THE RELATIONSHIP BETWEEN CONSOLIDATION AND SWELLING CHARACTERISTICS OF EXPANSIVE SOILS (CASE OF MOJO TOWN, ETHIOPIA)

FIKADU ANBESSIE DADI

A Thesis Submitted to the Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

February, 2023

Adama, Ethiopia

THE RELATIONSHIP BETWEEN CONSOLIDATION AND SWELLING CHARACTERISTICS OF EXPANSIVE SOILS

(CASE OF MOJO TOWN, ETHIOPIA)

BY :FIKADU ANBESSIE DADI

Advisor: Argaw Asha (Ph.D.)

A Thesis Submitted to The Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

February, 2023

Adama, Ethiopia

Approval Page

I/we hereby certify that the recommendation and suggestion made by the board of examiners are appropriately incorporated into the final version of the thesis entitled “**The Relationship Between Consolidation And Swelling Characteristics Of Expansive Soils (Case Of Mojo Town)**” by Fikadu Anbessie.

Argaw Asha (Ph.D)	____________________	______________
Advisor	Signature	Date

We, the undersigned, members of the Board of Examiners of the thesis open defense by Fikadu Anbessie have read and evaluated the thesis entitled “**The Relationship Between Consolidation And Swelling Characteristics Of Expansive Soils (Case Of Mojo Town)**” and examined the candidate during open defense .This is, therefore, to certify that the thesis is accepted partial fulfillment of the requirement of the Degree of Master of Science in Civil Engineering.

____________________	____________________	______________
Chairperson	Signature	Date
____________________	____________________	______________
Internal Examiner	Signature	Date
Dr. Tezera Firew Azmatch	____________________	March 28, 2023
External Examiner	Signature	Date

Finally approval and acceptance of the thesis is contingent upon submission of its final copy to the Office of Postgraduate Studies (OPGS) through the Department Graduate Council (DGC) and School Graduate Committee (SGC).

____________________	____________________	______________
Department Head	Signature	Date
____________________	____________________	______________
School Dean	Signature	Date
____________________	____________________	______________
Office of Postgraduate Studies, Dean	Signature	Date

DECLARATION

I hereby declare that this Master thesis entitled "**The Relationship between Consolidation and Swelling Characteristics Of Expansive Soils (Case Of Mojo Town)**" is my original work That is, it has not been submitted for the award of academic degree, diploma or certificate in any other. All sources of materials that are used for this thesis have been duly acknowledged through citations.

Fikadu Anbessie	______________________	_______________
Name of student	Signature	Date

RECOMMENDATION

I the advisor of this thesis, hereby certify that I have read the revised version of the entitled "**The Relationship Between Consolidation And Swelling Characteristics Of Expansive Soils (Case Of Mojo Town)"**prepared under my guidance by Fikadu Anbessie_submitted in partial fulfillment of the requirements of the Degree of Master's of Science in Civil Engineering. Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

Argaw Asha(Ph.D)	______________________	_______________
Advisor	Signature	Date

ACKNOWLEDGEMENT

First and most of all, I would like to thank almighty God for blessing and being with me in every step I pass through, and the strength to finish this work. I wish to express my deepest gratitude to my advisor, Dr. Argaw Asha for encouraging me during the progress of this research and for his unlimited support through supervision. He has been devoting his precious time and providing all necessary relevant literatures and contents to carry out the research. I would also like to thank Mr. Shume Deme of Adama Science and Technology Geotechnical Lab Technician and Mr.Tamirat Dibaba of Oromia Laboratory Services Engineering Company for their support in the experimental work. Last but not least, I would like to express my sincere thanks to my families and friends for their support.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

TABLE PAGE

List of Acronyms and Abbreviations

A……………………….Activity

AASHTO…………… ...American Association of State Highway and Transportation Officials

ASTM…………………American Society for Testing and Material

Cc……………………...Compression index

C_V……………………...Coefficient of consolidation,

CH……………………..Inorganic Highly Plastic Clays

CL ………………..……Inorganic Clays of Low Plasticity

e_o ……………………....Initial void ratio

e ……………………….Void ratio

FS……………………....Free Swell

H…………………………...Height of soil strata

LL……………………....liquid limit

MDD…………………....Maximum Dry Density

MH……………………...Inorganic Silt of High Plasticity

ML……………………....Inorganic Silt of Low Plasticity

NMC…………………….Natural Moisture Content

OH……………………....Organic Silts and Organic Clays of High Plasticity

OMC…………………….Optimum Moisture Content

OL……………………....Organic Silts and Organic Clays of Low Plasticity

P………………......…….Pressure

PI……………………...…plasticity index

PL………………………..plastic limit

SPR………………………Swelling Pressure

SPO………………………Swelling Potential

t50 ………………………Time during which 50% of consolidation takes place

t90 …………………….…Time during which 90% of consolidation takes place

TP………………………...Test Pit

USCS……………………..Unified Soil Classification System

ABSTRACT

The swell characteristics of the soil are influenced by the variation in the values of initial moisture content, dry density, coefficient of consolidation and compression. The objective of this thesis is to identify the relationship between consolidation and swelling characteristics of expansive soils of mojo town. The one-dimensional consolidation test was conducted for disturbed and undisturbed soil sample with different initial dry density and moisture Content. The expansive soil samples were classified as A-7-5and A-7-6 as per classification AASHTO system and MH and CH according to USCS. The test results show that the initial dry density increases with increase in swelling pressure. The compression index C_C*, ranges from 0.397-0.416 & the coefficient of consolidation Cv ranges from0.12-0.57 m2/year, while initial dry density & initial moisture content range of 1.057-1.126 g/cm3 & 45.66-49.66% respectively. The swelling potential and swelling pressure obtained from the test result are found to be 4.65%, 14.56% and 100kPa, 420kPa respectively. It was observed from the test results, those factors that affect the swelling characteristics of expansive soils have also affected the consolidation characteristics of the expansive soil.*

Keywords:*-one-dimensional consolidation test, coefficient of consolidation, compression index.*

CHAPTER ONE

INTRODUCTION

1.1 Back ground

Expansive soils present significant geotechnical and structural engineering challenges the world over, with costs associated with expansive behavior estimated to run into several billion annually (Lee D Jones,2012).

Clayey soil, also known as problematic or expansive soil has peculiar cyclic swell-shrink behavior and for this reason construction on expansive soil always creates many problems for Civil engineers. When moisture content increases, the soil shows its swelling behavior, but when the moisture decreases it shows shrinkage behavior. During the volume change behavior expansive soils cause large uplift pressures and upheaval of structures built on them. Due to this movement, lightly loaded structures such as foundations, pavement, canal beds and linings, and residential structures established on them are severely damaged (Alemayehu T. & Mesfin L. 1999).

An expansive soil possesses their expansive characteristics mainly due to the presence of clay mineral which is montmorillonite. The most important clay mineral cause for expansive nature is montmorillonite has an octahedral sheet sandwiched between two silica sheets. When this mineral was exposed to moisture, water absorbed between interlayering lattice structures and exert an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with expansive soils.

Expansive soils form a major soil group in Ethiopia and occur in the highlands mostly in the western, central and southwestern part of the country and in most cases the major clay mineral component is montmorillonite. The road sectors in Ethiopia are suffering from the high shrink-swell behavior of this expansive soil. Many damages occur each year and road construction over such expansive soil creates serious problems including increasing cost of construction and maintenance. The probability of damages increases for structures on swelling foundation soils if the climate and other field environment, effect of construction, and effect of occupancy tend to promote moisture changes in the soil (Dinkie Adimas, 2015).

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis Gohatsion, Addis- Modjo are some of the areas covered by expansive soils (Ayenew Z., 2004).

Substantial damage has been occurring in Ethiopia on buildings and roads that are constructed on expansive soils with severe economic consequences, psychological effects and loss of proper functioning of structures. As the engineering properties of expansive soil of Ethiopia are different from the same soil in other locality, a detailed investigation and research should be carried out on expansive soils to know in more detail the behavior of the soils and make remedial measures that are safe and economical.

The degree of expansiveness of soils varies from place to place depending upon type of parent material, climate and topography. Amount and type of clay minerals in the soil, thickness of expansive soil zone, and thickness of the active zone also affect the degree of shrinkage and swell (Aklilu A., 2015).

Introduced by Karl von Terzaghi, the "father of soil mechanics and geotechnical engineering", the term "Soil Consolidation" refers to the process in which the volume of a partially or fully saturated soil decreases due to an applied stress. Consolidation of clayey soils play a crucial role in geotechnical engineering; viz.; in improving the strength of soft soils, stability analysis, settlement behavior, foundation problems and achieving safety and economy in the construction of various civil engineering projects. The effects of consolidation are more pronounced where a building sits over a layer of soil with low stiffness and low permeability, such as marine clay, leading to massive settlement over many years. There are various types of civil engineering construction projects such as embankments construction, land reclamation, tunnel and basement excavation in clay etc. where consolidation often poses huge technical risk. Process of consolidation has been formulated by many researchers as a combination of two phenomena: the permeability and compressibility. On one hand, permeability controls the rate at which the water is expelled out of the soil and thus affects the rate of settlement at any time. On the other hand, compressibility controls the evolution of excess pore pressures and thus effects the duration of the consolidation. During the consolidation process, compressibility and permeability are intertwined. In Terzaghi's theory, a linear stress-strain relationship at a constant permeability is assumed. The rate of consolidation is controlled by the coefficient of consolidation,Cv(Taslima Nasrin,2020).In order to successfully apply consolidation theory in civil engineering problems, it is necessary to determine a reliable value of the coefficient of consolidation, Cv and compression index Cc, is useful for the determination of the settlement in the field which can replicate the field characteristics of the soil. Therefore, Most of the researches shown that the engineering properties of expansive soils of Ethiopia are covered, but still consolidation characteristics of soil have not that much assessed.

1.2 Statement of the problem

The premature failure of structures like lightweight buildings and roads caused by expansive soil is tremendously high and alarming. These failures are mainly dominated in light weight-engineering structures founded in expansive soils. The problem associated with expansive soils is not yet properly solved. Study on Relationship between Consolidation and Swelling Characteristics of Expansive Soils of the study area had not been studied. Therefore, this thesis would be helpful to the study area.

1.3 Research Question

At the end of this study the research would try to answer the following question:

- What are the ranges of values of the consolidation parameters of expansive soil of mojo town?
- What are the factors that influence the values of the Consolidation-swell results expansive soil of mojo town?

1.4 Objectives

1.4.1 General Objective

The main objective of this thesis is to identify the relationship between consolidation and swelling characteristics of expansive soil of mojo town.

1.4.2 Specific Objectives

The specific objective of this thesis is to:

- Establish relevant relationship between consolidation and swelling characteristics of expansive soil Mojo town.

- Determine the ranges of values of the consolidation parameters of expansive soil of mojo town.
- Determine the index properties of the mojo expansive soil.

1.5Scope of the study

The scope of this research is limited to study the relationship between consolidation and swelling characteristics of expansive soils of mojo town. This study would be supported by different types of literatures and a series of laboratory experiment. The relevant laboratory tests were conducted like, Free swell test, Specific Gravity test, Atterberg limits test, Grain Size distribution test Standard Compaction test, Swell –Consolidation test on undisturbed samples, Swell–Consolidation test on disturbed samples by remolding at different initial moisture content & dry density. The results were formulated and drawn by ASHTO, ASTM, and USCS Standard.

1.6 Significance of the study

This study would give a significant understanding of the relationship between consolidation and swelling characteristics of expansive soils. Moreover, the study would give some input parameters for future researchers in the area of consolidation-swelling characteristics of soil of mojo town.

CHAPTER TWO

LITERATURE REVIEW

2.1 Expansive soils

Expansive soils are materials that increase and decrease significantly in volume as moisture content changes. The expansive soils swell if their moisture content increases and they shrink if their moisture content decreases. The phenomenon depends on the mineralogical combination of the clay soils. When water is added to expansive clays, the water molecules are pulled into gaps between the clay plates. As more water is absorbed, the plates are forced further apart, leading to an increase in soil pressure or an expansion of the soil's volume. If the liquid limit of a soil sample exceeds 50% and the plasticity index exceeds 30%, the soil shall be considered as having expansive clay mineralogy. The clay mineral responsible for most expansive damage is smectite, although bentonite and illite also have some expansive potential. Another category of expansive soil known as swelling bedrock contains a special type of mineral called clay stone. The montmorillonite clay minerals, one of the smectite group, are considered as a highly expansive and the most effective ones for swelling behavior. This type of soil is found in many places all over the world especially in the arid and semi-arid regions. More than 2.4 million square km (240 million ha) of Vertisols, clayey soils high in smectite, are distributed throughout the world. Extensive areas are found in Australia (80 million ha), India (73 million ha), Sudan (50 million ha), southern and western states of United States (12.8 million ha), Egypt, Ethiopia, Chad, Ghana, Cuba, Puerto Rico, and Taiwan (Wang J. D. et al. 2016).Clay mineralogy dictates the potential for clays to shrink and swell under various moisture levels. Although there are many types of expansive clays, most consist of kaolinite, illite, and smectite, with montmorillonite being a very common and important member of the smectites. The size of the clay particles is directly related to its sensitivity to water, with larger clay particles being less sensitive to water. Kaolinite has the largest clay mineral crystal, lowest specific surface area, and typically low activity. Activity is defined as the plasticity index of the soil divided by the percent of clay-sized particles (soil particles less than 2 micrometers). An activity higher than 1.25 indicates that a soil is susceptible to larger changes in volume when wetted and dried.Montmorillonites have the smallest clay mineral crystal, greatest specific surface, and highest activity. Montmorillonites are generally considered the most susceptible to swelling as their water content increases (Eli Cuelho and Michelle Akin 2017).

2.2. Origin of Expansive Soil

The origin of expansive soil is related to a combination of conditions and processes that results in the formation of clay minerals having a particular chemical and mineralogical make up, which, when in contact with water expands. Variations in the conditions and processes may also form other clay minerals, most of which are non expansive. The conditions or processes that determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected (Ayenew Z., 2004).

2.3. Distribution of expansive soils

Potentially expansive soils can be found almost anywhere in the world. In the underdeveloped nations, many of the expansive soil problems may not have been recognized because of less intensity of construction. It is to be expected that more expansive soil regions related problems would be reported each year as the amount of construction increases. Expansive soils are in abundance where desiccation phenomenon is common i.e., where the annual evaporation exceeds the precipitation. The problem of expansive soil is widespread throughout the five continents (Ayenew Z., 2004).

2.3.1. Distribution of Expansive soils in Africa

Expansive soils are widespread in Africa occurring in South Africa, Ethiopia, Kenya, Mozambique, Morocco, Nigeria etc. In other parts of the world cases of expansive soils have been widely reported in USA, Australia, Canada, India, Spain, Israel, Turkey, Argentina, Venezuela, etc. .Large parts of Ethiopia and Sudan are covered with black clay soils, as are smaller areas in Tanzania, Kenya, Uganda, Malawi and Zambia. The associated volcanic rocks are the main cause of the black clays formation in Ethiopia, Kenya and Sudan (Aklilu F., 2015).

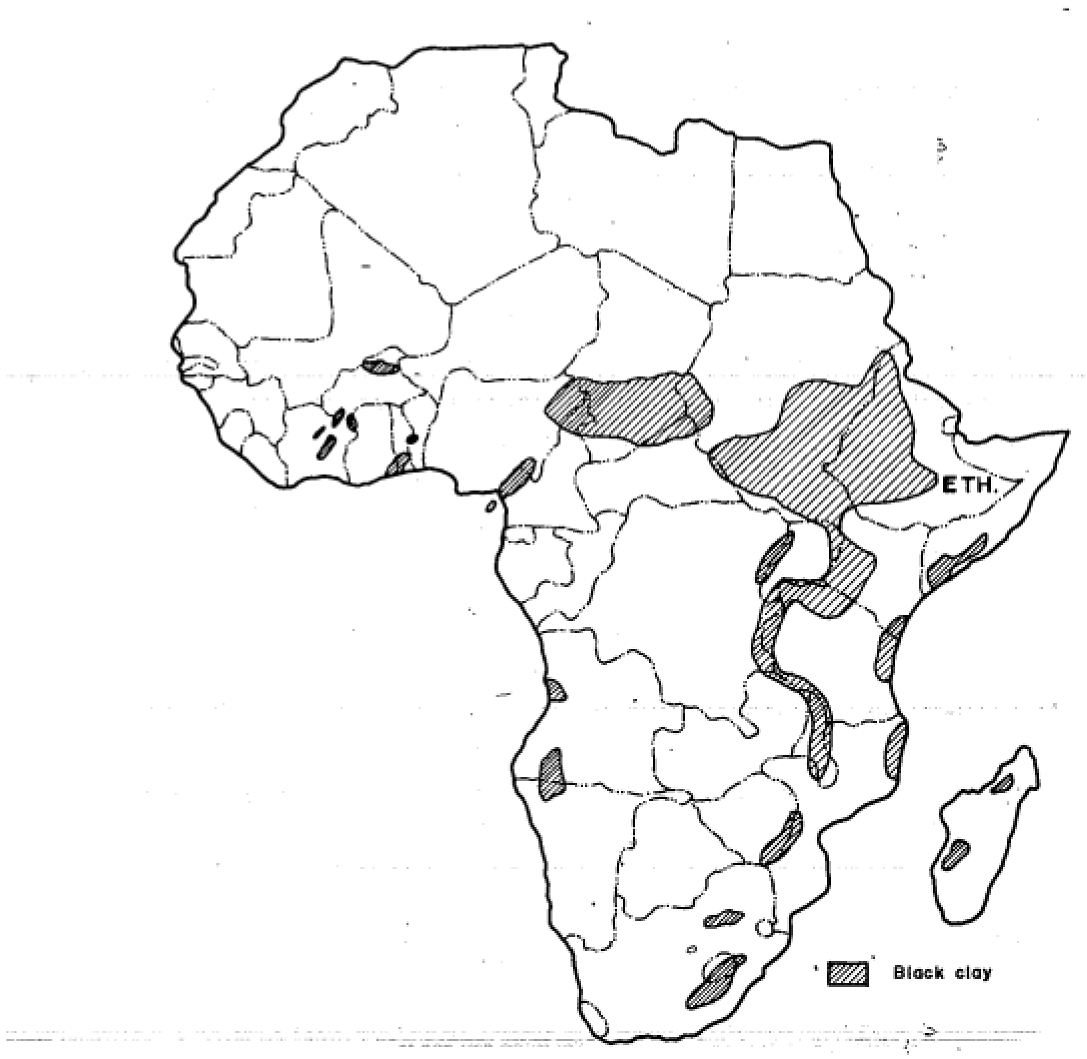


Figure 1: Distribution of Expansive soils of Africa (Alemayehu T. & Mesfin L. 1999).

2.3.2. Distribution of Expansive Soils in Ethiopia

Most of the Ethiopian plateau and part of the Somalian plateau are covered with tropical black clays. The intervening Rift Valley has a variety of soils - alluvial, lacustine, and residual black clays over the more recent volcanics .The Ethiopian dark clay soils located in the Rift Valley and Ethiopian plateau are estimated to cover 10 million hectares.Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis Ababa-Ambo, Addis Ababa-Woliso, Addis Ababa –Debre Berhan, Addis Ababa -

Gohatsion, Addis Ababa - Modjo are some of the areas covered by expansive soils. Areas like some part of Mekele, Gondor, Bahirdar, Debre berhan and Gambela are also known to be partly covered by expansive soils. Expansive soil is a major problem soil of Africa and other parts of the world. Large parts of Ethiopia are covered by black and gray expansive soil. The expansive clays of Ethiopia are residual, derived from the weathering of basic volcanic rocks, which cover part of Ethiopia. In the capital, Addis Ababa, it has been noticed that expansive soils covers large parts of the city where recent constructions are carried out. Most of the structural damage on expansive soil results from the differential rather than the total movement of the foundation soil as a result of swell. Damages can occur within a few months following construction, may develop slowly over a period of about few years, or may not appear for many years until some activity occur to disturb the soil moisture equilibrium (Aklilu., 2015).

Figure 2: Distribution of Expansive soils in Ethiopia (Aklilu F., 2015).

2.4. Mineralogy of expansive soil

The behavior of the soil mass mainly depends on micro scale factors such as:-the amount and type of clay minerals in the soil, the chemical structure, the specific area of the clay particles, the soil water chemistry contained within the voids. According to ASTM the term clay is applied to the fraction of grains whose equivalent diameter is less than 0.005mm. The individual grains are fragments of a single mineral i.e. a solid compound with a definite chemical composition and unique crystalline structure. The minerals of clays are formed by the weathering of rocks (Asamnew G.,2016).

There are three common types of clay minerals are Kaolinite, Illite and Montmorillonite (Alemayehu T. & Mesfin L. 1999).

Kaolinite - low degree of expansiveness

Illite - moderate degree of expansiveness

Montmorillonite - very high degree of expansiveness

Kaolinite

Kaolinite has a structural unit made up of alumina sheets joined to silica sheet. The bond that exists between layers is tight and hence it is difficult to separate the layers. As a result, kaolinite is relatively stable and water is unable to penetrate between the layers. Consequently, kaolinite has low degree of expansiveness.

Illite

Illite has a basic structure similar to that of montmorillonite. However, the basic illite units are bonded together by potassium ions which are non-exchangeable. Because of this, the illite units are reasonably stable and so that mineral swells much less than montmorillonite. Hence, illite has moderate degree of expansiveness.

Montmorillonite

Montmorillonite is the most common of all the clay minerals and is well known for its swelling properties. Its basic structure consists of an alumina sheet sandwiched between two silica sheets the basic montmorillonite units are stacked one on top of the other, but the bond between the individual units is relatively weak and water is easily able to penetrate between the sheets and cause their separation and hence swelling. Therefore, montmorillonite has very high degree of expansiveness.

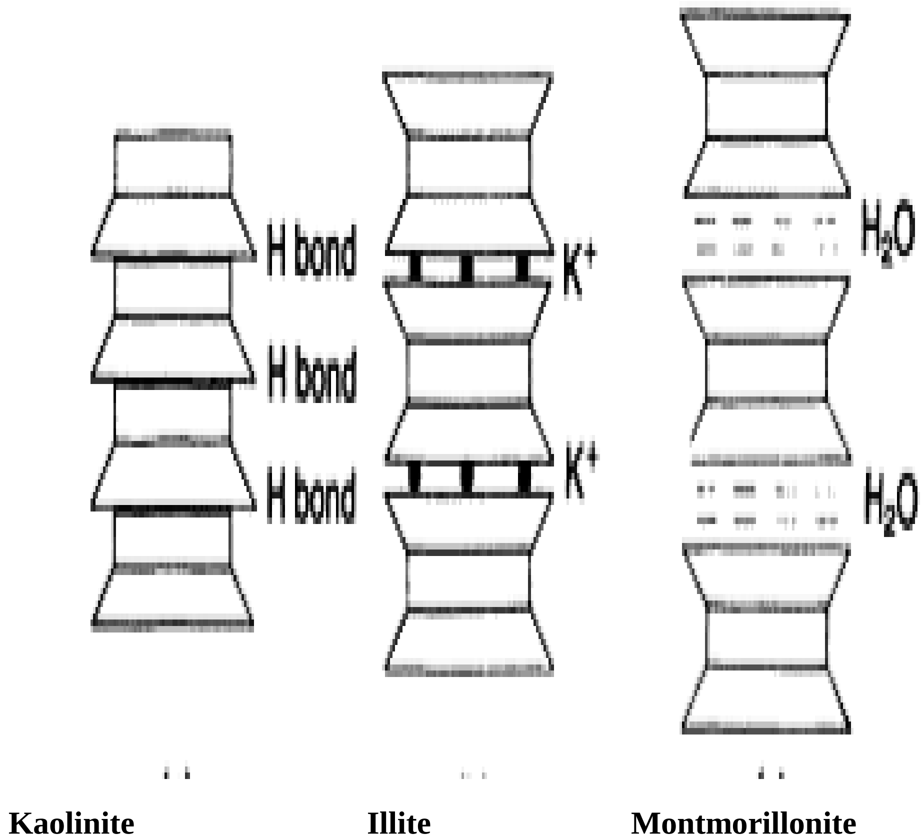


Figure 3: Clay Minerals (R.F Craig 2004)

2.5. Identification and Classification of Expansive Soils

2.5.1 Identification of Expansive Soils

A. Field Identification

It is evident that expansive soil can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include (Tirsit D. 2017).

- Usually have a color of black or gray
- Wide or deep shrinkage cracks.
- High dry strength and low wet strength.
- Stickiness and low trafficability when wet.

- Cut surfaces have a shiny appearance
- Appearance of cracks in nearby structures.
- Arid and semiarid areas are particular trouble spots because of large variations in rainfall and temperature

B. Laboratory Identification

There are three different methods of identifying potential expansive soil in laboratory. These are: (Tirsit D. 2017).

- Mineralogical methods of identification
- Direct measurement
- Indirect methods of identification

I. Mineralogical Methods of Identification

Type of clay mineral is a fundamental factor controlling expansive soil behavior. Clay minerals can be identified using a variety of techniques, some of the common types of mineralogical tests are:

- X-ray Diffraction
- Differential Thermal Analysis (DTA)
- Electron Microscopy

ii. Direct Measurement

These methods offer the most useful data by direct measurement and tests are simple to perform and do not require complicated equipment. Testing should be performed on a number of samples to avoid erroneous conclusions. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer.

iii. Indirect measurement

These methods include the index property, Potential Volume Change (require special PVC apparatus) and activity methods, which are valuable tools in evaluating the swelling property. However, none of the indirect methods should be used independently.

2.5.2. Classification of expansive soils

A soil classification is a systematic method of categorizing soils into various groups and subgroups according to their probable engineering behavior but without detailed descriptions. Most of the earlier classification systems were based on grain size distribution (e.g. MIT classification) and soil texture (e.g. textural classification). However, with the introduction of Atterberg limits, new classification systems have come into existence.

It is possible to trace many soil classification systems such as the Casagrande's unified soil classification system (USCS), the American Association of State Highway and Transportation Officials (AASHTO) system, the pedologic soil classification system, Federal Aviation Agency (FAA) system, US public roads administration (PRA) system, and the textural classification system. Currently, the USCS and the AASHTO system are in use in civil engineering practice. The USCS was originally developed by Casagrande and was later modified by the US Bureau of Reclamation (USBR) and the US Army Corps of Engineers, to enhance its applicability to many more fields. Many national standard codes of practice such as ASTM designation D2487-93, BS 5930 and IS 14984 follow this modified version of the USCS as it stands or with slight modification. Both systems, namely modified USCS and AASHTO, base their classification of soils for engineering purposes on particle size characteristics, liquid limit (WL) and plasticity index (Ip) of soils. The sub grouping of coarse-grained soils is done with the help of parameters such as uniformity coefficient (Cu) and coefficient of curvature (Cc) to account for the gradation of soils. The sub grouping of fine-grained soils is entirely based on a plasticity chart (i.e. Ip plotted against wL). In addition, some codes of practice give some useful criteria that need to be followed to obtain a rough estimation of soil characteristics such as angularity (of coarse-grained soils), moisture content, consistency, cementation, dry strength, plasticity, dilatancy, toughness, organic content (e.g. ASTM designation D2488-93, IS 14984). However, apart from IS 1498, these systems do not have the criteria to assess the expansivity of the soil (A. Sridharan and K. Prakash 2000).

The identification/classification techniques represented by the index property indicator were selected generally on the basis of their simplicity and the indicated correlations with measured volume change. With regard to field experience, several of the techniques have been used considerably with reasonable success. The techniques selected for evaluation are described in some detail.

(Louisiana Department of Transportation).

The Louisiana Department of Transportation uses the Atterberg limits (liquid limit (LL) and plasticity index (PI)) balanced with field experience to identify potentially expansive soils. Table 1: The criteria used for identifying and classifying potential swell are:

LL, %	PI, %	Potential Swell Classification
20-49	15-24	Low to medium
50-70	25-46	High
>70	>46	Very high to severe

(Raman,V. 1967).

This method uses the Atterberg limits (LL, PI, and shrinkage limit (SL)), but in a different configuration. Baman defines the shrinkage index (SI) as the difference between the LL and the SL (LL- SL).

Table 2: The criteria recommended by Raman for identification/classification are

PI %	SI %	Degree of Expansion
>12	<15	Low
12-23	15-30	Medium
23-32	30-40	High
>32	>40	Very high

(Dakshanamurtthy,V and Raman,V. 1973).

This method is based on a modification of Casagrande's plasticity chart, which includes PI and LL, with the addition of the SI (LL- SL).

Table 3: A simplification of the procedure using the LL is:

LL, %	Potential Swell Classification
0-20	Non swelling
20-35	Low swelling
35-50	Medium swelling
50-70	High swelling
70-90	Very high swelling
>90	Extra high swelling

(Seed,woodward and Lundgren,1962).

The potential swell of an expansive soil is defined from correlations of percent swell from odometer tests using laboratory prepared and compacted samples with percent clay size (-2 ll m) and soil activity. A statistical relationship is defined for potential swell in terms of clay content and activity and compared with measured volume change.

Table 4: The potential swell may be categorized as follows:

PI, %	Degree of Expansion
Potential Swell, %	Low Very high
0-1.5	Medium
5-25	High
>25	Very high

2.6. Damages caused by expansive soils

Expansive soils owe their characteristics to the presence of swelling clay minerals. As they get wet, the clay minerals absorb water molecules and expand; conversely, as they dry they shrink, leaving large voids in the soil. Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties. Expansive soils are found in areas with poor internal drainage and low to moderate rainfall (J. David Rogers, Robert Olshansky, et al).

A potentially expansive soil is not necessarily damaging unless it is subjected to moisture changes, which may result from seasonal climatic changes. Foundation soils which are expansive will “heave” and can cause lifting of a building or other structure during periods of high moisture. Conversely during periods of falling soil moisture, expansive soil will “collapse” and can result in building settlement. Either way, damage can be extensive (Aklilu A., 2015).

2.6.1 Foundation Damage

The most obvious way in which expansive soils can damage foundations is by uplift as they swell with moisture increases. Swelling soils lift up and crack lightly-loaded, continuous strip footings, and frequently cause distress in floor slabs. Because of the different building loads on different portions of a structure's foundation, the resultant uplift will vary in different areas. As shown in Figure: 5 the exterior corners of a uniformly-loaded rectangular slab foundation will only exert about one-fourth of the normal pressure on a swelling soil of that exerted at the central portion of the slab. As a result, the corners tend to be lifted up relative to the central portion. This phenomenon can be exacerbated by moisture differentials within soils at the edge of the slab. Such differential movement of the foundation can also cause distress to the framing of a structure (J. David Rogers, Robert Olshansky, et al).

Figure 4: Polygonal pattern of surface cracks in the dry season. These cracks are approximately one inch wide at the top. Note sewer manhole in background (J. David Rogers, Robert Olshansky, et al).

Figure 5: This crack is at least 32 inches deep. The yardstick was easily inserted to this depth; narrower, less straight cracks may extend much deeper (J. David Rogers, Robert Olshansky, et al).

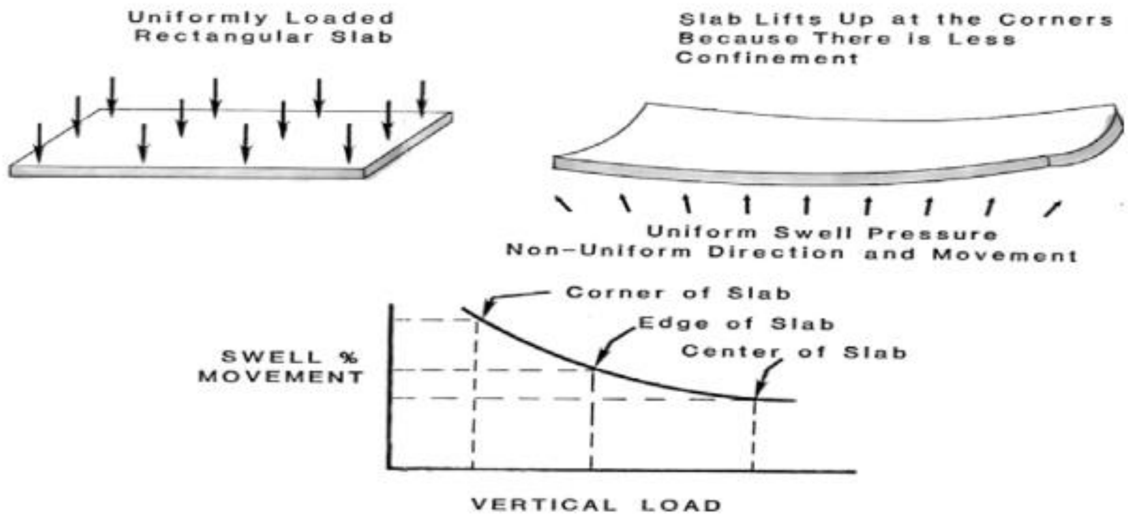


Figure 6: A rectangular slab, uniformly loaded, will tend to lift up in the corners because there is less confinement (J. David Rogers, Robert Olshansky, et al).

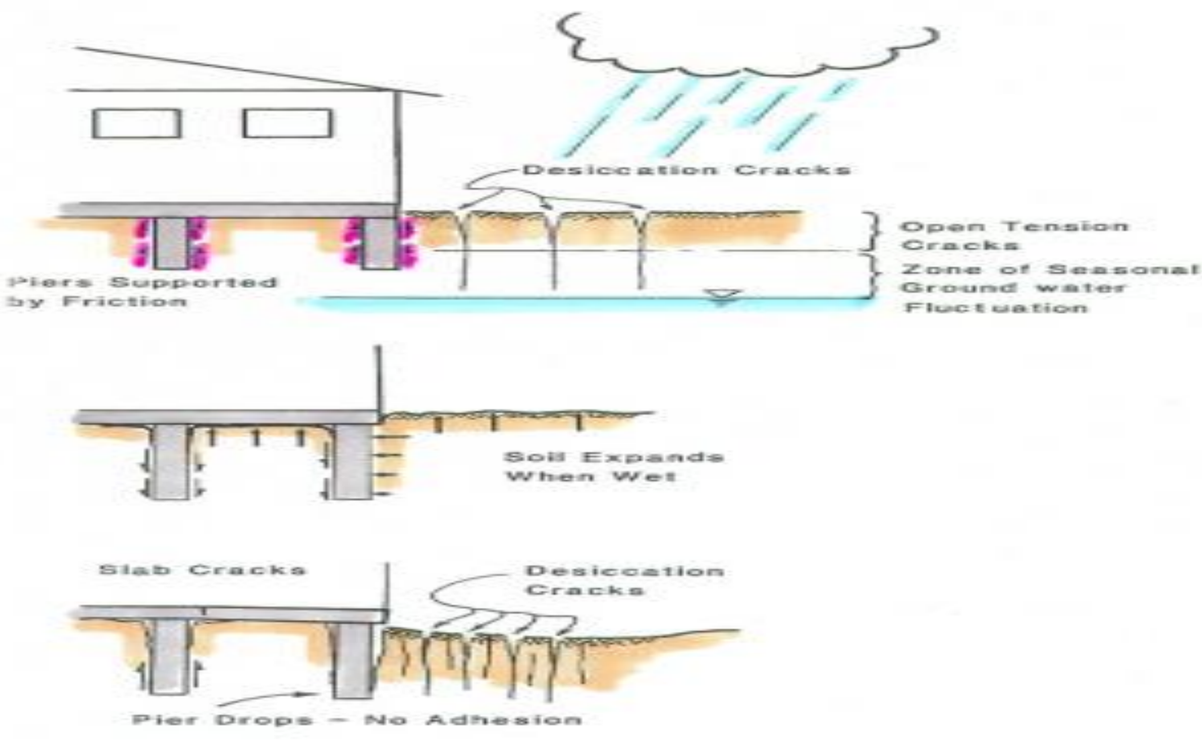


Figure 7: Damage to home supported on shallow piers (J. David Rogers, Robert Olshansky, et al).

Table 5: Recommended Foundations on Expansive Soils (Girma Tessema ., 1984).

Type of Building	Degree of Expansiveness	Recommended Foundation
Long store, workshop and similar buildings	Moderate to High	3.0m deep isolated footing foundation with 1m thick soil replacement under the ground slab and improved drainage systems
Compact: light residential and single storey office Buildings	Moderate	3.0m deep isolated footing foundation with lm thick soil replacement under the ground slab and improved drainage systems
Compact: light residential and single storey office Buildings	High	slab foundations, with split or flexible type of construction if the building is longer than 15m. Improved drainage system is essential
Multi-storey and movement sensitive factory or other Buildings	Moderate to High	Under-reamed pile foundation with suspended floor slabs and grade beams, and increased rigidity of the building

2.6.2. Damages to Buildings

Expansive soils, which swell on absorption of water and shrink on removal, cause severe damage to structures if no measures are adopted beforehand. Foundations resting on expansive soils may move differentially in the vertical direction and show sign of unacceptable cracks as the position of ground water table changes with season. Observations of deformations of structural units show that both heave and settlement may occur in structures founded in swelling clays. The predominant deformation observed in buildings is that of upward heave. From data available in literature, heaving of the building generally starts some time after completion of the structure, of the order of one year. It should be noted that cracks are more likely to occur in lightly loaded structures than in heavily loaded structures; the former being easier to be moved vertically by the pressure exerted by underlying expansive soil. This fact appears clearly in severe cracking of single story domestic houses of rigid construction like load bearing brick walls which are constructed in many places of arid and semi-arid areas. The more heavily loaded buildings and more flexible constructions, e.g., multi-story frame buildings are less affected because of their large applied load, greater depths of foundations, and their flexibility. Damage caused by expansive soil appears firstly as cracks on external and internal walls, joints of brick and concrete elements or stones, and interior plaster walls. The types of cracks appearing in structures are: diagonal shear cracks, horizontal and vertical shear cracks, horizontal and vertical cracks caused by bending moments and horizontal and vertical tension cracks. It is expected that for buildings on expansive soils, the size of cracks increase with time until it leads to total damage of collapse. The engineering problems of expansive soil are the determination of parameters and criteria which can be used in the design of structures built on or with such soil (Hussein Elarabi.,2010).

Figure 8: Structural damage to house caused by ‘end lift’ (Lee D Jones ,2012).

Figure 9: Example of differential settlement due to influence of trees (Lee D Jones ,2012).

2.7. Mechanics of swelling

2.7.1 General

Swelling of expansive soils will take place under change in the environment of the soil. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. By far the most important element and of most concern to the practicing engineer is the effect of water on expansive soil. There must be a potential gradient, which can cause water migration and a continuous passage through which water transfer can take place. The potential gradient in expansive soils can be due to seasonal moisture fluctuation or thermal gradient, which can cause vapor and liquid moisture transfer. It is well recognized that the heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soils (Mesfin K., 2005).

2.7.2 .Factors influencing Swelling

The swell potential of a Expansive Soil may be affected by either the soil properties influencing the nature of the internal force field, the environmental factors those may change the internal force system or the state of stress present on the soil. Some physical factors such as initial water content, initial density, amount and type of compaction also influence the swell potential and swell parameters of soils. These factors are summarized below: (Masoumeh Mokhtari and Masoud Dehghani 2012).

2.7.2.1Soil Properties Influencing Swell Potential

i. Clay Mineralogy: Clay minerals which typically cause soil volume changes are montmorillonites, vermiculates, and some mixed layer minerals. Illites and Kaolinites are frequently inexpensive, but can cause volume changes when particle sizes are extremely fine.

ii. Soil Water Chemistry: Swelling is repressed by increased cation concentration and increased cation valence. For example, Mg2+ cations in the soil water would result in less swelling than Na+ cations.

iii. Soil Suction: Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is related to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water.

iv.Plasticity: In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinkage. Plasticity is an indicator of swell potential.

v. Soil Structure and Fabric: Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at high water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water contents.

vi. Dry Density: Higher densities usually indicate closer particle spacing, which may mean greater repulsive forces between particles and larger swelling potential.

2.7.2.2. Environmental Factors Affecting Swell Potential

i. Initial Moisture Content: A desiccated expansive soil will have high affinity for water, or higher suction than the same soil at higher water content, lower suction. Conversely, a wet soil profile will loose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions.

ii Moisture Variations: Changes in moisture in the active zone near the upper part of the profile primarily define heave, it is in those layers that the widest variation in moisture and volume change will occur.

iii.Climate: Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have short wet periods.

iv.Groundwater: Shallow water tables provide source of moisture and fluctuating water tables contribute to moisture.

v.Drainage: Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth.

vi.Vegetation: Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation.

vii.Permeability: Soils with higher permeability, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.

viii.Temperature: Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.

2.7.2.3. Stress Conditions Affecting Swell Potential

i. Stress History: An over consolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting-drying cycles, swell is unaffected.

ii.In situ Conditions: The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in-situ measurements and observations.

iii.Loading: Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance inter-particle repulsive forces and reduces swell.

Iv.Soil Profile: The thickness and location of potentially expansive layers in the profile considerably influence potential movements. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by non-expansive material or overlies bedrock at shallow depth.

2.8Consolidation

2.8.1 General

The ultimate change in volume of soil occurring under a change in applied stress depends on the compressibility of the soil skeleton of the soil particles. However, the water in the voids of a saturated soil is relatively incompressible and, if no drainage takes place, change in applied stress results in a corresponding change in pore pressure, and the volume change is negligible. As the drainage takes place, water flows from zone of high excess pore pressure to zone of less or zero excess pore pressure, and thus the excess pore pressure dissipate. The applied stress is then transferred to the soil skeleton and volume change takes place. It is this time dependent volume change of cohesive soils resulting from dissipation of excess pore pressure, which is known as consolidation. The consolidation of a soil may be considered to consist of two stages: the primary consolidation stage during which the applied pressure increment is transferred from the pore water to the skeleton, and the secondary consolidation stage that follows the end of primary phase. A study of consolidation requires knowledge of the compressibility of the soil skeleton and the rate at which excess pores pressure dissipates which is related to the permeability. The compressibility characteristics of a soil relating both the magnitude and the rate of settlement are usually determined from the consolidation test, using an apparatus called oedometer (Fasil A., 2003).

2.8.2 Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. The soil layers are unable to spread laterally as the surrounding soil strata confines them. Hence there must be adjustment to the new pressure by vertical deformation. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. The vertical compression of the soil mass under increased pressures is thus made up of the following components:

i. Deformation of the soil grains

ii. Compression of water and air with in the voids

iii. An escape of water and air from the voids

It is quite reasonable and rational to assume that the solid matter and the pore water relatively are incompressible under the loads encountered. The change in volume of the soil mass under imposed stresses must be only due to the escape of water and air (Mesfin K., 2005).

The consolidation of a soil deposit can be divided in to three stages:

a) Initial Consolidation

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles (Dr.K.R.Arora. 2004).

b) Primary Consolidation

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as an excess pore water pressure, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs .the decrease depends upon the permeability of the soil and is, therefore, time dependent .this reduction in volume is called primary consolidation

In fine grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability. As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles. Thus, the effective stress is increased: (Dr.K.R.Arora. 2004)

c) Secondary consolidation

The reduction in volume continues at a very slow rate even after the excess hydrostatic pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in the volume is the called secondary consolidation.the causes for particles and adsorbed water to the new stress system. In most inorganic soils, it is generally small: (Dr.K.R.Arora. 2004)

2.8.3 Factors Affecting the Consolidation Characteristics of Soils

The consolidation behavior of clay soil in its natural state is highly dependent on stress history and permeability. The effects of these factors are explained below.

i. Effect of soil type

For saturated fine-grained soils the major factor in the escape of pore water from the soil is the time required. When compared to granular soils where expulsion of pore water takes place unimpeded with a small time lag, much longer time is needed in fine-grained soils for pore water to escape. The basic difference in the compression behavior of a granular soil and that of a fine-grained soil can therefore be expressed as granular soil compresses almost immediately up on loading. But the compression is relatively small whereas fine-grained soil exhibits time dependent compression and the resulting compression is rather large. The magnitude of time lag is basically influenced by the permeability of the soil (Aklilu F., 2015).

ii. Stress history of soils

Soils tend to retain the effects of stress changes that have taken place in their Geological history in the form of their structure. A soil which is subjected to a certain effective stress for the first time in its history will obviously be more compressible than when it has been subjected to a large effective stress in its earlier history but is now relieved of that effective stress due to some reason. Based on the stress history of saturated cohesive soils, they are considered to be under consolidated, normally Consolidated, or over consolidated which are usually described by the over Consolidation ratio (OCR) and defined as

OCR = Pc/ Po

Where:

Pc is Preconsolidation pressure

Po is insitu overburden pressure

OCR is over consolidation ratio

a) Pre-consolidation pressure Pc

A soil may have been pre-consolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. The practical significance of the pre-consolidation load appears in calculating settlements of structures (Aklilu F., 2015).

b) Under consolidated (OCR < 1)

A saturated cohesive soil is considered under consolidated if the soil is not fully consolidated under the existing overburden pressure and excess pore water pressures exist within the soil. Under consolidation occurs in areas where a cohesive soil is being deposited very rapidly and not enough time has elapsed for the soil to consolidate under its own weight (Aklilu F., 2015).

c) Normally Consolidated (OCR = 1)

A saturated cohesive soil is considered normally consolidated if it has never been subjected to an effective stress greater than the existing overburden pressure and if the deposit is completely consolidated under the existing overburden pressure (Aklilu F., 2015).

d) Over consolidated or Pre consolidated (OCR > 1)

A saturated cohesive soil is considered over consolidated if it has been subjected in the past to a vertical effective stress greater than the existing vertical effective stress. For structures constructed on top of saturated cohesive soil, determining the over consolidation ratio of the soil is very important in the settlement analysis. For example, if the cohesive soil is under consolidated, then considerable settlement due to continued consolidation owing to the soil's own weight as well as the applied structural load would be expected. On the other hand, if the cohesive soil is highly over consolidated, then a load can often be applied to the cohesive soil without significant settlement (Aklilu F., 2015).

iii. Effect of effective stress

Due to application of stress to a saturated soil, the water in the pore starts to flow out. The flow continues as long as there is hydraulic gradient. As this transient flow continuous there will be reduction in volume causing an increase in effective stress. Free water and/or gas bubbles in the voids of mineral soils cannot transfer shear stresses. Hence the soil skeleton alone transfers shear stresses. Thus the effective normal stresses govern the internal resistance of granular soils irrespective of the Shear stress. As a soil is subjected to a stress the soil undergoes a decrease in void ratio. If the same stress is applied again the soil undergoes a decrease in void ratio which is of course less than the decrease in initial void ratio (Aklilu F., 2015).

2.8.4 One-dimensional consolidation

Since water can flow out of a saturated soil sample in any direction, the process of consolidation is essentially three-dimensional. However, in most field situations, water will not be able to flow out of the soil by flowing horizontally because of the vast expanse of the soil in horizontal direction. Therefore, the direction of flow of water is primarily vertical or one-dimensional. As a result, the soil layer undergoes one-dimensional or 1-D consolidation settlement in the vertical direction (Jaafar A Mohammed. 2015).

The standard one-dimensional consolidation test is usually carried out on saturated specimen using an Odometer.In this test a small representative sample of soil is carefully trimmed and fitted into a rigid metal ring. The soil sample is mounted on a porous stone base and a similar stone is placed on top to permit water, which is squeezed out of the sample to escape freely at the top and bottom. Prior to loading, the height of the sample should be accurately measured. Also, a micrometer dial is mounted in such a manner that the vertical strain in the sample can be measured as loads are applied. The consolidation test apparatus is designed to permit the sample to be submerged in water during the test to simulate the position below a water table of the prototype soil sample from which the test sample was taken. Loads are applied in steps in such a way that the successive load intensity, P, is twice the preceding one; the load intensities commonly used being ¼, ½, 1, 2, 4,8,16 kg/cm2. Each load is allowed to stand until primary consolidation is practically ceased. The dial readings are taken at elapsed time of 0, .0.25, 0.50, 1, 2, 4, 8, 15, 30,60minute.......24hours. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitude of plastic or permanent deformation. The consolidation characteristics (or parameters) of a soil which are the compression index, Cc, and the coefficient of consolidation, Cv will be determined from the test. The compression index relates to how much consolidation or settlement will take place. The coefficient of consolidation on the other hand relates to how long it will take for an amount of consolidation to take place (Mesfin K., 2005).

Figure 10: A schematic diagram of an oedometer (Mesfin K., 2005).

The results of the typical oedometer test are usually presented in the form of an e-P, e-log P, and dial reading- time plots.

i. Consolidation index

Consolidation index Cc, is numerically equal to the slope of the straight portion of the void ratio versus log pressure plot from e-log P curve (Figure: 9). Its value is constant beyond the range of the recompression, since beyond this point the plot of e against log P is a straight line. Cc is an important index used to calculate the ultimate settlement of a foundation founded on a clay layer.

Thus

$$C_c = \frac{e_1 - e_2}{\log^{P_2} - \log^{P_1}} \qquad S = \frac{C_c}{1+e} H \log_{10} \frac{p_o + \Delta p}{p_o}$$

Where:

Cc-Consolidation Index

e -Void ratio

p - Pressure in log scale

H-Height of soil strata

S-Ultimate Settlement

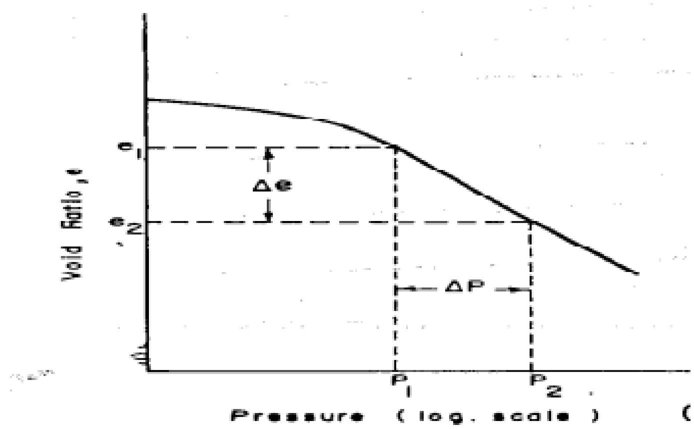


Figure 11: Determination of consolidation index (Aklilu F., 2015).

ii. Coefficient of consolidation, Cv

The coefficient of consolidation Cv may be found experimentally from conventional laboratory 1-D consolidometer test results. Both Casagrande and Taylor methods has determined as follows. The coefficient of consolidation Cv as determined by Casagrande's semi logarithmic plot method is

$$Cv=\frac{(0.197)H^2}{t_{90}}\text{..........}\underline{\text{Cm2/S}}$$

Cv = coefficient of consolidation of stratum, in cm2 /sec

H = equivalent specimen thickness, in cm

t50 = time at 50 percent of primary consolidation, in sec

The Cv value as determined by Taylor's square root of time fitting method is (0.848).

$$Cv = \frac{(0.848)H^2}{t_{90}}\text{..........}\underline{\text{Cm2/S}}$$

Cv = Coefficient of consolidation of stratum, in cm2 /sec

H = Equivalent specimen thickness, in cm

t90 = Time at 90 percent of primary consolidation, in sec

2.84.1Determination of Coefficient of consolidation, Cv

Two methods are available for evaluation of Cv from consolidometer test data. The first called the square root of time fitting method is due to Taylor & the second called the logarithm of time fitting method is due to Casagrande. The two methods are based on the comparison of the laboratory and theoretical time curves. Since the natural scale doesn't offer the best representation of time, the square root & the logarithm of time are used (Alemayehu T. & Mesfin L. 1999).

i. Square-root-of-time fitting method

A straight line is drawn through the points representing the initial readings that exhibit a straight line trend. Then the line is extrapolated back to t=0 and the deformation ordinate representing 0% primary consolidation is obtained. A second straight line through the 0% ordinate is drawn so that the abscissa of this line is 1.15 times the abscissa of the first straight line through the data. The intersection of this second line with the deformation-square root of time curve is the deformation, d90, and time, t90, corresponding to 90% primary consolidation Alemayehu T. & Mesfin L. 1999).

The deformation at 100% consolidation is 1/9 more than the difference in deformation between 0 and 90% consolidation. The time of primary consolidation, t100, may be taken at the taken as the intersection of the deformation-square root of time curve and this deformation ordinate. The deformation, d50, corresponding to 50% consolidation is equal to the deformation at 5/9 of the difference b/n 0 and 90% consolidation (Alemayehu T. & Mesfin L. 1999).

ii. Logarithm-of-time-fitting method

A straight line is drawn through the points representing the final readings which exhibit a straight line trend and constant slope. A second straight line is drawn tangent to the steepest part of the deformation-log time curve. The intersection represents the deformation, d100, and time, t100, corresponding to 100% primary consolidation. Compression in excess of the above estimated 100% primary consolidation is defined as secondary compression. The deformation representing 0% primary consolidation is selected by choosing any two points that have a time ratio of 1 to 4.The deformation at the larger of the two times should be greater than , but less than of the total deformation for the load increment. The deformation corresponding to 0% primary consolidation is equal to the deformation at the smaller time, less the difference in the deformation for the two selected times. The deformation, d50 corresponding to 50% primary consolidation is equal to the average of the deformations corresponding to the 0 and 100% deformations. The time, t50, required for 50% consolidation may be found graphically from the deformation-log time curve by observing the time that corresponds to 50% of primary consolidation on the curve (Alemayehu T. & Mesfin L. 1999).

2.9 Previous Works

Some related studies on relationship between Consolidation and swelling characteristics of of expansive soils found in Ethiopia were conducted in the past for academic purposes. Some of these works are described as follows:-

2.7.1 Study of Mesfin kasa

A consolidation-swell test is carried out on nine undisturbed and eleven disturbed samples of expansive soils of Addis Ababa. The test results obtained from consolidation swell test on disturbed as well as undisturbed samples of expansive soil of different density and initial moisture content are presented in the form of void ratio- log pressure, and dial reading- square root of time plots. The test results are grouped into two for the purpose of the analysis. The first group includes test results of the undisturbed samples these results are used to determine the range of values of the consolidation parameters, Cc and Cv. the second group includes test results of both disturbed and undisturbed samples used to determine the relationship between consolidation and swelling characteristics of expansive soil of Addis Ababa. and the following conclusions were made

- It is known that the magnitude of the swelling of expansive soils varies with environmental conditions and it is observed from this research that the consolidation Characteristic of expansive soils is influenced by the swelling. Therefore, the consolidation characteristic of expansive soil is not inherent property of the soil mass.
- It is pointed out from this thesis that consolidation parameters are not constant for a given expansive soil. But the Atterburg limits are inherent property of the given soil. Therefore, empirical relations developed to determine compression index from theplasticity (Liquid limit) in non expansive soil will no more be applied for expansive soil.
- This thesis has shown that the compression index, Cc, and coefficient of consolidation, Cv, of expansive soil of Addis Ababa as determined from the consolidation-swell test on undisturbed samples ranges from 0.3-0.4 and from 0.139-0.356 m^2/year, respectively in an initial moisture content range of 37.5-41.5%.

2.7.2 Study of Aklilu Fikadu

A total of twelve samples that may represent the study area were gathered within a reasonable sampling interval. Routine laboratory standard tests were conducted on basic index property tests of the soils just for identification and classification of the soils.

The swell and swell pressure are generally determined in the laboratory with onedimensional consolidometer. One-Dimensional consolidation test method also used for determining the magnitude and rate of consolidation of soil. The data from the consolidation tests are used to estimate the magnitude and rate of both differential and total settlement of a structure of earth fill. Estimates of this type are of key important in the design of engineered structures and the evaluation of their performance.

In this research one dimensional swelling and consolidation characteristics of the soil were determined. The main purpose of the test is to obtain information on the swell & compression properties of the soil for use in examining the relationship between swelling & consolidation characteristics of the expansive soil of Galan Town and the following conclusions were made.

- The expansive soil of Galan Town, according to USCS & AASHTO is classified as Inorganic Clay of high plasticity, CH & A-7-5 respectively.
- The amount & rate of swell decreases as moisture content increases. The degree of swell depends on the amount of water absorbed by the soils at the time of swelling. On the other hand, for constant moisture content swelling potential increase with increase in dry density. Therefore, swelling of expansive soil is significantly influenced both by moisture content & dry density.
- Increasing the initial moisture content results in low swelling potential. From this, one can conclude that pre-wetting on the expansive soil reduces the amount of swell causing pre-expansion of the soil in which further expansion pressure on a new foundation will be minimized

- Maximum swelling pressure is obtained from the samples with optimum moisture content & maximum dry density. This implies that, swelling pressure is highly influenced by dry density.
- When the confining pressure of a structure does not exceed the pressure exerted by the expanding soil (swelling pressure), foundation movement will occur on the form of heave or upward movement.
- The compressibility index (C_c) of expansive soils is considerably influenced by its swelling & degree of disturbance due to the amount of water absorbed by the soil mass. Therefore, the consolidation characteristic of expansive soil is not inherent property of the soil mass thus swelling is additional factor that Influence its consolidation characteristics.
- Maximum value of compressibility index (C_c) is obtained from dry samples with high degree of expansion. For the saturated samples, the degree of swelling is low due to low amount of absorbed water; as a result of which low value of compressibility index (C_c) is obtained. This implies that consolidation index (C_c) is highly influenced by the degree of expansion.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

In this study, the material and methods were illustrated to achieve objectives. Significant insights for study area and sample preparations were presented. A relevant laboratory works were made to overcome the mentioned objectives.

3.2 Material

Material such as expansive soil of mojo town was used in this study and explained in detail here under this section.

Figure 12: (a) Typical Test pit Depth b) Typical crack of Building due to Expansive soil around study area

3.2.1 Expansive Soil

The soil Sample was collected from mojo town that expansive soil are dominant and the sample was collected below a depth of 1.5 up to 3m from ground level.

3.3 Description of Study Area and Methods

3.3.1 Description of Study Area

The study was performed on mojo town of expansive soil. Mojo is a town located in the central Ethiopia, named after Mojo river, located in East Shewa zone of the Oromia region 70km east of Addis Ababa and about 26kms to west of Adama town, it is an administrative center of Lome Woreda. Mojo is not only accessible by road but also has served as a train station of Addis Ababa –Djibouti railway line since 1915.

Mojo has an estimated total population of 39,316 of whom 19,278 were males and 20,038 were female (Alemayehu T. & Mesfin L., 1999).

It is also worth mentioning that, Mojo is located in rift valley of Ethiopia thus placing it in a seismic active zone. According to EBCS 8, 1995 the city is located in zone four. The location of the research area on the map of Ethiopia is shown in Figure 13.

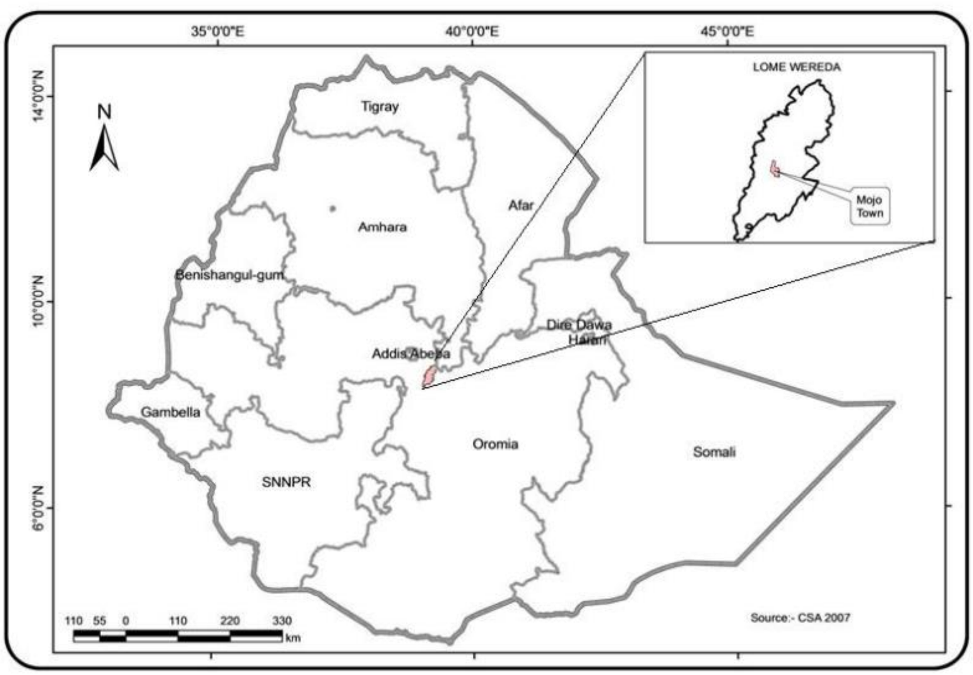


Figure 13: Location of the research area on the of Ethiopia (Mastewal G., 2016).

3.2 .2 Soil and Geology

3.2.1 Soil

According to (Mastewal G., 2016) there are three types of soils observed in the town and its surrounding:

I. Vertisols (black cotton) found in the central, south and south western part of the town.

II. Sandy and Silly soils: - these soils are observed in different colors in the areas of the town i.e. light brown, dark brown and red. Although clay soils are found in these category Silty clays, sandy silt and loam soil which is the combination of clay and silt are the main components of these soils. These soils are found in north eastern, eastern and central part of the town.

III. Alluvium: - the main compositions of these soils are sandy silt, sandy clay and flood plain. These deposits are found in the southern, northern and eastern part of the town.

3.2.2 Geology

Mojo and its surrounding areas are supposed to have been covered by the ancestral lake during the pluvial period of the Quaternary. Currently in the areas of Mojo Town the lacustrine sedimentation is found. These lacustrine sediments are the redeposit of volcanic sands, silt stone, sand stone, and diatomite with intercalations of water-laid tuff (Mastewal G.,2016)

3.3 Topography and drainage conditions

The topography of Mojo town ranges from 1730 to 1890 m.a.s.l. in the northern part of the town. The southern part of the town is the area around the highway towards Shashemane. From this highway, the general characteristic of the terrain decreases in elevation towards Mojo River. On the other hand the South-eastern part of the town is ascending in elevation around Ethiopian Road Authority camp. In the western and the central part of the town the topography of the terrain is plain except in gorges of Mojo river, where there is descending in elevation. In the eastern direction, the topography is plain but the peripheral area of the land is ascending in elevation from Malk-Lami towards Tedde high lands. Because of the topographic configuration of the town and its environment, the alignment of flow of the Mojo River is from north western to the south eastern direction of the town. Mojo town is located in Awash drainage basin (Mastewal G.,2016).

3.4 Climate

3.4.1 Rainfall

The records of National Meteorological Service Agency from Adama Observatory Substation show that the Mean Annual rainfall for 29 years i.e. from 1982 to 2011 is shown in Figure 14.

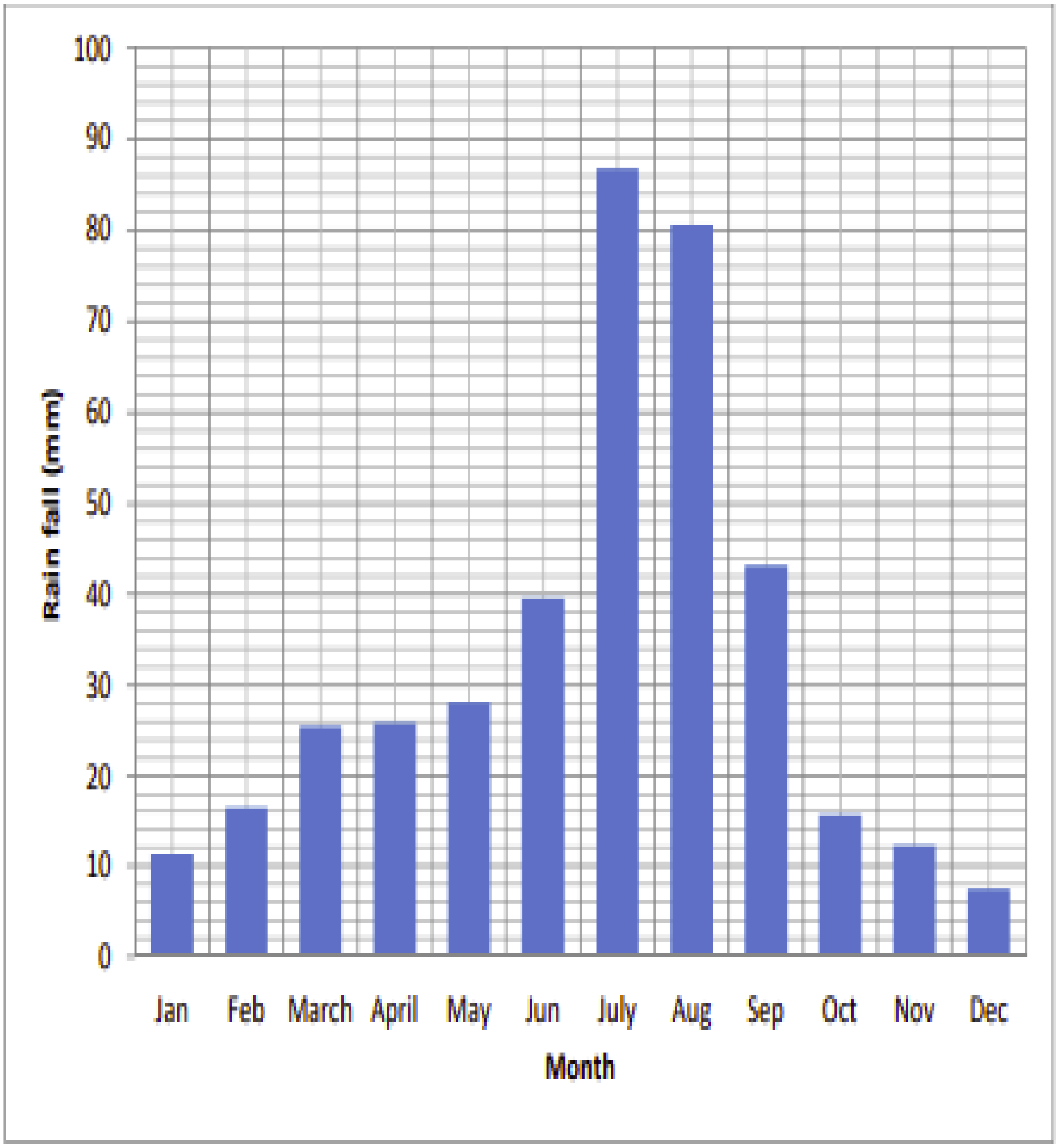


Figure 14: the Mean Annual rainfall for 29 years (Mastewal G., 2016).

3.4.2 Temperature

In a mountainous tropical country like Ethiopia, altitude is by far the most important factor controlling climate. It affects distribution of both temperature and rain fall. Generally, regions between 1500-2300 m.a.s.l. (categorized as 'woinadega' or subtropical climates) have a temperature that ranges between 15 – 20^{o}C, areas between 500 - 1500 m.a.s.l (i.e. 'kola' or tropical climate) have 20 – 30^{o}C area below 500 m.a.s.l. ('bereha' or desert climate) have a temperature of 30oC and above. The town of Mojo, with altitude ranging from 1730 – 1890 m.a.s.l., has a mean minimum, mean maximum and mean average monthly temperatures of 11.8, 28.6 and 20.2 ^{o}C respectively. The highest temperature is during months of March, April, May and June where October, December and January have low temperature. From Figure 3.4 the mean monthly average temperature ranges from 18.3^{o}C to 22.3^{o}C. This shows the temperature variation is almost the same throughout the year.

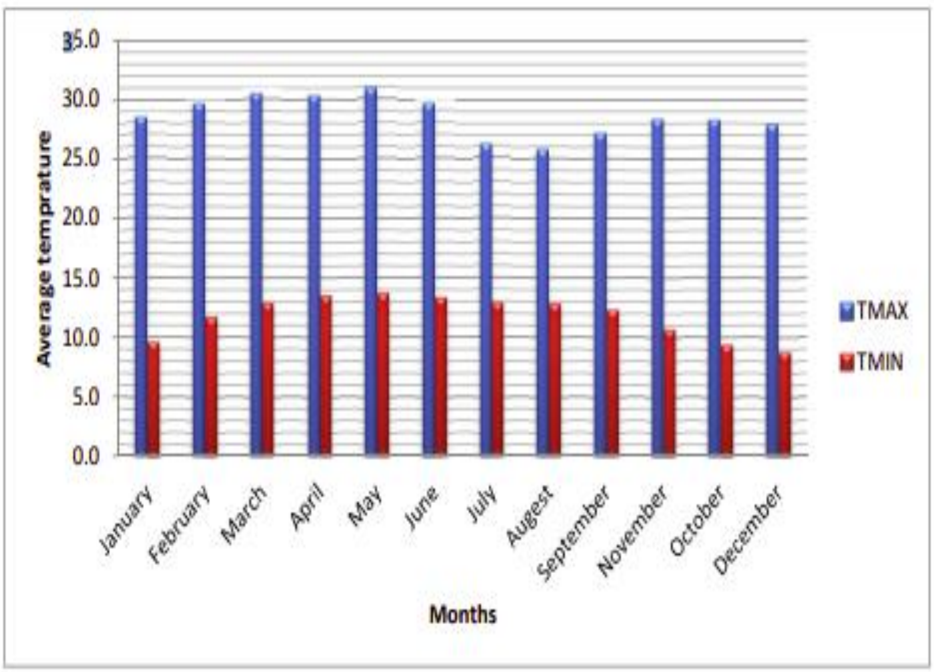


Figure 15: Average Monthly Maximum and Minimum temperature distribution of Mojo Town (Mastewal G.,2016).

3.5 Methods of Soil sampling and sample size

The soil profile of Mojo town varies from place to place within the town. Generally, the soil profiles could be treating into two major groups, namely expansive and non-expansive soils. For the laboratory works of this thesis disturbed and undisturbed Soil sample were collected from different sites in mojo town, which are covered by expansive soil. Most of the disturbed samples were collected around undergoing construction projects during the excavation stage usually taken from the bucket of the excavator, or from the side of a shallow pit. Undisturbed samples may be obtained by hand cutting blocks of soil in shallow pits, but it is more common to use sample tubes to obtain standard undisturbed samples. After visual identification and information gathered from the community expansive soils of ten test pits were excavated to the maximum depth of three meters. Disturbed and undisturbed samples were collected for this work and taken to laboratory for testing .for a total of fifteen laboratory test samples from ten test pits. Especially samples from places where structural damage and road failures occurred are given special attention. Each collected disturbed and undisturbed soil samples are tested based on ASTM standard. Samples collected from site would be transported to the laboratory to conduct tests.

3.6Procedurs

While carrying out this study, the following procedures were followed

- Literature Review: different types of literatures; such as text books, academic journals, seminars and research papers pertaining to consolidation and swelling characteristics of expansive soil were reviewed.
- Materials and Methods: material sampling and testing methods were undertaken as per ASTM, to investigate the relationship between swelling & consolidation behavior of expansive soils of Mojo Town using one-dimensional consolidation theory of Terzaghi. To attain the purpose of this research, representative disturbed & undisturbed samples of expansive soil were collected from different locations of Mojo town & a series of swell-consolidation tests would be carried out in the laboratory.
- Analysis and Discussion of test results: based on the theories and laboratory tests were performed, the results obtained were analyzed and discussed thoroughly.
- Formulation of conclusions and Recommendations were based on the results obtained.
- Finally compiled, wrote and documented of the thesis work.

CHAPTER FOUR

4.1 Laboratory Result and Discussion

4.1.1 General

For the successful accomplishment of the study, the following tests were done in the laboratory. these tests are carried out in accordance with the ASTM standard testing methods. The detail test results are presented in the Appendices. Total of fifteen soil samples were gathered from mojo town where expansive soil are dominant. Based on the samples retrieved from the sites, laboratory tests on the fifteen samples were conducted in the geotechnical laboratory of Adama Science and Technology University and Oromia Laboratory Services Engineering Company.

Accordingly, the following different kinds of tests have been performed

- Free swell test
- Specific Gravity of soil(ASTM D854)
- Atterberg limits (ASTM D4318)
    - Liquid Limit
    - Plastic Limit
    - Plasticity Index
- Grain Size Analysis test (ASTMD422)
    - Sieve Analysis
    - Hydrometer Analysis
- Standard Proctor Compaction test (ASTM D698)
- Swell-Consolidation Test (ASTM D2435)
    - Swell –Consolidation test on undisturbed and disturbed samples by remolding at different initial moisture content & dry density.

4.1.2 Free swell test

These tests were carried out in accordance to Gibbs and Holtz, 1969, test Procedure.The procedure consists of pouring very slowly 10 cubic centimeters of that part of the dry soil passing No.40 sieve into a 100 cubic centimeters graduated measuring cylinder and letting the content stand for approximately twenty-four hours until all the soil completely settles on the bottom of the graduating cylinder. Then the final volume of the soil is noted.

$$\text{Free swell} = \frac{\text{Final Volume} - \text{Initial Volume}}{\text{Initial Volume}} * 100\%$$

In this study the laboratory test results show that, free swell tests of the study area falls in the range of (101-143) %.

Table 6: Classification of the soil based on test results obtained from Free swell test

Sample ID	**Location**	**Depth (m)**	**Sample Type**	**Sample No**	**Color**	**Free Swell (%)**
Pit-1	Dibora	1.5	Disturbed	1	Black	103
Pit-1	Dibora	3	Disturbed	2	Black	109
Pit-2	Dry Port	1.5	Disturbed	3	Black	101
Pit-3	Dry Port	1.5	Disturbed	4	Black	104
Pit-4	Seyyo	1.5	Disturbed	5	Black	116
Pit-4	Seyyo	3	Disturbed	6	Black	133
Pit-5	Seyyo	1.5	Disturbed	7	Black	123
Pit-5	Seyyo	3	Disturbed	8	Black	135
Pit-6	Dry Port	1.5	Disturbed	9	Black	109
Pit-7	Dibora	1.5	Disturbed	10	Black	115
Pit-7	Dibora	3	Disturbed	11	Black	118
Pit-8	Seyyo	1.5	Disturbed	12	Black	139
Pit-8	Seyyo	3	Disturbed	13	Black	143
Pit-9	Seyyo	1.5	Disturbed	14	Black	132
Pit-10	Dibora	3	Disturbed	15	Black	120

4.1.3 Specific Gravity of soil solid

The specific gravity of soil solids, Gs, is the mass density of the mineral solids in soil normalized relative to the mass density of water. The laboratory test results show that, Specific Gravity of the study area falls in the range of (2.62-2.70).

Table 7: Specific Gravity test Results of samples under investigation

Sample ID	**Location**	**Depth(m)**	**Sample Type**	**Sample No**	**Color**	**Specific Gravity**
Pit-1	Dibora	1.5	Disturbed	1	Black	2.67
Pit-1	Dibora	3	Disturbed	2	Black	2.68
Pit-2	Dry Port	1.5	Disturbed	3	Black	2.62
Pit-3	Dry Port	1.5	Disturbed	4	Black	2.68
Pit-4	Seyyo	1.5	Disturbed	5	Black	2.67
Pit-4	Seyyo	3	Disturbed	6	Black	2.68
Pit-5	Seyyo	1.5	Disturbed	7	Black	2.69
Pit-5	Seyyo	3	Disturbed	8	Black	2.70
Pit-6	Dry Port	1.5	Disturbed	9	Black	2.63
Pit-7	Dibora	1.5	Disturbed	10	Black	2.67
Pit-7	Dibora	3	Disturbed	11	Black	2.67
Pit-8	Seyyo	1.5	Disturbed	12	Black	2.68
Pit-8	Seyyo	3	Disturbed	13	Black	2.63
Pit-9	Seyyo	1.5	Disturbed	14	Black	2.69
Pit-10	Dibora	3	Disturbed	15	Black	2.69

4.1.4 Atterberg limits

The Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils The liquid and plastic limit are water contents at which the mechanical properties of soil change. They apply to fine-grained soils and are performed on soil fractions that pass through 0.425μm sieve. The difference between liquid limit (LL) and plastic limit (PL) is defined as the plasticity index (PI).

$$PI = LL - PL$$

Figure 16: Atterberg test

The detail summary of laboratory test results of the Atterberg Limits and Indices of the study area falls in the ranges of:-Liquid Limit (75.50-119.59) %; Plastic Limit (28.92-49.09) %; and Plastic Index (35.42-83.03) %.

Table 8: Test Results of Atterberg Limit test & general classification of the soil of Mojo town.

Sample No	Location	Sample ID	Depth (m)	% Pass No 200 sieve	Atterberg			Soil Classification	
					LL	PL	PI	AASHTO	USCS
1	Dibora	Tp-1	1.50	91.60	82.75	37.85	44.9	A-7-5	MH
2	Dibora	Tp-1	3.0	95.60	82.12	37.87	44.25	A-7-5	MH
3	Dry Port	Tp-2	1.50	91.90	79.99	37.2	42.79	A-7-5	MH
4	Dry Port	Tp-3	1.50	93.50	85.25	49.09	36.16	A-7-5	MH
5	Seyyo	Tp-4	1.50	94.10	77.01	37.72	39.29	A-7-5	MH
6	Seyyo	Tp-4	3.0	93.00	75.50	40.08	35.42	A-7-5	MH
7	Seyyo	Tp-5	1.50	93.80	105.3	38.77	66.53	A-7-5	CH
8	Seyyo	Tp-5	3.0	94.40	82.12	38.6	43.52	A-7-5	MH
9	Dry Port	Tp-6	1.50	94.00	87.17	40.34	46.83	A-7-5	MH
10	Dibora	Tp-7	1.50	93.90	*119.59*	*39.51*	80.08	A-7-5	CH
11	Dibora	Tp-7	3.0	91.90	96.57	35.72	60.85	A-7-5	CH
12	Seyyo	Tp-8	1.50	92.60	115.72	35.72	80.00	A-7-5	CH
13	Seyyo	Tp-9	3.0	90.80	117.23	39.74	77.49	A-7-5	CH
14	Seyyo	Tp-9	1.50	92.20	111.95	28.92	83.03	A-7-6	CH
15	Dibora	Tp-10	3.0	93.20	106.34	30.77	75.57	A-7-5	CH

4.1.5 Soil Classification

The soil under study is plotted on the plasticity chart are classified based on the two most widely used engineering soil classifications:-

- AASHTO
- USCS

American Association of State Highway and Transportation Officials (AASHTO)

The expansive soil samples which were extracted from Mojo Town for this study have fallen under group A-7-5and A-7-6 as per classification AASHTO system.

Unified Soil Classification System (USCS)

The classification of expansive soils found in Mojo town has fallen under MH and CH group according to USCS. Seven of the samples are classified under CH group and the rest of them are in the MH group which is inorganic clay with high compressibility.

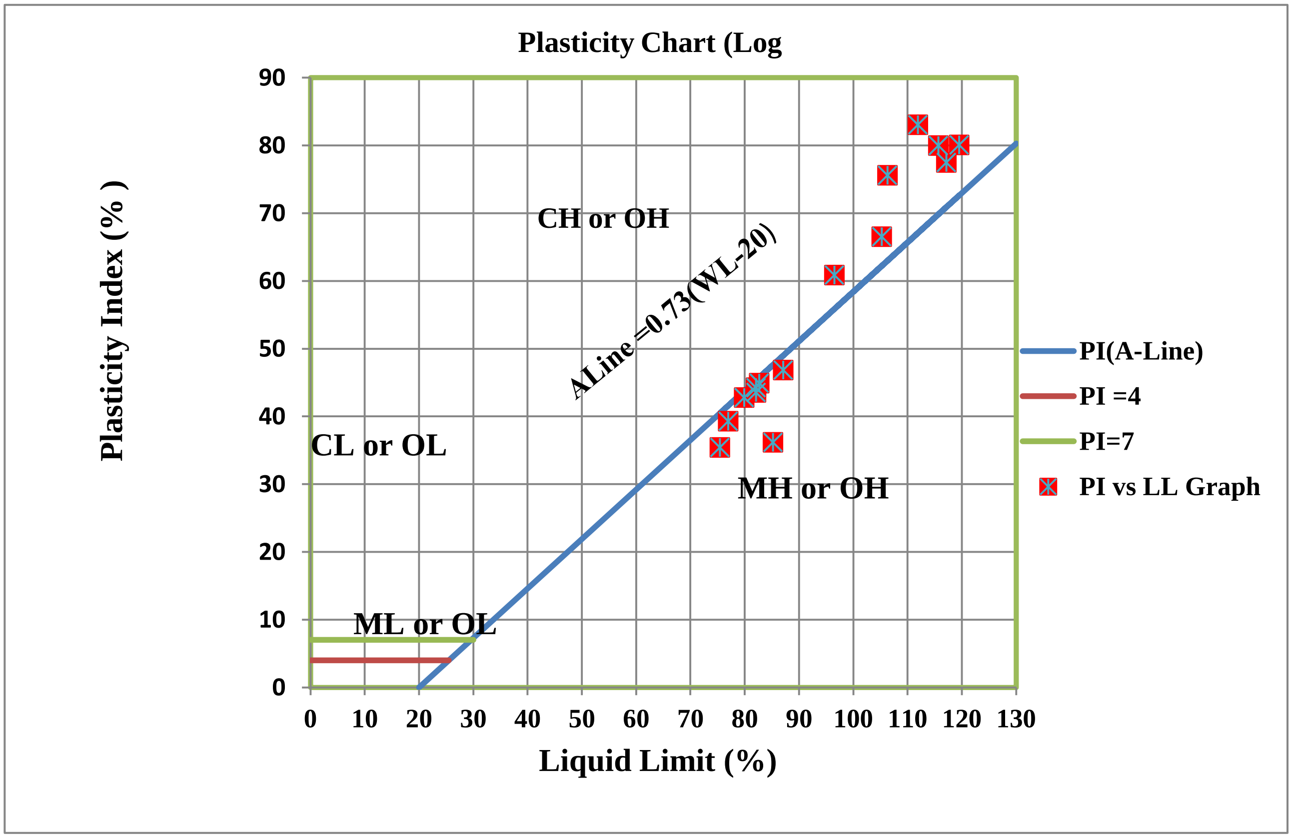


Figure 17: Plasticity chart of soil in the study area according to Unified Soil Classification System- USCS.

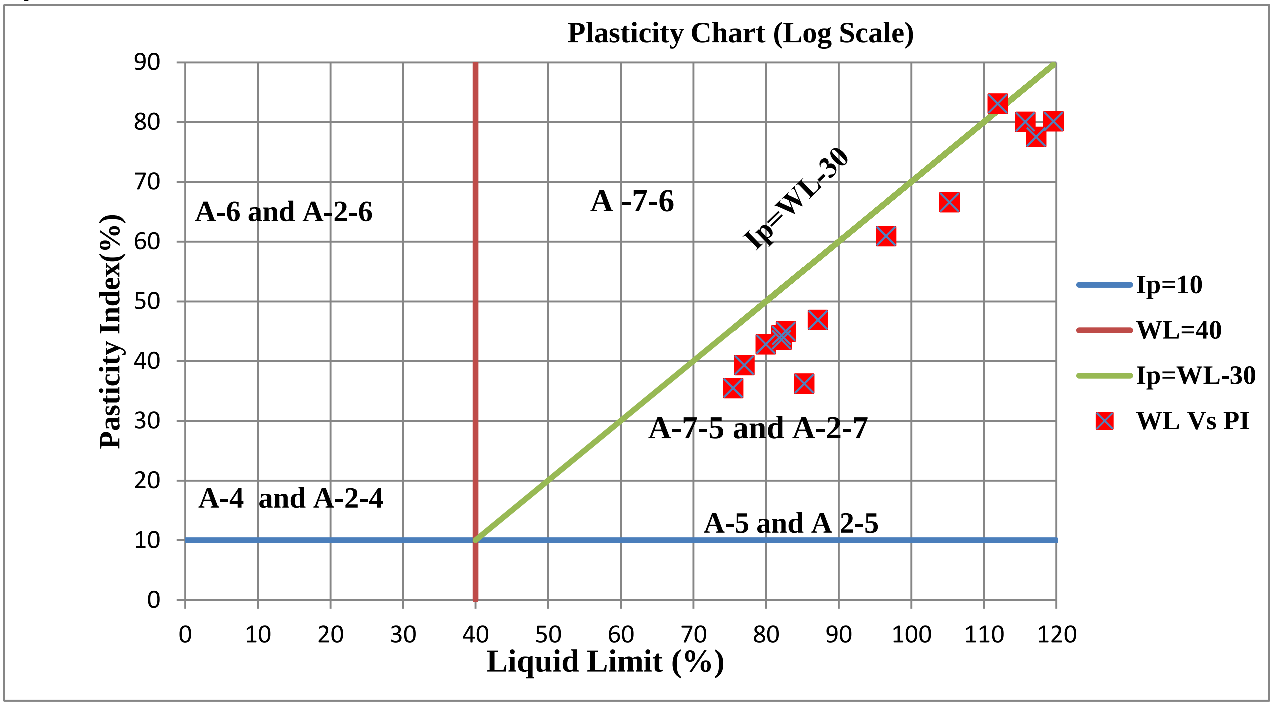


Figure 18: Plasticity chart of soil in the study area according to AASHTO system of classification

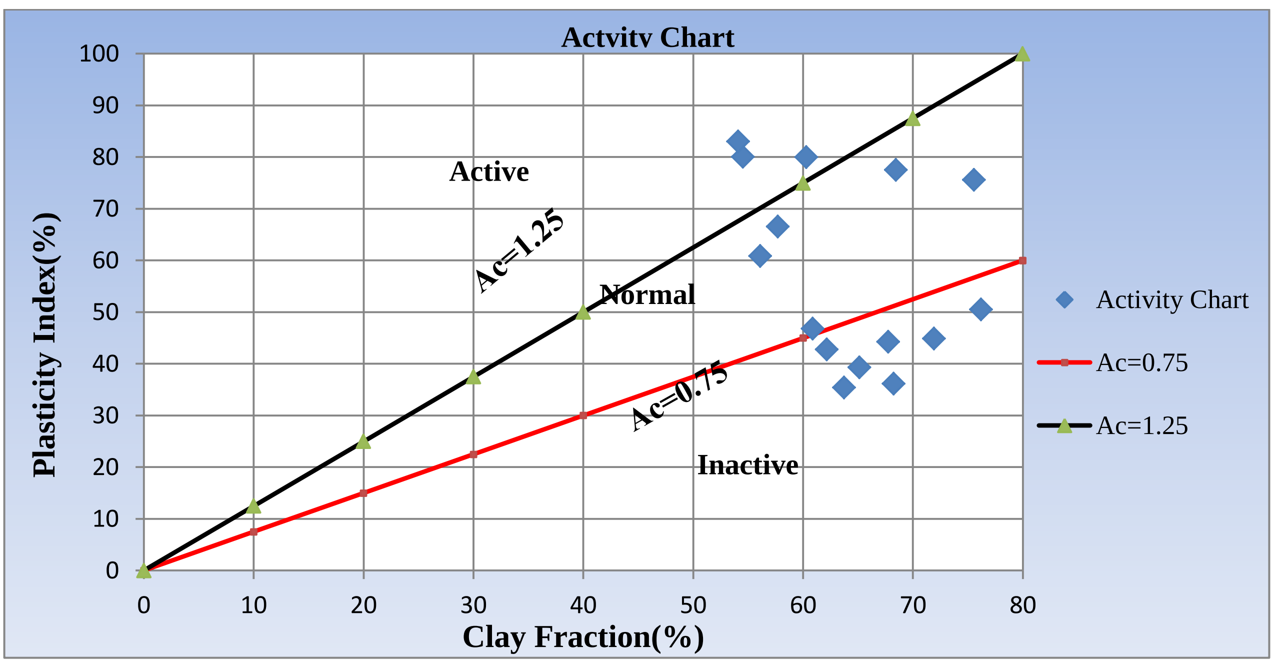


Figure 19: Activity charts of the soil under investigation

4.1.6 Grain size analysis (Wet Sieve and Hydrometer Analysis Tests)

4.1.6.1. Wet Sieve analysis

In wet sieve analysis, the oven-dried soil sample is soaked with water for 24hrs. Then, washed through 75μm sieve until the wash water becomes clear. The material retained on 75μm was dried in the oven and sieved through the set sieves. Then the material on each sieve was collected and weighed. The purpose of these tests is to determine the percentage of fines in soil

4.1.6.2 Hydrometer analysis

The test was conducted as per ASTM D422: To perform a hydrometer analysis, the soil is mixed with water and sodium hexameta phosphate (a dispersing agent) to create slurry of dispersed soil particles. Information regarding how low the hydrometer floats in the slurry was recorded as a function of time, and this information is used to calculate points of grain size versus percent passing for the gradation curve.

Figure 20: Recorded hydrometer floats in slurry with the function of time

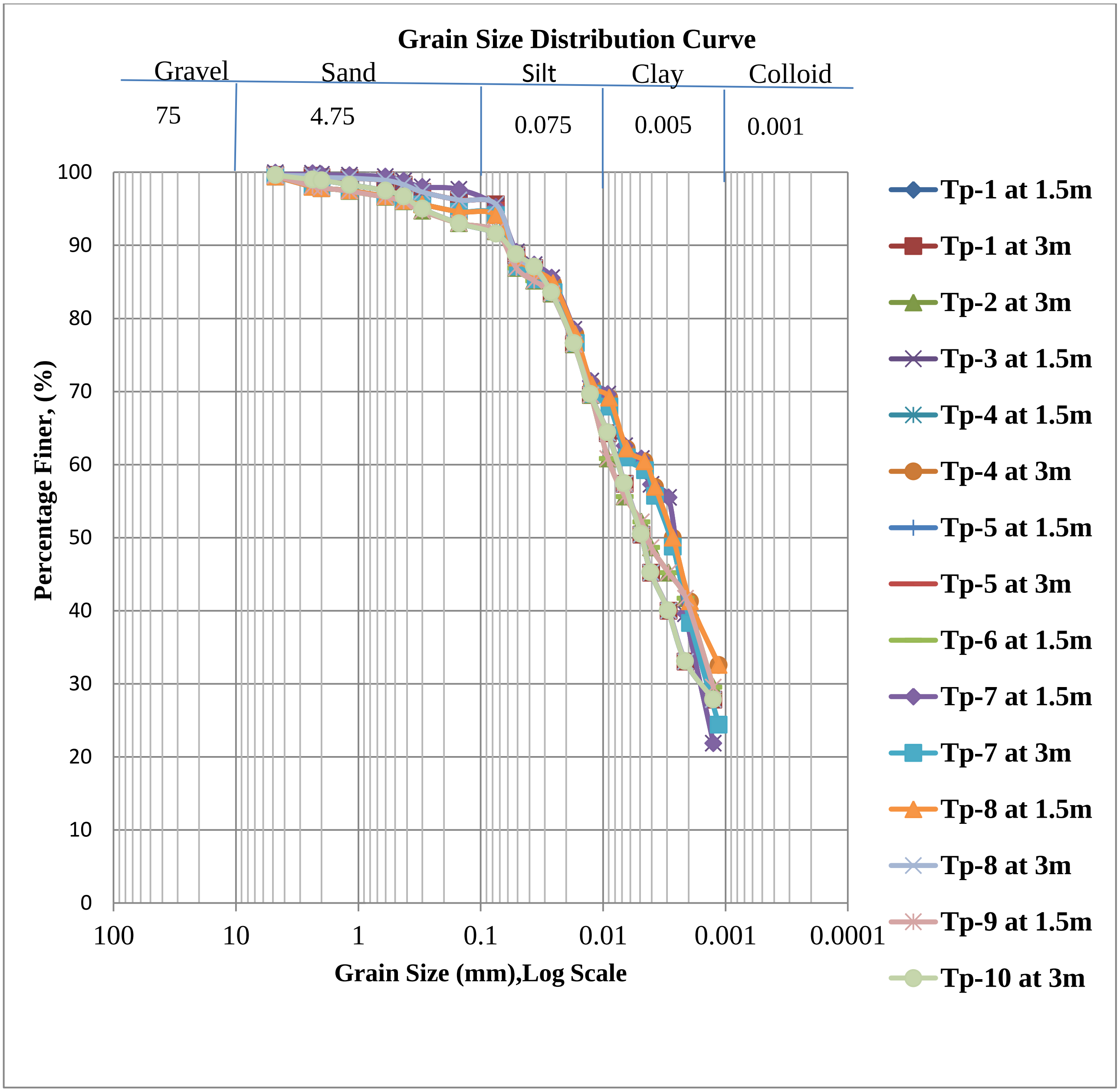


Figure 21:The combined grain size distribution curve for mechanical sieve & hydrometer tests

4.1.7 Standard Proctor Compaction test

The standard proctor compaction test to determine the optimum moisture content and maximum dry density of each sample from the compaction curve. Every sample was compacted in 3 layers by a hammer that delivers 25 blows to each.

Figure 22: Standard compaction test

Table 9: Summary of optimum moisture content and maximum dry density

Sample ID	Location	Depth (m)	Sample Type	Sample No	Color	OMC (%)	MDD (g/cm3)
Pit-1	Dibora	1.5	Disturbed	1	Black	34	1.40
Pit-1	Dibora	3	Disturbed	2	Black	38	1.27
Pit-2	Dry Port	1.5	Disturbed	3	Black	42	1.30
Pit-3	Dry Port	1.5	Disturbed	5	Black	40	1.28
Pit-4	Seyyo	1.5	Disturbed	6	Black	37	1.33
Pit-4	Seyyo	3	Disturbed	7	Black	36	1.32
Pit-5	Seyyo	1.5	Disturbed	8	Black	31	1.42
Pit-5	Seyyo	3	Disturbed	9	Black	34	1.47
Pit-6	Dry Port	1.5	Disturbed	10	Black	42	1.32
Pit-7	Dibora	1.5	Disturbed	11	Black	37	1.23
Pit-7	Dibora	3	Disturbed	12	Black	34	1.32
Pit-8	Seyyo	1.5	Disturbed	13	Black	36	1.25
Pit-8	Seyyo	3	Disturbed	14	Black	33	1.46
Pit-9	Seyyo	1.5	Disturbed	15	Black	32	1.33
Pit-10	Dibora	3	Disturbed	16	Black	38	1.24

4.2.8 Swell-Consolidation Test

4.2.8.1 General

The standard test method for one-dimensional consolidation properties of soil using incremental loading on the sample prepared for consolidation test. The load was maintained for 24 hours during which the soil consolidates with drainage from the porous stones. Afterward, the applied load was increased incrementally by doubling the applied stress at each stage. The numbers of the load stages were 8, 16, 32, 64, and 128 kg. During the loading process, water was provided into the cell so that the specimen remains fully saturated. At each loading stage, the deformation was measured at 6, 15, and 30 seconds, then at 1, 2, 4, 8, 16, 30 min, and 1, 2, 4, 8, and 24 hours, respectively. When the maximum load was reached, unloading stages were conducted by reducing the load. Typically, the load was reduced by a factor of 4 at each step. After the test, the final height of the sample and its water content were measured. Finally, compressibility characteristics such as compression index, Cc, coefficient of volume compressibility, the coefficient of consolidation, Cv, and void ratio were determined from data of several one dimensional consolidation tests using oedometer apparatus for varying loading increments. The collected data were plotted as graphical construction, from which the consolidation and Swelling characteristics were determined.

Figure 23: Consolidation test machine

Sample preparation

In order to have sufficient and reliable data for the target analysis, the soil sample was collected from mojo town. Prior to sampling, visual site investigations and information from residents and construction firms were collected to consider the different soil types and to take representative samples evenly in the whole town. Accordingly, 15 soil samples obtained from 10 test pits in different parts of Mojo town where expansive soil is dominant. Pits were excavated to the maximum depth of three meters. Disturbed and undisturbed samples were prepared for One-Dimension Swell- Consolidation tests.

a) Undisturbed samples

Undisturbed samples of expansive soil collected from Mojo Town and the specimens for odometer test were extracted from the tube sampler using the odometer ring which has a diameter of 75mm and a height of 20mm.The samples were then prepared for the test by trimming the ends.

b) Disturbed samples

Since the in-situ Disturbed soil samples was difficult to find samples of reasonably varying moisture content and dry density of in-situ soil, it was quite necessary to prepare the soil specimen for swell- consolidation tests from disturbed samples by remolding. For this purpose disturbed samples of expansive soils were collected from different locations of Mojo Town and standard compaction tests were conducted in the laboratory to determine the compaction curve by plotting the moisture content versus dry density. From the compaction curves different soil specimens were prepared for swell-consolidation tests by varying the moisture content and dry density. Using the compaction curve was helpful to increase the size of samples to be tested & also to visualize the trends of series of swell consolidation test results from the same compaction curve. Plots of the compaction curves from which the soil specimens were prepared for swell and consolidation tests are presented in Figure 30 below.

4.2.8.2Test Results

The swell-consolidation test was carried out on six undisturbed and ten remolded soil samples of expansive soils of Mojo Town. The test results obtained from swell-consolidation test on undisturbed & remolded expansive soil samples with different initial moisture content and dry density are presented in the form of void ratio versus log pressure, and dial reading versus square root of time plots. The test results presented in the main body of the paper are only the typical ones which can better explain the argument. The test results of all the other samples are in the appendices.

Table 10: The initial condition of the soil under investigation is presented.

Sample ID	**Location**	**Depth(m)**	**Sample Type**	**Sample No**	**Initial Moisture content (%)**	**Initial dry density**	**Initial Saturation S (%)**
Pit-1	Dibora	1.5	Undisturbed	1	47.56	1.075	85.61
Pit-1	Dibora	3	Undisturbed	2	48.38	1.071	86.28
Pit-2	Dry Port	1.5	Undisturbed	3	47.81	1.076	87.37
Pit-3	Dry Port	1.5	Undisturbed	4	48.12	1.065	85.05
Pit-4	Seyyo	1.5	Undisturbed	5	48.31	1.079	87.53
Pit-4	Seyyo	3	Remolded	6	48.21	1.082	87.44
Pit-5	Seyyo	1.5	Remolded	7	47.81	1.091	87.71
Pit-5	Seyyo	3	Remolded	8	47.61	1.095	87.74
Pit-6	Dry Port	1.5	Remolded	9	47.47	1.101	89.89
Pit-7	Dibora	1.5	Remolded	10	46.41	1.110	88.88
Pit-7	Dibora	3	Remolded	11	46.48	1.116	89.07
Pit-8	Seyyo	1.5	Remolded	12	45.70	1.126	88.70
Pit-8	Seyyo	3	Remolded	13	49.66	1.057	87.78
Pit-9	Seyyo	1.5	Remolded	14	45.99	1.088	84.03
Pit-10	Dibora	3	Remolded	15	45.46	1.100	84.57

The following consolidation curves for both undisturbed and disturbed soil samples are shown below.

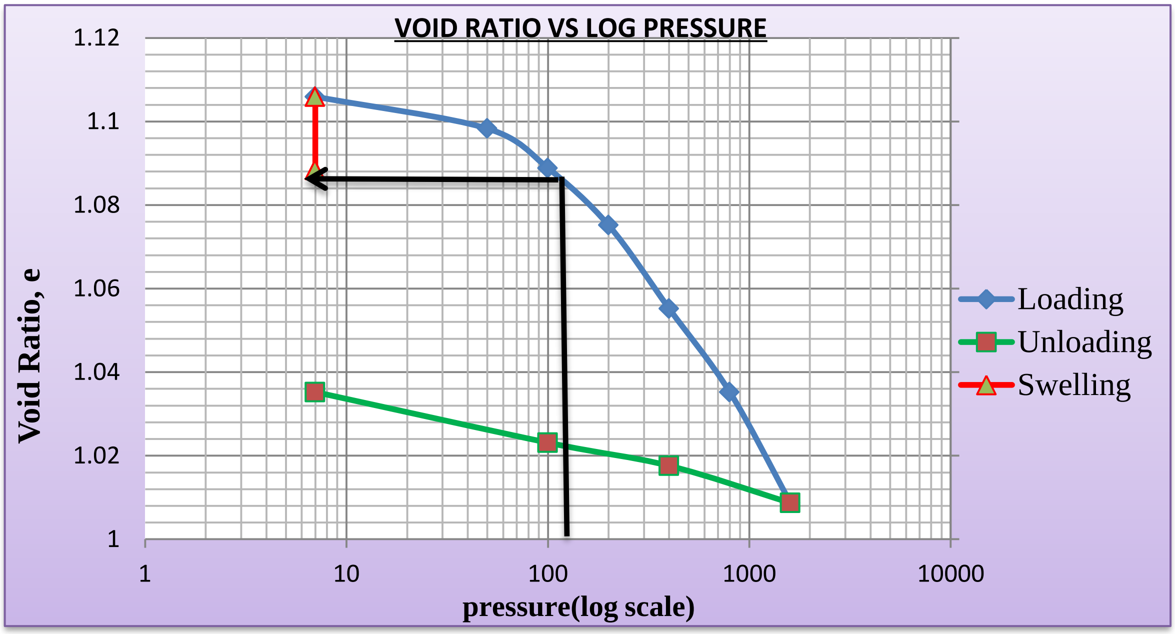


Figure 24 (a): Typical plot of e- log P of undisturbed sample at initial dry density ρd =1.075& initial moisture content of w=47.56(sample-1)

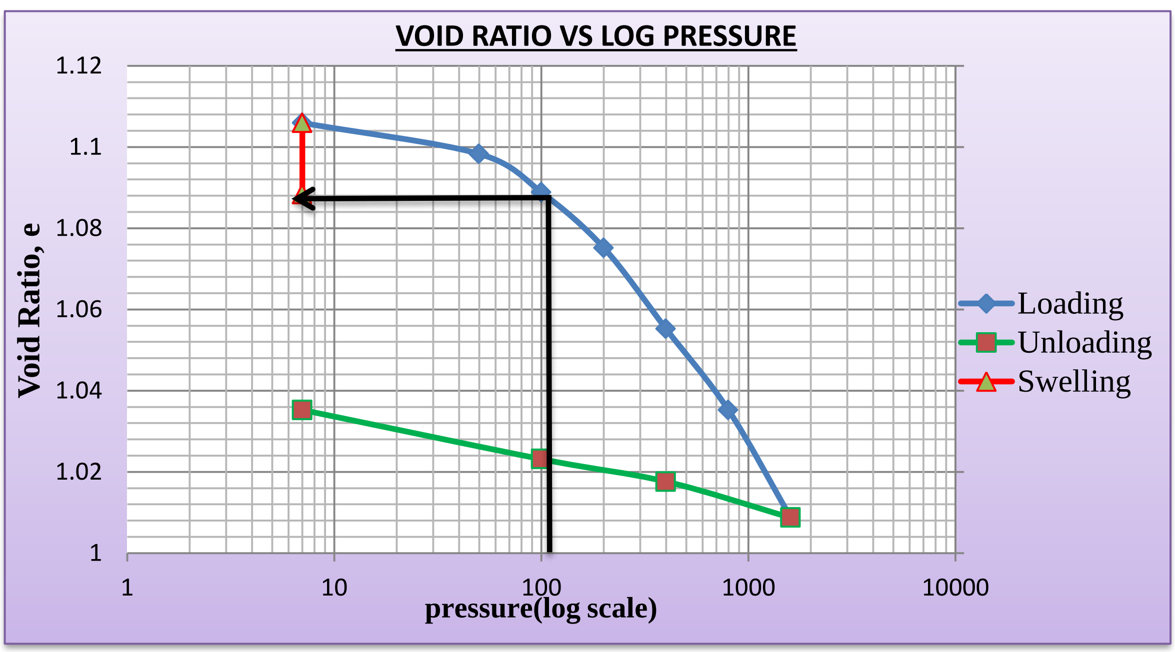


Figure 25 (a): Typical plot of e- log P of undisturbed sample at initial dry density ρd =1.071& initial moisture content of w=48.38(sample-2)

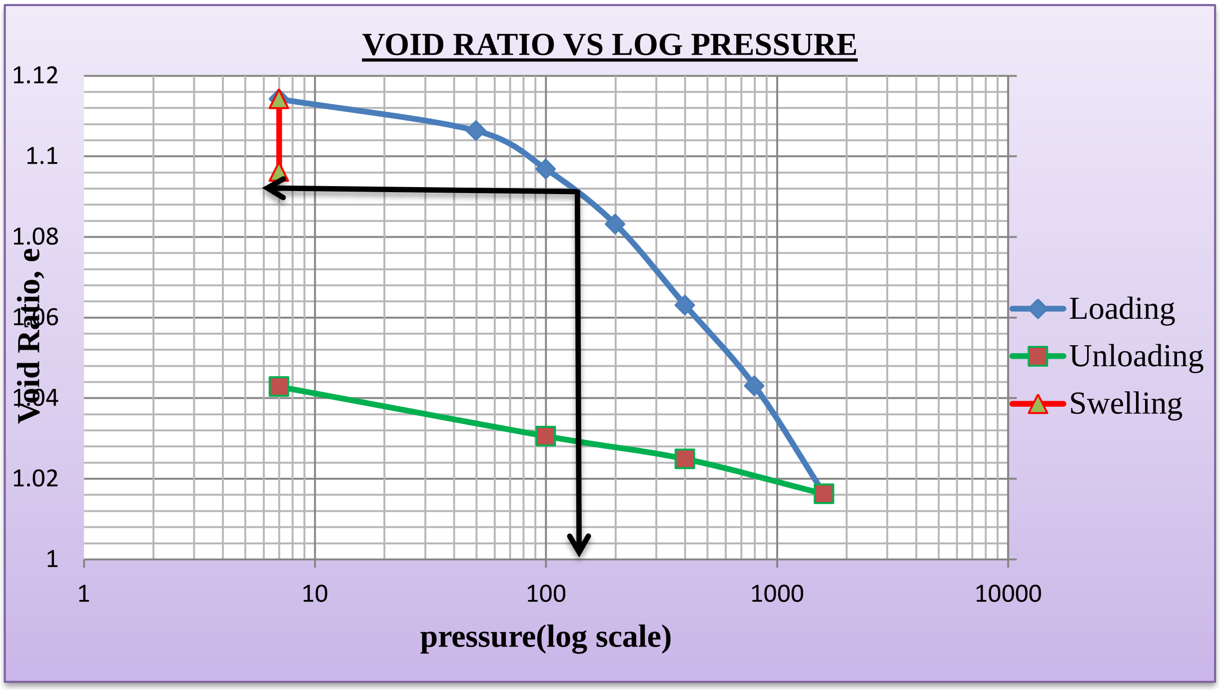


Figure 26 (a): Typical plot of e- log P of Remolded sample at initial dry density ρd =1.076& initial moisture content of w=47.81(sample-3)

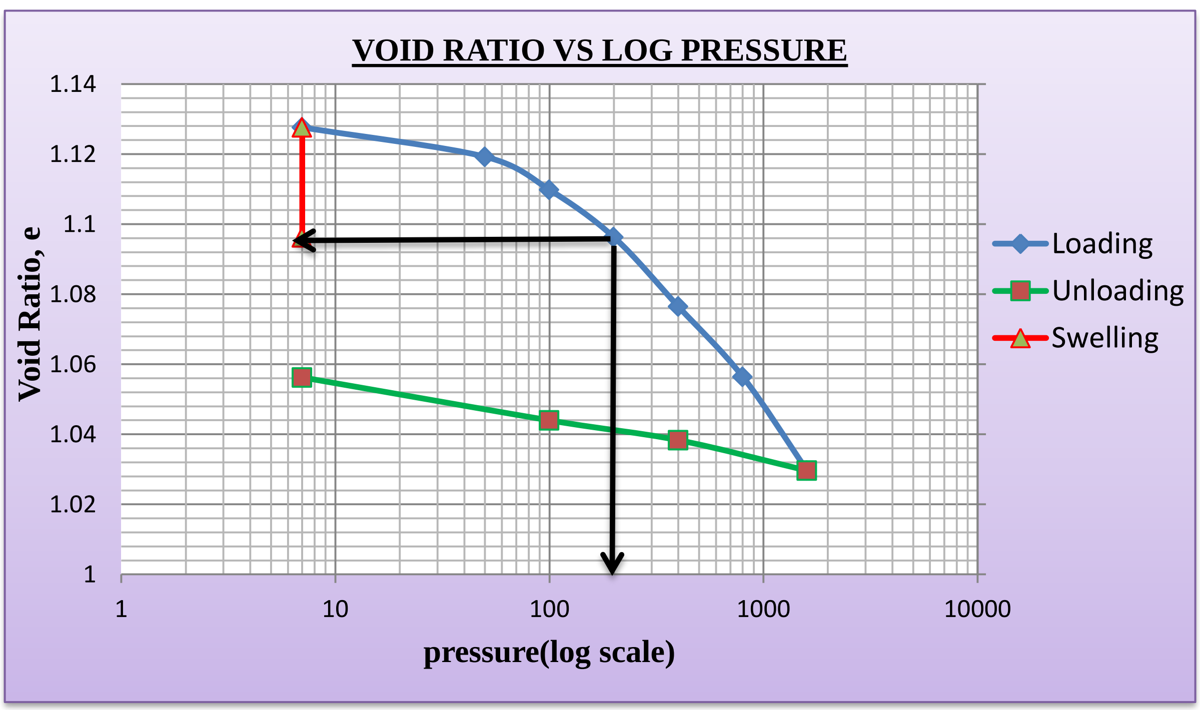


Figure 27 (a): Typical plot of e- log P of Remolded sample at initial dry density ρd =1.065& initial moisture content of w=48.12(sample-4)

Table: 11 Results of laboratory tests conducted on swell – consolidation characteristics of expansive soil samples of mojo town

Sample ID	**Location**	**Sample No**	**Depth(m)**	**Sample Type**	Initial Moisture content (%)	Initial dry density	Swelling Potential %	Swell Pressure kPa	Consolidation Index Cc	Consolidation Coef. Cv m2 /Year
Pit-1	Dibora	1	1.5	Undisturbed	47.56	1.075	7.20	140	0.420	0.32
Pit-1	Dibora	2	3	Undisturbed	48.38	1.071	4.65	100	0.410	0.21
Pit-2	Dry Port	3	1.5	Undisturbed	47.81	1.076	5.36	150	0.405	0.37
Pit-3	Dry Port	4	1.5	Undisturbed	48.12	1.065	5.40	200	0.421	0.57
Pit-4	Seyyo	5	3	Undisturbed	48.31	1.079	9.56	280	0.411	0.42
Pit-4	Seyyo	6	1.5	Remolded	48.21	1.082	14.48	300	0.412	0.49
Pit-5	Seyyo	7	3	Remolded	47.81	1.091	6.21	290	0.409	0.50
Pit-5	Seyyo	8	1.5	Remolded	47.61	1.095	6.32	370	0.409	0.32
Pit-6	Dry Port	9	1.5	Remolded	47.47	1.101	14.56	400	0.401	0.12
Pit-7	Dibora	10	1.5	Remolded	46.41	1.110	9.13	380	0.397	0.37
Pit-7	Dibora	11	3	Remolded	46.48	1.116	7.31	400	0.401	0.25
Pit-8	Seyyo	12	1.5	Remolded	45.70	1.126	8.04	420	0.397	0.28
Pit-8	Seyyo	13	3	Remolded	49.66	1.057	7.68	410	0.416	0.52
Pit-9	Seyyo	14	1.5	Remolded	45.99	1.088	7.93	380	0.411	0.46
Pit-10	Dibora	15	3	Remolded	45.46	1.100	8.19	350	0.414	0.30

4.2 8.4 Discussion on the test results

The main objective of this thesis is to identify the relationship between consolidation and swelling characteristics of expansive soils of mojo town using one dimensional consolidation test method.

The laboratory test results for the Swell-Consolidation tests are tabulated and some of them are shown graphically in the main body of the paper. In addition to swell-consolidation test the result of index property tests on sample collected from different part of the mojo town are shown in Table.

The swell characteristics of the soil are influenced by the variation in the values of initial moisture content, dry density, coefficient of consolidation and compression and this implies that the swell potential, swell pressure and rate of settlement are affected.

These consolidation- Swell characteristics were obtained from data of several one dimensional consolidation tests using oedometer apparatus for varying loading increments. The collected data were plotted as graphical construction, from which the consolidation characteristics were determined. The one-dimensional consolidation test was conducted for disturbed and undisturbed soil sample with different dry density and moisture Content. The coefficient of consolidation, compression index, and void ratio versus pressure, Swelling pressure and swelling potential were determined for each sample. The compression index of each sample was determined from void ratio-pressure curves (logarithm).At a given pressure level, the ratio of samples decrease after a given pressure increase.

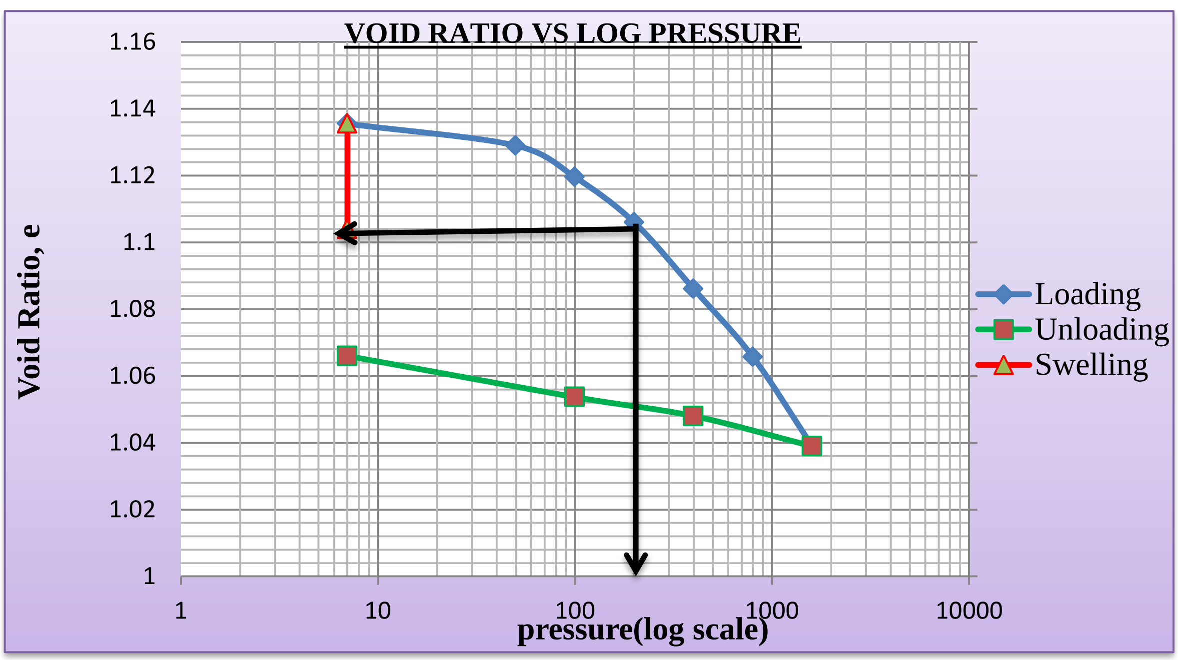


Figure: 28 Typical plot of e- log P of Remolded sample at initial dry density ρd =1.079& initial moisture content of w=48.31(sample-5)

In other words, the compression index decreased when soil is consolidated by increasing pressure. e-log p curve in Figure: 28 above shown that the relationship between the void ratio and consolidation pressure. The compression index with the increase in pressure, the soil void ratio decreases gradually. When the consolidation pressure is high, the decreasing trend of the soil void ratio slows down.

The compression index (Cc) is one of the critical parameter used to characterize soil compressibility. Compression index of the test soil in various pressure ranges were calculated according to the e–log p curves. Table :11 shown that the compression index varies from 0.397 to 0.416.When the applied consolidation pressure was relatively small, both the void ratio and the compression index varied considerably increased.at the same time With the increase in pressure, the void ratio of the test soil decreased and the compression index also decreased.

The coefficient of consolidation (Cv) is a parameter reflecting the drainage rate of soil during consolidation. According to the relationship between soil compression and time that acquired in the test, the coefficient of consolidation of samples Soil under various loads can be obtained by using the time square root method as shown in Figure below. Table: 11 shown that the coefficient of consolidation of the test soil varies in the range of 0.12m2/year to 0.57 m2/year.

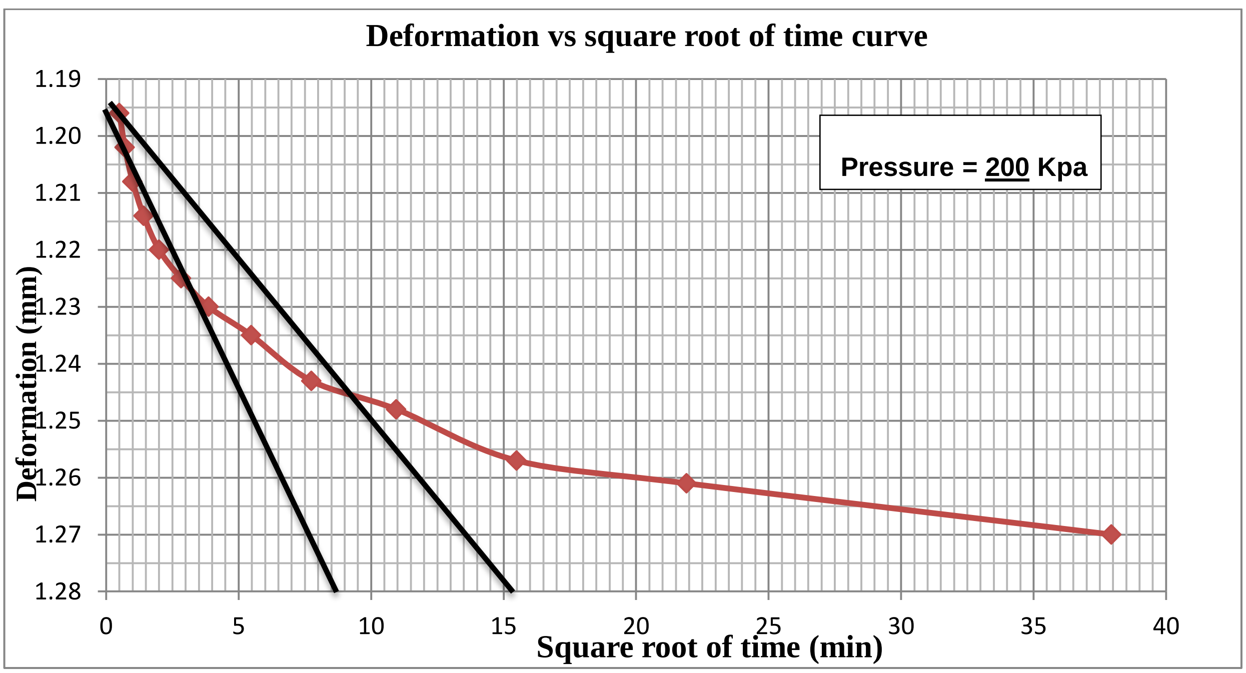


Figure: 29 Typical plots of Deformation versus Square root of time expansive soil of mojo Town, sample: 1 (P=200kPa), On the other hand, the standard compaction test method was used for preparing the soil samples at maximum dry density and Optimum moisture content. The soil samples were then fully saturated in a vacuum chamber. Afterward, the 1D consolidation test was conducted according to ASTM standard D2435. A standard compaction test was reflected which was important for remolded soil sample at OMC and MDD to get compaction curve. The compaction test results were used to remold the samples from which test specimens have been prepared for the swell-consolidation tests.

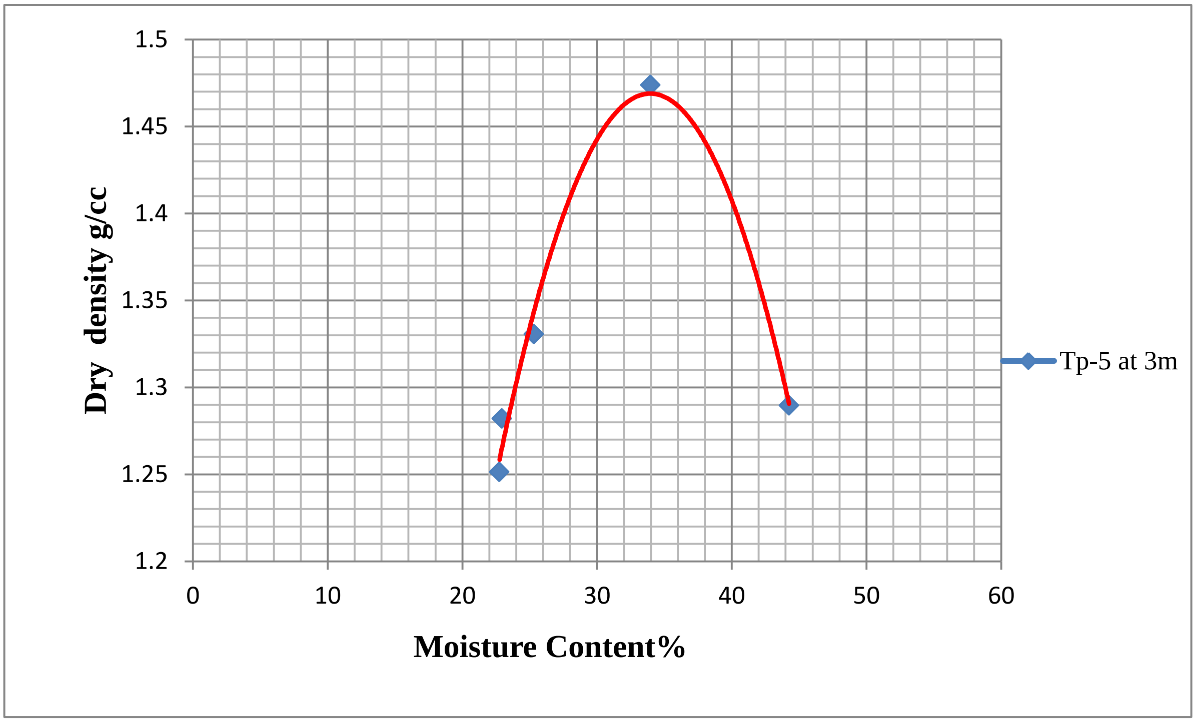


Figure 30 Standard Compaction Curve Dry density Vs. Moisture Tp-5 at 3m

The Compaction test result of expansive soil indicates that the increased the dry density of the soil result showed that the swelling potential and swelling pressure of soil is increased. Also the coefficient of consolidation increased more when the soil is densed. Additionally, the increasing coefficient of consolidation is due to the soil have less affinity with water. A higher coefficient of consolidation can reduce the duration of consolidation and leads to an improvement in the stability and bearing capacity of the expansive soil. Tests on remolded samples prepared to have the different initial dry density and different initial moisture content has shown the same consolidation characteristics, i.e. the same compression index Table 11.For instance, soil sample 7 and sample 8 had different initial dry density and different initial moisture content. The corresponding compression indexes obtained are 0.409 and 0.409, respectively. It can be easily observed that the samples subjected to more swelling show a larger value of compression index. The same observation can also be made on the other test results in appendix F. The effect of swelling on consolidation characteristics can also be explained by using soil samples of the same initial moisture content and different dry density. The lower swelling pressure is due to higher initial moisture content of the specimen prior to testing. The initial moisture content of the expansive soils controls the amount of swelling. Expansive soil with lower initial moisture content will have higher swelling potential. As can be observed from the test results, those factors that affect the swelling characteristics of expansive soils have also affected the consolidation characteristics of the expansive soil. As a result, high swelling pressure is obtained. Thus for expansive soil, maximum swelling pressure is obtain at OMC & MDD. This implies that, swelling pressure of expansive soil is highly influenced by its dry density. The variation of swelling pressure is presented in the appendix. The swelling characteristics of an expansive soil increases with increase in dry density for constant moisture content. This is because; higher density results in closer particle spacing which causes greater particle interaction. Since the swelling characteristics of expansive soil is influenced by the type & amount of the clay mineral constituted in the given volume, high density contains large amount of clay minerals so that when such soil is exposed to moisture it results in high volume change. The Swell-Consolidation tests results showed that as the initial dry density increases, both the percent swell as well as the swelling pressure increase. For expansive soil with high dry density, to reach its full saturation, high water content is required & causes high swelling potential which results in high degree of disturbance. This can be observed from Table 13 sample- 3 & 7 which have the same initial moisture content of 47.81 & different initial dry density of 1.076 & 1.091 g/cm3 respectively. Accordingly, the corresponding swelling potential and swelling pressure obtained from the test result are found to be 5.36%, 6.21% and 150kPa, 290kPa respectively. The swelling pressure was determined from the e-log pressure curve as the pressure corresponding to the initial void ratio. Further conventional loading beyond the swelling pressure has caused the soil to undergo consolidation from which the consolidation characteristic has been determined. As the test results indicated that the swelling potential and swelling pressure of expansive soils of Mojo Town for sample is found to range from 4.65 – 14.56 % and 100 – 420kpa respectively.

CONCLUSION AND RECOMMENDATION

CONCLUSION

In this study, the relationship between consolidation and swelling characteristics of expansive soils of mojo town under different initial moisture content & dry density using one dimensional consolidation test method was investigated. From laboratory results, the following conclusions are drawn.

- ❖ The expansive soil of Mojo Town, according to USCS & AASHTO is classified as Inorganic Clay of high plasticity, MH, CH & A-7-5, A-7-6 respectively.

- ❖ The value of swelling pressure at OMC & MDD is greater than the swelling pressure obtained from the standard compaction curve Maximum swelling pressure is obtained from the samples with optimum moisture content & maximum dry density. This implies that, swelling pressure is highly influenced by dry density

- ❖ A higher coefficient of consolidation can reduce the duration of consolidation and leads to an improvement in the stability and bearing capacity of the soil.

- ❖ The Compaction test result of expansive soil indicates that the increased the dry density of the soil result showed that the swelling potential and swelling pressure of soil is increased. Also the coefficient of consolidation increased more when the soil is densed additionally; the increasing coefficient of consolidation is due to the soil having less affinity with water.

- ❖ The swelling characteristics of an expansive soil increases with increase in dry density for constant moisture content. This is because; higher density results in closer particle spacing which causes greater particle interaction.

- ❖ The compression index (Cc) with the increase in pressure, the soil void ratio decreases gradually. When the consolidation pressure is high, the decreasing trend of the soil void ratio slows down.

- ❖ This thesis shown that the compression index C_C, ranges from 0.397-0.416 & the coefficient of consolidation Cv ranges from 0.12-0.57 m2/year, while initial dry density & initial moisture content range of 1.057-1.126 g/cm3 & 45.66-49.66% respectively and The swelling potential and swelling pressure obtained from the test result are found to be 4.65%, 14.56% and 100kPa, 420kPa respectively

- ❖ The Consolidation-swell characteristics of the soil are influenced by the variation in the values of Dry Density, Initial Moisture Content, coefficient of consolidation, compression and swell index .This implies that the swell potential, swelling pressure and rate of settlement are affected.

RECOMMENDATION

- Mojo is found in seismic active zone so that Dynamic property should be studied before designing any civil structure
- It is recommended to perform such a similar study in other parts of the country especially areas where expansive soils are dominant.
- It is recommended that the above values should be refined with larger size of data.

REFFERENCE

Lee D Jones. (2012). British Geological Survey Ian Jefferson, School of Civil Engineering, University of Birmingham.

Taslima Nasrin. (2020).Critical Appraisal of Settlement Vs. Rate of Settlement (Srs) Approach in Vertical Consolidation.

Aklilu A. (2015).Relationship between Swelling and Consolidation Characteristics of Expansive Soils of Galan Town.

Ayenew Z. (2004). Investigation into Shear Strength Characteristics of Expansive Soil of Ethiopia.

Wang, J. D. et al. (2016). Field Tests of Expansive Soil Embankment Slope Deformation under the Effect of the Rainfall Evaporation Cycle.

R.F Craig (2004). Soil Mechanics

Tirsit D. (2017).Empirical Model For Estimating Compression Index From Index Properties Of Fine Grained Soils (Case of Modjo Town).

A. Sridharan and K. Prakash (2000). Classification procedures for expansive soils.

Alemayehu T., & Mesfin L. (1999) 'Soil Mechanics' Faculty of Technology, Addis Ababa University.

Donald R, Lawrence D.et al (1977). An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils.

Asamnew G.(2016) Developing Correlation Between The Index Properties And Swelling Potential Of The Expansive Soils Found In Woliso town.

J. David Rogers, Robert Olshansky, et al, Damage to Foundations from Expansive Soils.

Hussein Elarabi (2010).Damage mechanism of expansive soils.

Girma Tessema (1984). Foundations On Expansive Soils.

Mesfin K. (2005).'Relationship between Consolidation and Swelling Characteristics of Expansive Soils of Addis Ababa', Ethiopia.

Masoumeh Mokhtari and Masoud Dehghani (2012). Swell-Shrink Behavior of Expansive Soils, Damage and Control.

Fasil A. (2003). Investigation In to Some of the Engineering Properties of Red Clay Soils In Bahir Dar.

Eli Cuelho and Michelle Akin (2017).Mitigation of Expansive Soils in South Dakota.

Dinkie Adimas . (2015).Effect of Sugarcane Bagasse Ash and Cement on Swelling Characteristics of Expansive Soil around Mojo.

Dr.K.R.Arora. (2004). Soil Mechanics and Foundation Engineering.

Jaafar A Mohammed . (2015).Consolidation of Soils Testing and Evaluation.

Mastewal (G. 2016)investigation Into Some Of The Engineering Properties Of Soils Found In. Mojo Town.

APPENDICES

Appendix :A Atterberg limit test results

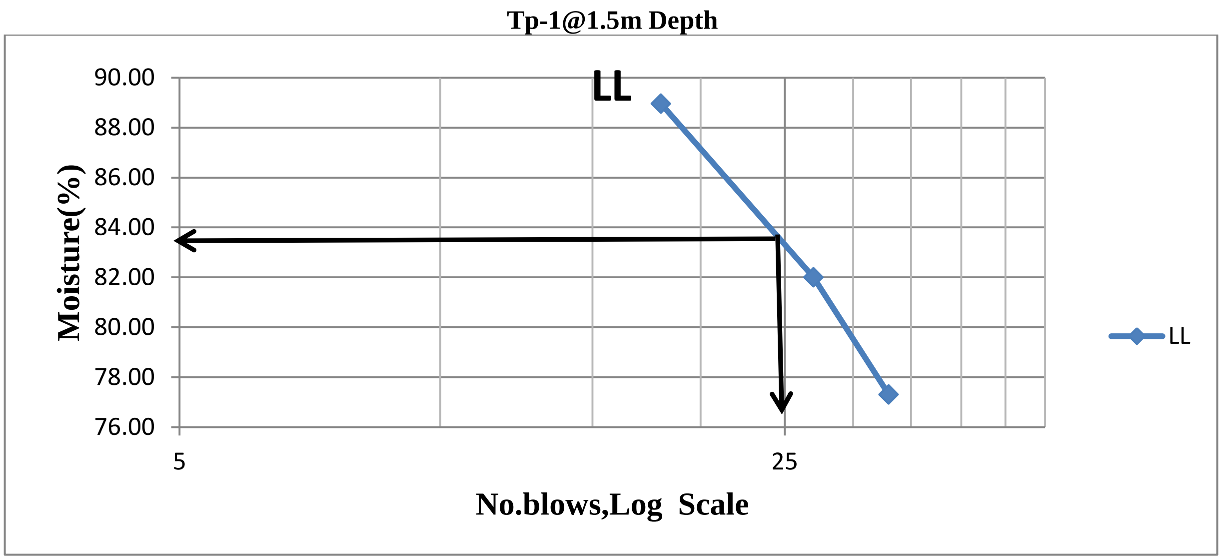


Figure A-1 Atterberg limit test results of TP-1 @ 1.5m, LL = **82.90**%, PL = **37.85**%, PI = **45.06**%

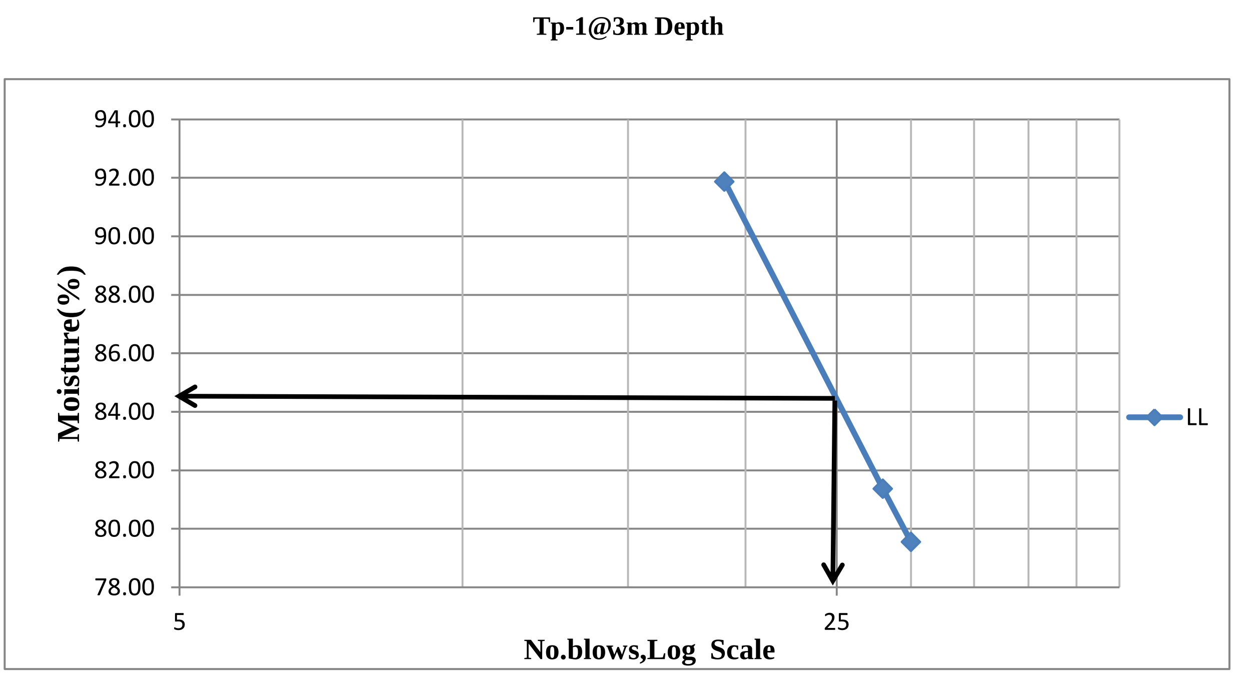


Figure A-2 Atterberg limit test results of TP-1 @ 3m, LL = **84.26**%, PL = **37.87**%, PI = 46.39

Tp-2@1.5m Depth

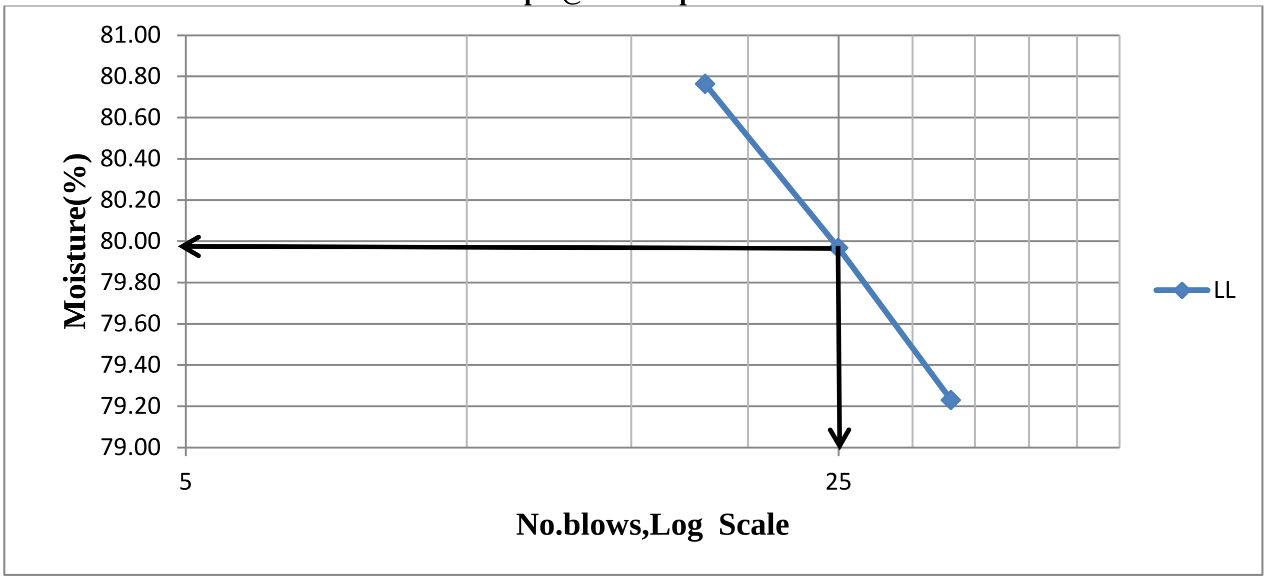


Figure A-3 Atterberg limit test results of TP-2 @ 1.5m, LL = **79.99**%, PL = **37.20**%, PI = 42.79

Tp-3@1.5m Depth

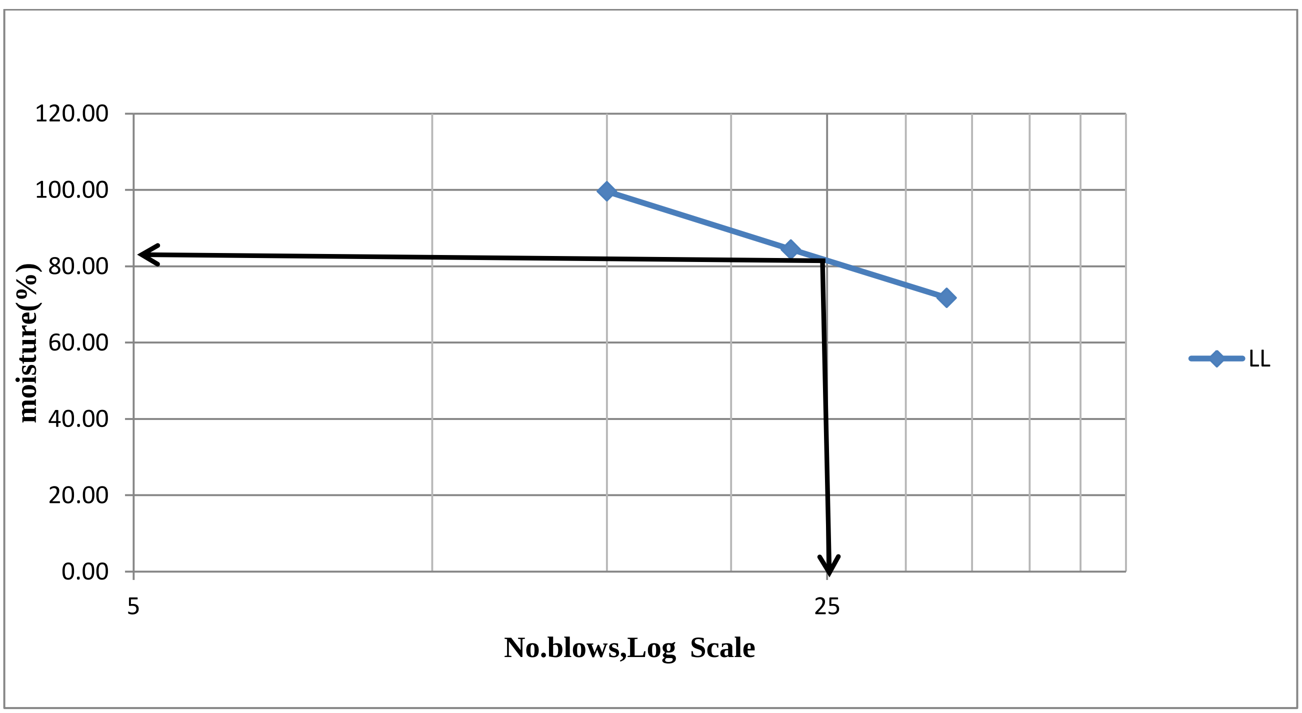


Figure A-4 Atterberg limit test results of TP-3 @ 1.5m, LL = **82.25**%, PL = **49.09**%, PI = 36.16

Tp-4 @3m Depth

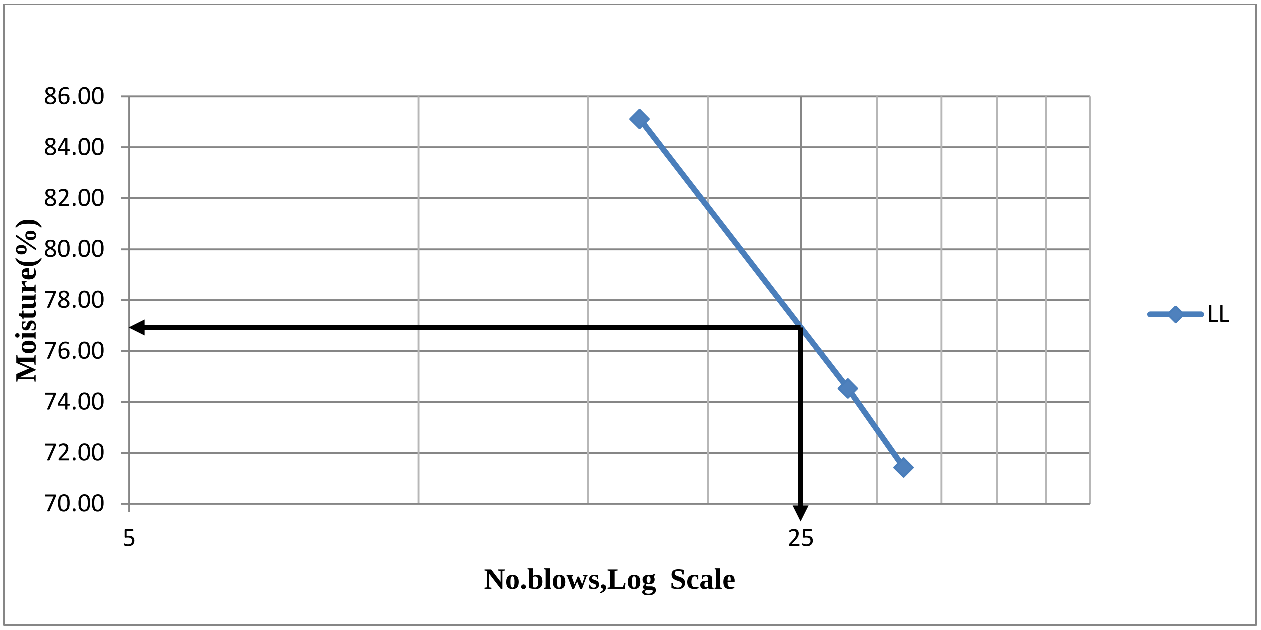


Figure A-5 Atterberg limit test results of TP-4 @ 3m, LL = **77.01**%, PL = **37.72**%, PI = 39.29%

Tp-4@1.5m Depth

Figure A-6 Atterberg limit test results of TP-4 @ 1.5m, LL = **75.50**%, PL = **40.08**%, PI = 5.43%

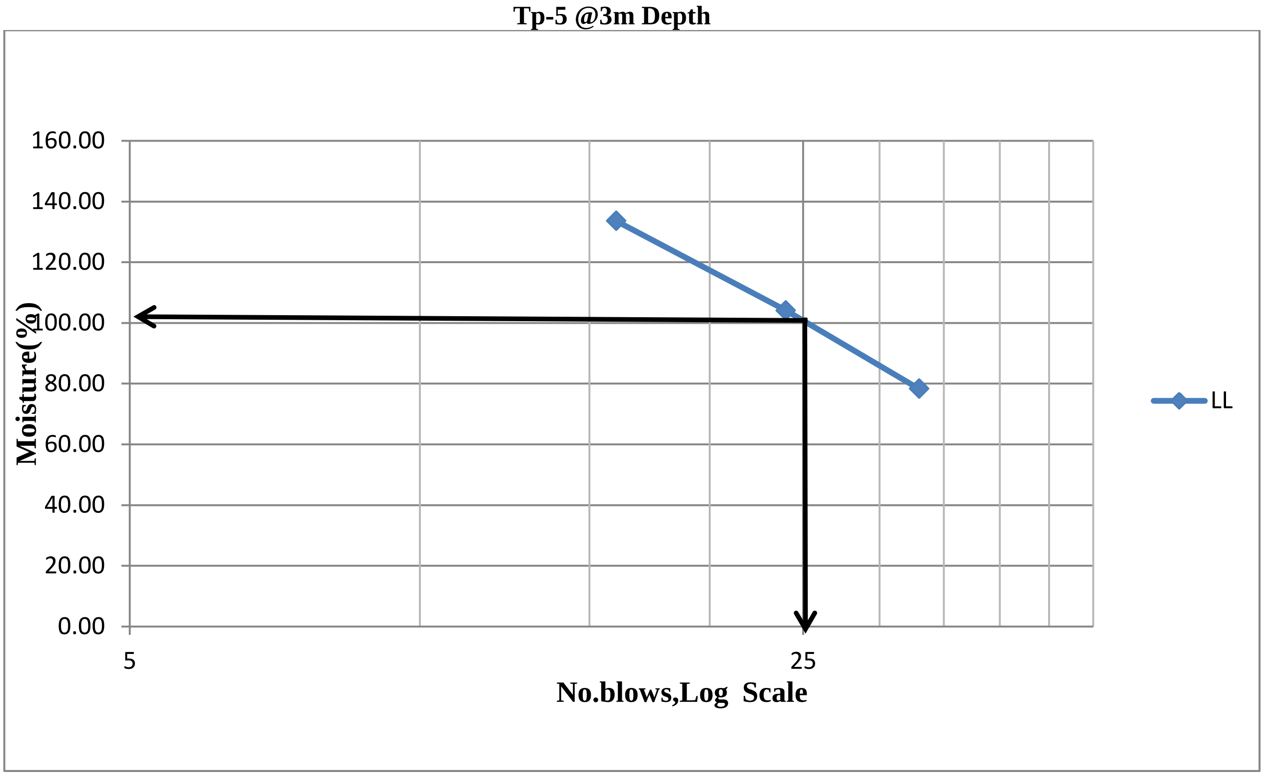


Figure A-7 Atterberg limit test results of TP-5 @ 3m, LL = **105.04**%, PL = **38.77**%, PI = 66.27%

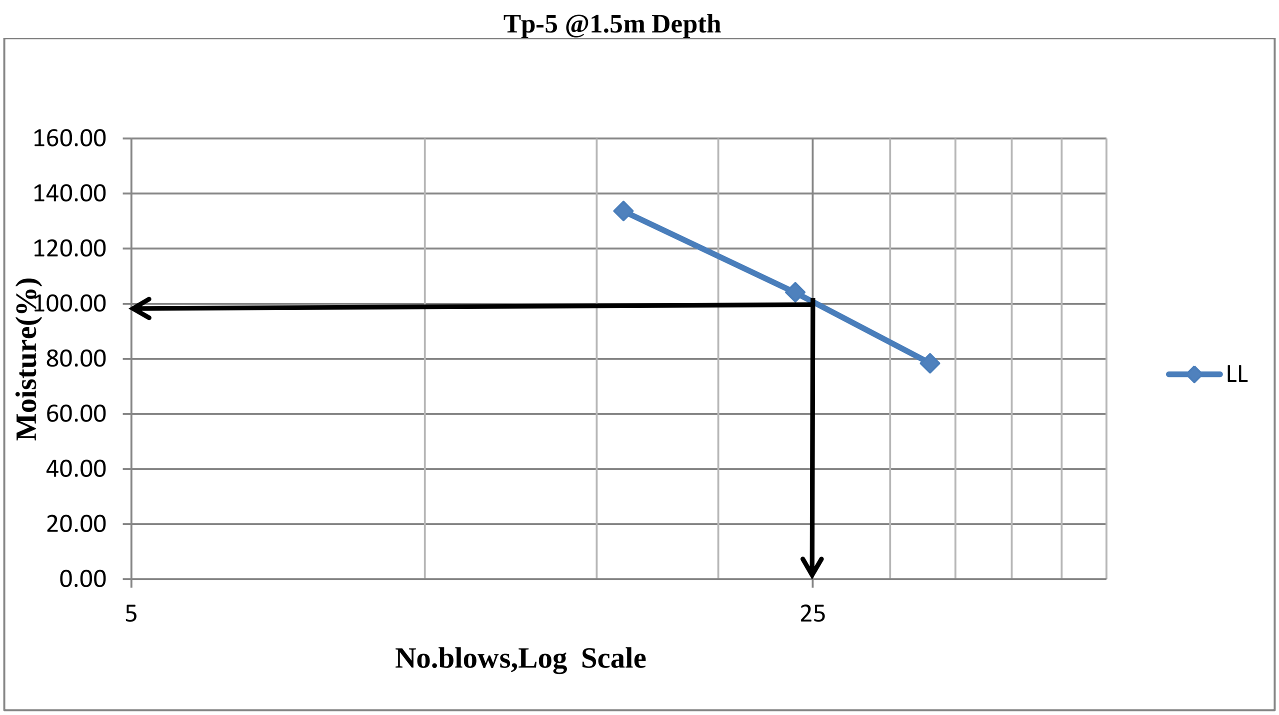


Figure A-8 Atterberg limit test results of TP-5 @ 1.5m, LL = **89.12**%, PL = **38.60**%, PI = 50.51%

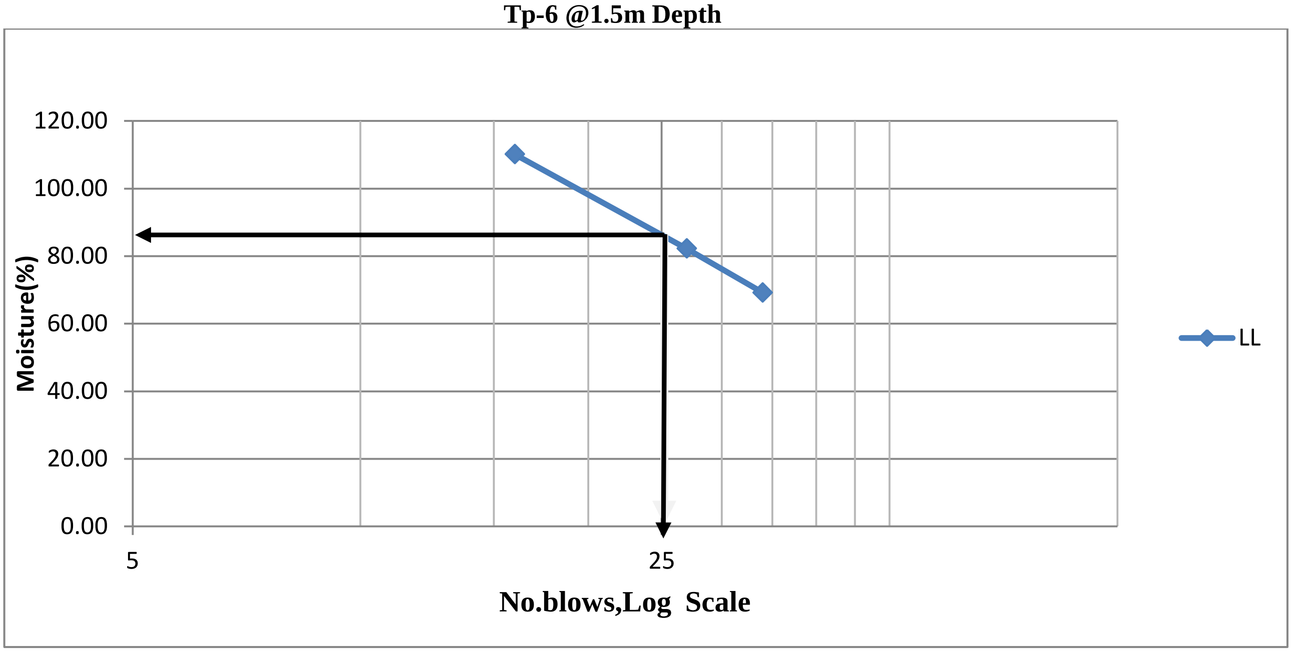


Figure A-9 Atterberg limit test results of TP-6 @ 1.5m, LL = **87.17**%, PL = **40.34**%, PI = 46.83%

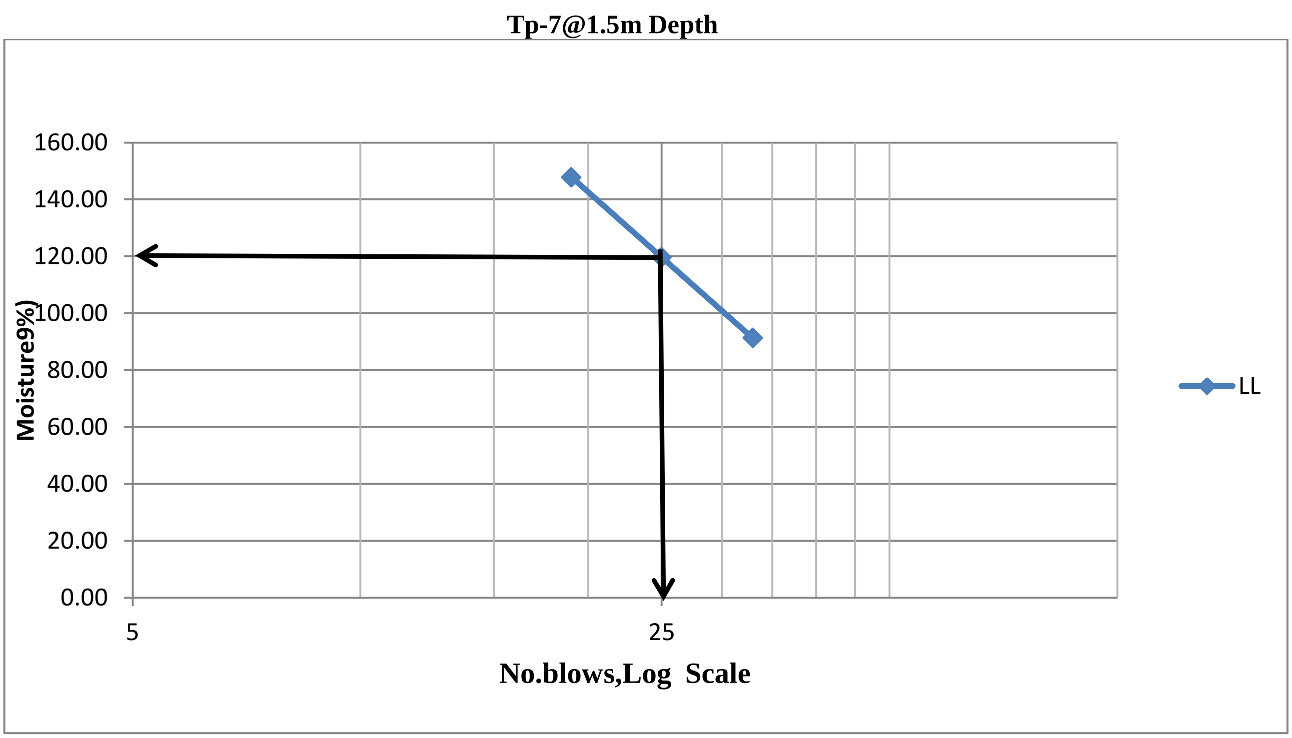


Figure A-10 Atterberg limit test results of TP-7 @ 1.5m, LL = **119.59**%, PL = **39.51**%, PI = 80.08%

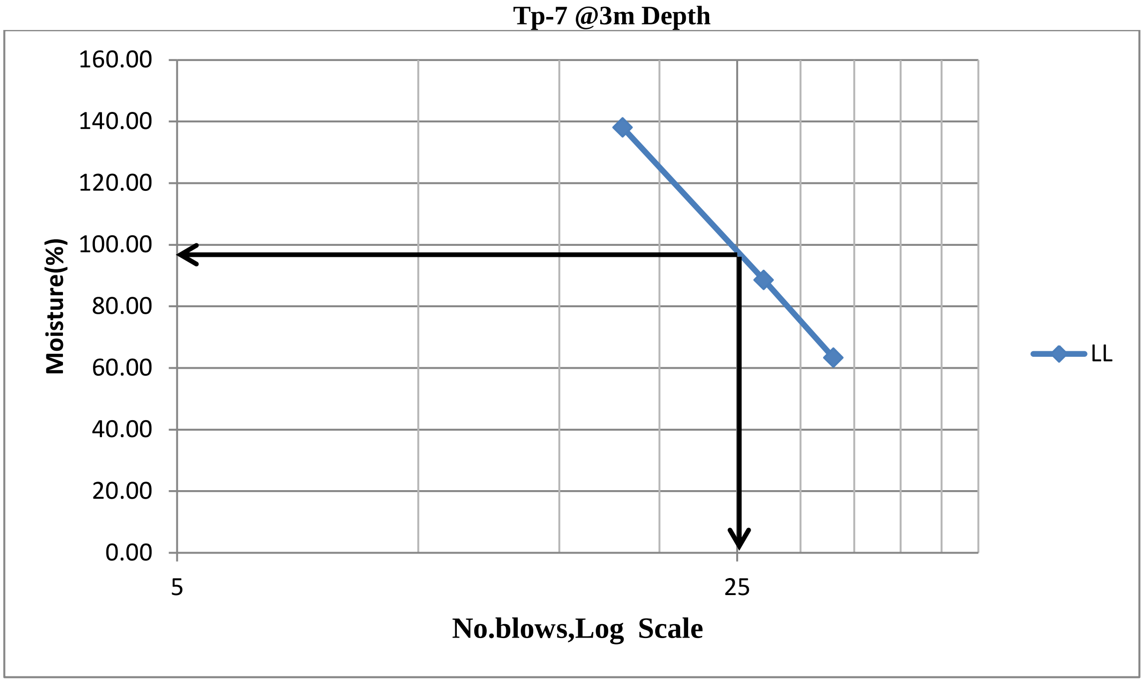


Figure A-11 Atterberg limit test results of TP-7 @ 3m, LL = **96.57**%, PL = **35.72**%, PI = 60.86%

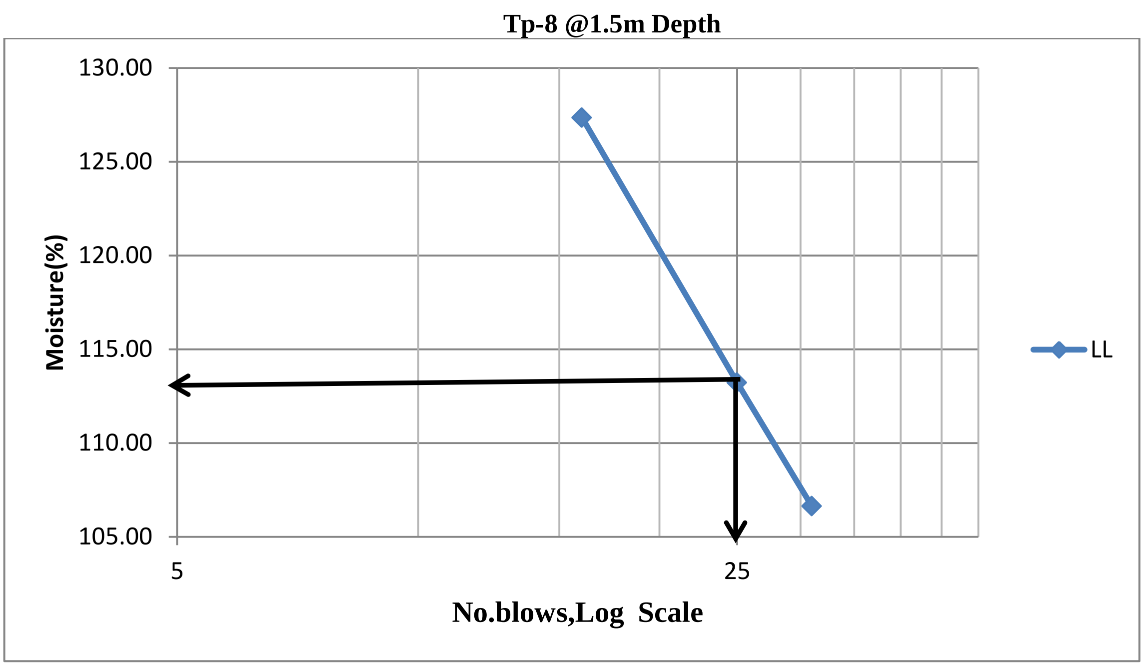


Figure A-12 Atterberg limit test results of TP-8 @ 1.5m, LL = **115.72**%, PL = **35.72**%, PI = 80.01%

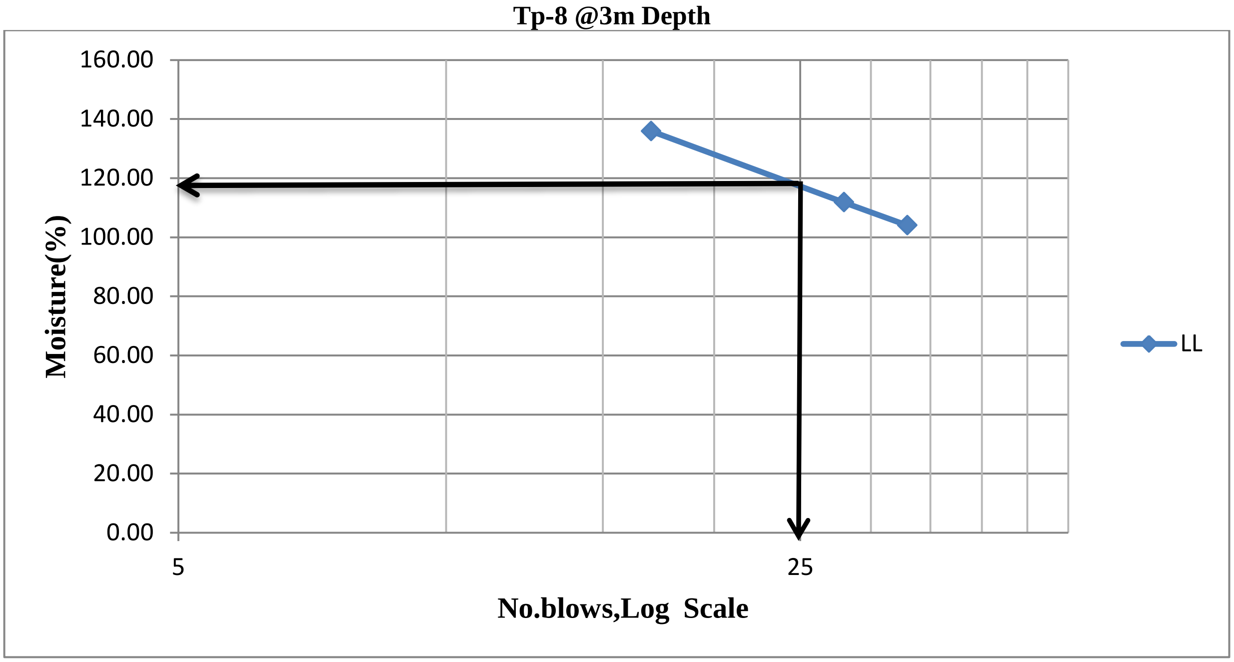


Figure A-13 Atterberg limit test results of TP-8 @ 3m, LL = **117.23**%, PL = **39.74**%, PI = 77.49%

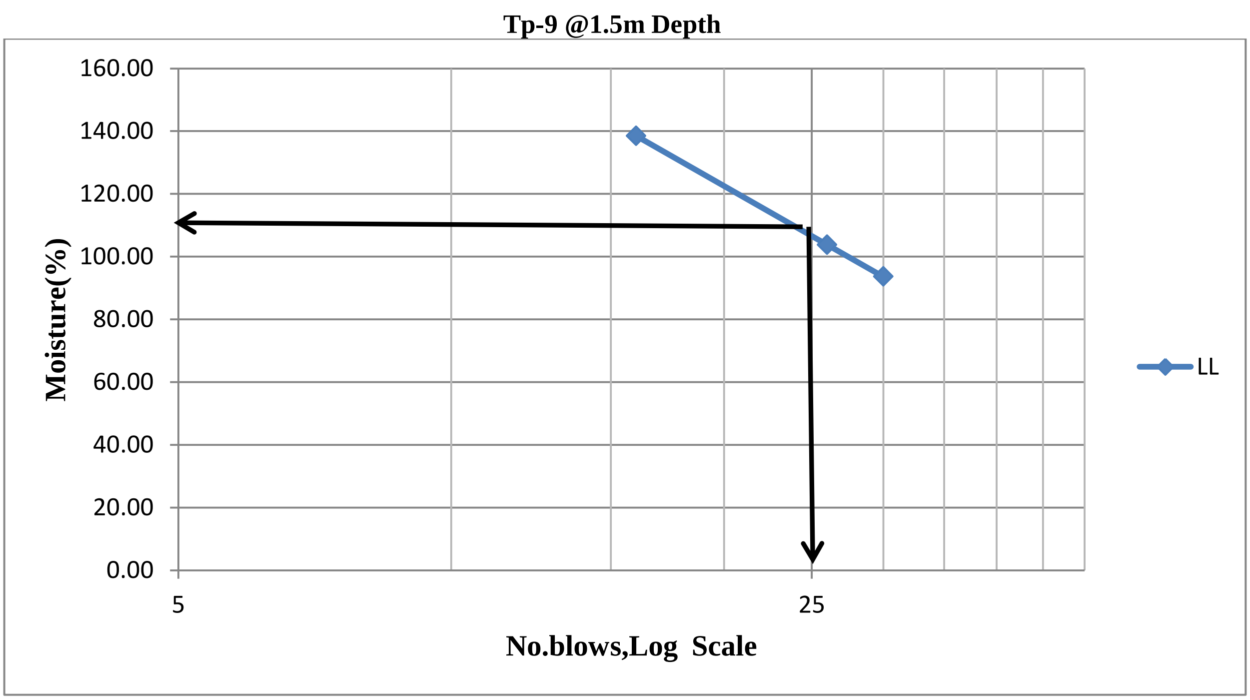


Figure A-14 Atterberg limit test results of TP-9 @ 1.5m, LL = **111.95**%, PL = **28.92**%, PI = 83.03%

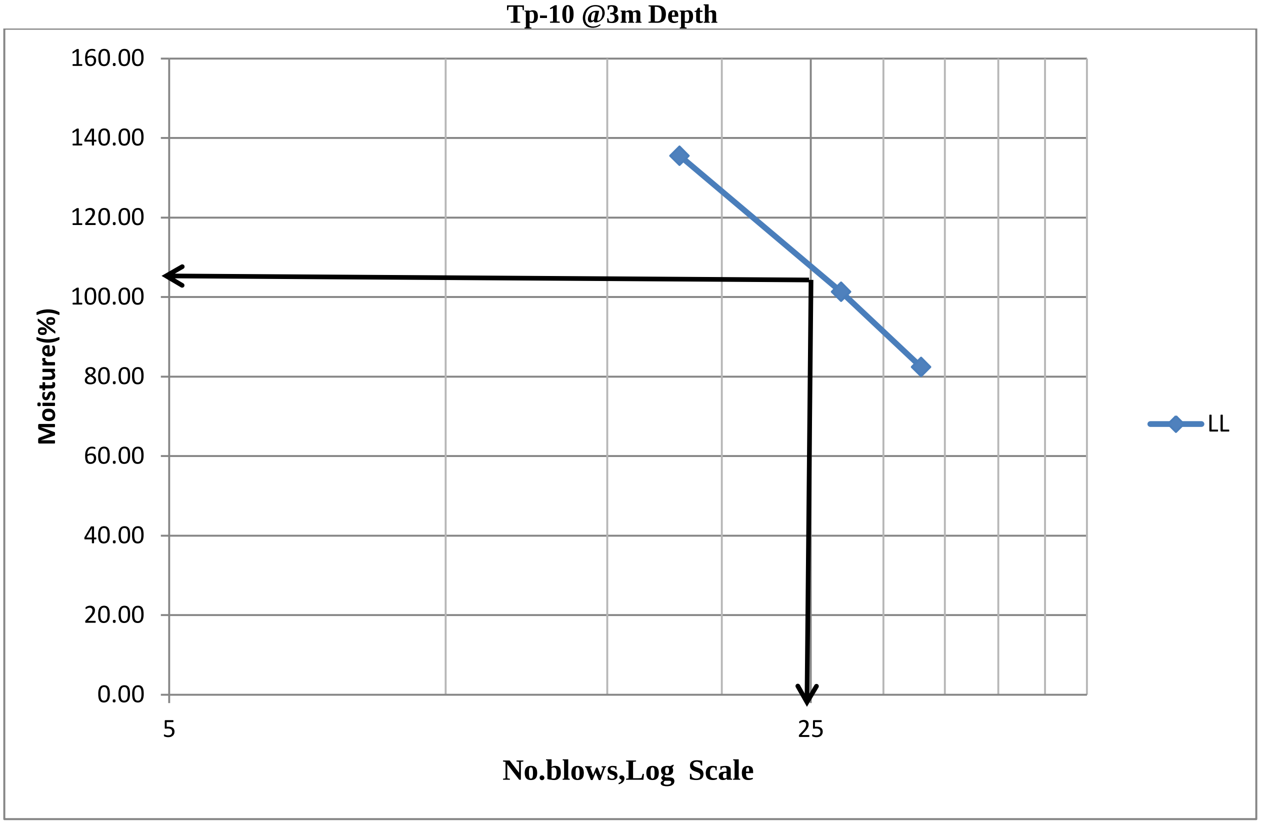


Figure A-15 Atterberg limit test results of TP-10@ 3m, LL = **106.34**%, PL = **30.77**%, PI = 75.57%

Appendix: B Data for free swell test results

Determination No.	Measuring Cylinder No.		Reading After 24 Hours		Free Swell Index,%
	Kerosene (ml)	Distilled Water (ml)	Kerosene (ml)	Distilled Water (ml)	
Pit-1at 1.5m Depth(Dibora)	10	10	11.3	24	112.39
Pit-1at 3m Depth(Dibora)	10	10	11	23	109.09
Pit-2 at 1.5m Depth(Dry Port)	10	10	10.3	18.5	79.61
Pit-3 at 1.5m Depth(Dry Port)	10	10	10.6	19	79.25
Pit-4 at 1.5m Depth(Seyyo)	10	10	10.7	22	105.61
Pit-4 at 3m Depth(Seyyo)	10	10	10.3	21.3	106.80
Pit -5 at 1.5m Depth(Seyyo)	10	10	10.2	21.7	112.75
Pit-5 at 3m Depth(Seyyo)	10	10	10.1	21.2	109.90
Pit-6 at 1.5m Depth(Dry Port)	10	10	10.8	19.2	77.78
Pit-7 at 1.5m Depth(Dibora)	10	10	11.5	22.7	97.39
Pit-7 at 3m Depth(Dibora)	10	10	11.2	22.4	100.00
Pit-8 at 1.5m Depth(Seyyo)	10	10	10.9	21.5	97.25
Pit-9 at 3m Depth(Seyyo)	10	10	11.1	21.1	90.09
Pit-9 at 1.5m Depth(Seyyo)	10	10	11	22.6	105.45
Pit-10 at 3m Depth(Dibora)	10	10	11.2	22.3	99.11

Appendix :C Grain Size Distribution Curve

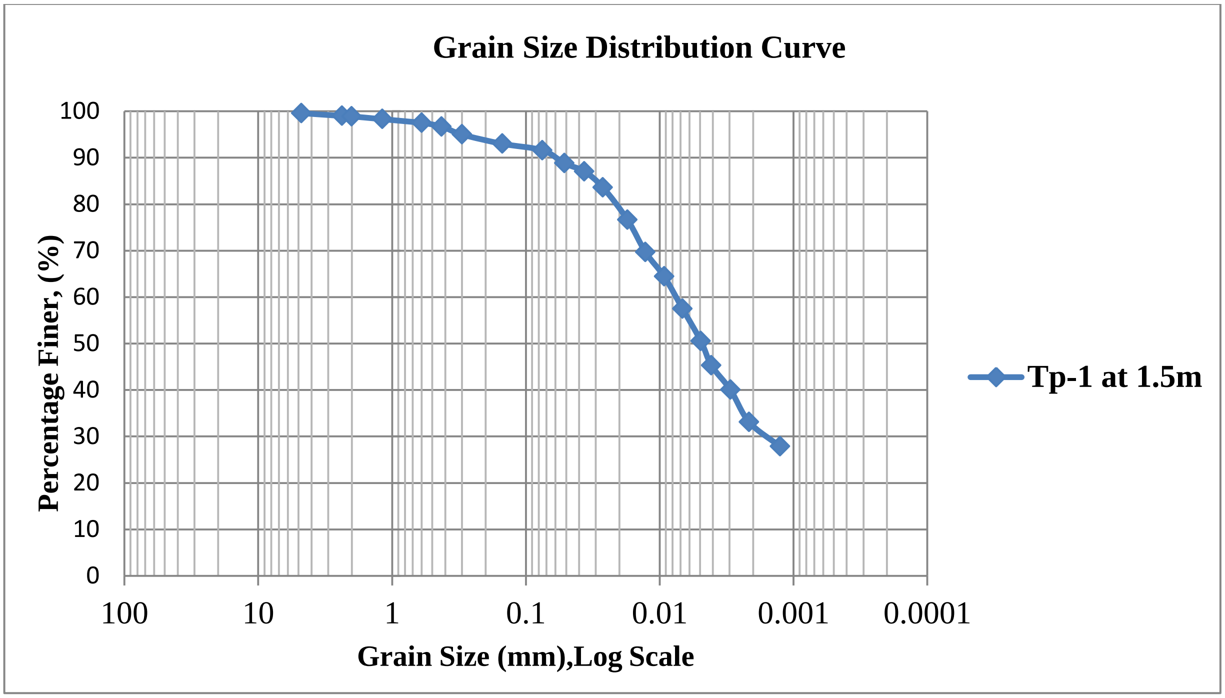


Figure C-1 Grain Size Distribution curve of TP-1 @ 1.5m Depth

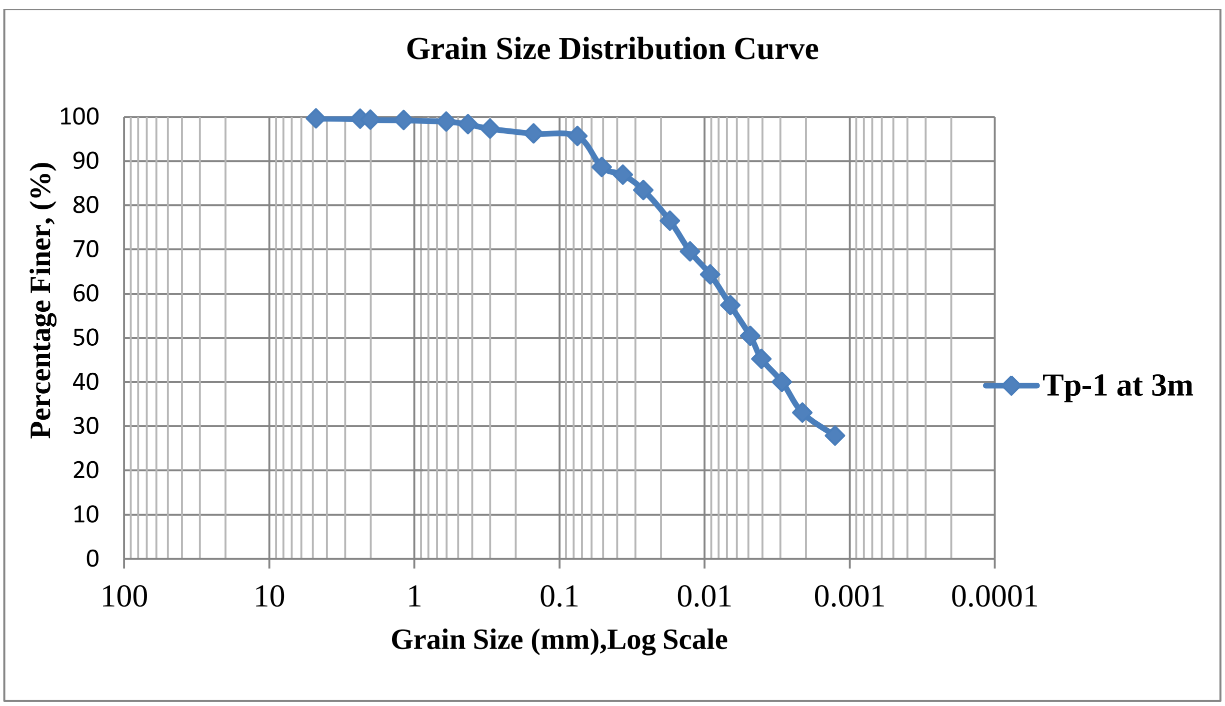


Figure C-2 Grain Size Distribution Curve of TP-1 @3m Depth

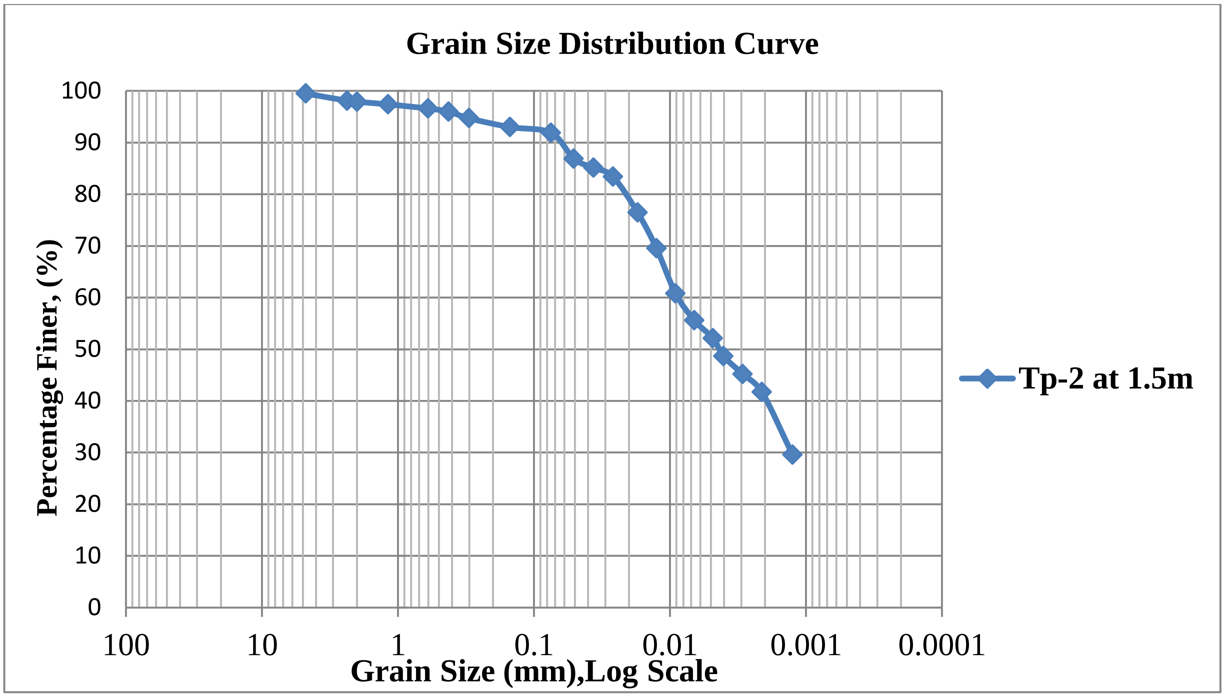


Figure C-3 Grain Size Distribution Curve of TP-2 @1.5m Depth

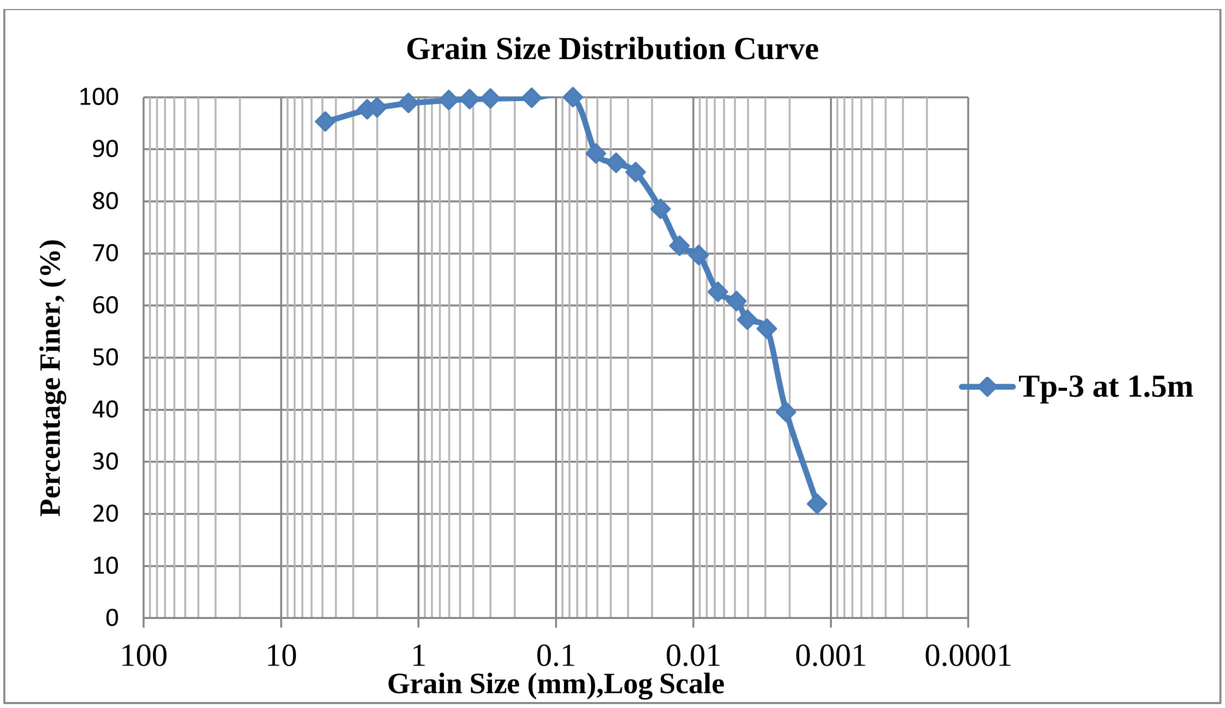


Figure C-3 Grain Size Distribution Curve of TP-3 @1.5m Depth

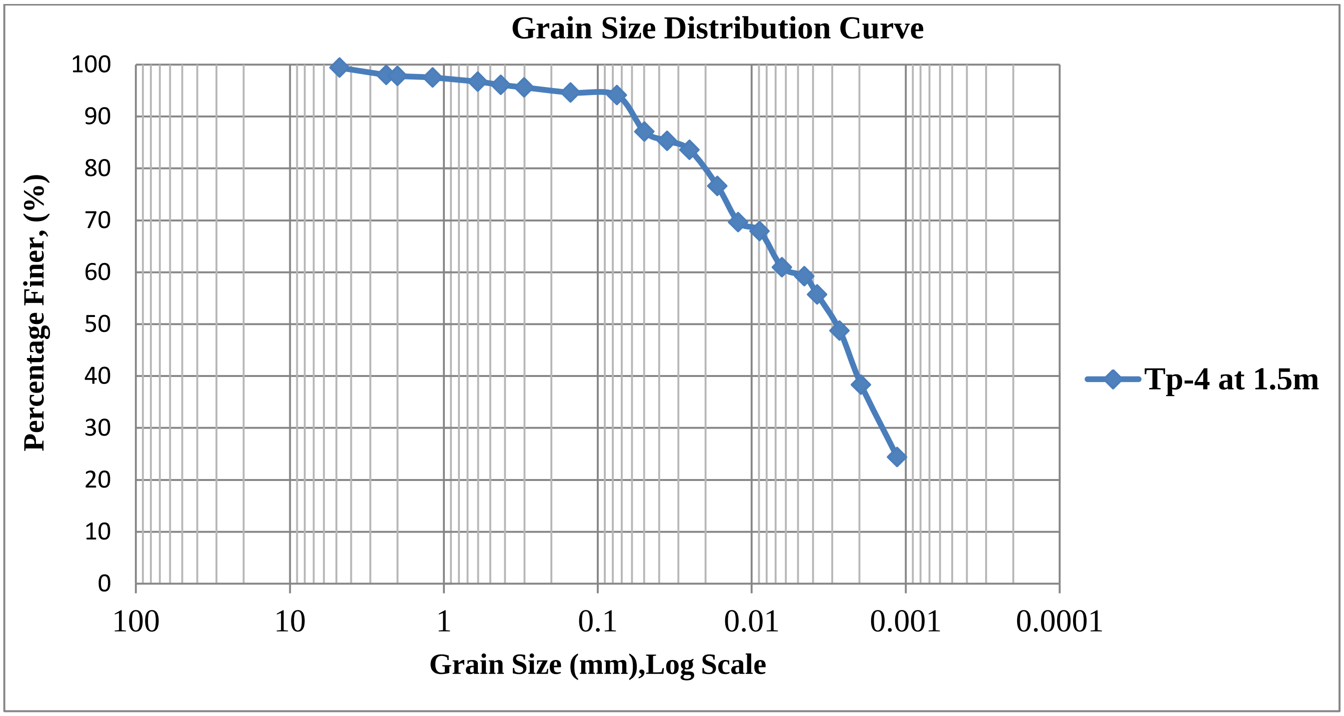


Figure C-3 Grain Size Distribution Curve of TP-4 @1.5m Depth

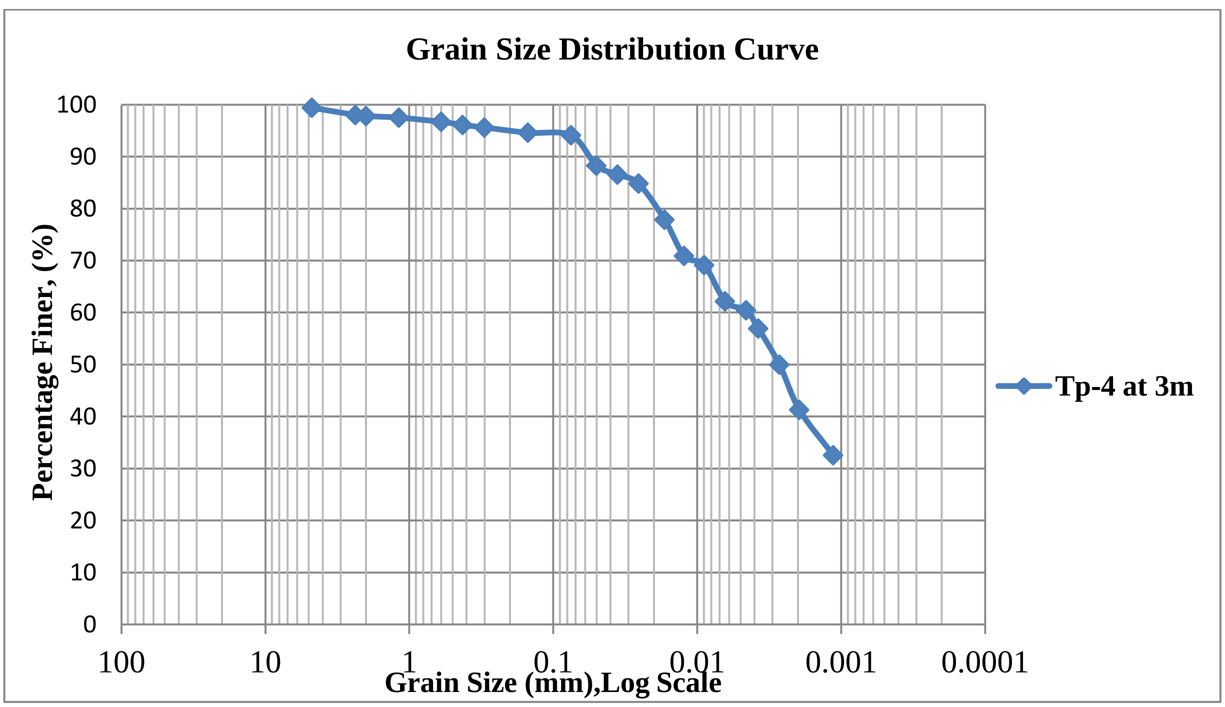


Figure C-3 Grain Size Distribution Curve of TP-4 @3m Depth

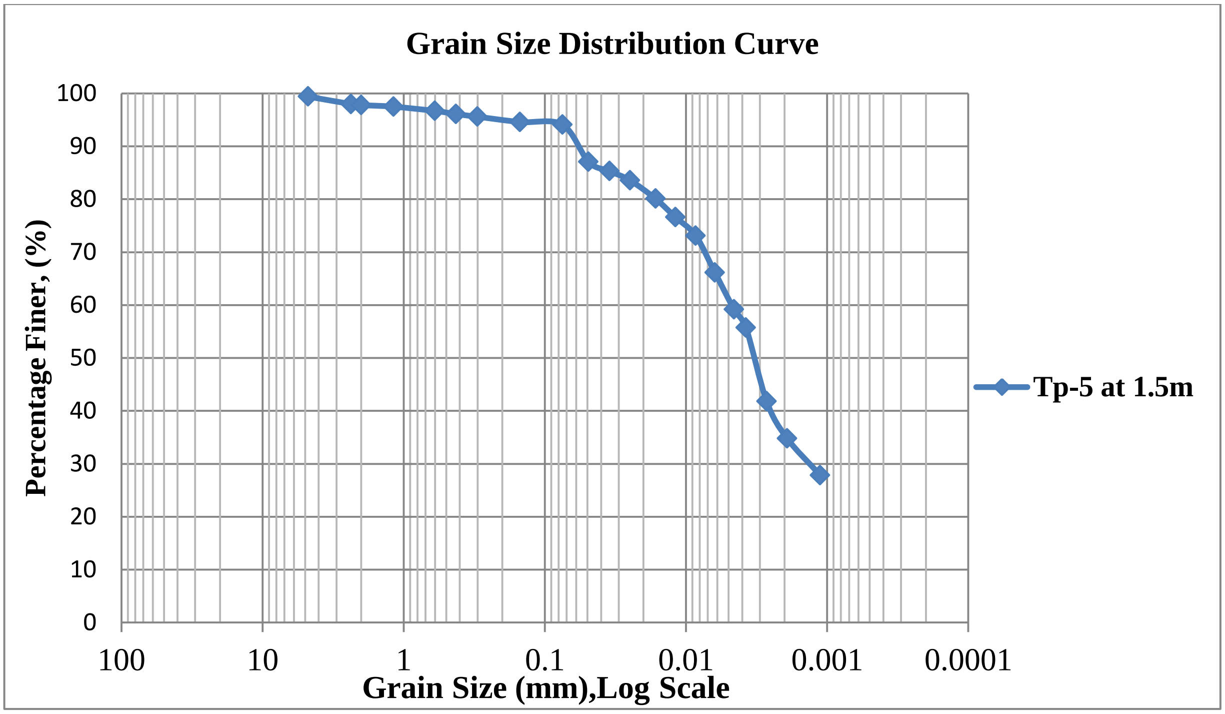


Figure C-3 Grain Size Distribution Curve of TP-5 @1.5m Depth

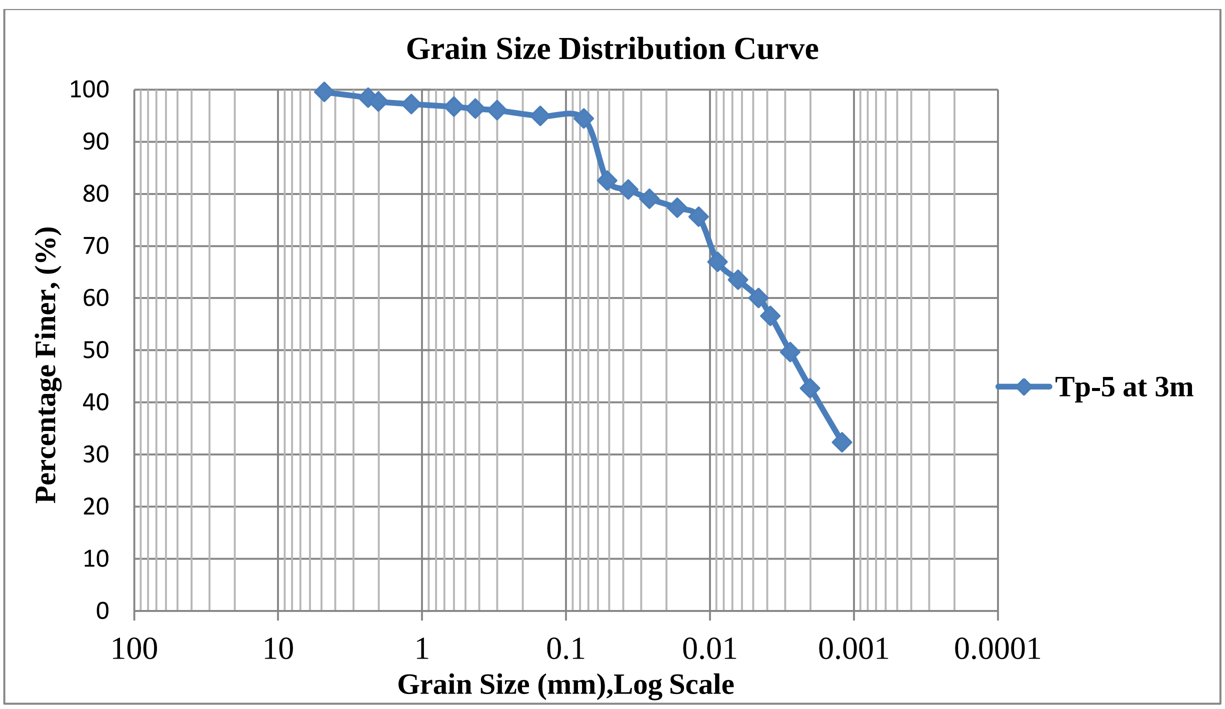


Figure C-3 Grain Size Distribution Curve of TP-5 @3m Depth

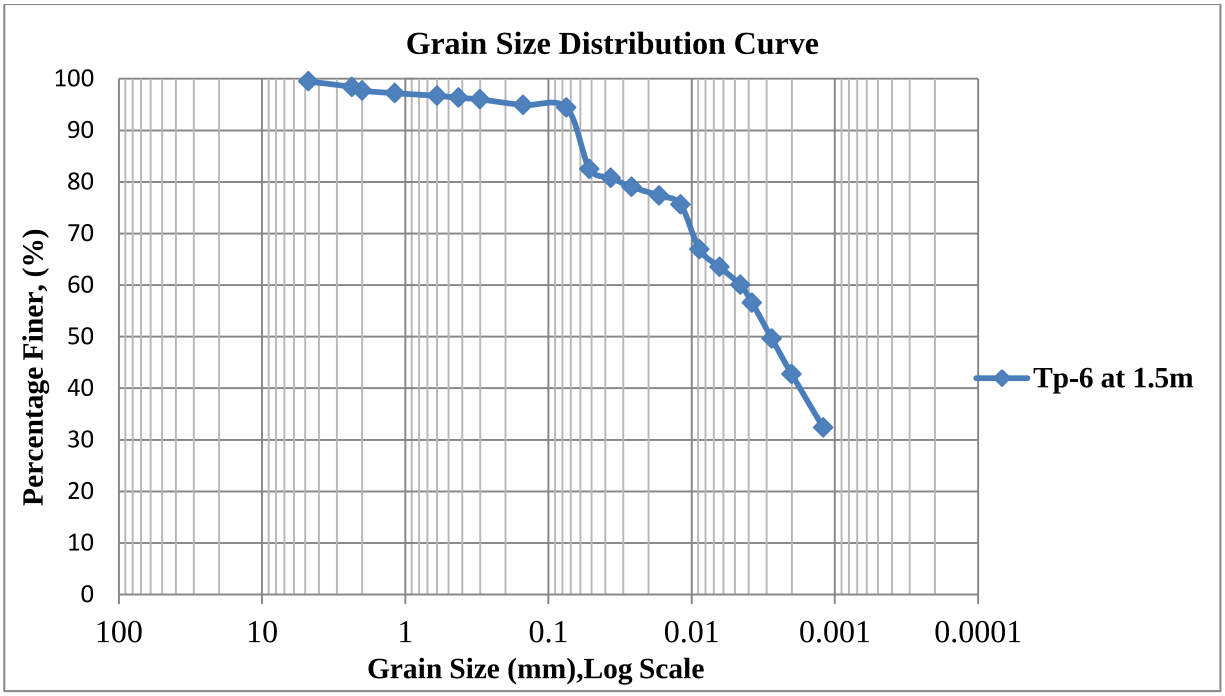


Figure C-3 Grain Size Distribution Curve of TP-6 @1.5m Depth

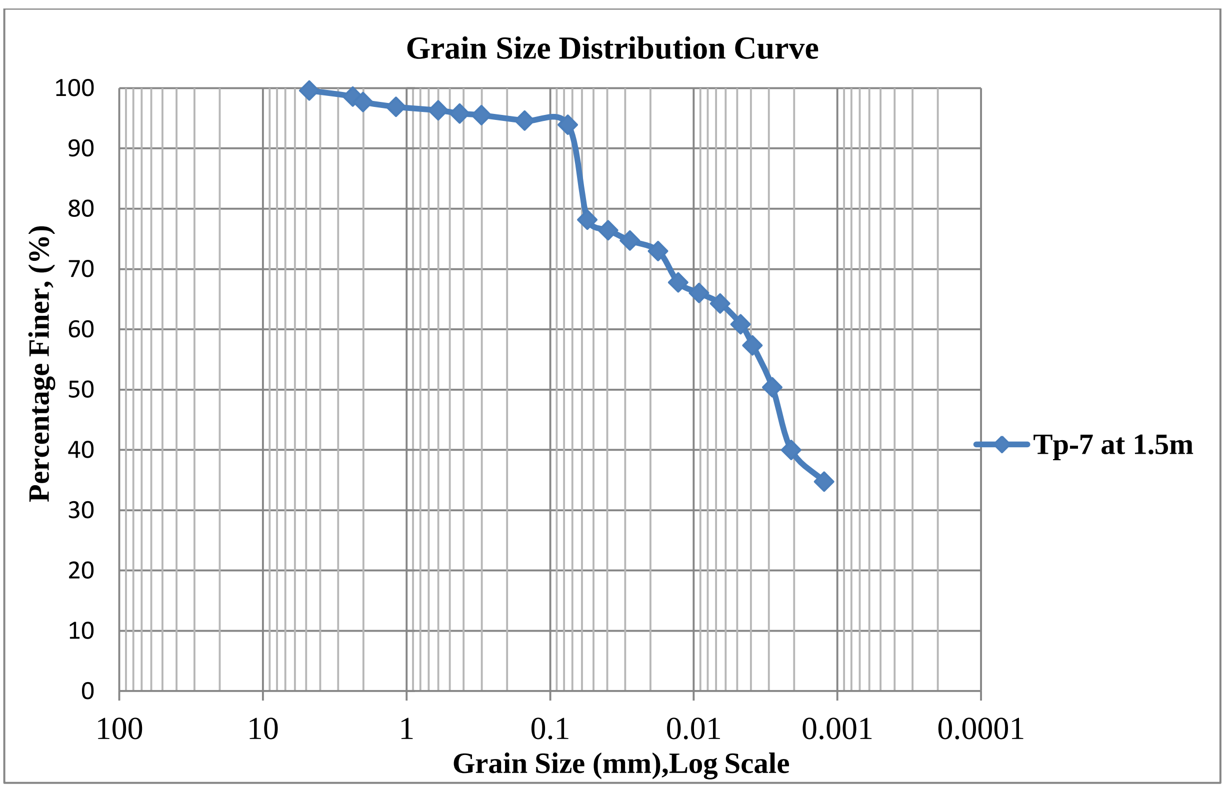


Figure C-3 Grain Size Distribution Curve of TP-7 @1.5m Depth

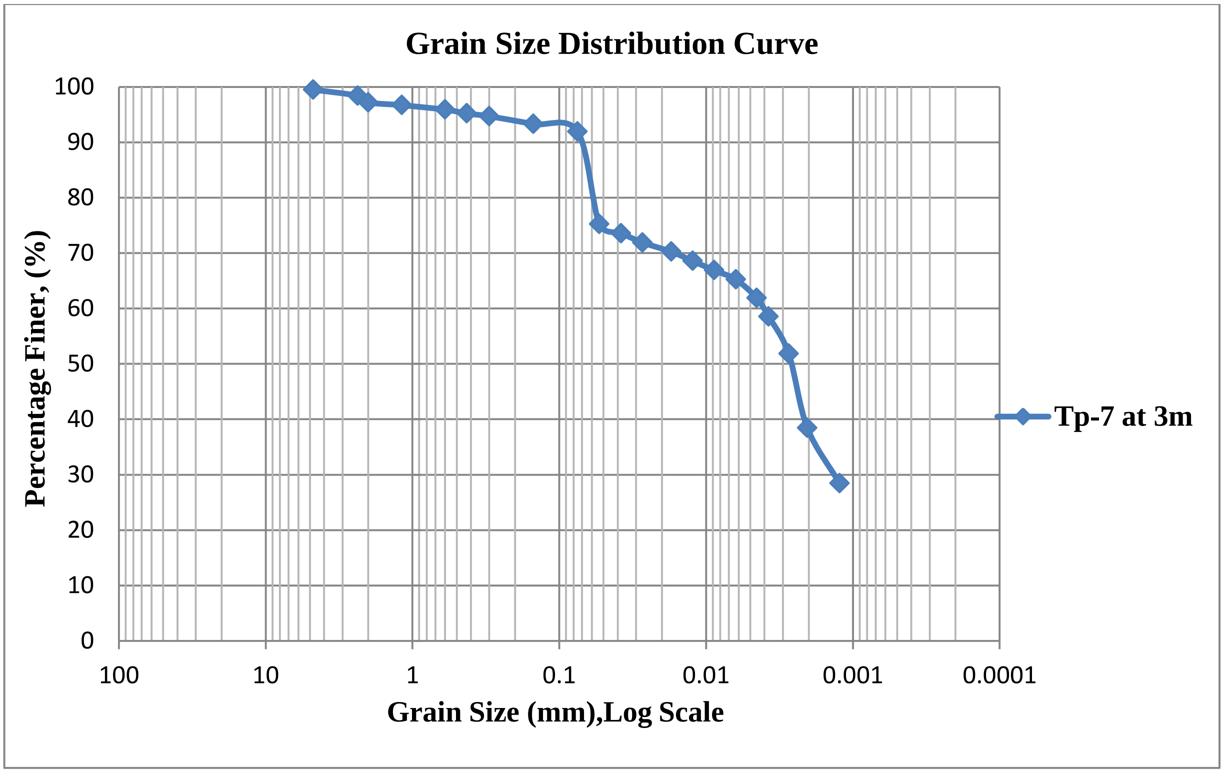


Figure C-3 Grain Size Distribution Curve of TP-7 @3m Depth

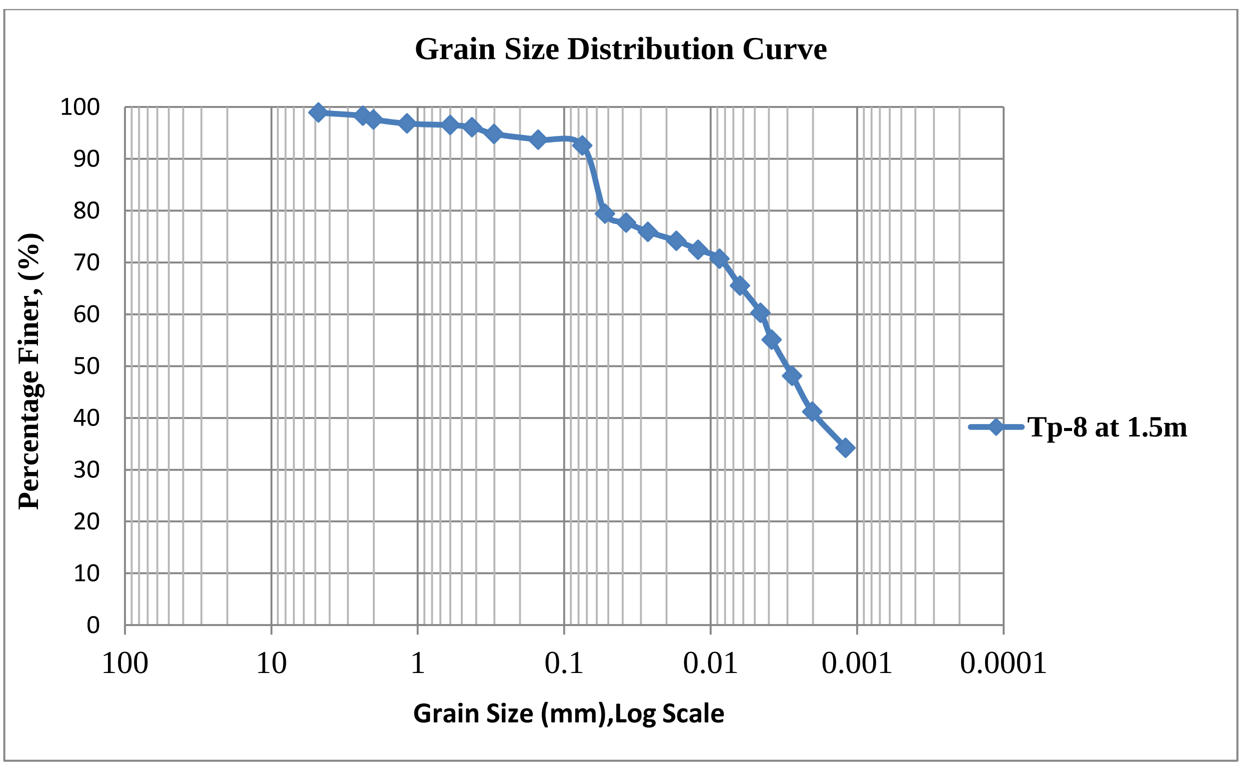


Figure C-3 Grain Size Distribution Curve of TP-8 @1.5m Depth

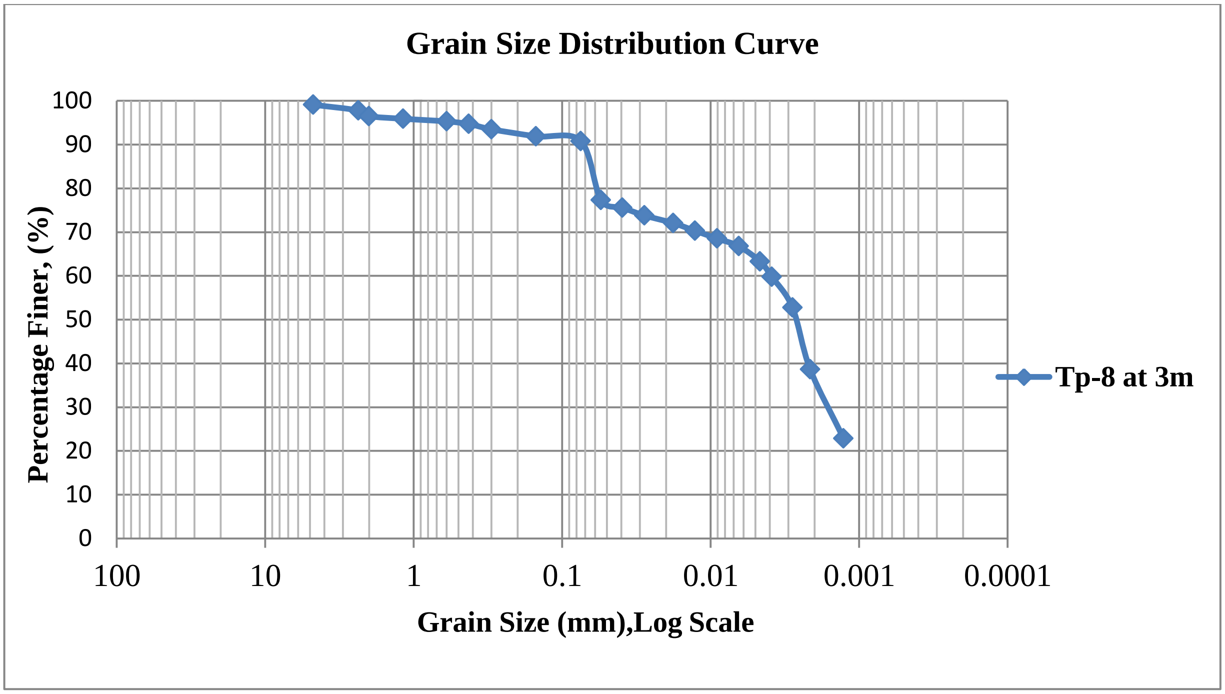


Figure C-3 Grain Size Distribution Curve of TP-8 @3m Depth

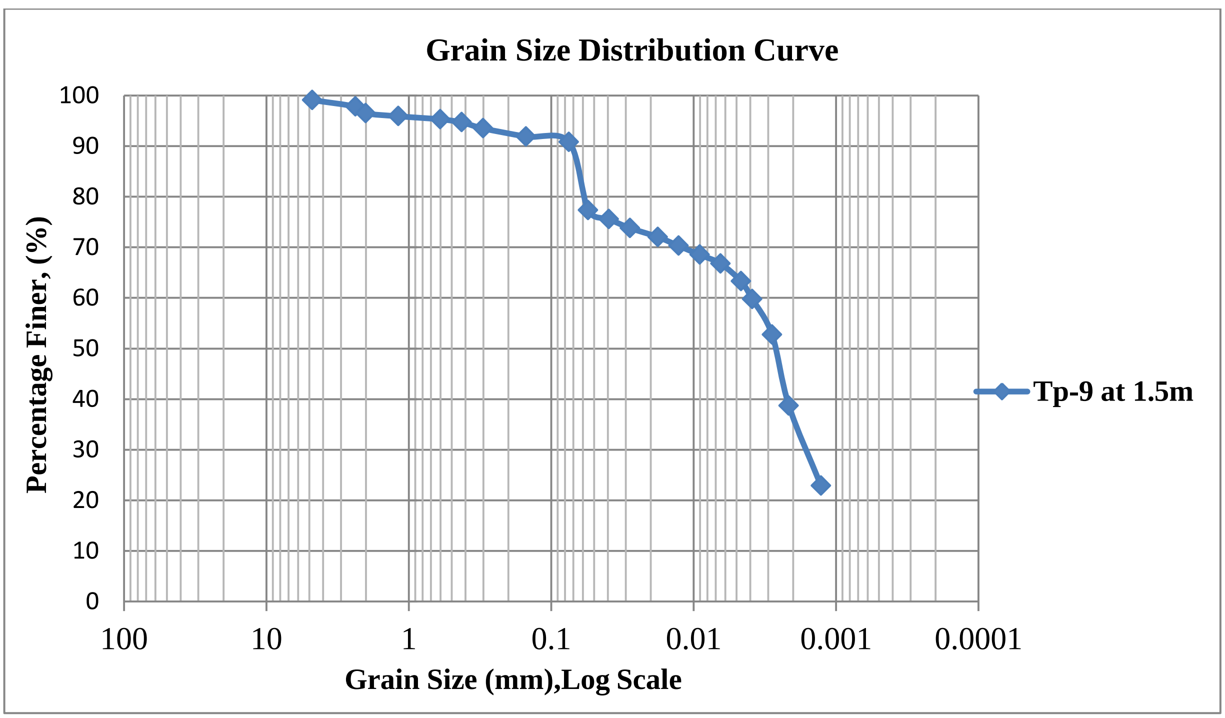


Figure C-3 Grain Size Distribution Curve of TP-9 @1.5m Depth

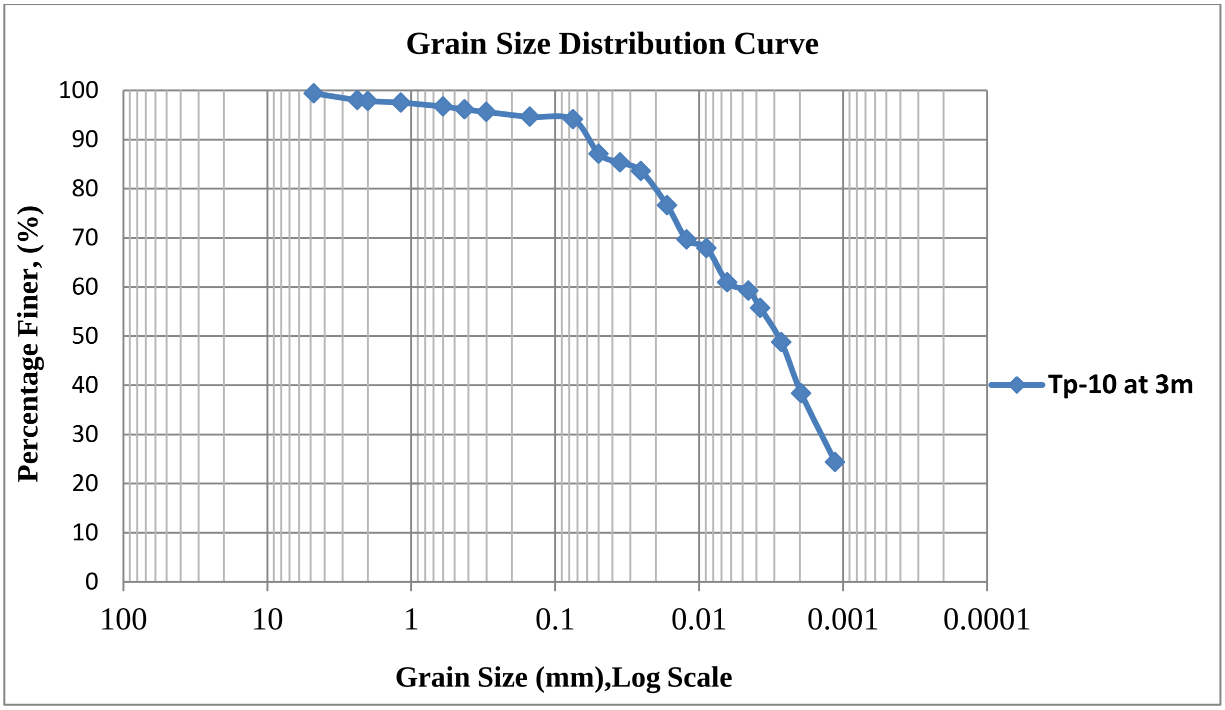
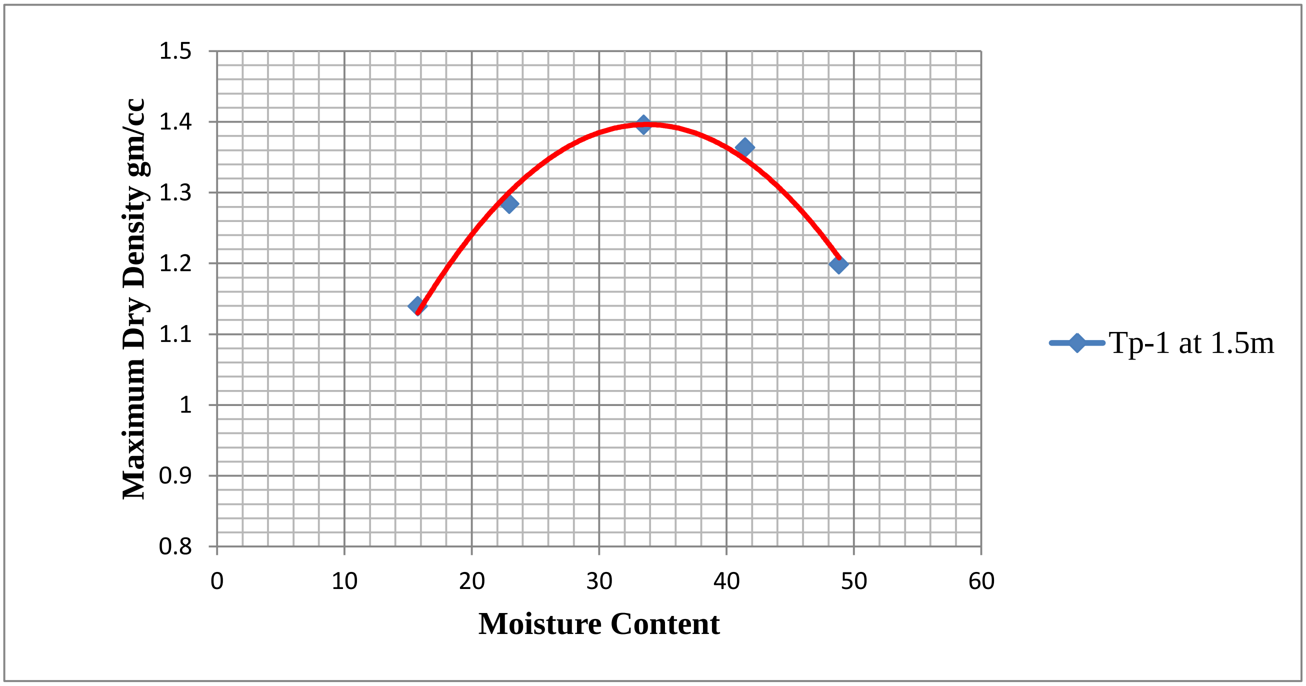
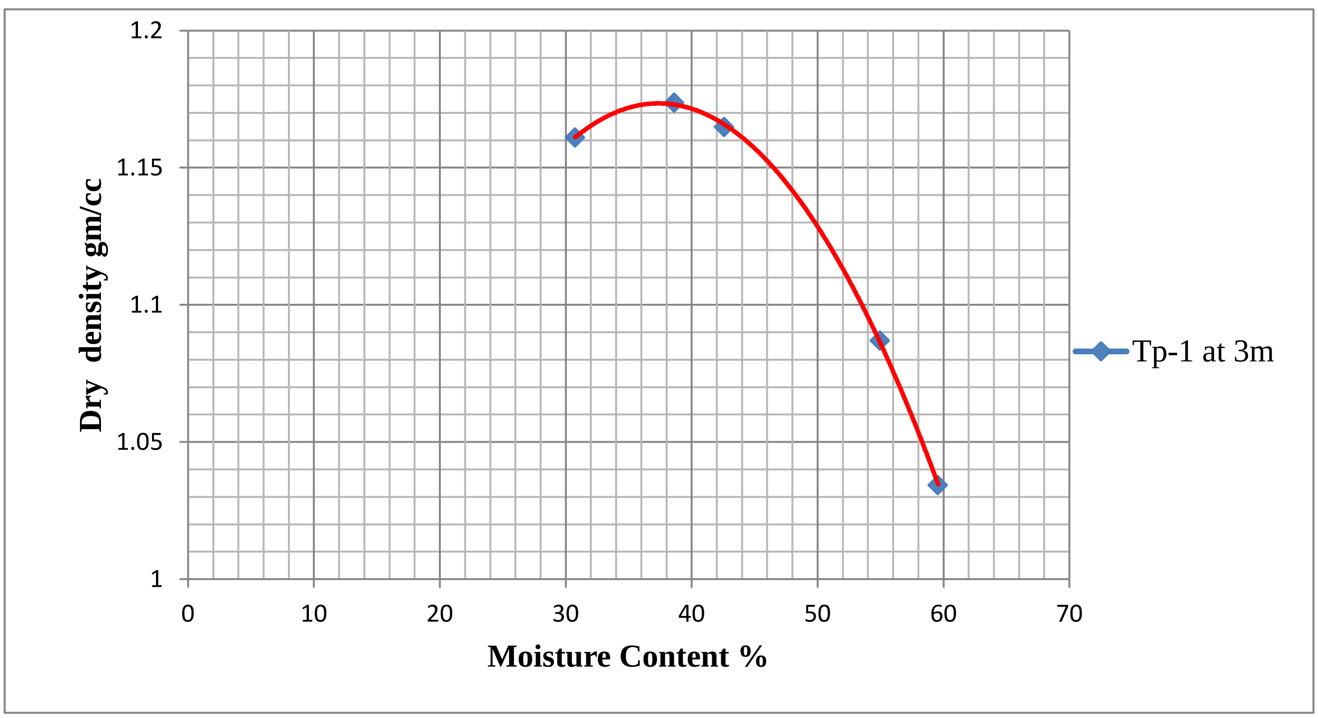


Figure C-3 Grain Size Distribution Curve of TP-10 @3m Depth

Appendix: D Standard Compaction Curve Dry density Vs. Moisture





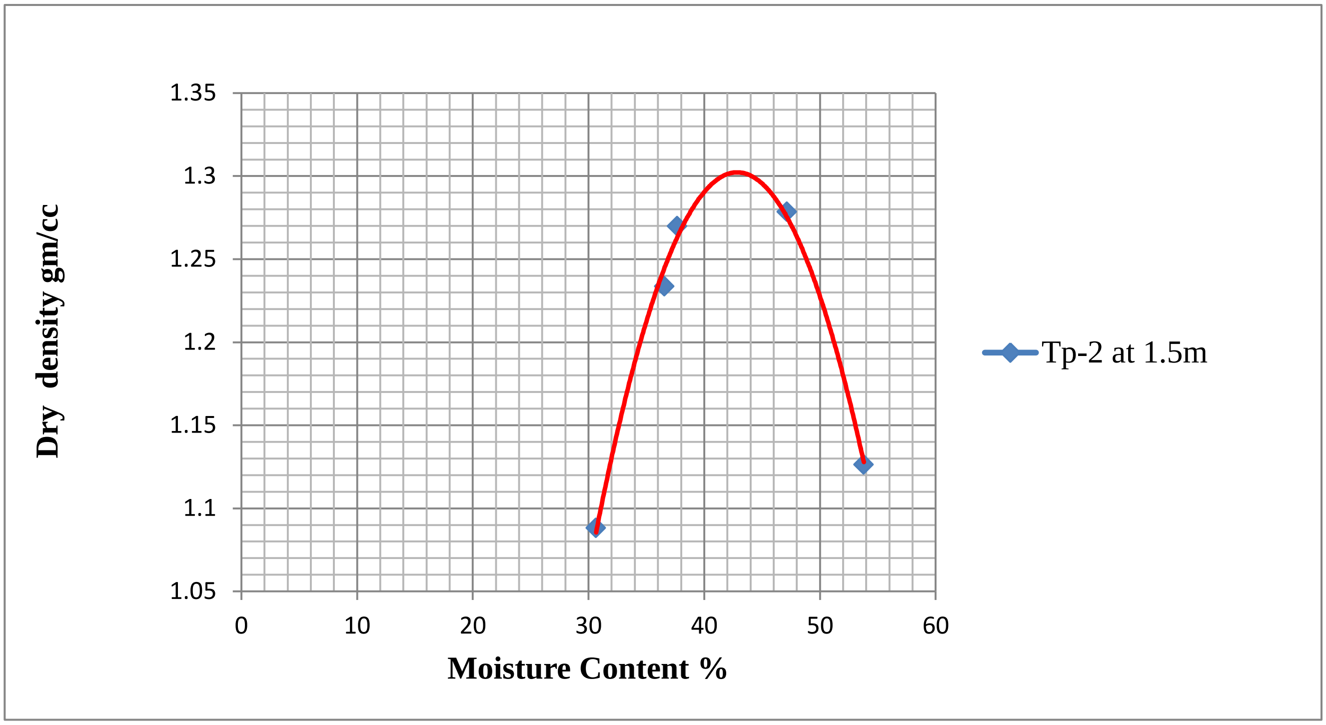


Figure -3 Standard Compaction Curve Dry density Vs Moisture Tp-2 at 1.5m

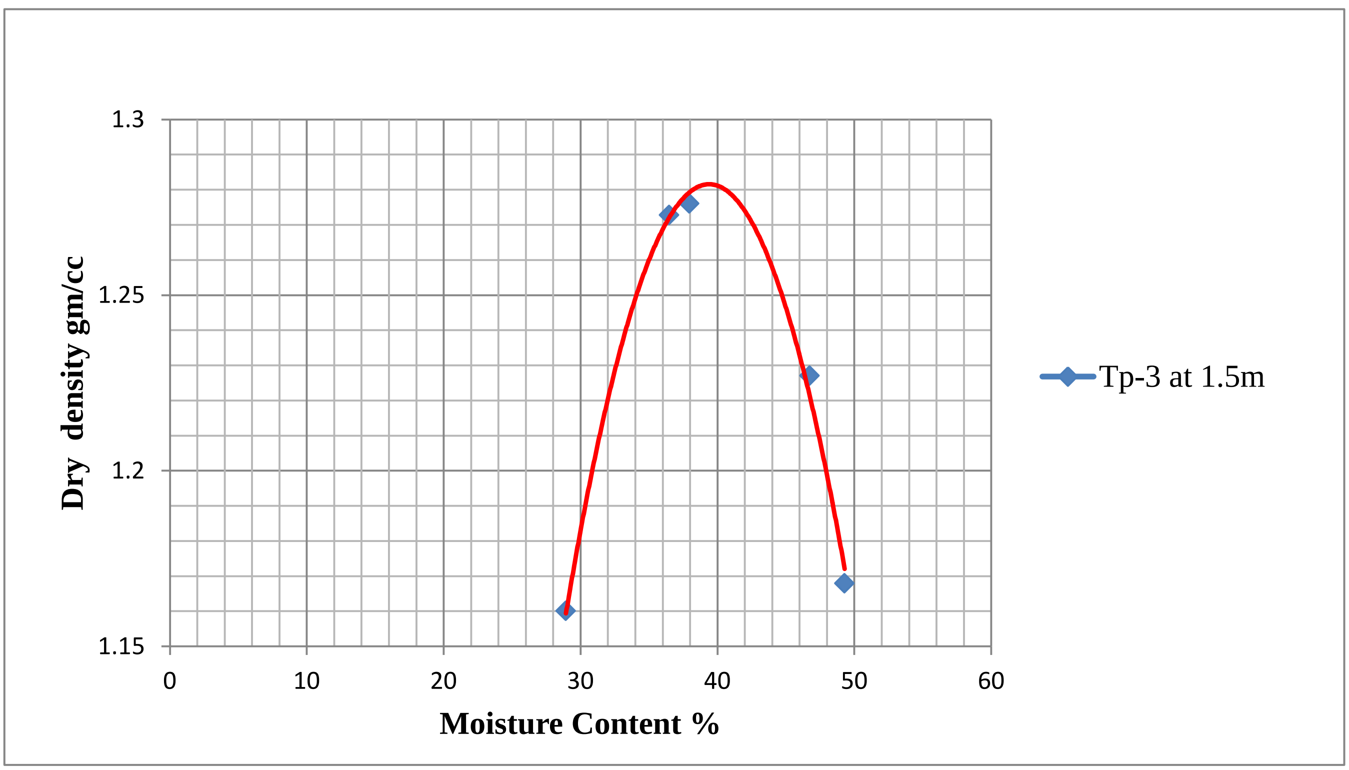


Figure -4 Standard Compaction Curve Dry density Vs Moisture Tp-3 at 1.5m

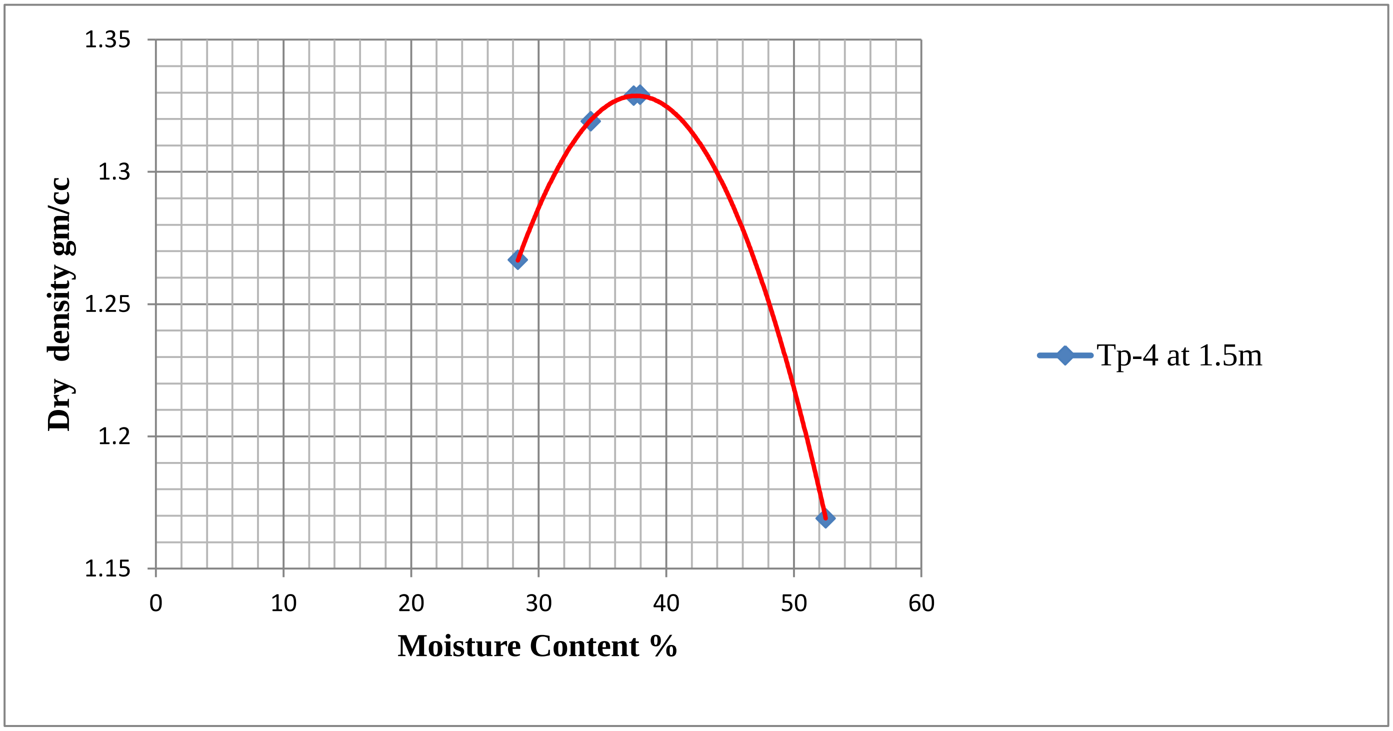


Figure -5 Standard Compaction Curve Dry density Vs Moisture Tp-4 at 1.5m

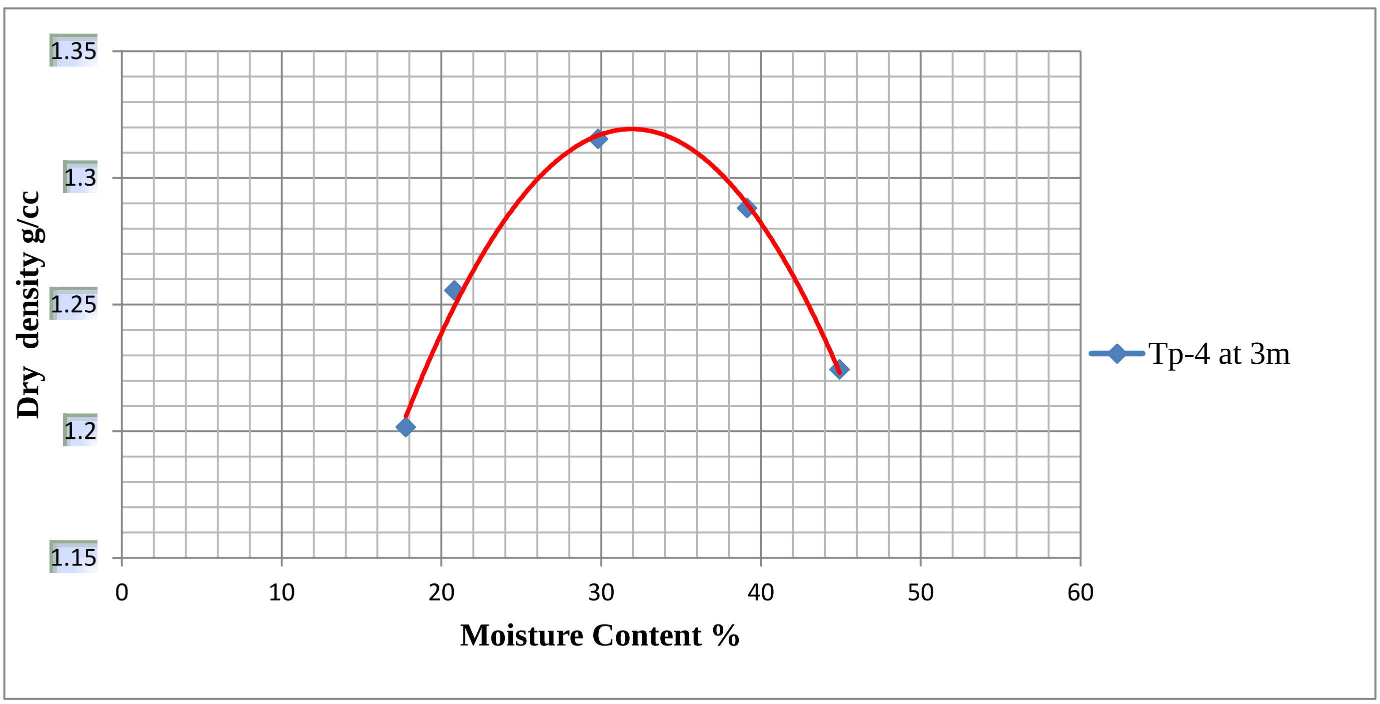
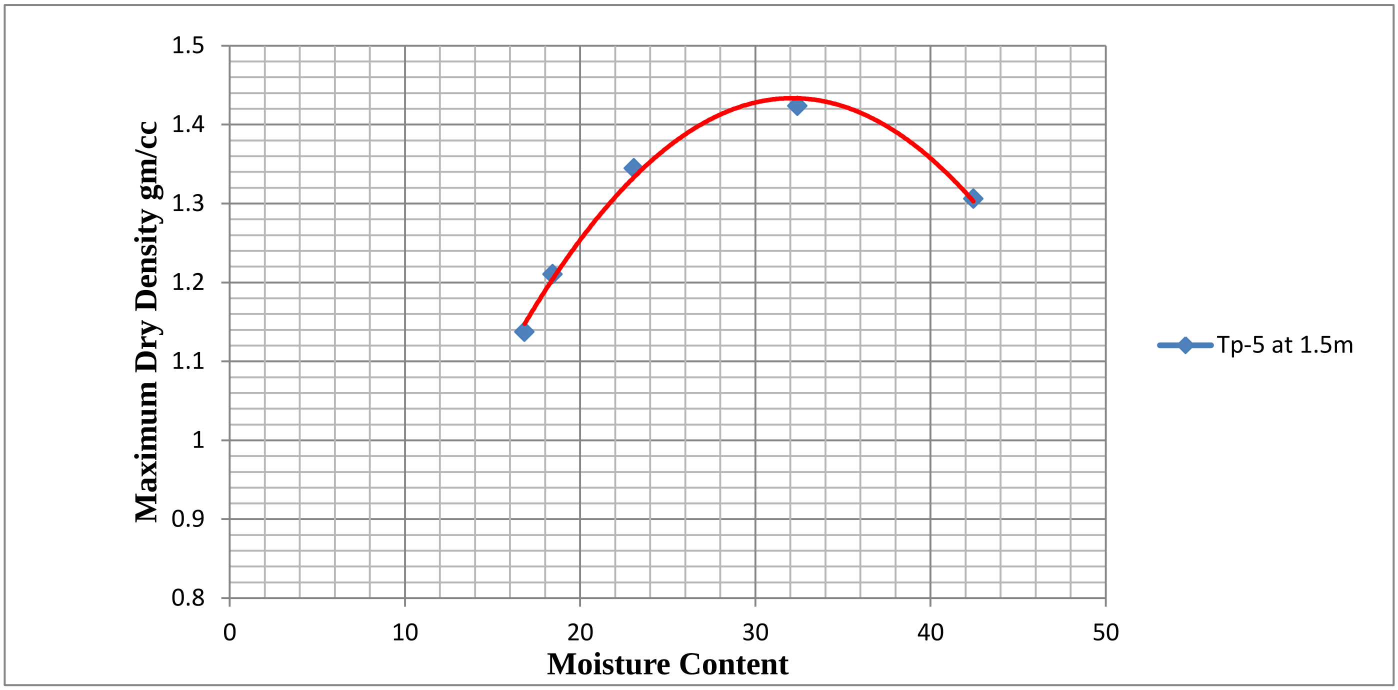


Figure -6 Standard Compaction Curve Dry density Vs Moisture Tp-3 at 1.5m



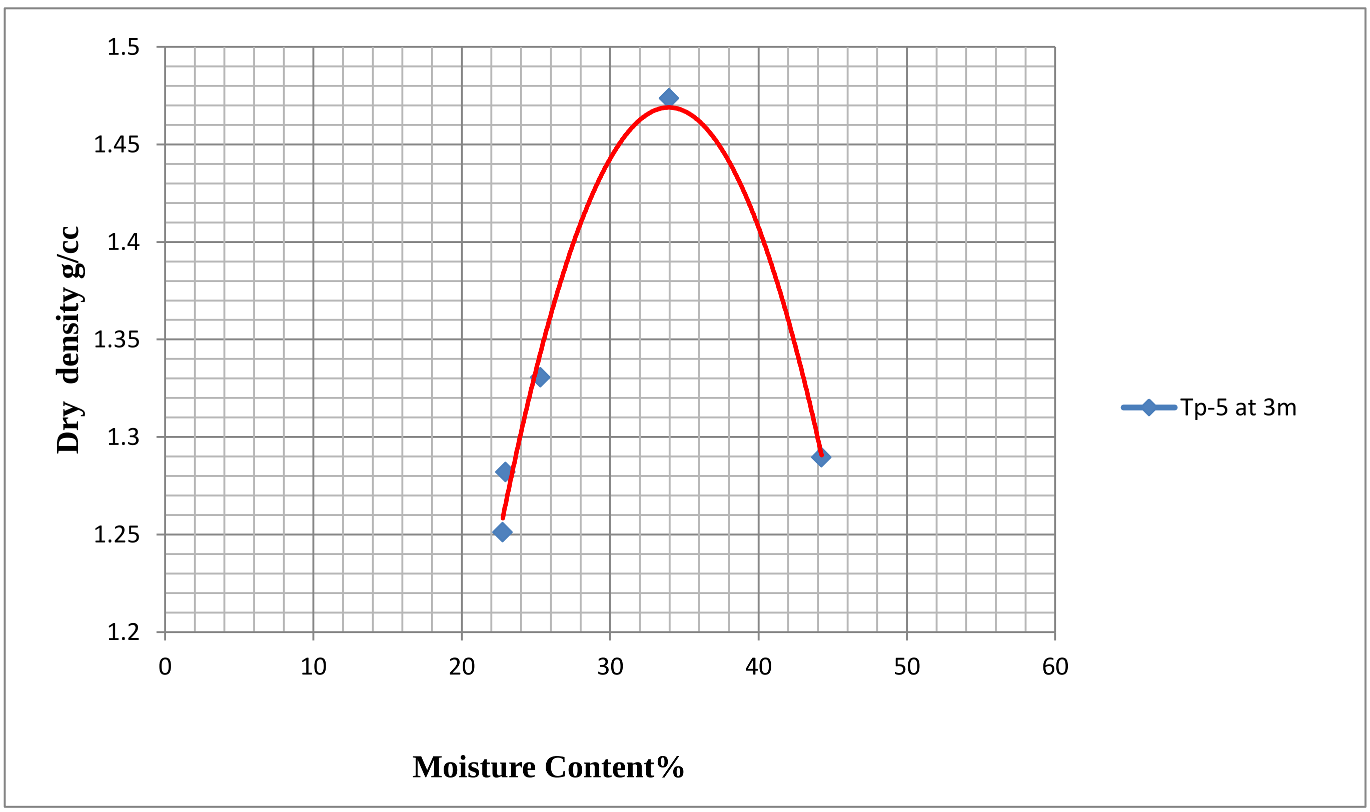
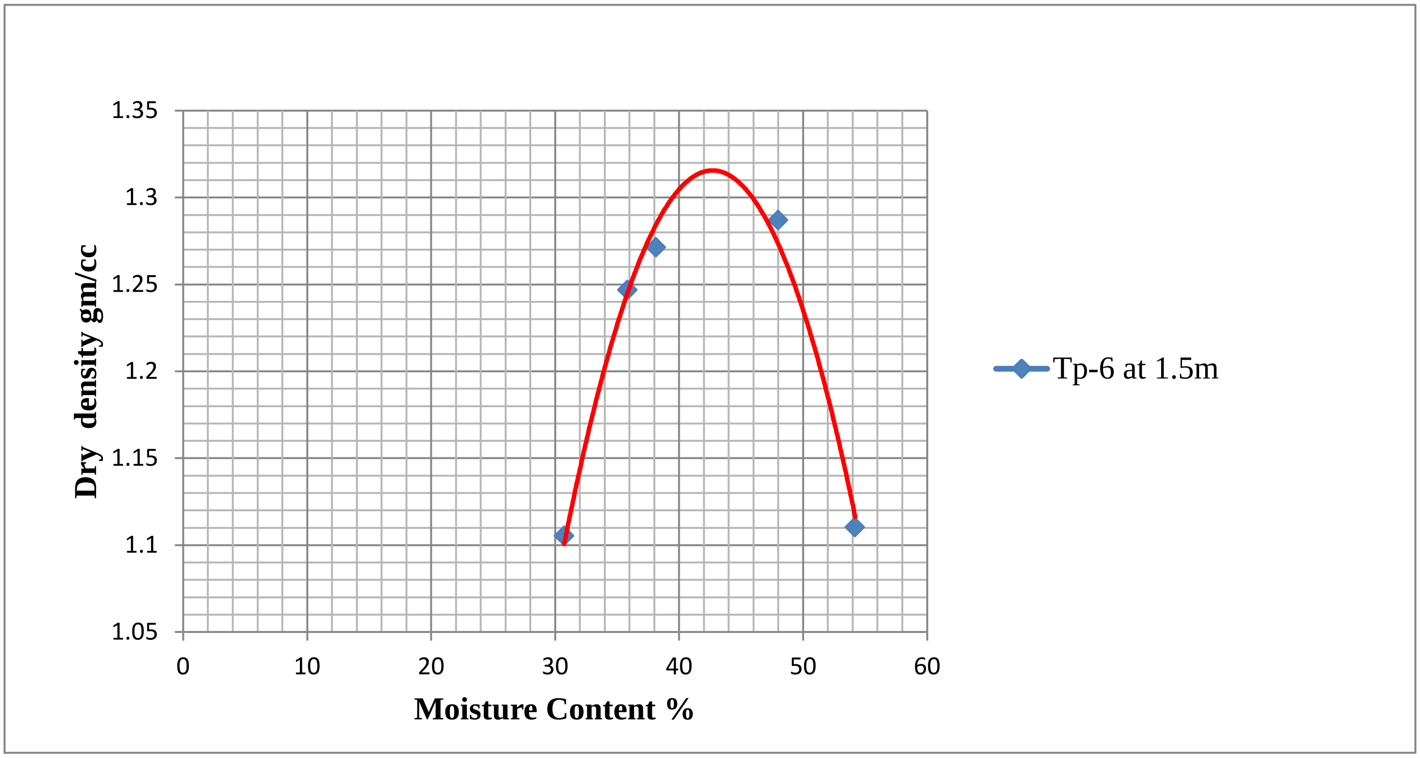


Figure -8 Standard Compaction Curve Dry density Vs Moisture Tp-5 at 3m



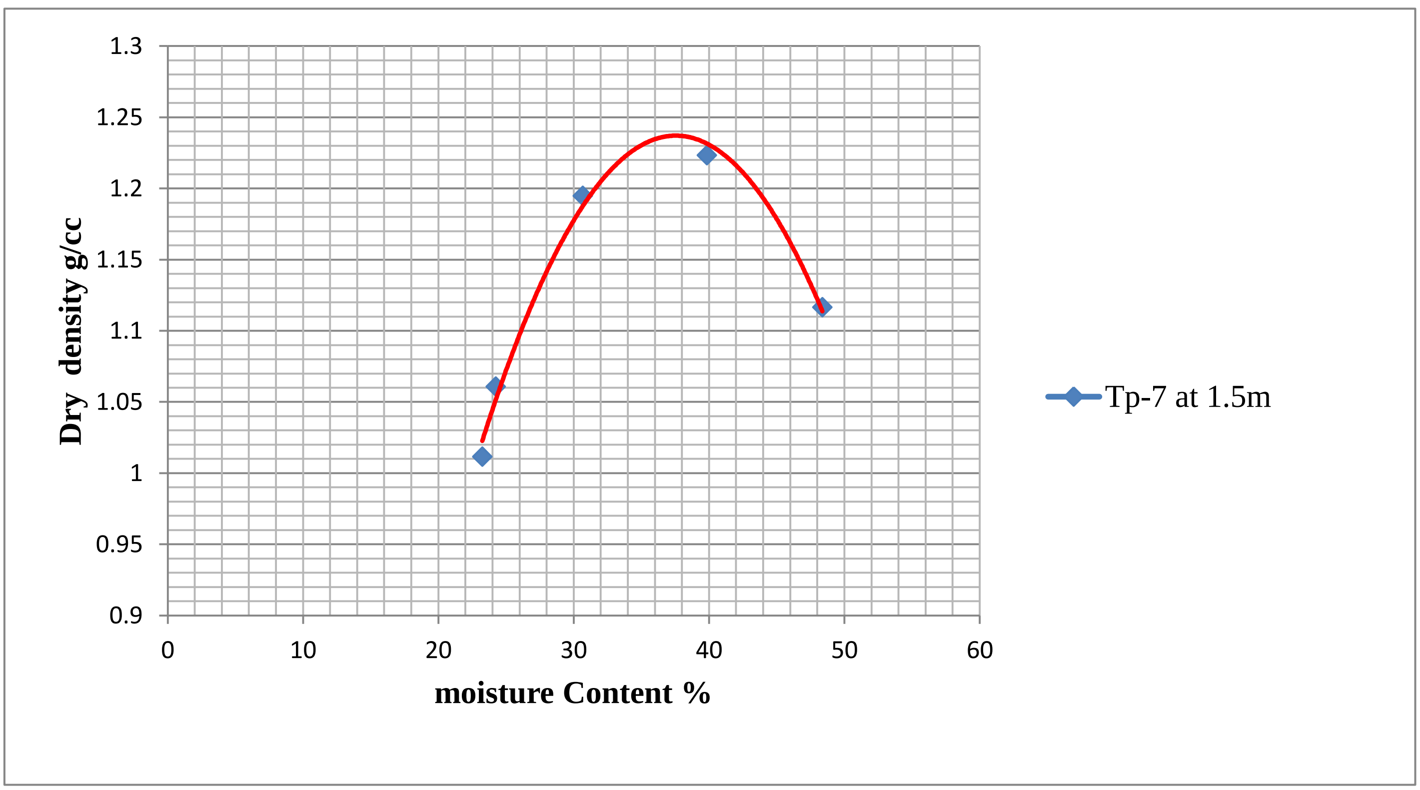
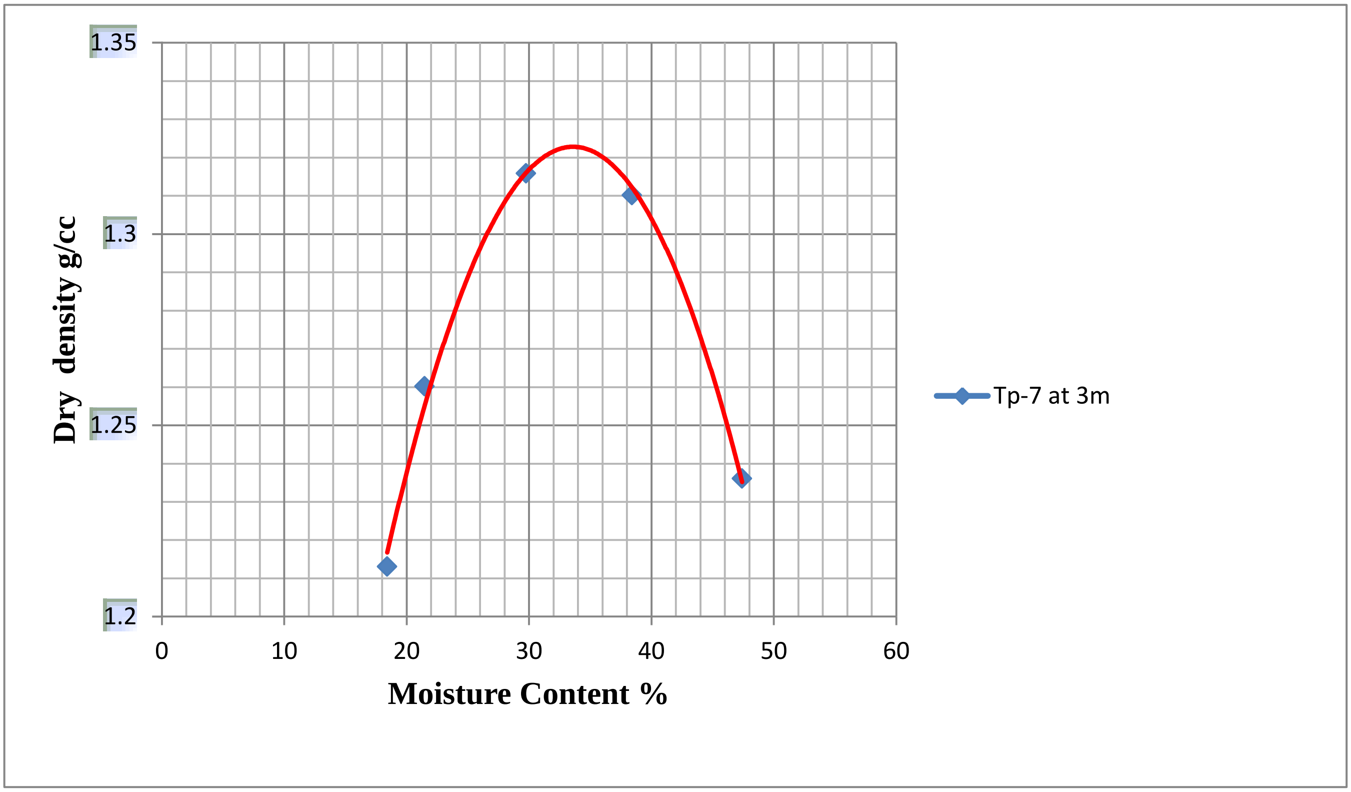


Figure -10 Standard Compaction Curve Dry density Vs Moisture Tp-7 at 1.5m



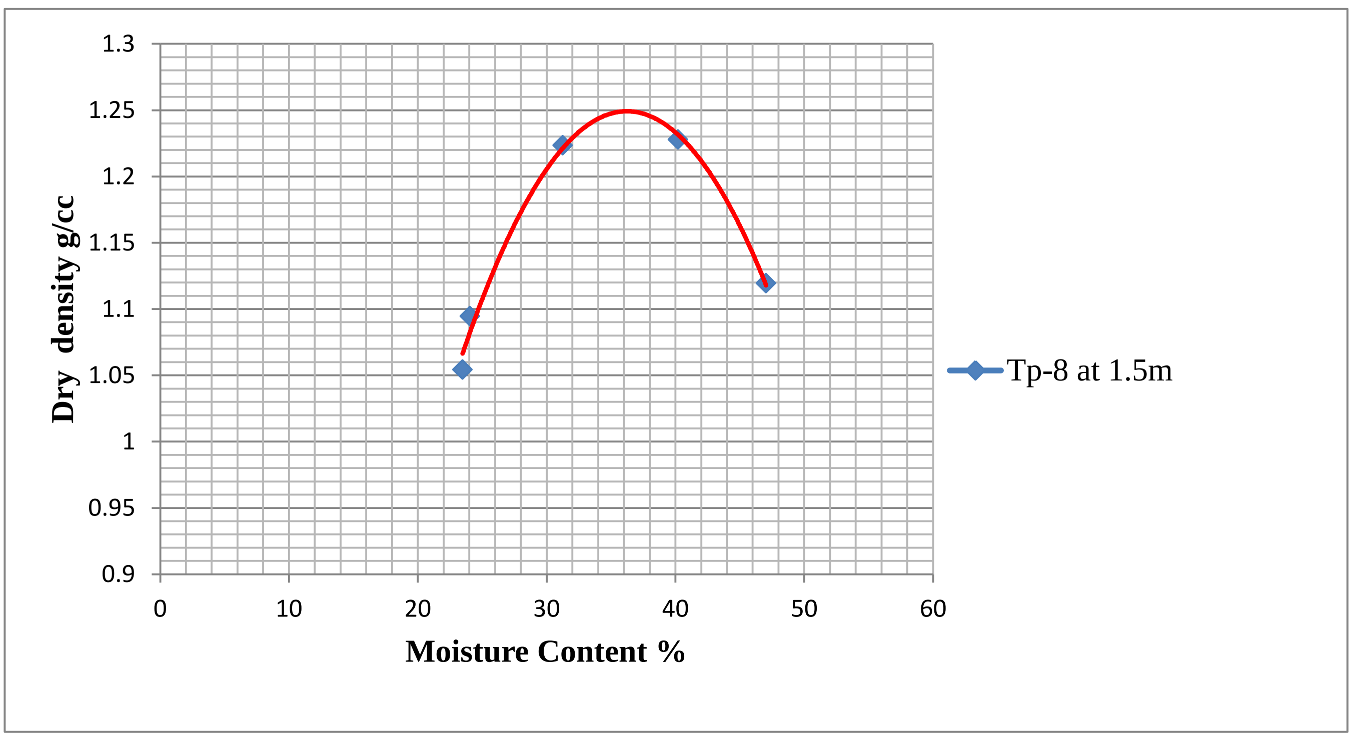
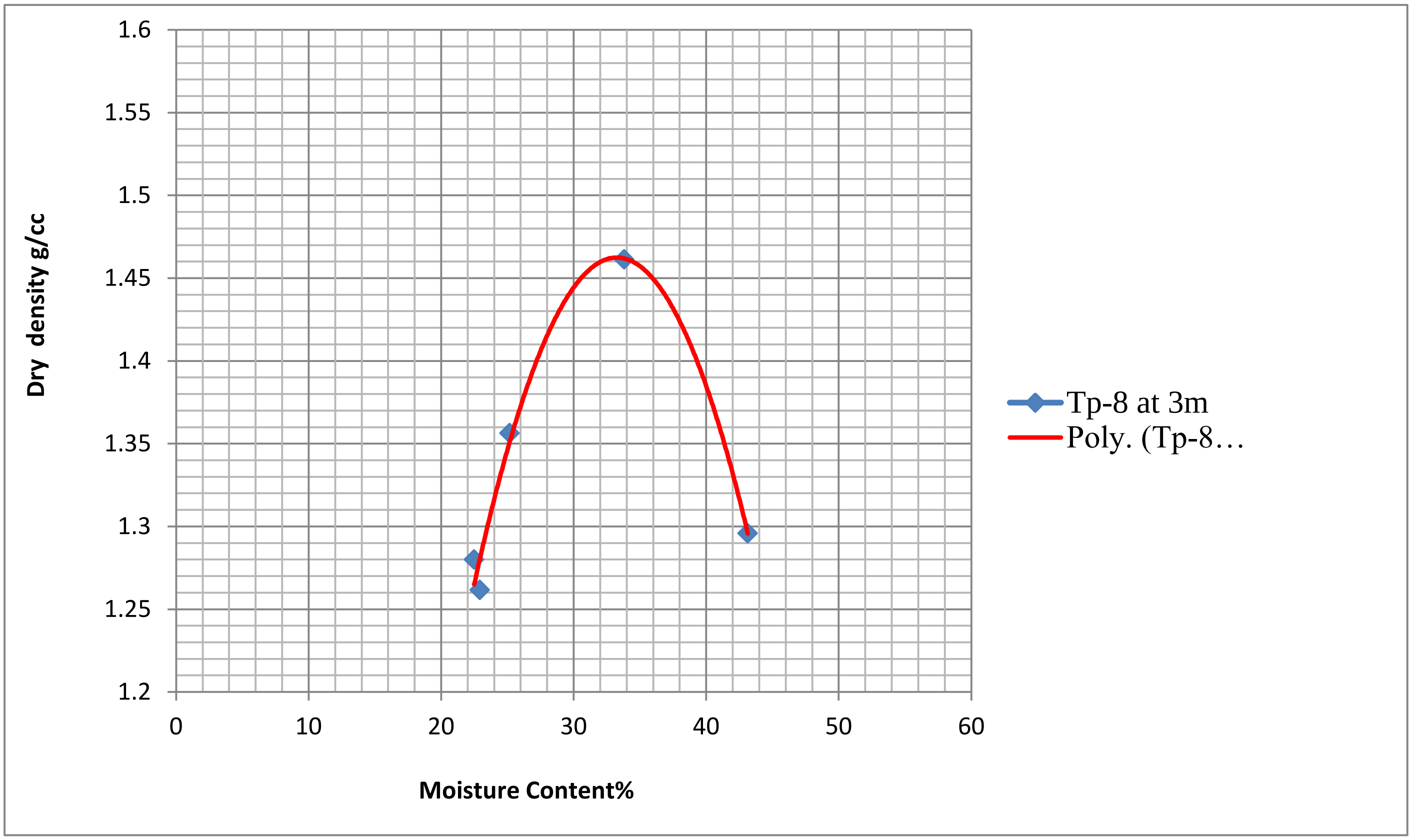


Figure -12 Standard Compaction Curve Dry density Vs Moisture Tp-8 at 1.5m



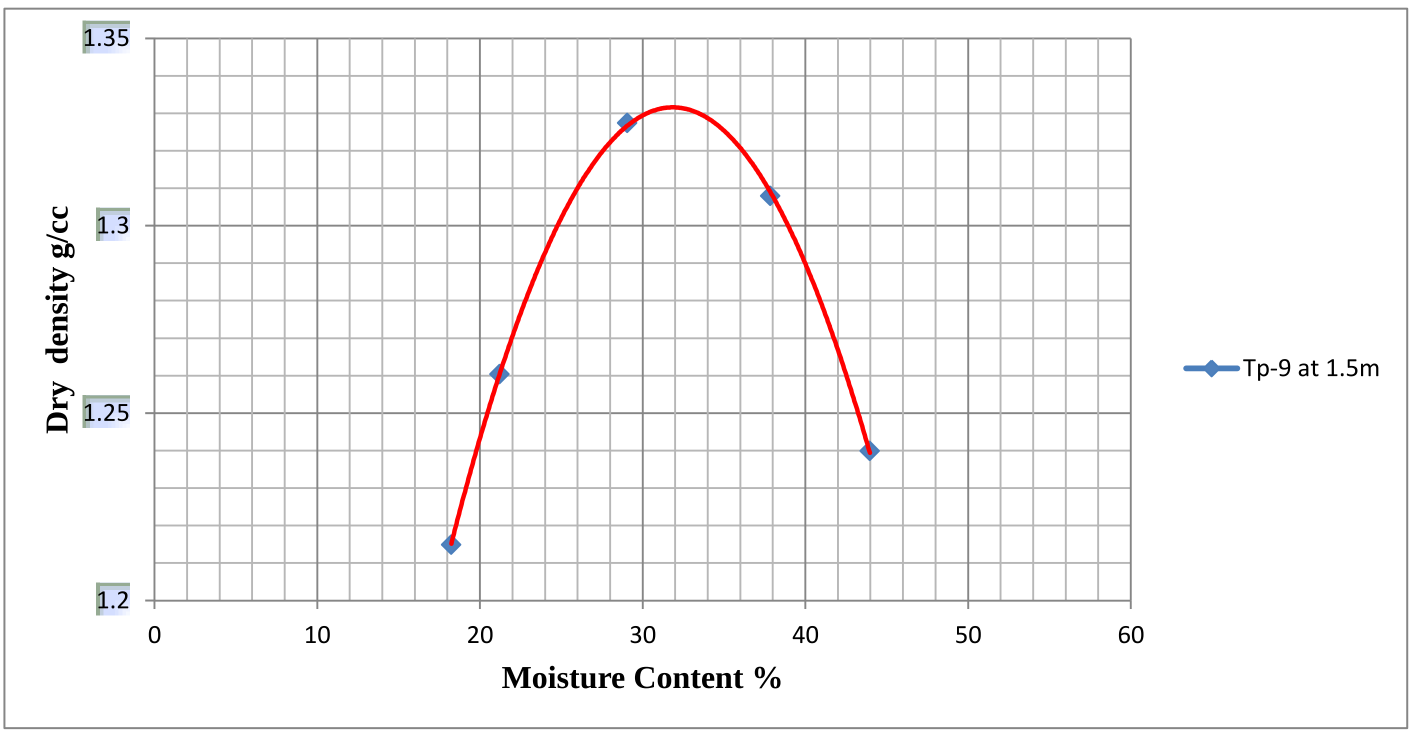


Figure -14 Standard Compaction Curve Dry density Vs Moisture Tp-9 at 1.5m

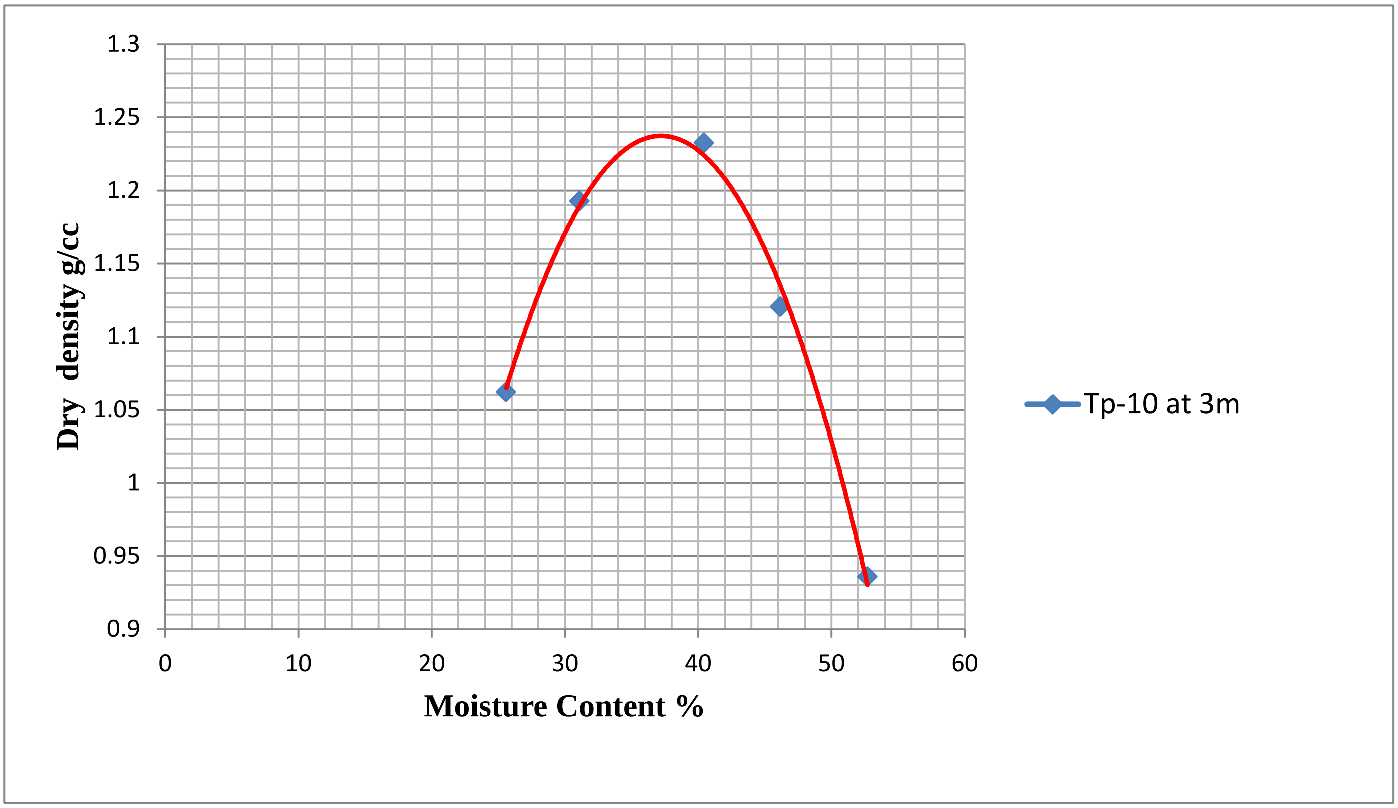


Figure -15 Standard Compaction Curve Dry density Vs Moisture Tp-10 at 1.5m

Appendix E: One-Dimensional Consolidation Test Result

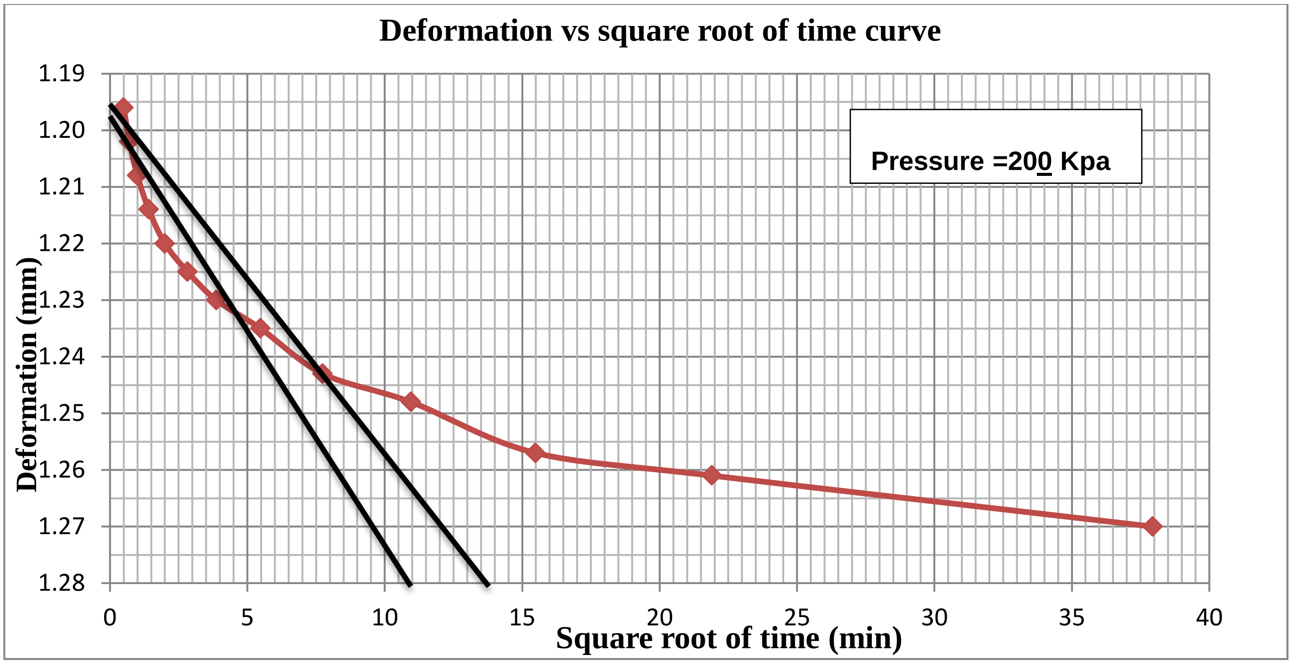


Figure E -1 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 1 (P=200KPa)

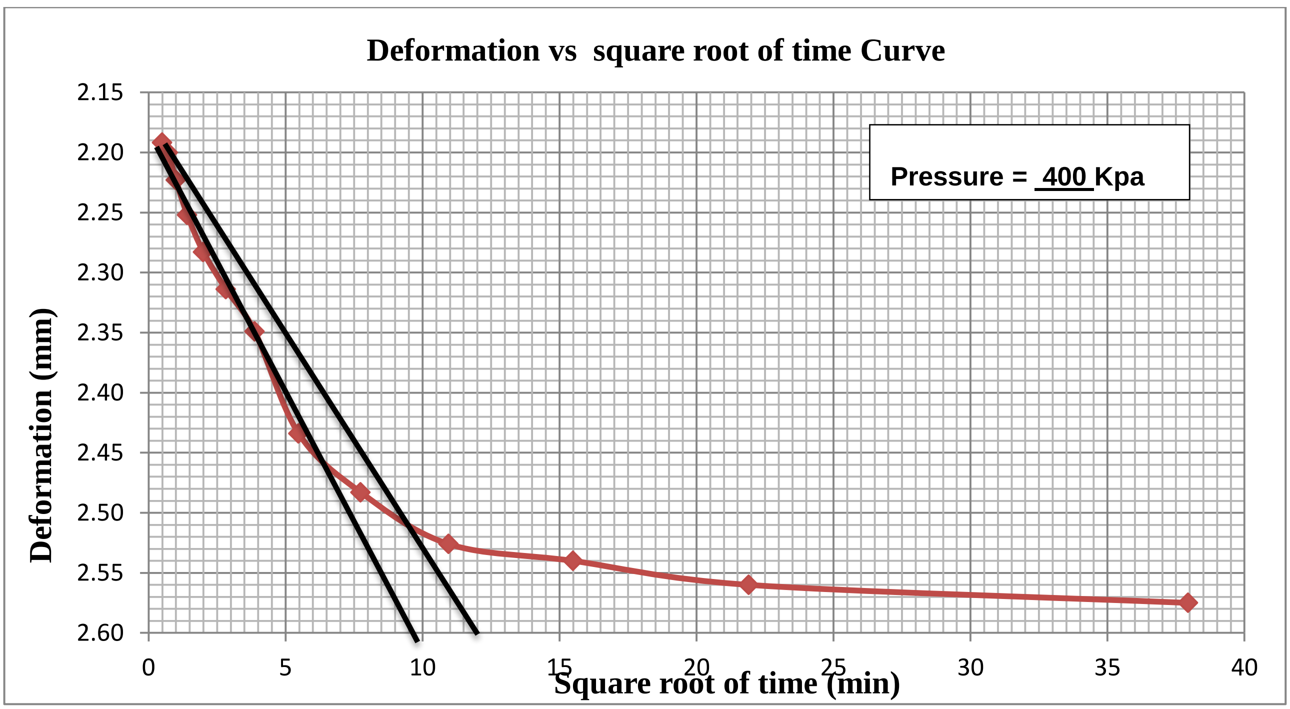


Figure E -2 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 1 (P=400KPa),

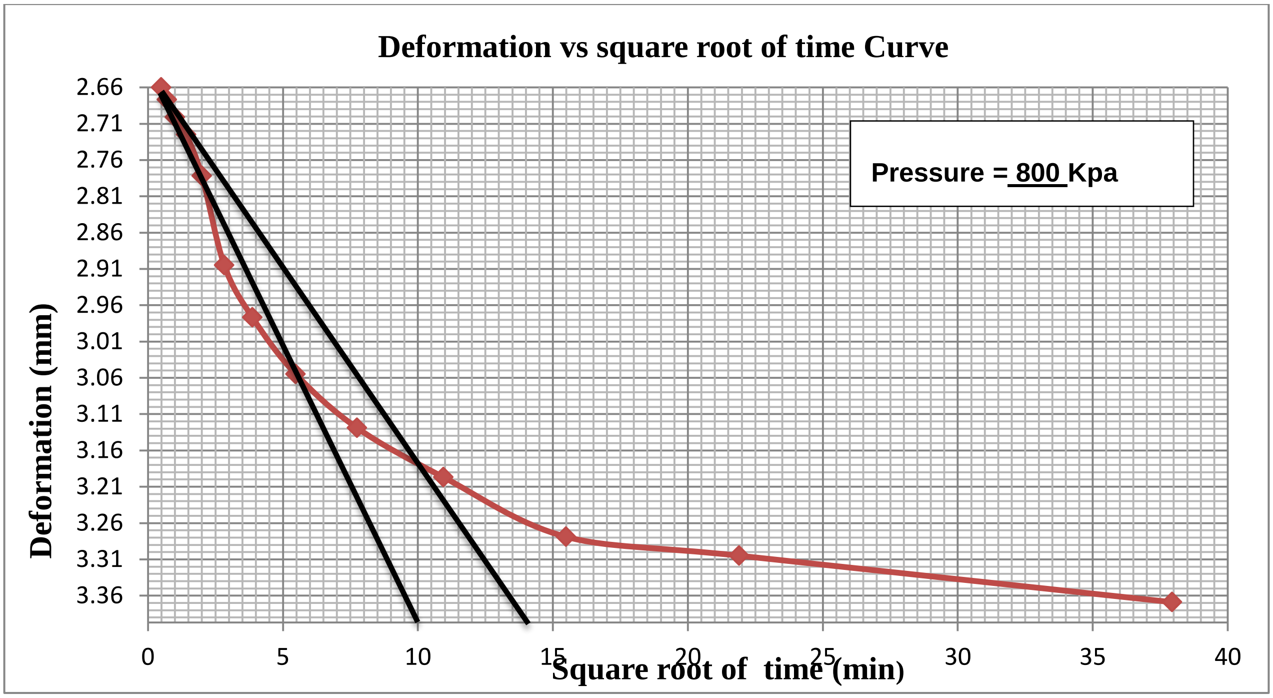


Figure E -3 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 1 (P=800KPa)

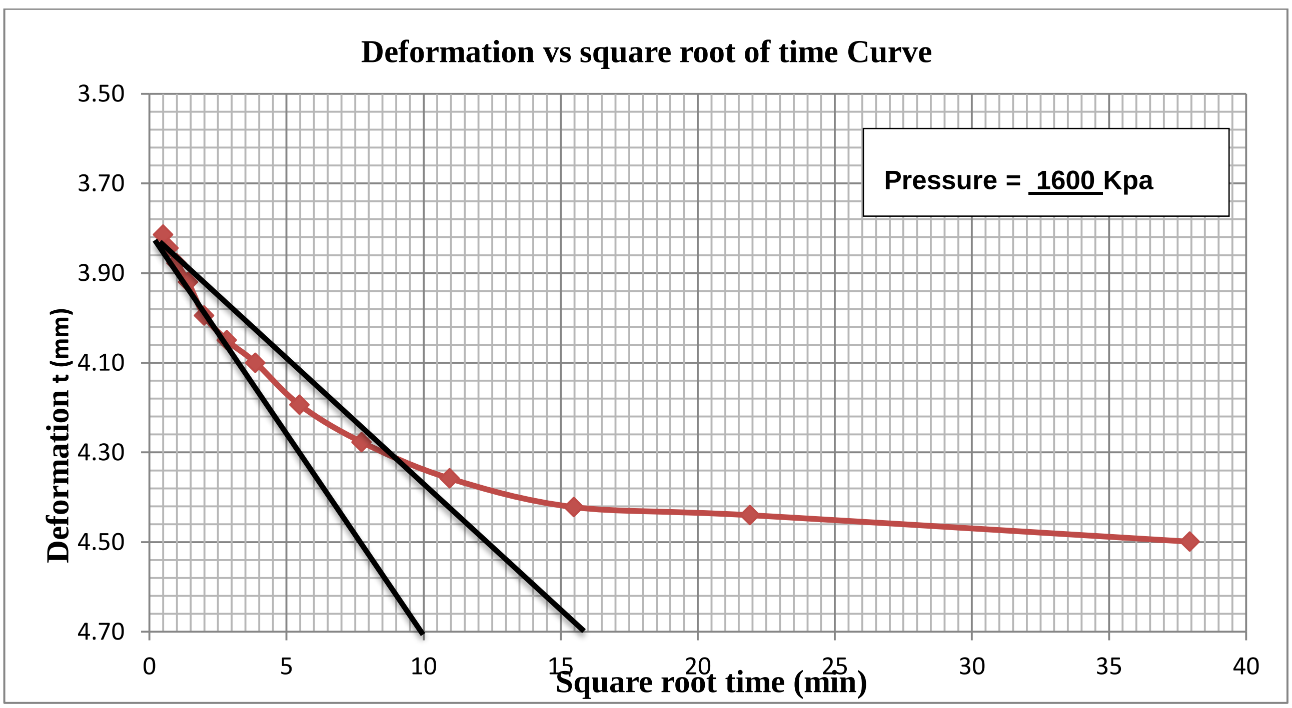


Figure E -4 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 1 (P=1600KPa)

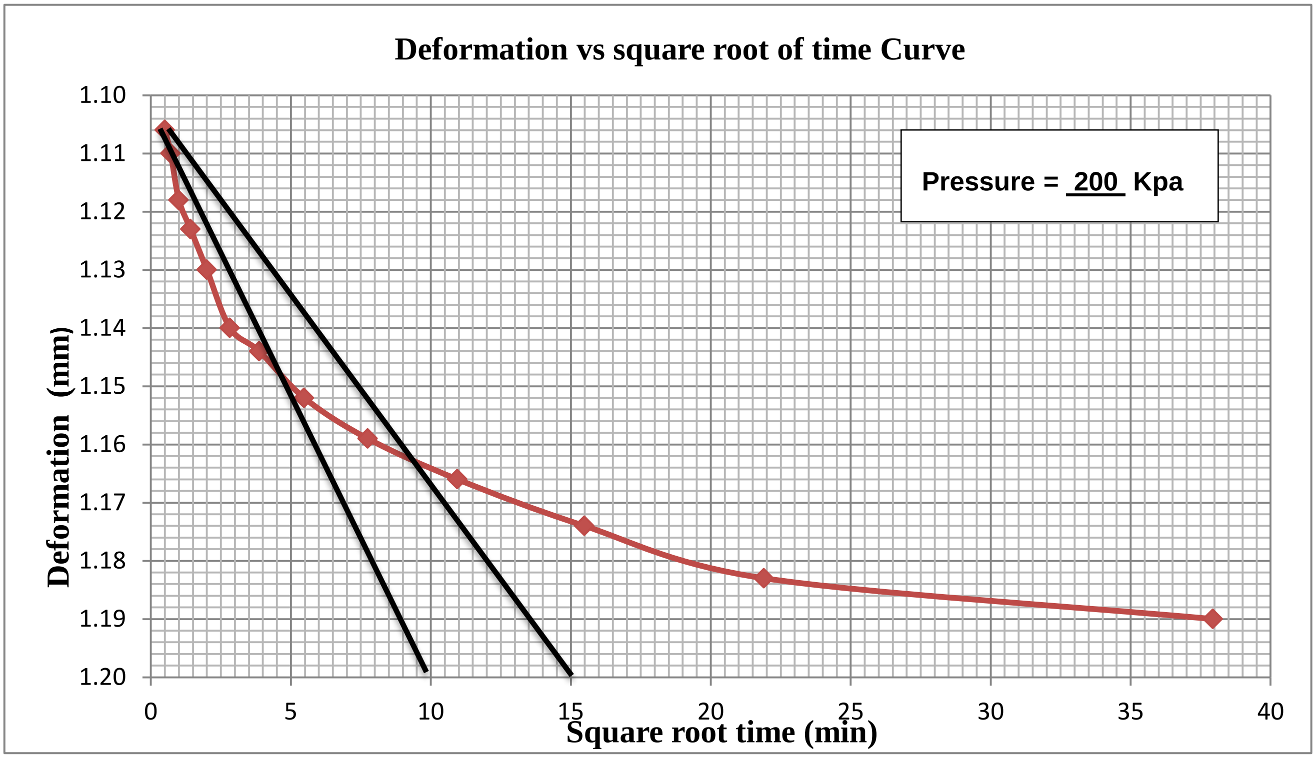


Figure E -5 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 2 (P=200KPa)

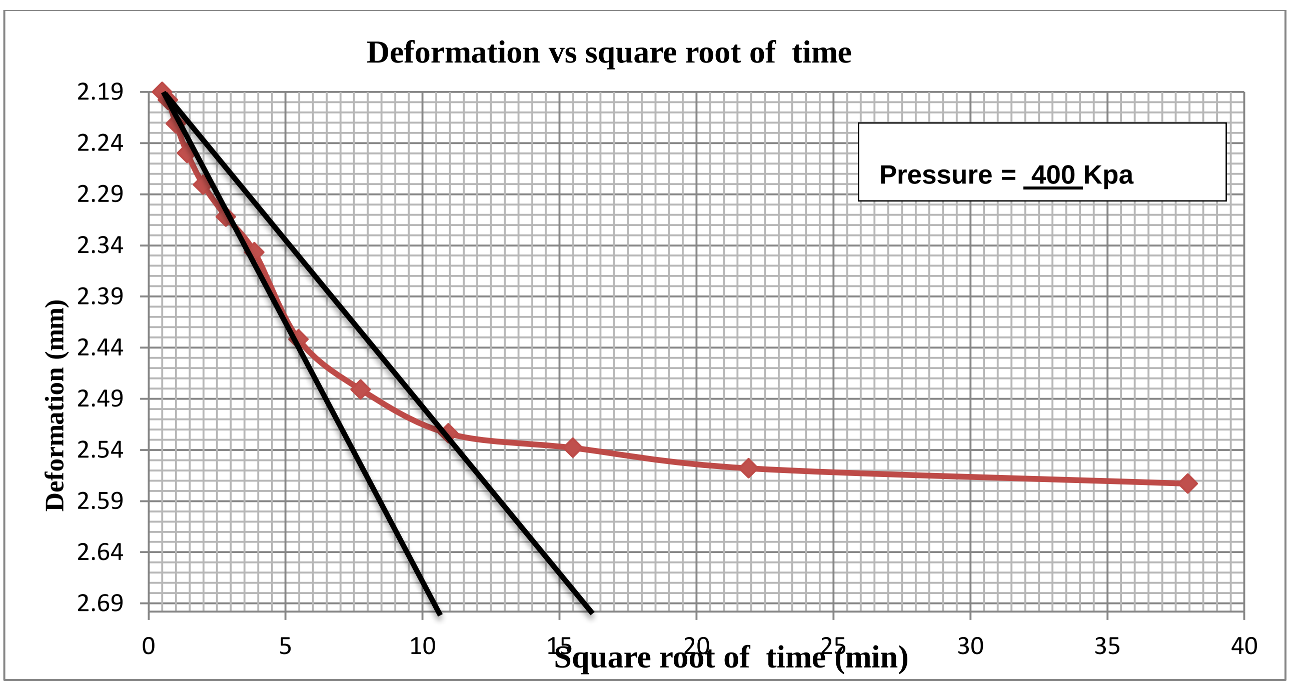


Figure E -6 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 2 (P=400KPa)

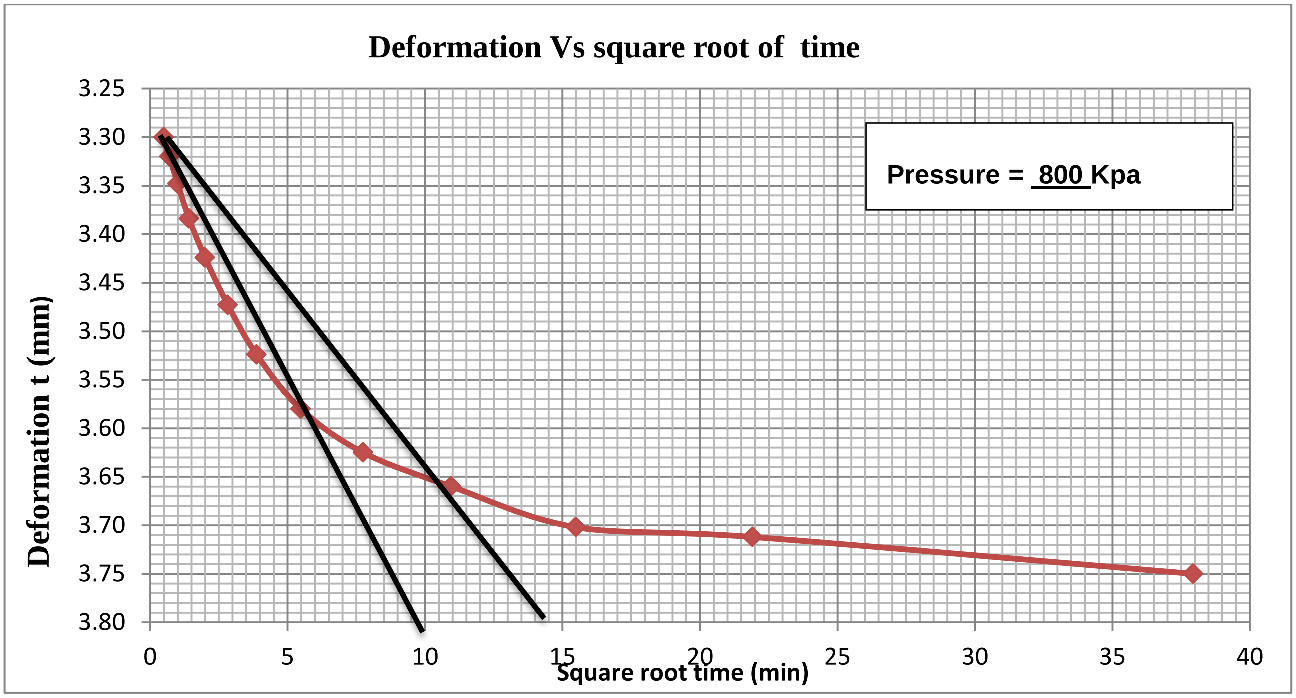


Figure E -7 Typical plots of Deformation t versus Square root of time for remolded expansive soil sample of Mojo town, Sample 2 (P=800KPa)

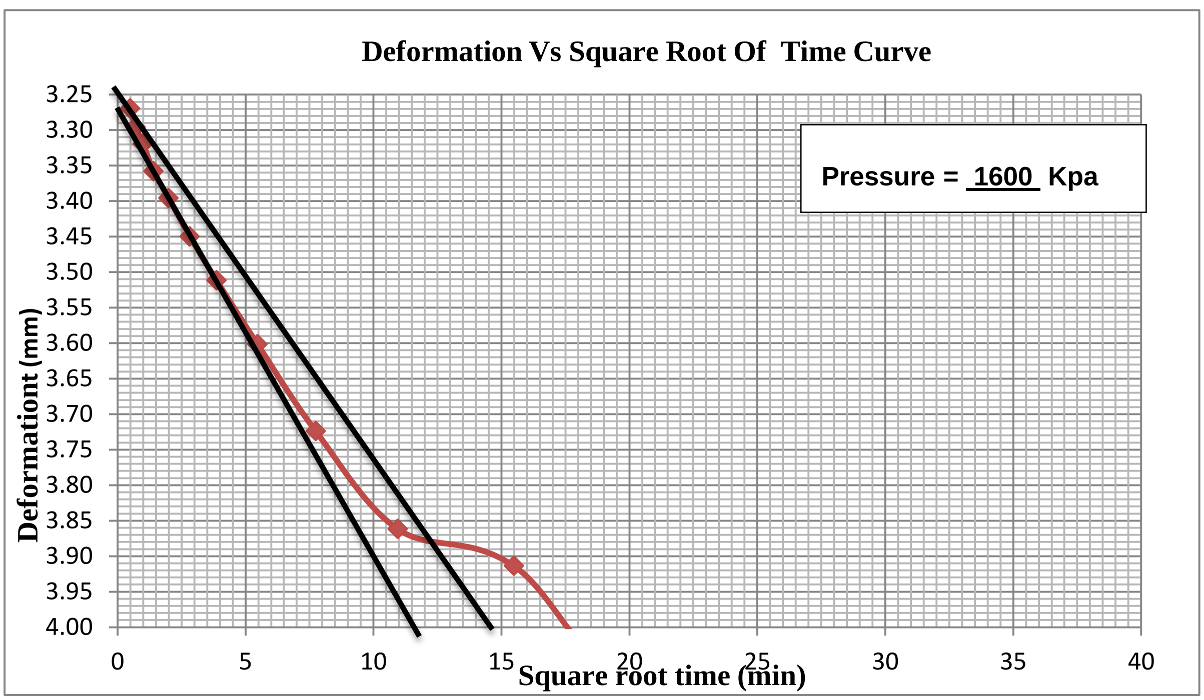


Figure E -8 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 2 (P=1600KPa)

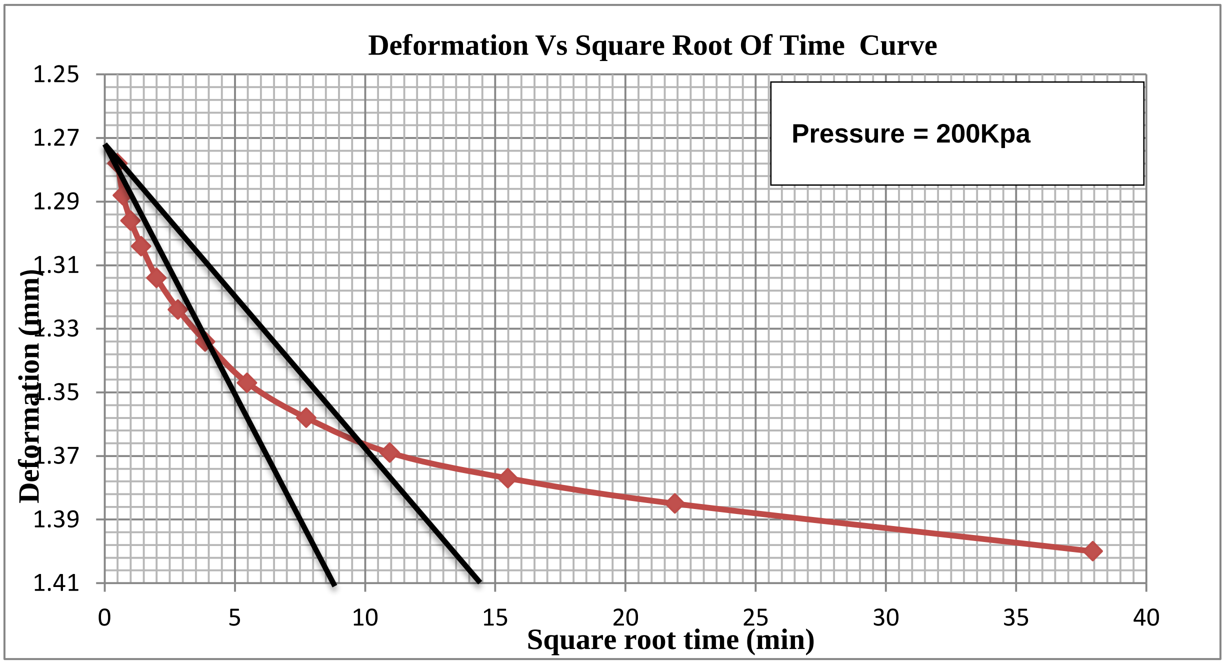


Figure E -9 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 3 (P=200KPa)

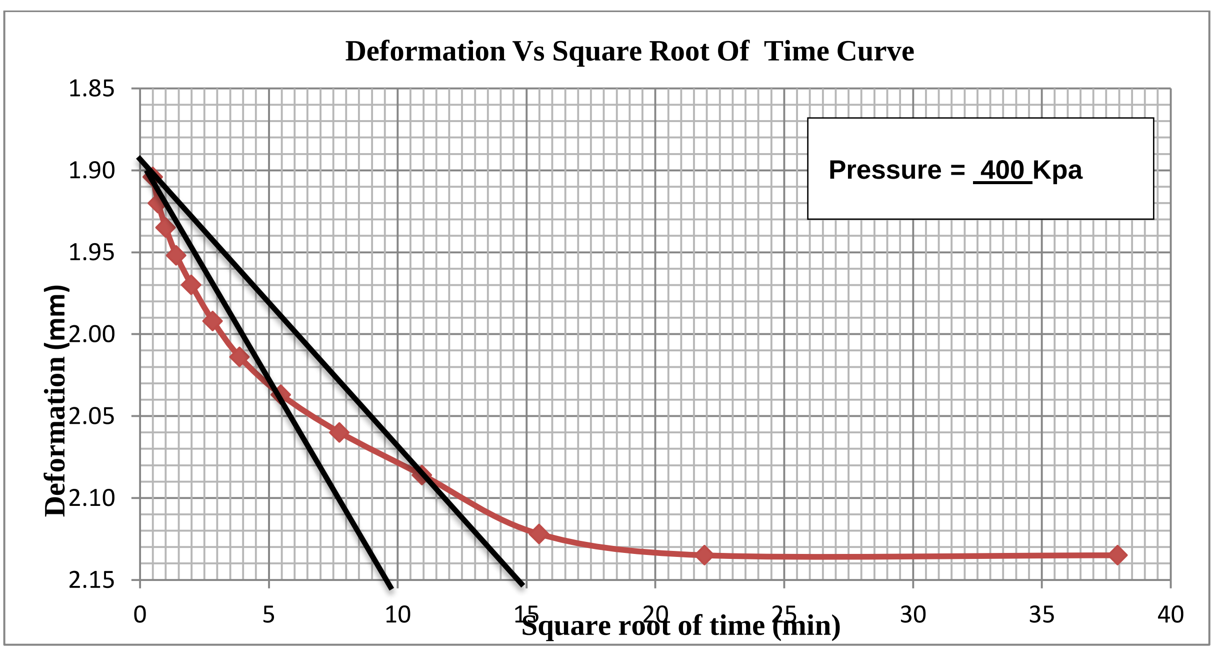


Figure E -10 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 3 (P=400KPa)

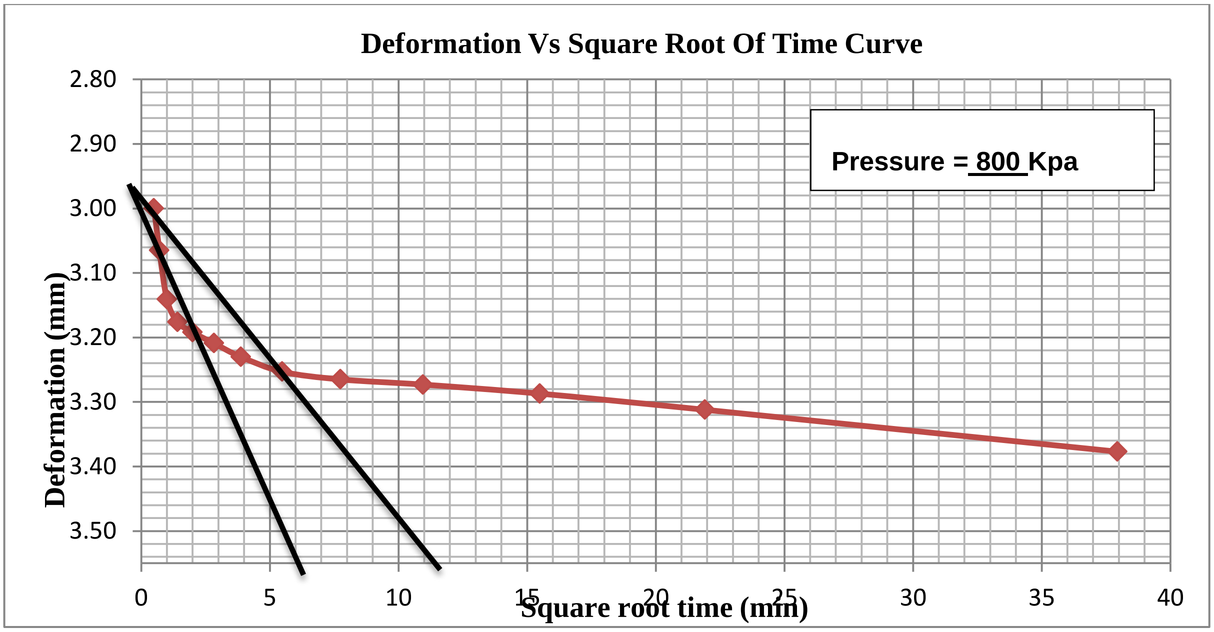


Figure E -11 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 3 (P=800KPa)

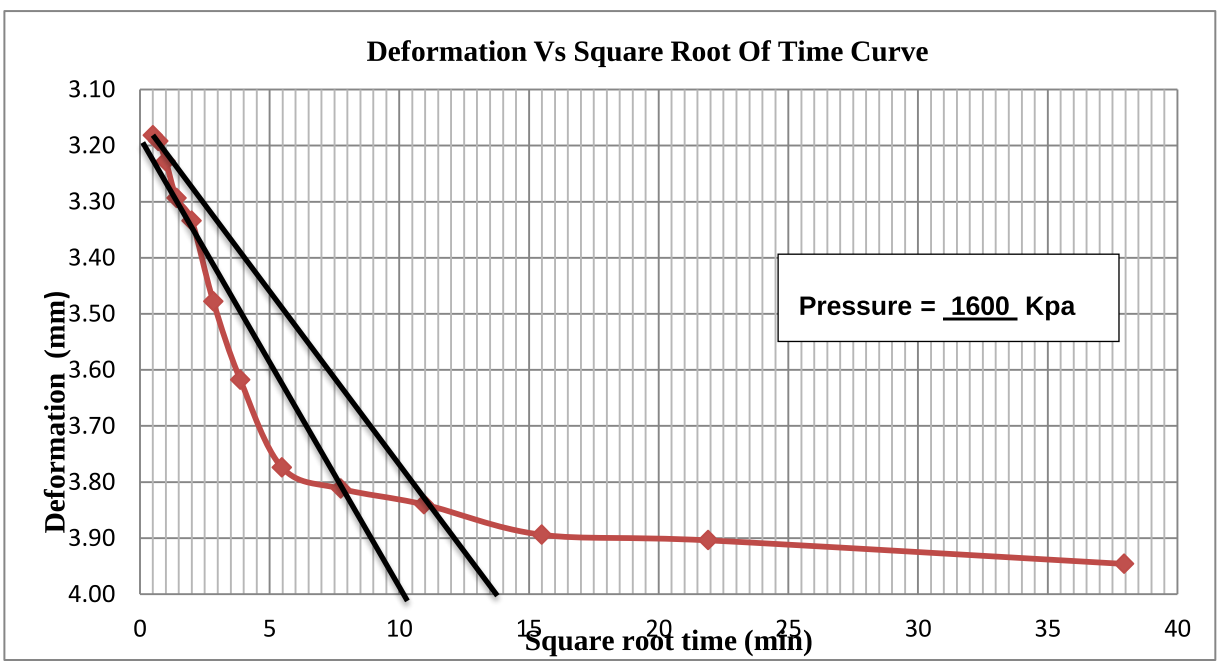


Figure E -12 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 3 (P=1600KPa)

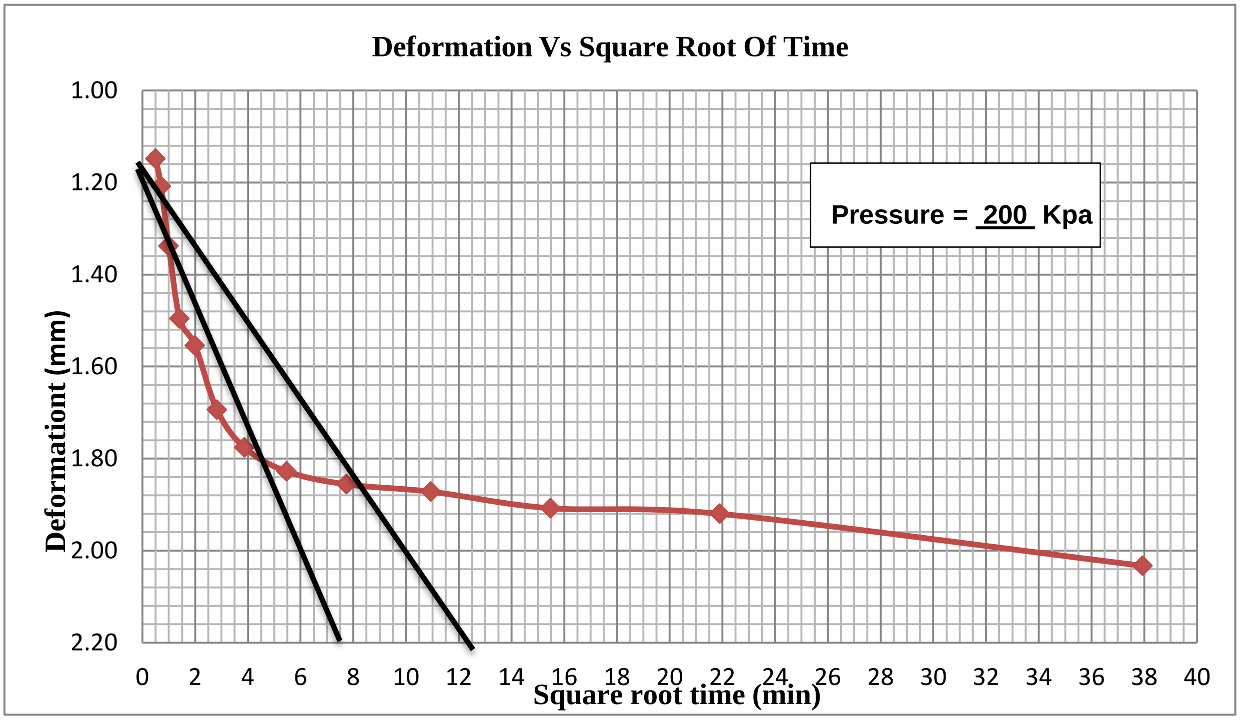


Figure E -13 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 4 (P=200KPa)

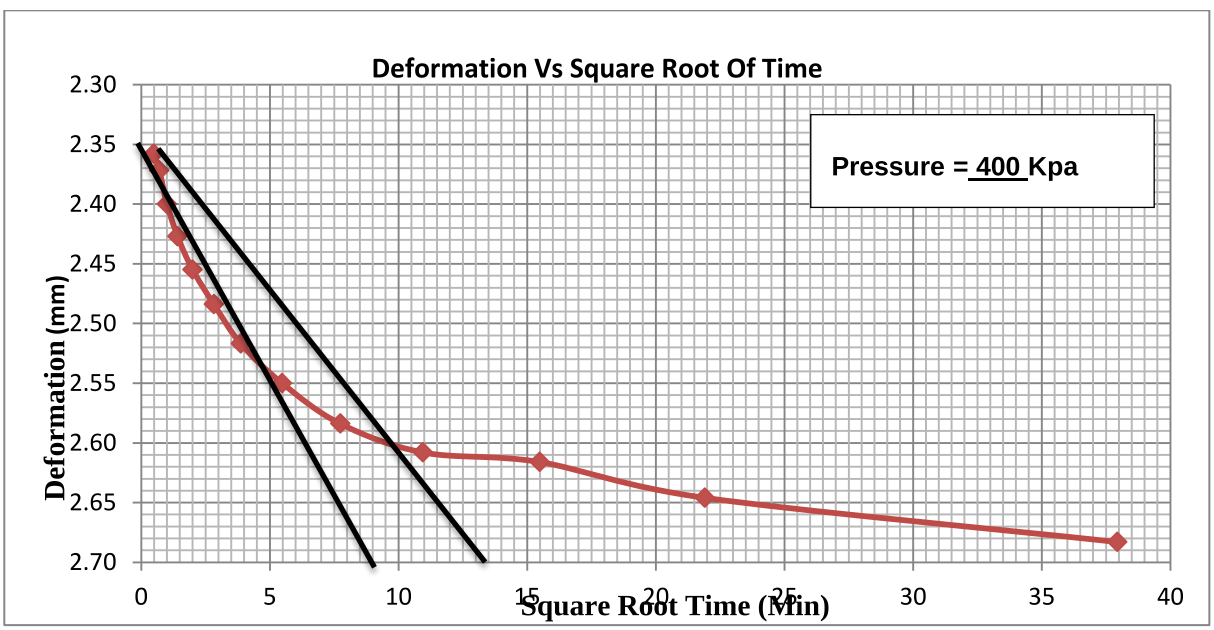


Figure E -14Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 4 (P=400KPa

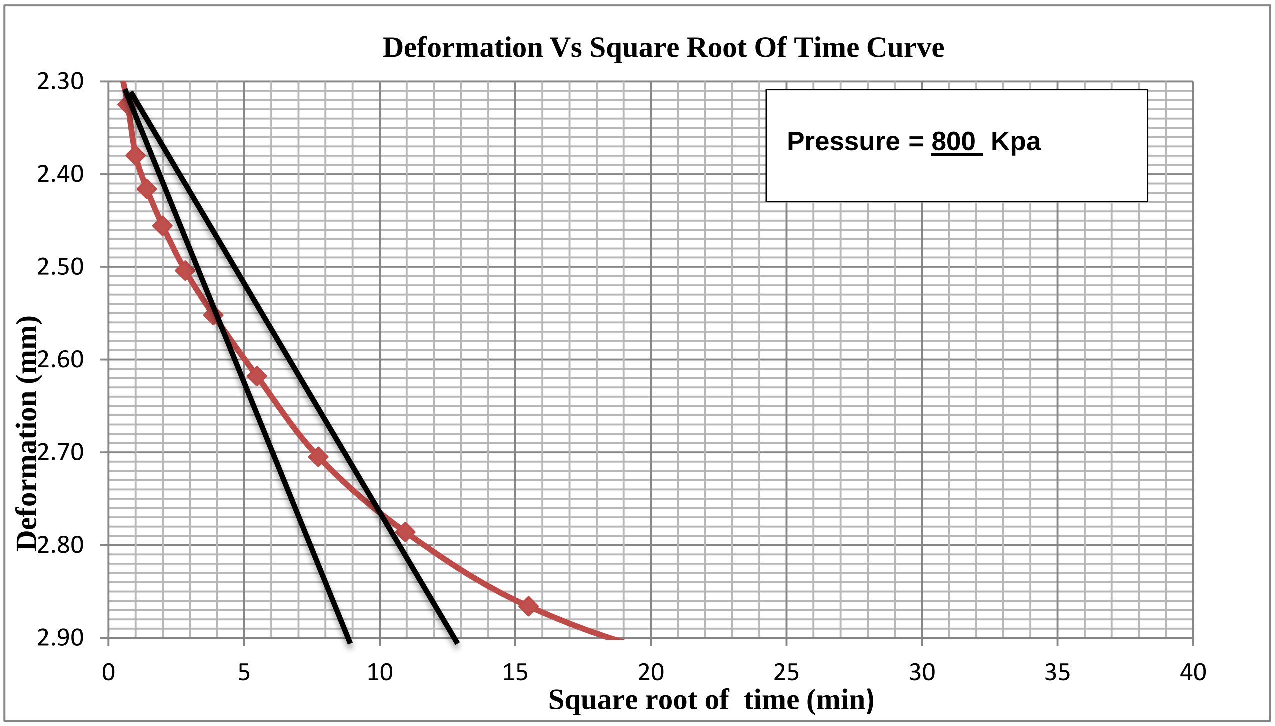


Figure E -15 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 4 (P=800KPa)

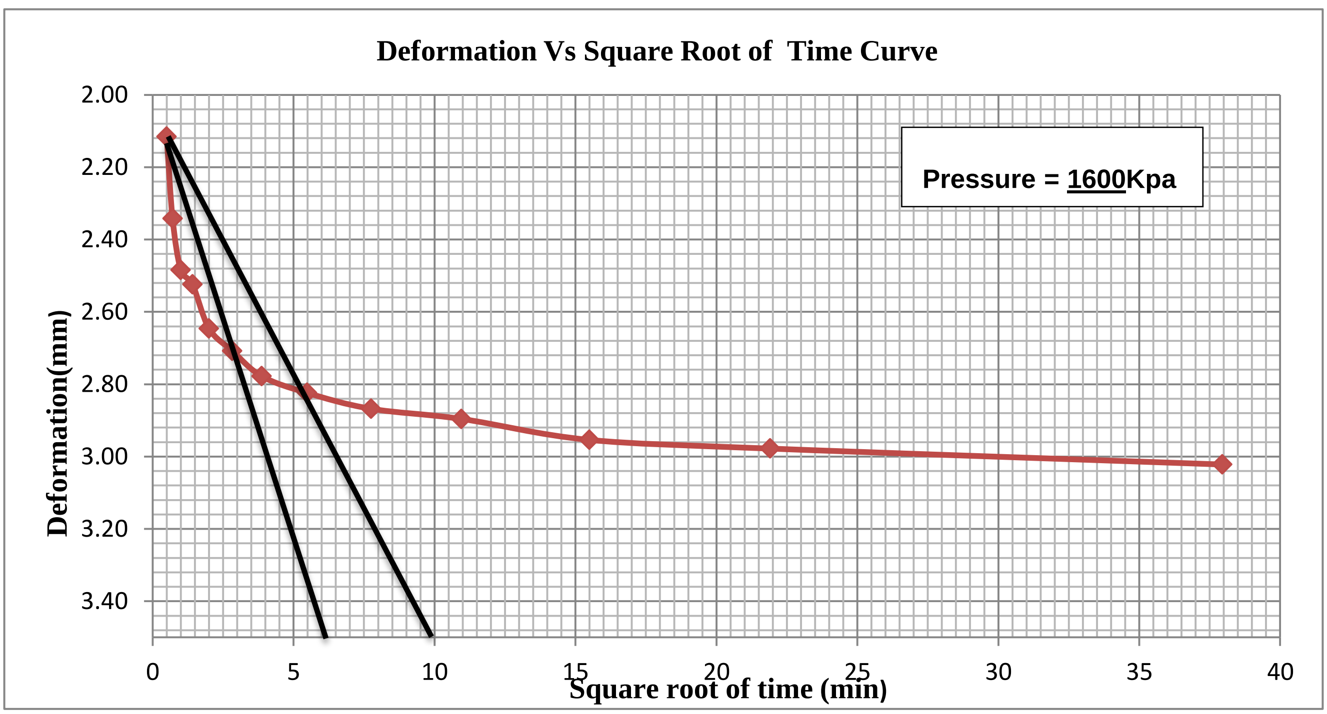


Figure E -16 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 4 (P=1600KPa)

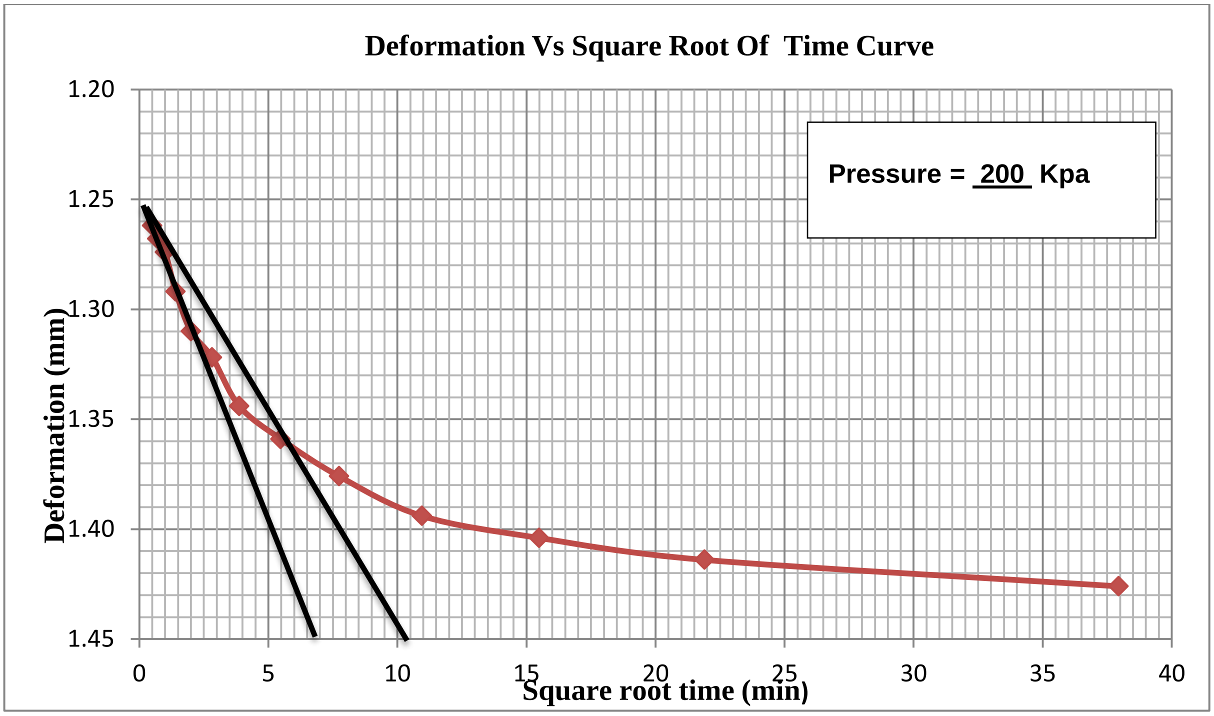


Figure E -17 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 5 (P=200KPa)

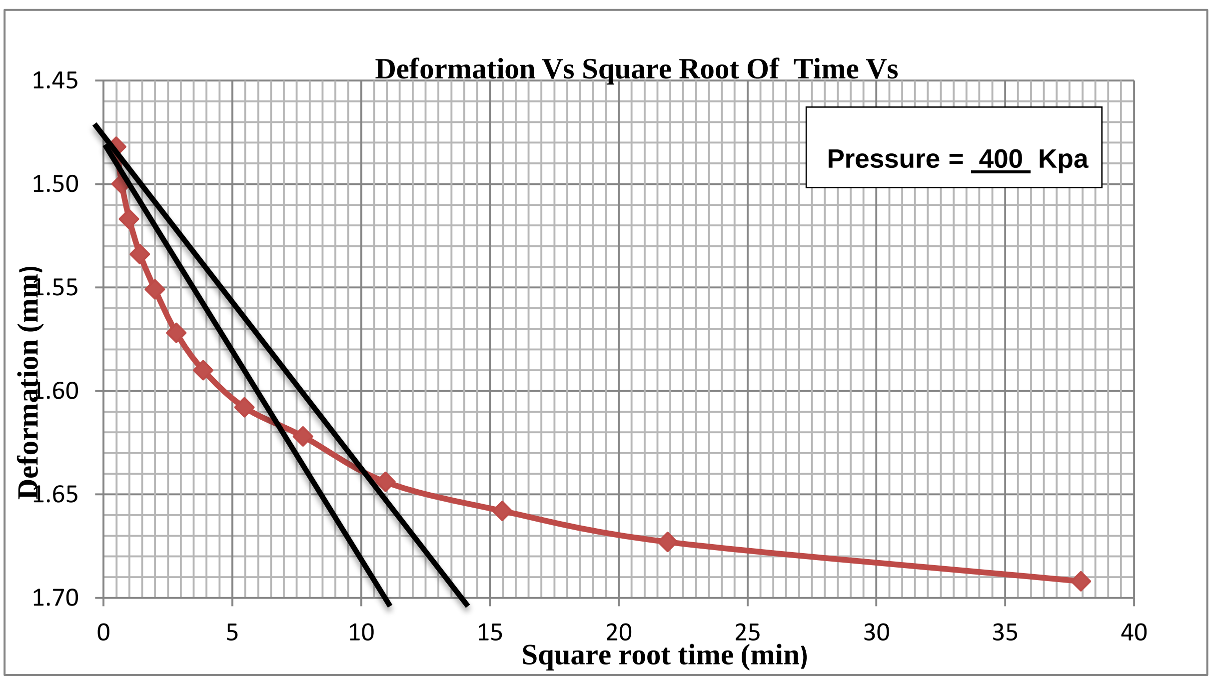


Figure E -18 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 5 (P=400KPa)

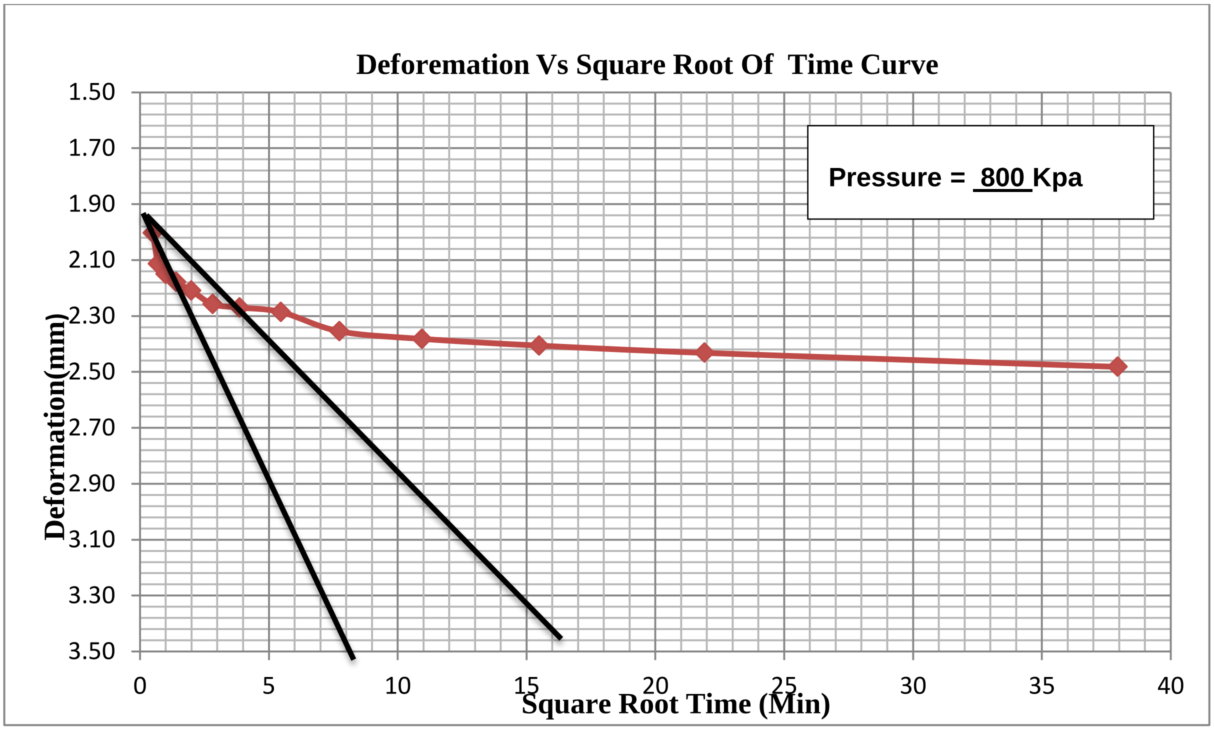


Figure E -19 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 5 (P=800KPa)

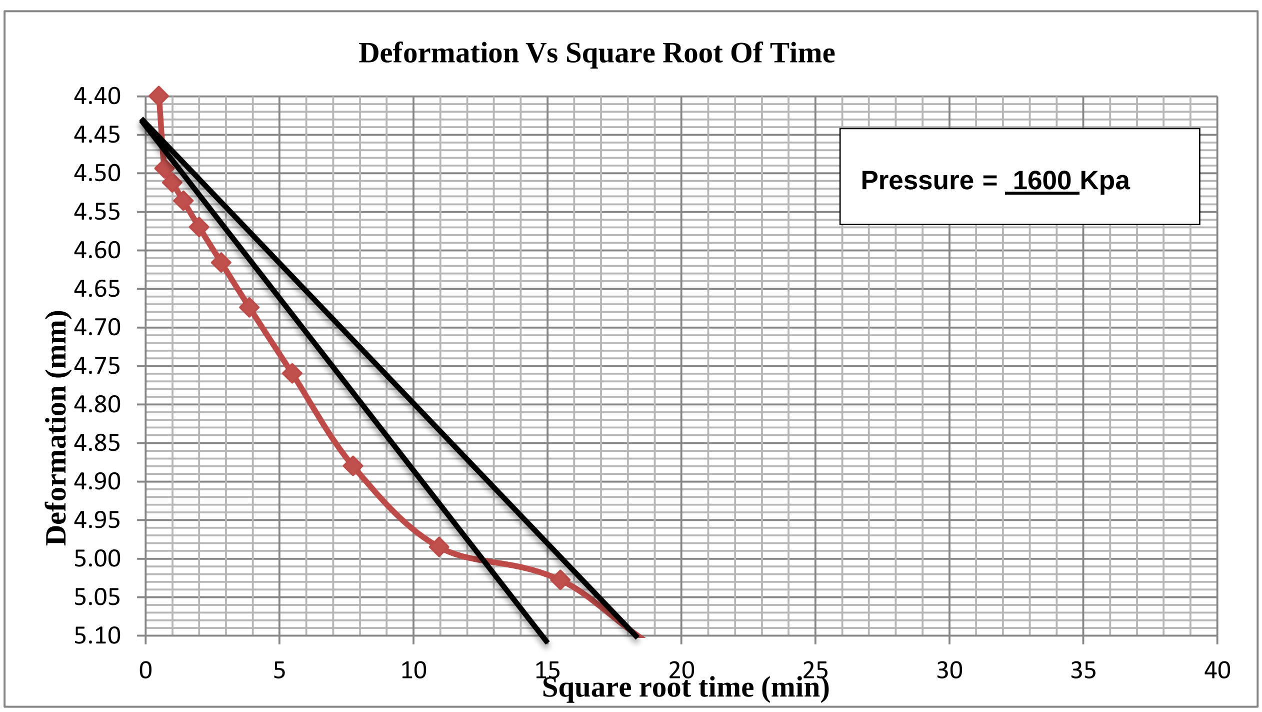


Figure E -20 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 5 (P=1600KPa)

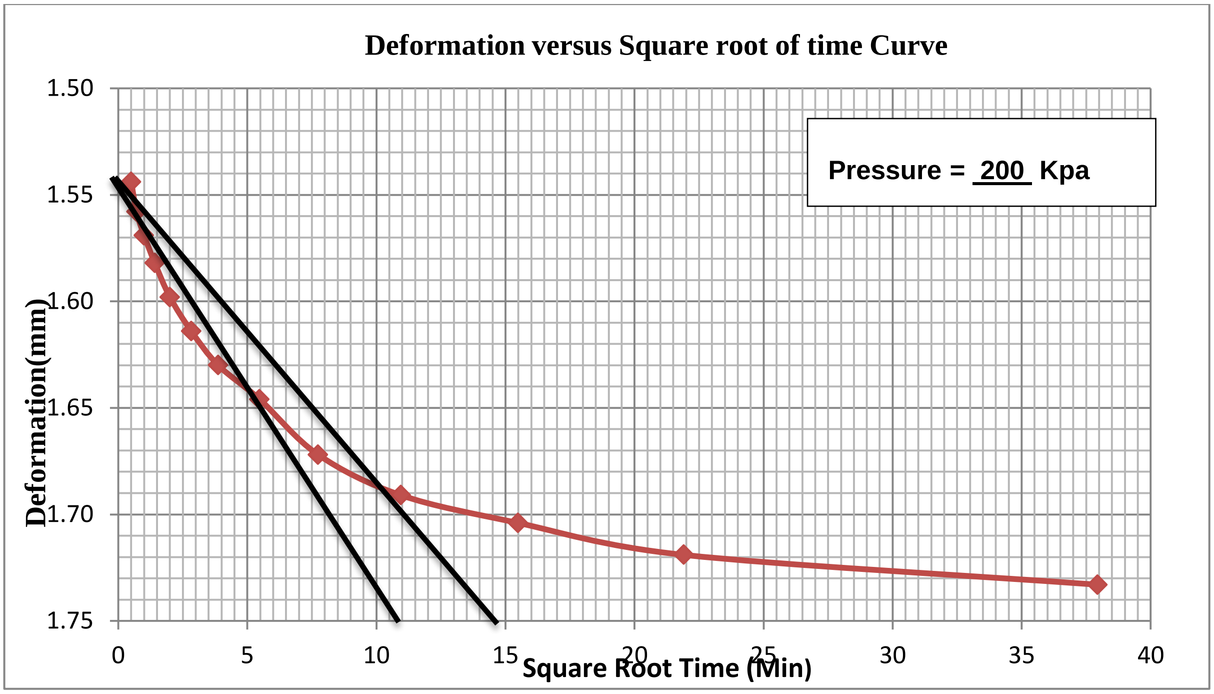


Figure E -21 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 6 (P=200KPa)

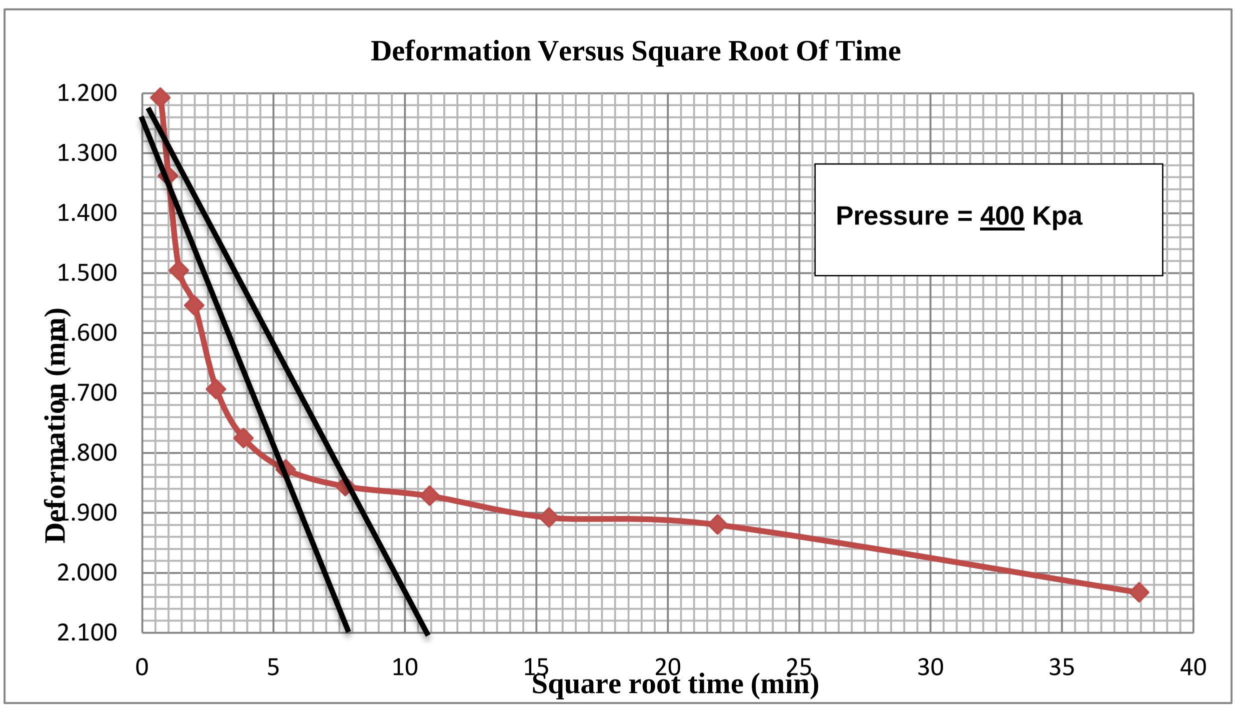


Figure E -22 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 6 (P=400KPa)

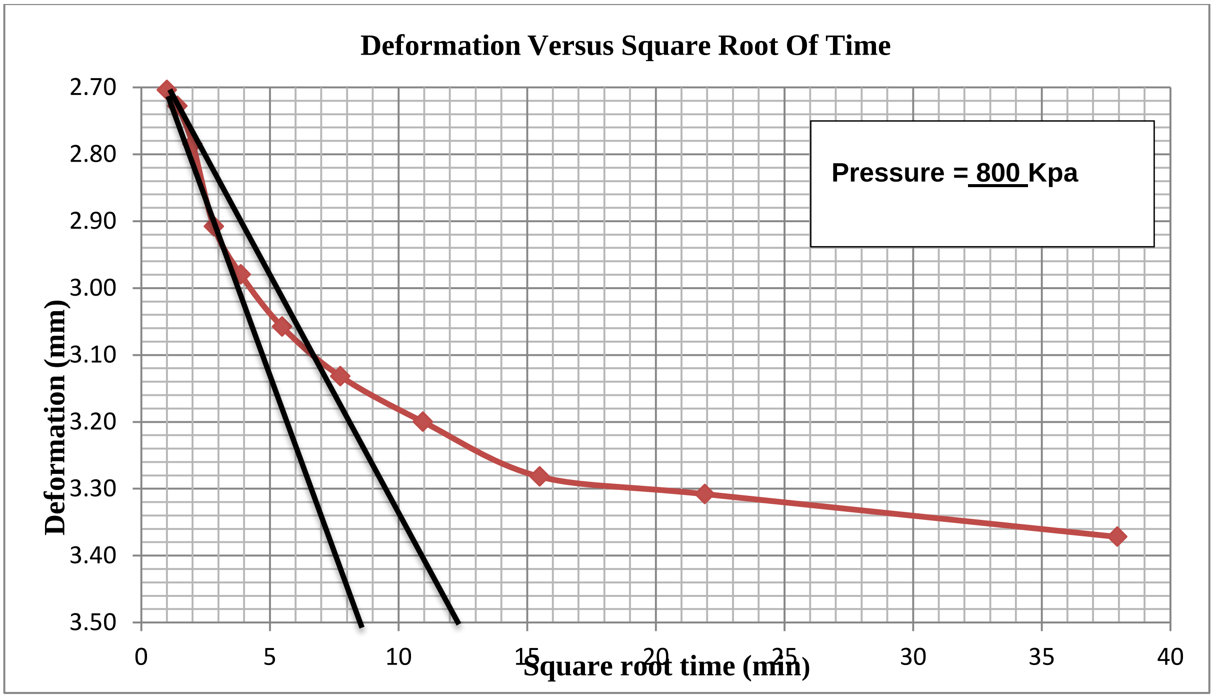


Figure E -23 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 6 (P=800KPa)

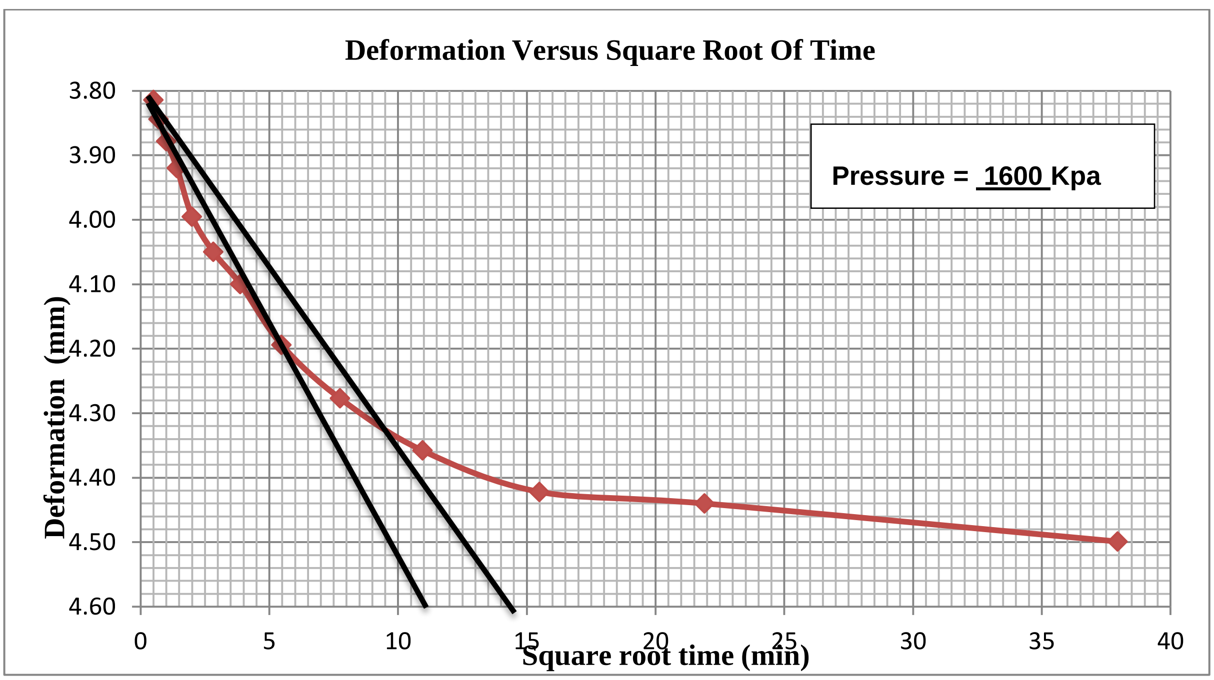


Figure E -24 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 6 (P=1600)

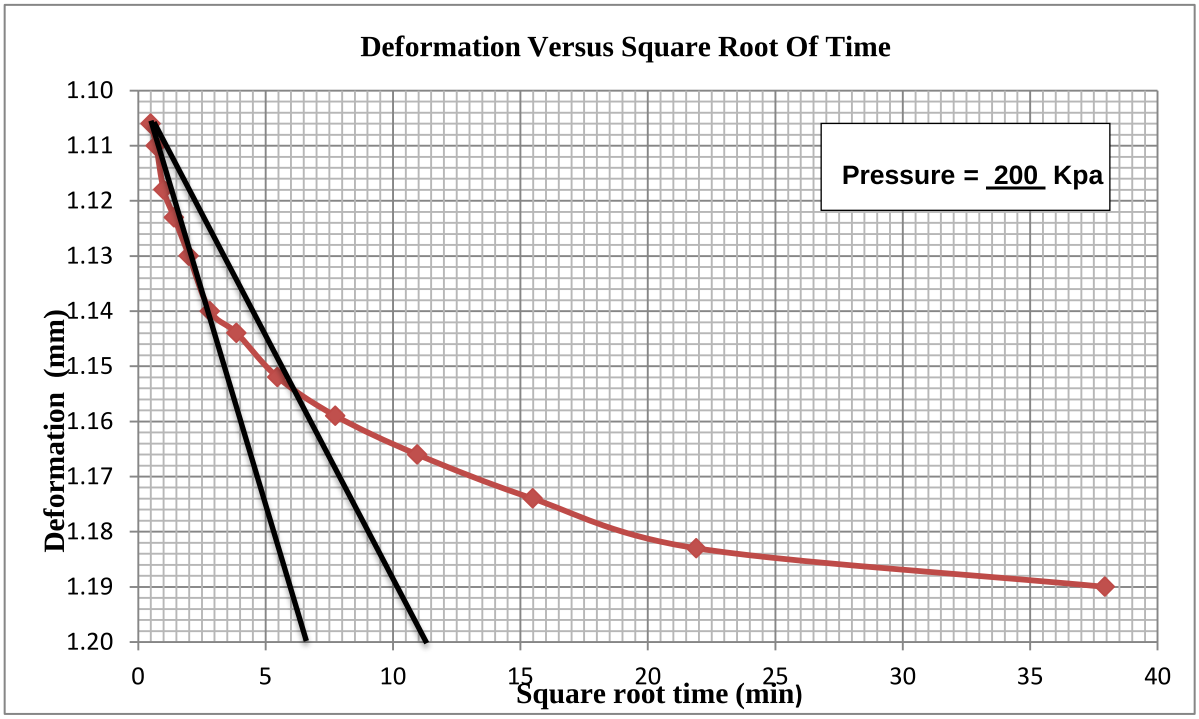


Figure E -25 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 7 (P=200)

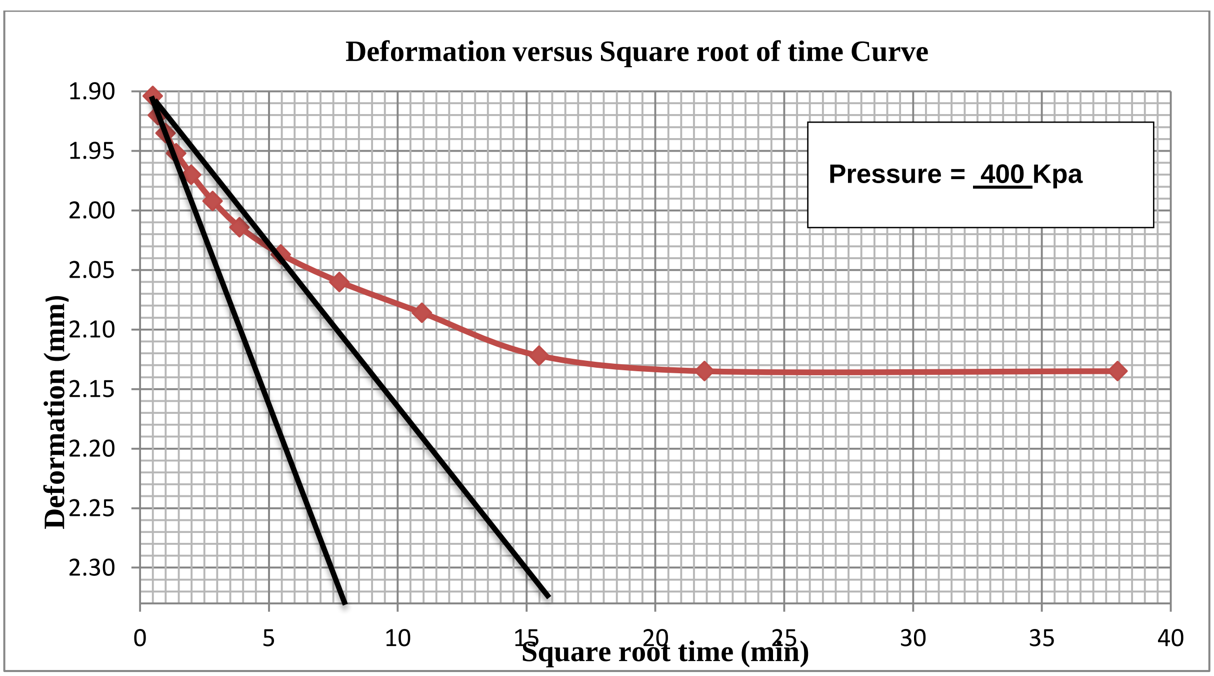


Figure E -26 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 7 (P=400KPa)

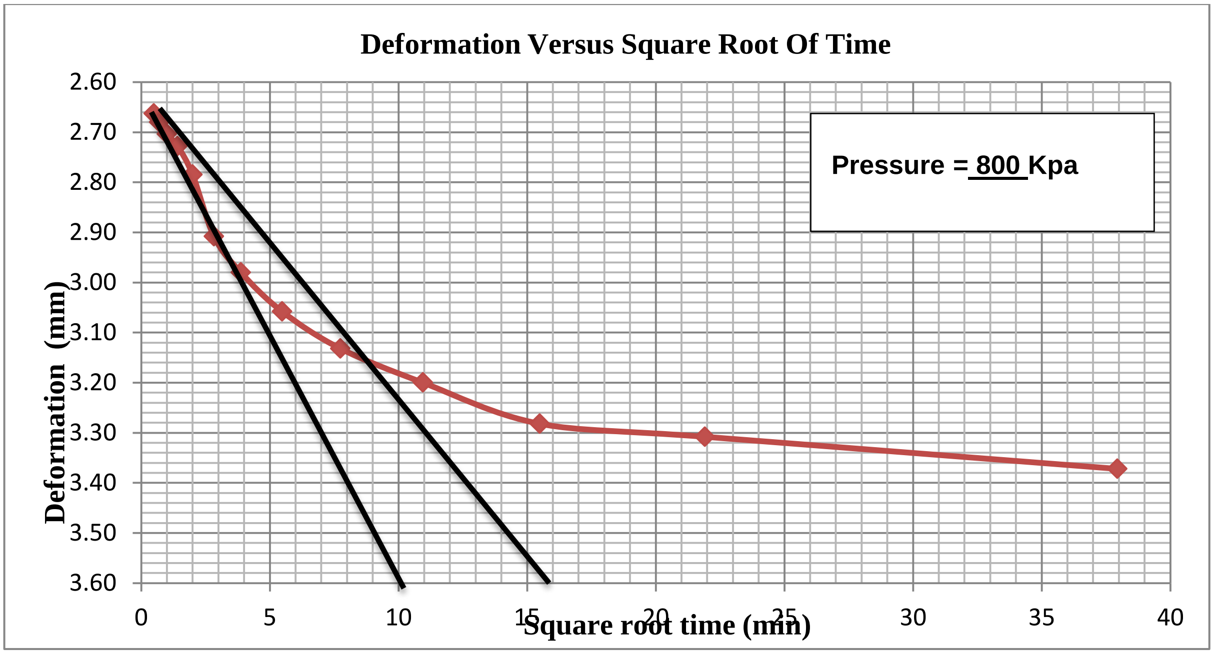


Figure E -27 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 7 (P=800KPa)

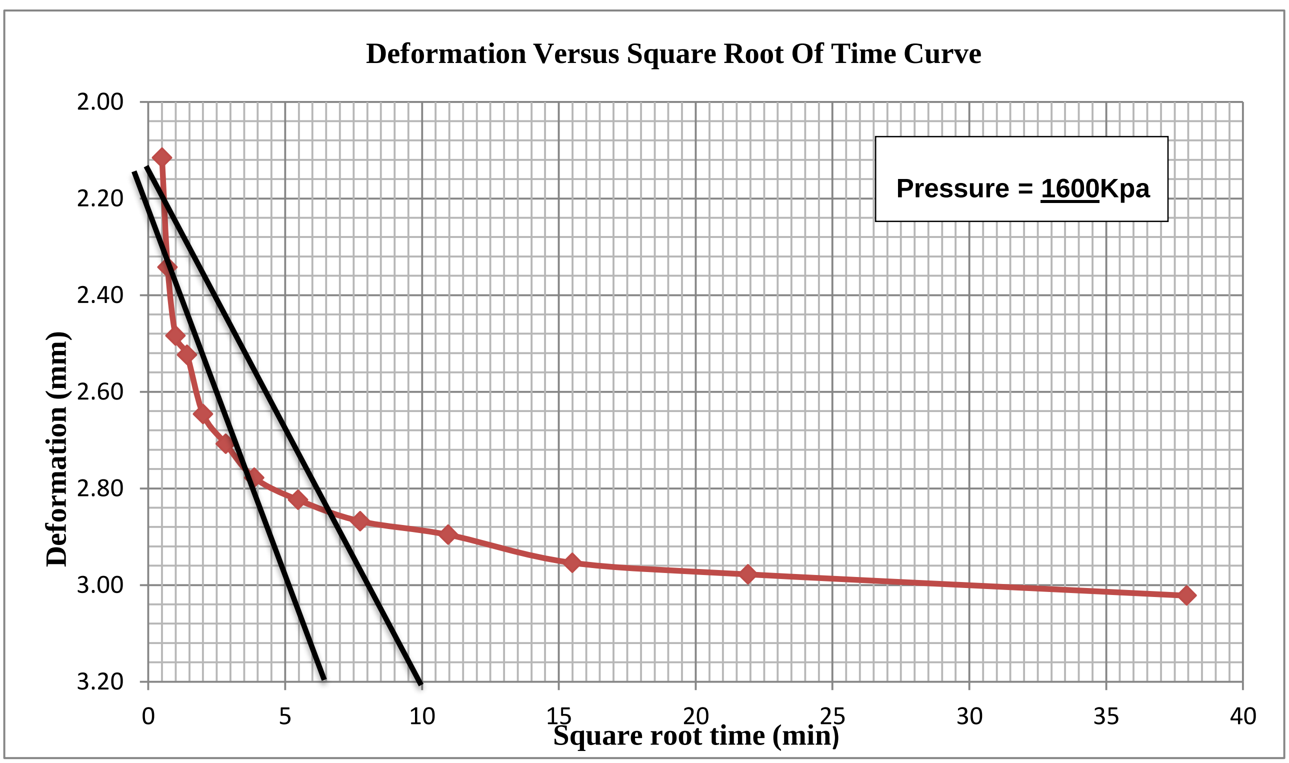


Figure E -28 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 7 (P=1600KPa)

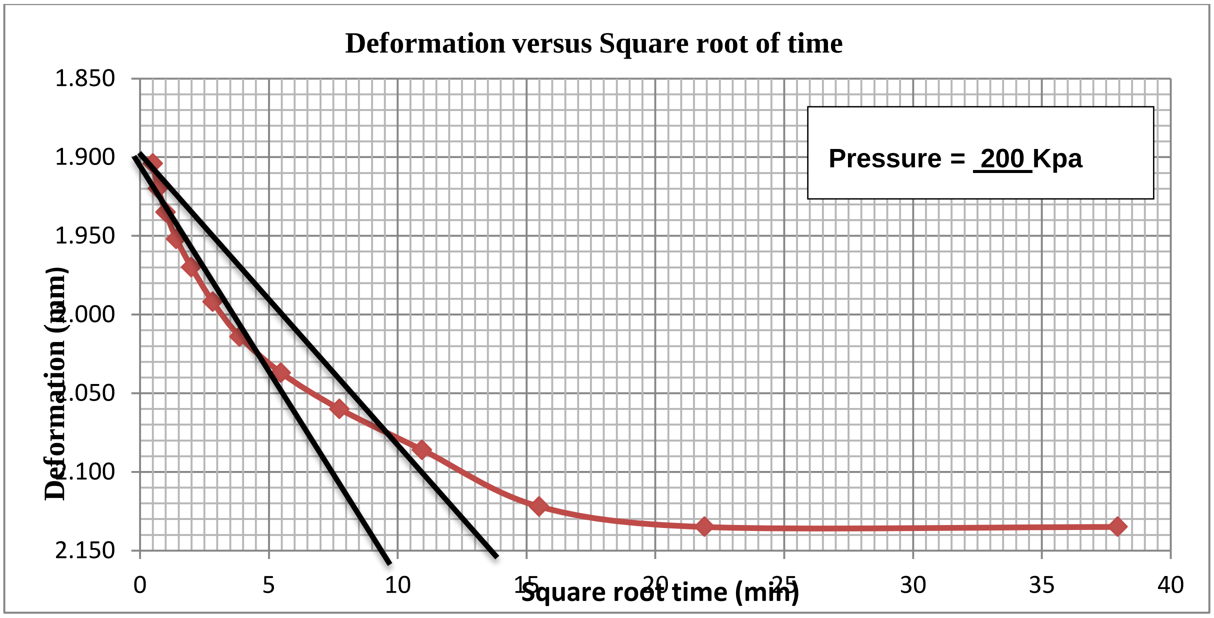


Figure E -29 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 8 (P=200KPa)

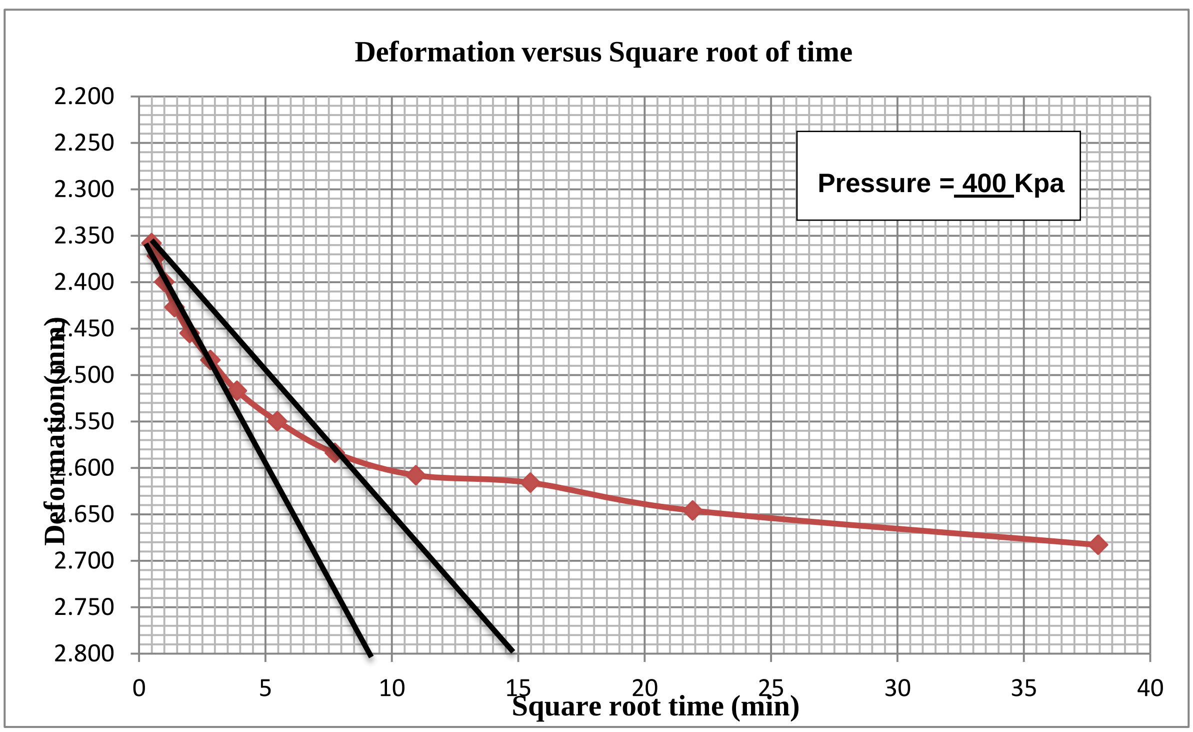


Figure E -30 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 8 (P=400KPa)

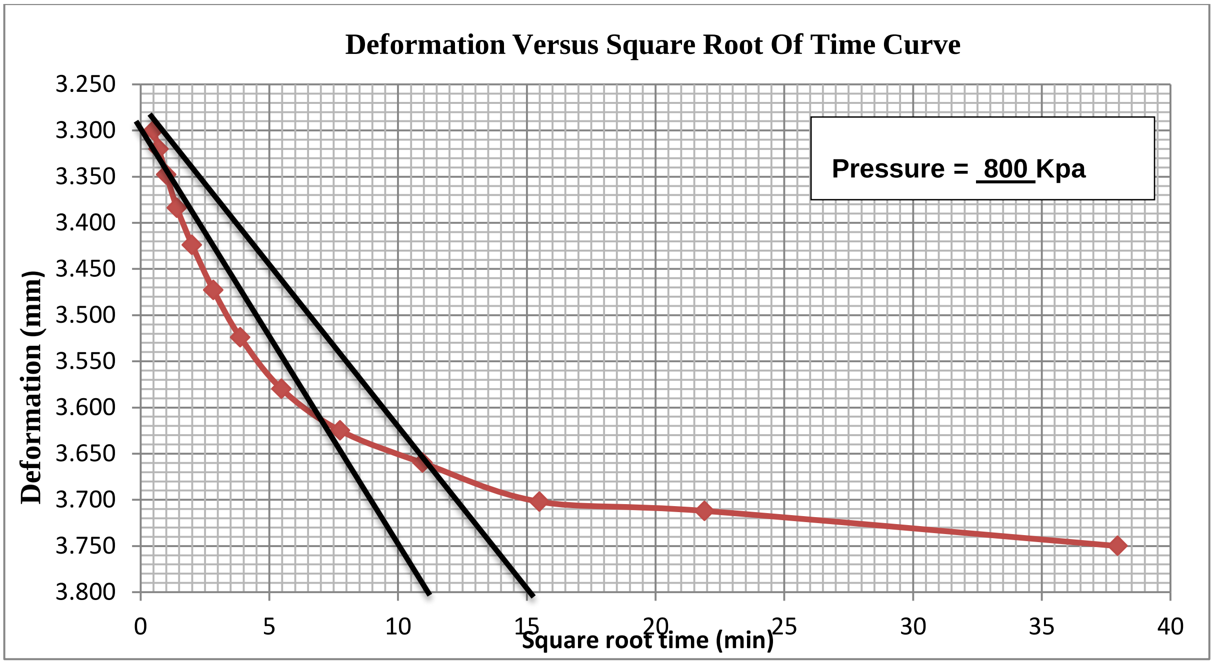


Figure E -31 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 8 (P=800KPa)

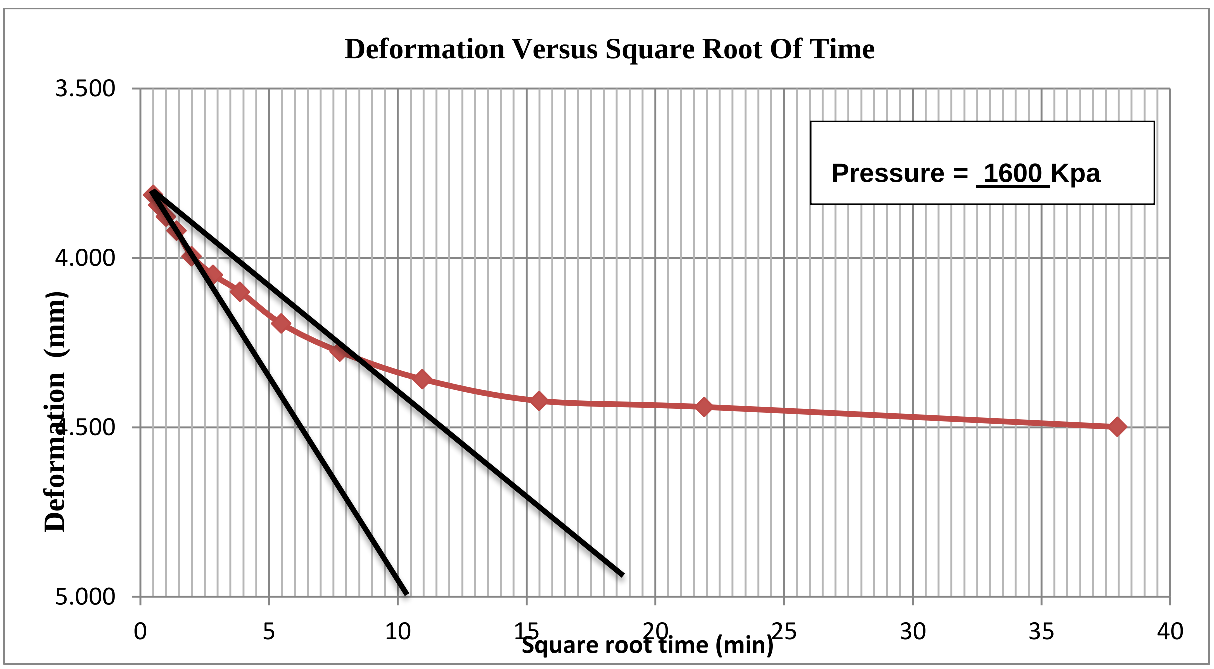


Figure E -32 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 8 (P=1600KPa)

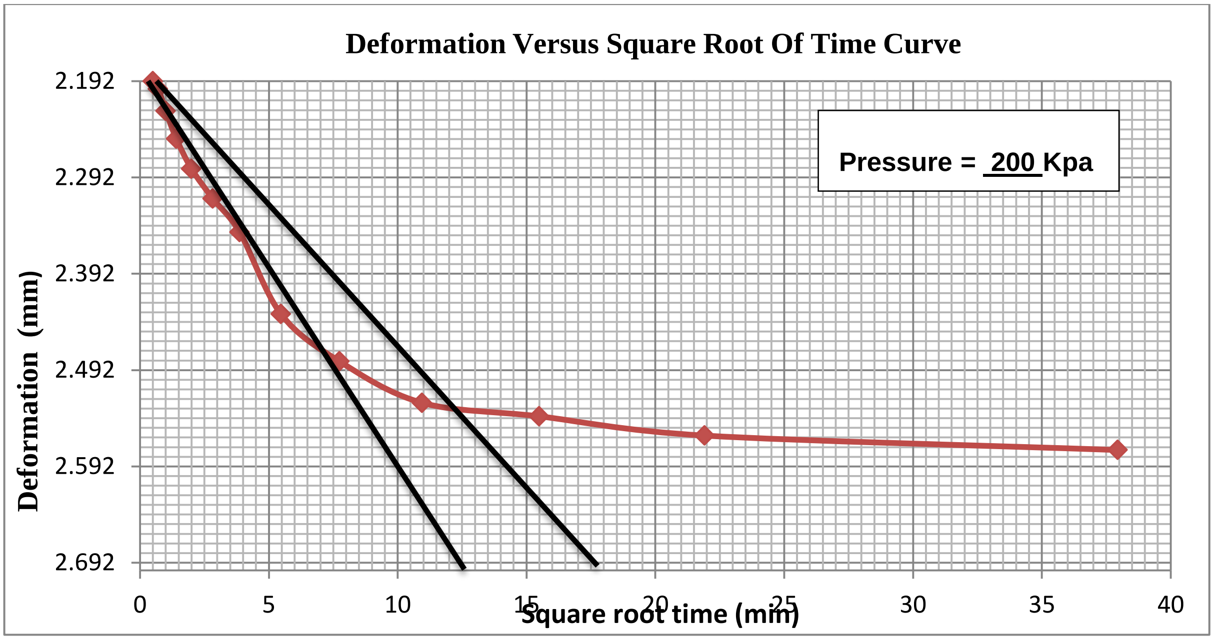


Figure E -33 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 9 (P=200KPa)

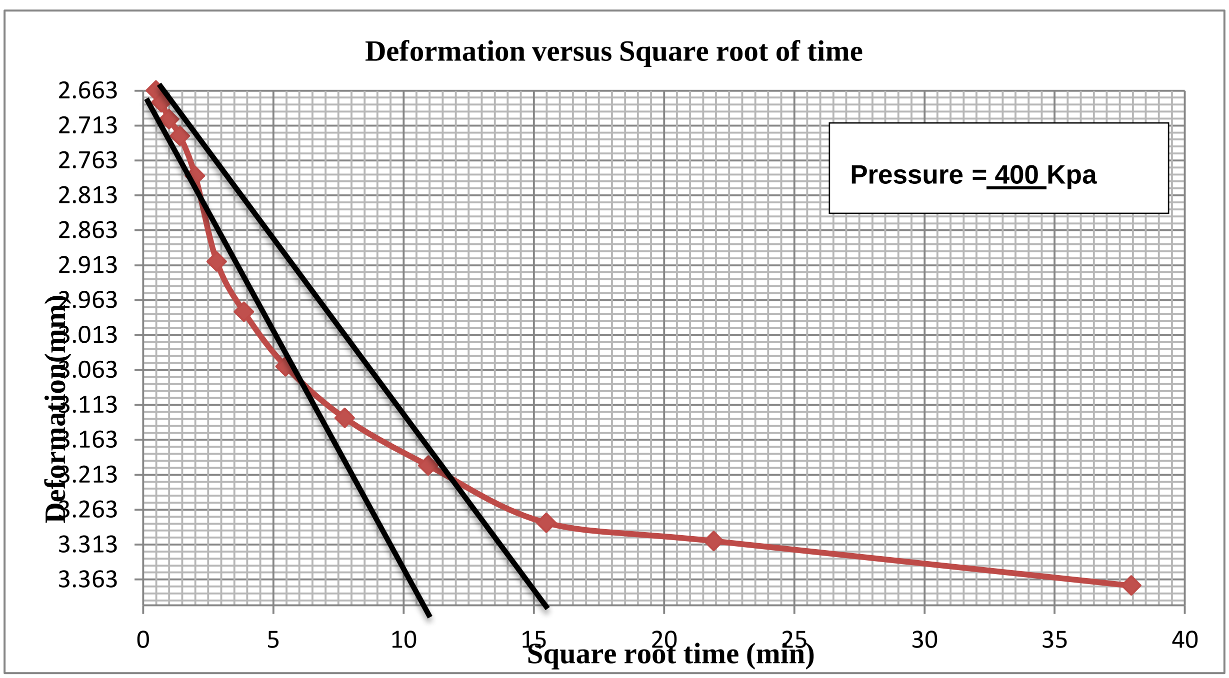


Figure E -34 Typical plots of Deformation t versus Square root of time for remolded expansive soil sample of Mojo town, Sample 9 (P=400KPa)

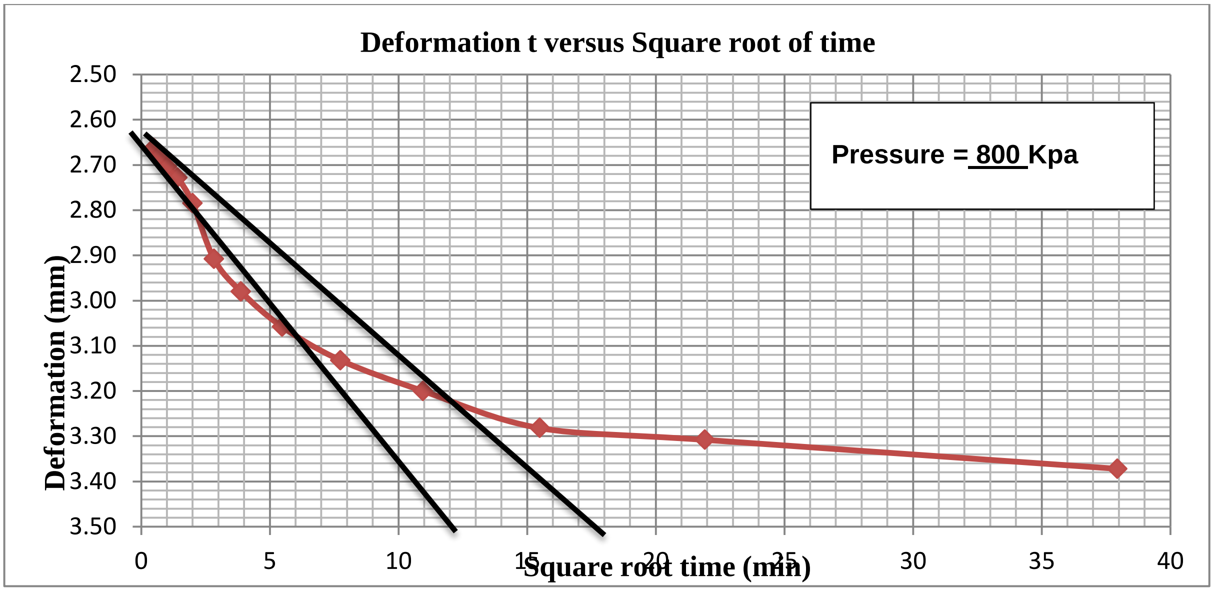


Figure E -35 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 9 (P=800KPa)

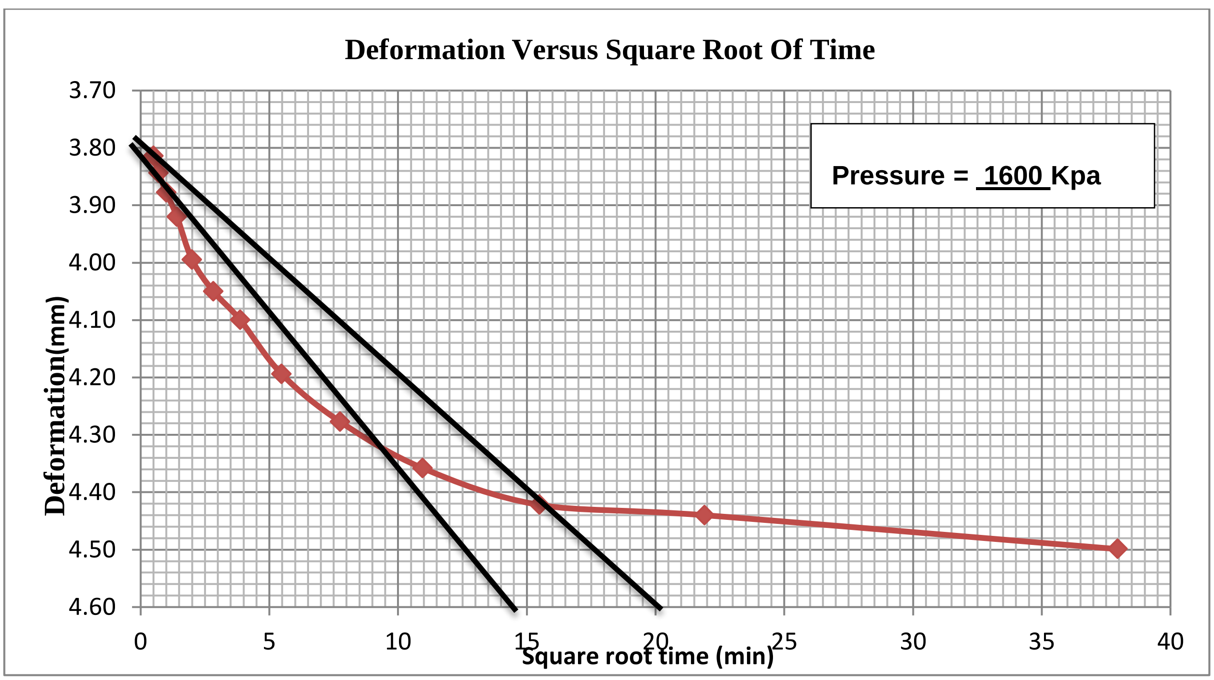


Figure E -36 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 9 (P=1600)

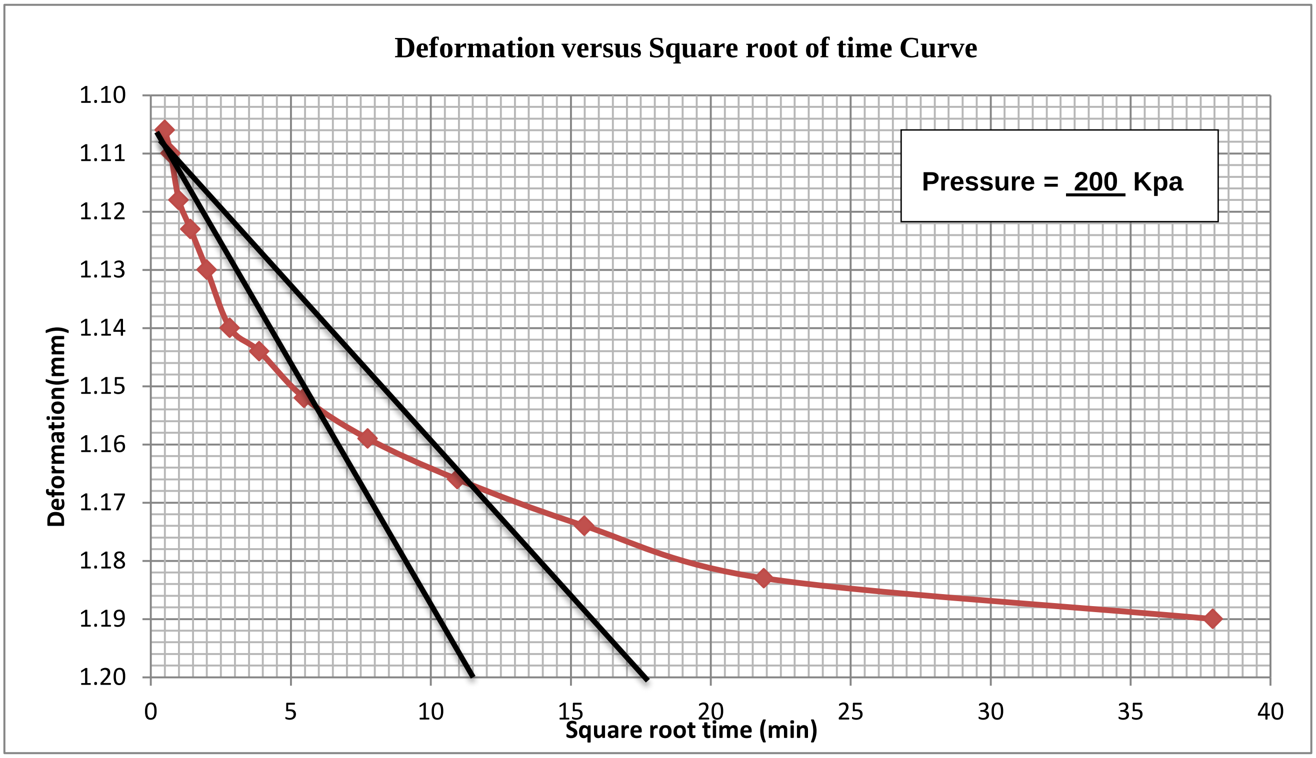


Figure E -37 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 10 (P=200KPa)

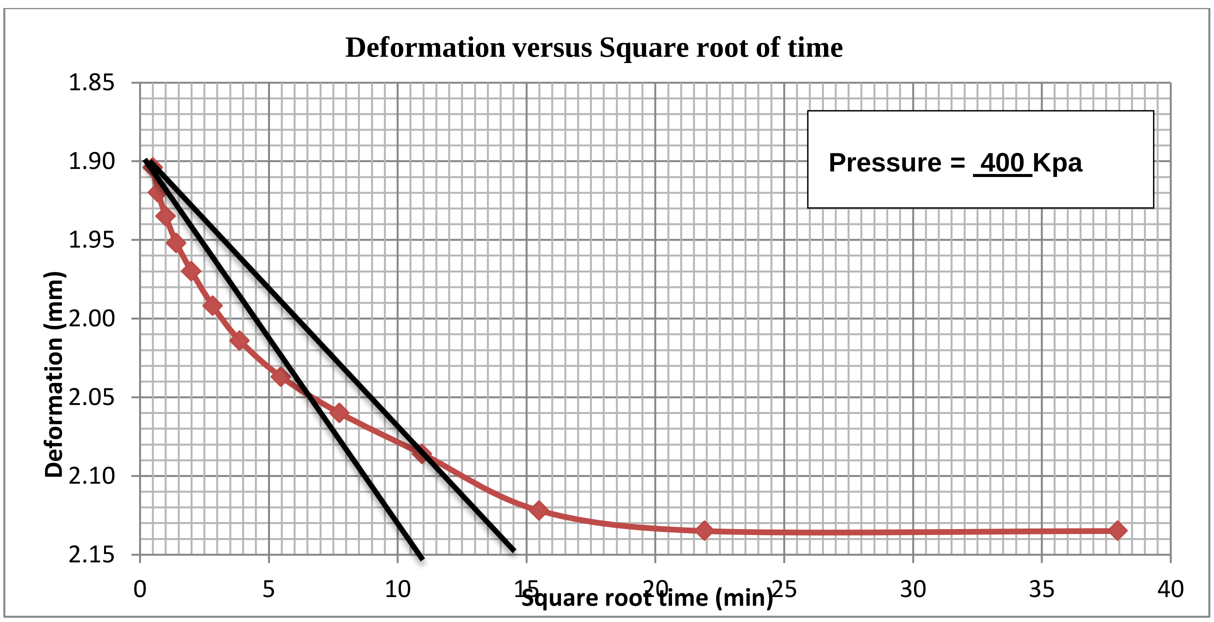


Figure E -38 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 10 (P=400KPa)

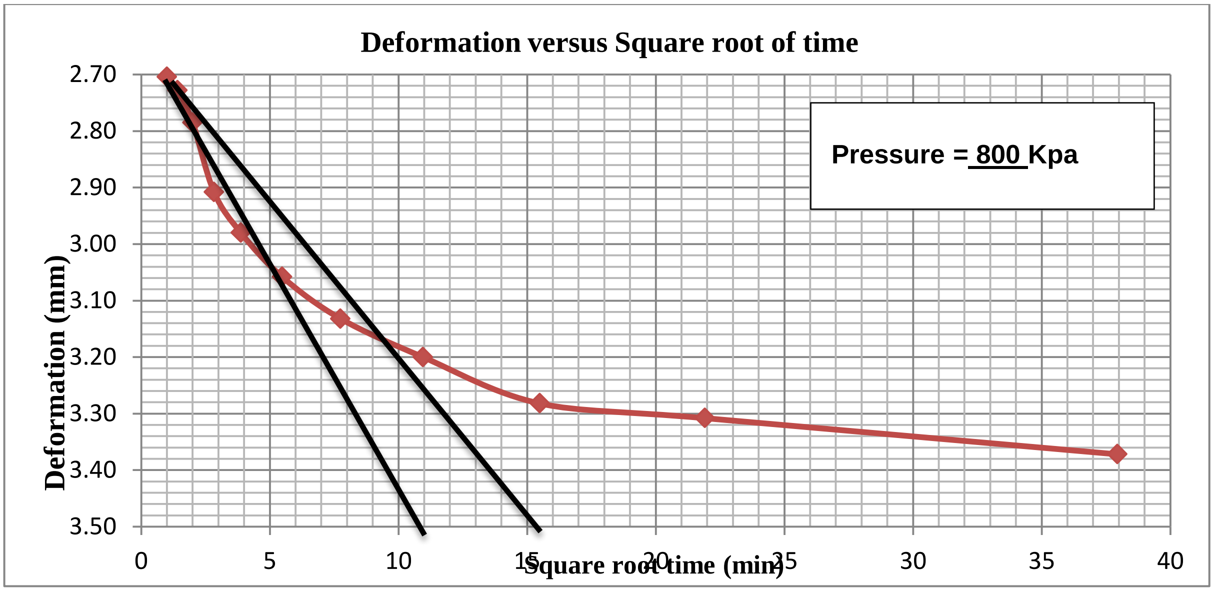


Figure E -39 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 10 (P=800KPa)

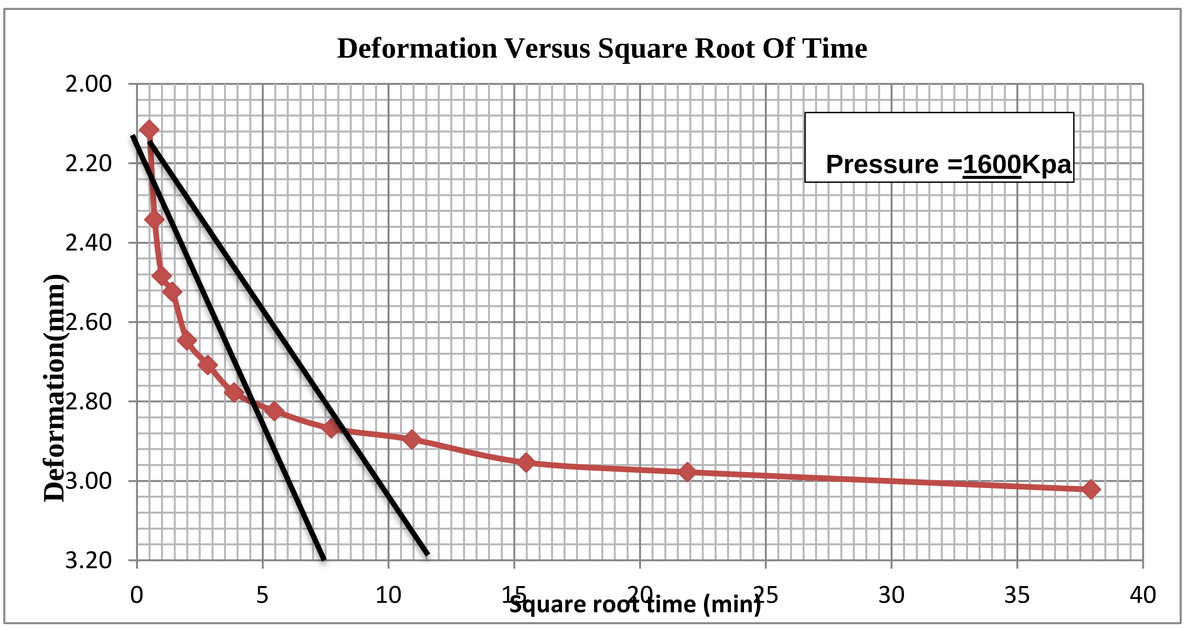


Figure E -40 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 10 (P=1600KPa)

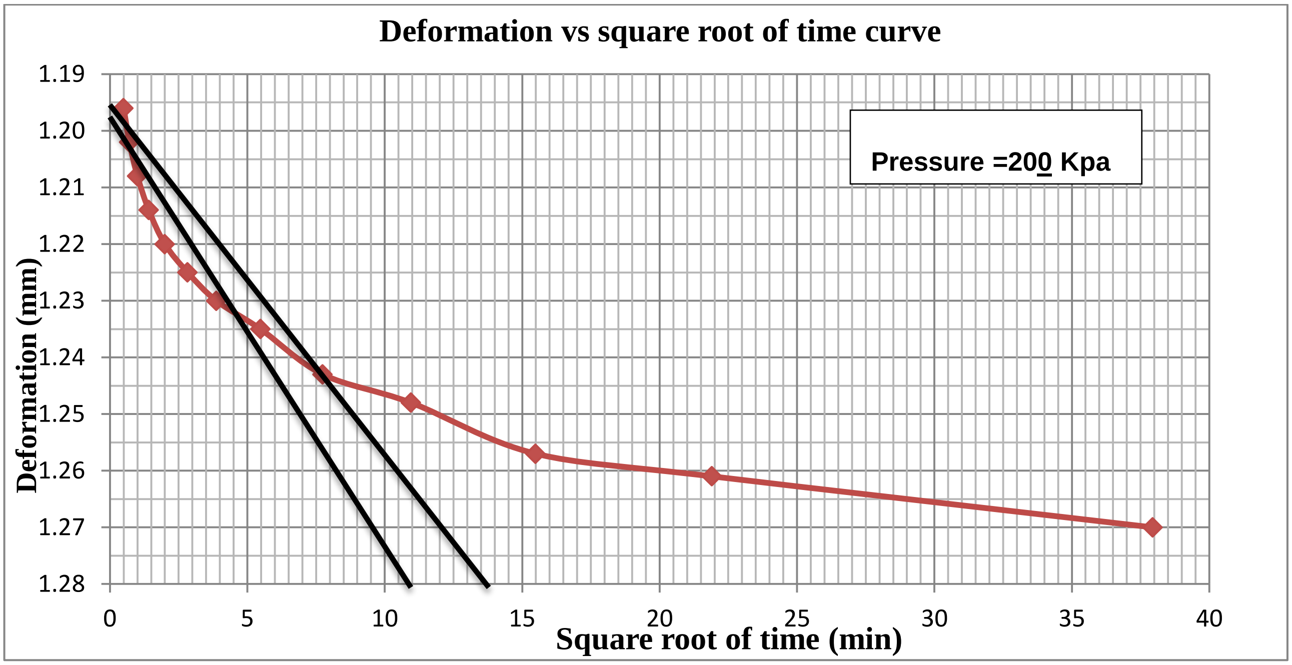


Figure E -41 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 11 (P=200KPa)

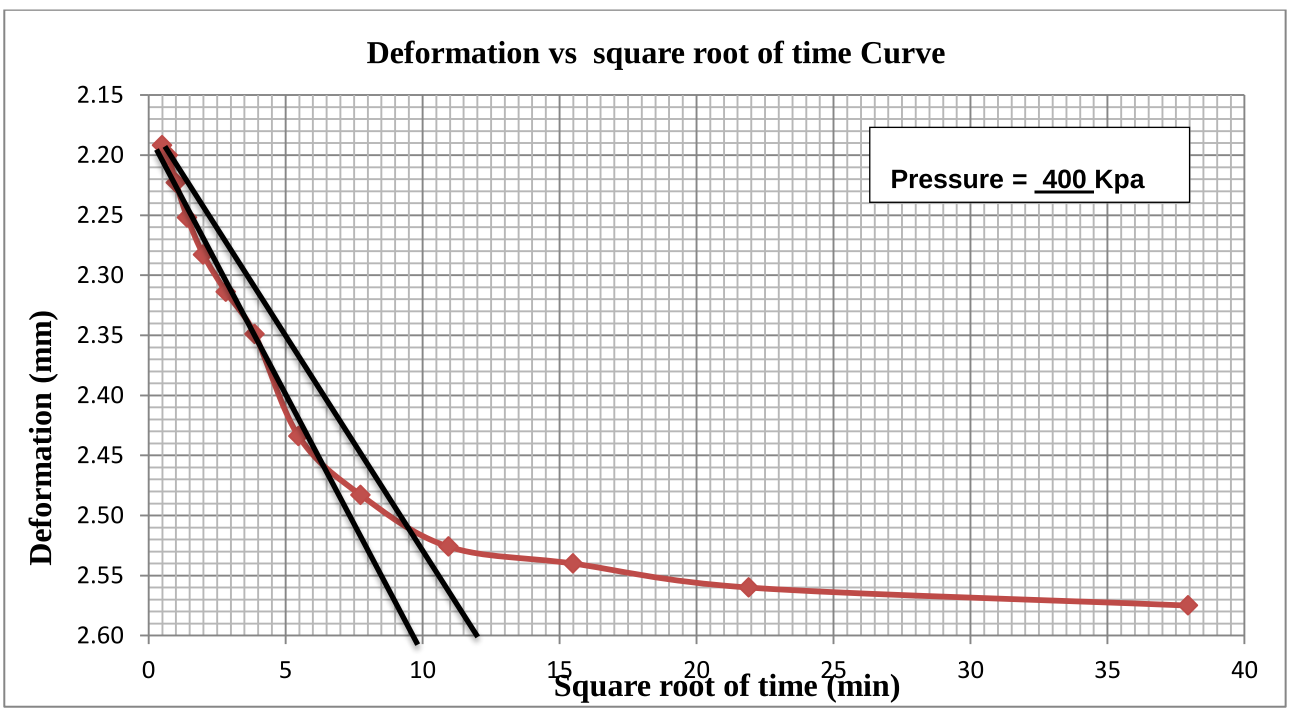


Figure E -42 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 11 (P=400KPa),

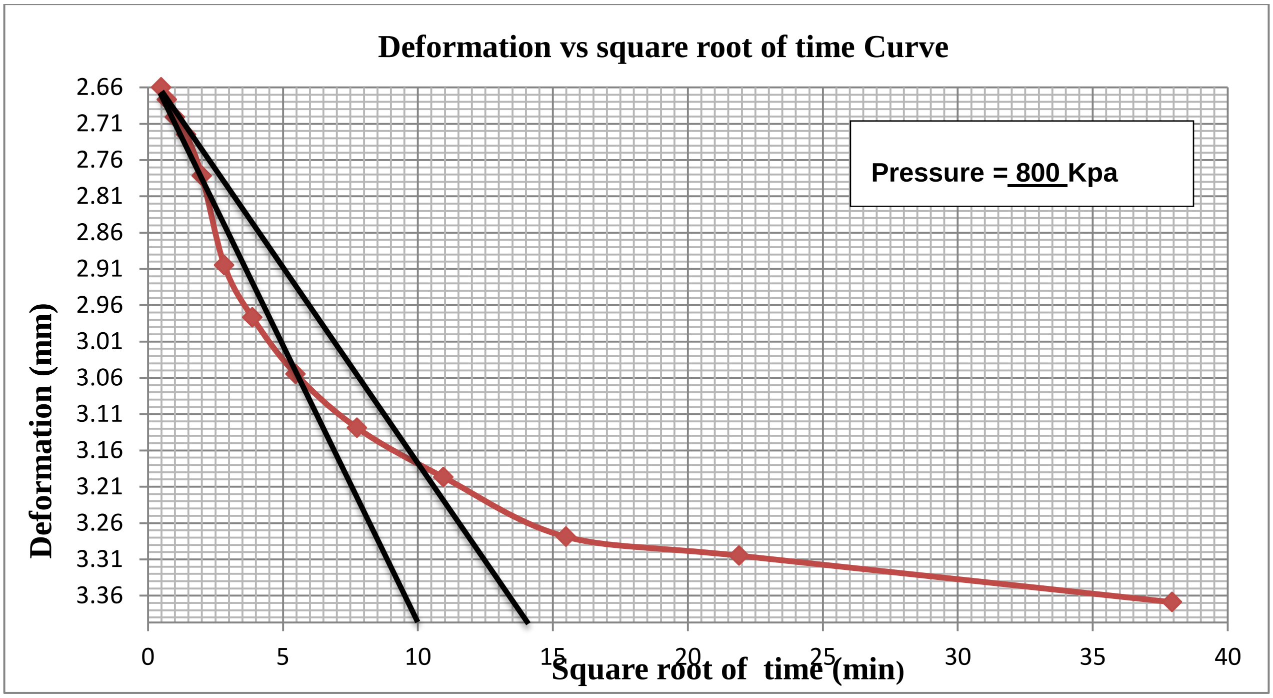


Figure E -43 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 11 (P=800KPa)

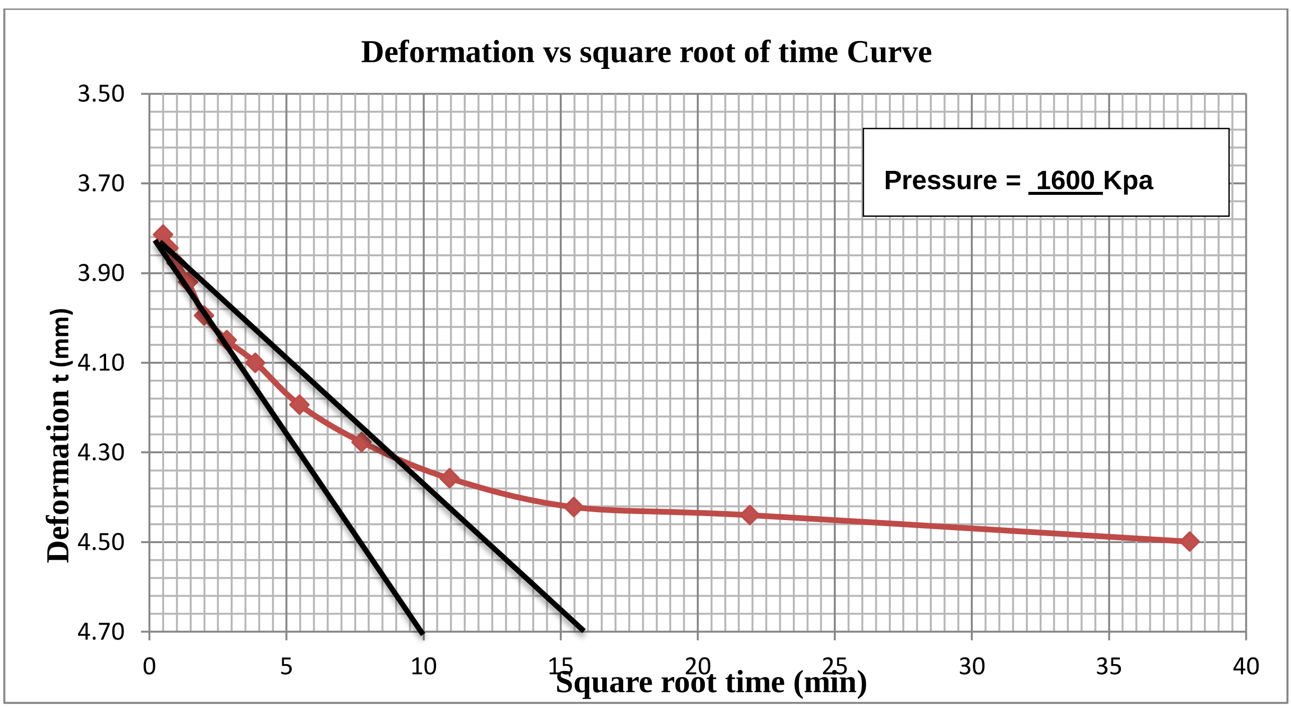


Figure E -44 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 11 (P=800KPa)

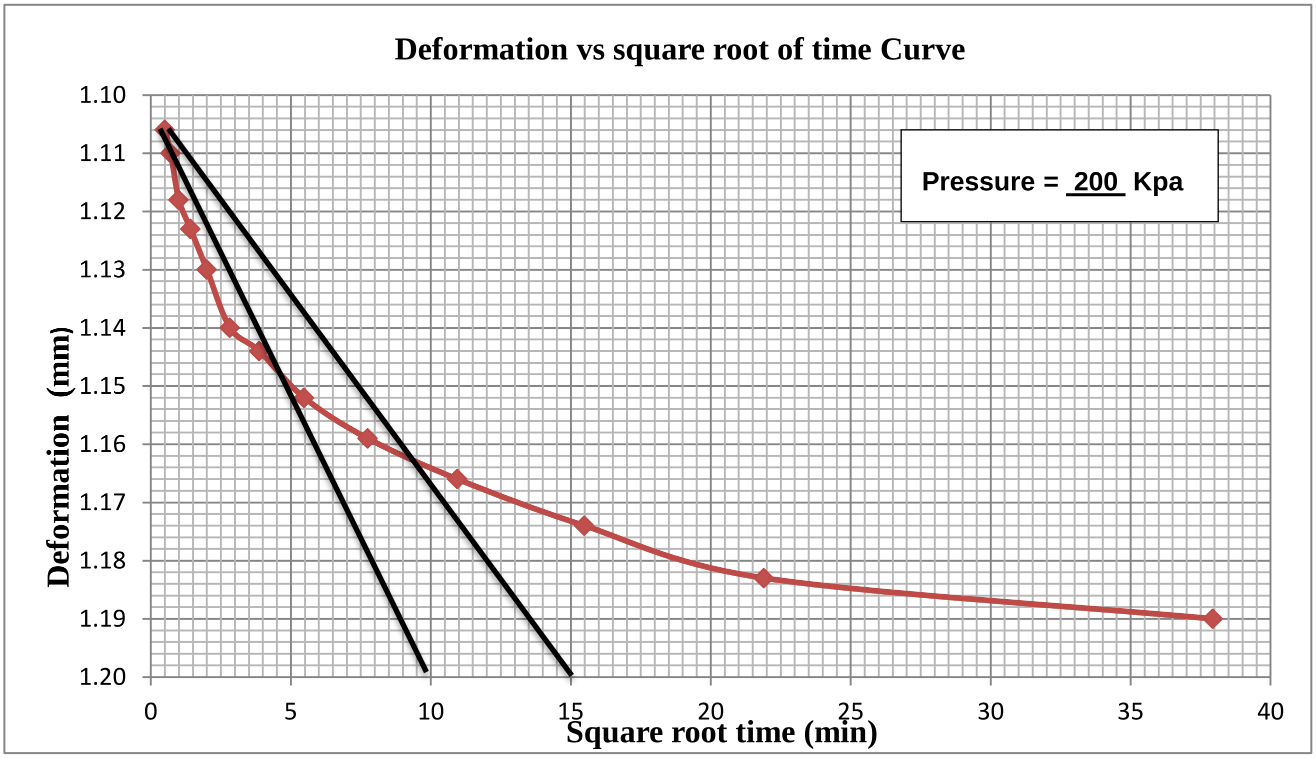


Figure E -45 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 12 (P=200KPa)

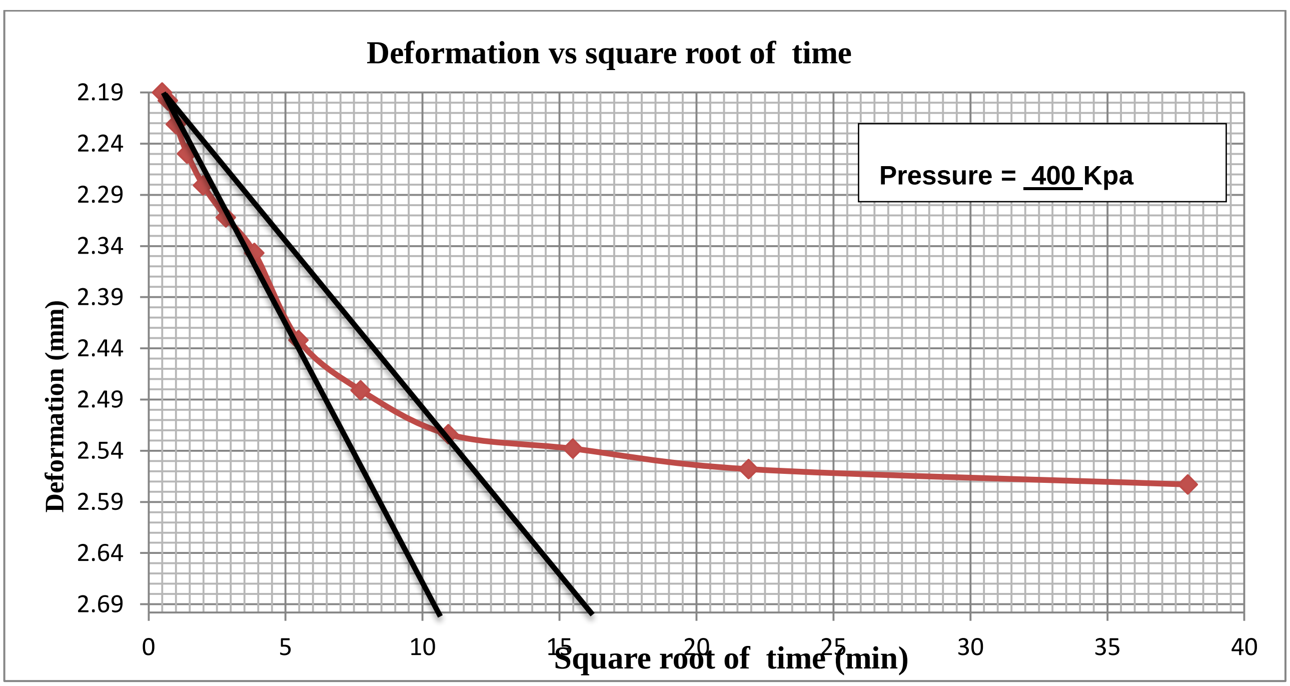


Figure E -46 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 12 (P=400KPa)

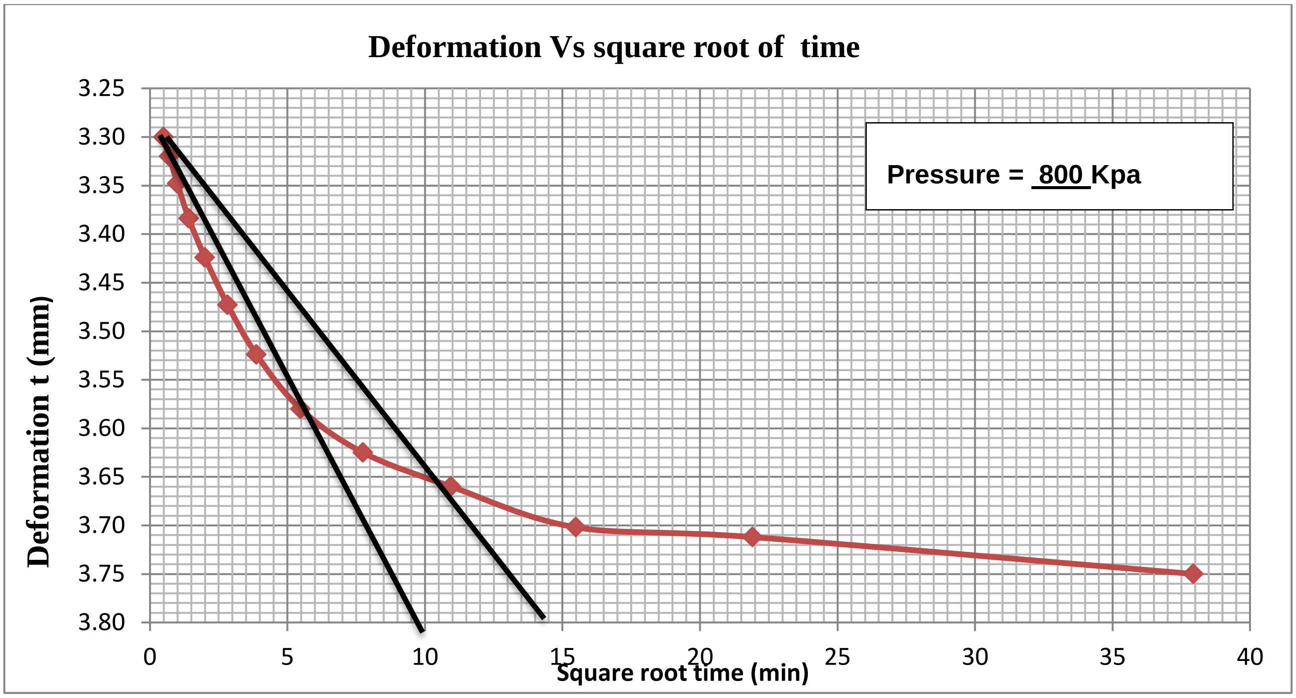


Figure E -47 Typical plots of Deformation t versus Square root of time for remolded expansive soil sample of Mojo town, Sample 12 (P=800KPa)

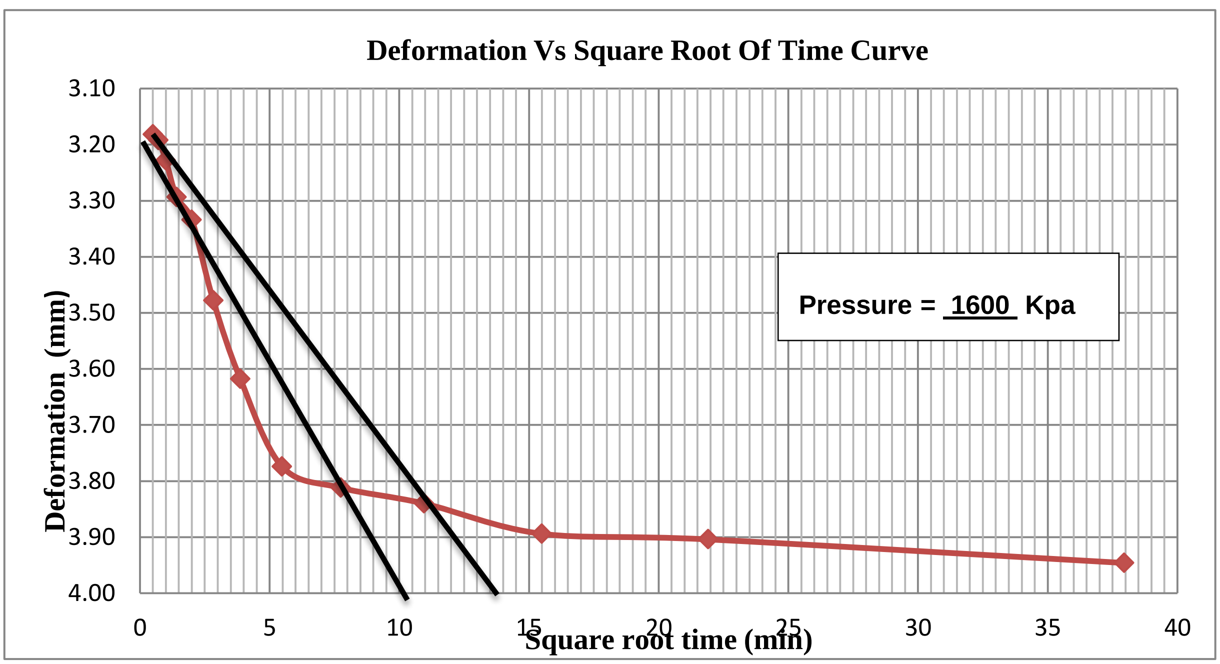


Figure E -48 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 12 (P=1600KPa)

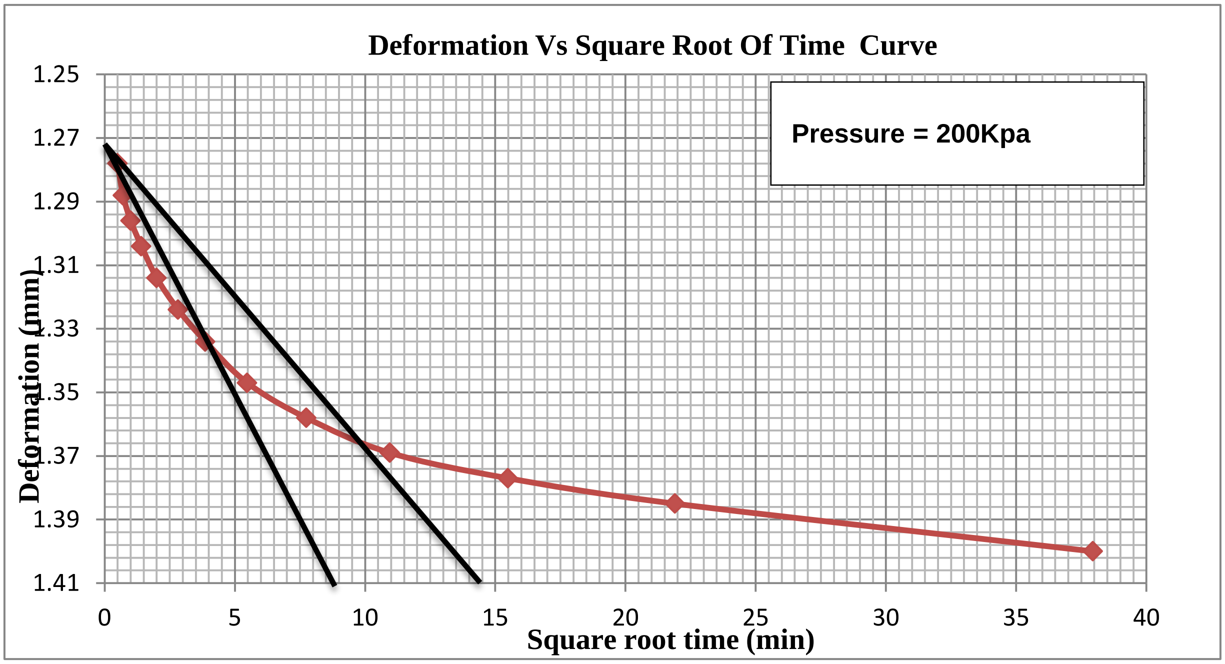


Figure E -49 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 13 (P=200KPa)

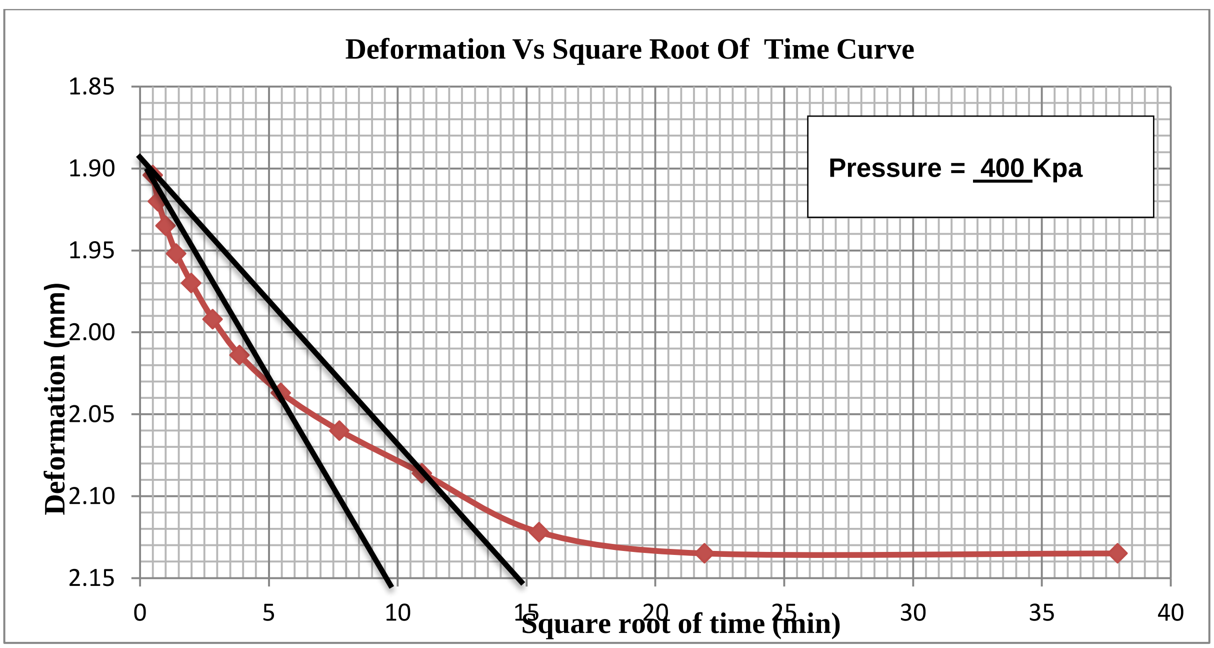


Figure E -50 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 13 (P=400KPa)

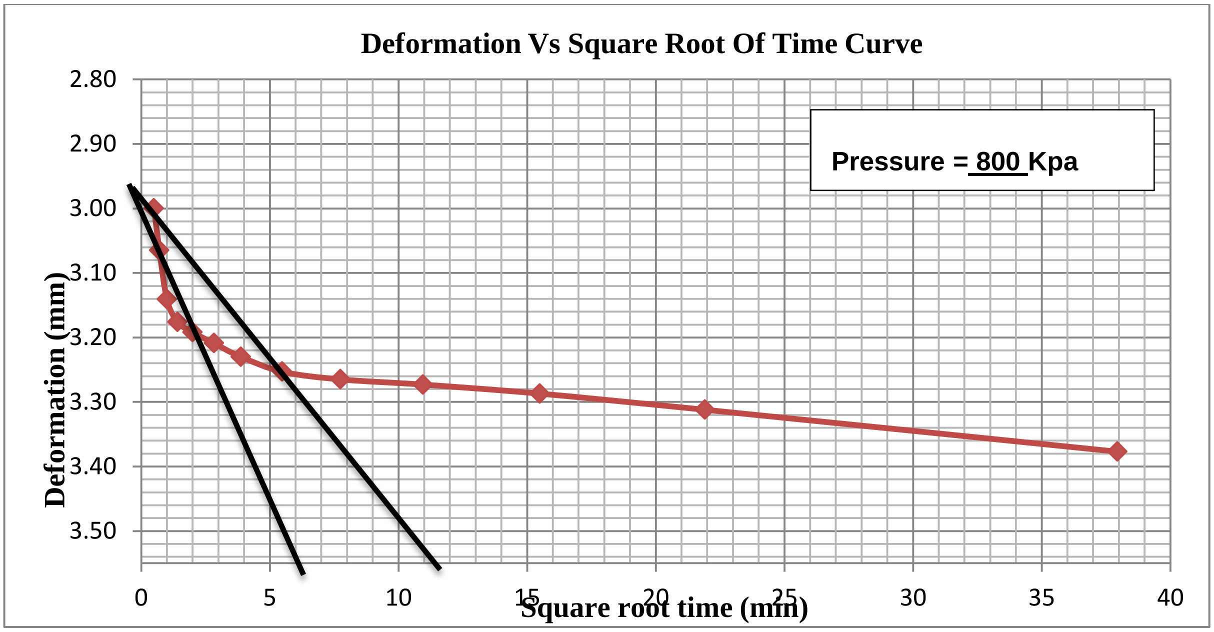


Figure E -51 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 13 (P=800KPa)

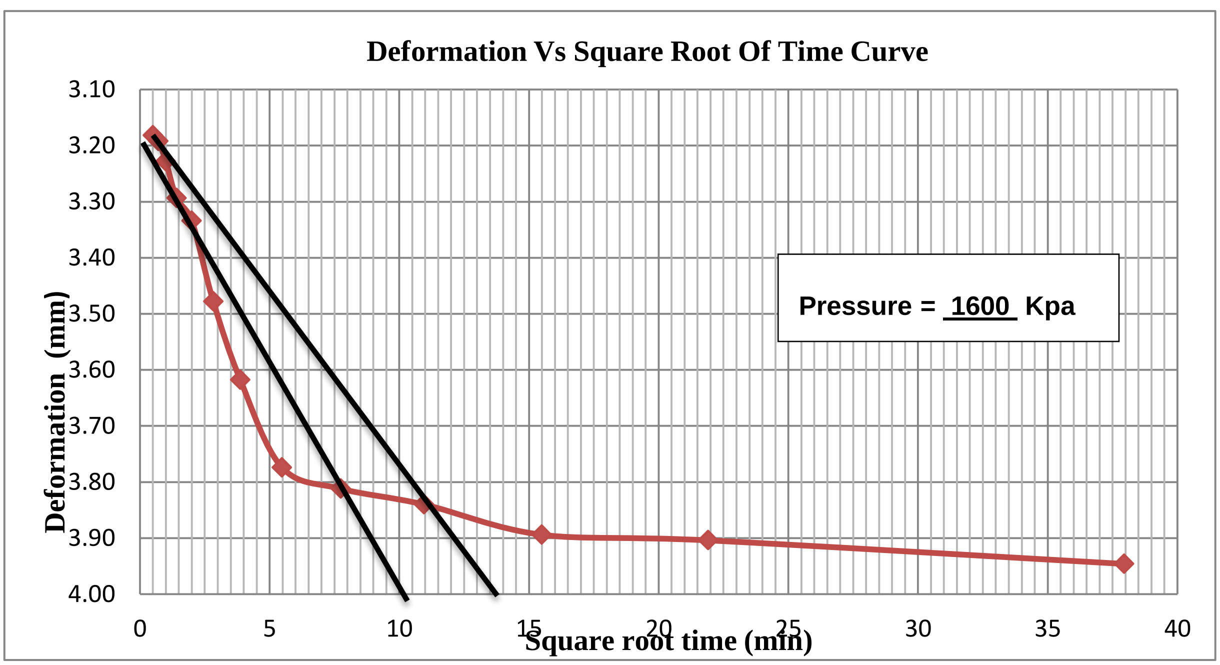


Figure E -52 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 13 (P=1600KPa)

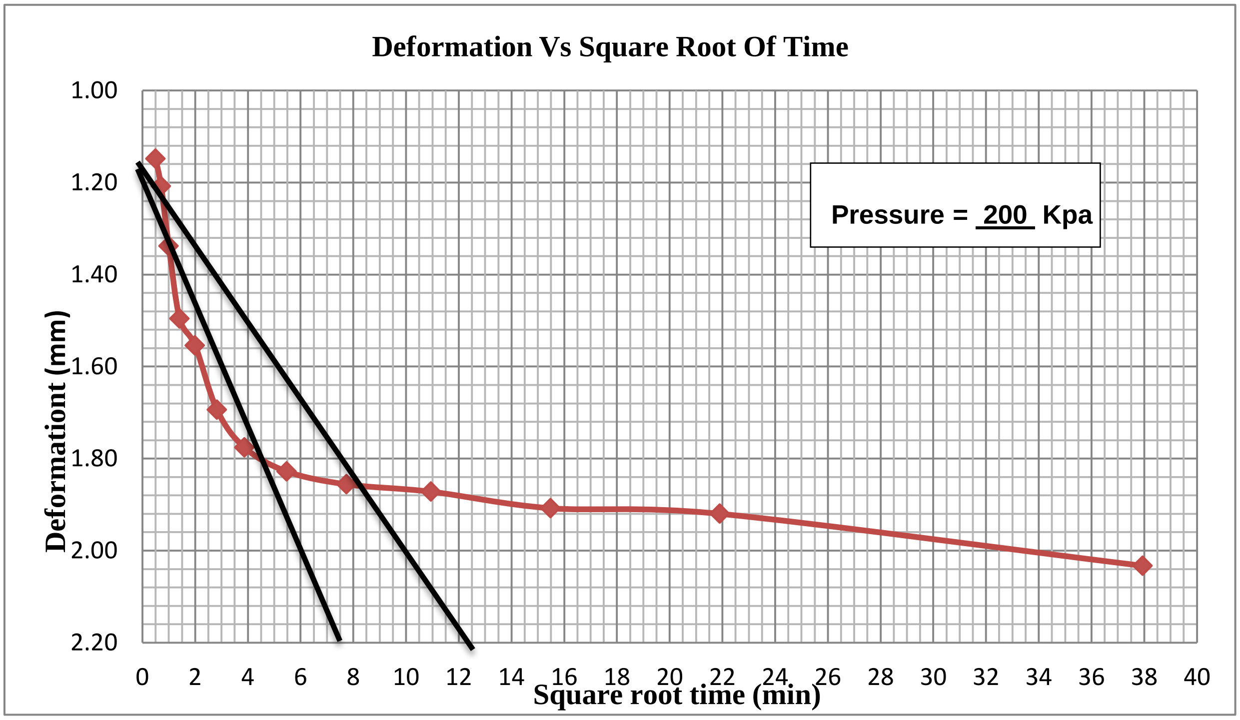


Figure E -53 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 14 (P=200KPa)

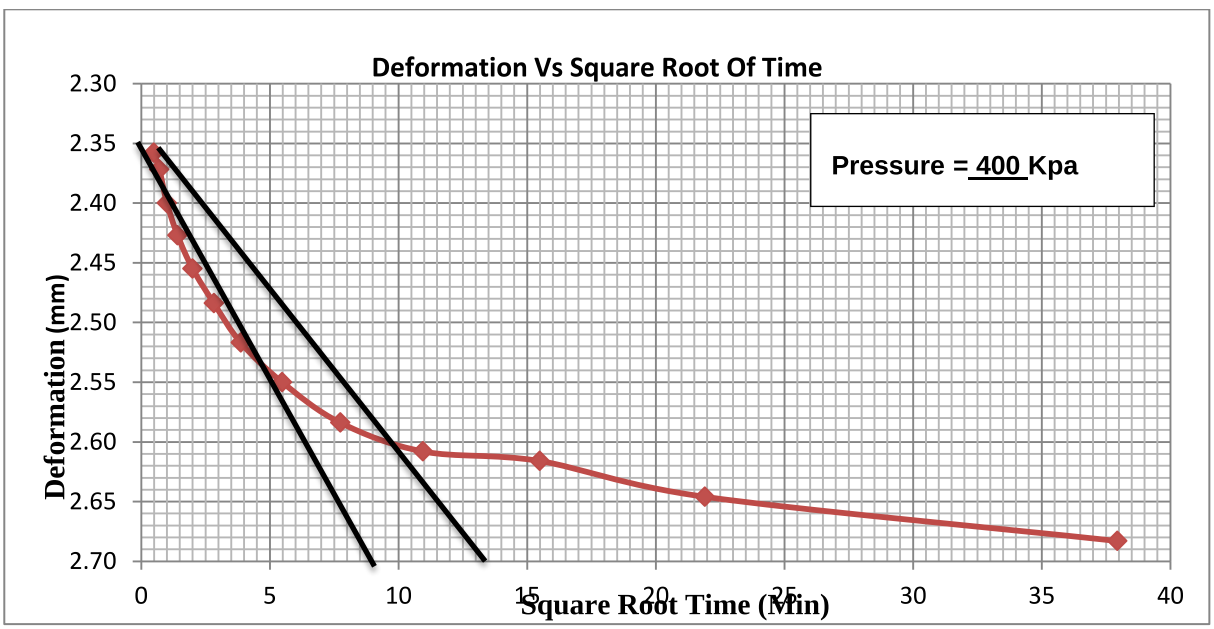


Figure E -54Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 14 (P=400KPa

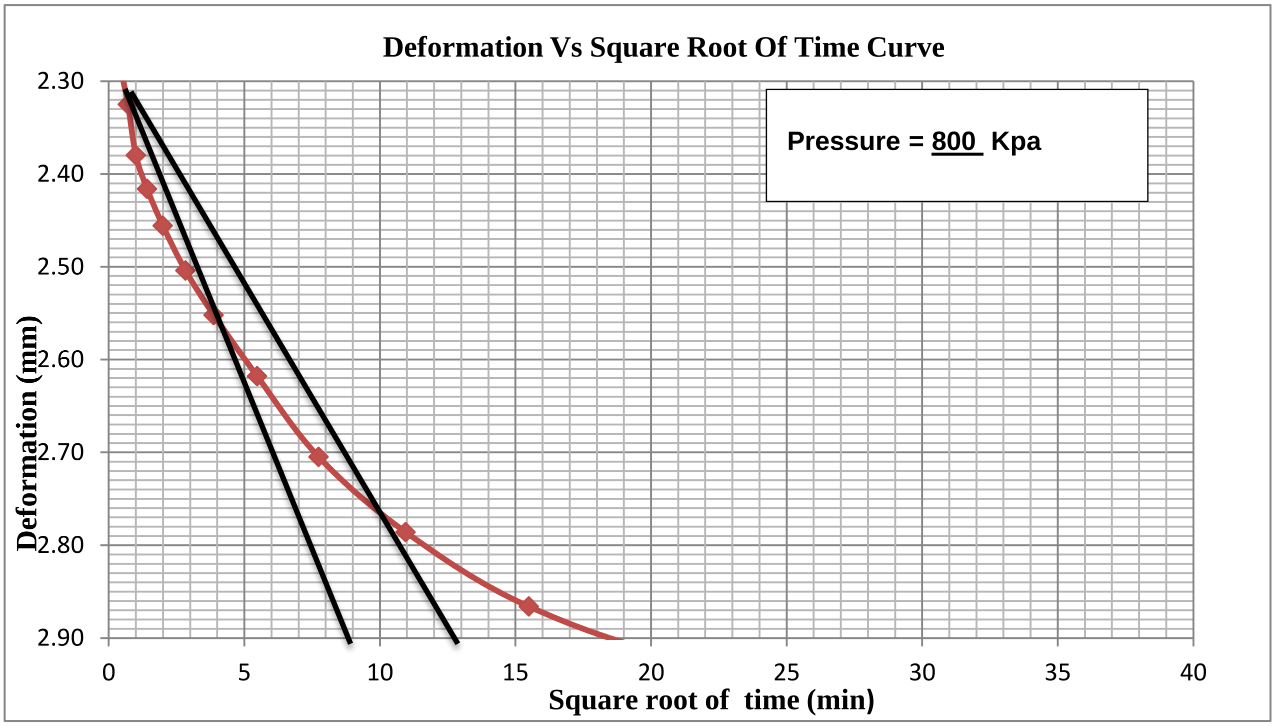


Figure E -55 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 14 (P=800KPa)

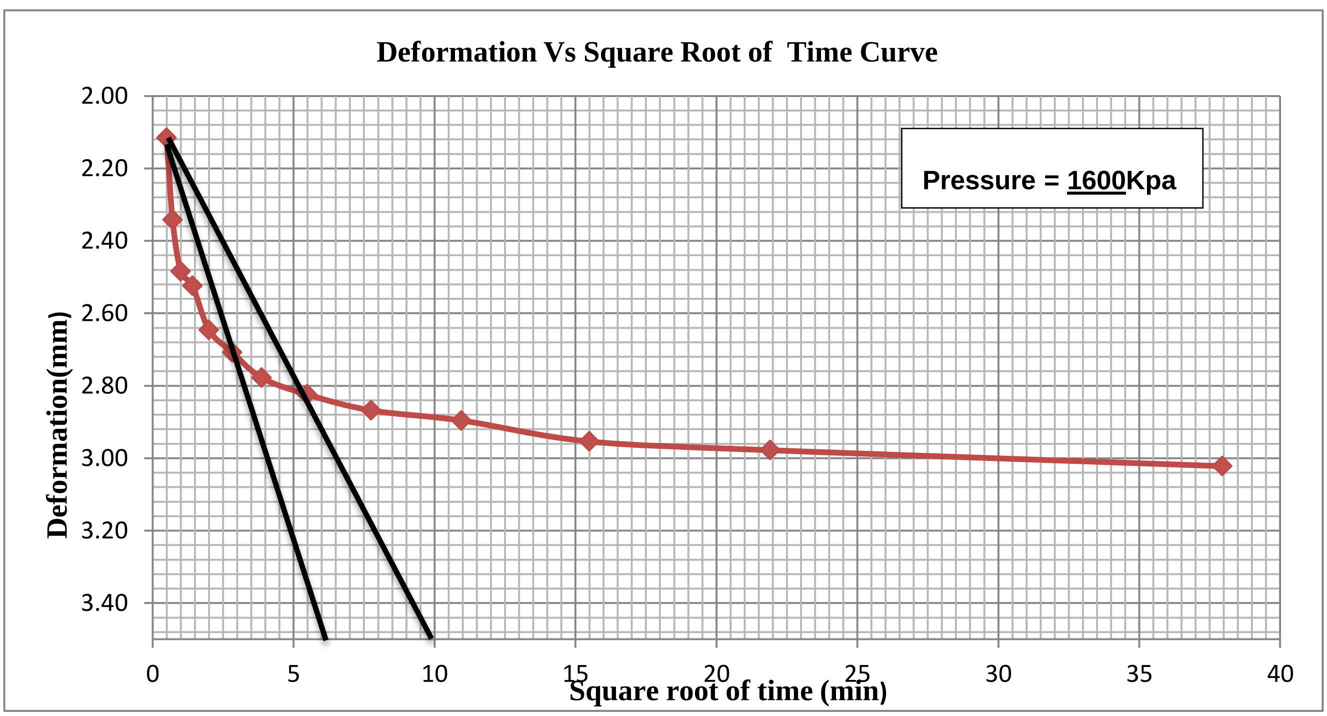


Figure E -56 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 14 (P=1600KPa)

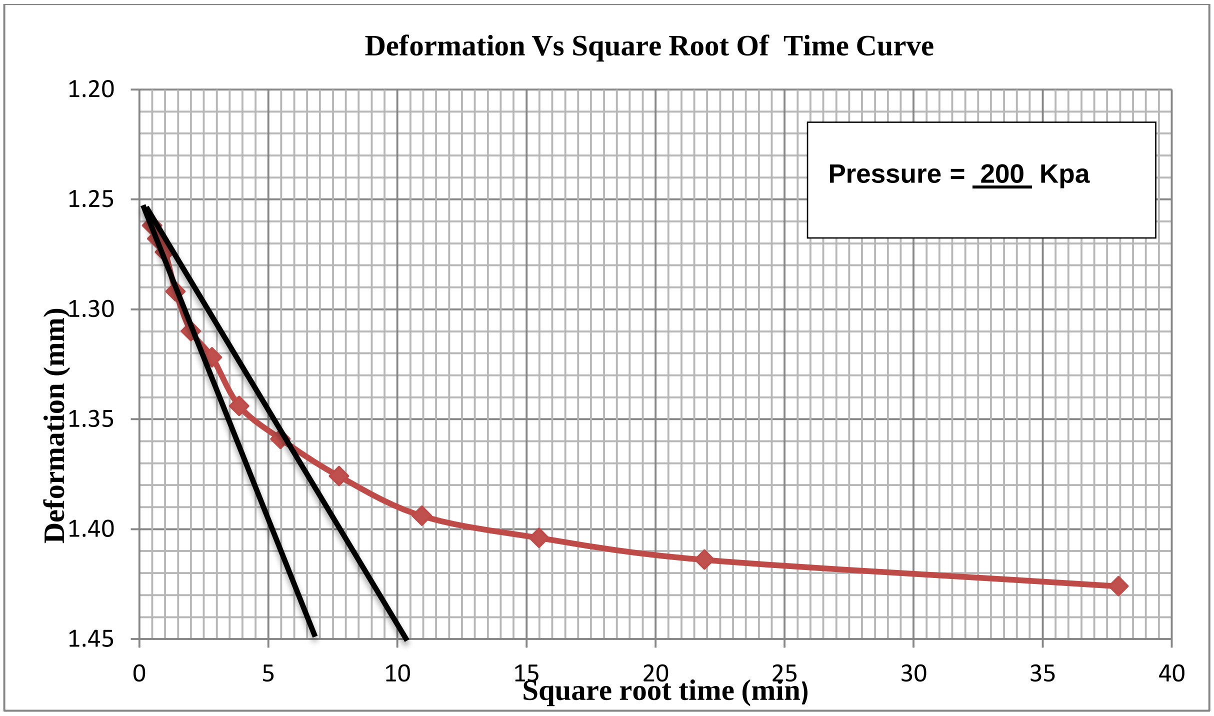


Figure E -57 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 15 (P=200KPa)

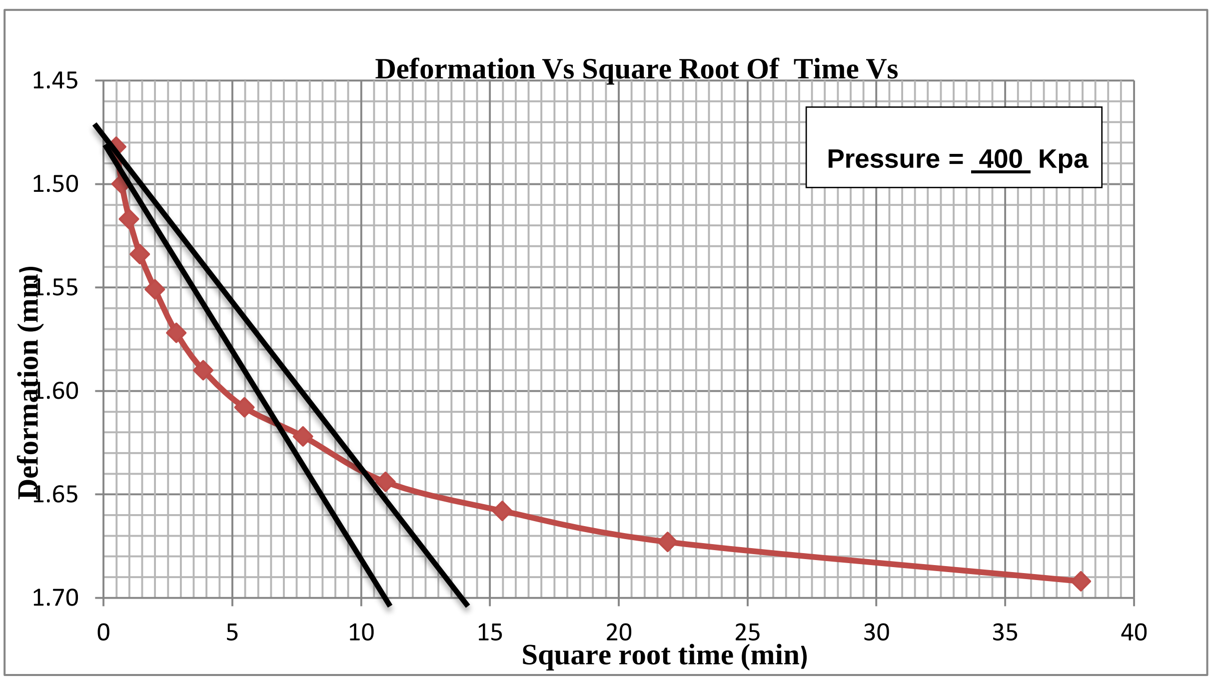


Figure E -58 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 15 (P=400KPa)

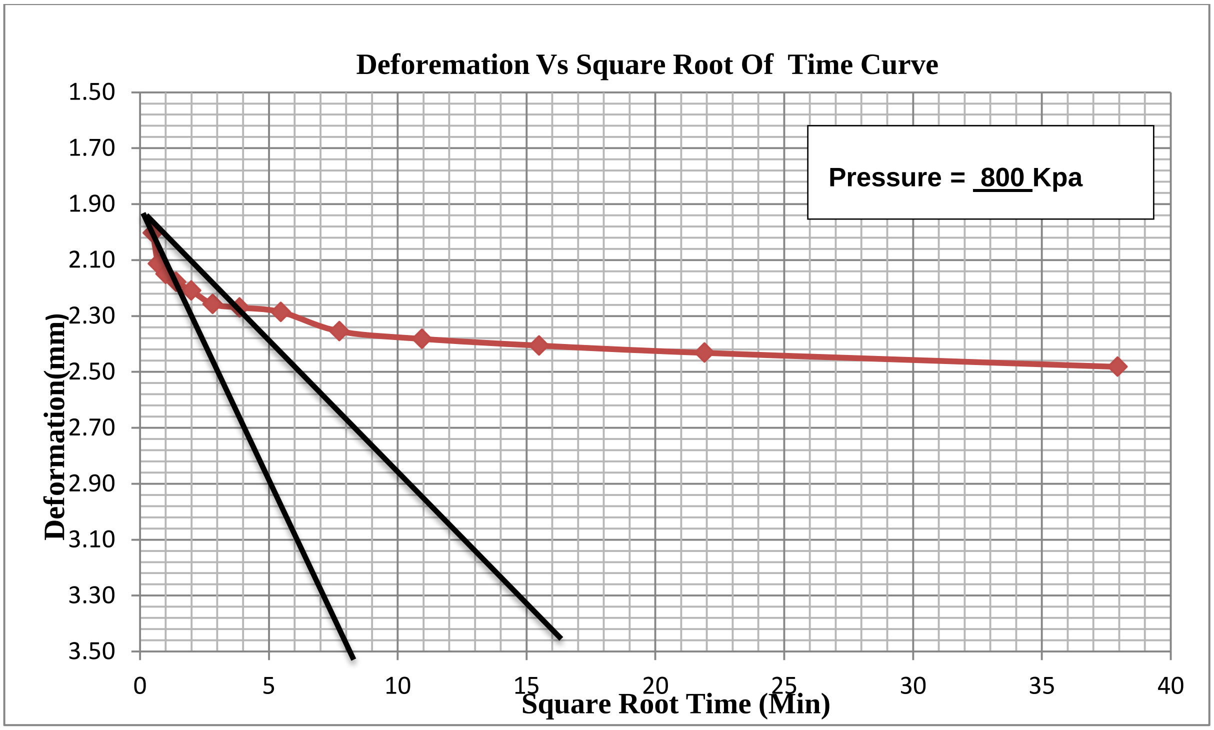


Figure E -59 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 15 (P=800KPa)

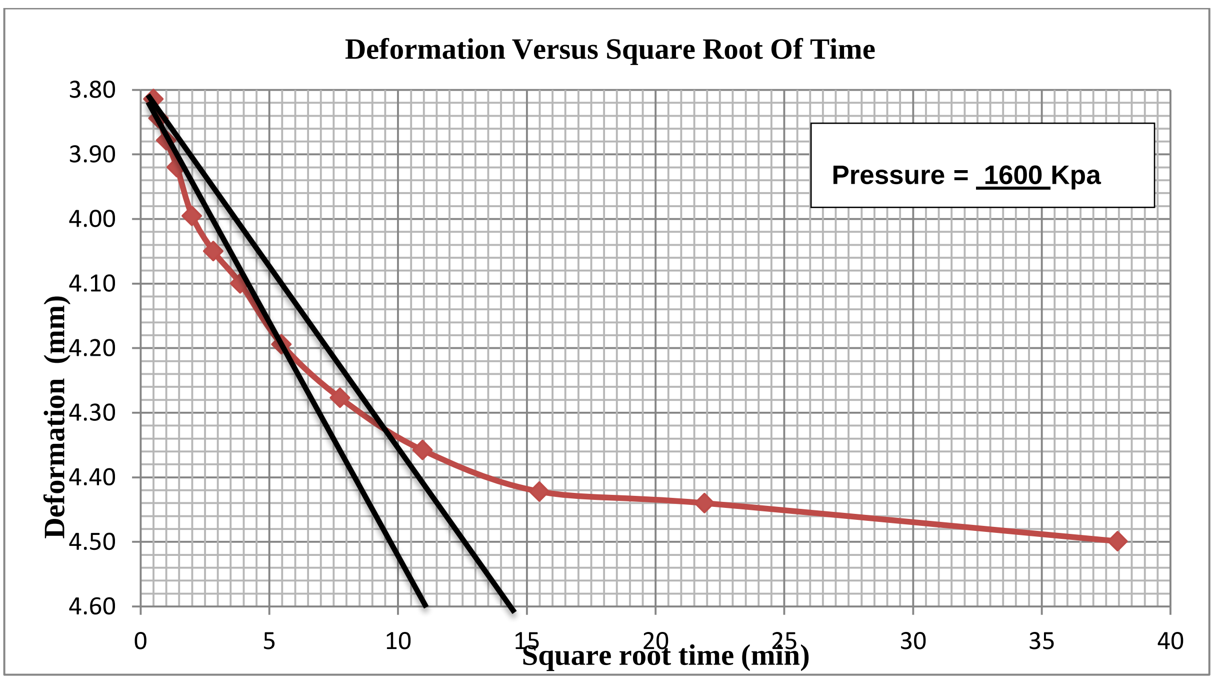


Figure E -60 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Mojo town, Sample 15 (P=1600)

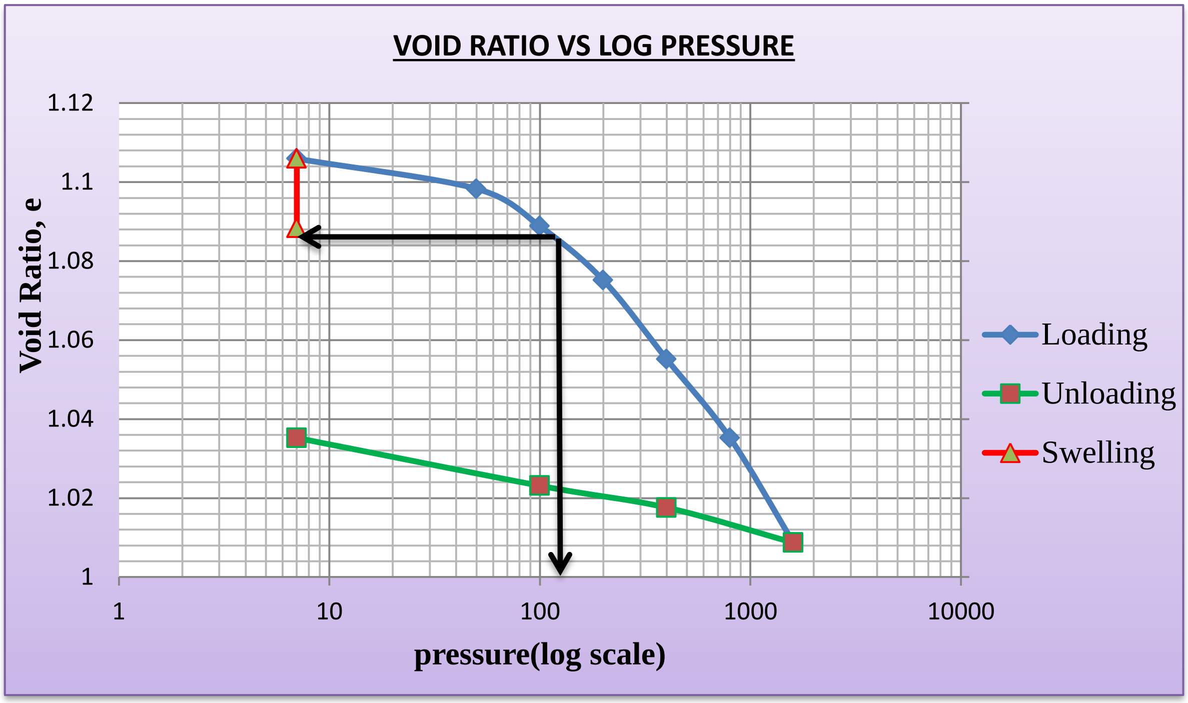


Figure E -62 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.33, w=22.82%) sample-1

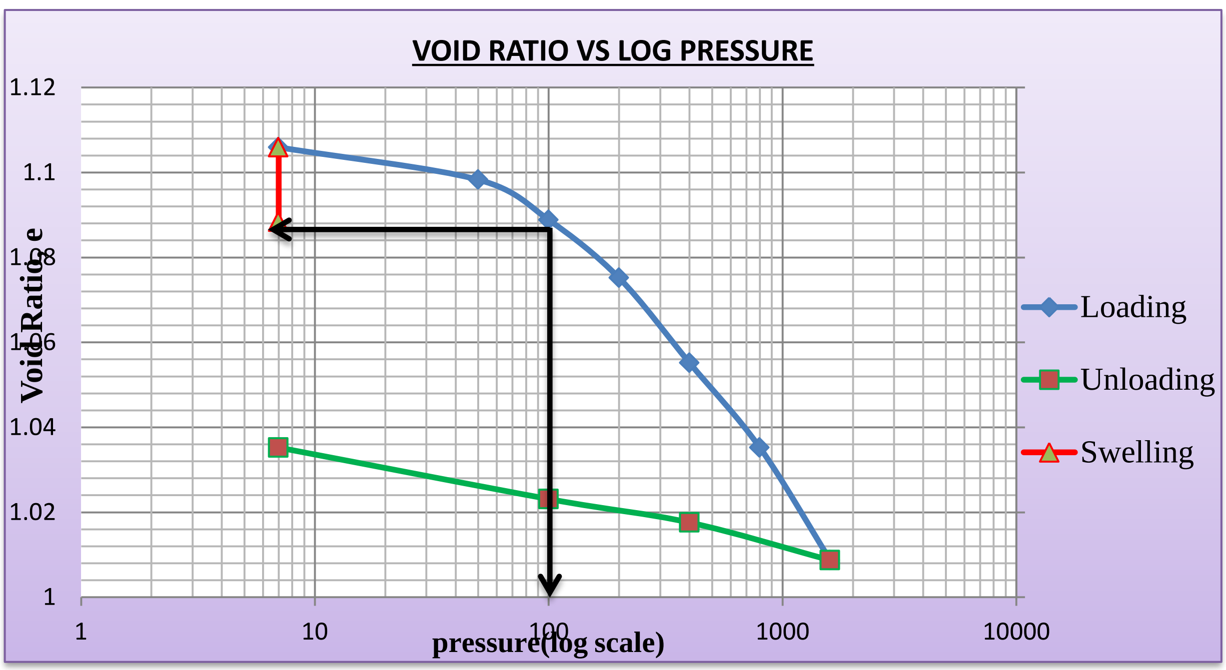


Figure E -63 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.071, w=48.38%) sample-2

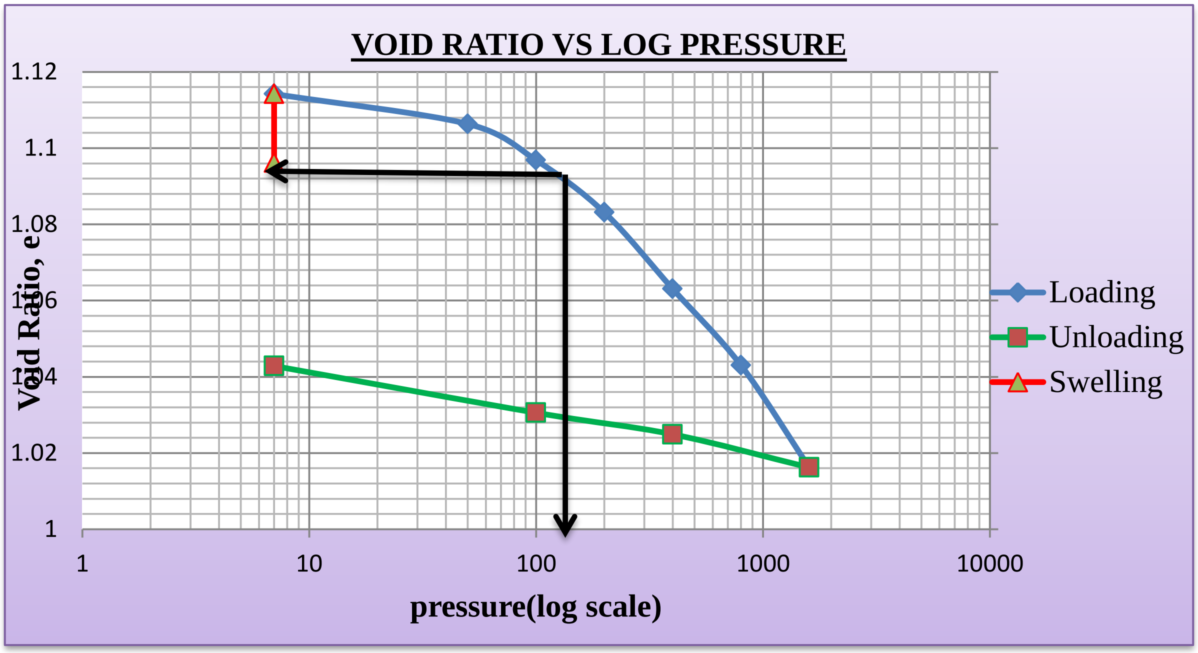


Figure F -64 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.076, w=47.81%) sample-3

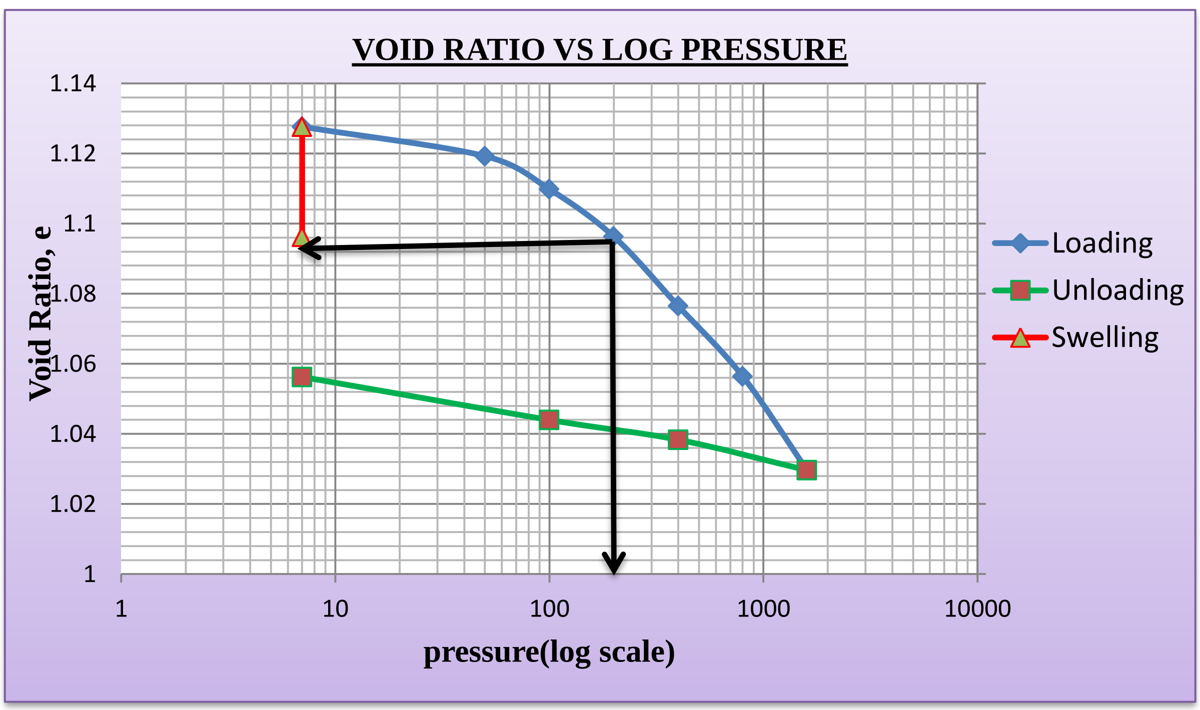


Figure E -65 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.065, w=48.12%) sample-4

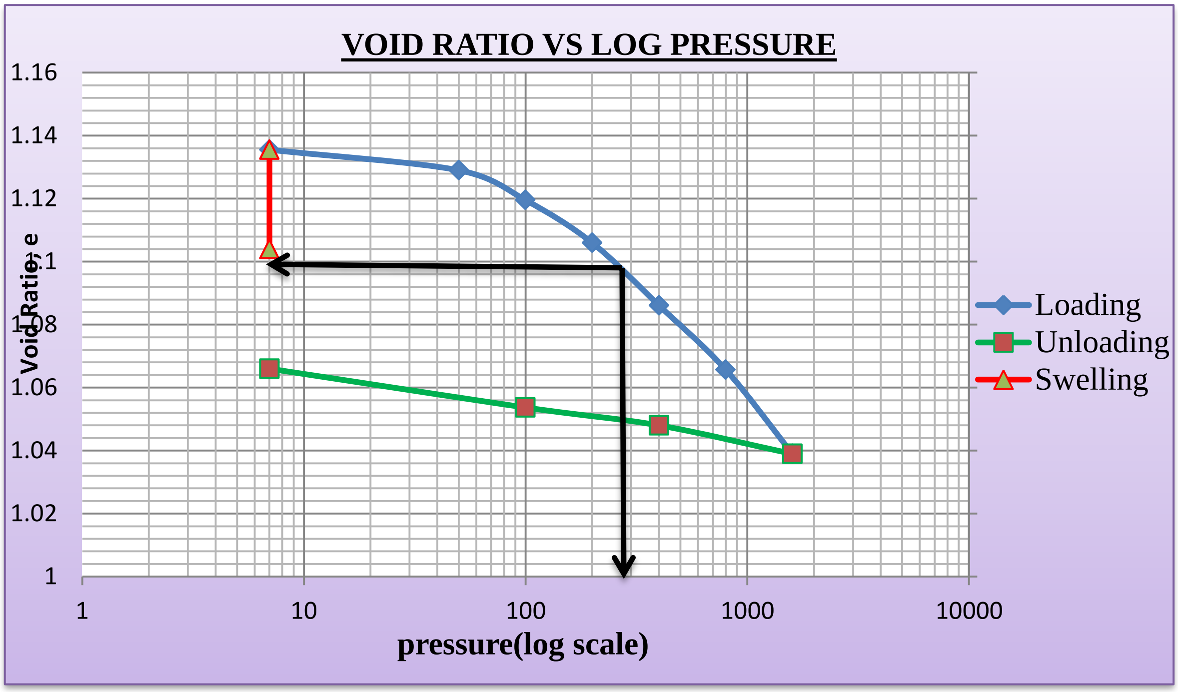


Figure E -66 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.079, w=48.31%) sample-5

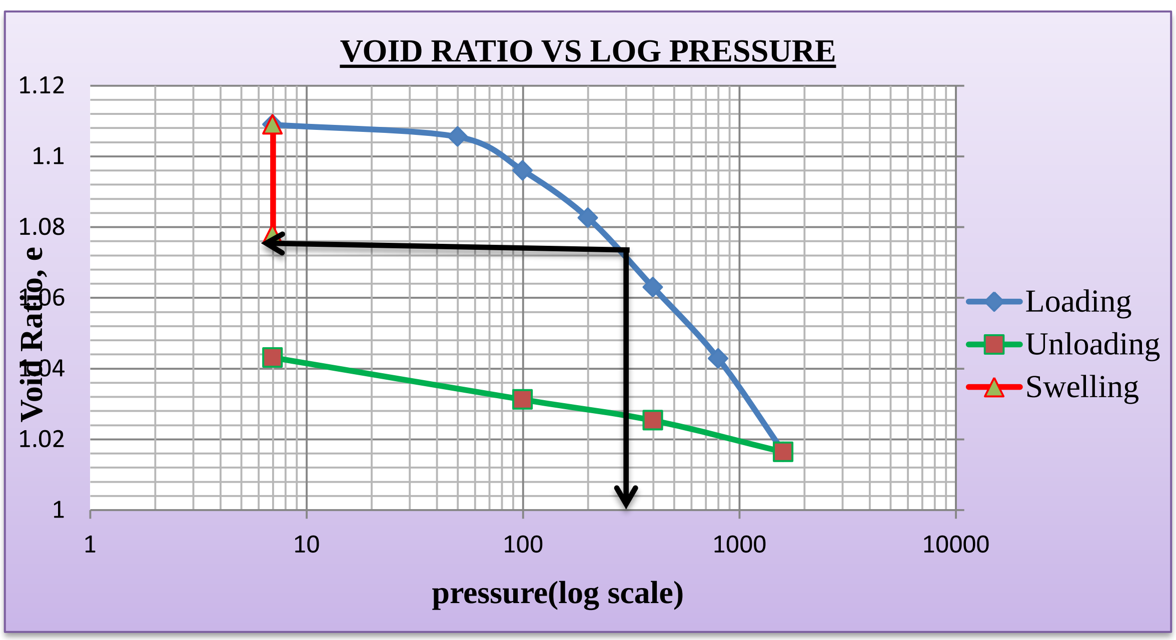


Figure E -67 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.082, w=48.21%) sample-6

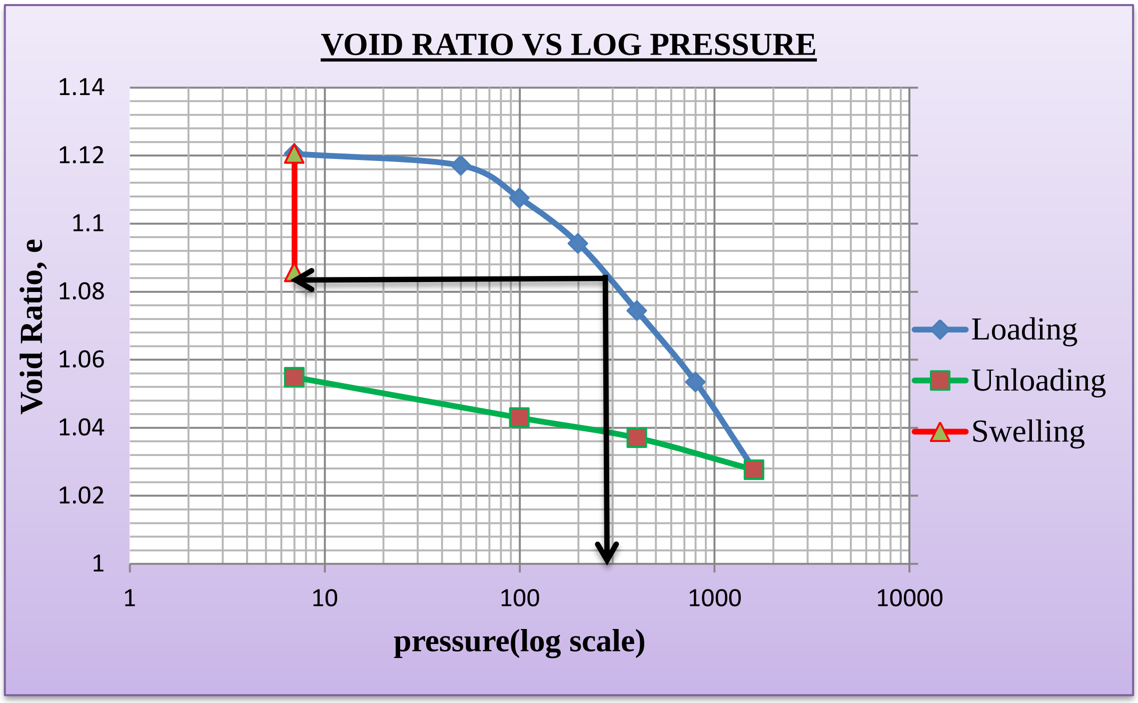


Figure E -68 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.091, w=47.81%) sample-7

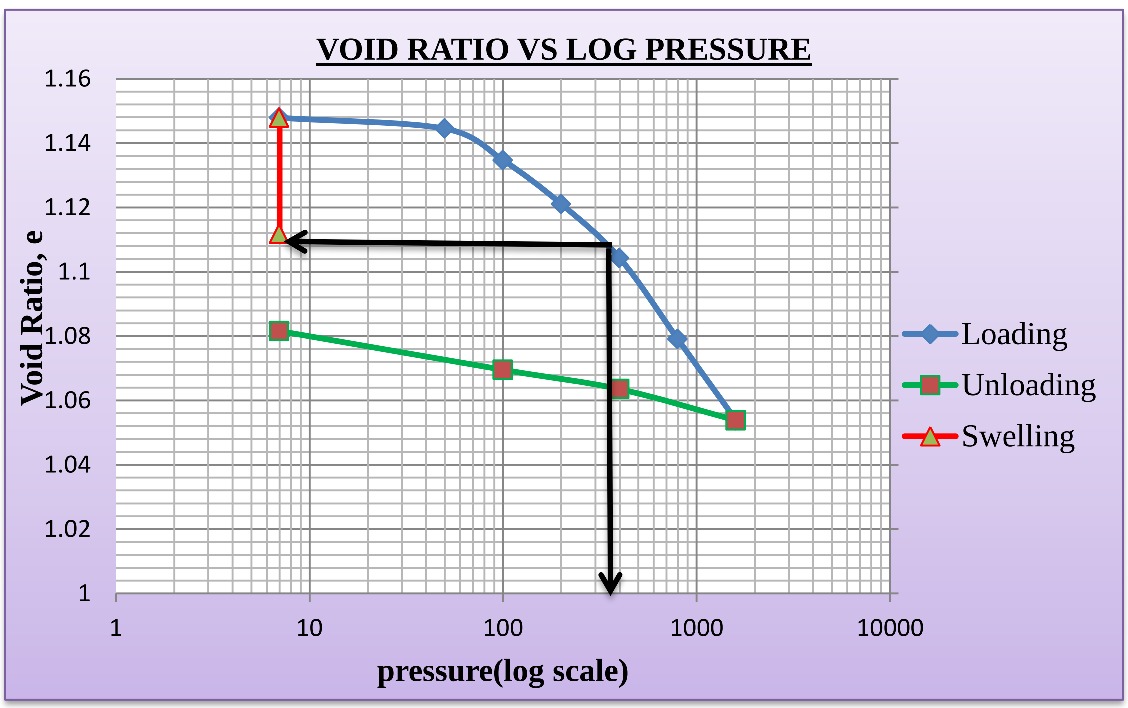


Figure E -69 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.095, w=47.61%) sample-8

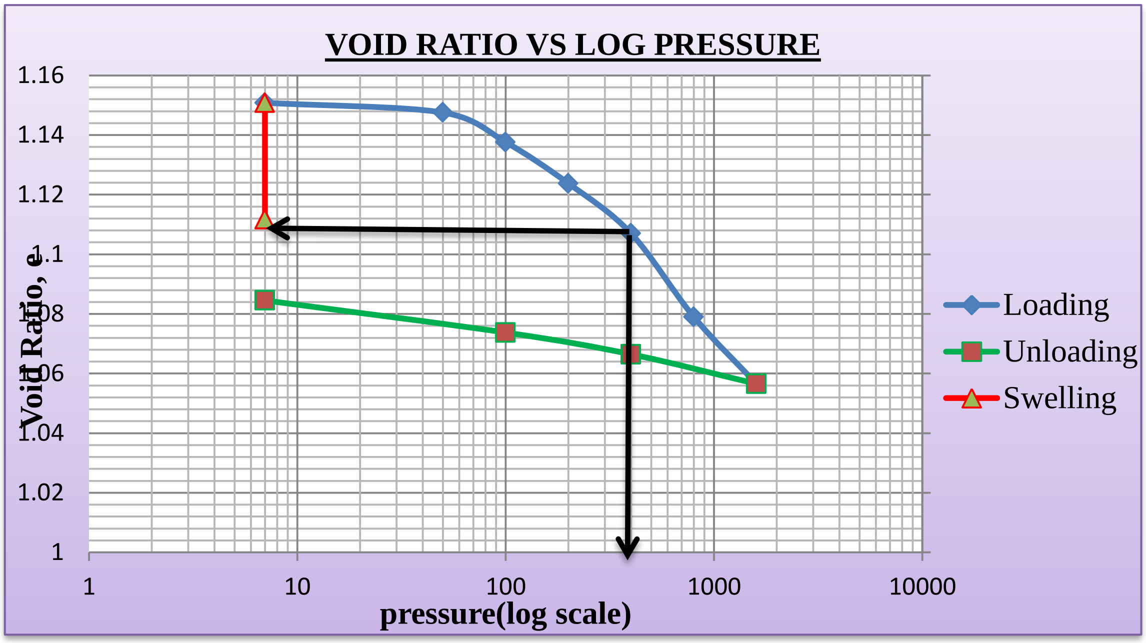


Figure E -70 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.101, w=47.47%) sample-9

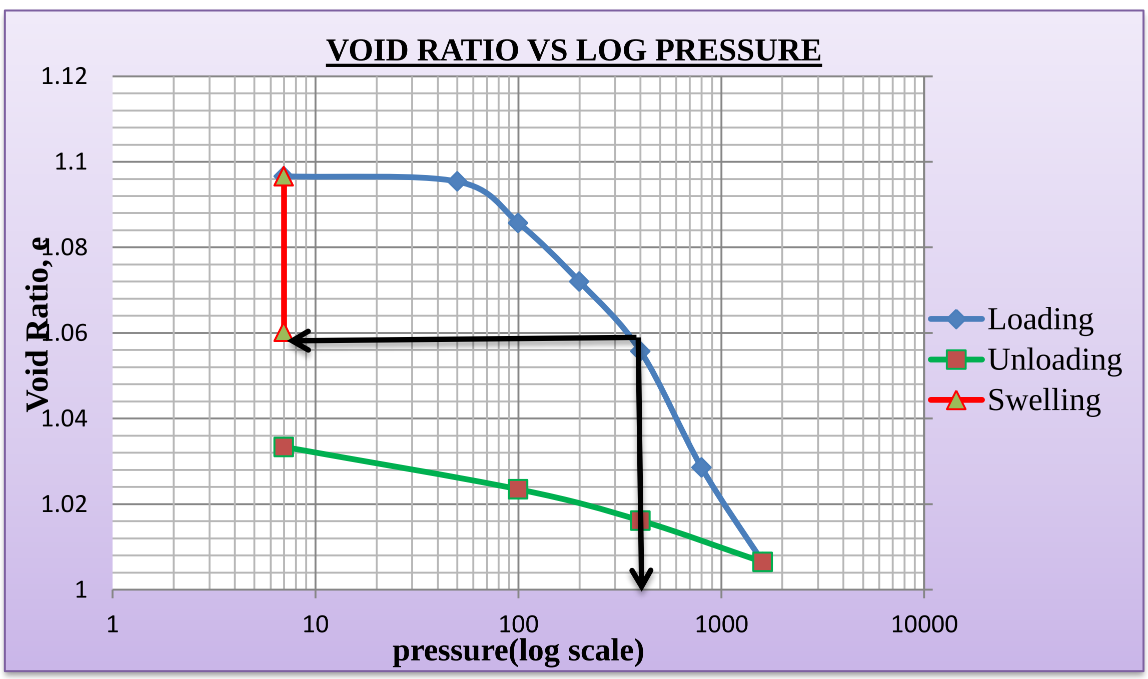


Figure E -71 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.110, w=46.41%) sample-10

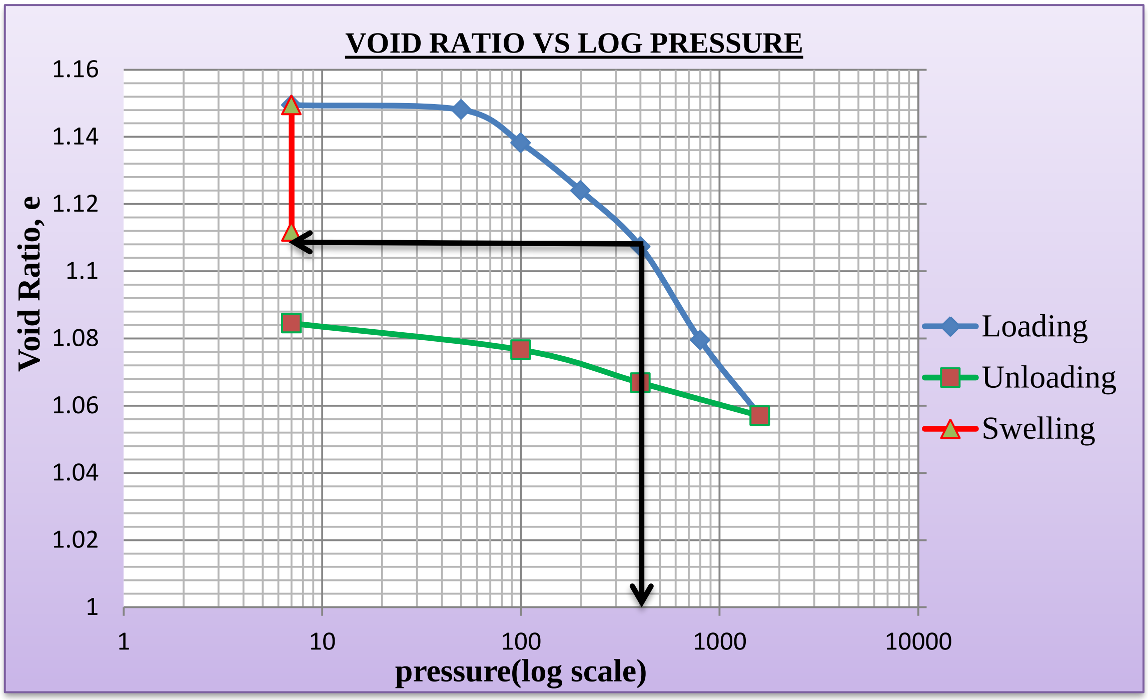


Figure E -72 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.116, w=46.48%) sample-11.

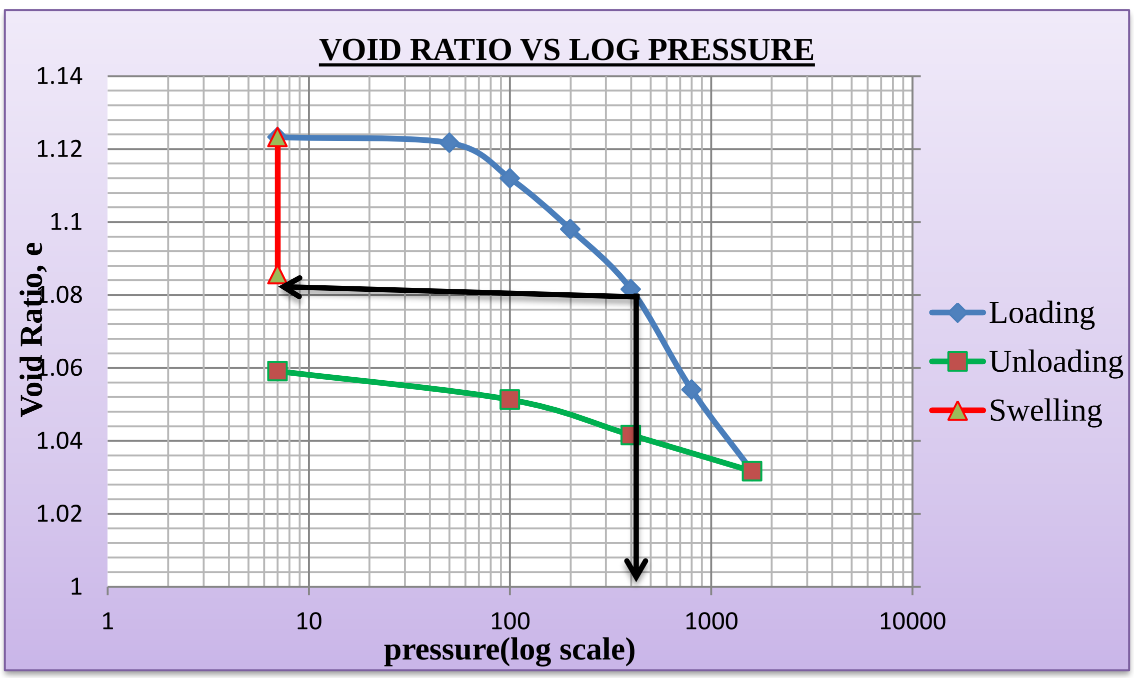


Figure E -73 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.126, w=45.70%) sample-12.

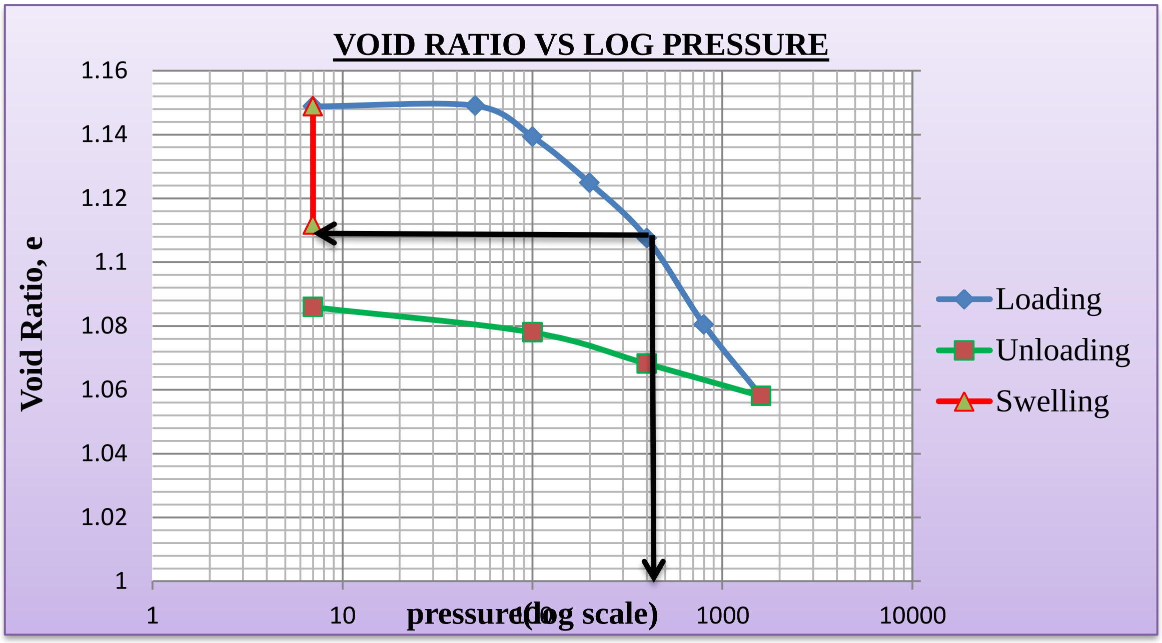


Figure E -74 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.057, w=49.66%) sample-13.

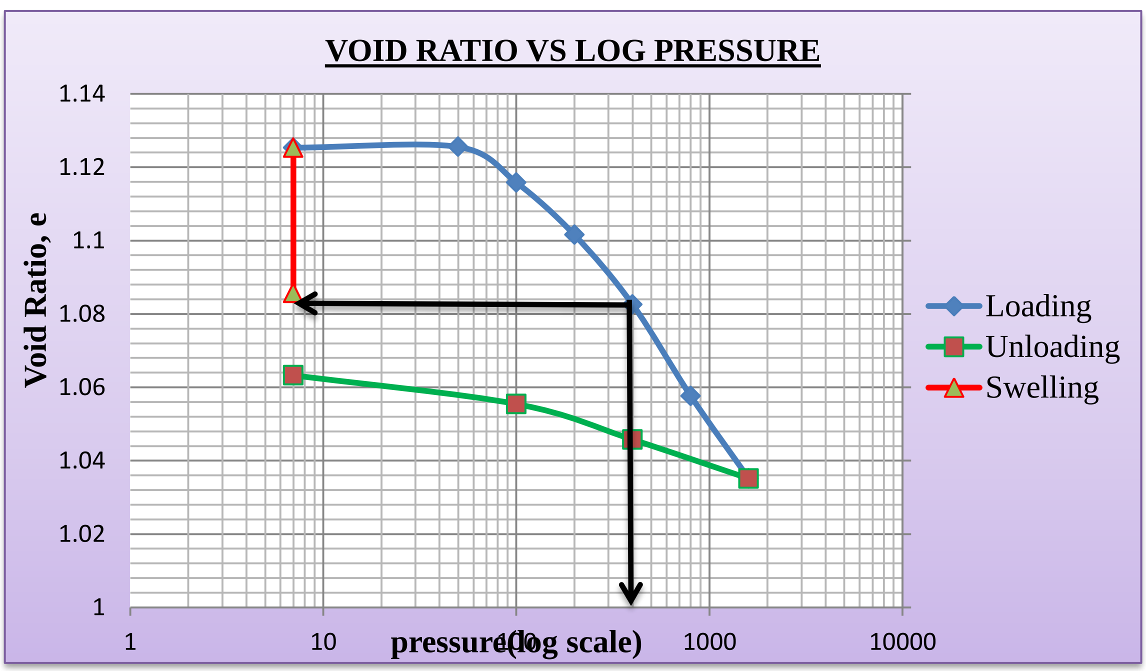


Figure E -75 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.088, w=45.99%) sample-14.

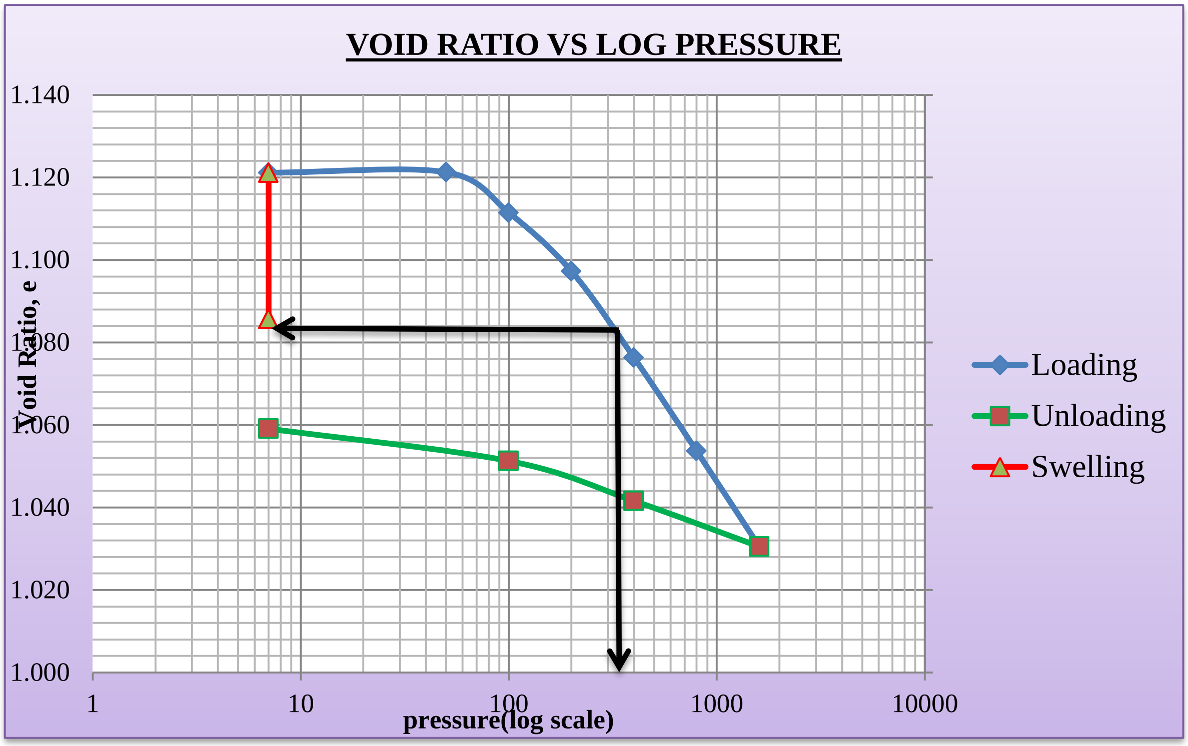


Figure E -76 Plot of Void ratio verses log Pressure of Remolded samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1..100, w=45.46%) sample-15.