of subgrade soil.

1.5. Significance of the Study

The study was investigated the moisture variation or saturation due to capillary based on the hydraulic conductivity of subgrade soil in its height of capillary and effects on the strength properties of subgrade soil. Based on this it propose the type of subgrade soil which can be provided to overcome the flows of water through them to sustain strength permanency and the clue for further cause of moisture variation in a subgrade soil.

1.6. Scope and Limitation of the Study

Studies on the properties of subgrade soil are very wide and vast. Because of this the properties of subgrade soil can be interpreted and described in a lot manner. This study or researching will focus on effects of capillary saturation on the properties of subgrade soil. Among the properties of subgrade soil affected by capillary saturation basically focus on the strength properties of subgrade soil. Based on the descriptive area that enhance the rise of capillary five pits of different station are taken through purposive sampling.

The following main tests and analysis of the result are conducted in this research; index properties of subgrade soil, height of capillary rise through the soil, rate of capillary rise through the soil at different level density and the variation of moisture by capillary saturation that increase the subgrade water content and effects on the density and bearing capacity (strength) of subgrade soil. This can be described in terms of CBR and the sequences of CBR reduction are observed.

The variation of moisture or saturation in subgrade soil can be influenced by a number of things such as drainage, ground water table elevation, capillary rise/ saturation, infiltration or pavement porosity (cracks in pavement). Among these studies will be focus on the saturation due to capillary rise in a subgrade soil. The depth of ground water and its fluctuation at different season are not considered due to lack of economies and time and these tests needs deep pit by machineries and longtime observation. Only capillary rise/saturation through the soil is taken for further incremental of moisture.

CHAPTER TWO

LITERATURE REVIEW

2.1. Introduction

Capillary rise is a well -known unsaturated soil phenomenon that describes the movement of pore water from lower elevation to higher elevation driven by the hydraulic head gradient acting across the curved pore air/pore water interface. Three fundamental physical characteristics related to capillary rise are of primary practical concern:

1. The maximum height of capillary rise,
2. The fluid storage capacity of capillary rise, and
3. The rate of capillary rise.

Each of these aspects has an important influence on the overall engineering behavior of unsaturated soil/water systems and is a complex function of both the soil and pore water properties. The development and improvement of new or existing experimental, analytical, and numerical techniques for measuring or modeling the height, storage capacity, and rate of capillary rise and the dependence of each on the relevant soil and pore water properties has historically been an active subject of basic and applied soils. Research (Malik et al. 1989; Parlange et al.1990; Lu et al. 1995; Stephens 1996).

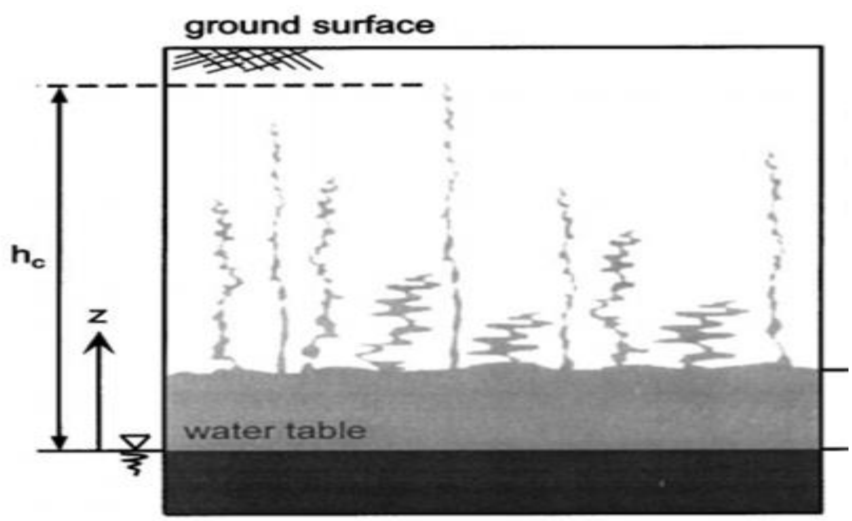


Figure 2.1: Conceptual model for capillary rise, (Geotechnical and Geo environmental journal, June 1, 2004.)

Figure 2.1 Show a conceptual model for capillary rise and the associated relationship between suction head and water content, i.e. the soil–water characteristic curve. The soil profile is delineated by three distinct zones: a saturated zone located below the water table where the pore water pressure is positive, a saturated zone located below the air-entry head but above the water table where the pore water pressure is negative with respect to atmospheric pressure, and an unsaturated zone located above the air-entry head where capillary water rises as a series of connected or disconnected fingers to an ultimate height (h_c).

2.2. Subgrade

The crust of a pavement, whether flexible or rigid, rests on a soil foundation on an embankment or cutting, normally known as subgrade. Sub grade can be defined as a compacted layer, generally of naturally occurring local soil, assumed to be 500/300 mm in thickness, just beneath the pavement crust, providing a suitable foundation for the pavement. The sub grade in embankment is compacted in two layers, usually to a higher standard than the lower part of the embankment .The soil in subgrade is normally stressed to certain minimum level of stresses due to the traffic loads and the subgrade soil should be of good quality and appropriately compacted so as to utilize its full strength to withstand the stresses due to traffic loads. This leads to economization of the overall pavement thickness. On the other hand the subgrade soil is characterized for its strength for the purpose of analysis and design of pavement.

2.3. Subgrade Performance

A subgrade's performance generally depends on three of the basic characteristics, which are briefly described below:

I. **Load bearing capacity**: The subgrade must be able to support loads transmitted from the pavement structure. This load bearing capacity is often affected by degree of compaction, moisture contents and soil type. A subgrade that can support a high amount of loading without excessive deformation is considered good.

II. **Moisture content**: Moisture tends to affect a number of subgrade properties including load bearing capacity, shrinkage and swelling. Moisture content can be influenced by a number of things such as drainage, groundwater table elevation, Capillary rise or capillary saturation, infiltration, or pavement porosity (which can be assisted by cracks in the pavement). Generally, excessively wet subgrades will deform excessively under load.

III. **Shrinkage and/or swelling**: Some soils shrink or swell depending upon their moisture content. Additionally, soils with excessive fines content may be susceptible to frost heave in northern climates. Shrinkage, swelling and frost heave will tend to deform and crack any pavement type constructed over them.

2.3.1. Desirable Properties

The desirable properties of subgrade soil as a highway material are;

- Withstand capability (Stability)
- Ease of compaction
- Strength permanency
- Low change in volume during adverse conditions of weather and ground water table (capillary saturation).
- Superior drainage
- Incompressibility

The most common parameter used to evaluate pavement layer strength is the California Bearing Ratio (CBR). The CBR value is influenced by the water content and the dry density as well as the texture (type) of the soil. Generally, the CBR test in the laboratory is conducted on test samples prepared at the dry density and water content likely to be achieved in the field. Whereas the Field dry density can be fairly well predicted the difficulty is to determine the stable moisture content at which to conduct the test.

2.4. California Bearing Ratio test

The California Bearing Ratio Test (CBR Test) is a penetration test developed by California State Highway Department (U.S.A.) for evaluating the bearing capacity of subgrade soil. The CBR test was first introduced or developed by O.J. Porter at California Highway Department in 1920. It is otherwise called as load-deformation test which is conducted in the laboratory or in the fields and these results are generally used to find the thickness of pavement layers, base course and other layers of a given traffic loading by the use of empirical design chart.

The CBR determines the thickness of different elements constituting the pavement. The CBR test is the ratio of force per unit area required to penetrate soil mass by a circular plunger of 50mm at the rate of 1.25mm/min. Observations are carried out between the load resistances (penetration) vs. plunger penetration. The California bearing ratio, CBR is expressed as the ratio of the load resistance (test load) of a given soil sample to the standard load at 2.5mm or 5mm penetration, expressed in percentage.

CBR = (Test load/Standard load) ×100……………………… (2.1)

The standard load for 2.5mm and 5mm penetrations are 1370 kg and 2055 kg respectively. The CBR test is carried out on a small scale penetration of dial reading with probing ring divisions. The proving ring divisions are taken corresponding to the penetrations at 0, 0.64,1.27, 1.96, 2.52, 3.18,3.81,4.45, 5.08, 7.62, 10.16, 12.7 and from which Test loads are calculated and hence CBR value of soil is being determined. The maximum Load and penetration is recorded if it occurs for a penetration of less than 12.7 mm. The curve is mainly convex upwards although the initial portion of the curve may be concave upwards due to surface irregularities. A correction is then applied by drawing a tangent to the curve at the point of greatest slope. The corrected origin will be the point where the tangent meets the abscissa. The CBR values are usually calculated for penetrations of 2.54 mm and 5.08mm. Generally the CBR values at 2.54mm penetration will be greater than 5.08mm penetration and in such a case the former is taken as the CBR value for design purposes. If the CBR value corresponding to a penetration of 5.08mm exceeds that for 2.54mm, the test is repeated. If identical results follow, the bearing ratio corresponding to 5mm penetration is taken for design.

2.5. Hydraulic conductivity of soil

The soil spaces or pores between soil grains allow water to flow through them. In soil mechanics and foundation engineering, know that how much water is flowing through a soil in unit time. This knowledge is required to design of earth dams, determine the quantity of seepage under hydraulic structures, and dewater before and during the construction of foundations.

(Darcy 1856) proposed that average flow velocity through soils is proportional to the gradient of the total head. The flow in any direction, j, is

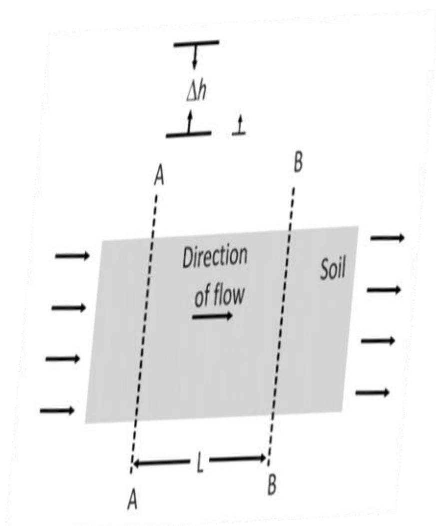
$$V_j = K_j \frac{dH}{dx_j} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

Where V is the average flow velocity, K is a coefficient of proportionality called the hydraulic conductivity (sometimes called the coefficient of permeability) and dH is the change in total head over a distance dx. The unit of measurement for k is length/time, and it is usually expressed in cm/sec or m/sec in SI units. Darcy's law becomes.

$$V = k\frac{\Delta h}{L} = ki \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.3)$$

Where i = h/L is the hydraulic gradient.

Figure 2.2: For calculating the velocity of flow of water through the soil.



.

Figure 2.2: Definition of Darcy's law

Δh = piezometric head difference between the sections

L = distance between the sections at AA and BB

(Note: Sections AA and BB are perpendicular to the direction of flow).

The average velocity calculated from the previous equation is for the cross-sectional area normal to the direction of flow. Flow through soils, however, occurs only through the interconnected voids. The velocity through the void spaces is called the seepage velocity (v_s) and is obtained by dividing the average velocity by the porosity of the soil:

$$V_s = \frac{k_j}{n} i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.4)$$

The volume rate of flow, q, is the product of the average velocity and the cross-sectional area.

$q = vA = Aki$ …………………….......................... (2.5)

The hydraulic conductivity of soils depends on several factors: fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of mineral particles, and degree of saturation. The value of hydraulic conductivity varies widely for various soils. Some typical values for saturated soils are given in the table below. The hydraulic conductivity of unsaturated soils is lower and increases rapidly with the degree of saturation. The value of the hydraulic conductivity of soils varies greatly. In the laboratory it can be determined by means of constant head or falling head permeability testing. The constant head test is more suitable for granular soils. Table 2.1, provides typical range for the values of k for various soils. In granular the value primarily depends on the void ratio. In the past, several equations have been proposed to relate the value of k with the void ratio in the granular soil:

$$\frac{k_1}{k_2} = \frac{e_1}{e_2} \quad \text{(2.6)}$$

$$\frac{k_1}{k_2} = \frac{\left(\frac{e_1^2}{1+e_1}\right)}{\left(\frac{e_2^2}{1+e_2}\right)} \quad \text{(2.7)}$$

$$\frac{k_1}{k_2} = \frac{\left(\frac{e_1^3}{1+e_1}\right)}{\left(\frac{e_2^3}{1+e_2}\right)} \quad \text{(2.8)}$$

Where, k_1 and k_2 are the hydraulic conductivities of a given soil at void ratios e_1 and at e_2 respectively.

Table 2.1: Typical range values of hydraulic conductivity for various soil type.

Soil type	K (cm/sec)
Clean gravel	$10^{2} – 1.0$
Coarse sand	$1.0 – 10^{-2}$
Fine sand	$10^{-2} – 10^{-3}$
Silty/ clay	$10^{-3} – 10^{-5}$
Clay	$< 10^{-6}$

(Hazen 1930) proposed and empirical relationship for hydraulic conductivity for fairly uniform sand in the form of:

$$k = c\ D_{10}^{2} \quad (2.9)$$

Where , k = coefficient of permeability in cm/sec

c = a constant varying from 1.0 to 1.5 (usually taken to be 1.0)

D_{10} = effective size in mm of 10% finer.

This equation is based on Hazen's observations of loose, clean, filter sands. A small quantity of silts and clays, when present in a sandy soil, may change the hydraulic conductivity substantially.

As early as 1943, Terzaghi formulated a simple theory based on Darcy's law and saturated hydraulic conductivity for predicting the rate of capillary rise in a one-dimensional column of soil. To arrive at his solution for the rate of capillary rise, (Terzaghi 1943) made two major assumptions:

1. That Darcy's law for saturated fluid flow is roughly applicable to unsaturated flown and
2. That the hydraulic gradient responsible for capillary rise can be approximated as follows:

$$i = \frac{h_c - z}{z} \quad (2.10)$$

Where, h_c = ultimate height of capillary rise; and

Z = distance measured positive upward from the elevation of the water table. Terzaghi's other assumption, Darcy's law is valid for capillary rise, can be expressed in familiar mathematical terms as follows:

$$q = k_s i = n\frac{dz}{dt}\text{...}(2.11)$$

Where, q = discharge velocity;

k_s = saturated hydraulic conductivity of the soil; and

n = soil porosity.

Solving Equation (2.9) and (2.10) and imposing an initial condition of zero capillary rises at zero time, Terzaghi arrived at the following solution describing the location of the capillary wetting front z as implicit function of time:

$$t = \frac{nh_c}{k_s}(\ln\frac{h_c}{h_c\text{-}z} - \frac{z}{h_c})\text{..}(2.12)$$

In the same decade, (Lane and Washburn 1946) conducted a systematic experimental study that included measurements of the height and rate of capillary rise in carefully controlled columns of gravel, sand, and silt. The Lane and Washburn study and subsequent laboratory studies have unambiguously showed that Terzaghi's original analytical solution significantly over predicts the rate of capillary rise, often by as much as 2 orders of magnitude.

For clayey soils in the field, a practical relationship or estimating the hydraulic conductivity (Tavenas et al., 1983) is;

$$\log k = \log k_o - \frac{e_o\text{-}e}{c_k}\text{...}(2.13)$$

Where, $k =$ Hydraulic conductivity at a void ratio e.

$k_o =$ In situ hydraulic conductivity at a void ratio e_o

$c_k =$ conductivity change index $\approx 0.5\,e_o$

For clayey soils, the hydraulic conductivity for flow in the vertical and horizontal directions may vary substantially. The hydraulic conductivity for flow in the vertical direction (k_v) for in situ soils can be estimated from figure 2.2. For marine and other massive clay deposits;

$k_h / k_v < 1.5$ ……………………………………… (2.14)

Where, k_h = hydraulic conductivity for flow in the horizontal direction. For verve clays, the ratio of k_h / k_h may exceed 10.

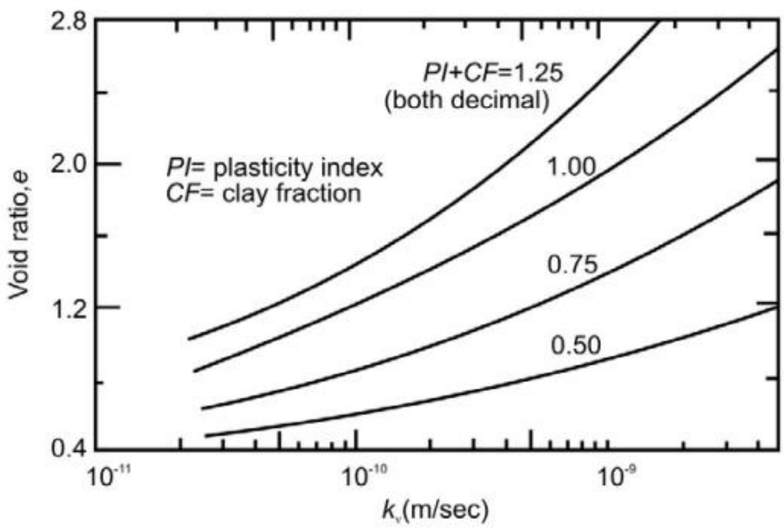


Figure 2.3: Variation of in situ kv for clay soils after (Tavenas et al. 1983).

The hydraulic conductivity is a measure of the ease with which water flows through Permeable materials. It is inversely proportional to the viscosity of water which decreases with increasing temperature. Therefore, permeability measurements at laboratory Temperatures should be corrected, before application to field temperature Conditions by means of the equation.

$$k_f = \frac{\mu_T}{\mu_f} k_T \quad \text{………………………………………... (2.15)}$$

Where k_f and k_T. are the hydraulic conductivity or coefficient of permeability values corresponding to the field and test temperatures respectively and μ_f and μ_T are the corresponding viscosities. It is customary to report the values of k_T. at a standard temperature of 20°C. The equation is:

$$k_{20} = \frac{\mu_T}{\mu_{20}} k_T \text{...(2.16)}$$

2.6. Laboratory determination of hydraulic conductivity

Two standard laboratory tests are used to determine the hydraulic conductivity of soil, the constant-head test and the falling-head test.

2.6.1. Constant-head test

The constant head test is used to determine the hydraulic conductivity of coarse-grained soils. A typical constant-head test arrangement is shown below. In this test, water supply at the inlet is adjusted in such a way that the difference of head between the inlet and the outlet remains constant during the test period. After a constant flow rate is established, water is collected in a graduated cylinder for a known duration.

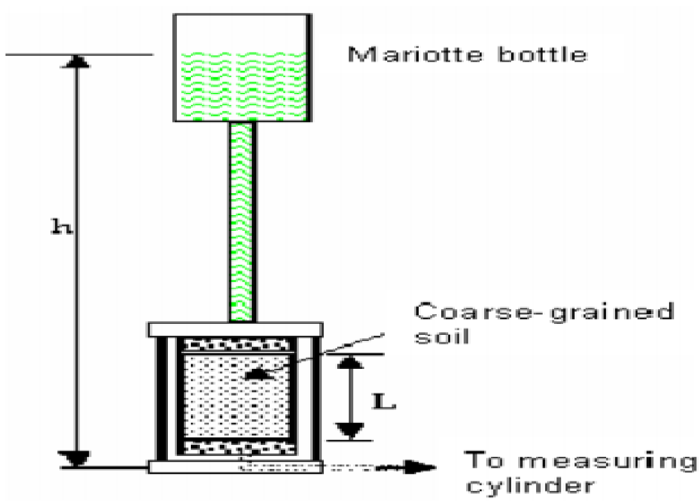


Figure 2.4: Illustrative diagrams for constant head tests.

The total volume of water collected may be expressed as:

$Q = Avt = A(ki)t$ ……………………………………(2.17)

Where, Q = volume of water collected

A = area of cross-section of the soil specimen

t = duration of water collection and since i = h/L for this test, where L is the length of the specimen (height),

$$Q = \frac{Akht}{L} \quad \text{…………………………………………} \quad (2.18)$$

$$k = \frac{QL}{Aht} \text{..}(2.19)$$

2.6.2. Falling-head test

Due to low hydraulic conductivity of fine-grained soils, it will take a considerable time to obtain reasonable discharge volume using the constant-head test. It is therefore customary to use the falling-head test for such materials. A typical arrangement of the falling-head permeability test is shown below.

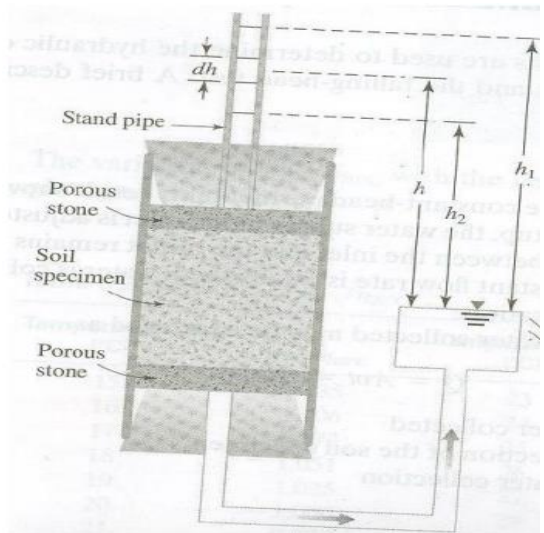


Figure 2.5: Illustrative diagrams for falling- head tests.

During these tests water from the stand pipe flows through the soil. The head of water (h) change with time as flow occurs through the soil. At different times the head of water is recorded. Let dh be the drop in head over a time period dt. The velocity or rate of head loss in the tube is;

$$v = -\frac{dh}{dt} \quad \text{..(2.20)}$$

The rate of flow of water through the specimen at any time to can be given by:

$$q = k\frac{h}{L}A = -a\frac{dh}{dt} \quad \text{…………………………….. (2.21)}$$

Where, q = flow rate

a = cross-sectional area of the standpipe

A = cross-sectional area of the soil specimen

Rearranging the equation gives,

$$dt = \frac{aL}{Ak}\left(-\frac{dh}{h}\right) \quad \text{……………………………… (2.22)}$$

Integration of the left side of this equation with limits of time from 0 to t and the right side with limits of head difference from h1 to h2 gives:

$$t = \frac{aL}{Ak}\ln\left(\frac{h_1}{h_2}\right) \qquad (2.23)$$

$$k = 2.303\frac{aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right) \qquad (2.24)$$

2.7. Effect of capillary rise on the moisture content of subgrade soil.

The function of subgrade is very important in pavement performance. Like a foundation, it bears all the loads on the road and transfers them to the soil. The high change subgrade moisture content leads to pavement weakening and decreases the subgrade strength and stiffness. Reduction in subgrade bearing capacity results in longitudinal rutting in the wheel paths and associated cracking, and ultimately pavement failure.

As the mechanical properties of subgrade materials are highly dependent on their moisture content, predicting future variation of soil moisture content of a subgrade is important. There are many ways that water can reach the pavement structure. The infiltrated precipitation through cracks in the pavement, shoulders, and side ditches are considered as atmospheric sources. The other sources originate from saturated zone and depend on the depth of groundwater, capillary rise which cause saturation. (see Figure 2.5).

Studies addressing how the unexpected moisture in subgrade affects the pavement performance have mostly focused on change in precipitation and temperature. Groundwater table as a free-water surface can act as a source of capillary water. The range of water movement due to capillary potential mostly depends on the soil texture and structure. Both free-water surface and capillary water can be transformed to water vapor due to change in temperature and pressure conditions. A water table rise increases capillary rise, increasing the subgrade soil water content (SWC), Equilibrium moisture content (EMC) and Field moisture content (FMC). Groundwater level as a source of subgrade moisture can affect the water content of subgrade due to capillary rise, reducing CBR.

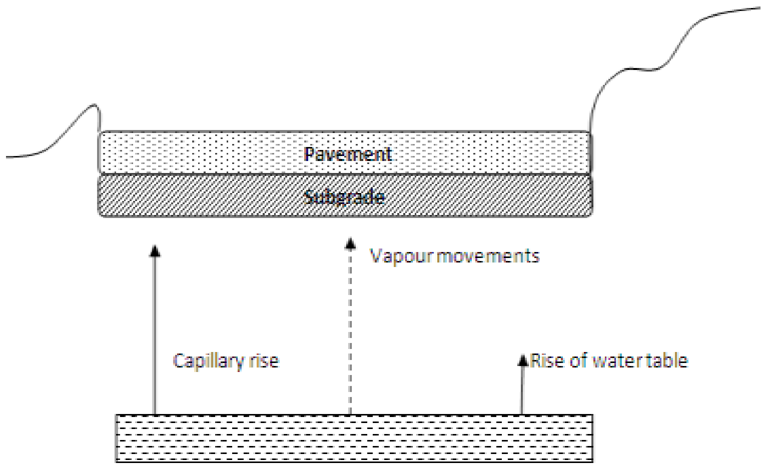


Figure 2.6: Source and movement of water in subgrade from saturated zone, (Houghten, N., 2004 and Vorobieff, G., 2005 Sydney.)

2.8. Effect of moisture variation on CBR

(Alayaki and Bajomo 2011) examined the effect of moisture variation on the strength characteristics of laterite soil in Abeokuta, Ogun State, Nigeria. The result showed that an increase in the soaking period of the compacted soil sample from 1to 5 days result in decrease in the CBR of the soil. He observed that the top face of the soaking soil has a CBR value greater than that at the bottom face.

(Jaleel 2011) studied the effect of soaking on the top and bottom CBR value of a sub-base material. He prepared fourteen CBR samples at 95% relative modified AASHTO compaction. The results showed that, a significant drop in the CBR for top and bottom due to the soaking was observed. Most of decrease in soaking CBR value was pronounced in the first days for top and bottom CBR, respectively. From the results of the testing conducted in this study on the effect of soaking period on top and bottom sub base for highway purpose, he concluded that the load applied on the sub base layer decreases with increase of period soaking.

(Ampadu 2006) examined the effect of water content (WC) on the CBR of a subgrade soil samples of soil from a study site were prepared by laboratory compaction at the optimum water content (OMC) using different levels of compaction to obtain samples different densities. The remolded samples were then subjected to different levels of wetting in a water tank and different degrees of drying in the laboratory and the CBR value were determined. From the laboratory CBR test results on a subgrade material at different water contents for three different dry densities, it may be conclude that the rate of change in CBR per percentage change in water content during drying from the OMC was 3 to 7 times larger than during wetting from OMC.

(Singh et al.2011) developed regression-based models for estimating soaking and un-soaking California Bearing Ratio (CBR) values for fine-grained subgrade soils. Five locally available soils were collected from different zones of West Bengal. The samples were compacted at four different levels of compaction (i.e.50, 56, 65, and 75 blows) and at five different levels of moisture contents on dry and wet sides of an optimum moisture content (OMC) of a soil (i.e., ± 2% OMC, ± 1% OMC, and OMC).Regression models were developed considering different independent parameters namely, index properties of soils, degree of compaction, and moisture content. It was observed that the CBR value, both soaking and un-soaking significantly affected by change in moisture content and compaction effort.

(Ningsih et al. 2012) studied correlation between index properties and CBR tests of Pekanbaru (Indonesia) soils with and without soaking. This research aims to make comparisons between CBR soaking test results for CBR un-soaking in some variation of clay content and make simple comparisons between CBR soaking for CBR un-soaking by considering the soil properties. The results showed that there was a linear correlation between the CBR soaking and CBR un-soaking also influenced by the nature of the index (the properties of the soil).

(Rahman 2010) studied the correlation between CBR results and physical properties of soil. Correlation had been proposed in the study to predict the CBR values at top face of the soil sample for Malaysia’s type of soil based on the collected soil data and results from laboratory works. These correlations were developed based on the Maximum dry density (MDD), Optimum moisture content (OMC) and the number of blows (of CBR test).

(ERES Division 2001) studied correlation of CBR values with soil index properties. The objective of this study was to develop general correlations that describe the relationship between Soil Index Properties and the California Bearing Ratio (CBR) and Resilient Modulus(MR) of unbound materials such as base, sub-base, and subgrade layers in pavement systems.

(Yasin et al. 2001), studied the Effect of Submergence on Subgrade Strength. His study aimed at determining the effects of depth of submergence and duration of submergence on the subgrade strength of soil samples collected from the Dhaka-Aricha highway. CBR tests were performed with different heights of submergence after normal soaking period and also after prolonged submergence. For the studied depth and duration of submergence, no effect of submergence on sub-grade CBR strength could be found for any of the three types of soils tested.

CHAPTER THREE

METHODOLOGY

3.1. Study Area

The research conducted in Jimma to Bedele roads, which covers 130 km in Oromia region of Ethiopia ,located in south west Oromia which connects Jimma town to Bedele town and un intermediate Woredas town like Yabbu, Agaro, Toba, Dembi, Yembero, Gachi, Bedele and Small Villages between them.

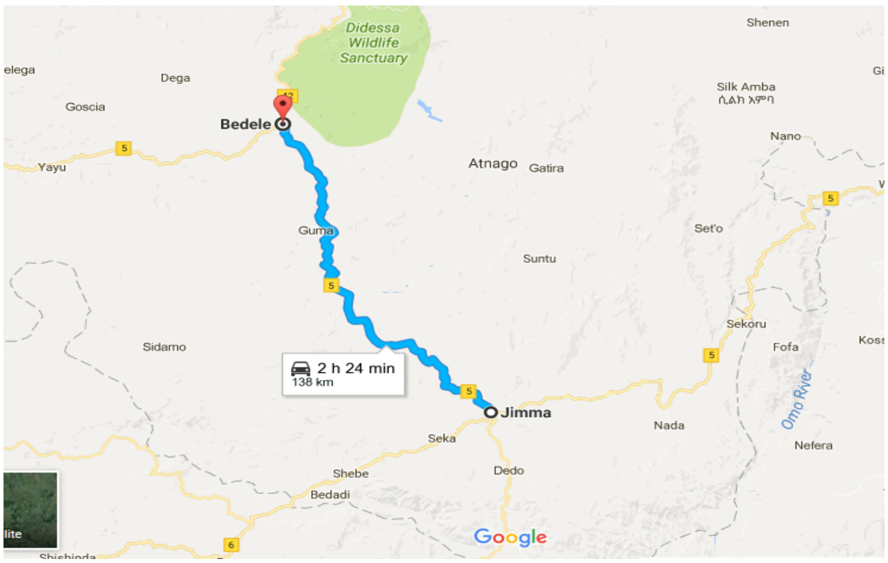


Figure 3.1: Google map location of Jimma to Bedele roads.

The entire study has been conducted on five types of soil and /or station, i.e. station 1, station 2, station 3, station 4 and station 5 with appropriate sampling and testing condition.

3.2. Study Design

Effects of capillary rise/saturation on some properties of subgrade soil by experimental study or researching. This experimental study investigates index properties, height of capillary rise through subgrade soil, rate of capillary rise (hydraulic conductivity) of subgrade soil and the moisture content variation due to capillary saturation. Moisture content variation leads to incremental of subgrade water contents and effects on properties of subgrade soil. These studies clearly interoperates the effects of moisture variation from optimum moisture on the bearing capacity of subgrade soil due to capillary saturation by testing the CBR values at different degree of saturation and percentage changed.

3.3. Study Population

The study populations to be considered to complete this research are index properties of subgrade soil; which are liquid limit, plastic limit, plastic index, free swell index and specific gravity can be tested. Particle size distribution through sieve analysis (dry or wet) can be done for soil classification, so that the tested soil can be identified in percentage (empirically) to gravel, sand and silt /clay depend on their size. The grain size at which the subgrade soil can 10,30 and 60 percent finer can investigated experimentally, Which can indicate the probable flow of water due to capillary through them. Besides the coefficient of uniformity and curvature evaluated to fix weather the soil are well- graded, gap-graded, uniformly graded and poorly graded.

The modified proctor test had been done to attain the maximum dry density with respect to optimum moisture contents. Taking this proctor result as bench mark we can directly go to CBR test for identification of the strength of subgrade soil. This can be conducting from OMC (EMC) and dry density which is un soaking condition (day 0) which is not affected by capillary saturation to different day of soaking (capillary saturation), i.e from un soaked (day 0) to soaked (day 4)

Initially experiments were conducted to find out different properties of subgrade soil such as index properties, grain size distribution, specific gravity, free swell index, rate of capillary rise (hydraulic conductivity)and height of capillary. Later on heavy compaction tests were conducted to find out the optimum moisture content & corresponding maximum dry density. Then CBR tests were made at different moisture contents including OMC and analysis made to investigate the variation of CBR with respect to different days of soaking (capillary saturation) , i.e. from un soaked (day 0) to soaked (day 4). The variations were also made with regard to moisture content at different layers along with different positions (Top, Center and Bottom positions) and also the variations of moisture content with respect to different days of soaking were observed.

3.4. Sampling Frame

Soil sample from the study area or case study can be taken based on purposive sampling on the descriptive area on the existing road listed below, that can enhance our research finding, sag area or lowland area and at cuts of natural ground level, besides to sag area the probable marsh land exist along the cross section of the road without embankment provision and poor (unfunctional) drainage system exist and feasible rutting of wheel path with associated crack exist and ultimate pavement failure exists.

3.5. Sample Size and Sampling procedures

Purposive sampling techniques are used for preparation of subgrade soil sample as described in frame sampling. Accordingly five pits of different station that can enhance the effects of capillary saturation are taken. The bulk soil samples are taken for each station with proper sampling techniques, equipment and methodologies.

- ✓ The Subgrade soil samples are taken from the road near to the shoulder at adequate depth to get subgrade material.
- ✓ Sample taken can be placed inside the thick plastic bags or gauges in order to prevent moisture variation and other adverse change on soil properties.
- ✓ From each pit 50kg of subgrade material are taken, which is sufficient for each variable test needed and some the materials like any rocks and other mixed materials are removed.
- ✓ After completion of sampling they can fill the pits sample by extra filler materials and then go to experimental investigation.

3.6. Study Variable

3.6.1. Dependent Variable

Effects of capillary rise/saturation on the strength properties of subgrade soil

3.6.2. Independent Variable

- Index properties of subgrade soils.
- Height of capillary rise/saturation.
- Rate of capillary rise (Hydraulic conductivity).

- Optimum Moisture Content (OMC) and Maximum Dry Density (MDD).
- Bearing capacity of subgrade soil through (CBR) at different level of saturation (time of soaking).

3.7. Experimental Investigations

Soils are classified with different engineering properties which affect the behavior of soil under different conditions. These properties are described briefly here.

3.7.1. Attreberg Limits.

3.7.1.1. Liquid Limit

The liquid limit (LL) is the water content at which a soil changes from plastic to liquid behavior. At this limit, the soil possesses a small value of shear strength, losing its ability to flow as a liquid. In other words, the liquid limit is the minimum moisture content at which the soil tends to flow as a liquid.

Prepare un air dried soil mass that pass on a sieve No.40 (425μm Sieve) were prepared and 250g of soil passing this sieve taken per station. This quantity of soil is sufficient for both liquid and plastic limits. Mix thoroughly with distilled water to form a uniform paste and place the mixed soil in a storage dish, cover it to prevent loss of moisture, allow to stands for at least 16h and then get water content using the multipoint liquid limit methods. The liquid limit is identified in the laboratory as that water content at which the groove cut into the soil pat in standard liquid limit device requires 25 blows from a height of 1cm to close along a distance of 13mm.

Plot the relation-ship between the water content, W, and the corresponding number of drops, N, of the cup on a semi-logarithmic graph with the water content as ordinate on the arithmetic scale, and the number of drops as abscissas on the logarithmic scale. Draw the best straight line through the plotted points, On the plot (known as flow curve), take the water content corresponding to the intersection of the line with the 25-drop abscissa as a liquid limit of the soil.

3.7.1.2. Plastic Limit

Plastic limit (PL) is the arbitrary limit of water content at which the soil tends to pass from the plastic state to the semi-solid state of consistency. Thus, this is the minimum water content, at which the change in shape of the soil is accompanied by visible cracks, i.e., when worked upon, the soil crumbles.

3.7.1.3. Plasticity Index

Plasticity Index (PI) is the range of water content within which the soil exhibits plastic properties, that is, it is the difference between liquid and plastic limits. If the PL is greater than the LL, report the soil as non-plastic, NP

3.7.2. Differential Free Swell

Free Swell Index is the increase in volume of a soil, without any external constraints, on submergence in water.

$$\text{Free swell index} = \frac{V_D - V_K}{V_K} x100\% \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

Where, V_D = volume of soil specimen read from the graduated cylinder containing distilled water.

V_K = volume of soil specimen read from the graduated cylinder containing kerosene.

3.7.3. Specific Gravity

Specific gravity of soil solids is defined as the ratio of unit weight of solids to the unit weight of water at the standard temperature (4°C). Specific gravity of soil may be defined as the ratio of the unit mass of solids (mass of solids divided by volume of solids) in the soil to the unit mass of water. In equation form,

$$G_s = \frac{M_s}{V_s \rho_w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

Where, G_s =specific gravity of soil

M_S =mass of solid, g

Vs = volume of solid, cm^3

ρ_w = unit mass of water(1g/cm^3)

The specific gravity of most natural soil falls in the general range of 2.60-2.80; the smaller the values are for coarse-grained soil. The specific gravity is essential in relation to other soil tests. It is used when calculating porosity and void ratio and is particularly important when compaction and consolidation properties are being investigated and preparing a soil sample of 20g soil passing 2mm sieve size or 100g of soil passing 4.75mm sieve size using ASTM recommendation.

3.7.4. Coefficient of permeability (Hydraulic conductivity)

The hydraulic conductivity (K) is a measure of the ease with which water flows through Permeable materials. It is inversely proportional to the viscosity of water which decreases with increasing temperature. The falling-head method, covered in this part, may be used to determine the permeability of both fine-grained soils (such as silts and clays) and coarse-grained soils or granular soils.

The coefficient of permeability (hydraulic conductivity) can be computed using the equation.

$$k = R_t\left(2.303\frac{aL}{At}\log\left(\frac{h_0}{h_1}\right)\right) \text{..} (3.3)$$

Where, k = Coefficient of permeability, cm/s

a = cross-sectional area of standpipe, cm^2

L = length of the specimen, cm

A = cross-sectional area of soil specimen, cm^2

h_0 = hydraulic head at beginning of test, cm

h_1 = hydraulic head at end of test, cm

t = total time for water in burette to drop from h_0 to h_1, s

R_t = is the temperature correction factor for the viscosity of water

$= \eta_T/\eta_{20^0c}$

Where η_T =Viscosity of water at temperature T.

η_{20^0c} = Viscosity of water at 20^0c.

Since the hydraulic conductivity of soil depends on several factors, among that fluid viscosity, pore-size distribution, grain size distribution and viscosity are the major one, so that mostly the permeability of the soil depend on the void ratio of the soil. Based on this it can be obtained rate of capillary or coefficient of permeability of soil with different density, to achieve this it can combine different empirical formula which is;

$$\gamma_d = \frac{\gamma}{1+\omega} \;,\; \gamma_d = \frac{G_s \gamma_w}{1+e} \text{ and } \frac{k_1}{k_2} = \frac{\left(\frac{e_1^3}{1+e_1}\right)}{\left(\frac{e_2^3}{1+e_2}\right)} \quad \text{..................................} \quad (3.4)$$

Then placing the same specimen in a sink which water is about some height above the cover and soaked at least for 24hrs. The sample will be saturated until minimum amount of entrapped air, discharge in equivalent to discharge out through the soil specimen or fully saturation. Then connect the plastic tube on valves in the tops specimen (mold) at the same time closing the bottom (outlet) valves of specimen (mold) and then reading the rise water through the tube. When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink (allowing for capillary rise in the tube)

3.7.5. Particle size distribution

A soil consists of particles of various shapes, sizes and quantity. Grain-size (particle size) analysis is a method of separation of soils in to different fractions based on particle size as per test method ASTM D422. It expresses quantitatively the proportions, by mass, of varies sizes of particles present in a soil. It is shown graphically on a particle size distribution curve.

3.7.5.1. Sieve Analysis

The sieve analysis determine the grain size distribution curve of soil samples by passing them through a stack of sieves of decreasing mush opening sizes and by measuring the weight retained on each sieve. Soil with grain size larger than 4.75mm are taken as Gravel, those grain size between 2mm and 0.75mm as Sand and those less than 0.75mm are taken as combination of silt and clay. The data are recorded in each sieve size and computing percentage of finer. Grain size distribution curve can be obtained by plotting grain size as abscissa on logarithmic scale versus percentage fines as ordinate on arithmetic scale. The corresponding grain size 10% (D_{10}), 30% (D_{30}) and 60% (D_{60}) finer in terms of weight can be obtained. It can be obtained by using semi logarithmic interpolation between the data points of the grain size distribution curve. Coefficient of uniformity (C_u) and curvature (C_c)

$$C_u = \frac{D_{60}}{D_{10}} \qquad C_c = \frac{(D_{30})^2}{D_{60} x D_{10}} \quad (3.5)$$

The diameter corresponding to 10% finer in the distribution curve are indicate particularly or important in regulating the flow of water through soils. The higher this value, the coarser the soil and the better the drainage characteristics

A well graded soil has a uniformity coefficient greater than about 4 for gravels and 6 for sands. A soil that has a uniformity coefficient of less than 4 contains particles of uniform size. The minimum value of Cu is 1 and corresponds to a collection of particles of the same size.

Coefficient of curvature, also called coefficient of gradation or coefficient of concavity (C_c or sometimes C_z). The coefficient of curvature is between 1 and 3 for well-graded soils. Gap-graded soils have values outside this range.

3.7.5.1.1. Wet sieve Analysis

Wet sieve analysis is for soil containing a substantial of fine particles. For sample finer particles governing it is more convincing to do wet analysis. About 1kg of soil was taken and it was washed thoroughly with water on 75 micron sieve, soil retained on sieve was dried and weighed and used for sieve analysis. These dried soils were passed through stack of sieves like 4.75mm, 2.36mm, 1.18mm, 600µm, 300µm, 150µm, 0.75µm.

3.7.5.1.2. Dry sieve Analysis

About 2 kg of soil was taken and sieved with different sieve size and soil retained on each sieve size was recorded for the sake analysis. Then the subgrade soil can be classified as per designation of ASTM D-2487.

3.7.6. Modified Proctor Test

Compaction means to press soil particles tightly together by expelling air from void spaces between the particles. Compaction increase soil unit weight, there by producing three important effects: (1) an increase in shear strength, (2) a decrease in future settlement, and (3) a decrease in permeability. These three changes in soil characteristics are beneficial for some types of earth construction, such as highway and other.

The Proctor compaction test is a laboratory method of experimentally determining the optimal moisture content at which a given soil type will become most dense and achieve its maximum dry density. The term Proctor is in honor of R. R. Proctor, who in 1933 showed that the dry density of a soil for a given comp active effort depends on the amount of water the soil contains during soil compaction. His original test is most commonly referred to as the standard Proctor compaction test; later on, his test was updated to create the modified Proctor compaction test.

Modified proctor compaction test follow the same procedure as the standard compaction test, but use the heavier rammer (44.5kN instead of 24.4kN) with larger height of drop (457 mm instead of 305 mm). Also compact the soil in 5layers (instead of 3) by applying 56 blows per layer (instead of 25). As per ASTM D 1558 , preparing a soil sample of fraction passing 4.75mm sieve of amount at least 4.3kg for computing of moist- unit weight and moisture content in each trial.

The moisture-unit weight relationship for the soil sample being tested can be analyzed by plotting a graph with moisture contents along the abscissa and corresponding dry unit weight along the ordinate. The moisture content and dry unit weight corresponding to the peak of the plotted curve are termed "optimum moisture content" and "maximum dry unit weight," respectively.

3.7.7. California Bearing Ratio Test

The CBR is a measure of resistance of a material to penetration of standard plunger under controlled density and moisture conditions. The test procedure should be strictly adhered if high degree of reproducibility is desired. The CBR test may be conducted in re-molded or undisturbed specimens in the laboratory. The test has been extensively investigated for field correlation of flexible pavement thickness requirement.

Briefly, the test consists of causing a cylindrical plunger of 50mm diameter to penetrate a pavement component material at 1.25mm/minute. The loads, for 2.54mm and 5.08mm are recorded. This load is expressed as a percentage of standard load value at a respective deformation level to obtain CBR value and Molding the soil sample into standard molds keeping its moisture content and dry density exactly same as its optimum moisture content and proctor density respectively. Determination of CBR strength of the respective soil samples in molds using the CBR instrument. Soil sample is tested for its CBR strength after being soaked in water for 1 day, 2 days, 3 days and 4 days. Unsoaked CBR is also determined for each sample. Generally the experimental investigation of our study can be described in the flow chart of listed bellows.

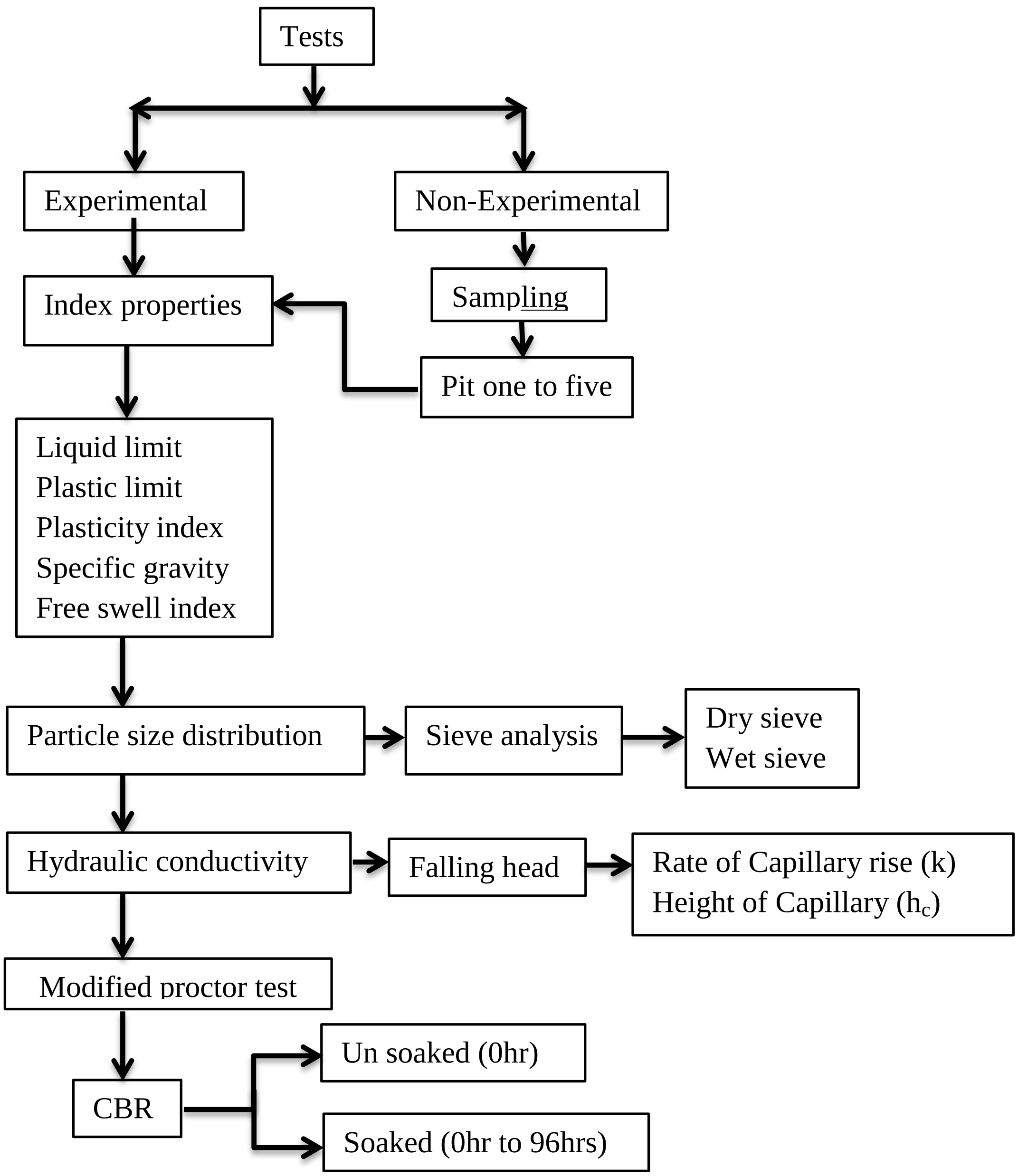


Figure 3.2: Flow chart of experimental investigation.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1. Type 1 Soil (Soil at station 25+200)

4.1.1. Index properties

In this soil the index properties such as Liquid limit, Plastic limit, Plasticity Index, Free swell index and specific gravity value are used as an initial input for rest tests result and analysis, are presented in a table 4.1;

The laboratory detail analysis is attached in Appendix A.1.1, 2 and 4.

Table 4.1: Index properties of type-1 soil at station 25+200

Index properties	Experimental value
Liquid Limit	36.86%
Plastic Limit	18.25%
Plasticity Index	18.61%
Specific Gravity	2.65
Free Swell Index	Non expansive

The result indicated in the table 4.1 shows that this soil is not further affected by shrinking and/or swelling by moisture variation based on the results of free swell index indicates non expansive and stays under plastic characteristics within the above range value shown.

4.1.2. Particle size distribution

The sieve analysis test is done to determine the grain size distribution of the soil sample and then plotting the grain size as abscissa on logarithmic scale versus percentage of fines as ordinate on arithmetic scale. The sieve analysis test result is under appendix A.1.3.

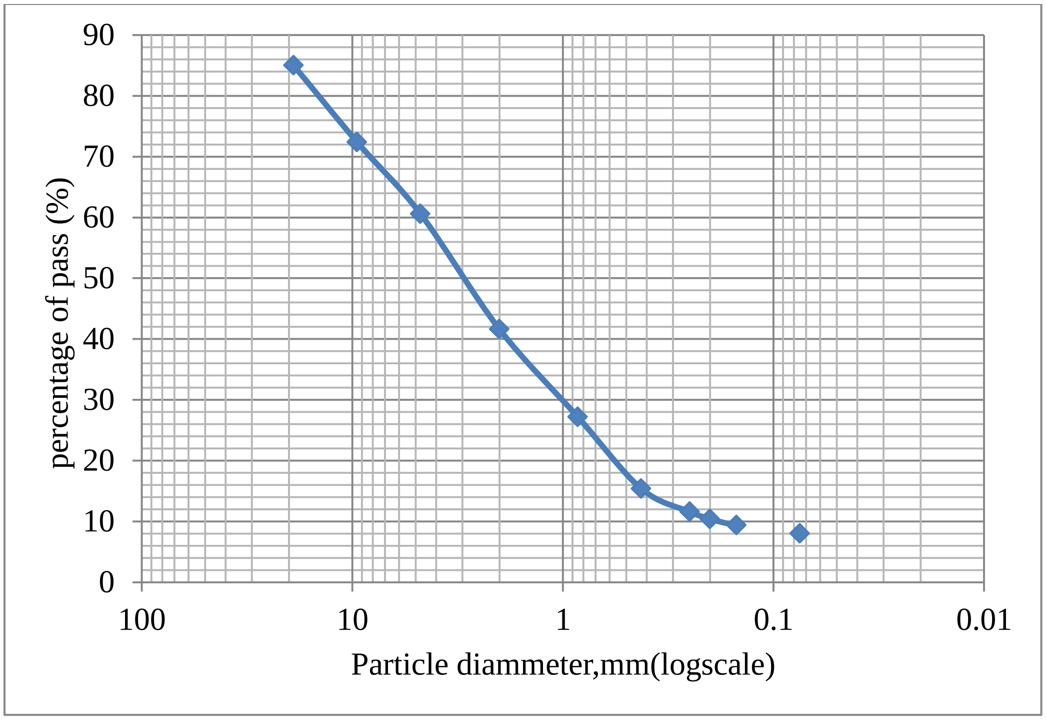


Figure 4.1: Particle size distribution curve of type-1 soil at 25+200.

The figure 4.1 shows that soil with grain size larger or equal to 4.75mm which Gravel, takes 39.4% in a soil, soil with grain size between 2mm and 0.75mm as Sand, takes 52.6% in a soil and soil with grain size less than 0.75mm are taken as combination of silt and clays, takes 8% in a soil.

The grain size corresponding to10%, 30%, and 60 % by weight finer can be obtained by using semi logarithmic interpolation between the data points of the grain size distribution curve. Therefore D_{10}, D_{30} and D_{60} values are 0.18mm, 1.074mm and 4.66mm respectively. The diameter size corresponding to 10% finer in the distribution indicate 0.18mm, this implies that there was probable flow of water through the soil.

The coefficient of uniformity ($C_u = D_{60}/D_{10}$) is 25.88 which is greater than 4 for gravel and less than 6 for sand, particle size are non- uniform and coefficient of curvature or gradation($C_c = D_{30}^2/D_{60}*D_{10}$) is 1.375 which is between the range of 1 and 3. This implies this subgrade soil is well graded soil. As per ASTM D – 2487 indicates the ratio of percent retained above sieve size 4.75mm (R_4= 39.4%) to percent retained above sieve

0.075mm (R_{200} = 92%) is 0.43 < 0.5 , percent of passing sieve size 0.075mm (F_{200}) is 8% < 50% and PI = 18.61% which is greater than PI of A- Line (PI = 0.73(LL-20). So Subgrade soil is classified as Well graded sand with clay and gravel (SW- SC).

4.1.3. Modified proctor compaction test

Modified proctor compaction test has been done for this soil sample to obtain the maximum dry density and optimum moisture contents of the soil.

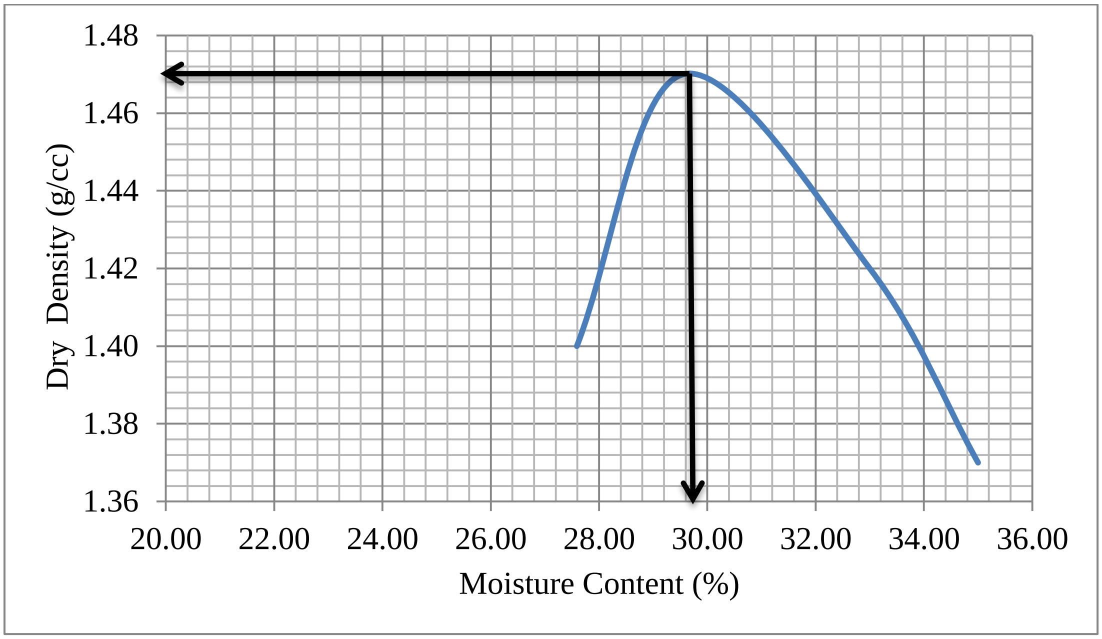


Figure 4.2: Modified proctor compaction test result of type-1soil at 25+200.

The figure 4.2 shows that graph of moisture content and maximum dry density. The purpose of drawing the curve shown in the figure is to extract the Maximum (peak) Dry Density (MDD) and Optimum Moisture Content (OMC). From the figure (OMC = 29.62% and $\gamma_D =$ 1.47g/cm^3)

4.1.4. Falling head permeability test

The falling-head method, covered in this part, may be used to determine the permeability of both fine-grained soils (such as silts and clays) and course- grained soils or granular soils.

The specimens of different density or voids ratio are prepared for testing the flows water through them or permeability. Accordingly the specimen data which is ready for testing are shown in the table 4.2:

A. **Specimen data**

Table 4.2: Specimen data for falling head test of type-1soil at 25+200.

Specimen Mass(M) (kg)	0.845
Specimen Height, L (cm)	11.5
Specimen diameter, D (cm)	10.16
volume of specimen (V) (cm^3)	931.87
Bulk density ($\gamma = M/V$)(g/cm^3)	0.91
Water Content (w) (%)	3.96
Dry density $(\gamma_{dry} = \frac{\gamma}{1+w})$ (g/cm^3)	0.87
Specific gravity of soil, G_s	2.65
Initial void ratio ($e = \frac{G_S \gamma_w}{\gamma_{dry}} - 1$)	2.04

The specimen placed in a sink which water is about 2 cm above the cover and soaked at least for 24 hours. The sample saturated until minimum amount of entrapped air, discharge in equivalent to discharge out (fully saturated). When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink (allowing for capillary rise in the tube)

Height of capillary from the mold (H_C) >= 4.3cm

Height of capillary from the top of the water (H_C) >= 2cm

This result can indicate that at this density there is water flow through the soil due to capillary in its height determined above. This can leads to increasing Subgrade Water Contents or Saturation which can affects mechanical properties like bearing capacity, strength permanency, stability and incompressibility of the soil.

B. Falling head test

The result of rate of capillary rise or coefficient of permeability can be determined at several densities or void ratio of the subgrade soil can be obtained. Using the same specimen data indicated before in table 4.2 and Cross-sectional area of stand pipe, a= 1.664cm, Length of soil specimen in a permeameter (L= 11.5cm) and Cross-sectional area of soil specimen (A= 81.073cm2). The rate of capillary rise at initial void ratio of (e = 2.04) or dry density (γ_{dry}) of 0.87 g/cm^3 indicated in table 4.2 is shown in the table 4.3: using the equation $k = 2.303\frac{aL}{At}\log_{10}\left(\frac{h_1}{h_2}\right)$ indicated in review equation (2.24) and multiplying by temperature correction factor ($R_t = \frac{\mu_T}{\mu_{20}}$) in equation (2.15).

Table 4.3: Rate of capillary rise at initial void ratio (e = 2.04) of type-1 soil at 25+200.

Trial	1	2
Head, h_o(cm)	83.6	83.8
Head,h_1 (cm)	35.6	37.7
Time, t (sec)	909.6	911.4
Temperature, T (^{o}c)	27	26
Permeability at T^{o}c, K_T	0.000220946	0.000206903
R_t for Temperature.	0.854477489	0.872404144
Permeability at 20^{o}C, K_{20}	0.000188794	0.000180503
Average K_{20} (cm/s)	0.000184648	

Figure 4.3: Falling head permeability (rate of capillary) test determination

The result of rate of capillary rise gained above indicate that, in a soil there is a flow water with the above rate value within specified density or voids until to reach it height of capillary. Depend up on these values which is rate capillary at dry density of 0.87 g/cm^3, we can determine the rate of capillary through the soil at different values of density or voids, starting from this initial value up to the Maximum Dry Density (MDD) obtained during the compaction (1.47 g/cm^3) using the equation $\gamma_d = \frac{G_s \gamma_w}{1+e}$ and equation (2.8) in a review which relate the value of k with void ratio or density.

The rate capillary rises with respect to several densities or void ratio of the sample are shown in the figure 4.4:

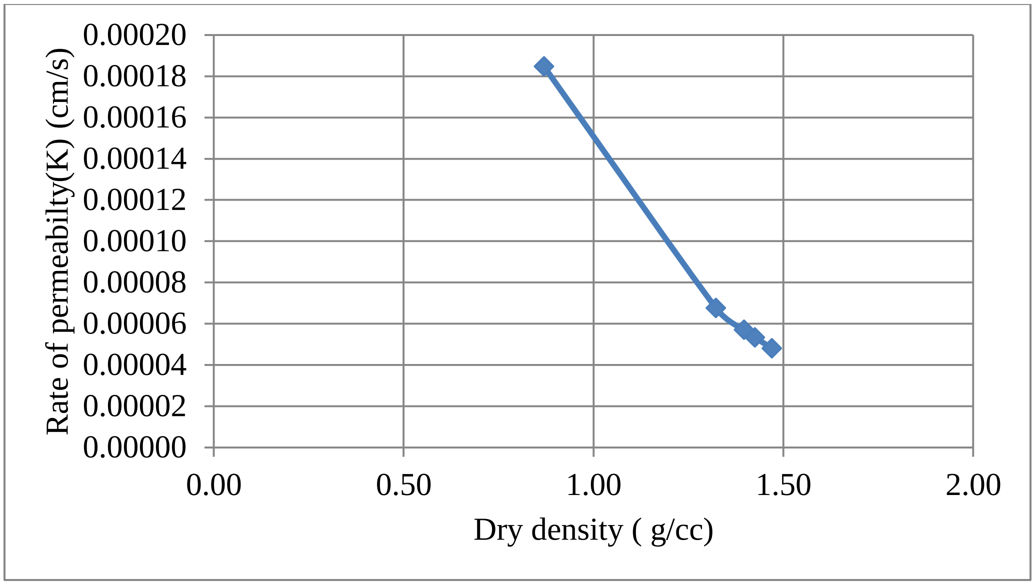


Figure 4.4: Relationship between the densities and permeability of type-1soil at 25+200.

The result shown in a graph 4.4 indicates that the propensity of a soil to allow the flow of water through the soil. In the first two densities (1.32 to 0.87) rate flows is high due to high void spaces and saturation. And from densities (MDD to 1.40) the rates of flows is slow due to decreasing in voids but the result indicates that at MDD there is still probable flow water which can change Subgrade Water Content (SWC) or (OMC) and further incremental of saturation due to capillary. This can effect on the subgrade strength or strength permanency and other. The variation of strength due to capillary saturation has been tested by CBR at different time of soaking.

4.1.5. California Bearing Ratio Test results

The CBR test at different level of saturation (moisture content) due to capillary including OMC and different level compaction (10, 30 and 65 blows) are investigated. The analysis are made to investigate the variation of CBR with respect to different type of soaking, which is from un soaked (day 0) to soaked (day 4) can be observed. The detail laboratory observation is attached in Appendix A.1.6.

Test conducted under OMC (29.62%) and MDD (1.47g/cc).

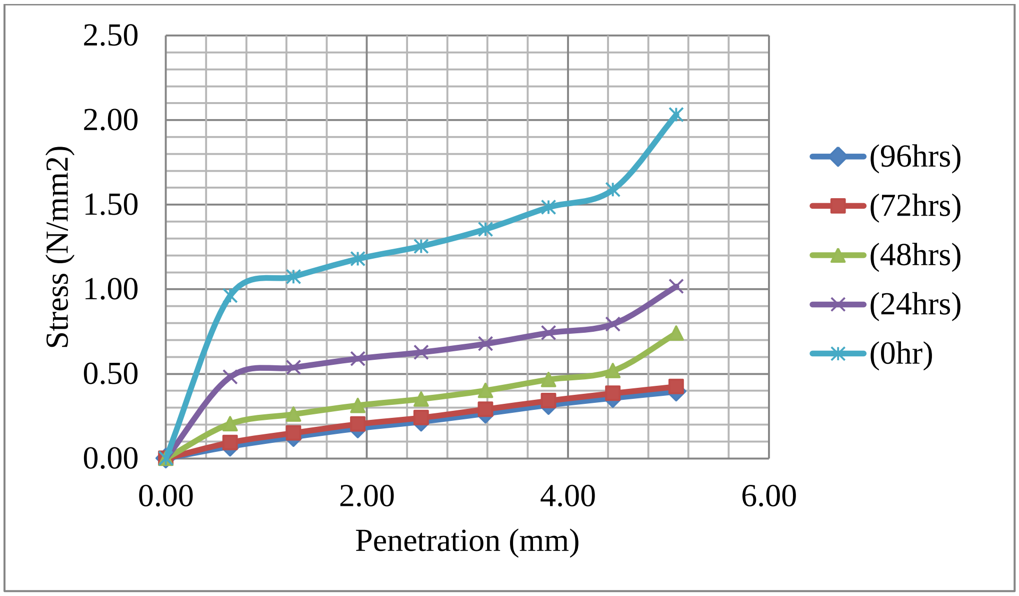


Figure 4.5: Stress vs penetration graph for type-1 soil at 25+200 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition.

The result shown in the graph 4.5 observes that the soil can reduce the ability to withstand the stress as the saturation of the soil increasing. The CBR value or bearing capacity of subgrade soil decreasing.

4.1.6. Variation of CBR with time of soaking

The variation of CBR with respect to time of soaking or saturation due to capillary can be shown in the table 4.4:

Table 4.4: Variation of CBR with time of soaking for type-1 soil at 25+200

Variation of CBR with time of soaking										
Time of soaking in hours										
Time	0Hr		24Hrs		48Hrs		72Hrs		96Hrs	
Pen(mm)	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08
CBR (%)	18.21	19.65	9.1	9.83	5.1	7.16	3.49	4.1	3.16	3.82

The CBR values decreasing with time of saturation increasing but sequence of decreasing are rapid in the first three days saturation and slowly in the last one day of saturation as indicated in table 4.4 and figure 4.6. The moisture of subgrade soil increasing from OMC (at peak dry density obtained). This variation in moisture can lead subgrade soil to reduction in density, incompressibility, strength and bearing capacity.

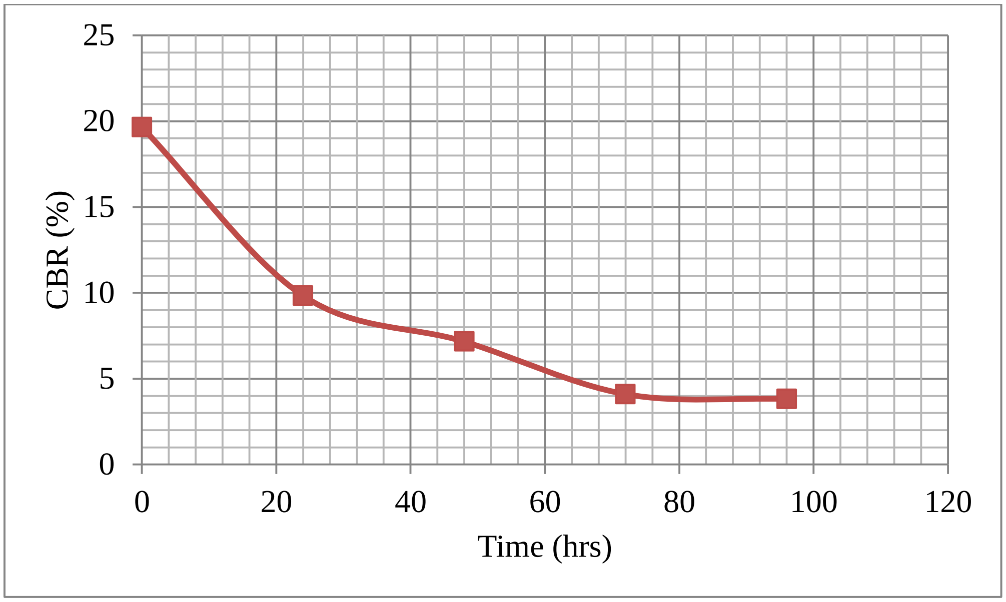


Figure 4.6: Variation of CBR with time of soaking for type-1 soil at 25+200.

4.1.7. Moisture variation in a soil sample

Take a soil sample from various part of CBR sample for determination of moisture content. The schematic diagram is given blow in a figure 4.7. Middle parts or around center line axis are taken, accordingly take the moisture content of a sample around the Top, Middle and Bottom are observed.

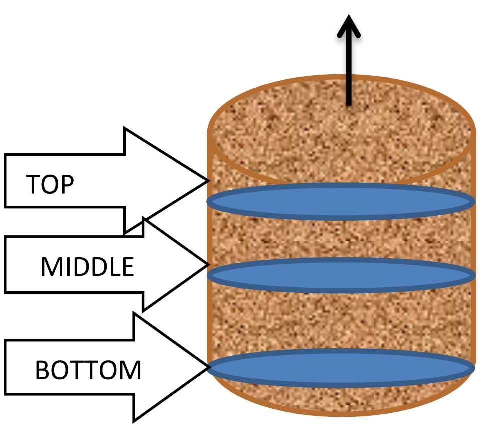


Figure 4.7: Schematic diagrams for moisture variation in type-1 soil sample at 25+200.

Table 4.5: Moisture variation in type-1 soil at 25+200 with depths and time of soaking.

Moisture variation in soil sample (%)						
Position	Axis	Time of soaking				
		0Hr	24Hrs	48Hrs	72Hrs	96Hrs
Top	Centre		17.4	17.52	18.01	18.61
Middle			13.46	13.8	14.92	15.5
Bottom			9.02	9.8	10.4	10.8

The moisture results indicated above in a table 4.6 is the variation of moisture in percent from OMC to the maximum time of soaking.

4.1.8. Variation of CBR with respect to moisture

The variation of CBR values with variation of moisture due to capillary saturation in a soil sample are shown in a figure 4.8:

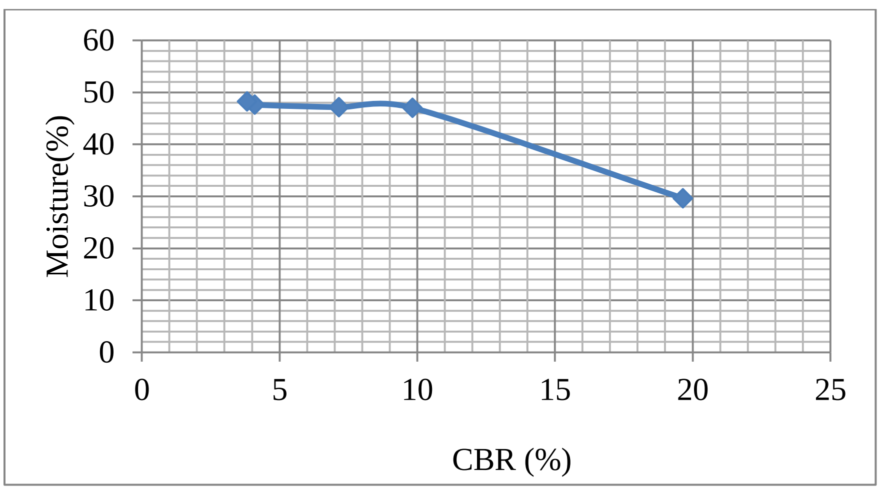


Figure 4.8: Variation of CBR with moisture for type-1soil at 25+200).

The CBR values decreasing highly in the first three day of saturation due to sudden change in moisture and gradually decreasing in the last one day due to no further change in moisture content shown in table 4.6.

4.2. Types of soil -2 (station 35+000)

4.2.1. Index properties

The soil index properties such as Liquid limit, Plastic limit, Plasticity Index, Free swell index and specific gravity value are presented as follow in a table 4.6;

The detail laboratory test result attached in Appendix A.2.1, 2 and 4.

Table 4.6: Index properties of type-2 soil at 35+000

Index properties	Experimental value
Liquid Limit	49.20%
Plastic Limit	29.29%
Plasticity Index	19.91%
Specific Gravity	2.79
Free Swell Index	Non expansive

The result indicated in the table 4.6 shows that this soil also is not further affected by shrinking and/or swelling by moisture variation based on the results of free swell index indicates non expansive and stays under plastic characteristics within the above range value shown.

4.2.2. Particle size distribution

The grain size distribution of this soil sample done by sieve analysis test and plotting the graph as shown below;

The sieve analysis test result is under appendix A.2.3.

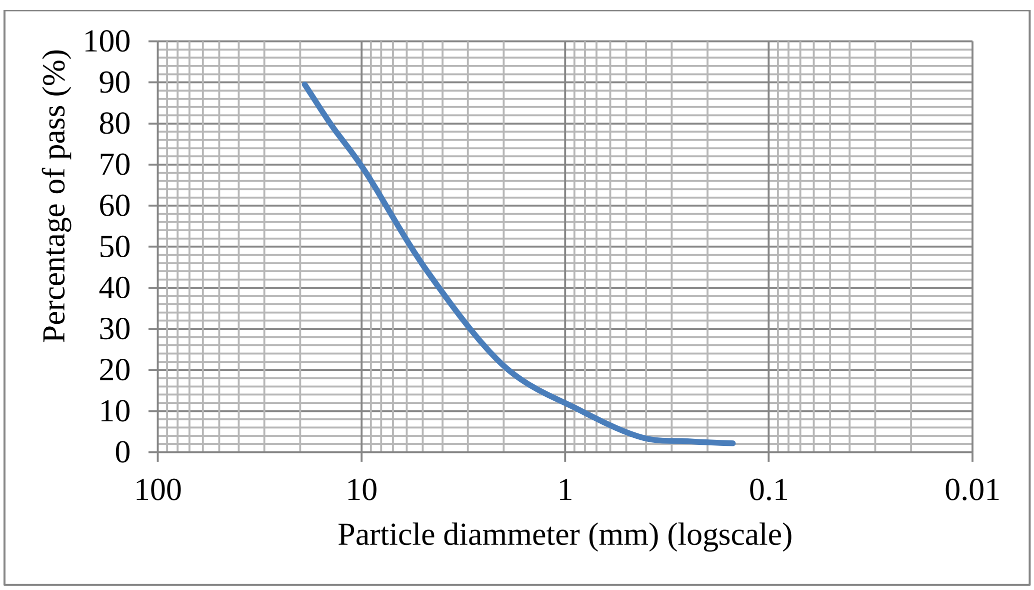


Figure 4.9: Particle size distribution curve of type-2 soil at 35+000.

The result shown in a figure (4.9) implies that in this soil sample 56 % of Gravel, 42.25%of Sand and 1.75%of Silt and/or Clay are exist. The grain size corresponding to10%, 30%, and 60 % by weight finer can be obtained by using semi logarithmic interpolation between the data points of the grain size distribution curve. Therefore D_{10}, D_{30} and D_{60} values are 0.83mm, 3.08mm and 7.92mm respectively. The diameter size corresponding to 10% finer in the distribution indicate 0.83mm, this implies that there was probable flow of water through the soil.

The coefficient of uniformity (C_u) are 9.54 which is greater than 4 for gravel and less than 6 for sand, particle size are non- uniform and coefficient of curvature or gradation (C_c) is 1.44 which is in the range of 1 and 3. This implies this subgrade soil is well graded soil. Percent of subgrade soil passing sieve size 0.075mm (F_{200}) is 1.75 % < 5% and ratio of R_4 to R_{200} is 0.57 > 0.5 and 56% gravel exist, So subgrade soil is classified as Well-graded gravel with sand (GW).

4.2.3. Modified proctor compaction test

The results of modified proctor compaction test are represented in figure 4.10:

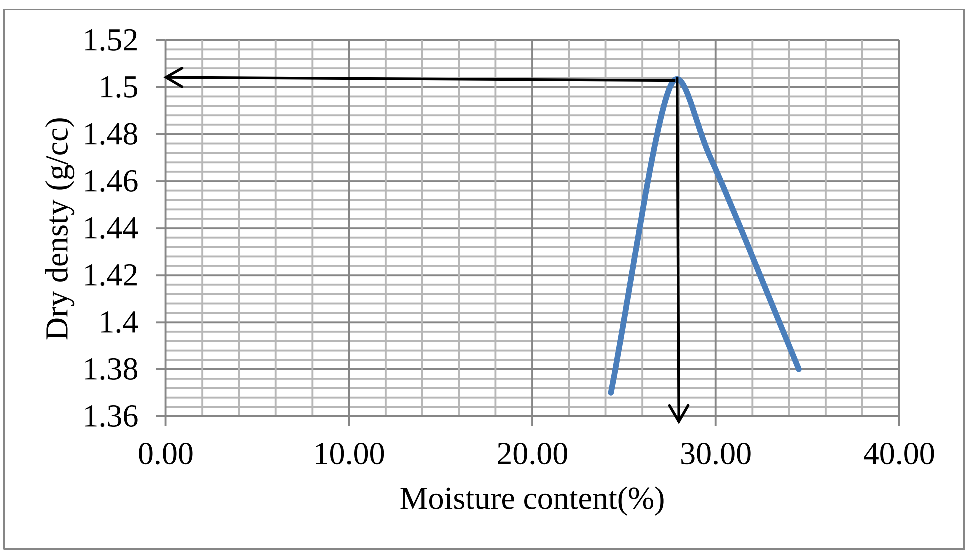


Figure 4.10: Modified proctor compaction test result of type-2 soil at 35+000.

From the figure 4.10, we can observes (OMC = 28.00% and $\gamma_D =$ 1.51g/cm^3)

4.2.4. Falling head permeability test result.

The specimens of different density or voids ratio are prepared for testing the flows water through them or permeability. Accordingly the specimen data which is ready for testing are shown in the table 4.7:

A. Specimen Data

Table 4.7: Specimen data for falling head test of type-2 soil at 35+000.

Specimen Mass(M) (g)	1.08
Specimen Height, L (cm)	11.5
Specimen diameter, D (cm)	10.16
volume of specimen V(cm^3)	931.87
Bulk density, g (g/cc)	1.16
Water Content, w (%)	4.54
Dry density, (γ_d)	1.11
Specific gravity of soil, Gs	2.79
Initial void ratio,(e)	1.52

The specimen placed in a sink which water is about 3 cm above the cover and soaked at least for 24 hours. The sample will be saturated until minimum amount of entrapped air, discharge in equivalent to discharge out (fully saturated).

When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink again it is allowing for capillary rise in the tube.

Height of capillary from the mold (H_C) >= 8cm

Height of capillary from the top of the water (H_C) >= 5cm.

The above height of capillary is larger that of type 1 soil. This is due to grain size corresponding to 10% finer in a soil is 0.83mm which is larger than that type one as shown in particle distribution and high amount of water content exist in a soil.

B. Falling head test

The result of rate of capillary rise or coefficient of permeability can be determined at several densities or void ratio of this subgrade soil can be obtained. Using the same specimen data indicated before in table 4.8. The rate of capillary rise at initial void ratio of (e = 1.52) or dry density (γ_{dry}) of 1.11 g/cm^3 indicated in above table 4.8 is shown in the table 4.9:

Table 4.8: Rate capillary rise at initial void ratio (e = 1.52) of type-2 soil at 35+000.

Trial	1	2
Head, ho(cm)	85.2	88.5
Head,h1 (cm)	40.4	41.8
Time, t (sec)	420.6	420.6
Temperature, T (^{o}c)	26.5	25
Permeability at T(^{o}c) , K_T	0.000418815	0.000421024
R_t for T	0.863356252	0.891033983
Permeability at 20^{o}c, K_{20}	0.000361587	0.000375147
Average K_{20} (cm/s)	0.000368367	

The rate of capillary with respect to several void ratios or densities of the sample are shown in the figure 4.11;

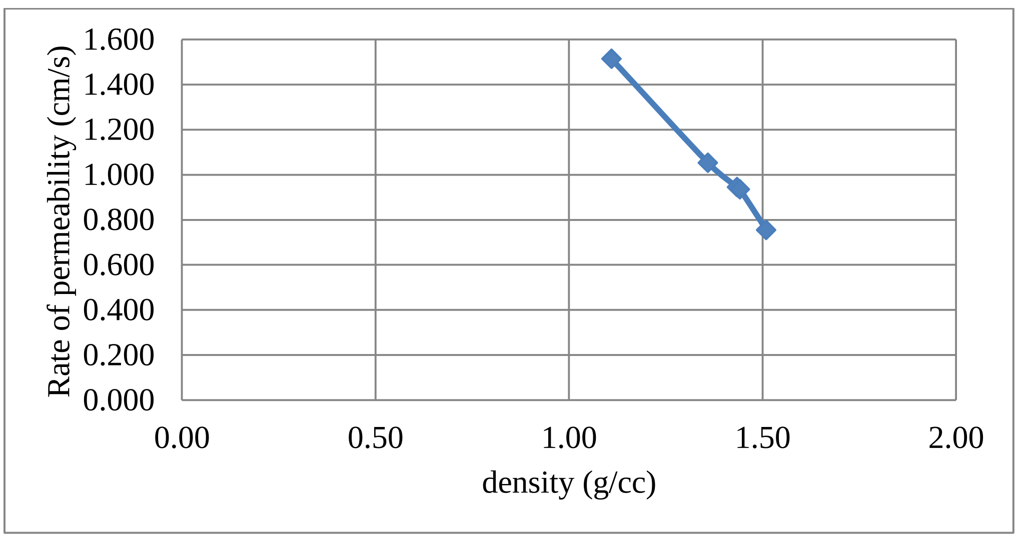


Figure 4.11: Relationship between the densities &permeability of type-2 soil at 35+000.

The result shown in a graph 4.11 indicates that again there is the propensity of a soil to allow the flow of water through the soil. From densities 1.44 (g/cc) to 1.11(g/cc) the rate flows is highly increasing due to high void spaces and saturation. And from densities (MDD to 1.44 (g/cc)) the rates of flows is slowly increasing due to decreasing in voids but the result indicates that at MDD there is still probable flow water which can change Subgrade Water Content (SWC) or (OMC) and further incremental of saturation due to capillary.

4.2.5. California Bearing Ratio Test results

The CBR test at different level of saturation (moisture content) due to capillary including OMC and at level compaction (56 blows) is investigated. The analysis are made to investigate the variation of CBR with respect to different type of soaking capillary saturation, which is from un soaked (day 0) to soaked (day 4) can be observed. The detail laboratory observation is attached in Appendix A.2.6.

Test conducted under OMC (28.00%) and MDD (1.51g/cc).

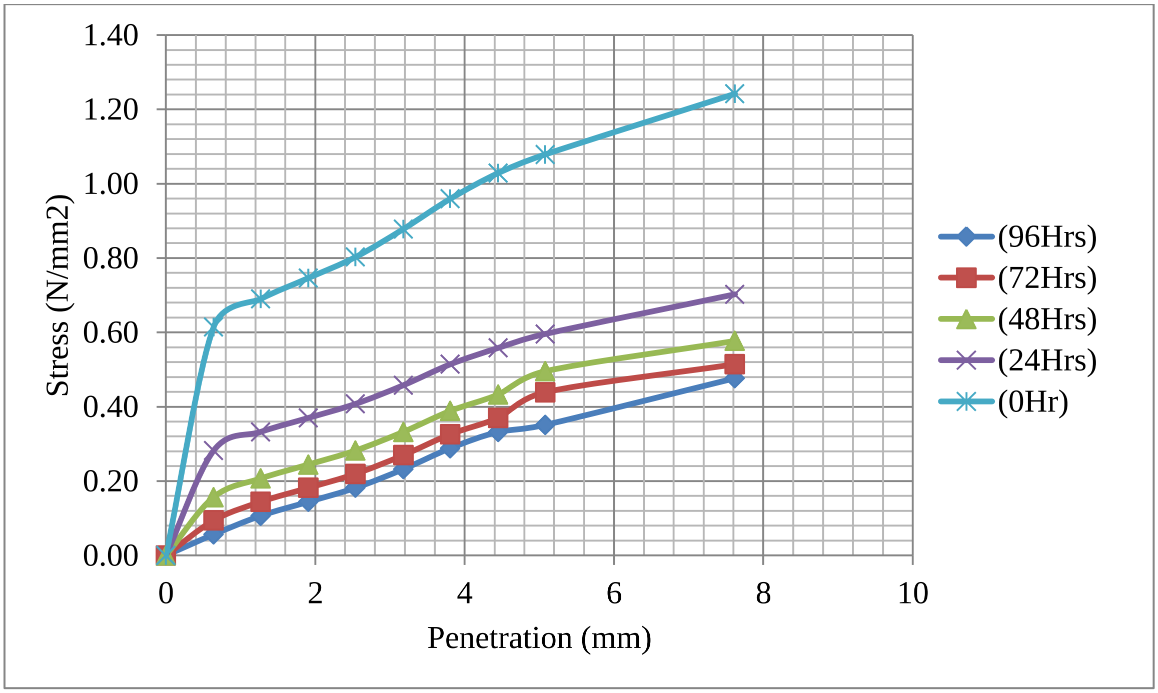


Figure 4.12: Stress vs penetration graph for type-2 soil at 35+000 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition.

The result shown in the above graph (4.12) observes that the soil can reducing the ability to withstands the stress as the saturation of the soil increasing. The CBR value or bearing capacity of subgrade soil decreasing.

4.2.6. Variation of CBR with time of soaking

The variation of CBR with respect to time of soaking or saturation due to capillary can be shown in the table 4.9:

Table 4.9: Variation of CBR with time of soaking for type-2 soil at 35+000.

Variation of CBR with time of soaking										
Time of soaking in hours										
Time	0Hr		24Hrs		48Hrs		72Hrs		96Hrs	
Pen(mm)	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08
CBR (%)	11.65	10.43	5.92	5.76	4.1	4.79	3.19	4.25	2.64	3.4

The CBR values decreasing with time of saturation increasing but sequence of decreasing are rapid in the first days (24hrs) saturation and slowly in the last three days (48, 72 and 96hrs) of saturation as indicated in table 4.11and figure 4.13. The moisture of subgrade soil increasing from OMC (at peak dry density obtained).

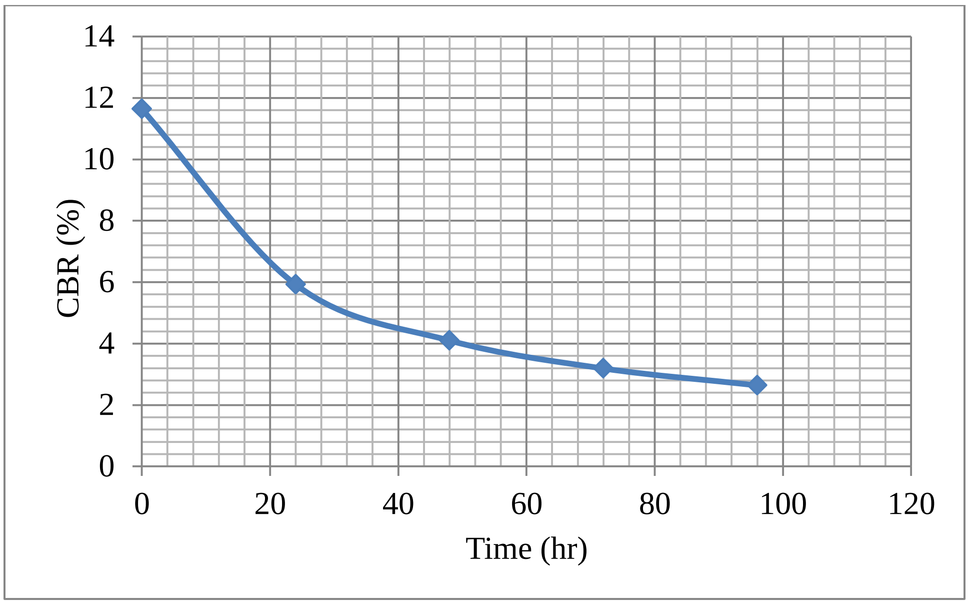


Figure 4.13: Variation of CBR with time of soaking for type- 2 soil 35+000.

4.2.7. Moisture variation in a soil sample

Moisture content variation in a soil sample with depths (top, middle and bottom) through the center axis shown previously in schematic diagrams and time of soaking (capillary saturation) are observed as shown in a table 4.10:

Table 4.10: Moisture variation in type-2 soil at 35+000 with depths and time of soaking.

Moisture variation in soil sample (%)						
Position	Axis	Time of soaking				
		0Hr	24Hrs	48Hrs	72Hrs	96Hrs
Top	Centre		11.5	12.52	13.6	13.85
Middle			11	11.3	11.9	12.4
Bottom			9.02	9.6	10.4	10.7

4.2.8. Variation of CBR with respect to moisture

The variation of CBR values with variation of moisture due to capillary saturation in a soil sample are shown in a figure 4.14:

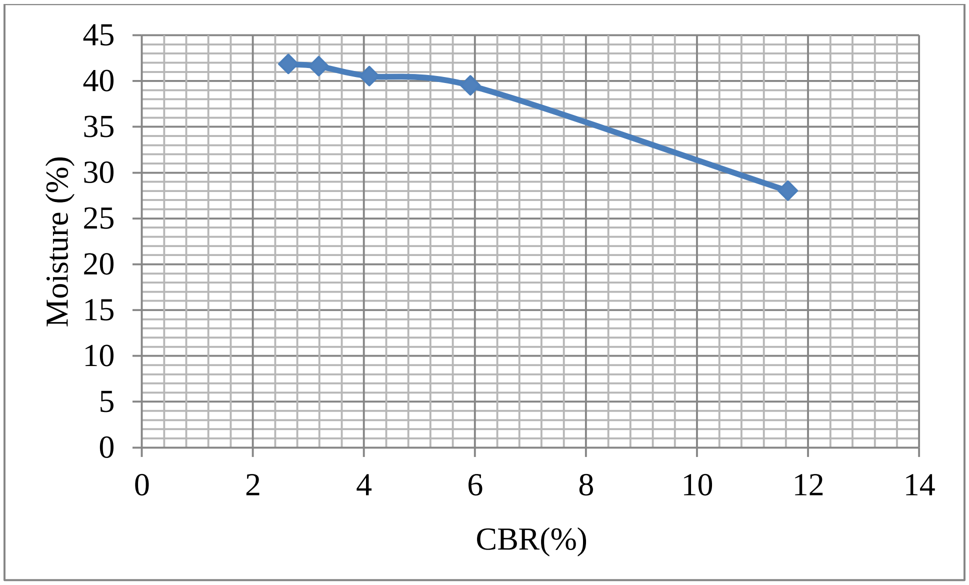


Figure 4.14: Variation of CBR with moisture for type-2 soil at 35+000.

The CBR values decreasing highly in the first day of saturation due to sudden change in moisture and gradually decreasing in the last three days due to no further change in moisture content shown in table 4.10.

4.3. Types of soil-3 (station 17+600)

4.3.1. Index Properties

The index properties such as Liquid limit, Plastic limit, Plasticity Index, Free swell index and specific gravity value are presented as follows;

The detail laboratory test result attached in Appendix A.3.1, 2 and 4.

Table 4.11: Index properties of type-3 soil at 17+600.

Index properties	Experimental value
Liquid Limit	52.37%
Plastic Limit	34.19%
Plasticity Index	18.18%
Specific Gravity	2.73
Free Swell Index	Non expansive

4.3.2. Particle size distribution

The grain size distribution of this soil sample done by sieve analysis test and plotting the graph as shown in figure 4.15;

The sieve analysis test result is under appendix A.3.3.

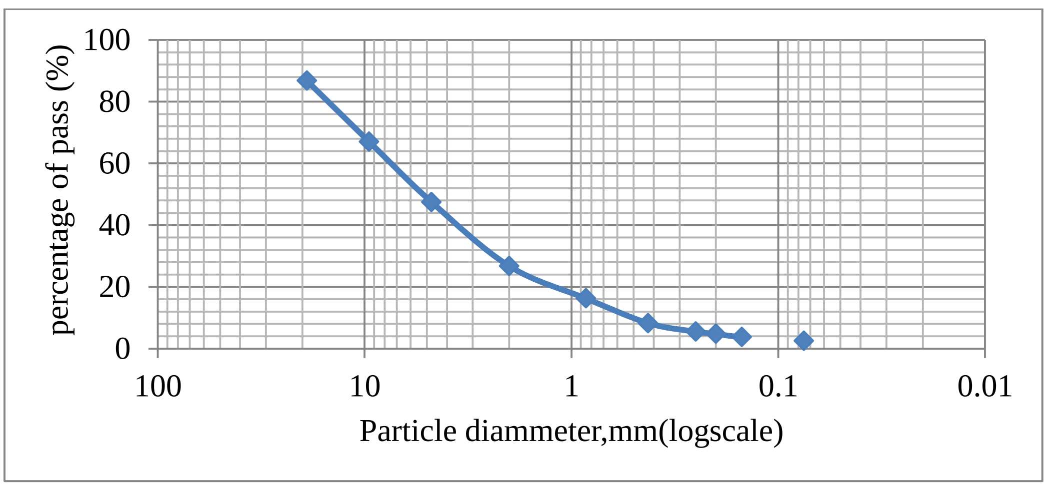


Figure 4.15: Particle size distribution curve of type-3 soil at 17+600.

This implies that in this soil sample 52.5 % of Gravel, 45 %of Sand and 2.5 % of Silt and/or Clay are exist and D_{10}, D_{30} and D_{60} values are 0.518mm, 2.431mm and 7.795mm respectively. The diameter size corresponding to 10% finer in the distribution indicate 0.518mm, this implies that there was probable flow of water through the soil.

The coefficient of uniformity (C_u) are 15.05 which is greater than 4 for gravel and less than 6 for sand, particle size are non- uniform and coefficient of curvature or gradation (C_c)is 1.46 which is between the range of 1 and 3 . This implies this subgrade soil is well graded soil. Percent of subgrade soil passing sieve size 0.075mm (F_{200}) is 2.5 % < 5% and ratio of R_4 to R_{200} is 0.54 > 0.5 and 52.5 % gravel exist, So subgrade soil is classified as Well- graded gravel with sand (GW).

4.3.3. Modified proctor compaction test

The results of modified proctor compaction test are represented in figure 4.16;

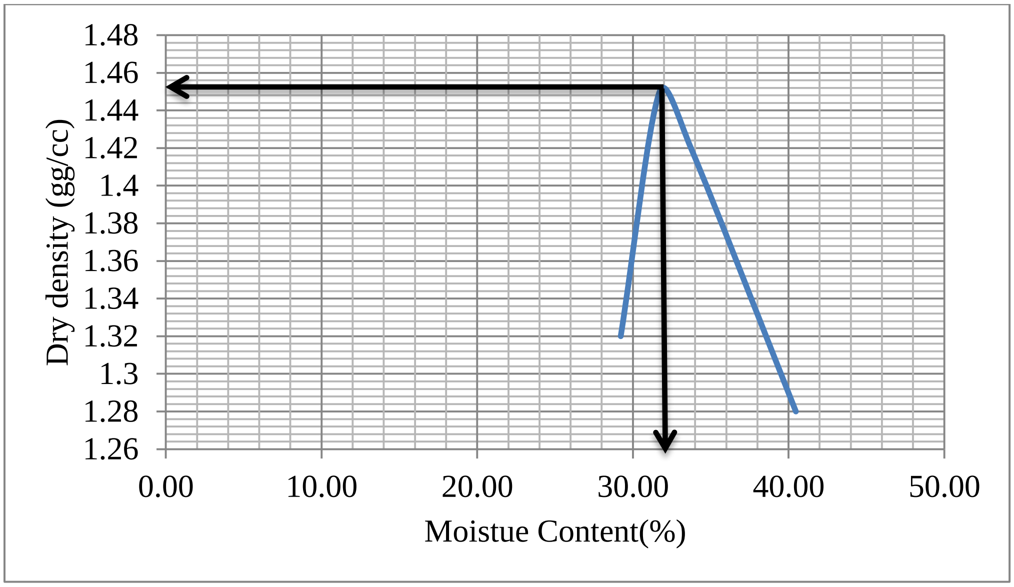


Figure 4.16: Modified proctor compaction test result of type-3 soil at 17+600.

From the figure 4.16, we can observes (OMC = 32.00% and $\gamma_D =$ 1.45g/cm^3)

4.3.4. Falling head permeability test result.

The specimens of different density or voids ratio are prepared for testing the flows water through them or permeability. Accordingly the specimen data which is ready for testing are shown in the table 4.12:

A. Specimen Data

Table 4.12: Specimen data for falling head test of type-3 soil at 17+600.

Specimen Mass (M) (kg)	1.015
Specimen Height, L (cm)	11.5
Specimen diameter, D (cm)	10.16
volume of specimen, V (cm^3)	931.87
Bulk density, (γ) (g/cc)	1.09
Water Content, w (%)	2.86
Dry density, (γ_{dry})	1.06
Specific gravity of soil, G_s	2.73
Initial void ratio,(e)	1.58

The specimen placed in a sink which water is about 3 cm above the cover and soaked at least for 24 hours. The sample will be saturated until minimum amount of entrapped air, discharge in equivalent to discharge out (fully saturated).

When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink again it is allowing for capillary rise in the tube.

Height of capillary from the mold (H_C) >= 6.5cm

Height of capillary from the top of the water (H_C) >= 3cm

B. Falling head test

The result of rate of capillary rise or coefficient of permeability can be determined at several densities or void ratio of this subgrade soil can be obtained. Using the same specimen data indicated before in table 4.14. The rate of capillary rise at initial void ratio of (e = 1.58) or dry density (γ_{dry}) of 1.06 g/cm^3 indicated in a table 4.12 is shown in the table 4.13:

Table 4.13: Rate of capillary rise at initial void ratio (e = 1.58) of type-3 soil at 17+600.

Trial	1	2
Head, h_o(cm)	82.2	84.7
Head,h_1 (cm)	41.8	42.3
Time, t (sec)	520	627
Temperature, T (^{o}c)	25	25
Permeability at T^oc, K_T	0.000220946	0.000179849
R_t for T	0.891033983	0.891033983
Permeability at 20^oC, K_{20}	0.000196871	0.000160252
Average K_{20} (cm/s)	0.000178561	

The rate of capillary rises or coefficient of permeability with respect to several void ratios or densities of the sample are shown in the figure 4.17;

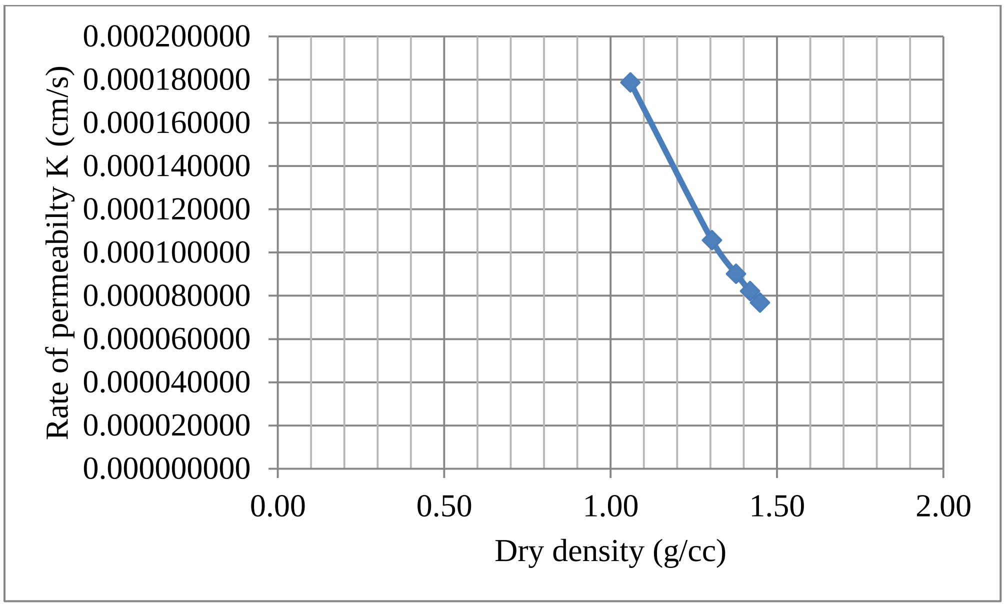


Figure 4.17:Relationship between the densities and permeability of type-3 soil at 17+600.

The result shown in a graph 4.17 indicates that again there is the propensity of a soil to allow the flow of water through the soil. From densities 1.38 (g/cc) to 1.06(g/cc) the rate flows is highly increasing due to high void spaces and saturation. And from densities (MDD to 1.38 (g/cc)) the rates of flows is slowly increasing due to decreasing in voids but the result indicates that at MDD there is still probable flow water which can change Subgrade Water Content (SWC) or (OMC) and further incremental of saturation due to capillary.

4.3.5. California Bearing Ratio Test results

The CBR test at different level saturation(moisture content) including OMC and at level compaction (56 blows) are investigated and the analysis are made to investigate the variation of CBR with respect to different type of soaking (capillary saturation) which is from un soaked (day 0) to soaked (day4) can be observed. The detail laboratory observation is attached in Appendix A.3.6.

Test conducted under OMC (32.00%) and MDD (1.45g/cc)

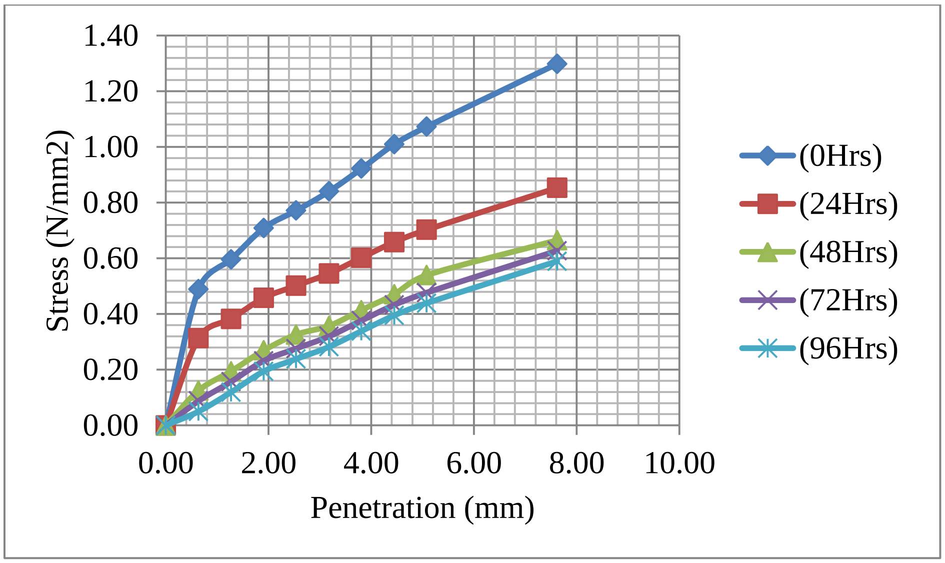


Figure 4.18: Stress vs penetration graph for type-3 soil at 17+600 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition.

4.3.6. Variation of CBR with time of soaking

The variation of CBR with respect to time of soaking or saturation due to capillary can be shown in the table 4.14:

Table 4.14: Variation of CBR with time of soaking for type-3 soil at 17+600.

Variation of CBR with time of soaking										
Time of soaking in hours										
Time	0Hr		24Hrs		48Hrs		72Hrs		96Hrs	
Pen(mm)	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08
CBR (%)	11.2	10.37	7.28	6.79	4.73	5.22	4.01	4.61	3.46	4.25

The CBR values decreasing with time of saturation increasing but sequence of decreasing are rapidly in the first two days (24hrs and 48hrs) and slowly decreasing in the last two days(72hrs and 96hrs).

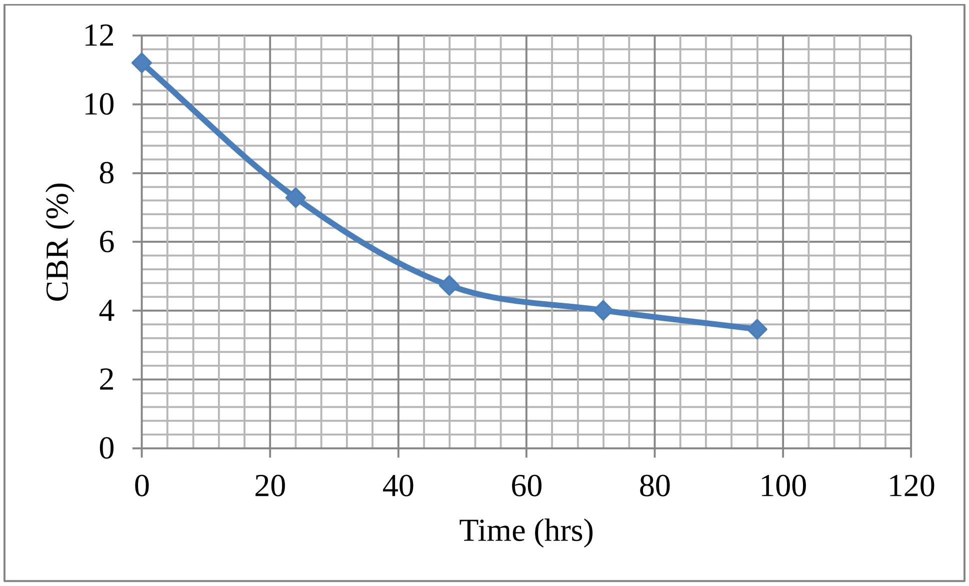


Figure 4.19: Variation of CBR with time of soaking for type-3 soil at 17+600.

4.3.7. Moisture variation in a soil sample

Moisture content variation in a soil sample with depths (top, middle and bottom) through the center axis shown previously in schematic diagrams and time of soaking (capillary saturation) are observed as shown table 4.15:

Table 4.15: Moisture variation in type-3 soil at 17+600 with depths and time of soaking.

Moisture variation in soil sample (%)						
Position	Axis	Time of soaking				
		0Hr	24Hrs	48Hrs	72Hrs	96Hrs
Top	Centre		13.65	14.7	15.01	15.36
Middle			13.46	14.32	14.92	15.1
Bottom			13.42	14.01	14.4	14.7

4.3.8. Variation of CBR with respect to moisture

The variation of CBR values with variation of moisture due to capillary saturation in a soil sample are shown in a figure 4.20:

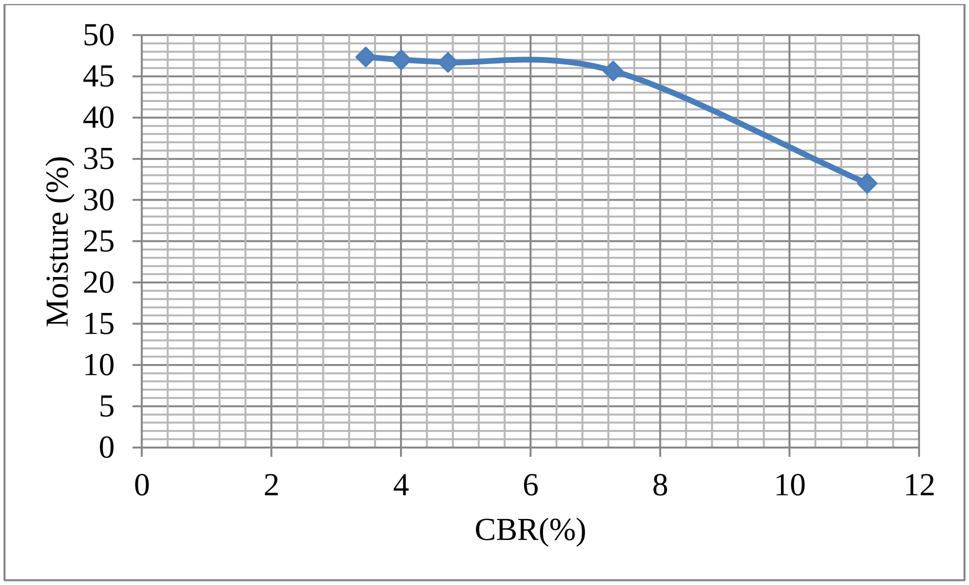


Figure 4.20: Variation of CBR with moisture for type-3 soil at 17+600.

The CBR values decreasing highly in the first two day of saturation due to sudden change in moisture and gradually decreasing in the last two days due to no further change in moisture content shown in table 4.15.

4.4. Type of soil 4 (station 30+500)

4.4.1. Index Properties

The index properties such as Liquid limit, Plastic limit, Plasticity Index, Free swell index and specific gravity value are presented as follows;

The detail laboratory test result attached in Appendix A.4.1, 2 and 4.

Table 4.16: Index properties of type -4 soil at 30+500.

Index properties	Experimental value
Liquid Limit	44.25%
Plastic Limit	31.57%
Plasticity Index	12.68%
Specific Gravity	2.77
Free Swell Index	Non expansive

4.4.2. Particle size distribution

The grain size distribution of this soil sample done by sieve analysis test and plotting the graph as shown in figure 4.21;

The sieve analysis test result is under appendix A.4.3.

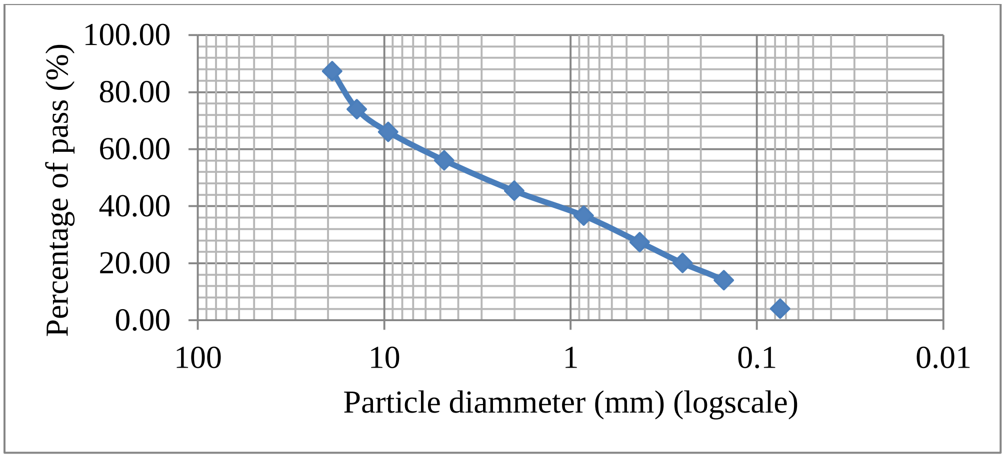


Figure 4.21: Particle size distribution curve of type-4 soil at 30+500.

The result shown in figure 4.21 implies that in this soil sample 44.00% of Gravel, 52.00%of Sand and 4.00%of Silt and/or Clay are exist and D_{10}, D_{30} and D_{60} values are 0.12mm, 0.55mm and 6.65mm respectively. The diameter size corresponding to 10% finer in the distribution indicate 0.12mm, this implies that there was probable flow of water through the soil.

The coefficient of uniformity (C_u) are 55.42 which is greater than 4 for gravel and less than 6 for sand, particle size are non- uniform and coefficient of curvature or gradation (C_c)is 0.38 which is out of the range between 1 and 3. This implies that this subgrade soil is Gap- graded soil. Percent of subgrade soil passing sieve size 0.075mm (F_{200}) is 4 % < 5% and ratio of R_4 to R_{200} is 0.46 < 0.5 and 44 % gravel exist, So subgrade soil is classified as Poorly - graded sand with gravel (SP).

4.4.3. Modified proctor compaction test

The results of modified proctor compaction test are represented in figure 4.22:

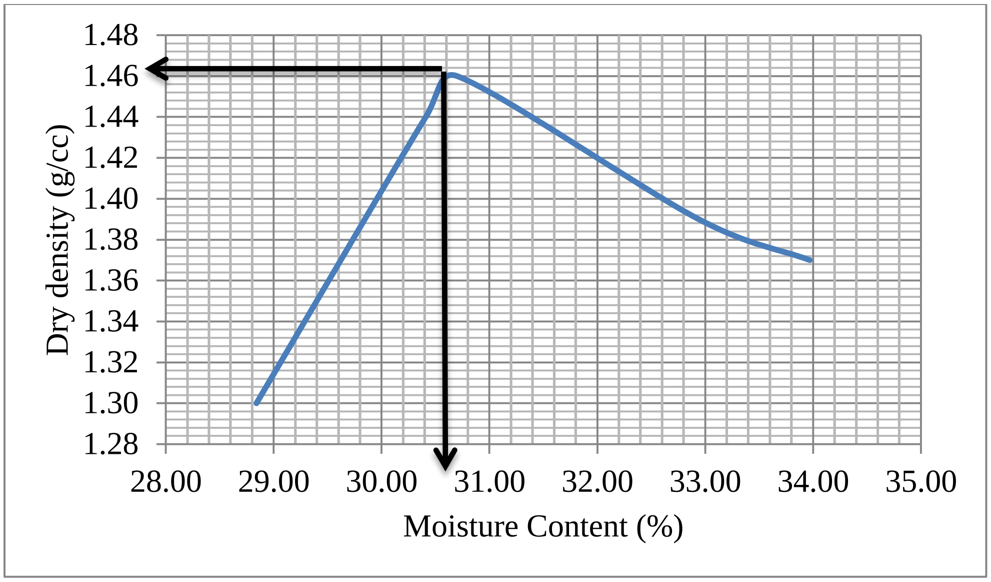


Figure 4.22: Modified proctor compaction test result of type-4 soil at 30+500.

From the figure 4.22 we can observe that (OMC = 30.70% and $\gamma_D =$ 1.462g/cm^3)

4.4.4. Falling head permeability test result.

The specimens of different density or voids ratio are prepared for testing the flows water through them or permeability. Accordingly the specimen data which is ready for testing are shown in the table 4.17:

A. Specimen Data

Table 4.17: Specimen data for falling head test of type -4 soil at 30+500.

Specimen Mass(M) (kg)	0.95
Specimen Height, L (cm)	11.5
Specimen diameter, D (cm)	10.16
volume of specimen, V (cm^3)	931.87
Bulk density, γ (g/cc)	1.02
Water Content, w (%)	6.36
Dry density,(γ_d)(g/cc)	0.96
Specific gravity of soil, G_s	2.77
Initial void ratio,(e)	1.89

The specimen placed in a sink which water is about 3 cm above the cover and soaked at least for 24 hours. The sample will be saturated until minimum amount of entrapped air, discharge in equivalent to discharge out (fully saturated).

When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink (allowing for capillary rise in the tube)

Height of capillary from the mold (H_C) >= 6.5cm

Height of capillary from the top of the water (H_C) >= 3.5cm

B. Falling head test

The result of rate of capillary rise or coefficient of permeability can be determined at several densities or void ratio of this subgrade soil can be obtained. Using the same specimen data indicated before in table 4.20. The rate of capillary rise at initial void ratio of (e = 1.89) or dry density (γ_{dry}) of 0.96 g/cm^3 indicated in above table 4.17 is shown in the table 4.18:

Table 4.18: Rate of capillary rise at initial void ratio of type-4 soil at 30+500.

Trial	1	2
Head, h_o(cm)	83.4	83.4
Head,h_1 (cm)	45.2	47.4
Time, t (s)	1200	1320
Temperature, T (^{o}c)	23.5	23
Permeability at T^{o}c, K_T	0.000220946	0.000146357
R_t for T	0.9204248	0.930640247
Permeability at 20^{o}C, K_{20}	0.000203364	0.000136205
Average K_{20} (cm/s)	0.000169785	

The rate of capillary rise or coefficient of permeability with respect to several void ratios or densities of the sample are shown in the figure 4.23;

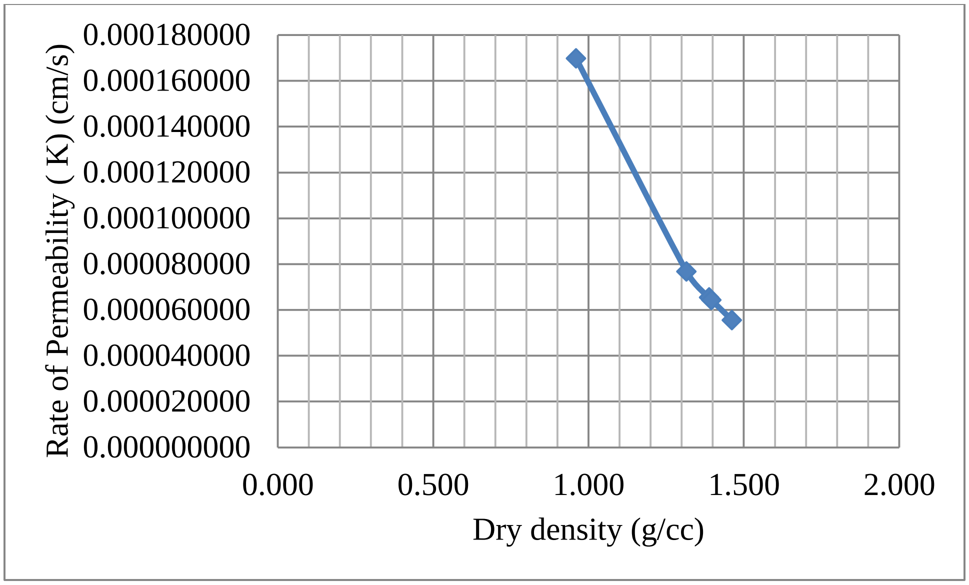


Figure 4.23: Relationship between the densities &permeability of type-4 soil at 30+500.

The result shown in a graph 4.23 indicates that again there is the propensity of a soil to allow the flow of water through the soil. From densities 1.389 (g/cc) to 0.96(g/cc) the rate flows is highly increasing due to high void spaces and saturation. And from densities (MDD to 1.389 (g/cc)) the rates of flows is slowly increasing due to decreasing in voids but the result indicates that at MDD there is still probable flow water which can change Subgrade Water Content (SWC) or (OMC) and further incremental of saturation due to capillary.

4.4.5. California Bearing Ratio Test results

The CBR test at different moisture content including OMC and at different level compaction (10, 30 and 65 blows) are investigated and the analysis are made to investigate the variation of CBR with respect to different type of soaking (capillary saturation), i.e. from un soaked (day 0) to soaked (day4) can be observed. The detail laboratory observation is attached in Appendix A.4.6.

Test conducted under OMC (30.7%) and MDD (1.462g/cc).

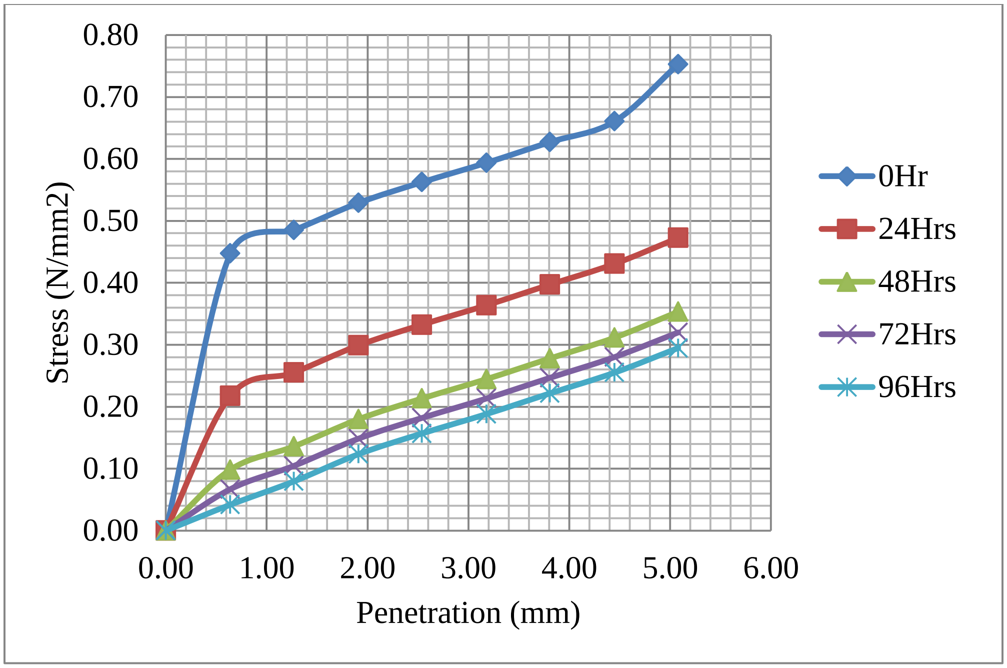


Figure 4.24: Stress vs penetration graph for type-4 soil at 30+500 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition.

4.4.6. Variation of CBR with time of soaking

The variation of CBR with respect to time of soaking or saturation due to capillary can be shown in the table 4.19:

Table 4.19: Variation of CBR with time of soaking for type-4 soil at 30+500.

Variation of CBR with time of soaking										
Time of soaking in hours										
Time	0Hr		24Hrs		48Hrs		72Hrs		96Hrs	
Pen(mm)	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08
CBR (%)	8.16	7.28	4.82	4.57	3.1	3.42	2.64	3.09	2.28	2.85

The CBR values decreasing with time of saturation increasing but sequence of decreasing are rapidly in the first two days (24hrs and 48hrs) and slowly decreasing in the last two days(72hrs and 96hrs).

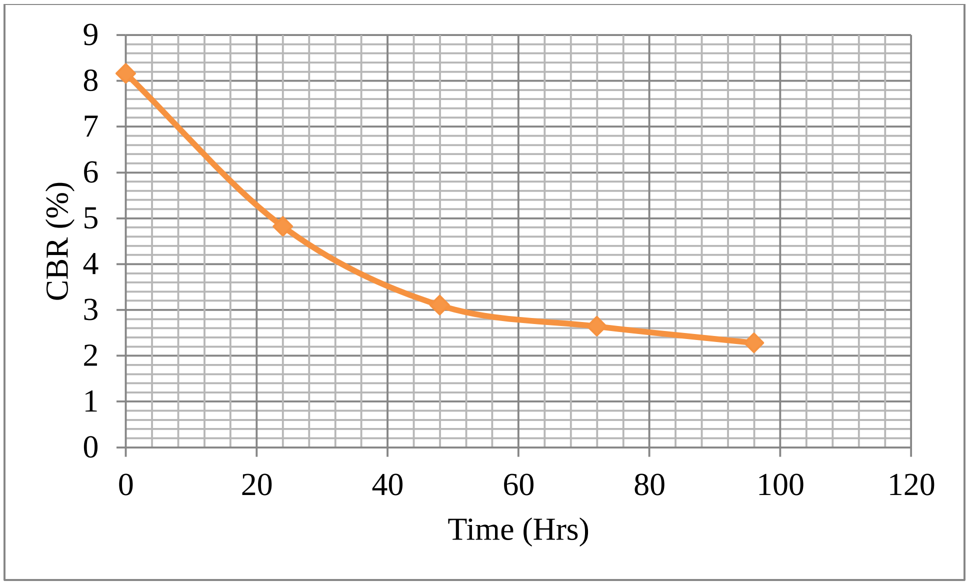


Figure 4.25: Variation of CBR with time of soaking for type-4 soil at 30+500.

4.4.7. Moisture variation in a soil sample

Moisture content variation in a soil sample with depths (top, middle and bottom) through the center axis shown previously in schematic diagrams and time of soaking (capillary saturation) are observed as shown table 4.20:

Table 4.20: Moisture variation in type-4 soil at 30+500 with depths and time of soaking.

Moisture variation in soil sample (%)						
Position	Axis	Time of soaking				
		0Hr	24Hrs	48hrs	72Hrs	96Hrs
Top	Centre		15.83	17.02	20.8	21.75
Middle			13.46	13.8	14.92	15.5
Bottom			10.92	11.45	12.3	13.3

4.4.8. Variation of CBR with respect to moisture

The variation of CBR values with variation of moisture due to capillary saturation in a soil sample are shown in a figure 4.26:

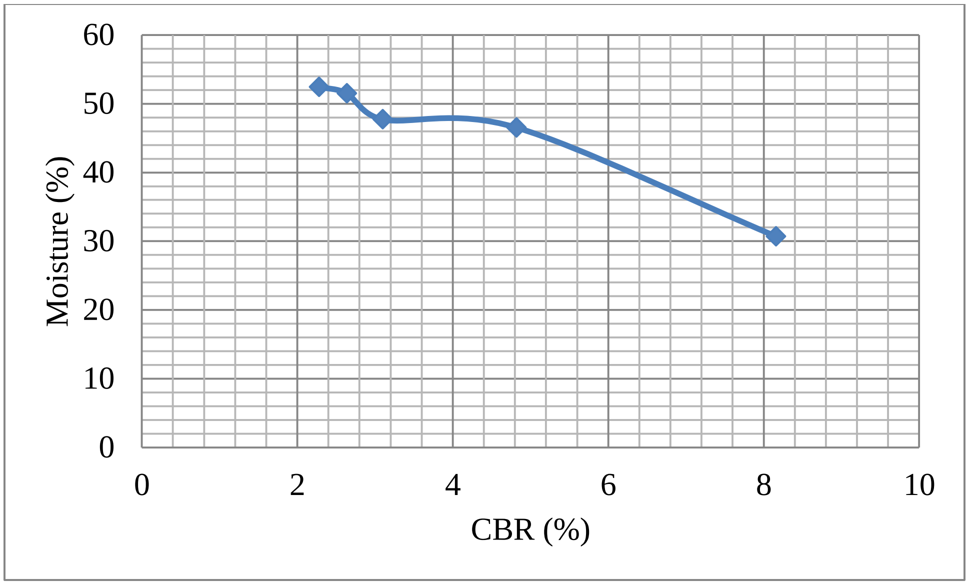


Figure 4.26: Variation of CBR with moisture for type-4 soil at 30+500.

The CBR values decreasing highly in the first two day of saturation due to sudden change in moisture and gradually decreasing in the last two days due to no further change in moisture content shown in table 4.20.

4.5. Type of soil 5 (station 11+800)

4.5.1. Index Properties

The index properties such as Liquid limit, Plastic limit, Plasticity Index, Free swell index and specific gravity value are presented as follows;

The detail laboratory test result attached in Appendix A.5.1, 2 and 4

Table 4.21: Index properties of type-5 soil at 11+800.

Index properties	Experimental value
Liquid Limit	52.44%
Plastic Limit	34.34%
Plasticity Index	18.10%
Specific Gravity	2.73
Free Swell Index	Non expansive

4.5.2. Particle size distribution

The grain size distribution of this soil sample done by sieve analysis test and plotting the graph as shown in figure 4.27;

The sieve analysis test result is under appendix A.5.3.

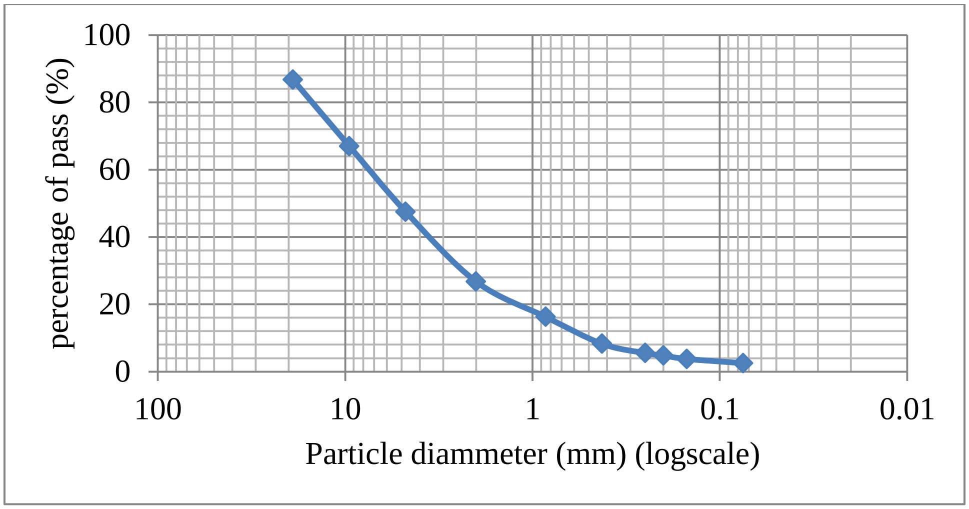


Figure 4.27: Particle size distribution curve of type-5 soil at 11+800.

The result shown above in a figure 4.27 implies that in this soil sample 52.50 % of Gravel, 45.00%of Sand and 2.50 %of Silt and/or Clay are exists and D_{10}, D_{30} and D_{60} values are 0.518mm, 2.431mm and 7.795mm respectively. The diameter size corresponding to 10% finer in the distribution indicate 0.518mm, this implies that there was probable flow of water through the soil.

The coefficient of uniformity (C_u) are 15.048 which is greater than 4 for gravel and less than 6 for sand, particle size are non- uniform and coefficient of curvature or gradation (C_c) is 1.464 which in the range between 1 and 3. This implies this subgrade soil is Well-graded soil. Percent of subgrade soil passing sieve size 0.075mm (F_{200}) is 2.5% < 5% and ratio of R_4 to R_{200} is 0.538 > 0.5 and 52.5 % gravel exist, So subgrade soil is classified as Well - graded gravel with sand (GW).

4.5.3. Modified proctor compaction test

The results of modified proctor compaction test are represented in figure 4.28;

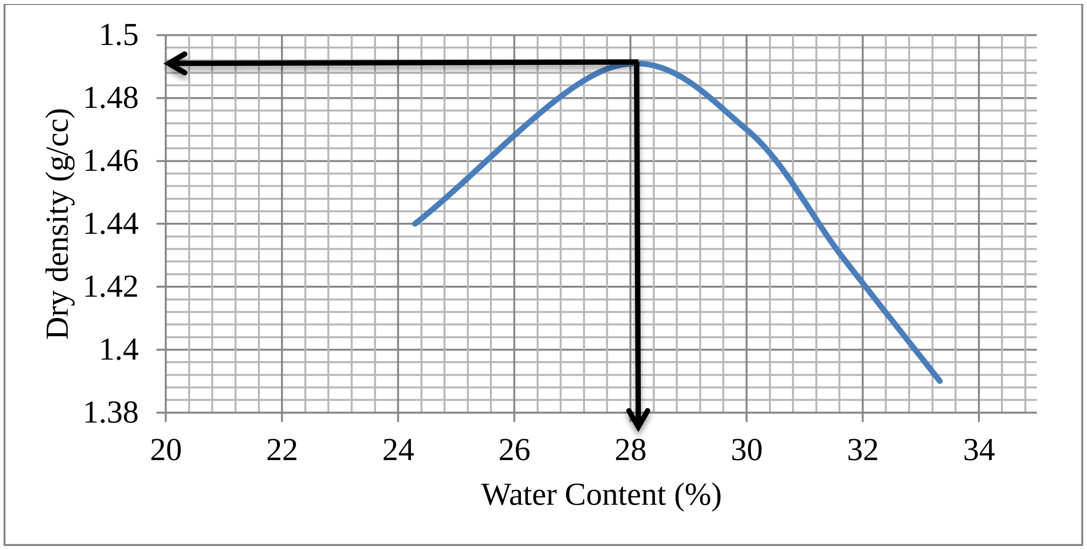


Figure 4.28: Modified proctor compaction test result of type-5 soil at 11+800.

From the above figure we can observe (OMC = 28.20% and $\gamma_D = 1.49 g/cm^3$)

4.5.4. Falling head permeability test result.

A. Specimen Data

Table 4.22: Specimen data for falling head test of type-5 soil at 11+800.

Specimen Mass (M) (kg)	1.06
Specimen Height, L (cm)	11.5
Specimen diameter, D (cm)	10.16
volume of specimen, V (cm^3)	931.87
Bulk density,(γ)(g/cc)	1.14
Water Content, w (%)	3.14
Dry density, (γ_d) (g/cc)	1.10
Specific gravity of soil, G_s	2.73
Initial void ratio,(e)	1.48

The specimen placed in a sink which water is about 2 cm above the cover and soaked at least for 24 hours. The sample will be saturated until minimum amount of entrapped air, discharge in equivalent to discharge out (fully saturated).

When water in the plastic inlet tube on the top of the mold reaches equilibrium with water in the sink (allowing for capillary rise in the tube)

Height of capillary from the mold (H_C) >= 4.3cm

Height of capillary from the top of the water (H_C) >= 2.3cm

B. Falling head test

The result of rate of capillary rise or coefficient of permeability can be determined at several densities or void ratio of this subgrade soil can be obtained. Using the same specimen data indicated before in table 4.25. The rate of capillary rise at initial void ratio of (e = 1.48) or dry density (γ_{dry}) of 1.10 g/cm^3 indicated in table 4.22 is shown in the table 4.23:

Table 4.23: rate of capillary rise at initial void ratio of type-5 soil at 11+800.

Trial	1	2
Head, h_o(cm)	88.8	88.7
Head, h_1 (cm)	52.3	46.7
Time, t (sec)	60	75
Temperature, T (^{o}c)	25.5	25.5
Permeability at T^{o}c, K_T	0.000220946	0.00016617
R_t for T	0.881627735	0.881627735
Permeability at 20^{o}C, K_{20}	0.000194792	0.0001465
Average K_{20} (cm/s)	0.000170646	

The rate of capillary or coefficient of permeability with respect to several void ratios or densities of the sample are shown in the figure 4.29;

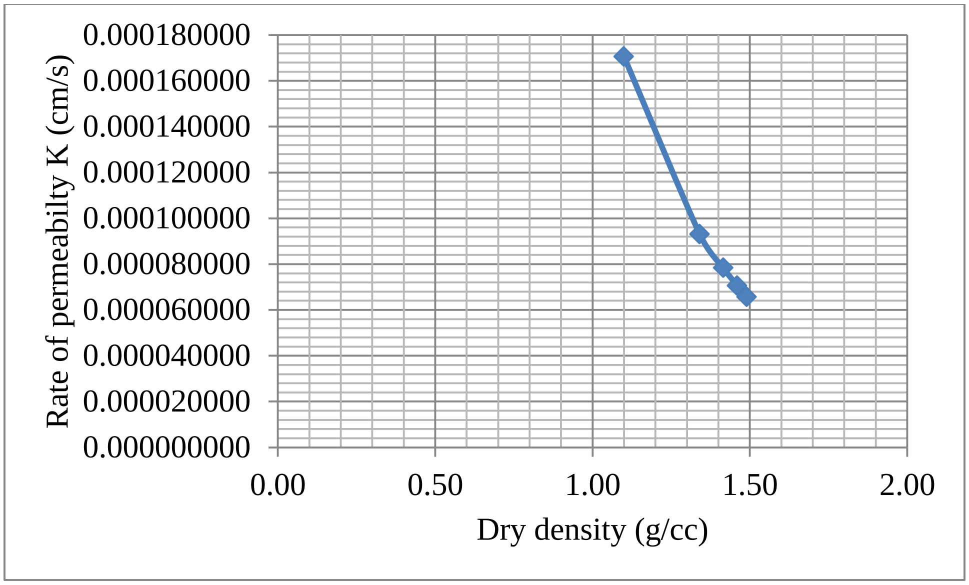


Figure 4.29: Relationship between the densities and permeability of type-5 soil at 11+800.

The result shown in a graph 4.29 and in a table 4.27 indicates that again there is the propensity of a soil to allow the flow of water through the soil. From densities 1.42 (g/cc) to 1.10 (g/cc) the rate flows is highly increasing due to high void spaces and saturation. And from densities (MDD to 1.42 (g/cc)) the rates of flows is slowly increasing due to decreasing in voids but the result indicates that at MDD there is still probable flow water which can change Subgrade Water Content (SWC) or (OMC) and further incremental of saturation due to capillary.

4.5.5. California Bearing Ratio Test results

The CBR test at different moisture content including OMC and at different level compaction (10, 30 and 65 blows) are investigated and the analysis are made to investigate the variation of CBR with respect to different type of soaking (capillary saturation), i.e from un soaked (day 0) to soaked (day 4) can be observed. The detail laboratory observation is attached in Appendix A.5.6.

Test conducted under OMC (28.20%) and MDD (1.49g/cc)

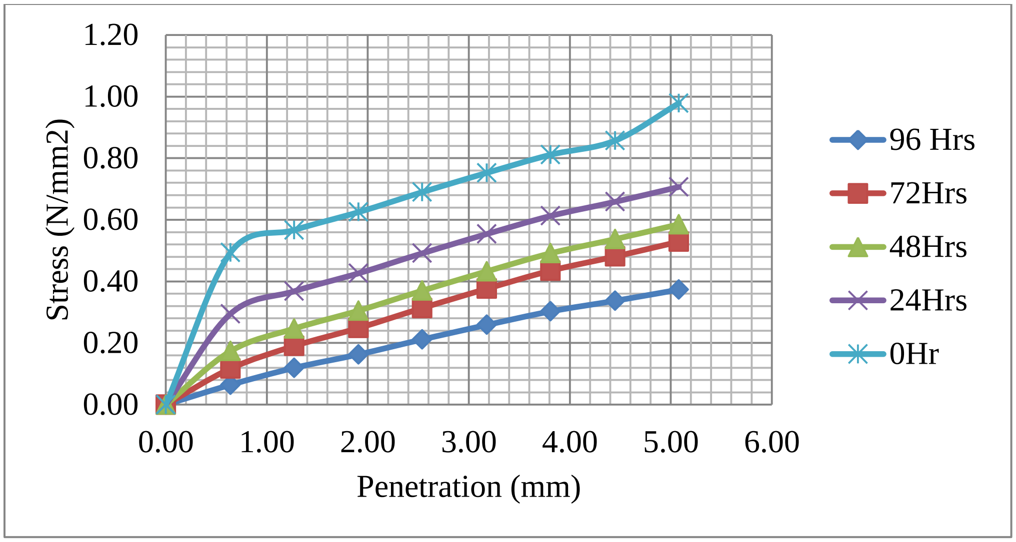


Figure 4.30: Stress vs penetration graph for type-5 soil at 11+800 for soaked (0hr, 24hrs, 48hrs, 72hrs and 96hrs) condition.

4.5.6. Variation of CBR with time of soaking

The variation of CBR with respect to time of soaking or saturation due to capillary can be shown in the table 4.24:

Table 4.24: Variation of CBR with time of soaking for type-5 soil at 11+800.

Variation of CBR with time of soaking										
Time of soaking in hours										
Time	0Hrs		24Hrs		48Hrs		72Hrs		96Hrs	
Pen(mm)	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08	2.54	5.08
CBR (%)	10.01	9.46	7.13	6.83	5.37	5.66	4.55	5.12	3.07	3.61

The CBR values decreasing with time of saturation increasing but sequence of decreasing are rapidly in the first two days (24hrs and 48hrs) and slowly decreasing in the last two days(72hrs and 96hrs).

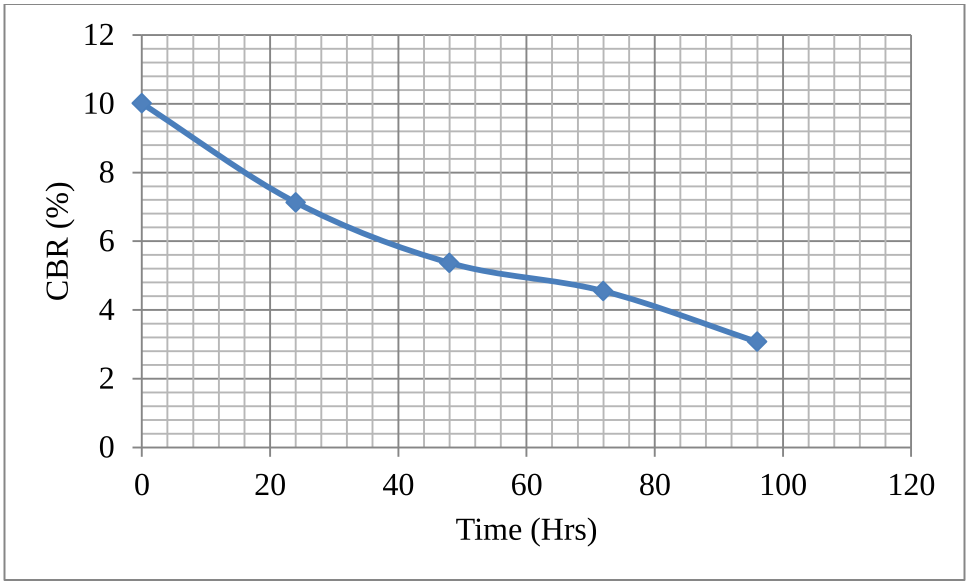


Figure 4.31: Variation of CBR with time of soaking for type -5 soil at11+800.

4.5.7. Moisture variation in a soil sample

Moisture content variation in a soil sample with depths (top, middle and bottom) through the center axis shown previously in schematic diagrams and time of soaking (capillary saturation) are observed as shown:

Table 4.25: Moisture variation in type-5 soil at 11+800 with depths and time of soaking.

Moisture variation in soil sample (%)						
Position	Axis	Time of soaking				
		0hr	24Hrs	48Hrs	72Hrs	96Hrs
Top	Centre		13.2	17.94	20.48	21.97
Middle			9.7	10.9	11.2	12.3
Bottom			9.02	9.7	10.4	11.0

4.5.8. Variation of CBR with moisture

The variation of CBR values with variation of moisture due to capillary saturation in a soil sample are shown in a figure 4.32:

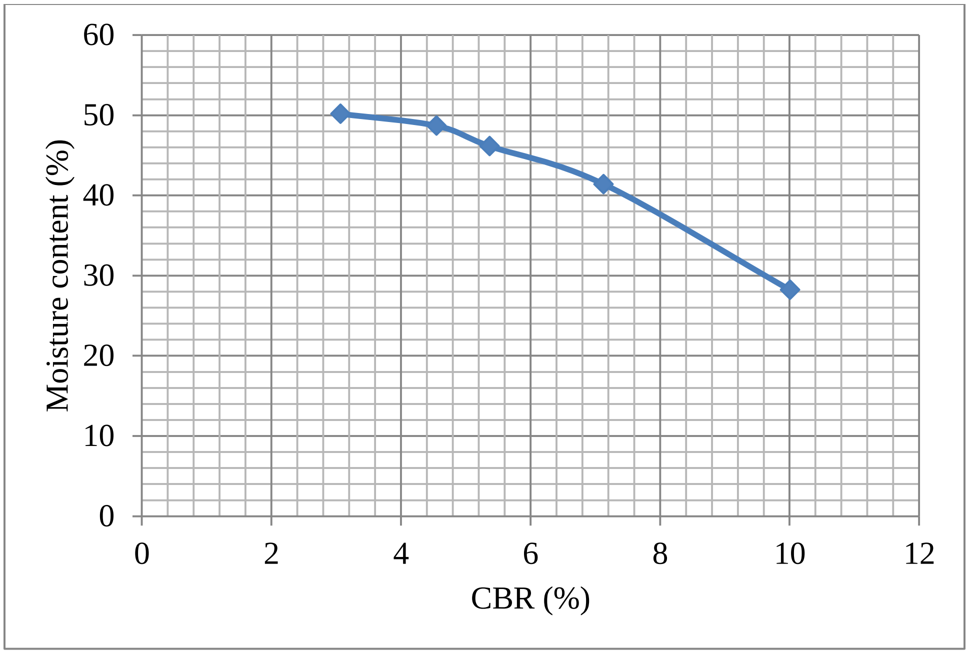


Figure 4.32: Variation of CBR with moisture for type-5 soil at 11+800.

The CBR values decreasing highly in the first two day of saturation due to sudden change in moisture and gradually decreasing in the last two days due to no further change in moisture content shown in table 4.25.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1. Conclusion

An attempt has been made in this research are to explore effect of capillary rise/saturation on the subgrade properties of soil, namely the capillary rise/height through the subgrade soil, the hydraulic conductivity of subgrade soil with varies densities or voids and the strength properties of subgrade soil in terms of the most widely parameter for pavement design as CBR, i.e. with varies soaking on the strength properties of subgrade soil are considered. For all five types of subgrade soil at different station, the effects of capillary rise/saturation have been considered in this research. From the result and discussion presented earlier the following conclusions are drawn:

- It observed that for each type of soil (at each station) the subgrade soil are conductive to water flow in its height of capillary rise/saturation which are greater than or equal to 4.3cm, 8cm, 6.5cm, 6.5cm and 4.3cm respectively.
- Rate of capillary rise or hydraulic conductivity of subgrade soil with any densities exists at each station soil sample. The rate of conductivity rapidly increasing from 90 % of MDD to the initial dry density taken and conductive slowly from 95 % of dry density to the MDD.
- It also observed that CBR values decrease with time of soaking or capillary saturation in each type (at each station) soil, the rate of reduction of CBR values can be varies for each type of soil. For first type of soil at station 25+200 the CBR values rapidly decrease in the first two days and slowly decrease in the last two days of saturation (soaking). While CBR values reduced by 5.76 times (82%) to the un soaked condition. For second type of soil at station 35+00 the CBR values rapidly in the first day and slowly decrease in the last three days, while CBR values reduces by 4.41 times (77.30 %) to the un soaked condition. For the third type of soil at station 17+600 the CBR values decreases in the first two days and slowly decrease in the last two days, while CBR values reduces by 3.24 times (69.11 %) to the un soaked condition. For fourth type of soil at station 30+500 the CBR values rapidly decrease in the first day and slowly decrease in the last three days, while CBR values reduces by 3.57 times (72.1 %) to the un soaked condition and for the last or fifth soil type at

station 11+800 the CBR values rapidly decrease in first two days and slowly decrease in the last two days of soaking. While CBR values reduced by 3.26 times (69.33%) to the un soaked condition.

- It can also observe that the variation of moisture content in a soil sample with capillary saturation or time of soaking. The rate of variation is high in the first two days and minimum changing in the last two days.

Generally capillary saturation changes the Subgrade Moisture Content (SWC) or Optimum Moisture Content (OMC) and reduces the ability to the soil to withstand stress or reduce subgrade strength.

5.2. Recommendation

- For Ethiopian Road Authority (ERA), it is better in the future construction or during maintenance of this road and others to provide a subgrade soil with non-susceptible to hydraulic conductivity, at least in the height of capillary rise / saturation in the areas of sensitive. Provide the subgrade soil with a minimum diameter of grain size with corresponding to 10 % finer (weight) in the distribution to overcome flow of water through them that can cause adverse effects on the strength properties of subgrade soil.
- During the construction or maintenance of this road or any others our hosting university can be gave a better consultation on the selection of proper material of subgrade soil that can reduces the flows of water through them and placing it at appropriate station with adequate depths in the road cross section and deals with further investigation or researches on the effects of capillary saturation on the strength properties of subgrade soil.
- Further study should be carried out on the other factors that cause variation of moisture or saturation in a subgrade soil.
- Further study should be carried out on the depth of ground water and its fluctuation at different season.

REFERENCES

Alayaki, F.M., Bajomo, O. S. (2011), Effect of Moisture Variation on the Strength Characteristics of Laterite soil. Proceedings of the *Environmental Management Conference,* Federal University of Agriculture, Abeokuta, Nigeria.

Ampadu, S, I. K. (2006), "Laboratory Investigation into the Effects of Water Content on the CBR of a Subgrade soil".

Bowles J.E. Foundation Analysis and design, fifth edition, us, The McGraw- Hill companies, Inc.1996.

ERES Division (2001), "Correlation of CBR values with soil index properties", Illinois.

Fasil, A., (2003). Investigation into some of the engineering properties of red clay soil in Bahirdar. M. Sc. thesis, A.A.U., Ethiopia.

Hazen, A. (1930). "Water supply." American Civil engineers handbook, Wiley, New York.

Jaleel, T. Z. (2011), Effects of Soaking on the CBR-Values of Sub Base Soil. *Eng. and Tech. journal,* Vol.29.

Lu, T.X et al (1995), "Water movement in glass bead porous media 3. Theoretical analyses of capillary rise into initially dry media." Water Resourses.,31(1), 11-18

Malik, R. S.et al (1989). "Maximal Capillary Rise flux as a function of height from the water table." Soil Sci., 148 (5), 322-326

Ningish, S.R (2012), Correlation between index properties and California bearing ratio test of pekanbaru soils with and without soaked. *Canadian Journal on Environmental, Construction and Civil Engineering Vol. 3,* Indonesia.

Parlange, J. Y.et al (1990), "Maximal Capillary rise flux as function of height from the water table." Soil Sci., 150 (6), 896-898.

Rahman, Q.B.A (2010), "Correlation between CBR results and physical properties of soils", Malaysia.

Singih. D (2011), Moisture and Compaction Based Statistical Mode for Estimating CBR of Fine Grained Subgrade Soils. *International Journal of Earth Science and Engineering,* pp 100-103

Stephens, D. B (1996). Vadose Zone hydrology, Chemical Rubber Corp., Boca Reton, Fla.

Terzaghi. K (1943). Theoretical Soil Mechanics, John Wiley and Sons, New York.

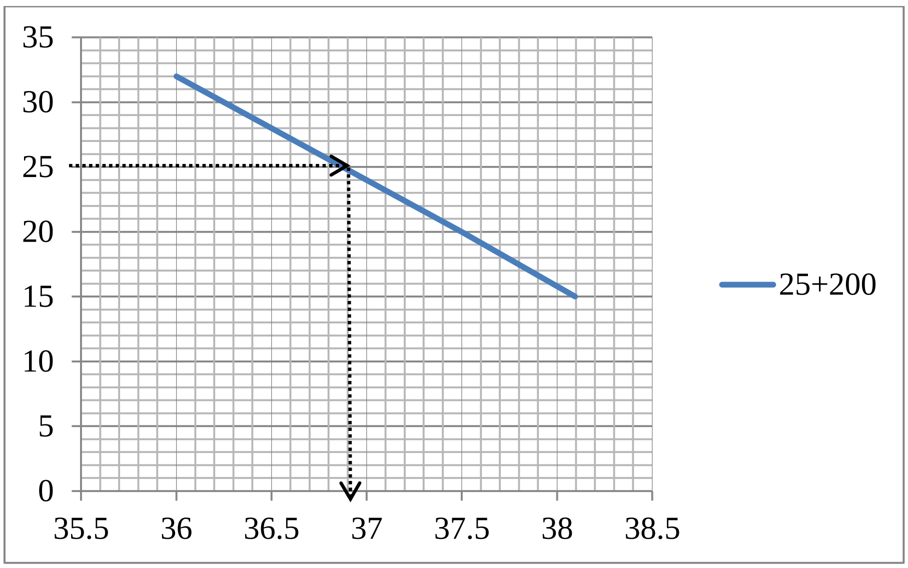
Yoder, E. J and M. W. Witezek (1975), Principle of Pavement Design, 2nd. , John Wiley and Sons, New York.

Appendix A: Test Outputs

A.1: For type 1(station 25+200) soil.

A.1.1: Liquid limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	LB4	W1	48
Mass of can	17	17	18
No of blows	32	20	15
Mass of can and wet soil	51	39	47
Mass of can and dry soil	42	33	39
WATER mass (%)	36	37.50	38.10

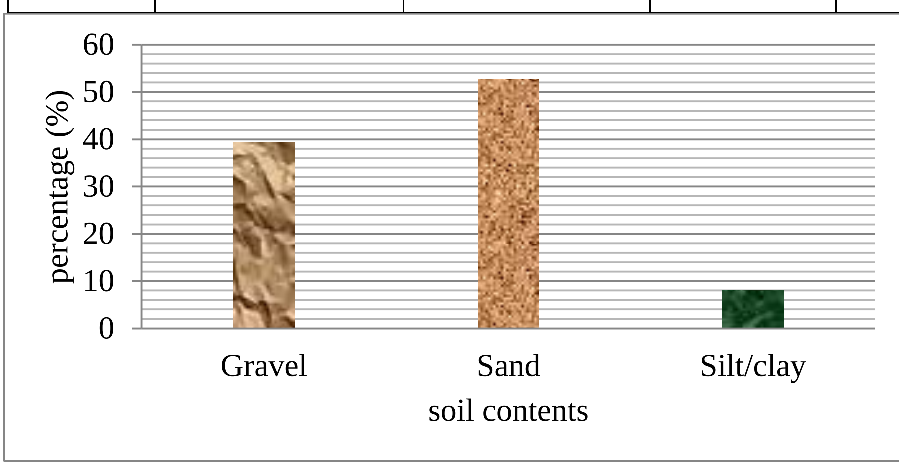


A.1.2: Plastic limits

Trial No	Trial-1	Trial-2	Trial-3
Name of can	44	A1	K60
Mass of can	18	18	17
Mass of can and wet soil	35	35	29
Mass of can and dry soil	32	33	27
Water content (%)	21.43	13.33	20.00
Plastic Limits (%)	18.25		

A.1.3: Particle size distribution.

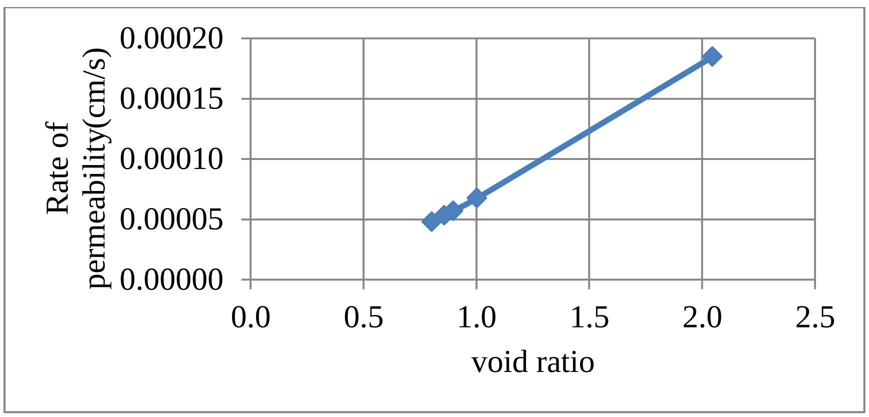
IS Seive No.(mm)	Weight retained in gm.	Percentage Weight retained	Cumulative Retained	Percentage weight passing
19	375	15	15	85
9.5	315	12.6	27.6	72.4
4.75	295	11.8	39.4	60.6
2	475	19	58.4	41.6
0.85	360	14.4	72.8	27.2
0.425	295	11.8	84.6	15.4
0.25	95	3.8	88.4	11.6
0.2	30	1.2	89.6	10.4
0.15	25	1	90.6	9.4
0.075	35	1.4	92	8
0	200	8	100	0
Total Mass	2500	100		



A.1.4: Specific gravity

Sample Name	L	K	H
Mass of Pycnometer, Mp	94.499	117.814	92.992
Mass of Pycnometer + Soil, Mps	130	158	161
Mass of Pycnometer + Soil + Water, Mpws	365	392	384
Mass of Pycnometer + Water, Mpw @ Ti	343	367	342
The water temprature, Ti	23	23	23
Temperature of contents of Pycnometer When Mpws was taken, Tx	25.5	25.5	25
Mass of Dry Soil, Ms	35.501	40.186	68.008
Density of water at Ti, ρW @ Ti	0.99757	0.99757	0.99757
Density of water at Tx ρW @ Tx	0.99694	0.99694	0.99707
$Mpw(at\ Tx) = \left[\frac{\rho w@Tx}{\rho w@Ti}\right][Mpw@Ti - Mp] + Mp$	342.8431	366.8426	341.8752
Conversion factor, K	0.99681	0.99681	0.99681
Specific Gravity, @ 20oc Gs = $k\left[\frac{Ms}{Ms+Mpw(at\ Tx)-Mpws}\right]$	2.65	2.67	2.62
AVERAGE		2.65	

A.1.5: Rate of permeability at different voids of soil sample.



A.1.6: CBR test result at different time of soaking.

A.1.6.1: Un soaked condition (0Hr)

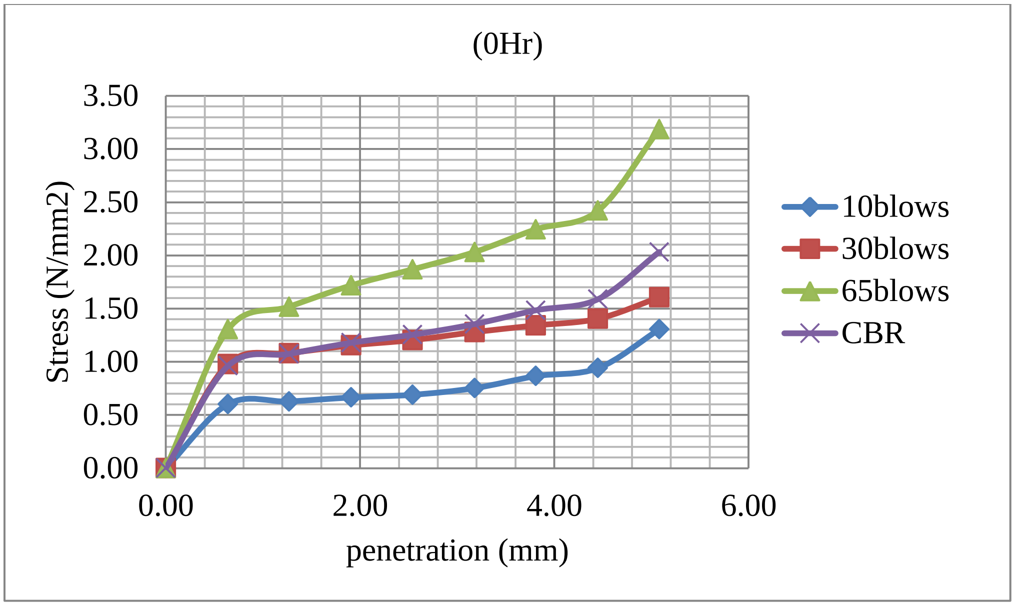
Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	153.33	1.86	0.96	
0.050	1.27	171.33	2.08	1.07	
0.075	1.91	188.00	2.28	1.18	
0.100	2.54	200.00	2.43	1.25	18.21

0.125	3.18	216.00	2.62	1.35	
0.150	3.81	236.67	2.87	1.48	
0.175	4.45	253.33	3.08	1.59	
0.200	5.08	324.00	3.93	2.03	19.65
0.300	7.62				
0.400	10.16				
0.500	12.70				

A.1.6.2: Soaked condition (96Hrs)

Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	11.33	0.14	0.07	
0.050	1.27	20.33	0.25	0.13	
0.075	1.91	28.33	0.34	0.18	
0.100	2.54	34.67	0.42	0.22	3.16
0.125	3.18	42.33	0.51	0.27	
0.150	3.81	50.33	0.61	0.32	
0.175	4.45	57.00	0.69	0.36	
0.200	5.08	63.00	0.76	0.40	3.82
0.300	7.62				
0.400	10.16				
0.500	12.70				

A.1.6.3 Stress versus Penetration graph and Analysis of CBR at 2.54mm and 5.08mm penetration



CBR corresponding 2.54 mm penetration = $\frac{\text{Test stress}}{\text{standard stress}} * 100$

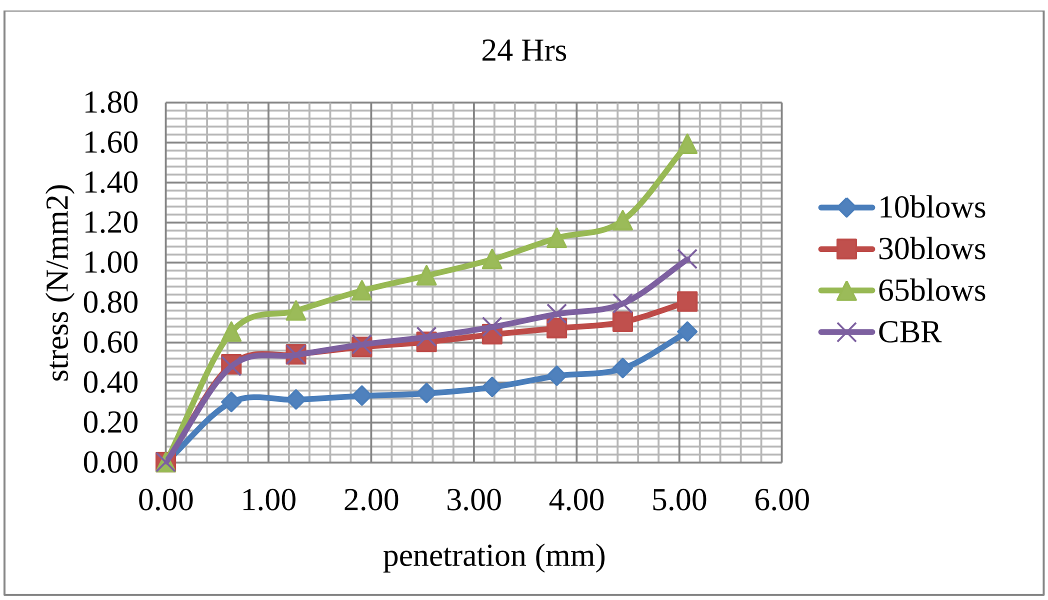
$= \frac{1.25}{6.89} * 100$

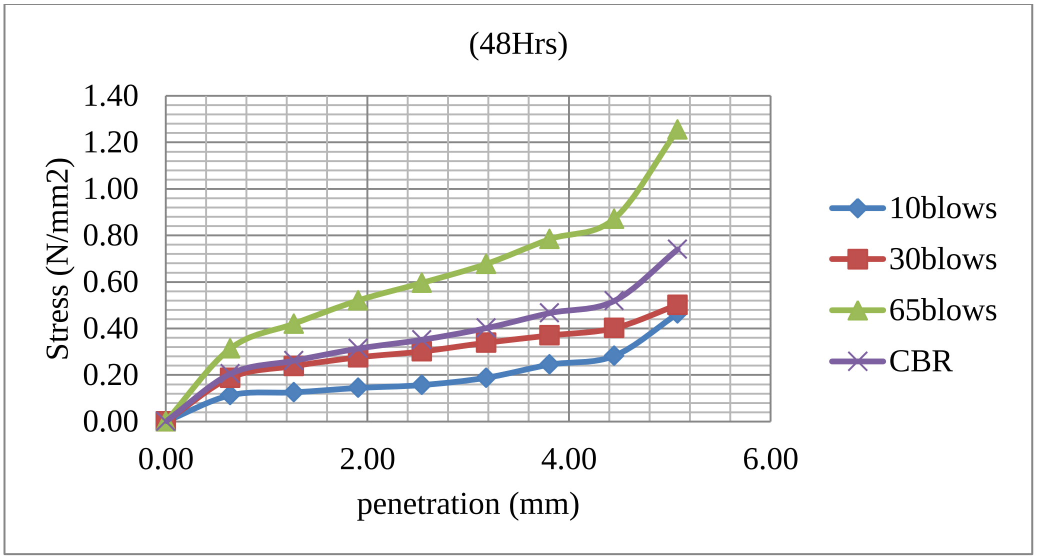
$= 18.21\%$

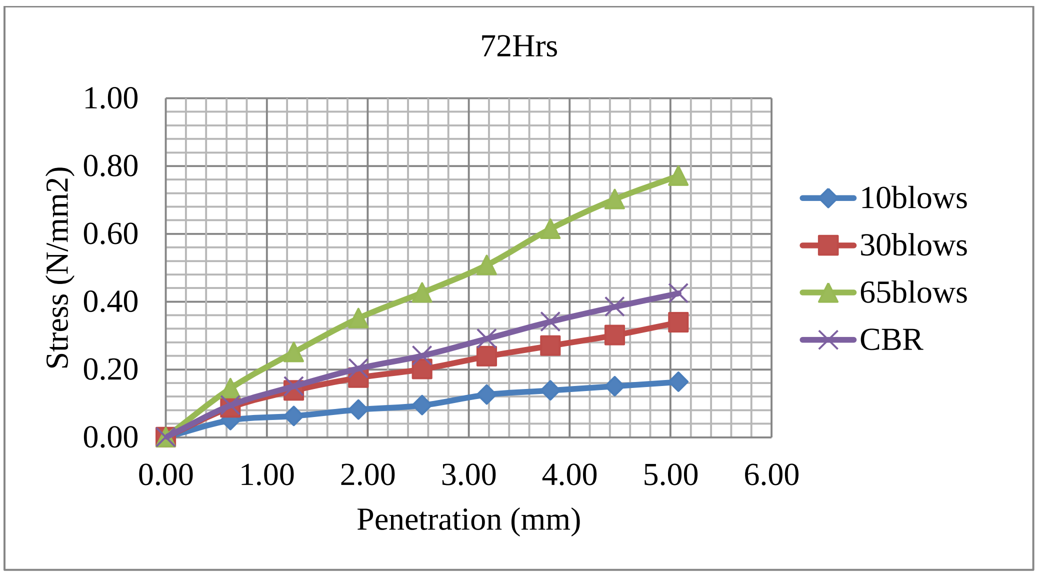
CBR corresponding 5.08 mm penetration = $\frac{\text{Test stress}}{\text{standard stress}} * 100$

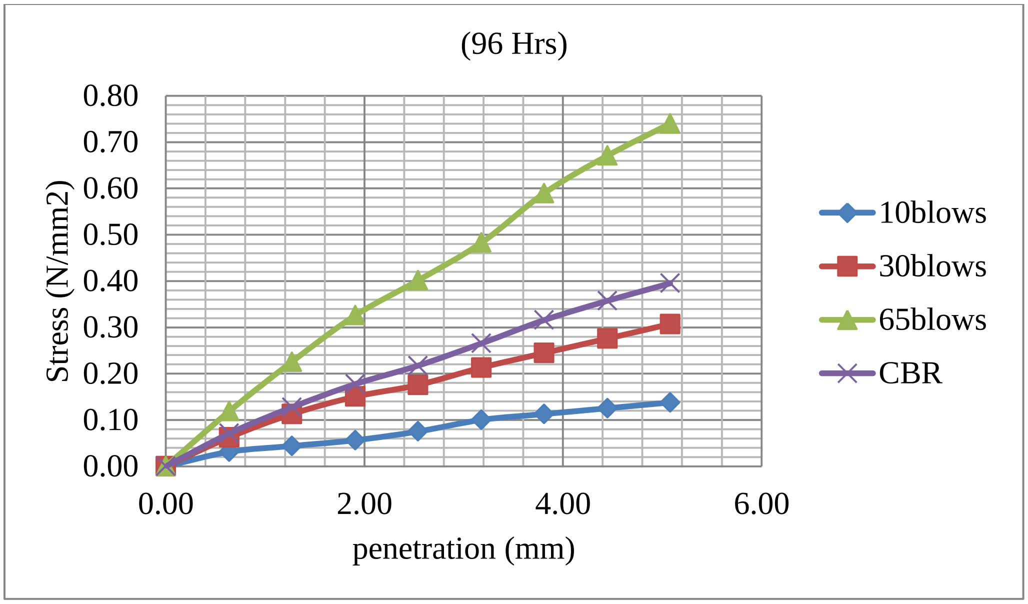
$= \frac{2.03}{10.34} * 100$

$= 19.65\%$




(48Hrs)
Stress (N/mm2)
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
2.00
4.00
6.00
penetration (mm)
10blows
30blows
65blows
CBR


72Hrs
Stress (N/mm2)
1.00
0.80
0.60
0.40
0.20
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Penetration (mm)
10blows
30blows
65blows
CBR



$$\text{CBR corresponding 2.54 mm penetration} = \frac{\text{Test stress}}{\text{standard stress}} * 100$$

$$= \frac{0.22}{6.89} * 100$$

$$= 3.16\%$$

$$\text{CBR corresponding 5.08 mm penetration} = \frac{\text{Test stress}}{\text{standard stress}} * 100$$

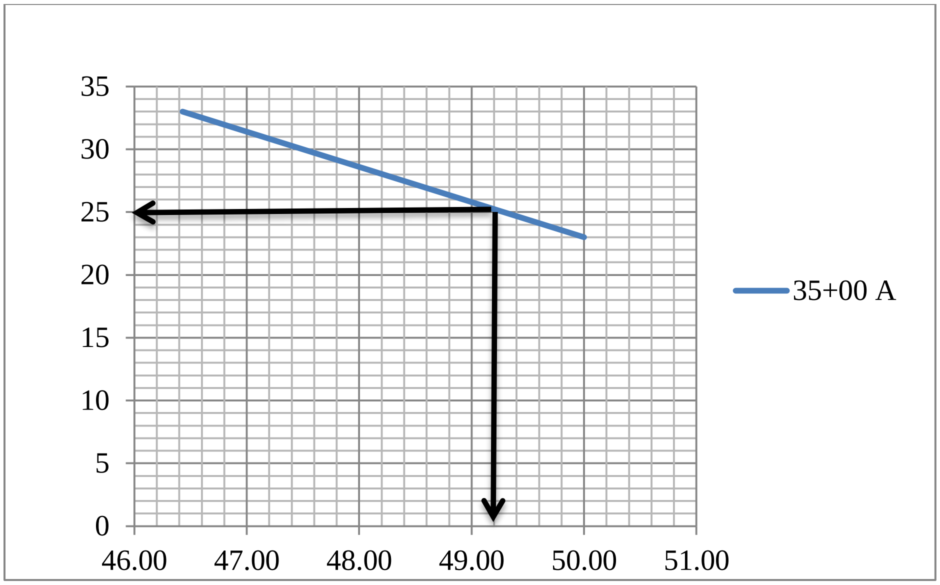
$$= \frac{0.40}{10.34} * 100$$

$$= 3.82\%$$

A.2: For type 2 (station 35+000) soil.

A.2.1: Liquid limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	D23	NC42	46
Mass of can	19	17	18
No of blows	33	27	23
Mass of can and wet soil	60	43	51
Mass of can and dry soil	47	34.5	40
Water mass (%)	46.43	48.57	50.00



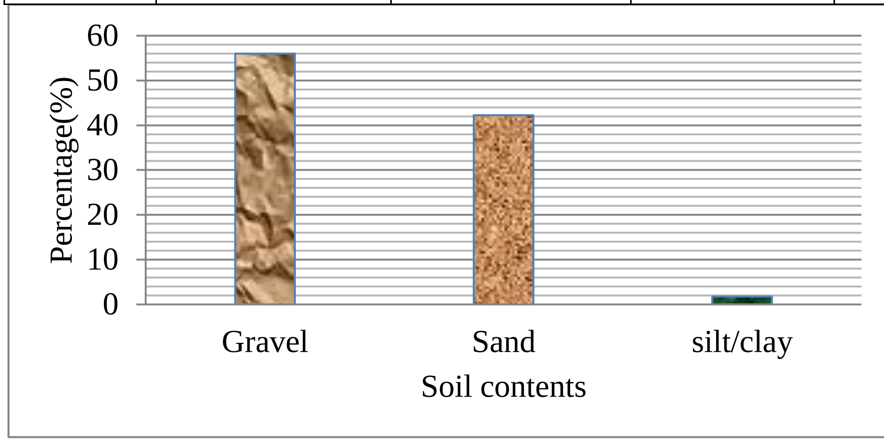
A.2.2: Plastic limits

Trial No	Trial-1	Trial-2	Trial-3
Name of can	A1	A2	FF
Mass of can	5	6	6
Mass of can and wet soil	19	20	22
Mass of can and dry soil	16	17	18
Water content (%)	27.27	27.27	33.33
Plastic Limits (%)	29.29		

A.2.3: Particle size distribution.

IS. Seive No.(mm)	Weight retained in gm.	Percentage Weight retained	Cumulative Retained	Percentage weight passing
19	210	10.5	10.5	89.5
14	200	10	20.5	79.5
9.5	230	11.5	32	68
4.75	480	24	56	44
2	460	23	79	21
0.85	215	10.75	89.75	10.25
0.425	132	6.6	96.35	3.65
0.25	20	1	97.35	2.65
0.15	10	0.5	97.85	2.15
0.075	8	0.4	98.25	1.75

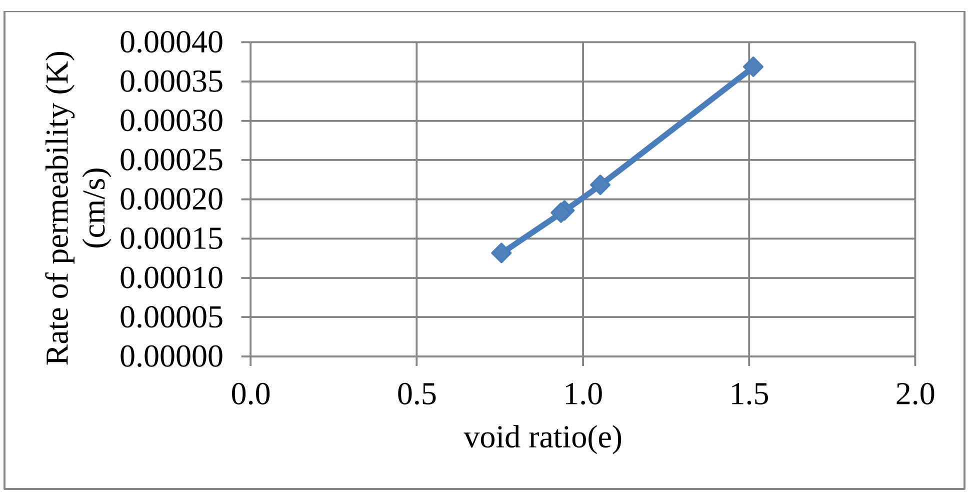
0	35	1.75	100	0
Total Mass	2000	100		



A.2.4: Specific gravity

Sample Name	O	N	SMALL
Mass of Pycnometer, Mp	162	162	152
Mass of Pycnometer + Soil, Mps	264	207	227
Mass of Pycnometer + Soil + Water, Mpws	724	687.5	697
Mass of Pycnometer + Water, Mpw @ Ti	659	659	649
The water temprature, Ti	23	23	23
Temperature of contents of Pycnometer When Mpws was taken, Tx	26	26	25.5
Mass of Dry Soil, Ms	102	45	75
Density of water at Ti, ρW @ Ti	0.99757	0.99757	0.99757
Density of water at Tx ρW @ Tx	0.99681	0.99681	0.99694
$Mpw(at\ Tx) = \left[\frac{\rho w@Tx}{\rho w@Ti}\right][Mpw@Ti - Mp] + Mp$	658.6214	658.6214	648.6861
Conversion factor, K	0.9986	0.9986	0.9987
Specific Gravity, @ 20oc Gs = $k\left[\frac{Ms}{Ms+Mpw(at\ Tx)-Mpws}\right]$	2.78	2.79	2.81
AVERAGE		2.79	

A.2.5: Rate of permeability at different voids of soil sample.



A.2.6: CBR test result at different time of soaking.

A.2.6.1: Un soaked condition (0Hr)

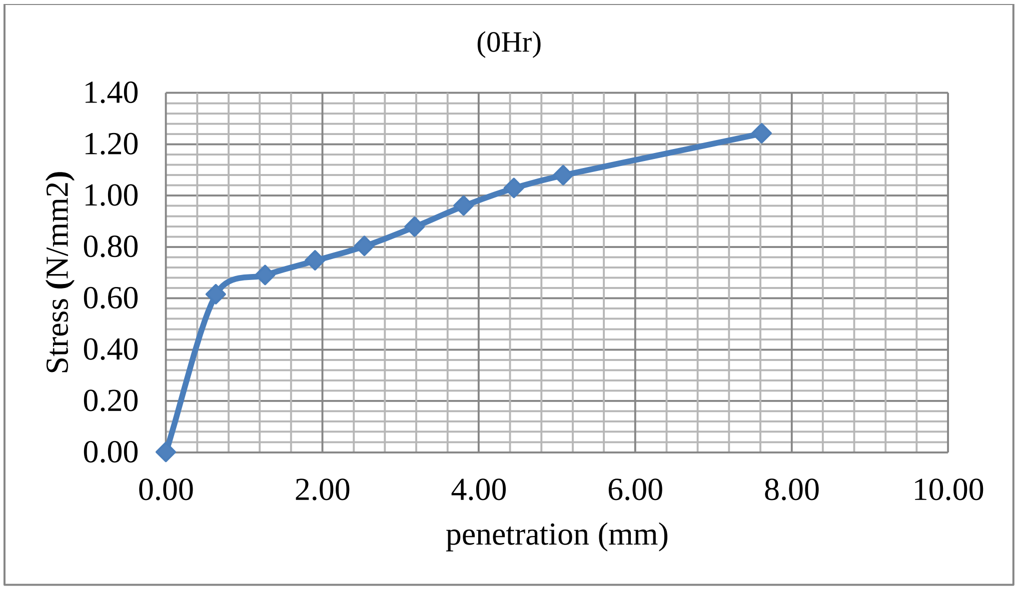
Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0	0.00	0.00	
0.025	0.64	98	1.19	0.61	
0.050	1.27	110	1.34	0.69	
0.075	1.91	119	1.44	0.75	
0.100	2.54	128	1.55	0.80	11.65
0.125	3.18	140	1.70	0.88	
0.150	3.81	153	1.86	0.96	
0.175	4.45	164	1.99	1.03	
0.200	5.08	172	2.09	1.08	10.43
0.300	7.62	198	2.40	1.24	
0.400	10.16				
0.500	12.70				

A.2.6.2: Soaked condition (96Hrs)

Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0	0.00	0.00	
0.025	0.64	9	0.11	0.06	
0.050	1.27	17	0.21	0.11	
0.075	1.91	23	0.28	0.14	
0.100	2.54	29	0.35	0.18	2.64
0.125	3.18	37	0.45	0.23	
0.150	3.81	46	0.56	0.29	
0.175	4.45	53	0.64	0.33	
0.200	5.08	56	0.68	0.35	3.40

0.300	7.62	76	0.92	0.48	
0.400	10.16				
0.500	12.70				

A.2.6.3: Stress versus Penetration graph and Analysis of CBR at 2.54mm and 5.08mm penetration



CBR corresponding 2.54 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

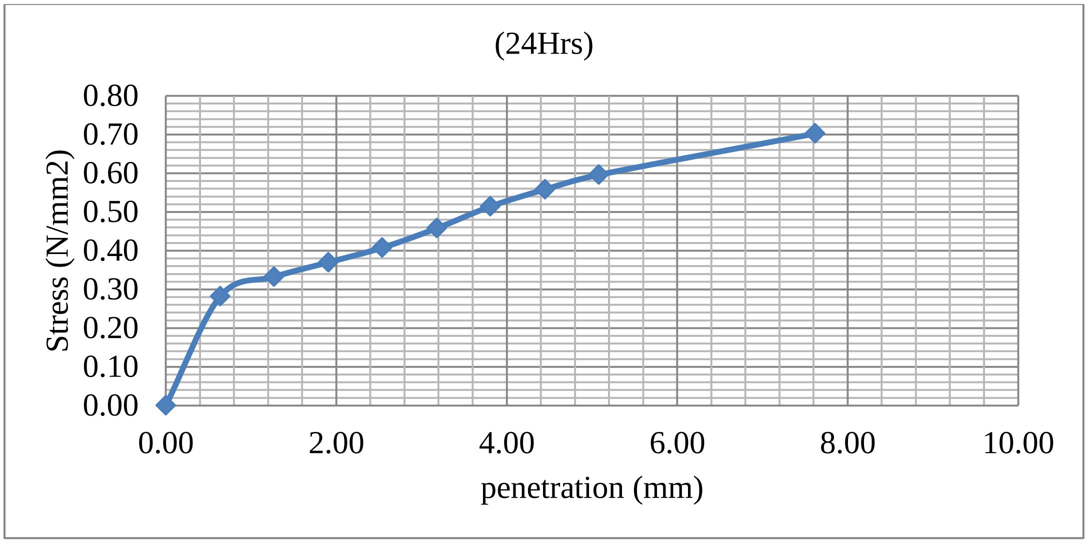
$= 0.80/6.89 * 100$

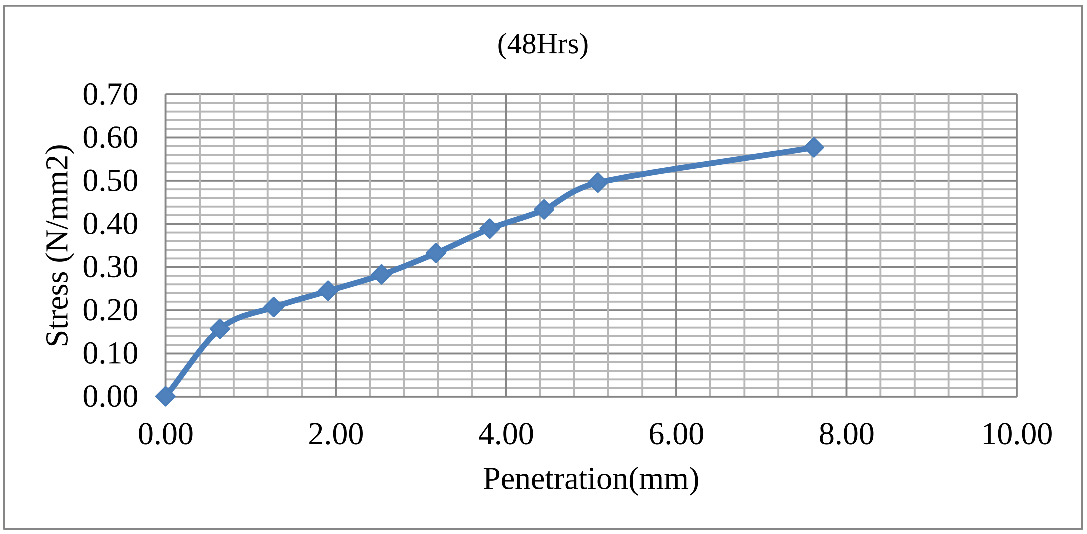
$= 11.65\%$

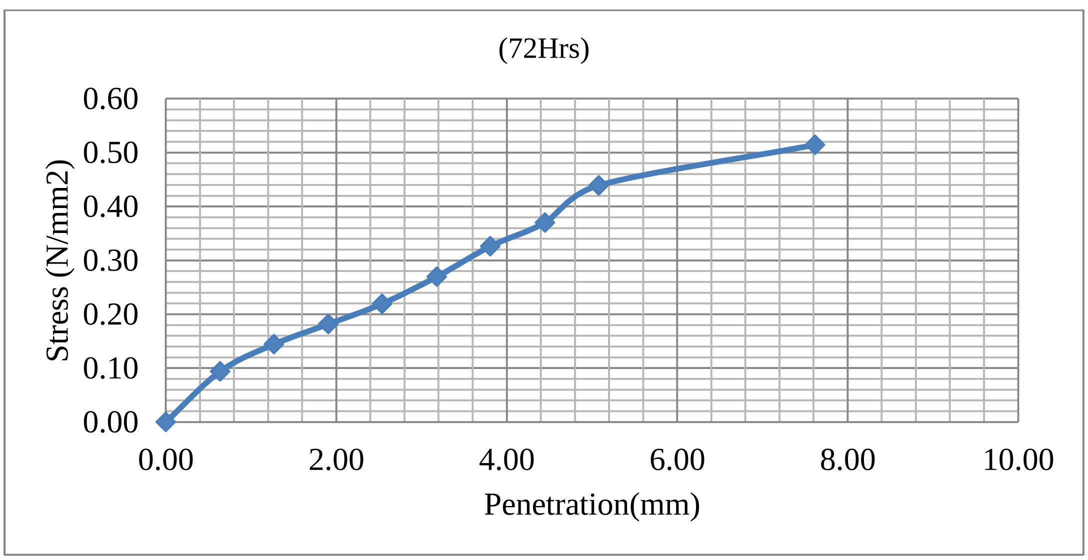
CBR corresponding 5.08 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

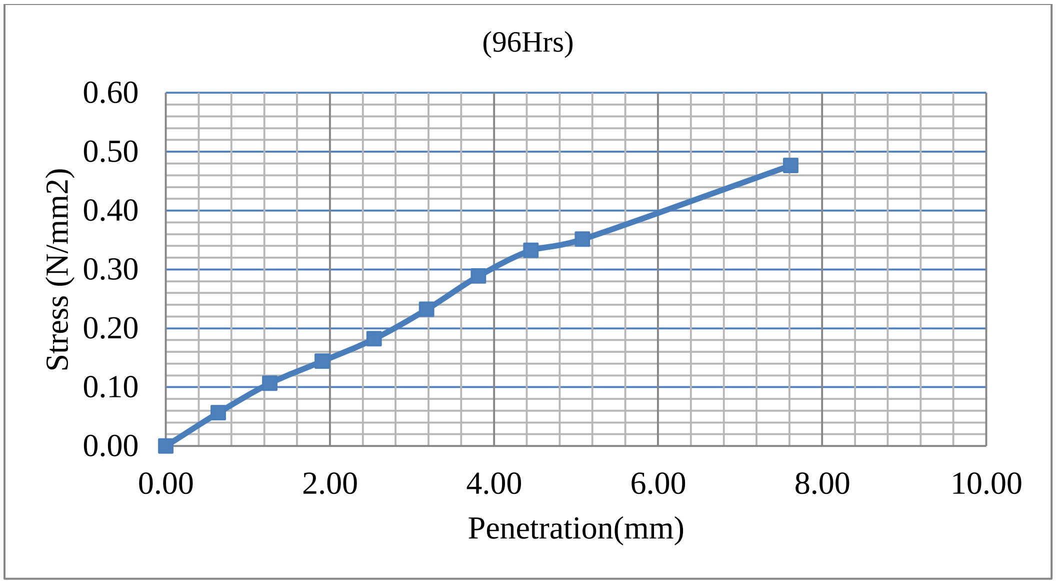
$= 1.08/10.34 * 100$

$= 10.43\%$


(24Hrs)
Stress (N/mm2)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
8.00
10.00
penetration (mm)


(48Hrs)
Stress (N/mm2)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
8.00
10.00
Penetration(mm)


(72Hrs)
Stress (N/mm2)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
8.00
10.00
Penetration(mm)



CBR corresponding 2.54 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

$= 0.18/6.89 * 100$

$= 2.64\%$

CBR corresponding 5.08 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

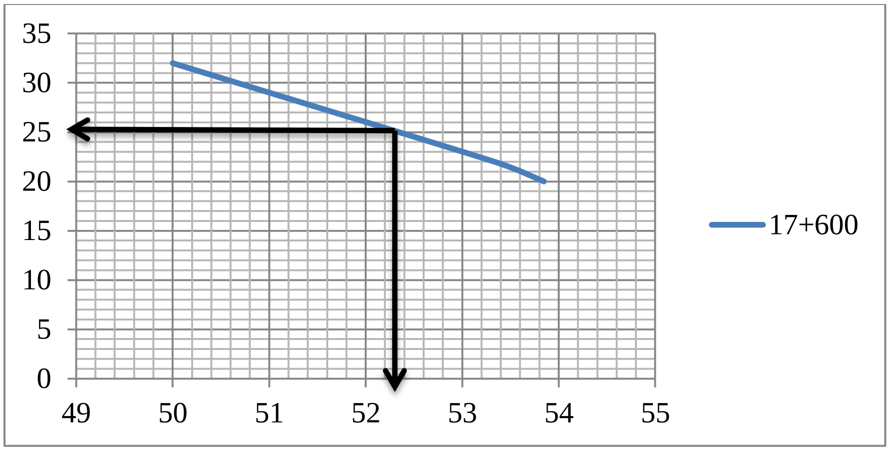
$= 0.35/10.34 * 100$

$= 3.40\%$

A.3: For type 3(station 17+600) soils.

A.3.1: Liquid limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	LC32	K3	46
Mass of can	18	18	18
No of blows	32	22	16
Mass of can and wet soil	42	41	38
Mass of can and dry soil	34	33	31
Water mass (%)	50	53.33	53.85

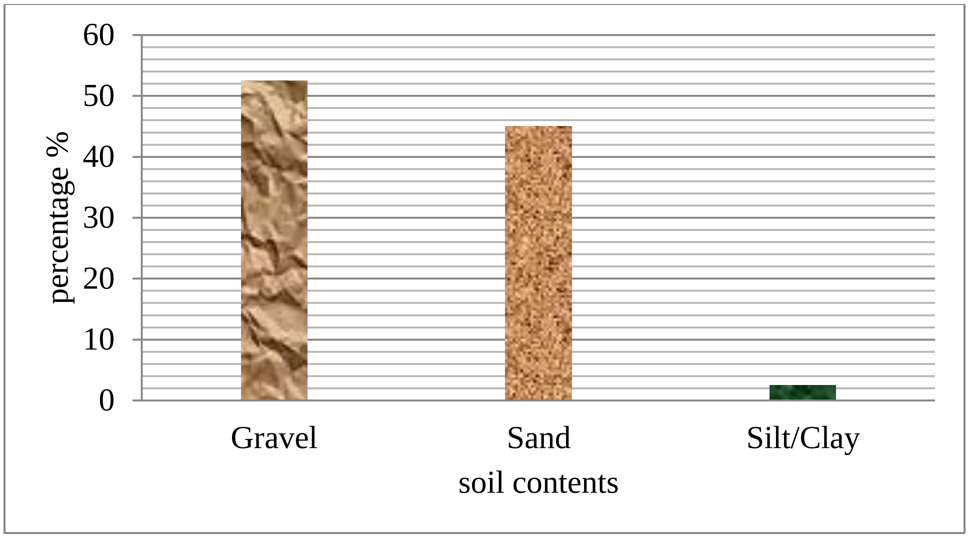


A.3.2: Plastic limits

Trial No	Trial-1	Trial-2	Trial-3
Name of can	A2	59	PL3
Mass of can	7	6	6
Mass of can and wet soil	18	17	16
Mass of can and dry soil	15	15	13
Water content (%)	37.50	22.22	42.86
Plastic Limits (%)	34.19		

A.3.3: Particle size distribution

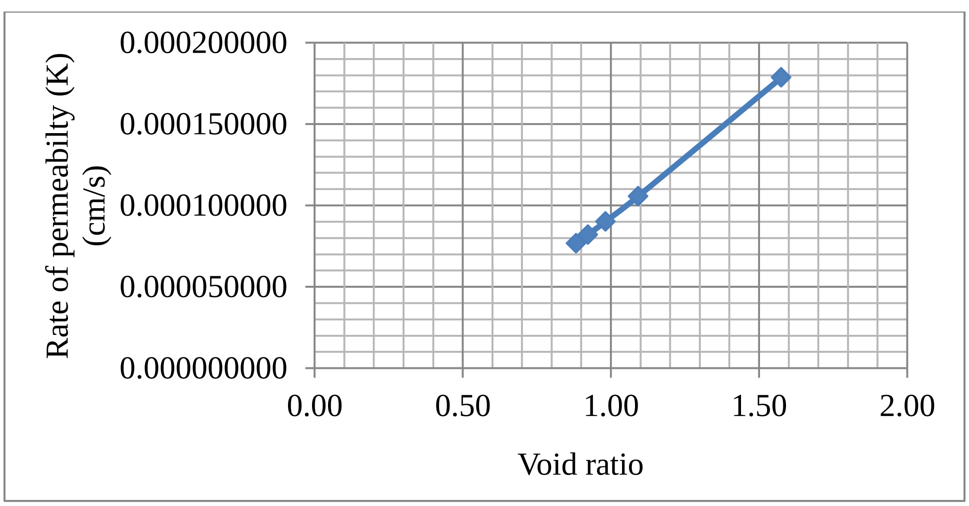
IS. Seive No.(mm)	Weight retained in gm.	Percentage Weight retained	Cumulative Retained	Percentage weight passing
19	265	13.25	13.25	86.75
9.5	395	19.75	33	67
4.75	390	19.5	52.5	47.5
2	415	20.75	73.25	26.75
0.85	210	10.5	83.75	16.25
0.425	160	8	91.75	8.25
0.25	55	2.75	94.5	5.5
0.2	15	0.75	95.25	4.75
0.15	20	1	96.25	3.75
0.075	25	1.25	97.5	2.5
0	50	2.5	100	0
Total Mass	2000	100		



A.3.4: Specific gravity

Sample Name	SMALL	N	O
Mass of Pycnometer, Mp	152	162	162
Mass of Pycnometer + Soil, Mps	174	210	225
Mass of Pycnometer + Soil + Water, Mpws	662	689	699
Mass of Pycnometer + Water, Mpw @ Ti	649	659	659
The water temprature, Ti	23	23	23
Temperature of contents of Pycnometer When Mpws was taken, Tx	26.5	27	27
Mass of Dry Soil, Ms	22	48	63
Density of water at Ti, ρW @ Ti	0.99757	0.99757	0.99757
Density of water at Tx ρW @ Tx	0.99668	0.99654	0.99654
$Mpw(at\ Tx) = \left[\frac{\rho w@Tx}{\rho w@Ti}\right][Mpw@Ti - Mp] + Mp$	648.5566	658.4868	658.4868
Conversion factor, K	0.9984	0.9983	0.9983
Specific Gravity, @ 20oc Gs = $k\left[\frac{Ms}{Ms+Mpw(at\ Tx)-Mpws}\right]$	2.57	2.74	2.80
AVERAGE	2.70		

A.3.5: Rate of permeability at different voids of soil sample.



A.3.6: CBR test result at different time of soaking.

A.3.6.1: Un soaked condition (0Hr)

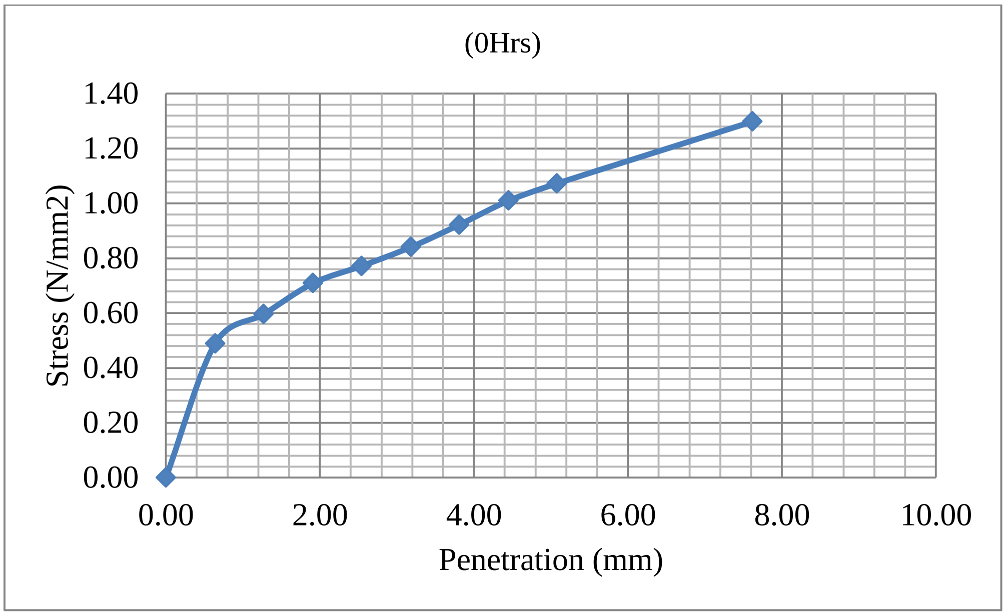
Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0	0.00	0.00	
0.025	0.64	78	0.95	0.49	
0.050	1.27	95	1.15	0.60	
0.075	1.91	113	1.37	0.71	
0.100	2.54	123	1.49	0.77	11.20
0.125	3.18	134	1.63	0.84	
0.150	3.81	147	1.78	0.92	
0.175	4.45	161	1.95	1.01	
0.200	5.08	171	2.08	1.07	10.37
0.300	7.62	207	2.51	1.30	
0.400	10.16				
0.500	12.70				

A.3.6.2: Soaked condition (96Hrs)

Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0	0.00	0.00	
0.025	0.64	8	0.10	0.05	
0.050	1.27	19	0.23	0.12	
0.075	1.91	31	0.38	0.19	
0.100	2.54	38	0.46	0.24	3.46
0.125	3.18	45	0.55	0.28	
0.150	3.81	54	0.66	0.34	
0.175	4.45	63	0.76	0.40	

0.200	5.08	70	0.85	0.44	4.25
0.300	7.62	94	1.14	0.59	
0.400	10.16				
0.500	12.70				

A.3.6.3: Stress versus Penetration graph and Analysis of CBR at 2.54mm and 5.08mm penetration



CBR corresponding 2.54 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

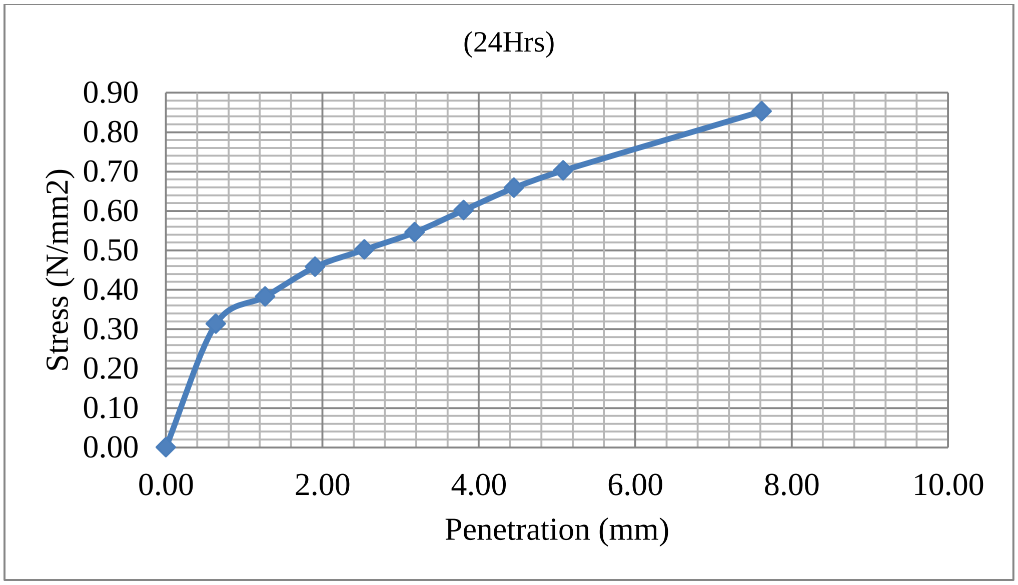
$= 0.77/6.89 * 100$

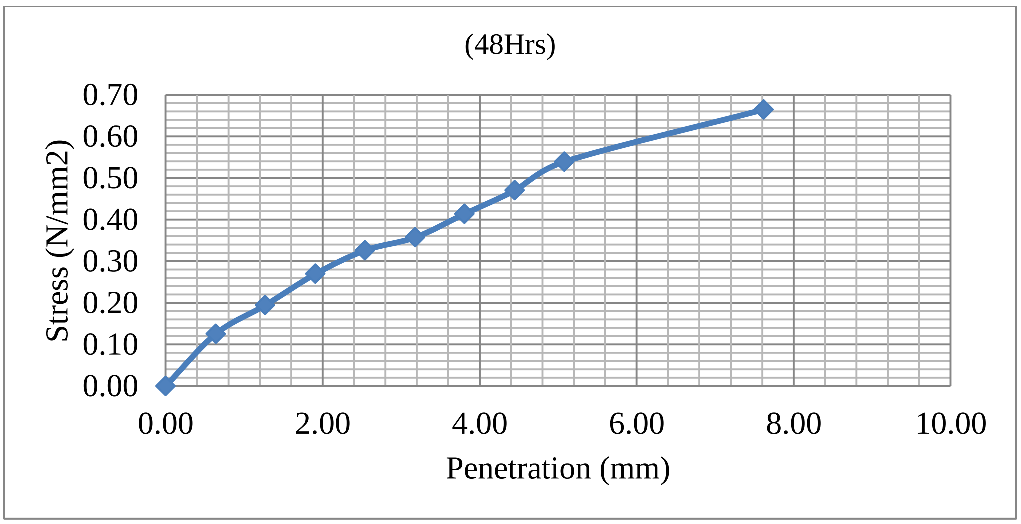
$= 11.20\%$

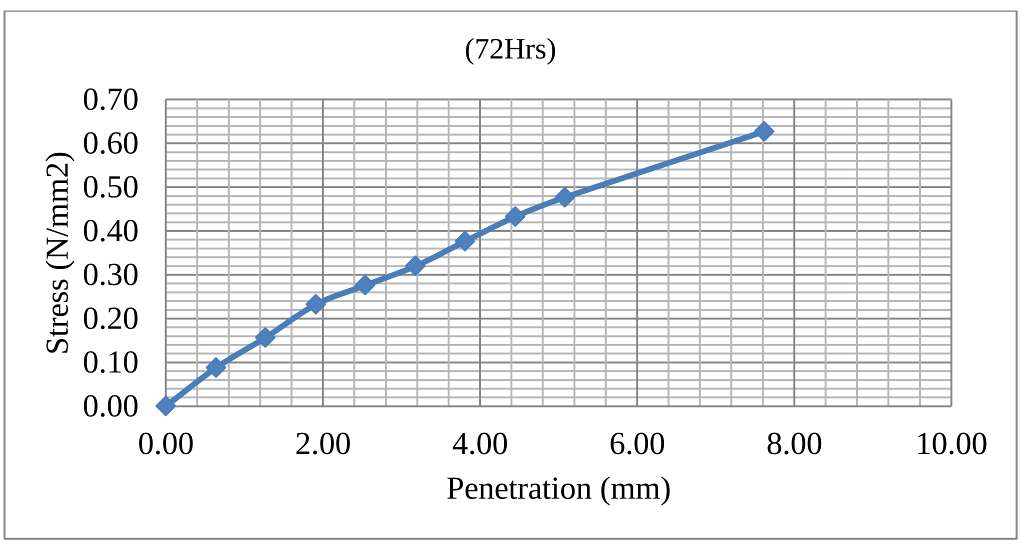
CBR corresponding 5.08 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

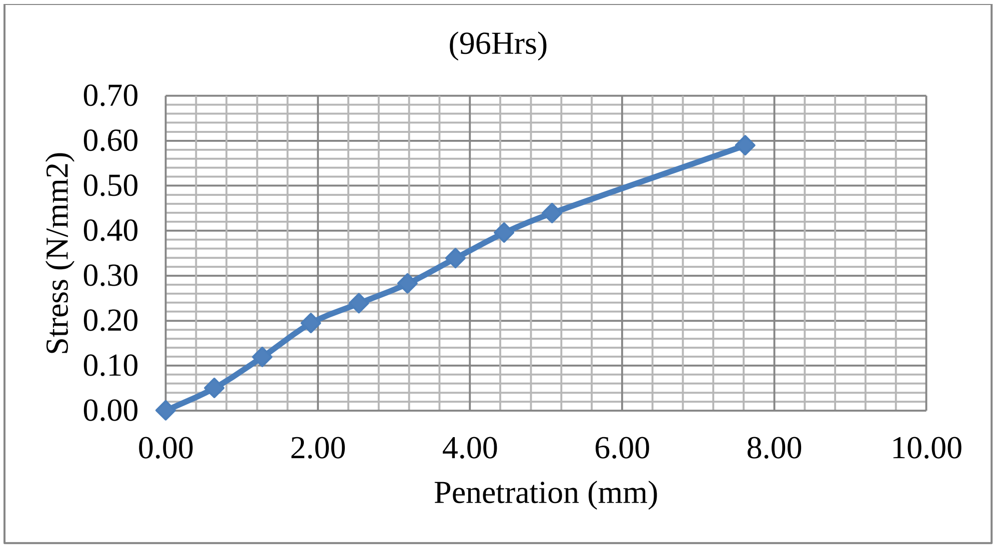
$= 1.07/10.34 * 100$

$= 10.37\%$


(24Hrs)
Stress (N/mm2)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
0.00
2.00
4.00
6.00
8.00
10.00
Penetration (mm)


(48Hrs)
Stress (N/mm2)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.00
2.00
4.00
6.00
8.00
10.00
Penetration (mm)


(72Hrs)
Stress (N/mm2)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.00
2.00
4.00
6.00
8.00
10.00
Penetration (mm)



CBR corresponding 2.54 mm penetration $= \text{Test stress} / \text{standard stress} * 100$

$= 0.24 / 6.89 * 100$

$= 3.46\%$

CBR corresponding 5.08 mm penetration $= \text{Test stress} / \text{standard stress} * 100$

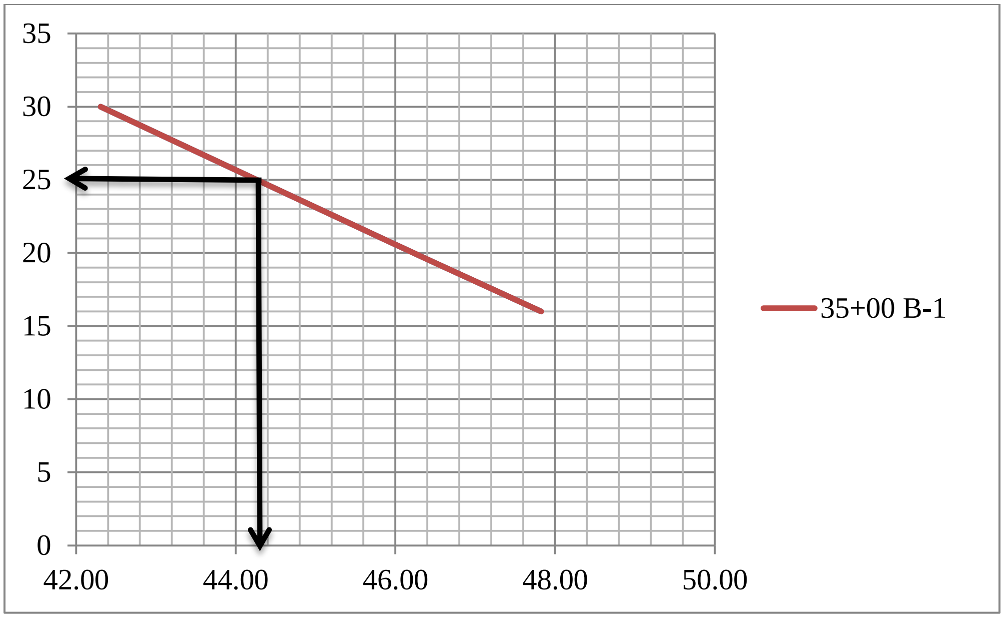
$= 0.44 / 10.34 * 100$

$= 4.25\%$

A.4: For type 4(station 30+500) soil.

A.4.1: Liquid limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	S	A3	3
Mass of can	6	6	6
No of blows	30	21	15
Mass of can and wet soil	43	41	40
Mass of can and dry soil	32	30	29
Water mass (%)	42.31	45.83	47.83



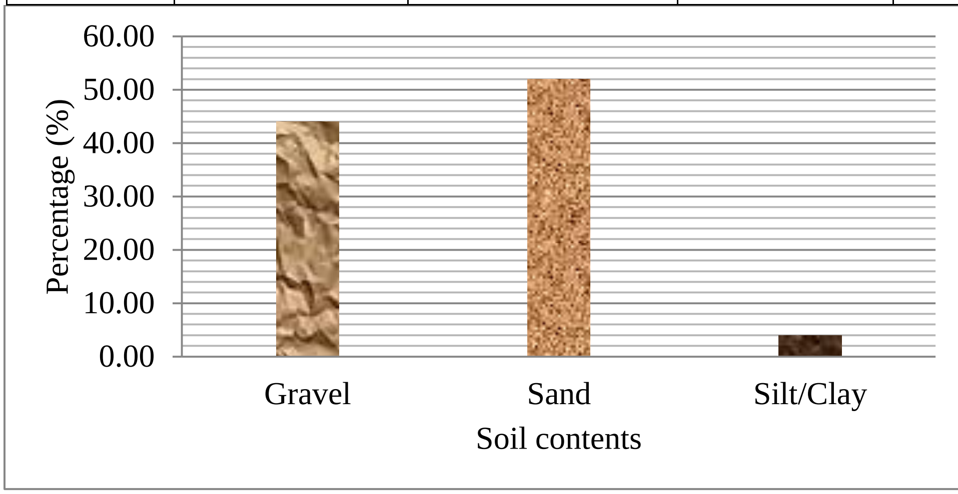
A.4.2: Plastic limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	11	EE	1
Mass of can	6	6	6
Mass of can and wet soil	18	21	16
Mass of can and dry soil	15	17	14
Water content (%)	33.33	36.36	25.00
Plastic Limits (%)	31.57		

A.4.3: Particle size distribution

IS Seive No.(mm)	Weight retained in gm.	Percentage Weight retained	Cumulative Retained	Percentage weight passing
19	19	12.67	12.67	87.33
14	20	13.33	26.00	74.00
9.5	12	8.00	34.00	66.00
4.75	15	10.00	44.00	56.00
2	16	10.67	54.67	45.33
0.85	13	8.67	63.33	36.67
0.425	14	9.33	72.67	27.33
0.25	11	7.33	80.00	20.00
0.15	9	6.00	86.00	14.00
0.075	15	10.00	96.00	4.00
0	6	4.00	100.00	0.00

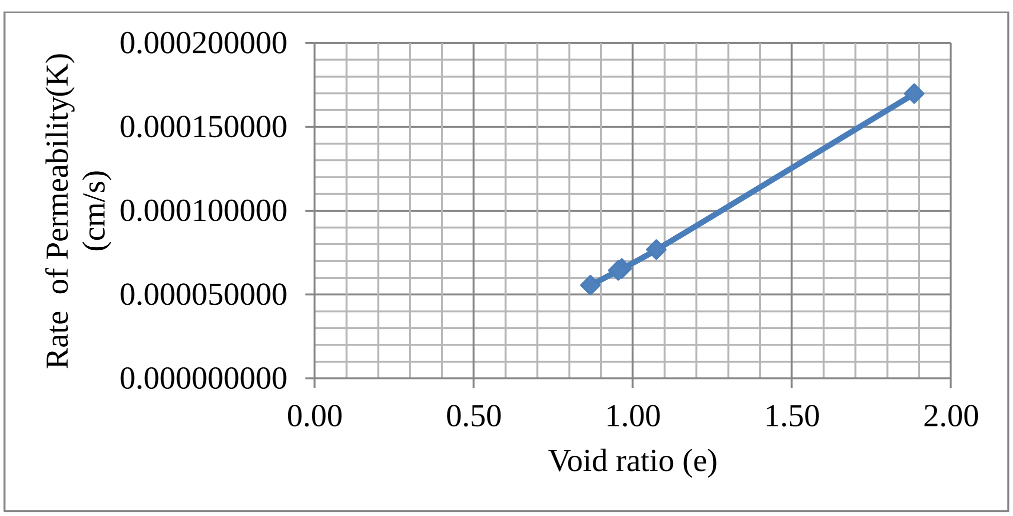
Total Mass	150			



A.4.4: Specific gravity

Sample Name	K	H	L
Mass of Pycnometer, Mp	117.814	92.992	94.499
Mass of Pycnometer + Soil, Mps	192	173	163
Mass of Pycnometer + Soil + Water, Mpws	414	393	386.5
Mass of Pycnometer + Water, Mpw @ Ti	367	342	343
The water temprature, Ti	23	23	23
Temperature of contents of Pycnometer When Mpws was taken, Tx	27.5	27.5	27.5
Mass of Dry Soil, Ms	74.186	80.008	68.501
Density of water at Ti, ρW @ Ti	0.99757	0.99757	0.99757
Density of water at Tx ρW @ Tx	0.9964	0.9964	0.9964
$Mpw(at\ Tx) = \left[\frac{\rho w@Tx}{\rho w@Ti}\right][Mpw@Ti - Mp] + Mp$	366.7077	341.708	342.7085
Conversion factor, K	0.9982	0.9982	0.9982
Specific Gravity, @ 20oc Gs = $k\left[\frac{Ms}{Ms+Mpw(at\ Tx)-Mpws}\right]$	2.75	2.78	2.77
AVERAGE		2.77	

A.4.5: Rate of permeability at different voids of soil sample.



A.4.6: CBR test result at different time of soaking.

A.4.6.1: Un soaked condition (0Hr)

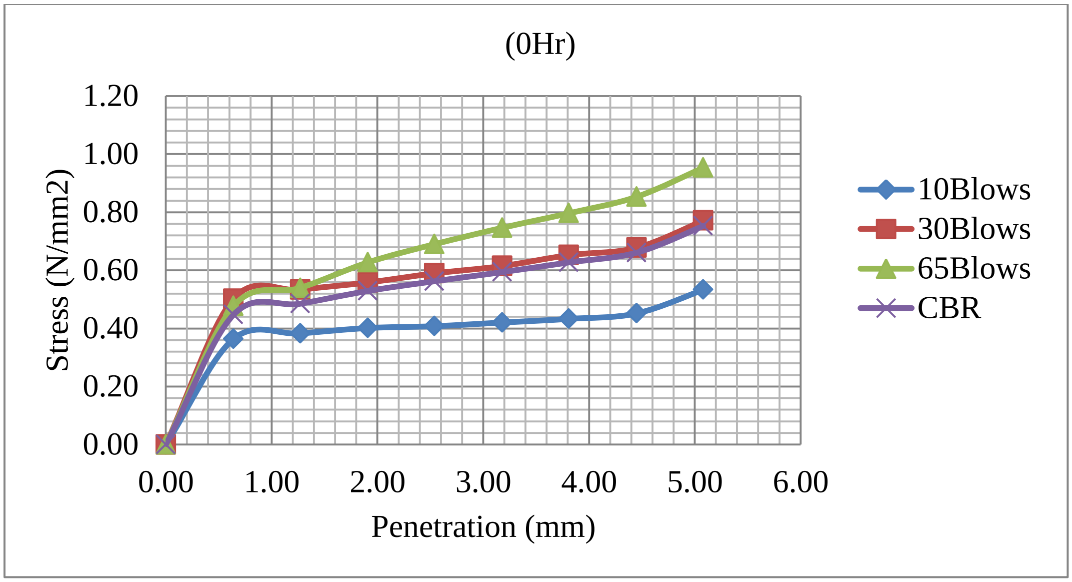
Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	71.33	0.87	0.45	
0.050	1.27	77.33	0.94	0.49	
0.075	1.91	84.33	1.02	0.53	
0.100	2.54	89.67	1.09	0.56	8.16
0.125	3.18	94.67	1.15	0.59	
0.150	3.81	100.00	1.21	0.63	
0.175	4.45	105.33	1.28	0.66	
0.200	5.08	120.00	1.46	0.75	7.28
0.300	7.62				
0.400	10.16				
0.500	12.70				

A.4.6.2: Soaked condition (96Hrs)

Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	6.67	0.08	0.04	
0.050	1.27	12.67	0.15	0.08	
0.075	1.91	19.67	0.24	0.12	
0.100	2.54	25.00	0.30	0.16	2.28
0.125	3.18	30.00	0.36	0.19	
0.150	3.81	35.33	0.43	0.22	
0.175	4.45	40.67	0.49	0.26	
0.200	5.08	47.00	0.57	0.29	2.85

0.300	7.62				
0.400	10.16				
0.500	12.70				

A.4.6.3: Stress versus Penetration graph and Analysis of CBR at 2.54mm and 5.08mm penetration



CBR corresponding 2.54 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

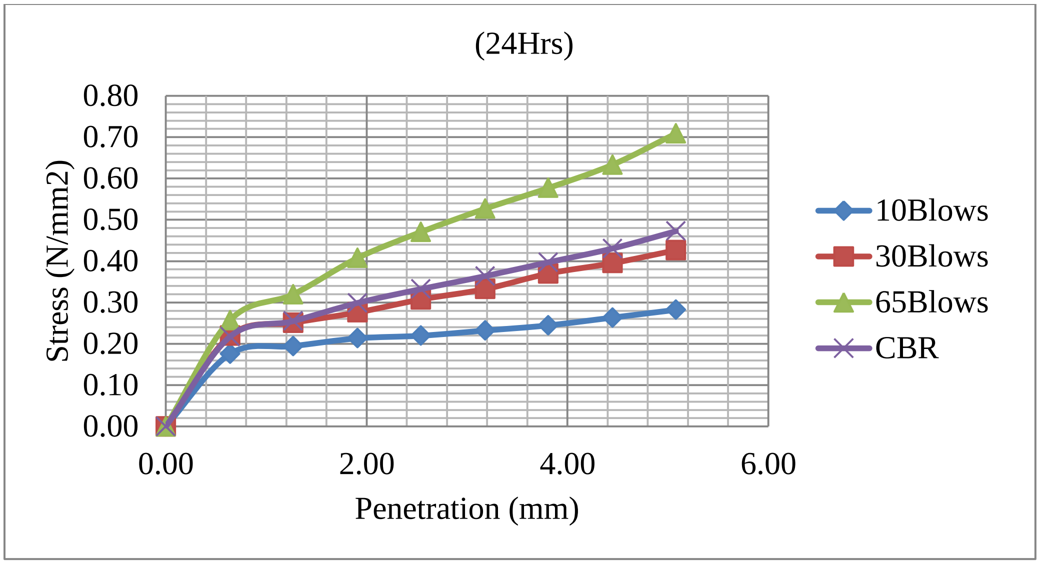
$= 0.56/6.89 * 100$

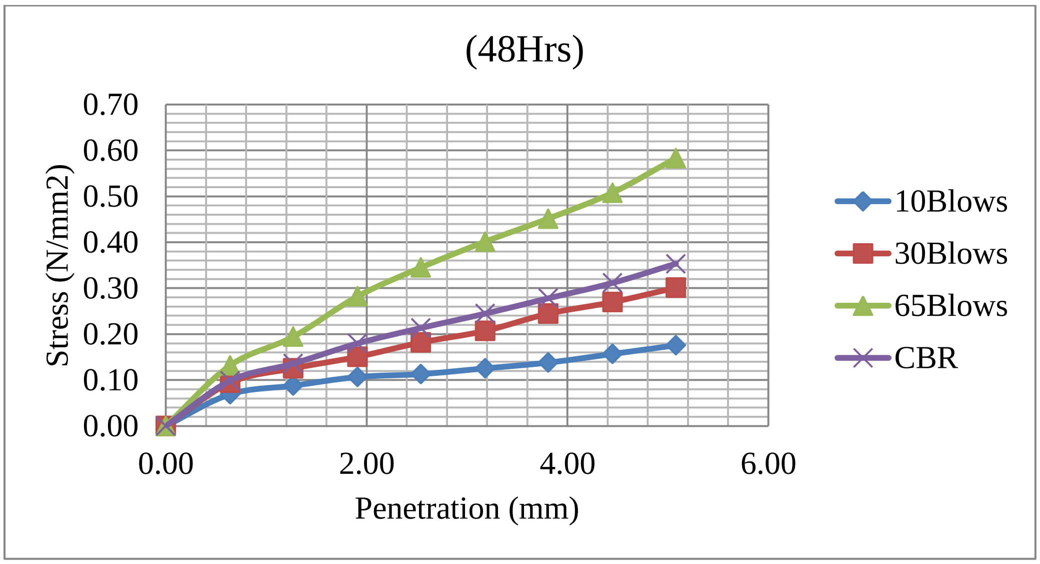
$= 8.16\%$

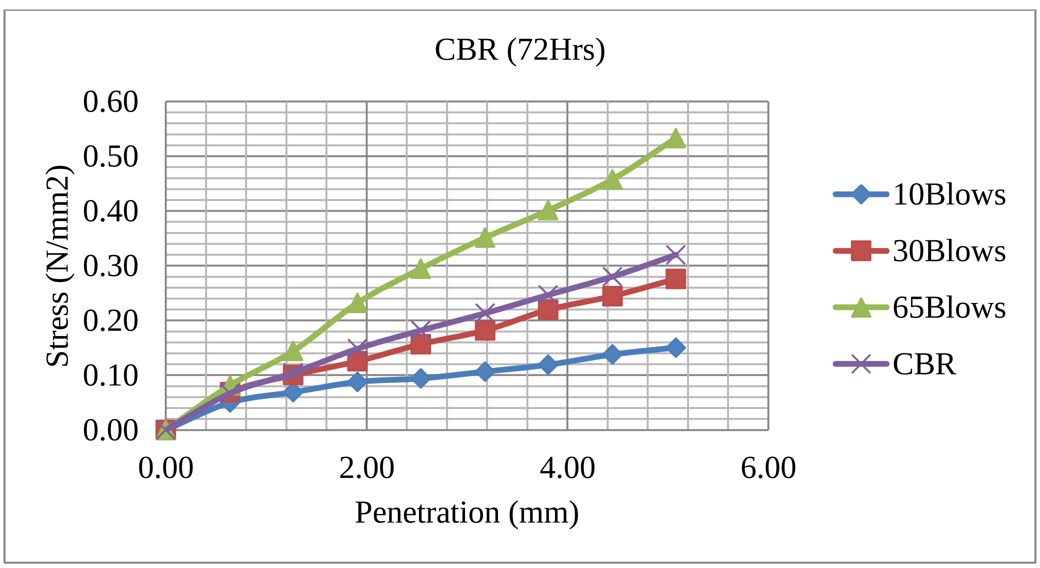
CBR corresponding 5.08 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

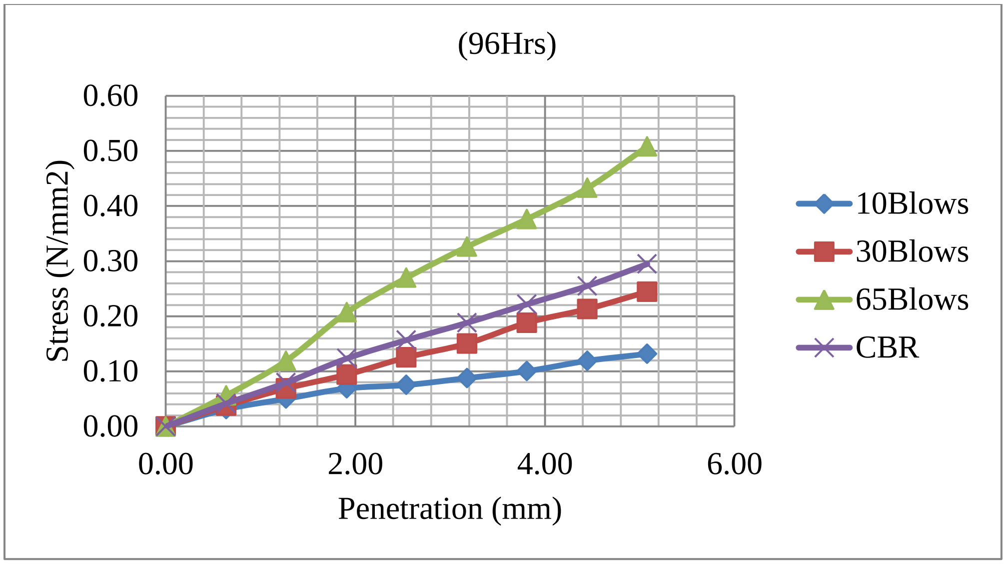
$= 0.75/10.34 * 100$

$= 7.28\%$


(24Hrs)
Stress (N/mm2)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
Penetration (mm)
10Blows
30Blows
65Blows
CBR


(48Hrs)
Stress (N/mm2)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
Penetration (mm)
10Blows
30Blows
65Blows
CBR


CBR (72Hrs)
Stress (N/mm2)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
2.00
4.00
6.00
Penetration (mm)
10Blows
30Blows
65Blows
CBR



$$\text{CBR corresponding 2.54 mm penetration} = {}^{\text{Test stress}}\!/_{\text{standard stress}} *100$$

$$= {}^{0.16}\!/_{6.89} *100$$

$$= 2.28\%$$

$$\text{CBR corresponding 5.08 mm penetration} = {}^{\text{Test stress}}\!/_{\text{standard stress}} *100$$

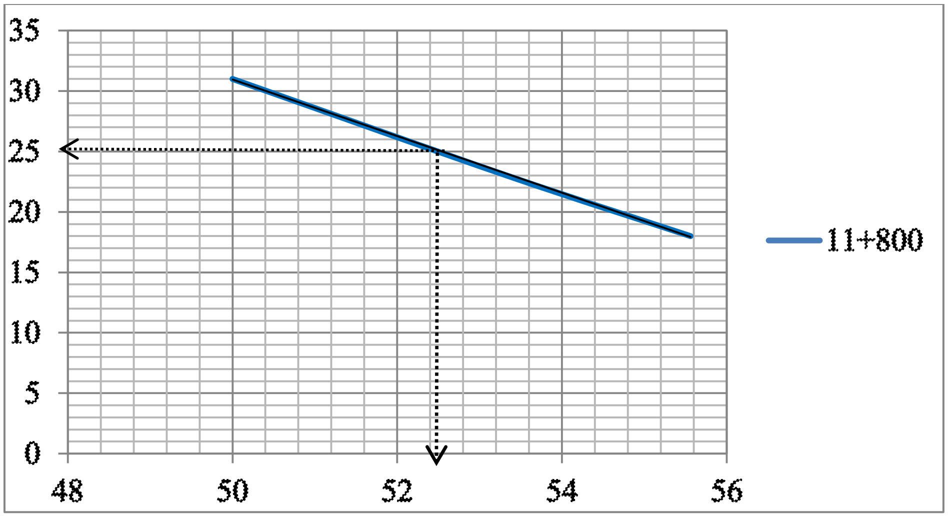
$$= {}^{0.29}\!/_{10.34} *100$$

$$= 2.85\%$$

A.5: For type 5 (station 11+800) soils.

A.5.1: Liquid limits.

Trial No	Trial-1	Trial-2	Trial-3
Name of can	Ls4	D53	D52
Mass of can	18	18	18
No of blows	31	23	18
Mass of can and wet soil	45	41	46
Mass of can and dry soil	36	33	36
WATER mass (%)	50	53.33	55.56



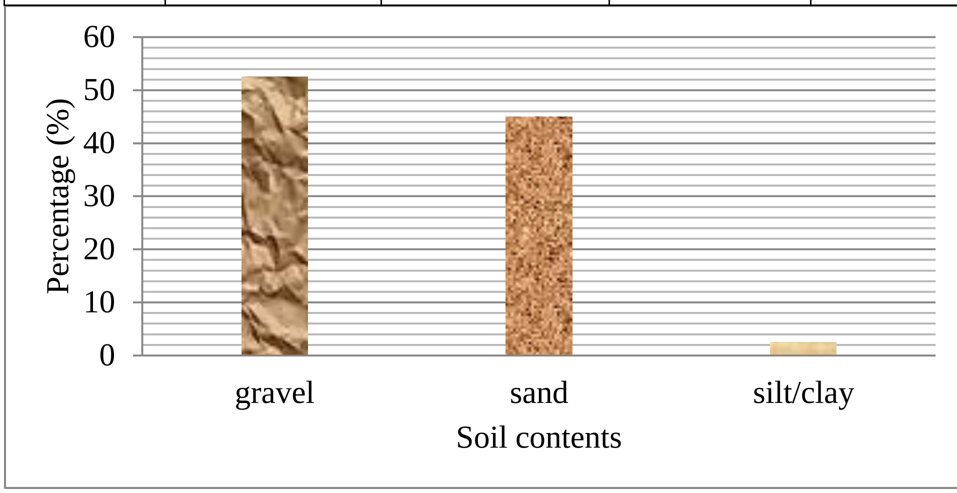
A.5.2: Plastic limits

Trial No	Trial-1	Trial-2	Trial-3
Name of can	K2	B2	F
Mass of can	8	6	6
Mass of can and wet soil	20	21	18
Mass of can and dry soil	17	17	15
Water content (%)	33.33	36.36	33.33
Plastic Limits (%)	34.34		

A.5.3: Particle size distribution

IS Seive No.(mm)	Weight retained in gm.	Percentage Weight retained	Cumulative Retained	Percentage weight passing
19	265	13.25	13.25	86.75
9.5	395	19.75	33	67
4.75	390	19.5	52.5	47.5
2	415	20.75	73.25	26.75
0.85	210	10.5	83.75	16.25
0.425	160	8	91.75	8.25
0.25	55	2.75	94.5	5.5
0.2	15	0.75	95.25	4.75
0.15	20	1	96.25	3.75
0.075	25	1.25	97.5	2.5
0	50	2.5	100	0

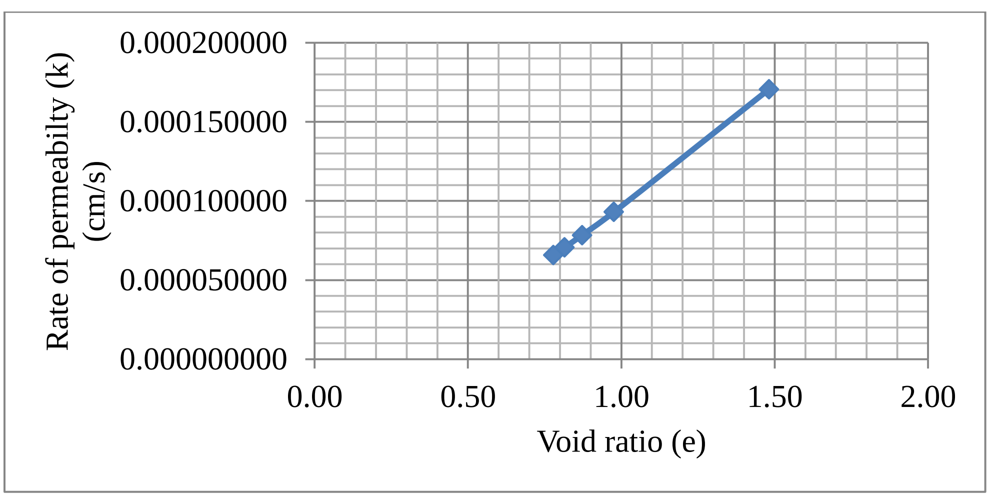
Total Mass	2000	100		



A.5.4: Specific gravity

Sample Name	N	O	SMALL
Mass of Pycnometer, Mp	162	162	152
Mass of Pycnometer + Soil, Mps	208	196	220
Mass of Pycnometer + Soil + Water, Mpws	688	680	692
Mass of Pycnometer + Water, Mpw @ Ti	659	659	649
The water temprature, Ti	23	23	23
Temperature of contents of Pycnometer When Mpws was taken, Tx	26	26	26
Mass of Dry Soil, Ms	46	34	68
Density of water at Ti, ρ_W **@ Ti**	0.99757	0.99757	0.99757
Density of water at Tx ρ_W **@ Tx**	0.99681	0.99681	0.99681
$Mpw(at\ Tx) = \left[\frac{\rho w@Tx}{\rho w@Ti}\right][Mpw@Ti - Mp] + Mp$	658.62	658.62	648.62
Conversion factor, K	0.99681	0.99681	0.99681
Specific Gravity, @ 20^{o}c Gs = $k\left[\frac{Ms}{Ms+Mpw(at\ Tx)-Mpws}\right]$	2.76	2.69	2.75
AVERAGE		2.73	

A.5.5: Rate of permeability at different voids of soil sample.



A.5.6: CBR test result at different time of soaking.

A.5.6.1: Un soaked condition (0Hr)

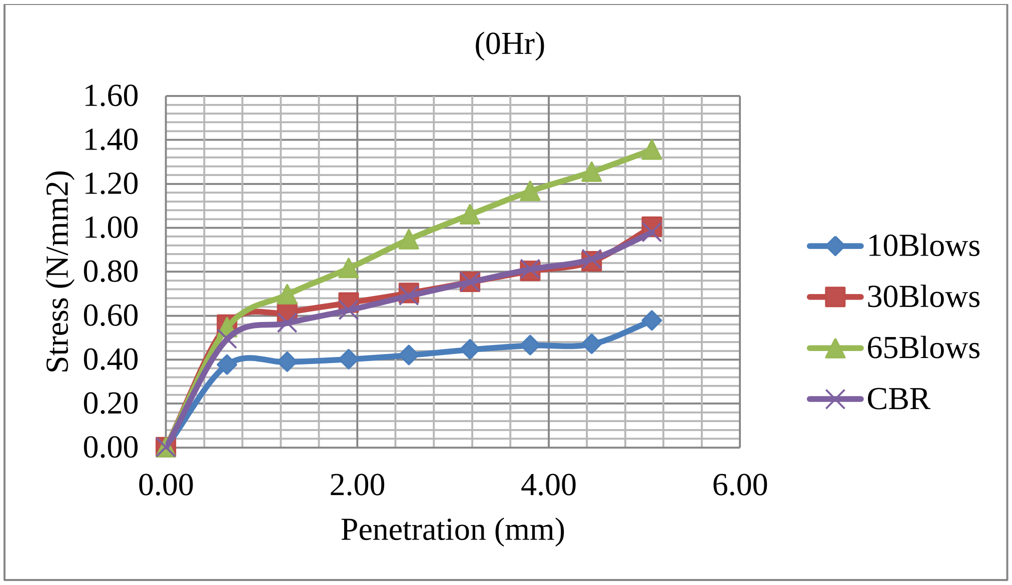
Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	78.67	0.96	0.49	
0.050	1.27	90.33	1.10	0.57	
0.075	1.91	99.67	1.21	0.63	
0.100	2.54	110.00	1.34	0.69	10.01
0.125	3.18	120.00	1.46	0.75	
0.150	3.81	129.33	1.57	0.81	
0.175	4.45	136.67	1.66	0.86	
0.200	5.08	156.00	1.89	0.98	9.46
0.300	7.62				
0.400	10.16				
0.500	12.70				

A.5.6.2: Soaked condition (96Hrs)

Ring Factor: N/Division		12.14			
Pen (in)	pen (mm)	Bottom		1935.5	
		Dial Reading	Load(KN)	Stress N/mm^2	CBR %
0	0.00	0.00	0.00	0.00	
0.025	0.64	13.67	0.17	0.09	
0.050	1.27	20.50	0.25	0.13	
0.075	1.91	24.00	0.29	0.15	
0.100	2.54	33.70	0.41	0.21	3.07
0.125	3.18	40.40	0.49	0.25	
0.150	3.81	45.67	0.55	0.29	
0.175	4.45	52.00	0.63	0.33	
0.200	5.08	59.50	0.72	0.37	3.61

0.300	7.62				
0.400	10.16				
0.500	12.70				

A.5.6.3: Stress versus Penetration graph and Analysis of CBR at 2.54mm and 5.08mm penetration



CBR corresponding 2.54 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

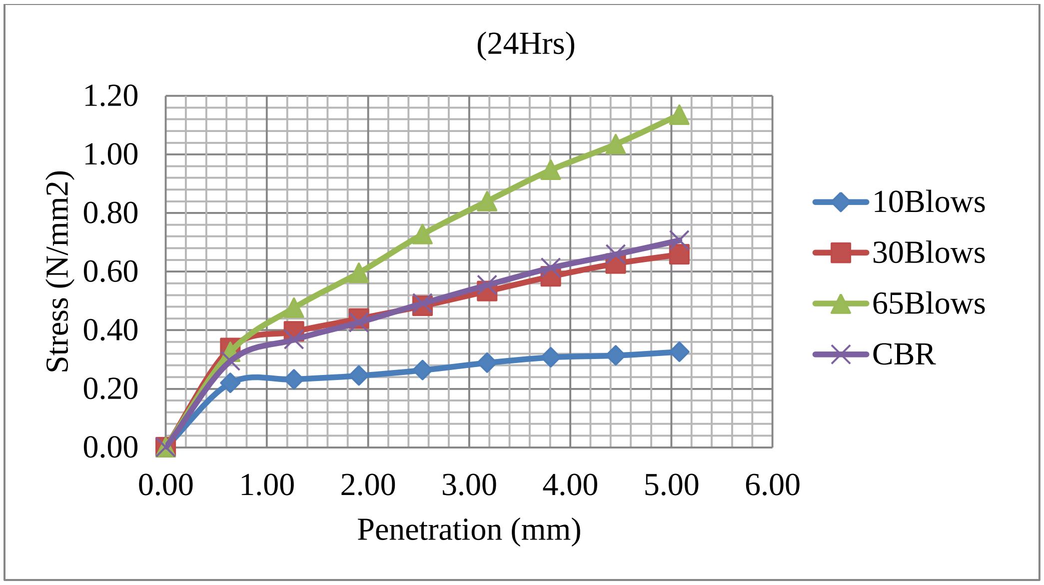
$= 0.69/6.89 * 100$

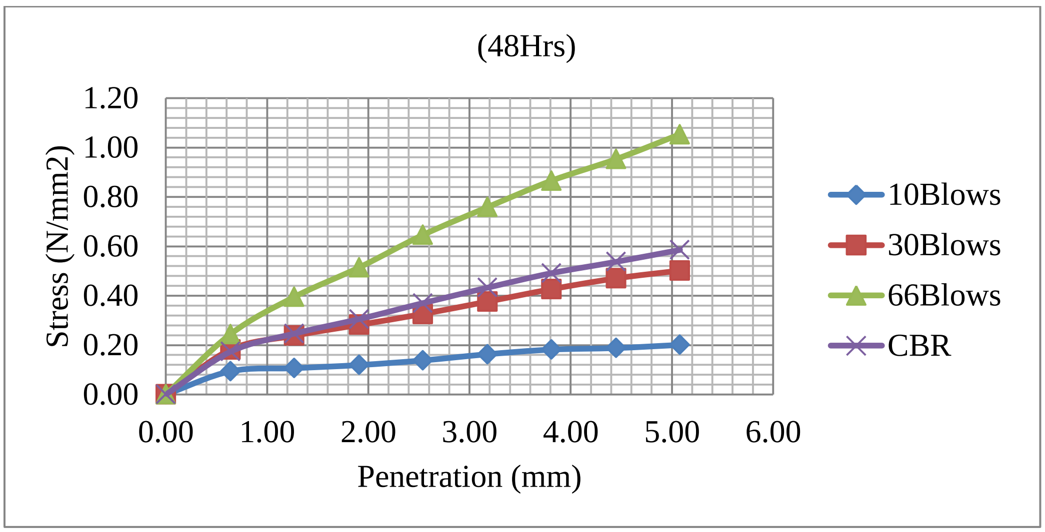
$= 10.01\%$

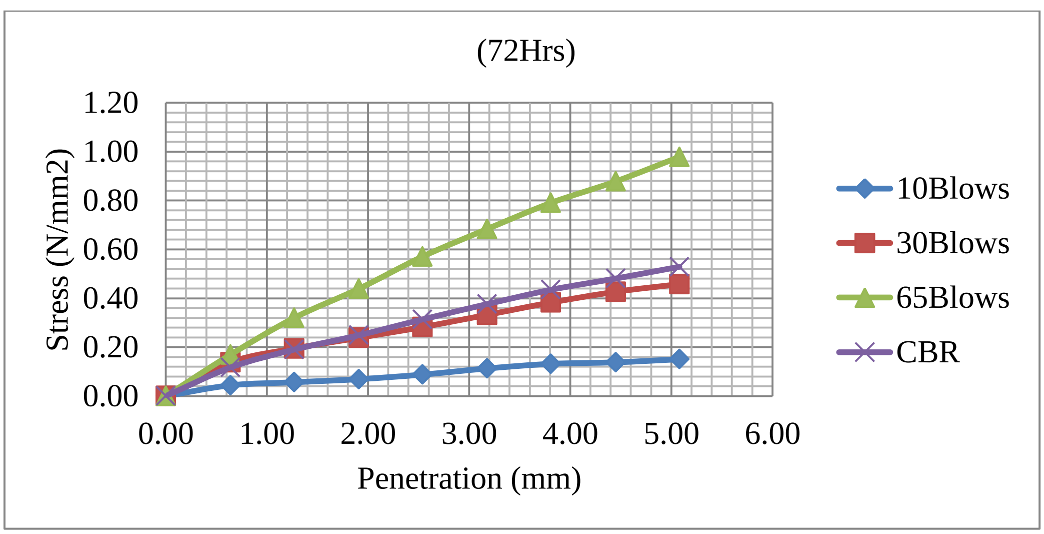
CBR corresponding 5.08 mm penetration = $\text{Test stress}/\text{standard stress} * 100$

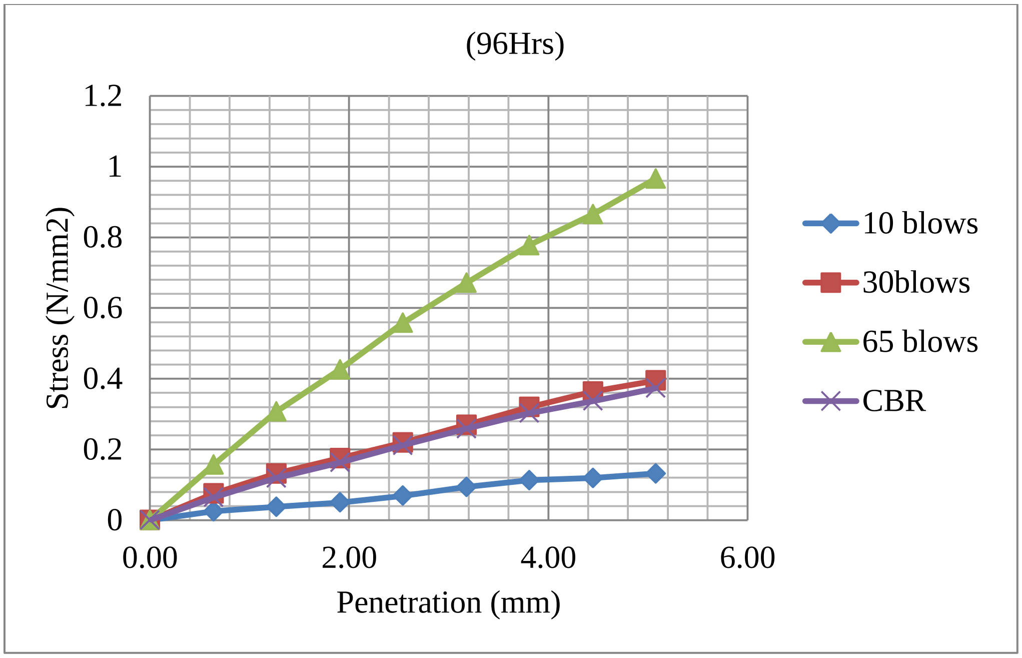
$= 0.98/10.34 * 100$

$= 9.46\%$


(24Hrs)
Stress (N/mm2)
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Penetration (mm)
10Blows
30Blows
65Blows
CBR


(48Hrs)
Stress (N/mm2)
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Penetration (mm)
10Blows
30Blows
66Blows
CBR


(72Hrs)
Stress (N/mm2)
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
Penetration (mm)
10Blows
30Blows
65Blows
CBR



CBR corresponding 2.54 mm penetration = $\text{Test stress} / \text{standard stress} * 100$

$= 0.21 / 6.89 * 100$

$= 3.07\%$

CBR corresponding 5.08 mm penetration = $\text{Test stress} / \text{standard stress} * 100$

$= 0.37 / 10.34 * 100$

$= 3.61\%$