ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

RELATIONSHIP BETWEEN SWELLING AND CONSOLIDATION CHARACTERISTICS OF EXPANSIVE SOILS OF GALAN TOWN

BY

AKLILU FIKADU

A thesis submitted to the school of graduate studies, Addis Ababa University, in partial fulfillment of the requirements for the Degree of Masters of Science in Civil Engineering

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(Geotechnical Engineering)

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr. Ing. Samuel Tadese and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged

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Abstract

Under the action of heavy loads, greater than the swelling pressure, the soil undergoes consolidation. The rate of consolidation depends on the applied load, coefficient of permeability, thickness, and compressibility of the soil.

The objective of this thesis is, therefore, mainly intended to investigate the relationship between swelling & consolidation behavior of expansive soils of Galan 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 Galan town & a series of swell-consolidation tests were carried out in the laboratory.

According to the test result obtained from Index property tests, the expansive soil of Galan Town, according to USCS is classified as Inorganic Clays soil of high plasticity, CH, & A-7-5 according AASHTO soil classification.

The value of compression index C_c obtained from expansive soils of Galan Town, ranges from 0.24-0.34 & the value of coefficient of consolidation C_v ranges from 0.32-0.42 m^2/year, while initial dry density & initial moisture content range of 1.17-1.37 g/cm3 & 30.6-50.1% respectively during the sample collection

In the course of this research, it has been found that swelling potential of expansive soils decrease with increase in initial moisture content for constant dry density.
On the other had the percent swell & swelling pressure increase with increasing initial dry density increases. Maximum swelling pressure is obtained from the samples prepared with optimum moisture content & maximum dry density.

The value of compression index C_c of sample with high swelling potential (high degree of disturbance) is greater as compared to samples with low swelling potential. Increase in moisture content decreases the amount & rate of both swell and consolidation.

Therefore, the consolidation characteristic of expansive soil is not inherent property of the soil mass because, swelling is also additional factor that can influence consolidation characteristics expansive soil.

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List of Nomenclature

C_c	Compression index
C_v	Coefficient of consolidation
e	Void ratio
Δe	Change in void ratio
H	Length of drainage path
k	Coefficient of permeability
m_v	Coefficient of volume compressibility
P	Pressure due to applied load
P_c	Preconsolidation pressure
P_o	Initial overburden pressure
Δp	Change in effective stress
S_c	Consolidation settlement
S_i	Immediate settlement
S_s	Secondary settlement
t_{50}	Time during which 50% of consolidation takes place
t_{90}	Time during which 90% of consolidation takes place
γ_d	Dry unit weight
γ_w	Unit weight of water
w	Moisture content
ρ_d	Dry density
S	Degree of Saturation
G_s	Specific Gravity

List of Acronyms

AASHTO	American Association of Highway and Transportation Officials
ASTM	American society for testing and materials
AAU	Addis Ababa University
Ac	Activity number
CEC	Cation Exchange Capacity
PVC	Potential Volume Change
SL	Shrinkage Limit
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
No.	Number
LL	Liquid limit
PI	Plasticity index
PL	Plastic limit
MDD	Maximum dry density
OMC	Optimum moisture content
S	Settlement
T	Time
NMC	Natural moisture content
CH	Inorganic highly plastic clays
OH	Organic silts and organic clays of high plasticity
OL	Organic silts and organic clays of low plasticity
CL	Inorganic clays of low plasticity
ML	Inorganic Silt of low plasticity
MH	Inorganic silt of high plasticity
SM	Silty sand
SW	Well-Graded Sand
GW	Well-Graded Gravel
OCR	Over Consolidation ratio
FS	Free swell
TP	Test Pit

1. Introduction

1.1 General

Expansive soils are clay soils with high plasticity commonly which are black or gray in color. They are characterized by their nature of expansion or shrinkage upon changes in moisture content. Foundations constructed on these clays are subjected to large up lifting forces caused by the swelling. These forces will induce heaving, cracking, and breakup of different structures. Most of structural damages due to expansive soils result from the differential rather than the total movement of the foundation soil as a result of swell. Damage can occur within a few months following construction, may develop slowly over a period of few years, or may not appear for many years until some activity occur to disturb the soil moisture equilibrium.

Expansive soils are distributed very widely over almost all geographical locations in the world, causing distress to the structures founded on them and discomfort to the users.

These soils are major problematic soils in Africa and other parts of the world. Our country Ethiopia is one of the victims of black and gray expansive soils. The expansive soils of Ethiopia are residual, derived from weathering of basic volcanic rock. In the capital Addis Ababa it has been known that expansive soils cover large part of the city, in which recent construction areas are extensively covered with expansive soils

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. [19]

Therefore, it is important to make localized study for the different areas.

1.2 Objective of the study

The main objective of this thesis is to examine the relationship between swelling and consolidation characteristics of expansive soils of Galan Town under different initial moisture content & dry density using one dimensional consolidation test method. In addition, the ranges of values of the consolidation parameters of expansive soil of Galan town are determined.

1.3 Methodology

In order to achieve the objective of this thesis, researches conducted on expansive soils of Ethiopia is reviewed; disturbed and undisturbed samples of expansive soils were collected from different sites of Galan Town. The following laboratory tests were conducted to attain the purpose of this work.

- 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.
- Swell & Swell pressure test by varying initial moisture content

1.4 Organization of the Thesis

The thesis is organized and presented in 6 chapters. Chapter 1 highlights introduction of the thesis. Chapter 2 deals with review of published literatures. In Chapter 3 description of the study area is presented. Chapter 4 comprises of the course of sample collection, preparation & results obtained from the tests conducted. In chapter 5 discussions on the test results were made. Conclusion and recommendations were given in chapter 6.

2. Literature Review

2.1 Origins of Expansive Soils & Parent Material

The parent materials associated with expansive soils are classified into two. The first group comprises of the basic igneous rocks, which are comparatively low in silica, generally about 45% to 52%. Rocks which are rich in metallic base such as the pyroxenes, amphiboles, biotitic and olivine fall within this category. Such rocks include the gabbros, basalts and volcanic glass.

The second group comprises of the sedimentary rocks that contain montmorillonite as constituent including shale and clay stones. Limestone and marls, rich in magnesium can also weather to clay. These constituents of the shale and clay stones contain varying amount of volcanic ash and glass, which were subsequently weathered to montmorillonite. Some of the fine grained sediments which accumulated to form these rocks also contain montmorillonite derived from weathering of continental igneous rocks and from ash, which fell on the continental areas as clouds of ash from volcanic eruptions can fall on continents and sea.

The constituents of the parent materials during the early and intermediate stages of the weathering process determine the type of clay formed. The nature of the parent material is much more important during these stages than after intense weathering for long periods of time. [19]

2.2 Characteristics of 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. Swelling clays can control the behavior of virtually any type of soil if the percentage of clay is more than about 5 percent by weight. Soils with smectite clay minerals, such as montmorillonite, exhibit the most profound swelling properties. [17]

Subsequent swelling and shrinkage of this soils due to change in moisture cause damages to civil engineering structures, particularly light structures which could not counteract the swelling pressure of the soil. The origin of expansive soils is related to a combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup 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, which determine the clay mineralogy, include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected. [19]

2.3 Distribution of Expansive soil

Potentially expansive soils can be found anywhere in the world. In the underdeveloped nations much of the expansive soil problems may not have been recognized. It is to be expected that more expansive regions will be discovered each year as the amount of construction increase. The countries where the expansive soil were reported are Argentina, Australia, Burma Canada, Cuba Ethiopia, Ghana, Ethiopia, India Israel, Iran, Mexico, Morocco Rhodesia, South Africa, Spain Turkey, U.S.A, Venezuela. 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 [7]

2.4 Clay Mineral

Most soil classification systems arbitrarily define clay particles as having an effective diameter of two microns (0.002 mm) or less. Particle size alone doesn't determine clay minerals. Probably the most important grain property of fine grained soils is the mineralogical composition. The three most important groups of clay minerals are montmorillonite, illite and kaolinite, which are crystalline hydrous aluminosilicates. Montmorillonite is the clay mineral that presents most of the expansive soil problems. Absorption of water by clay leads to expansion. From mineralogical standpoint the magnitude of expansion depends up on the kind and amount of clay mineral present, their exchangeable ions, electrolyte content aqueous phase and the internal structure. [7]

2.5 Formation of clay mineral

The clay minerals are formed through complicated process from an assortment of parent materials. The parent materials include feldspar, mica and limestone. The alteration process that takes place on land is referred to as weathering and that on sea floor or lake bottom as halmyrolysis. The alteration process includes disintegration, oxidation, hydration, and leaching. The setting for the formation of montmorillonite is extreme disintegration, strong hydration, and restricted leaching. The parent material for formation of montmorillonite often consists of ferromagnesium mineral calcic feldspar, volcanic glass and many volcanic rocks. Bentonite is clay composed primarily of montmorillonite which has been formed by the chemical weathering of volcanic ash. [7]

2.6 Clay Structure

There are two fundamental molecular structures as the basic units of the lattice structure of clay soils. These are the silica tetrahedron and the alumina octahedron. The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions. The alumina octahedron consists of an aluminum atom surrounded octahedrally by six oxygen ions. When each oxygen atom is shared by two tetrahedral, a plate-shaped layer is formed. Similarly, when each aluminum atom is shared by two octahedron, a sheet is formed. The silica sheets and the aluminum sheets combine to form the basic structural units of the clay particles. [7]

The basic building block of clay minerals is the silica tetrahedron and the alumina octahedron. The block combines into tetrahedral and octahedral sheets to produce the various types of clays.

Kaolinite has a structural unit made up of alumina sheets joined to silica sheet and is symbolized as indicated in Fig. below. Kaolinite consists of many such layers stacked one on top of the other.The bond that exists between layers is tight and hence it is difficult to separate the layers. As a result kaolinite is relatively stable & water is unable to penetrate between the layers. Consequently kaolinite shows little swelling on wetting. [2]

Illite has a basic structure similar to that of montmorillonite. However, the basic illite units are bounded 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. [2]

Montmorillonite is a three layer clay mineral having a single octahedral sheet sandwiched between two tetrahedral sheets to give a 2 to1 lattice structure. It is the most common of all clay minerals and is well known for its swelling properties. This basic structure consists of an alumina sheet sandwiches between two silica sheets. The basic montmorillonite units are stacked one on the other, but the bond between individual units is relatively weak so that water can easily able to penetrate between the sheets and cause their separation and hence swelling.[2]

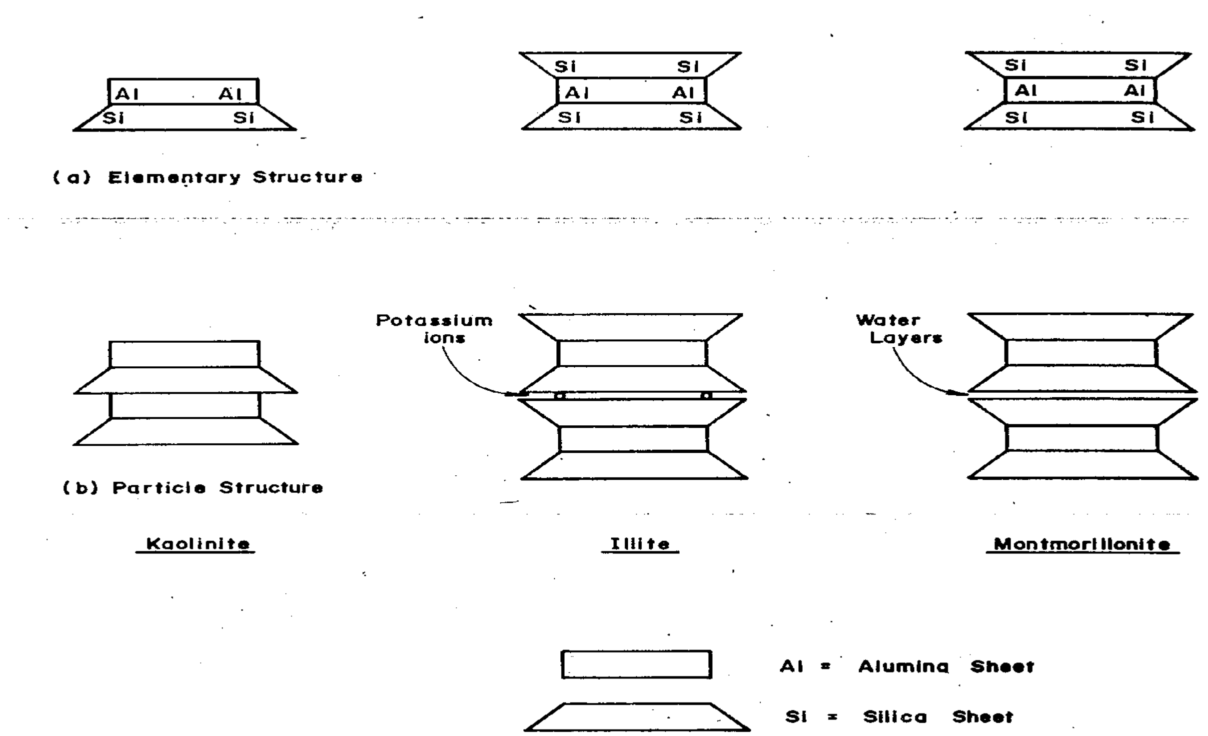


Figure 2.1: Symbolic Structure of clay minerals [2]

2.7 Identification and classification of Expansive soils

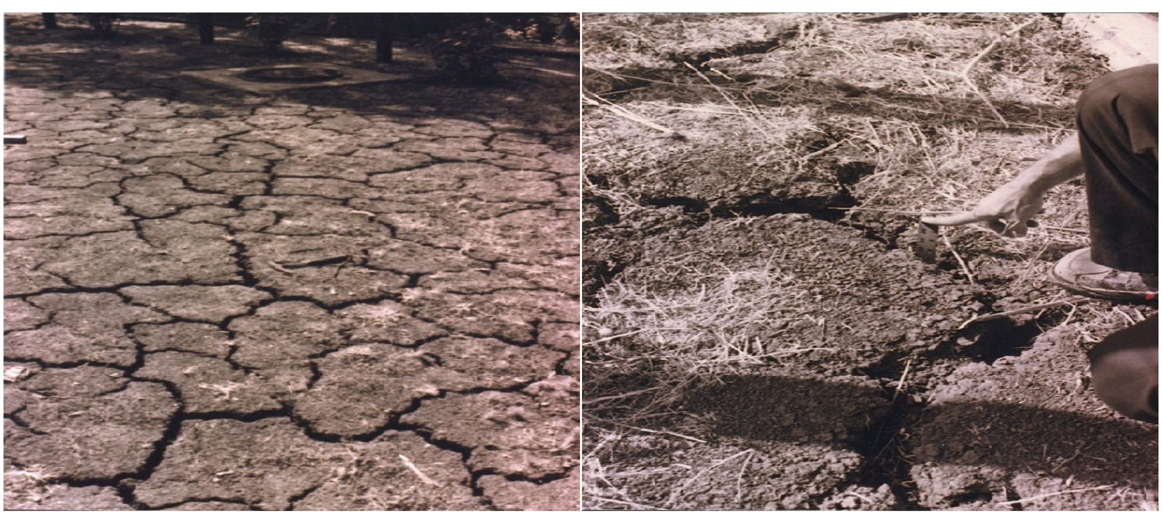
2.7.1. Identification of Expansive Soils

A. Field identification:

Currently it is evident that soil deposits can be recognized in the field through visual inspection. Some of the important field identification methods that indicate the potential for expansiveness of a soil are the following. [16]

Polygonal patterns of surface cracks in the dry season, Crack 2.5 cm wide and over 1 m depth is not uncommon. The cracks close down after rainy season (Figurer 2.2 a, b and c). Soils that can exhibit high swelling potential can be identified by field observations, mainly during reconnaissance and preliminary investigation stages. Important observations include: [19]

- Wide or deep shrinkage cracks
- High dry strength and low wet strength
- Stickiness and low traffic ability 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
- A shiny surface is easily obtained when a partially dry piece of the soil is polished with a smooth object such as the top of a fingernail
- The wet samples of the soil are sticky and it will be relatively difficult to clean the soil from the hands
- They are very hard when dry
- They usually have a color of black and/or gray
- As a rough guide, the presence of distinct cracks on light weight buildings



(a) (b)

Figure 2.2: Cracks in dry season. (a) Polygonal pattern of surface crack in dry season. (b) Deep crack in dry season [17]

B. **Laboratory identifications**

There are three different method of identifying potentially expansive soils.

i. Mineralogical Identification

Mineralogical identification can be useful in the evaluation of the material but is not sufficient in it when dealing with natural soils. The various methods of mineralogical identification are important in a research laboratory in exploring the basic concepts of clays but are impractical and uneconomical for practicing engineers. The techniques which may be used for mineralogical identification are listed below: [19]

- X-ray diffraction,
- Differential thermal analysis,
- Dye adsorption,
- Chemical analysis,
- Electron microscope resolution.

ii. **Indirect methods**

This method includes the index properties, Cation Exchange Capacity (CEC), and Potential Volume Change (PVC) test, activity method which are valuable tools in evaluating swelling property. Atterberg Limit, Grain Size Analysis and Free Swell tests are some, among the index property tests. [30]

The grain size analysis helps to determine the amount of colloidal sized particles existing in a soil. The term colloid describes a particle whose behavior is controlled by surface force (i.e. electrostatic and adsorptive force) rather than by gravitational force and smaller than 0.001 mm in diameter. These colloidal particles greatly influence the plasticity characteristics and volume change behavior of the soil. In this identification method, Atterberg limit and clay contents are combined to define activity. [30]

iii. Direct measurement

The direct measurement, offers the most useful data for a practicing engineer. The tests are simple to perform and do not require any costly and exotic laboratory equipment. The testing should be performed on number of sample rather than of a few to avoid erroneous conclusion. Direct measurement of expansive soils can be achieved by the use of conventional one-dimensional consolidometer which is available in most soil mechanics laboratories. [19]

2.7.2. Classification of expansive soils [1]

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 (w_L) 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 w_L). 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. [1]

A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas. Hence, it is very important to emphasize that design decision has to be based on predicting testing & analysis, which provide reliable information. [10]

A review of the identification and classification systems for expansive soils that appear in the technical literature follows:

<u>Skempton</u> [25]

Activity, A, the ratio of the plasticity index divided by the percent clay (% < 2 μm), was first defined and used by Skempton. This property has been used by various investigators to classify soils with regard to potential expansion is as follows.

A= PI / % < 2 µm

Activity, A	Rating potential expansion
< 0.75	Low
0.75 - 1.25	Medium
> 1.25	High

Chen [25] [12]

To simplify the USBR Method, a correlation is made between swell data and %< No. 200 sieve, liquid limit, and standard penetration resistance (ASTM 01586, AASHO T206).

< No. 200 Sieve, %	LL, %	Standard Penetration Blows/ft	Expansion, %	Probable Degree of Expansion
< 30	< 30	< 10	< 1	Low
30-60	30-40	10-20	1-5	Medium
60-95	40-60	20-30	3-10	High
> 95	> 60	>30	> 10	Very High

The method is based on data collected from a wide variety of samples and is very simple to use, i.e., all that is required is the natural water content and liquid limit. However, experience with regard to application of the method is relatively limited.

Krazynski [25]

A proposed test method for directly measuring expansion under a set of standard conditions is presented. The computed expansion index is then used to classify the soil for use beneath concrete slabs as follows:

Expansion Index	Potential Expansion
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
> 130	Very High

The expansion index is developed in conjunction with work on residential slabs and is intended only for classification purposes. There is little experience with this method, and no data for comparison with other methods are available.

Fernando, Smith, and Arulanandan [25]

With the method described earlier by Arulanandan, a comparison is made between expansion index and the magnitude of dielectric dispersion. The correlation is good for the soils tested and the authors establish the following criteria:

Magnitude of Dielectric Dispersion	Expansion Index	Potential Expansion
1-10	1-20	Very Low
11-25	21-50	Low
26-45	51-90	Medium
46-65	91-130	High
>65	>130	Very High

2.8. Mechanics of swelling

2.8.1. General

Swell is the process of imbibing available moisture to cause an increase in soil volume until the pore water pressure increases to an equilibrium determined by the environment. The amount of swell to satisfy the new pore pressure equilibrium depends on the magnitude of the vertical loading and soil properties that include soil composition, natural water content, density and soil structure. The rate of swell depends on the coefficient of permeability (hydraulic conductivity), thickness, and soil properties. The vertical confining pressure required to prevent volume expansion from absorbed moisture is defined as the swell pressure.

The swell and swell pressure are generally determined in the laboratory with the one-dimensional consolidometer. Swell is determined by subjecting the laterally confined soil specimen to a constant vertical pressure and by giving both the top and bottom of the specimen access to free water to cause swell. The swell pressure is determined by subjecting the laterally confined soil specimen to increasing vertical pressures, following inundation, to prevent swell. [29]

Swelling of expansive soils takes 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 and with the introduction of water volumetric expansion also takes place. [7]

2.8.2. Moisture Migration

The most common moisture transfer is by gravity. The seepage of surface water, precipitation and snow melting in to the soils are common example. The moisture migration can occur in all direction. Under artesian conditions, the flow can be upward. In stiff clays and in shale bedrock, the flow generally occurs in the bedding plans or follows continuous fractures and fissures. Shrinkage cracks which develop due to surface desiccation provide easy access of water in to the deep soil.

It is well recognized by observant soil engineers 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. Water vapor at a temperature higher than its surroundings will migrate toward the cooler area to equalize the thermal energy of the two areas. When water reaches the cooler area, generally the covered area beneath the structure, condensation can take place and provide sufficient moisture to initiate swelling.

Thermal gradient can also cause moisture migration through the liquid phase of the soils, Experiments conducted show that a temperature difference of 1^0 c was at least equivalent to a hydrostatic head of 3 feet in its ability to cause moisture migration. [7]

2.8.3. Factors influencing Swelling

2.8.3.1 Physical Properties of Expansive Soils

Physical properties of expansive soils determine the behavioral characteristics of the material. In many cases, attempts have been made to isolate the individual properties and explain the behavior on the basis of a single property or a combination of single property contributions. However, in both the laboratory and field situations, the actual behavior is a function of combinations and interrelationships among the properties. The following discussions are based on a categorization of the physical properties in order to point out and explain some of these interrelationships: (1) Inherent properties of the materials which contribute to or influence the actual volume change and (2) Environmental conditions, which enhance the probability and magnitude of expansivity and apply more to in situ materials. [13]

2.8.3.1.1 Inherent properties

The inherent properties which influence the behavior of expansive materials are presented in the following paragraphs. Confinement, time, and temperature are not inherent properties; however, they are factors which influence the role of the inherent properties in determining the amount and rate of volume change in both laboratory and in situ conditions. [13]

i. **Soil Composition**

This includes the type and amount of clay mineral within the soil and the size and specific surface area of the clay minerals. The type and amount of clay mineral are the inherent factors which determine whether or not the material will expand. In other words, the potential for volume change rests on the mineralogic composition; and the remaining inherent factors, combined with the ambient environmental conditions, determine the extent or magnitude of volume change.

Clay mineral size is not an independent parameter, but often is a characteristic of the specific clay mineral. For example, montmorillonite occurs as extremely small particles which may be considered colloid. On the other hand, kaolinite may occur as rather large particles which may be of a, fine silt size. Chlorite, vermiculite, illite, and mixed-layer clays are generally intermediate in size between montmorillonite and kaolinite. [13]

ii. **Soil characteristics**

Soil characteristics may be considered either as microscale or macroscale factors.

Microscale factors include the mineralogical and chemical properties of the soil. Macroscale factors include the engineering properties of the soil which in turn dictated by the microscale factors.

a) Microscale factors (clay mineralogy and soil water chemistry):- clay minerals of different types typically exhibit different swelling potentials because of variation in the electrical field associated with each mineral. The swelling capacity of an entire soil mass depends on the amount and type of clay mineral in the soil, the arrangement and specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles. In dry soils, salt cations are held close to the clay crystal surfaces by strong electrostatic forces. As water becomes available, Cation hydration energies are sufficiently large to overcome inter-particle attraction forces. Thus initially desiccated and densely packed particles are forced apart as adsorbed cations hydrate and become enlarged on the addition of water. [20]

b) Macroscale factors (plasticity and density):- Macroscale soil properties reflect the microscale nature of the soil. Because they are more conveniently measured in engineering work than microscale factors, macroscale characteristics are primary indicators of swelling behavior. Commonly determined properties such as soil plasticity and density can provide a great deal of insight regarding the expansive potential of the soils. Soil consistency, as quantified by the Atterberg limits, is the most widely used indicator of expansive potential. Most expansive soils can exist in a plastic condition over a wide range of moisture contents. This behavior results from the capacity of expansive clay mineral to contain large amount of water between particles and yet retain a coherent structure through the inter particle electrical forces. Soil plasticity, a useful indicator of swell potential, is influenced by the same microscale factors that control the swell potential. [20]

iii. **Dry density**

The dry density is an important factor in determining the magnitude of volume change. The swell or swelling pressure of an expansive soil increases with increasing dry density for constant moisture content. The reason, simply stated, is that higher densities result in closer particle spacing, therefore causing greater particle, interaction. This particle interaction, or more precisely, double-layer water interaction, results in higher osmotic repulsive forces and a greater volume change. This holds true for both remolded and undisturbed materials. [13]

iv. **Soil fabric**

The soil fabric refers to the arrangement in space of the constituent particles. In the case of argillaceous sediments and rocks, the fabric consists of the arrangements of the plate like clay minerals with each other and with the non clay components. The type of clay mineral arrangement present will influence the amount and to some degree the direction (lateral or vertical) of volume change exhibited by an expansive material. Individual clay mineral platelets generally occur in either agglomerated or non agglomerated arrangements. Agglomerated arrangements consist of independent groups of platelets which may be associated in several ways, while non agglomerated arrangements are void of discernible groups and the fabric is uniform throughout. [13]

v. **Pore water properties**

The phenomenon of volume change in expansive soils is the direct result of the availability and variation in the quantity of water in the soil. Therefore, the properties of the water will have a significant influence on the expansive behavior. The volume change of expansive soils is primarily due to the hydration of the clay minerals or, more precisely, the adsorption of water molecules to the exterior and interior surfaces of the clay mineral to balance the inherent change deficiency of the particle. The degree of hydration is influenced by the amount and type of ions adsorbed on the particle and the amount and type of ions in the pore fluids. Pore fluids containing high concentrations of cations, i.e., soluble salts, tend to reduce the magnitude of volume change of an expansive soil. On the other hand, pore fluids with low ionic concentrations may actually leach out the charge balancing cations and cementing agents and render the soil more susceptible to volume change. [13]

vi. **Confinement & State of Stress**

The application of a surcharge or external load to an expansive material will obviously reduce the amount of volume change that is likely to occur. For in situ conditions, the presence of a layer of no expansive overburden material may eliminate the probability of damage from the underlying expansive material. It may be noted that confinement has its greatest influence on expansive soils in a stress-related sense (swelling pressure).The greater the confinement, the greater the stress and the smaller the deformation. Generally, the load applied by a pavement is far less than that required to maintain minimal deformation; therefore, problems with expansive clays in highway sub grades are more related to deformation. [13]

Volume change is directly related to change in the state of stress in the soil. A reduction in the total stress due to excavation of overlying material will result in rebound and heave of the surface. Stress history is another factor which affects the swelling characteristics in that an over consolidated soil is more expansive than the same soil of the same void ratio but normally consolidated. The thickness and location of potentially expansive layers in the profile also considerably influence potential volume change. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. [20]

vii. **Time**

The influence of time on volume change is another interrelated property which has its major impact on the rate at which expansion occurs. The time to the first occurrence of volume change and the rate of expansion are functions of the permeability of the soil and the availability of water. Expansion occurs as soon as moisture is made available and continues until an equilibrium condition is reached with regard to the source of water or the hydration of the clay minerals. [13]

viii. **Permeability.**

As indicated in the previous discussion on the influence of time on volume change, the permeability plays an important role in the time rate of volume change. The permeability is a function of the initial moisture content, dry density, and soil fabric. For compacted soils, the permeability is greater at the lower moisture contents and dry densities and decreases to some relatively constant value at about the optimum moisture content. Above optimum, the permeability is essentially constant. The obvious reason for this minimum permeability near the optimum moisture content and maximum dry density is that the voids available for moisture movement are at a minimum because of the close particle spacing. Above optimum, the interaction of the double-layer water also minimizes the voids necessary for moisture movement. For in situ expansive soils, the permeability is normally enhanced by such structural discontinuities as fissures, fractures, and desiccation cracks. [13]

ix. **Temperature**

The influence of temperature is primarily limited to its effect on the viscosity and specific gravity of the adsorbed water. Increases in temperature tend to depress the double-layer water, while temperature decreases result in double-layer expansion. Of more importance is the influence of temperature on the movement of moisture, both vapor and liquid, as a result of thermal gradients within the soil mass. Water vapor at a higher temperature will migrate toward a cooler area in an effort to equalize the thermal energy in the system. [13]

2.8.3.1.2 Environmental conditions

The environmental conditions which influence volume change are presented in the following paragraphs:

i. Soil profile

The properties of the soil profile which may enhance or influence volume change include the total layer thickness, variations in the thickness, depth below ground surface, and the presence of lenses and layers of more permeable materials. Obviously, the thicker the layer of expansive soil, the greater the total potential volume change providing a source of moisture is available throughout the layer. Variations in thickness of the layer will result in variations of the magnitudes of volume change, or more precisely, differential volume change. Differential expansion, just like differential settlement, is the major problem with regard to damage to structures. The depth of the layer below ground surface may actually be a positive influence since the deeper the material, the greater the confinement on the expansive soil. [13]

ii. Depth of desiccation

The depth of desiccation is important to the magnitude & rate of volume change. The thickness of the desiccated layer represents the material in which a moisture deficiency exists. In addition, the layer normally has a large number of avenues (desiccation cracks) available for movement of moisture into the material. The depth of desiccation is generally defined as the depth to which a difference exists between the equilibrium moisture content profile resulting from evaporation and the ambient soil moisture content profile in which the soil stratum is in equilibrium with its environment. Generally, the hotter and drier the climate, the greater the depth of desiccation is. Changes in the overburden conditions and the proximity of the groundwater table have an important influence on the depth of desiccation. To date, no absolute method exists for defining the value. [13]

iii. Depth of seasonal moisture variation

This comprises some thickness of the surface material which is influenced by seasonal variations in climatic conditions. As would be expected, the greater depths of seasonal moisture change occur in areas in which the seasonal climatic changes are greatest, i.e., long droughts followed by excessive rainfalls. Ambient temperature conditions also influence the depth of seasonal variations. During colder seasons, moisture from the lower warmer zones will accumulate closer to the surface and dissipate back to depths during the warmer seasons. Seasonal moisture variations are relatively constant for given climatic conditions; however, the general trend is toward accumulation of total moisture content. [13]

iv. Vegetative cover

Vegetation such as trees, shrubs, and some grasses are conducive to moisture movement or depletion by transpiration. In areas where vegetation is removed and structures such as pavements are replaced, the moisture that was being used by the vegetative cover will tend to accumulate beneath the structure and enhance the volume change. Vegetation with large root systems located in close proximity to pavements (i.e., city streets) will result in differential moisture conditions and thus differential volume change. [13]

v. Surface drainage characteristics

Poor surface drainage leads to moisture accumulation or ponding which can provide a source of moisture for expansive sub grades by infiltration through the verge slopes. Poor surface drainage is a frequent problem associated with highways on expansive soils and occurs in cut, grade, and uphill sides of transition sections. The extent of the infiltration is a function of the transverse and longitudinal gradients in the ditches and the type of material in the section. [13]

vi. Modes of moisture transfer.

In situ soils are generally considered to be a three-phase system; that is, soil particles, water, and air. In such a system it is possible for water to move either in the liquid phase or vapor phase, or a combination of both. For water to move in either phase there must be a driving force within the system to provide a mode of transfer. These modes are generally described as gravity, capillarity, and thermal gradients. Gravitational movement of water is primarily limited to the liquid phase which includes simple infiltration of surface water, lateral seepage from available sources, and the upward movement of the groundwater table. Transfer of water by capillarity is again primarily limited to the liquid phase. The zone of capillary rise is the layer of material directly above the submerged material within the influence of the groundwater table and can extend upward from the groundwater table for considerable distances depending on the effective pore sizes of the soil. Moisture transfer as a result of thermal gradients is applicable to both the liquid and vapor phases; with the vapor phase predominant. Water vapor at a higher temperature in the surrounding area will migrate to the cooler area in an effort to equalize the thermal energies of the system. As the vapor moves into the cooler area, it will condense and form a source of free water. [13]

vii. Sources of water

Descriptions of the modes of moisture transfer in above discussed some of the possible sources of water which cause volume change. Infiltration of rainfall through the pavement proper (i.e., porous material or cracks) or through the verge slopes is the primary source of free water. This is evident from the fact that seasonal moisture variations are directly dependent on the frequency and amount of rainfall. Lateral or vertical migration of moisture from sources such as the groundwater table is another possible source of free water. These are the naturally occurring sources of water; however, variations can be caused by man through such activities as irrigation, which could influence surface infiltration and groundwater conditions. Faulty or leaking subsurface utilities (i.e., water or sewage) could adversely affect ambient moisture conditions. Impoundment of reservoirs could have profound effects on groundwater conditions. [13]

2.9 Consolidation

2.9.1 General

Consolidation is the process of gradual transfer of an applied load from the pore water to the soil structure as pore water is squeezed out of the voids. The amount of water that escapes depends on the size of the load and compressibility of the soil. The rate at which it escapes depends on the coefficient of permeability, thickness, and compressibility of the soil. The rate and amount of consolidation with load are usually determined in the laboratory by the one-dimensional consolidation test. In this test, a laterally confined soil is subjected to successively increase vertical pressure, allowing free drainage from the top and bottom surfaces. [29]

2.9.2 Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. The soil layers being confined by the surrounding soil strata adjust to the new pressure mainly through deformation. Soil may be considered to be skeleton of solid grains enclosing voids which may be filled with gas, liquid, or a combination of liquid and gas.

The vertical compression of the soil under increased pressure is thus made up of the following components.

- A compression of solid mater which under usual loading is account for very small compression.
- A compression of the pore fluid, which may be considerable where the pores contain air, but negligible when the pore are completely filled with water.
- Reduction of the pore space by expulsion of pore fluid, which forms the major component of the compression. [2]

The settlement of foundation in cohesion less soil and the elastic settlement of foundation in clay can be assumed to occur as soon as the load is applied. The consolidation settlement of foundation on clay will only take place as water seeps from the soil at the rate depending up on the permeability of the clay. [15]

According to Terzaghi (1943), a decrease of water content of saturated soil without replacement of the water by air is called a process of consolidation. [11]

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. In this regard the one – dimensional vertical compression under structure is considered.

The settlement of saturated clay or organic soil can have three different components: immediate (also known as ‘‘initial settlement’’), primary consolidation, and secondary compression. [26]

a) Immediate Settlement

Immediate settlement is due to untrained shear deformations, or in some cases contained plastic flow, caused by the two- or three-dimensional loading. Common examples of three-dimensional loading are from square footings and round storage tanks.

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. [20]

b) Primary Consolidation

The increase in vertical pressure due to the weight of the structure constructed on top of saturated soft clays and organic soil will initially be carried by the pore water in the soil. This increase in pore water pressure is known as an excess pore water pressure. The excess pore water pressure will decrease with time as water slowly flows out of the cohesive soil. This flow of water from cohesive soil (which has a low permeability) as the excess pore water pressures slowly dissipate is known as primary consolidation, or simply consolidation. [26]

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.

c) Secondary consolidation

The final component of settlement is due to secondary consolidation, which is that part of the settlement that occurs after essentially all of the excess pore water pressures have dissipated. The reduction in volume continues at a very slow rate even after the excess pore water 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. [20]

2.9.2.1 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 where as 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. [27]

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 underconsolidated, normally consolidated, or over consolidated which are usually described by the over consolidation ratio (OCR) and defined as

$$OCR = \frac{Pc}{Po}$$

Where:

Pc is Preconsolidation pressure

Po is insitu overburden pressure

OCR is over consolidation ratio

a) Pre-consolidation pressure P_c

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

There are a few graphical methods for determining the pre-consolidation pressure based on laboratory test data. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log p curve of an undisturbed sample as shown in Figure 2.4. From B, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the Preconsolidation pressure, P_c. [9]

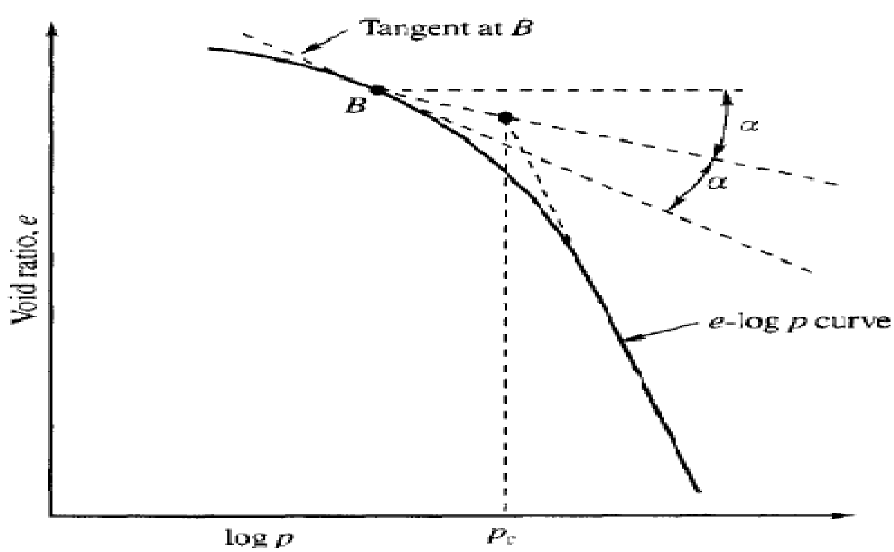


Figure 2.3: Different methods for determination of Pre consolidation pressure [21]

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.

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.

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 overconsolidation ratio of the soil is very important in the settlement analysis. For example, if the cohesive soil is underconsolidated, 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.

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. This means compressibility of soil decreases as effective stress increases

2.9.2.2 One-dimensional consolidation

The standard one-dimensional consolidation test is usually carried out on saturated specimen using an Odometer [11]. In this test a small representative sample of soil s 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, C_c, and the coefficient of consolidation, C_v, 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 [20].

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 2.5). 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. C_c 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

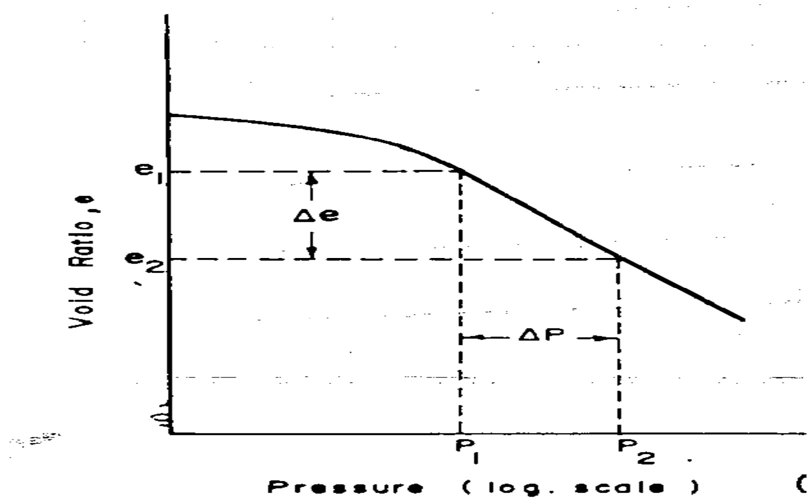


Figure 2.4: Determination of consolidation index [7]

ii. Coefficient of consolidation, C_v

The coefficient of consolidation C_v 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 C_v as determined by Casagrande's semi logarithmic plot method is

$$C_v = \frac{(0.197)H^2}{t_{90}} \ldots\ldots. \quad \frac{cm^2}{s}$$

C_v = coefficient of consolidation of stratum, in cm^2/sec
H = equivalent specimen thickness, in cm
t_{50} = time at 50 percent of primary consolidation, in sec
The C_v value as determined by Taylor's square root of time fitting method is (0.848).

$$C_v = \frac{(0.848)H^2}{t_{90}} \ldots\ldots. \quad \frac{cm^2}{s}$$

C_v = Coefficient of consolidation of stratum, in cm^2/sec
H = Equivalent specimen thickness, in cm
T_{90} = Time at 90 percent of primary consolidation, in sec

A. Determination of Coefficient of consolidation, C_v

Two methods are available for evaluation of C_v 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. [2]

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, d_{90}, and time, t_{90}, corresponding to 90% primary consolidation. [2]
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, d_{50}, corresponding to

50% consolidation is equal to the deformation at 5/9 of the difference b/n 0 and 90% consolidation. [2]

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, d_{100}, and time, t_{100}, 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, d_{50} corresponding to 50% primary consolidation is equal to the average of the deformations corresponding to the 0 and 100% deformations. The time, t_{50}, 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. [2]

2.10 Distribution of Expansive soils in Africa

Expansive soils are widespread in Africa (Figure 2.7) 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. [2]

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. 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 (Lyone Associates, 1971). [19]

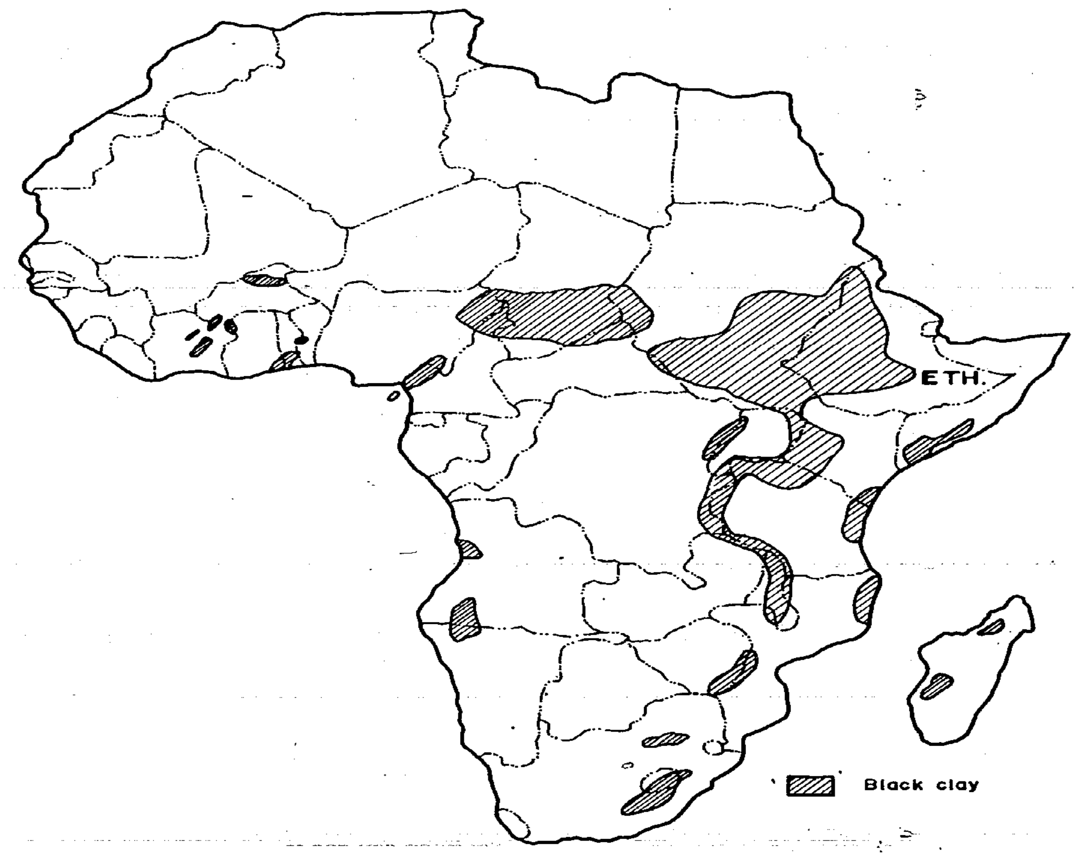


Figure 2.5: Distribution of Expansive Soils in Africa [7]

2.11 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 (Morine, 1971). The Ethiopian dark clay soils located in the Rift Valley and Ethiopian plateau are estimated to cover 10 million hectares (Lyon, 1971). [19]

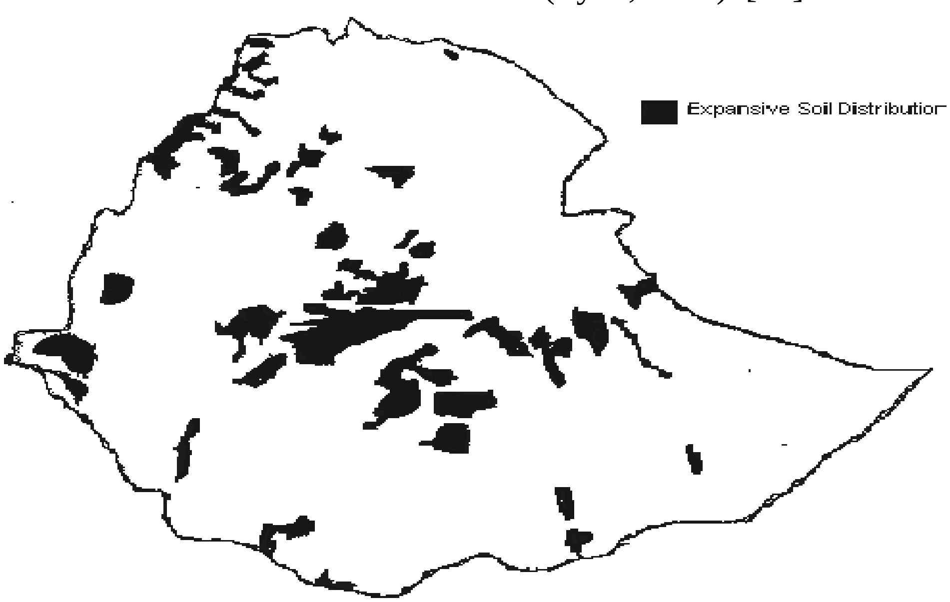


Figure 2.6 Distribution of Expansive soils in Ethiopia [14]

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, Debreberihan 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. [6]

2.12 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. 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.

2.12.1 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 2.11, 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.

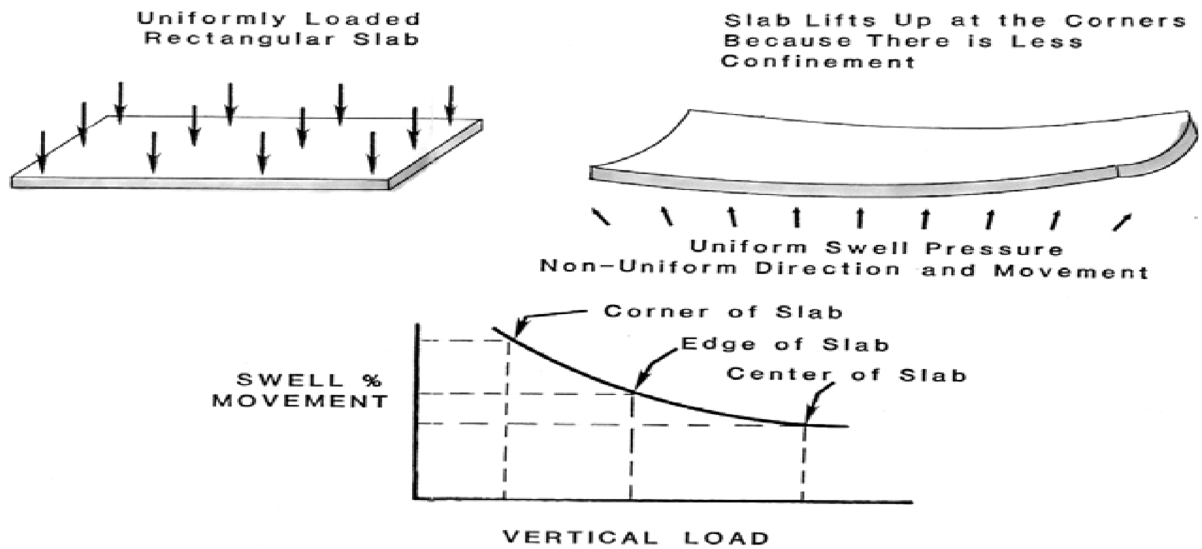


Figure 2.7: A rectangular slab, uniformly loaded, will tend to lift up in the corners because there is less confinement. [23]

Shallow pipes, especially plastic pipes, buried in the zone of seasonal moisture fluctuation, are exposed to enormous stresses by shrinking soils. If water or sewage pipes break, then the resultant leaking moisture can exacerbate swelling damage to nearby structures.

Concrete drainage devices can be adversely affected by expansive soils. Swelling clays can lift and crack concrete ditches, seriously impairing their ability to convey runoff. Subsequent contraction may leave a void under the concrete, leading to piping and erosion as runoff flows under the ditch.

Expansive soils pose the greatest hazard in regions with pronounced wet and dry seasons. The annual cycle of wetting and drying causes the expansive soils to shrink and swell each year. Thus, the arid regions of the country are much more susceptible to damage from expansive soils than regions that maintain moist soil conditions throughout the year.

The biggest problem in expansive soil areas is that of differential water content. A frequent source of damage is the differential swelling caused by pockets of moist soil adjacent to dry soil. For example, lawn and garden watering creates a moist zone on the exterior of a foundation, whereas the interior is dry; this creates differential swelling pressure on foundation elements. There is frequently a moisture differential between the soils beneath a house and those that are more directly exposed to changes in the weather. Cesspools, leaky pipes, and swimming pools are other common sources of water. [17]

Expansive soils are difficult to use in the construction of highways, airfields and lightweight structures, because such light structures can’t exert the necessary counter

load to overcome the swelling. Substantial damage has been occurring in Ethiopia on buildings and roads that are constructed on expansive soil with severe economic consequences, psychological effects and loss of proper functioning of structures. [6]
Extensive hairline cracks and some wider cracks with random orientations can be observed in the pavements. On the other hand, some experimental study of lateral restraint effects on the potential heave of expansive soils showed that, an area where many residential buildings, sidewalks, and pavements have been severely damaged due to the differential heave of the supporting weak rock shales. The damage is primarily in the form of diagonal cracks in beams and columns. [21]
In Ethiopia, a lot of damages have been occurring due to expansive soil. Recent researches in assessing the failures caused on structures built on expansive soils of Addis Ababa showed that more than 60% of the structures are damaged due to causes associated with expansive soils. [20] Although there is not an organized economic survey, it is assumed that most of the economic loss due to failures associated with civil engineering construction is attributed to expansive soils. Low level of understanding of the mechanisms of heaving and the measures required to counter such problem by civil engineers and architects, wrong concept of safety associated with the conservative design, improper soil investigation and interpretations, and the use of soil parameters, chart and tables developed for temperate zone soils have contributed for most of the failure caused by expansive soil area in Ethiopia. [6]
The problem observed on buildings varies from simple crack on walls to severe structural damages, which cause collapse of the whole building. Some of the problems observed in Addis Ababa expansive soil are shown blow in Figure 2.8. [31]

Figure 2.8 a) Shear of tie beam in Addis Ababa city car maintenance hangar [31]

Figure 2.8 b) Detachment of tie beam and column [31]

Figure 2.8 c) Cracks of walk way [31]

Figure 2.8 d) Sever diagonal cracks on building wall [31]

2.13 Review of Comparative Characteristics of Ethiopian Expansive Soils

Table 2.1 Indicative Properties of Ethiopian Expansive Soils [2]

Clay Content < 2µm or .002mm	50-80 %
Liquid Limit	80-120 %
Plasticity Limit	55-90 %
Shrinkage Limit	10-16 %
Free Swell	90-23 %

Table.2.2 Mineralogical compositions of expansive clay soils of Addis Ababa, [20]

Clay miner	**Mineral content (%)**	
	Black clays	**Gray clays**
Montmorillonite	7 0 - 80%	70-75%
Illite	10-15%	25-30%
Kaolinite	10-15%	10-15%

Table 2.3 Recent studies on Addis Ababa expansive soils give ranges of swelling pressure [19]
Swelling Pressure

Type of Soil	**Unit**	**Value**
Black Clay	kPa	36.9 - 420
Gray Clay	kPa	60.8 - 320

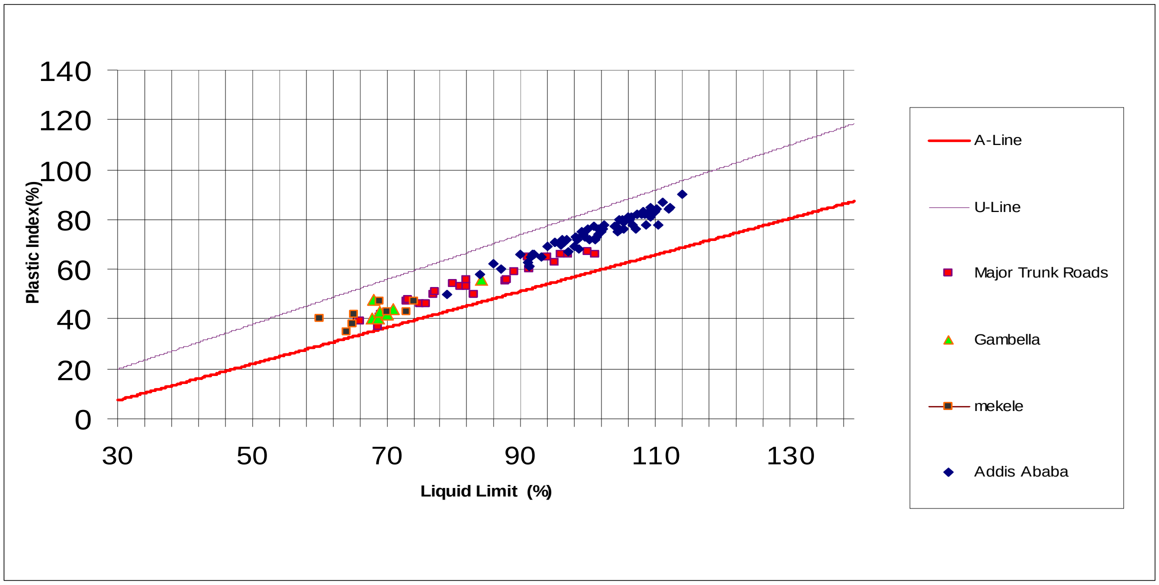


Figure 2.10 Plasticity chart of expansive soil of Ethiopia [20]

From the researches mention above, the engineering properties of the expansive soil of Addis Ababa such as density, specific gravity, consistency index, swelling characteristics, and shear strength characteristics are obtained. Summaries of outputs of the researches are presented in Table 2.5 & Table 2.6.

Table 2.5: Lithological properties of expansive soils of Addis Ababa [20]

Soil Properties	**Range of Values**	**Standard Deviation**	**Range of Values**	**Standard Deviation**	**Range of Values**	**Standard Deviation**
	Silty clay		Black clay		Gray clay	
Moisture content	35.6-52.1	2.71	34.0-42.6	5.24	38.9-45.1	3.50
Bulk density(g/c	1.60-1.76	0.03	1.65-1.70	0.03	1.69-1.70	0.03
Specific Gravity	2.70-2.88	0.03	2.77-2.81	0.02	2.80-2.82	0.01
Dry density(g/cm	1.10-1.30	0.04	1.18-1.26	0.05	1.15-1.27	0.06
Void ratio(e)	1.25-1.50	0.05	1.21-1.38	0.11	1.26-1.43	0.09
porosity(n)	0.53-0.61	0.11	0.54-0.58	0.22	0.55-0.58	0.18

Table 2.6 Range of values of index properties of expansive soils of Addis Ababa [20]

Soil type	**liquid limit**	**Plastic limit**	**Plasticity index**	**Clay content**	**Silt content**	**Sand**	**Activity**	**Free swell**
Black clay	98.50-104.30	25.50-26.90	75.47-76.40	75.40-79.80	24.4-29.5	2.00-3.4 0	1.60	109.0-120.0
Grey clay	101.40-104.50	25.6-27.8	68.30-74.90	71.10-75.30	22.10-26.20	2.70-2.80	1.36	96.60-106.30
Silty clay	78.80-95.00	24.60-27.00	56.80-67.80	65.00-71.00	36.15-46.40	2.20-3.80	0.89-1.10	86.00-117.00

Recent researches on the engineering properties of expansive soils of Ethiopia revealed that, the expansive soils in most parts of the country are classified as inorganic clays of high plasticity. (Figure 2.15) The clay content of the soil is found to be as high as 80%[5] and the amount of montmorillonite for Addis Ababa expansive soil is 70- 80%[Table 2.6]. These soils have the ability to hold significant amount of water that affects the shear strength, as well as, shrinkage and swelling characteristics. [20]

3. Description of the Study Area

3.1 Location

Galan Town is a city administration of Galan Zone which is one of the Zones of Oromia regional state around Addis Ababa city administration in Special Zone surrounding Finfine located at 25 km away from Addis Ababa in South-East direction or between 7^0 12'- 90 14' N Latitudes and $38^0$32' – $39^0$32' E Longitudes on the Addis Ababa – Djibouti asphalt road.

Galan town is surrounded in the East direction by Dukem city where the rail way and the high way cross each other. In the North East by Kora Mariam church, in the North by Wedesso Mountain and Abay Silto farmers 'Kebele'

Galan Town is one of the fastest growing cities in Addis Ababa area Zones so that there is a large volume of construction activities. Since it is the transit way on the road from Addis Ababa to Djibouti, the growth of trade and commerce is very high in the city. Due to its near location from Addis Ababa investors are highly attracted to construct huge number of commercial, Industrial, Business & residential buildings in Galan Town and the nearby areas. It has been noticed that the Town & nearby areas are largely covered by the expansive soils including areas where recent constructions are taking place.

3.2 Altitude

Galan is found between 1500-2300 m above sea level. The highest place (2,100m) is known as Gara Bushu & it is the place where the Oromos cultural Religion known as **Irrecha** was took place and which the place now exists as Millennium Park of the city. The lower elevation (1,750m) is around Merino Kebele.

3.3 Demography

According to the Galan city Administration, the population of the town is 32,689, out of which female are, 13,076 (40%) and males are 19,613 (60%). The annual growth percentage is 4.5% according to the office of the finance & economic development of Galan city. The economically active population (Productive age Group) categorized under (15-64) constituted 44.2% of the total population. Dwellers of the city are engaged in different agricultural and commercial activities of mostly retail trade. The city is well known by heavy manufacturing industries.

There are many nations & nationalities that live in the city. This includes Oromo, Amhara. Tigre, Guragie etc. Most of the residents are engaged in commercial activities, daily labor, farming and also workers in industries.

3.4 Ecology and Natural Resources

Ecologically the City lies in wet land ecosystem; its altitude lies between 1500-2300m above sea level. The City is bounded by forest and Nitosol and the climate is preferable both for work and residence.

The greenery coverage of the City is about 335.175 hectare, 4.46% including forests, urban agriculture, unoccupied areas and other vegetations (plantation) in parks or road sides. Every year about 100,000-200,000 different species of trees are planted in the City (including the dig out site)

3.5 Economic Activities

The economy of the City is dominated by manufacturing out puts of the Investment sectors and commercial activity (trade & delivering service) and the local exchange, its surrounding towns, Cities, and in general the whole country and the world have significant contribution for business activity. This is because of the commercial activities that are takes place among small and heavy manufacturers that involves in the country and also in the world market.

There are so many large-scale industrial activities in the city like pharmaceuticals, steel melting, paint factory; textile and garment factory, marble factory, chemical industry and car assembly are some of them. Also small-scale industries and cottage industries like grain mills, wood and metal workshops, car Battery, Coffee Hullers, hollow block manufacturing

Galan is widely known by its manufacturing products. The major manufacturing activities that account **91.8%** are metal products and Garment Products,

3.6 Climate

3.6.1 Rainfall

The records of National Meteorological Service Agency from Akaki observatory substation show that the mean annual rainfall for ten years i.e. from 2005 to 2014 at an altitude of 2057 above mean sea level, latitude of 38^0 47’10’’and longitude of 8^0 52’ 11’’ is 855.6 mm. As shown in Fig 3.1, the rainy season is only in months of July and August followed by rainfall of almost equal to or below evaporation rate of the city. Around 50% of the rainfall arrives in these two months.

3.6.2 Temperature

In a mountainous tropical country like Ethiopia altitude is by far the most important factor in controlling climate. It affects distribution of both temperature and rainfall. Generally, regions between 1500 - 2300 meters a.m.s.l. (categorized as 'woina dega' or sub tropical climate) have temperatures that range between 15 – 20 ^{0}c, areas between 500 – 1500 meters a.m.s.l. (i.e. 'kola' or tropical climate) have 20 -30 ^{0}c and areas below 500 meters a.m.s.l. (i.e. 'bereha' or desert climate) have a temperature of 30 ^{0}c and above[16].

The town of Galan with an altitude of 2057meters a.m.s.l., has a mean minimum, mean maximum and mean average monthly temperatures of 13.6, 26.5 and 20.0 ^{0}c respectively. The highest temperatures are during months of February, March, April, May and June where as July, August and December have low temperature. From Fig3.3 the Mean monthly average temperature ranges from 18.41^0c to 21.90 ^{0}c. This shows the temperature variation is almost similar throughout the year.

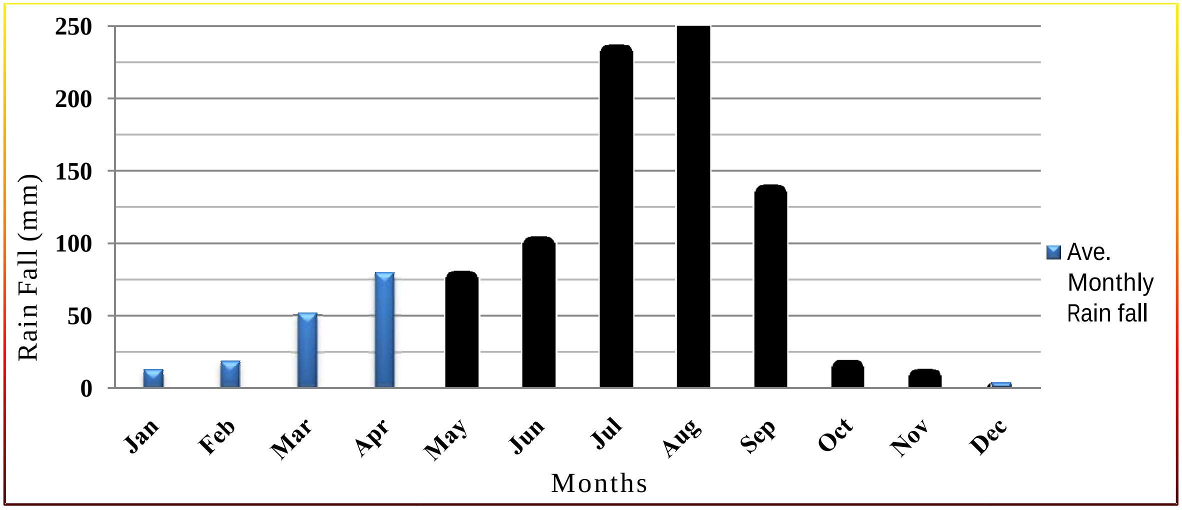


Figure 3.1 Mean monthly rainfall distribution of Galan Town (2005 - 2014)

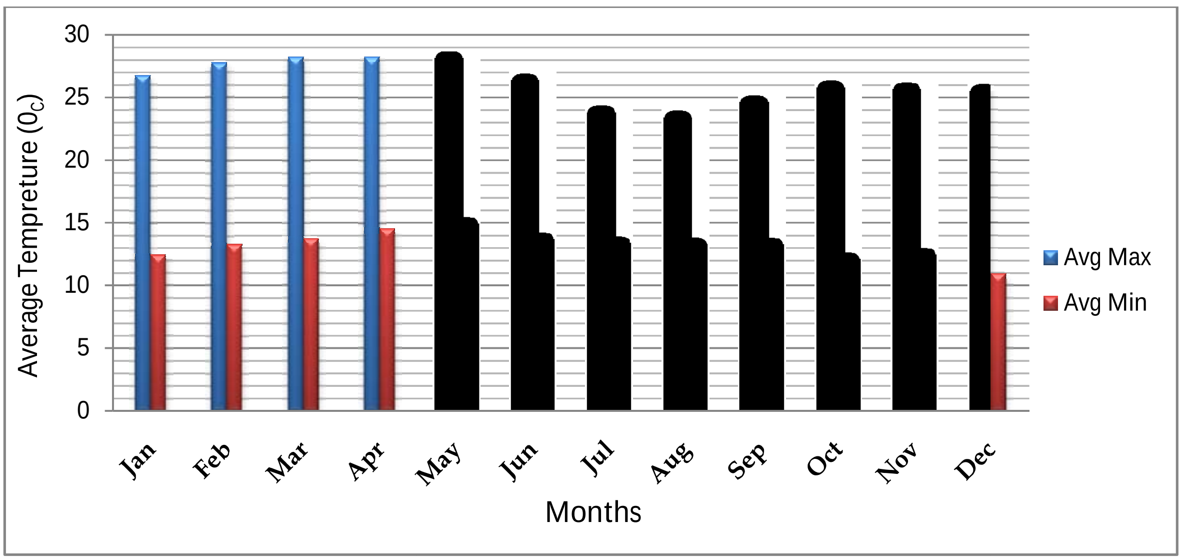


Figure 3.2 Average Monthly Maximum and Minimum temperature distribution of Galan Town, (2005 - 2014)

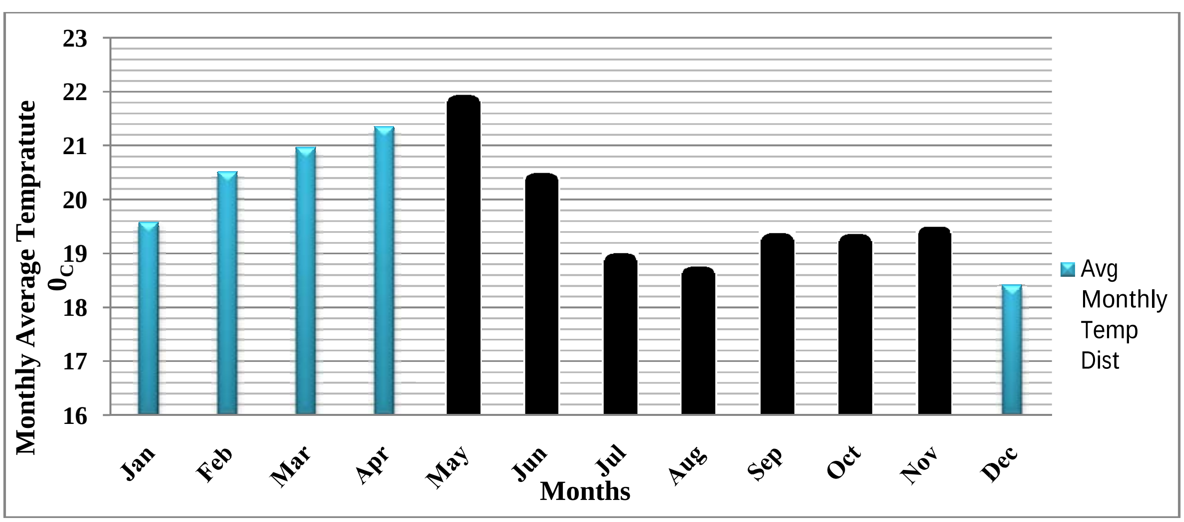


Figure 3.3 Monthly Average Temperature distribution of Galan Town (2005 - 2014)

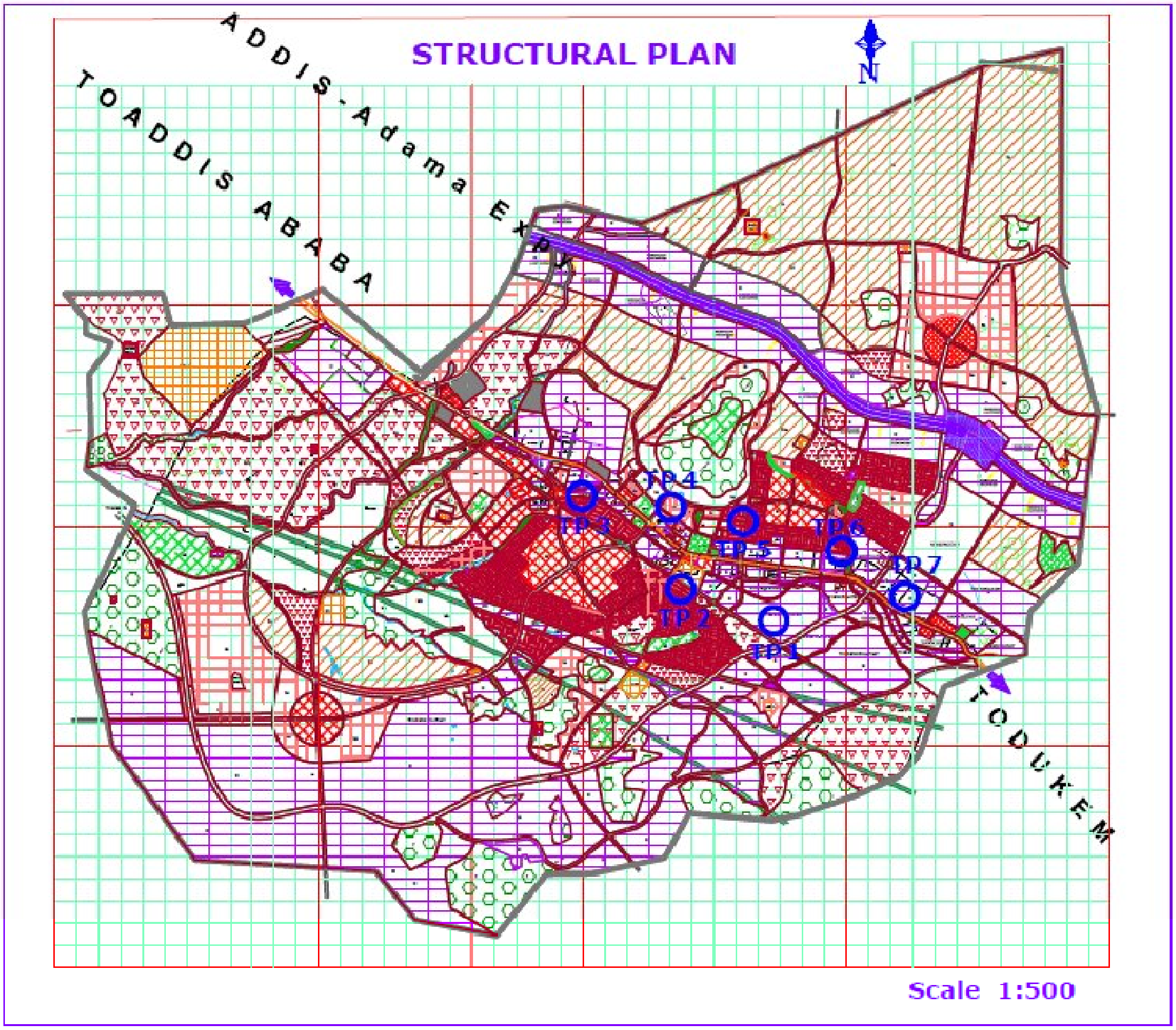


Figure 3.4 Map of Galan Town

4. Sample Collection & Preparation and Test Results

4.1 Sample Collection & Preparation

In order to achieve the intended purpose of this work disturbed & undisturbed representative samples of expansive soils of Galan Town emphasizing on residential and business areas where expansive soils are significantly dominant and spotted out & confirmed by the Town administration. To have sufficient and reliable data for the target analysis, standard laboratory tests were conducted on soil samples obtained from different location and depth of Galan Town. Most of the samples were collected around undergoing construction projects during the excavation stage to make use of clear visualization of the soil stratum. 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. According to the results obtained from laboratory tests the soils of Galan Town are classified as presented in Table 4.3. Initial condition of samples used for the laboratory swell-consolidation test is presented in Table 4.4.

Because it was difficult to find samples of reasonably varying moisture content and dry density of insitu soil, it was quite necessary to prepare the soil specimen for swell- consolidation tests from disturbed samples by remolding. For this purpose standard compaction test was conducted in the laboratory to determine the relationship between moisture content and dry density from the compaction curve. This approach was helpful for the thesis work, because it enabled to prepare the soil specimen for swell - consolidation test by varying the initial moisture content and dry density from the standard compaction curve.

4.2 Laboratory tests & results

4.2.1 General

A bulk soil, as it exists in nature comprises of soil particles, water and air. The properties and characteristics of soils are complex & variable which vary from point to point. The tests required for determination of engineering properties are generally elaborate and time consuming. Index properties of soil are properties, which are used to characterize the soils and determine their basic properties. Index tests include the water content (also known as moisture content), specific gravity tests, unit weight determinations, particle size distributions and Atterberg limits, which are used to classify the soil.

Laboratory tests should be performed in accordance with standard procedures, such as those recommended by the American Society for Testing and Materials (ASTM), American Assiciation of State highway and Transportation Officials (AASHTO) or those procedures listed in standard textbooks or specification manuals. [26]

Based on the samples retrieved from the sites, laboratory tests on the samples were conducted in the geotechnical and highway laboratories of Addis Ababa Institute of Technology as follows.

4.2.2 Free swell

To study the swelling property of the soils, the simplest test conducted is free swell test. This test is performed by slowly pouring 10ml of oven dry soil which has passed the No. 40(0.425mm) sieve in to 100 ml graduated cylinder filled with distilled (tap) water. After 24 hours, final volume of the suspension being read. Hence, free swell is defined as:

$$Free\ Swell = \frac{Final\ Volume - Initial\ Volume}{Initial\ Volume} x100\ \%$$

Free swell test results for the collected samples are summarized in Table 4.1. From the test result one can see that the free swell of the soil under investigation ranges from 175% to 230%. Those soils having a free swell less than 50% are considered as non expansive soils, those which have free swell between 50-100% are considered as marginal and those with free swell greater than 100% are considered as expansive soils. According to the results of this study all soil samples under investigation can be considered as expansive soils.

Table: 4.1 Clasification of the soil based on test results obtained from Free swell test

Sample ID	Depth (m)	Sample No	Sample Type	Physical Property	Free Swell (%)	Degree of Expansiveness
Pit-1	1.5	1	Disturbed	Black Clay	188	Highly Expansive
Pit-1	2.8	2	Disturbed	Black Clay	183	,,
Pit-2	1.5	3	Disturbed	Black Clay	183	,,
Pit-2	2.35	4	Disturbed	Gray Clay	190	,,
Pit-3	1.5	5	Disturbed	Black Clay	180	,,
Pit-4	1.5	6	Disturbed	Black Clay	175	,,
Pit-5	1.5	7	Disturbed	Black Clay	215	,,
Pit-5	2.6	8	Disturbed	Black Clay	213	,,
Pit-6	1.5	9	Disturbed	Black Clay	220	,,
Pit-6	2.6	10	Disturbed	Black Clay	195	,,
Pit-7	1.5	11	Disturbed	Black Clay	230	,,
Pit-7	2.9	12	Disturbed	Black Clay	193	,,

4.2.3 Specific Gravity Tests of the Soil Solids (Gs)

In this research the specific gravities were determined using ASTM D 854-00 Standard Test procedures. According to the test results the value of specific gravity of Black clay ranges from 2.60 to 2.7 & for Gray clay it ranges from 2.66 to 2.70. Therefore, the specific gravity of Galan Town ranges from 2.60 to 2.72. The test results are shown in Table 4.2.

Table: 4.2 Specific Gravity test Results of samples under investigation

Sample ID	Depth (m)	Sample No	Physical Property	Specific Gravity
TP-1	1.5	1	Black Clay	2.68
TP -1	2.8	2	Black Clay	2.71
TP -2	1.5	3	Black Clay	2.69
TP -2	2.35	4	Gray Clay	2.70
TP -3	1.5	5	Black Clay	2.69
TP -4	1.5	6	Black Clay	2.72
TP -5	1.5	7	Black Clay	2.66
TP -5	2.6	8	Black Clay	2.70
TP -6	1.5	9	Black Clay	2.60
TP -6	2.6	10	Black Clay	2.69
TP -7	1.5	11	Black Clay	2.61
TP -7	2.9	12	Black Clay	2.67

4.2.4 Moisture Content Tests

For many soils, the water content may be an extremely important index used for establishing the relationship between the way a soil behaves and its properties. The consistency of a fine-grained soil largely depends on its water content. The moisture content test was carried out in the laboratory as per the procedure of AASHTO T 265 or ASTM D 2216 – 98.

As observed from the laboratory test results the natural moisture content of the study area ranges from 30.66-50.06%.

4.2.5 Grain size analysis (Sieve and Hydrometer Analysis Tests)

The procedure followed to run this test was according to ASTM standard with designations D422-63 and D1140-97. According to ASTM D422-63 the distribution of particles, finer than 75μm can be done by hydrometer test and coarser than 75μm by mechanical sieve. The combined grain size distribution curve for mechanical sieve & hydrometer tests is shown in Figure 4.1.

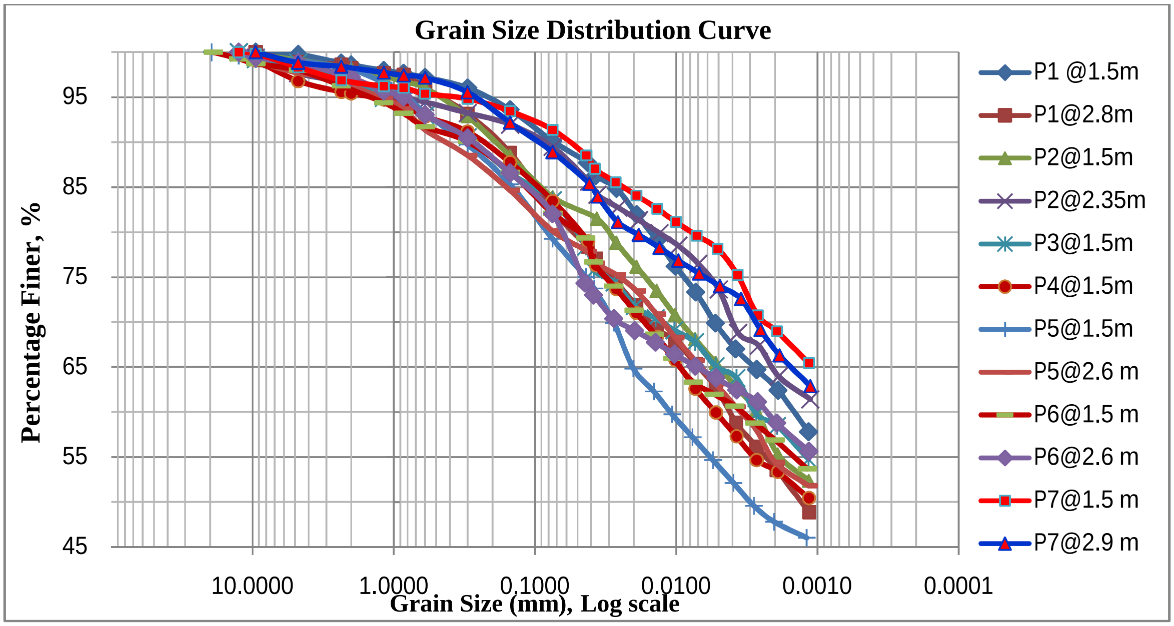


Figure 4.1: Grain size distribution cure for samples

4.2.6 Atterberg limit Tests

The Atterberg Limits were determined for air-dried samples based on the Standard Reference: ASTM D 4318-98 –Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. In addition the liquid limit tests were conducted on oven dry samples of five representative samples. These values are required, to know the ratio of liquid limit on oven dried and air dried sample, which will in turn used to classify the soil as organic and inorganic. The laboratory test result of Atterberg Limits of the soil under investigation and the classification made based on the results are summarized in Table 4.3. From this we can observe that liquid limit ranges from 98– 124%, plastic limit ranges from 25 – 37% and plastic index from 68 – 94%.

Table: 4.3 Result of Atterberg Limit test & general classification of the soil of Galan town

Sample ID	**Depth (m)**	**Sample No**	**% Pass No 200 Sieve**	**LL**	**PL**	**PI**	**PI**	**Soil Classification**	
							LL-30	**AASHTO**	**USCS**
Pit-1	1.5	1	86.99	114	35	78	84	**A-7-5**	**CH**
Pit-1	2.8	2	86.64	111	37	73	81	**A-7-5**	**CH**
Pit-2	1.5	3	87.48	105	32	73	75	**A-7-5**	**CH**
Pit-2	2.35	4	88.31	115	29	85	85	**A-7-5**	**CH**
Pit-3	1.5	5	85.98	109	30	79	79	**A-7-5**	**CH**
Pit-4	1.5	6	85.09	98	25	73	68	**A-7-5**	**CH**
Pit-5	1.5	7	93.40	120	37	89	90	**A-7-5**	**CH**
Pit-5	2.6	8	92.75	119	29	90	89	**A-7-6**	**CH**
Pit-6	1.5	9	94.31	122	32	90	92	**A-7-5**	**CH**
Pit-6	2.6	10	90.37	119	31	88	89	**A-7-5**	**CH**
Pit-7	1.5	11	95.06	124	30	94	94	**A-7-5**	**CH**
Pit-7	2.9	12	90.87	120	30	90	90	**A-7-5**	**CH**

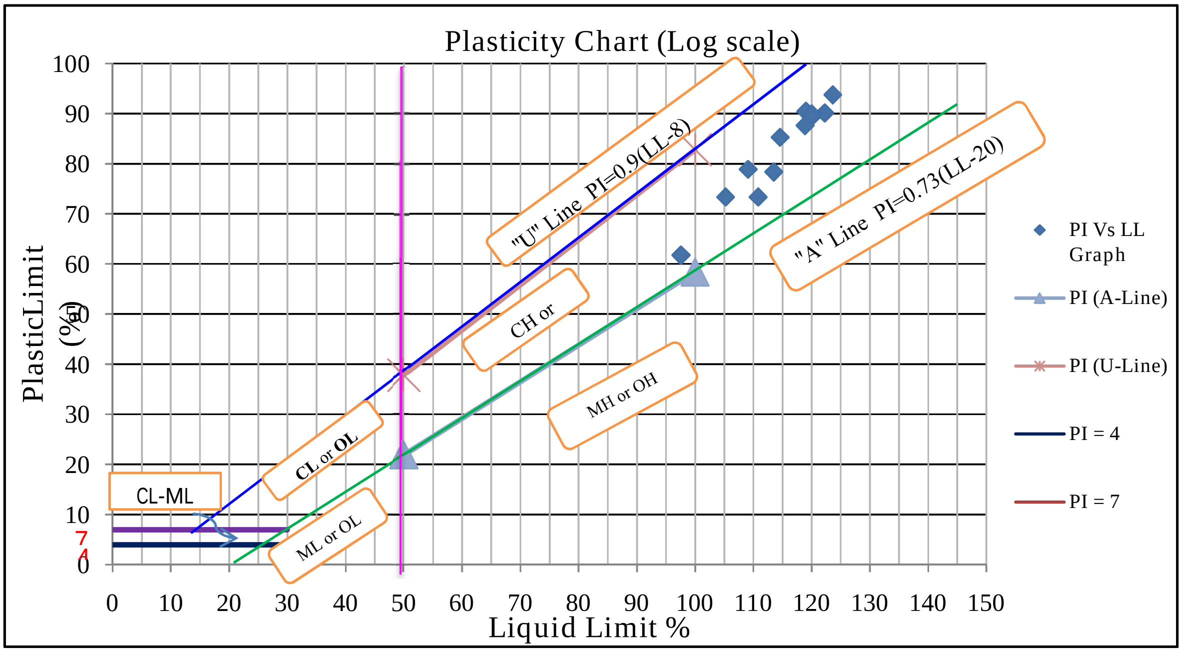


Figure 4.2: Plasticity chart of the study area according to Unified Soil Classification System - USCS

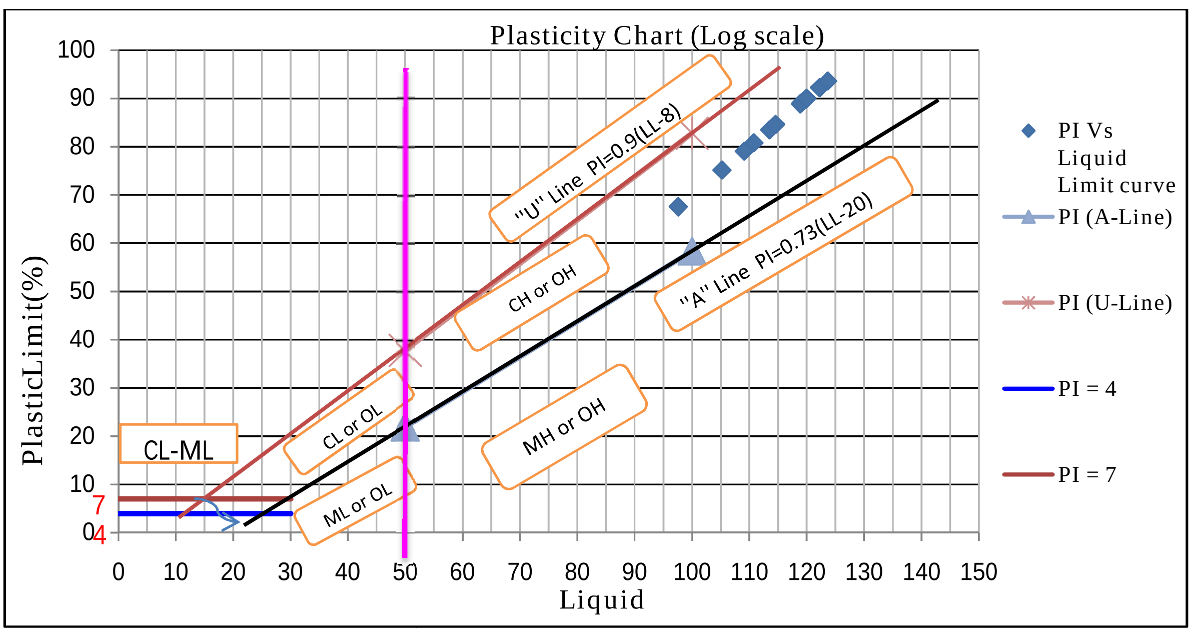


Figure 4.3: Plasticity chart of soil in the study area according to AASHTO system of classification

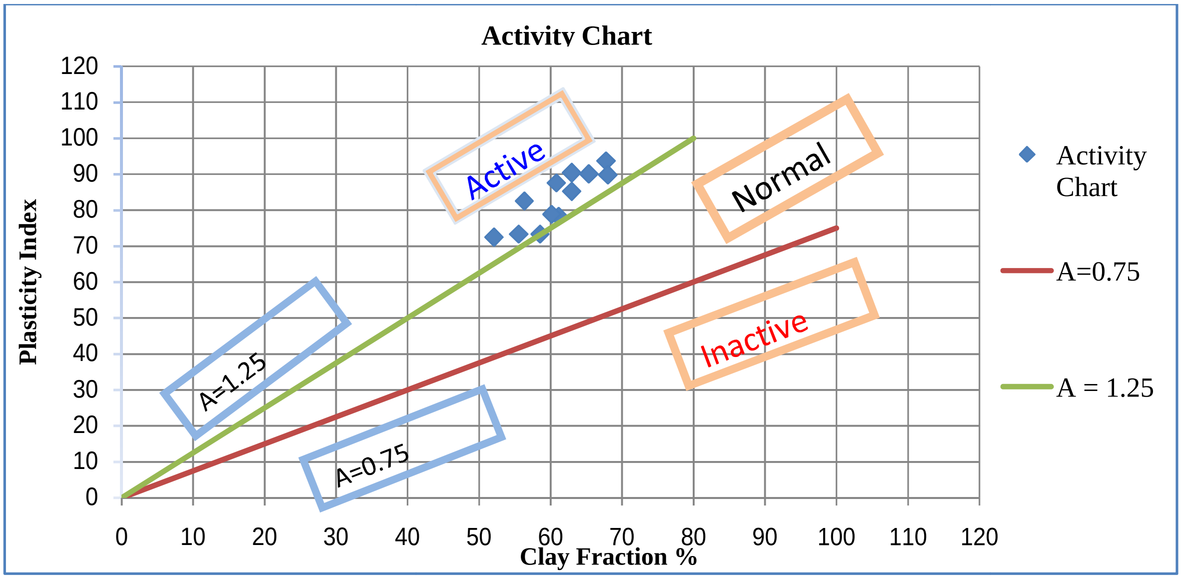


Figure 4.4: Activity charts of the soil under investigation

4.2.7 Laboratory Compaction Test

Compaction is defined as the densification of a fill by mechanical means. This physical process of getting the soil into a dense state can increase the shear strength, decrease the compressibility, and decrease the permeability of the soil.

In this research laboratory standard compaction tests were conducted on different samples to obtain the compaction curve. The compaction curves were used to remold the samples at different specified dry density & moisture content from wet side & dry side of the OMC for the purpose of swell-consolidation tests.

In this research ASTM D 698 - Standard Test Methods for Laboratory Compaction tests were conducted and the test results are shown in figure 4.5 below.

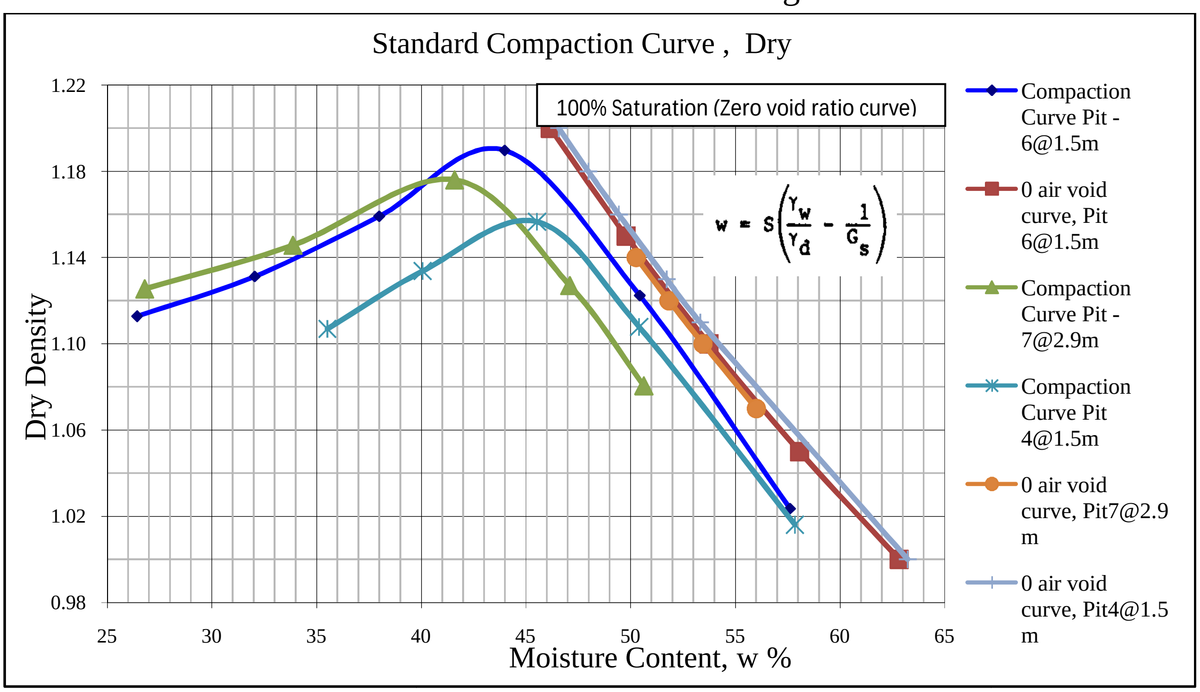


Figure 4.5 Compaction Curve obtained from Standard Compaction Test for different samples of

4.2.8 Laboratory Swell-Consolidation testing

4.2.8.1 General

The swell and swell pressure are generally determined in the laboratory with one-dimensional 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

4.2.8.2 Sample Preparation

For the One-Dimension Swell- Consolidation tests the specimens were prepared both from Undisturbed & Disturbed samples.

a) Undisturbed samples

From the undisturbed samples of expansive soil collected from Galan Town, the specimens for odometer test were extracted from the tube sampler using the odometer ring which has a diameter of 5cm and a height of 2cm.The samples were then prepared for the test by trimming the ends.

b) Disturbed samples

Because it was difficult to find samples of reasonably varying moisture content and dry density of insitu 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 Galan 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 4.6 below.

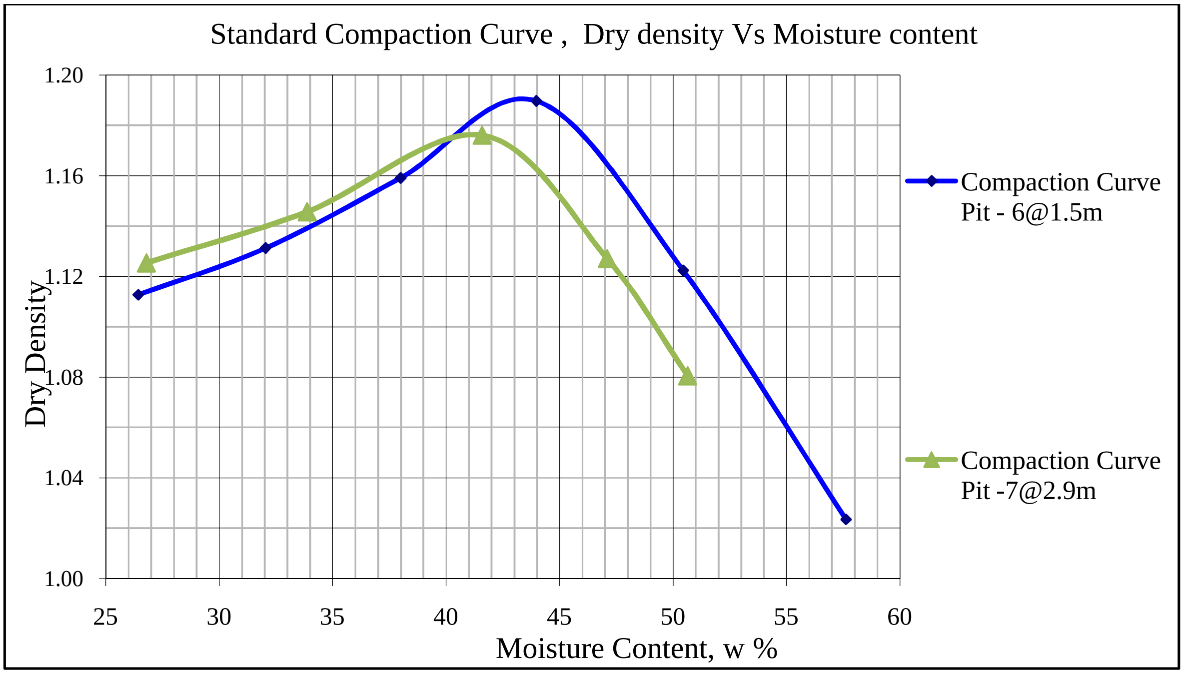


Figure 4.6: Compaction curve used for remolding from (Pit 6 @1.5 m depth) & (Pit 7 @ 2.9m depth)

On the compaction curve three points from wet & dry side of the curves including OMC were selected for remolding by varying the moisture content and dry density. Based on the selected points the amount of water and dry soil to mix was arithmetically obtained.

The disturbed samples were then air dried, sieved by 0.475 mm and the required mass for the remolding purpose was soaked with the specified moisture content for 24 hours in order to attain uniform moisture migration. The wet soil sample was then compacted to the required density and the test specimen was extracted using the ring of the odometer. This procedure was repeated to prepare a number of samples for remolding by varying initial moisture content & dry density from the same curve.

In order to achieve the intended objectives of this research, a swell-consolidation test was carried out on six remolded and five undisturbed samples of expansive soils obtained from seven test pits at different location of Galan Town where expansive soil is dominant. The ASTM procedures D2435-96 test methods for one-dimensional consolidation properties of soil, and D4546-96 Method-A Standard test method for one dimensional swell or settlement potential of cohesive soils, (which measures the free swell, the percentage heave for vertical confining pressure and the swell pressure) were conducted both on remolded & undisturbed soil samples. [26]

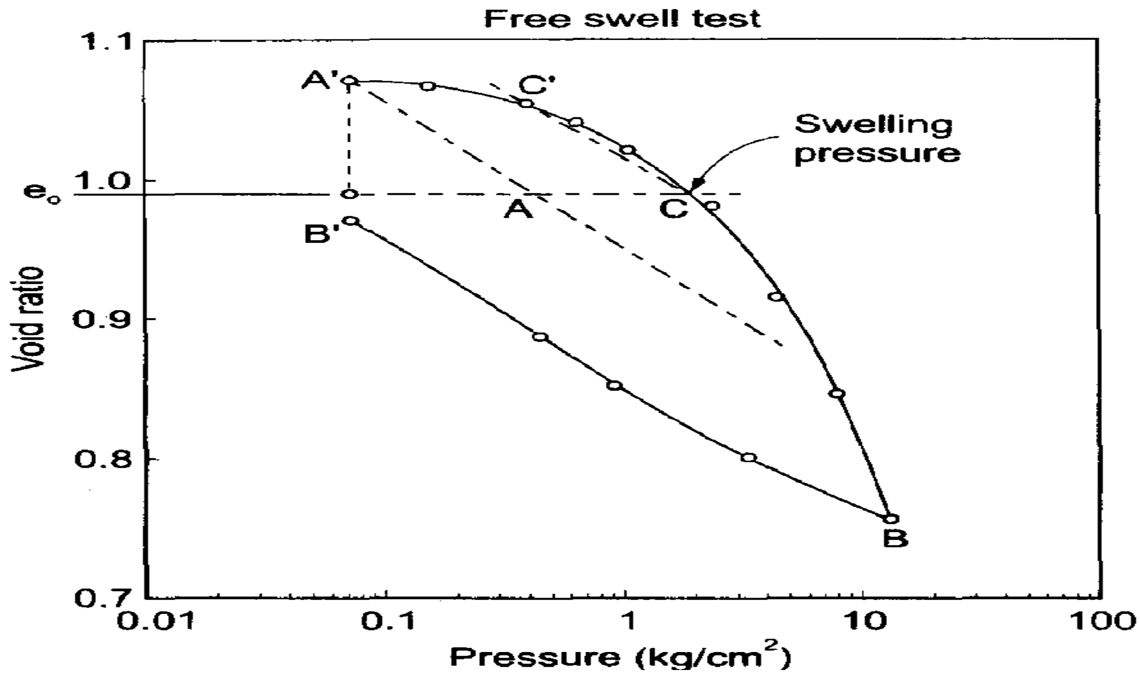


Figure 4.7: Typical plot of consolidation-swell test [26]

4.2.8.3 Test results

The swell-consolidation test was carried out on five undisturbed and six remolded samples of expansive soils of Galan 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 initial condition of the soil under investigation is presented in the table below.

Table: 4.4 Initial conditions of Undisturbed & Remolded samples prepared for swell-consolidation tests

Sample No	Depth (m)	Sample Type	Initial Moisture content (%)	Initial dry density	Initial Saturation S (%)
1	1.5	Undisturbed	45.77	1.18	96.55
2	2.8	Undisturbed	50.06	1.17	103.09
3	1.5	Undisturbed	36.48	1.30	91.43
4	2.35	Undisturbed	30.60	1.29	76.01
5	1.5	Undisturbed	30.66	1.37	85.60
6-1	1.5	Remolded	32.04	1.13	62.47
6-2	1.5	Remolded	44.00	1.19	93.96
6-3	1.5	Remolded	50.00	1.12	96.06
7-1	2.9	Remolded	27.00	1.13	52.92
7-2	2.9	Remolded	42.00	1.18	88.77
7-3	2.9	Remolded	47.00	1.13	92.21

In this study a number of one-dimensional consolidation tests were conducted to determine the Swell-consolidation characteristics of undisturbed and remolded soil samples. The corresponding consolidation curves for undisturbed soil samples are shown below.

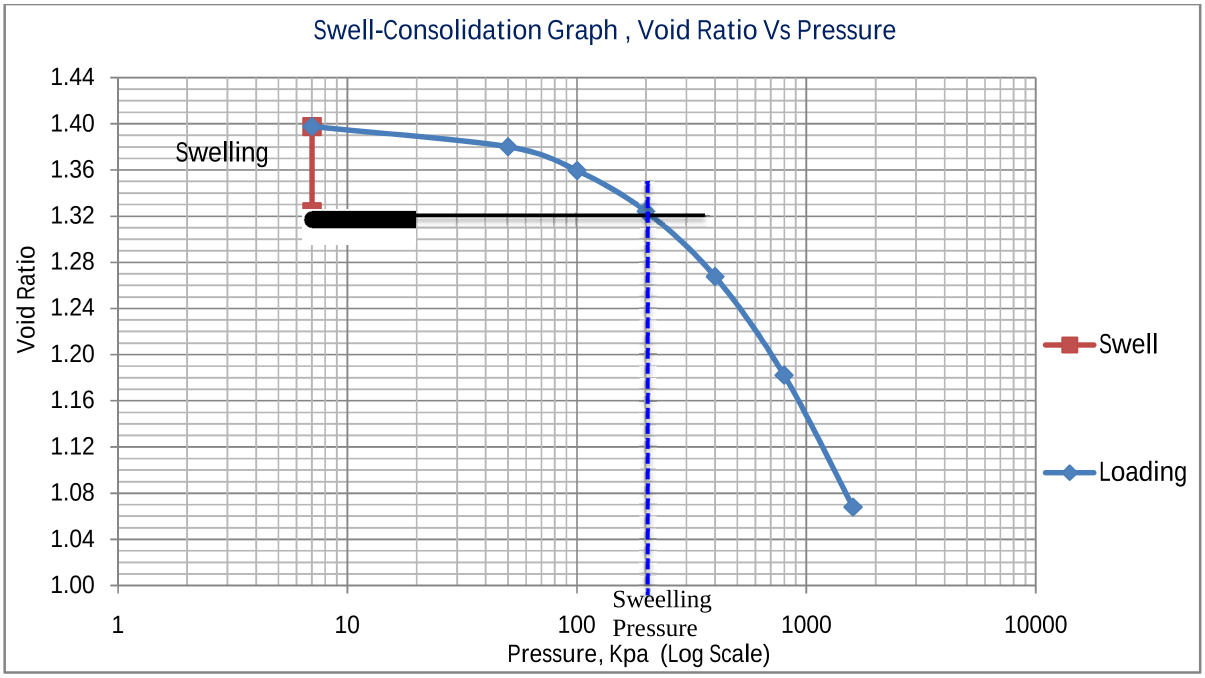


Figure 4.8 (a): Typical plot of e- log P of undisturbed sample at initial dry density of ρ_d =1.17& initial moisture content of w=50.06, (Sample-2)

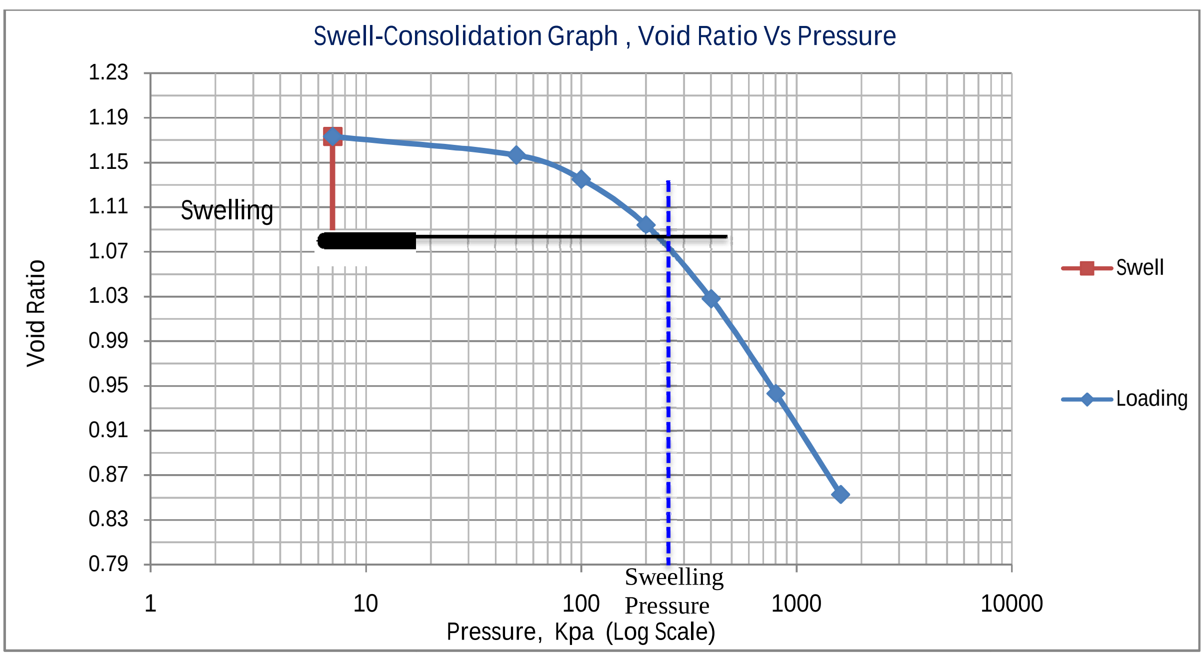


Figure 4.8 (b): Typical plot of e- log P of undisturbed sample at initial dry density of ρ_d =1.30 & initial moisture content of w =36.48, (Sample-3)

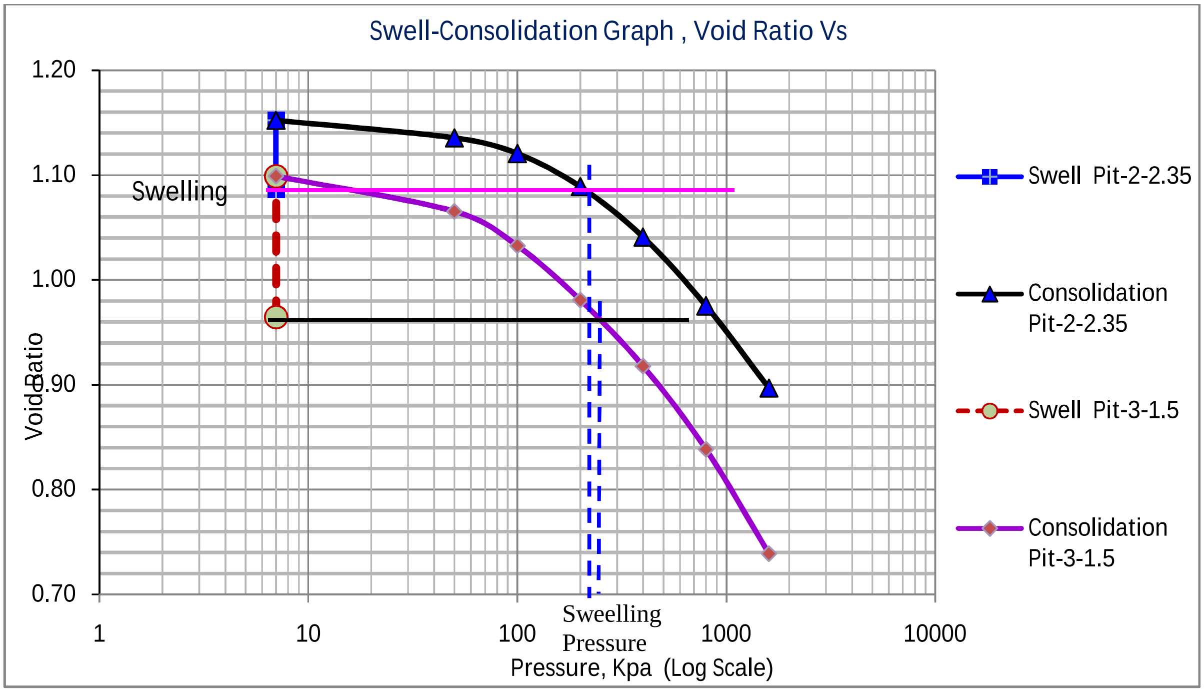


Figure 4.8 (c): Typical plot e- log P of undisturbed sample at the same initial moisture content (30.60%) & deferent initial dry density (ρ_d= 1.29 & ρ_d 1.37) of sample 4 & 5 respectively

The Swell-Consolidation test results are grouped into two for the purpose of the analysis. The first group includes the initial condition & the test results of the undisturbed samples. The test results are used to determine swelling potential, swelling pressure, & range of values of the consolidation parameters, C_c and C_v of the insitu soil sample. The second group includes test results of both undisturbed and remolded samples used to determine the relationship between consolidation and swelling characteristics of expansive soil of Galan Town. In the course of the study test results of pair and group of soil samples of the same dry density and different moisture content and soil samples of different dry density and the same moisture content were considered. The test results presented in the main body of the paper are only the typical ones which can better explain the relationship between swelling & consolidation characteristics of the expansive soil of the study area. The test results of all the other samples are presented in the appendices.

4.2.8.4 Determination of the consolidation Parameters

The important parameters of a soil in consolidation tests are the compression index C_c and the coefficient of consolidation C_v. Two methods are used to estimate C_v which are the square root of time method proposed by Taylor (1942) & the log time method proposed by Casagrande & Fadum (1940). The log time method makes use of the early (primary consolidation) and later time response (secondary compression) while the square root time method only utilizes the early time response, which is expected to be straight line. [22] In this thesis work, the square root of time method has been used.

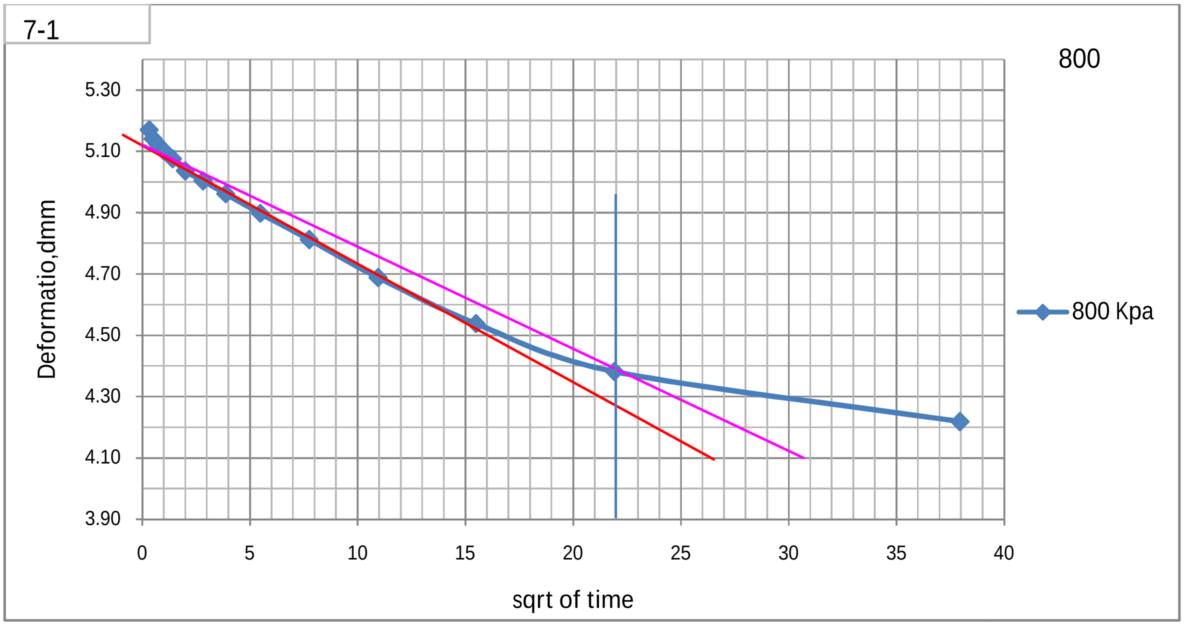


Figure 4.9 (a) Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-1 (P=800kPa),

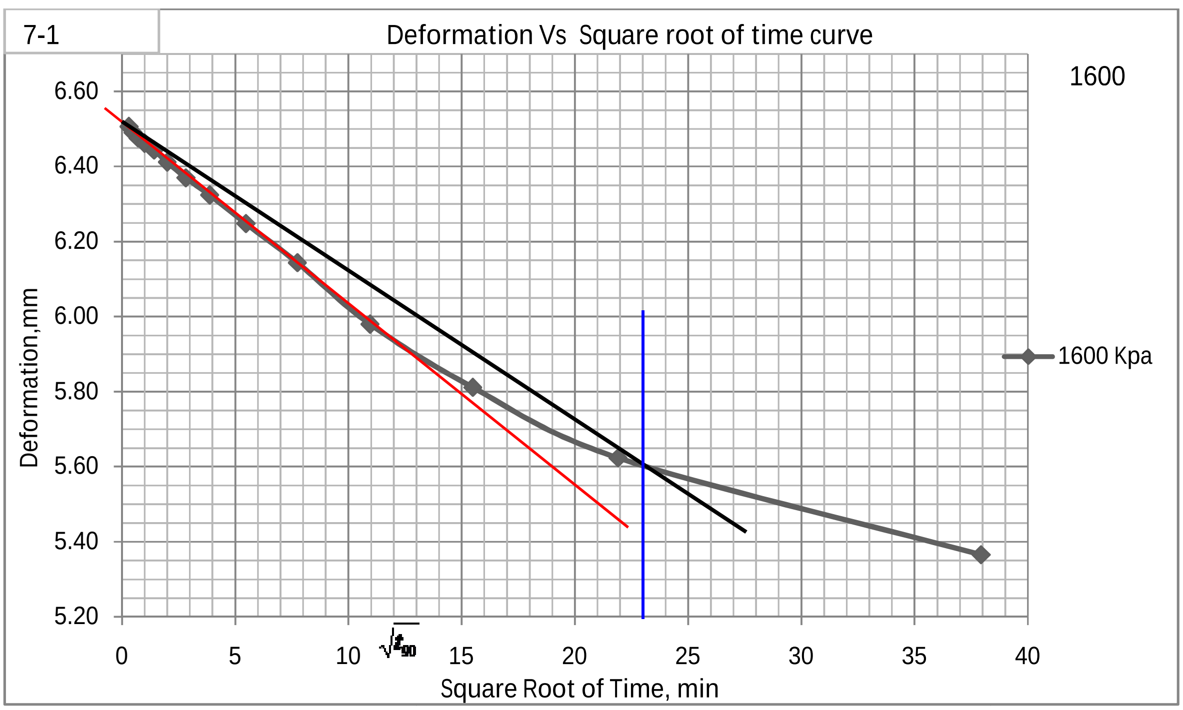


Figure 4.9 (b) Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-1 (P=1600kPa),

5. Discussion on the test results

Generally expansive soils are not subjected to volume change unless there is a change in moisture content. In this investigation a series of swell-consolidation tests were conducted to determine the relationship between Swelling & Consolidation characteristics of expansive soils of Galan Town by varying the initial moisture content and dry density of the samples.

The soil sample allowed to saturate in an oedometer swells until the equilibrium is reached under the seating load and this helps to obtain the swelling potential of the soil sample. Accordingly the swelling potential of the samples was found to be in the range between 2.18 & 6.83 %.

One-dimensional consolidation test method has also been conducted to determine the consolidation characteristics of the samples. As can be observed a series of the test results, for expansive soil to swell under varying moisture content and dry density, the overburden pressure has to be smaller than the swelling pressure. On the other hand for the expansive soil to have consolidated after its complete swelling under wetting, the overburden pressure has to be greater than its swelling pressure.

On this regard the knowledge of the stress history of expansive soil is essential which is used to determine the relative magnitude of present overburden pressure and the maximum pressure to which the soil was subjected to in its history. Stress history is also important to determine the relative magnitude of swelling pressure which in turn presents the relationship between swell-consolidation characteristics of expansive soils.

The swell-consolidation graph of undisturbed samples of expansive soil is found to be flat as compared to that of remolded samples. The range of values of initial void ratio, swelling potential, swelling pressure and compression index are also considerably different from the value of the remolded sample. This indicates that the disturbance of samples on expansive soil influences the insitu swell-consolidation characteristics of expansive soil. The observation made on the swell-consolidation test result of undisturbed samples of Galan Town show that the value of dry density ranges from 1.17 - 1.37 g/cm^3 and the initial moisture content ranges from 30.60-50.06%. As the test results indicate the swelling potential of expansive soils of Galan Town for undisturbed sample is found to range from 2.18 to - 6.83 %. The values obtained from the test results are evident for the expansiveness of soil of Galan town. The test result obtained from free swell test which ranges from 175 to 230% showed that the expansive soil of the study area is categorized under high degree of expansion.

The other observation made on swell-consolidation test result of undisturbed samples of Galan Town shows that the value compression index C_c of the Galan Town expansive soil is within the range of 0.240-0.340. The value of coefficient of consolidation C_v from the undisturbed expansive soil sample is found to be in the range of 0.32-0.42 m2/year. As the test results indicate the swelling potential found to range from 2.89 to - 6.83 % whereas the value of swelling pressure ranges from 150 to 270 presented. This shows that the initial condition of the soils & value of consolidation characteristics C_c & C_v vary from place to place, based on the depth from which the samples were collected and the moisture content during the sample collection. The swell consolidation test results on the undisturbed samples are presented in Table 5.1.

Table 5.1 Results of laboratory tests conducted on initial condition & swell – consolidation characteristics of undisturbed expansive soil samples

Sample ID	Depth (m)	Sample No	Sample Type	Initial Moisture content (%)	Initial dry density	Swelling Potential %	Swell Pressure kPa	Consolidation Index C_c	Consolidation Coef. C_v m^2/Year
Pit-1	1.5	1	Undisturbed	45.77	1.18	3.89	180	0.34	0.38
Pit-1	2.8	2	Undisturbed	50.06	1.17	2.18	150	0.33	0.41
Pit-2	1.5	3	Undisturbed	36.48	1.30	4.83	270	0.29	0.32
Pit-2	2.35	4	Undisturbed	30.60	1.29	3.15	210	0.24	0.42
Pit-3	1.5	5	Undisturbed	30.66	1.37	6.83	250	0.30	0.39

On the other hand, laboratory standard compaction tests have been conducted on disturbed samples to obtain the compaction curves. Data of the compaction curves were used to remold the samples from which test specimens have been prepared for the swell-consolidation tests. This technique has given a clear guide to utilize the compaction curve in such a way that selecting appropriate location of points on the curve from where to prepare the test specimens have to be prepared by varying initial moisture content & dry density. Three points were selected from wet side & dry side of the compaction curve including the point of optimum moisture content (OMC) & maximum dry density (MDD).

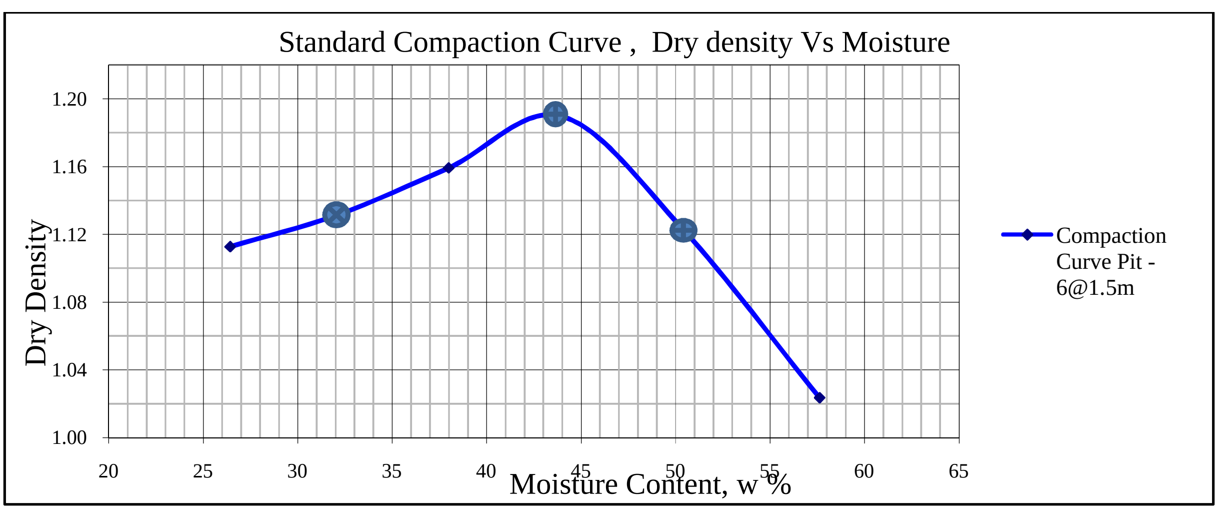


Figure 5.1 Compaction curve obtained from sample-9 (Pit 9 @1.5m depth) & used for remolding

Based on the points selected on the compaction curve, test specimens were prepared by remolding using different initial moisture content & initial dry density determined from the compaction curve. The specimens were prepared in the consolidometer ring from the remolded sample and mounted on consolidometer. As usual, conventional swell - consolidation tests have been conducted on a number of samples. From the tests conducted, the following typical e-log P curves have been obtained as presented in the body of this paper and the remaining is annexed at the end of the paper.

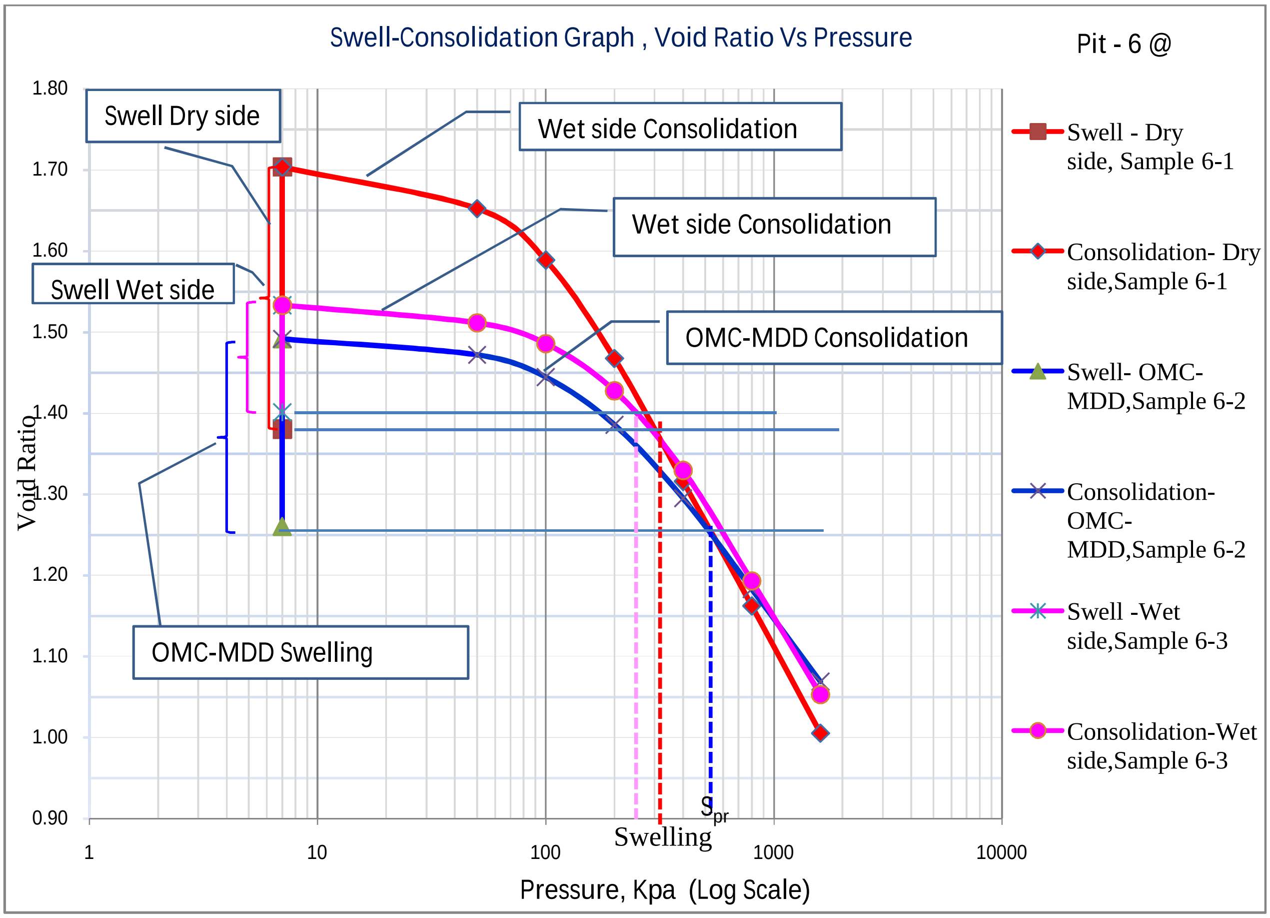


Figure 5.2 e- log P Plot of remolded samples at Dry side (ρ_d = 1.13, w=32%), OMC-MDD, (ρ_d = 1.19 and w=44%) & Wet side of the compaction curve,(ρ_d = 1.12, w=50%)

As can be seen from the figures the swelling potential is generally decreasing from the dry side of the compaction curve to the wet side as the initial moisture content increases. It is because expansive soils are clay soils which are characterized by their swelling property when exposed to moisture fluctuation. However, the amount of swell for lower initial moisture content is greater than the sample with the same dry density & increased initial moisture content. This implies that the degree of expansion is considerably influenced by the amount of water absorbed to cause the swell based on the initial moisture content for constant dry density.

Table 5.2 Determination of swelling potential & swelling pressure of remolded samples by varying initial moisture content & dry density

Sample No	Depth (m)	Sample Type	Initial Moisture content (%)	Initial dry density g/cm3	Swelling Potential %	Swelling Pressure kPa
6-1	1.5	Remolded	32.04	1.13	13.58	300
6-2	1.5	Remolded	44.00	1.19	10.24	560
6-3	1.5	Remolded	50.00	1.12	5.51	270
7-1	2.9	Remolded	27.00	1.13	13.24	280
7-2	2.9	Remolded	42.00	1.18	11.20	380
7-3	2.9	Remolded	47.00	1.13	7.31	300

It is evident from test result that the initial water content has a considerable influence on the percentage swelling of the soil samples. This is because as the initial water content increases, for specimens having the same initial dry density, the initial degree of saturation will also increase and the affinity of soil to absorb water will decrease. It follows that the amount of water absorbed for complete saturation will become smaller, and consequently the amount of swell will decrease as the initial water content increase.

On the other hand dry density is an important factor in determining the magnitude of volume change. The swell & swelling pressure 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 behavior 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 wetting it results in high volume change.

The 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 complete saturation, more water is required & causes high swelling potential which results in high degree of disturbance. This can be observed from Table 5.1 sample- 4 & 5 which have the same initial moisture content of 30.60 & different initial dry density of 1.29 & 1.37 g/cm3 respectively. Accordingly, the corresponding swelling potential and swelling pressure obtained from the test result are found to be 3.15%, 6.83% and 210kPa, 250kPa respectively.

Samples on the dry side of the compaction curve showed a maximum swelling potential which is high degree of disturbance. When it is subjected to vertical pressure, the rate of consolidation is found to be higher than samples both on at OMC-MDD & wet side of the compaction curve. Consequently the e-log P graph on the dry side of the compaction curve is steepest while the e-log P graph at wet side of the compaction curve is steeper & the e-log P graph of samples with OMC & MDD is steep as presented in Figure 5.2. This is because at OMC & MDD, the voids are filled with the soil particle and optimum moisture to attain its maximum dry density. Because of soils with maximum dry density has low permeability, it requires more time & pressure for final consolidation to take place. Due to this the consolidation curve gets smaller slope which results in smaller consolidation index (C_c).

On the other hand, on wet side of the compaction curve, the amount of soil particle is reduced while the moisture content is increased for the same volume. This causes increased in void ratio & degree of saturation. As a result when load is applied, the rate of consolidation becomes slow sine the large amount of water with in the void require more pressure & time to escape based on the permeability of the sample. So the graph shows lesser slope than the e-log P graph of dry side as clearly presented in Figure 5.3 &.5.4

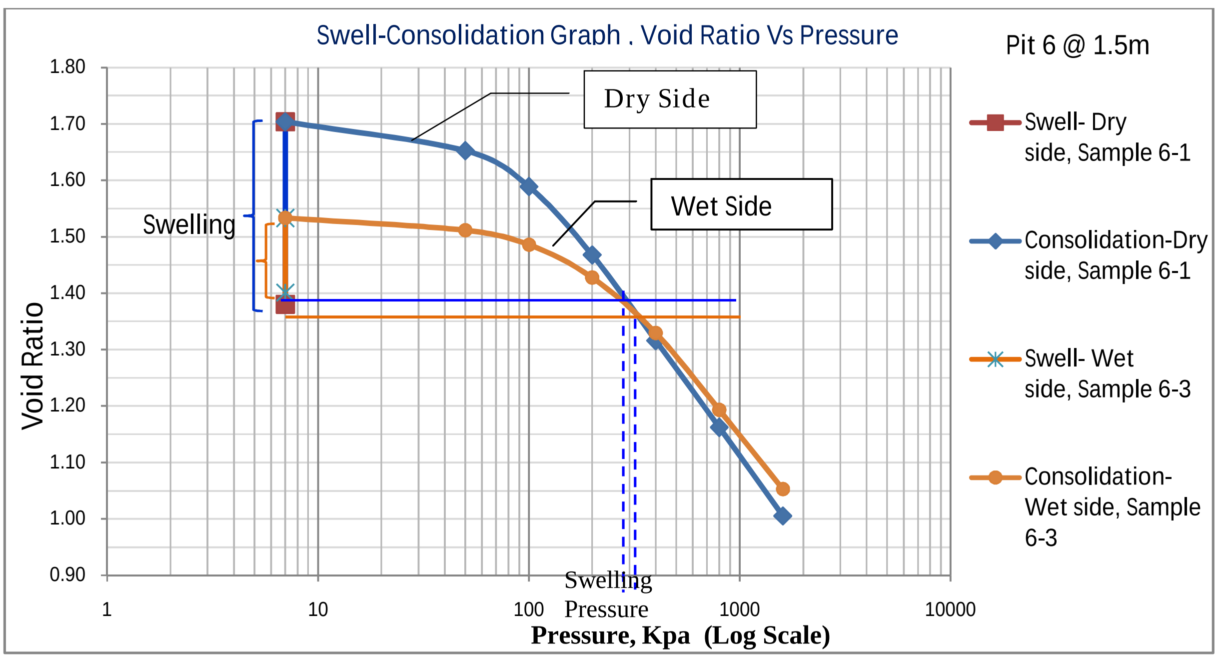


Figure 5.3 Plot of e- log P (dry side of the compaction curve), (ρ_d = 1.13, w=32%) & wet side of the compaction curve, with initial condition of (ρ_d = 1.12, w=50%).

The swelling pressure was obtained from the e-log pressure curve during the early loading of the sample. Further conventional loading beyond the swelling pressure has caused the soil to undergo consolidation from which the consolidation characteristic has been determined. According to the test results, in remolded sample, the value of swelling pressure ranges from 270 to 560 kPa. The swelling pressure of samples with OMC & MDD is greater than both samples from dry & wet side of the compaction curve. This is because at OMC & MDD due to its closer particles, when subjected to moisture high degree of swelling is obtained.

After final swelling & saturation, it requires more time to bring the sample to its original void ratio based on the permeability of the sample when subjected to loading. 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 Figure 5.2.

It can be summarized that, swelling potential of expansive soils decrease with increase in initial moisture content for constant dry density. The value of swelling pressure at OMC & MDD is greater than the swelling pressure obtained from the wet & dry side of the standard compaction curve.

The consolidation index (C_c) of samples with low initial moisture content & dry density is higher & with OMC & MDD the value consolidation index is lower while for wet side of the compaction curve, the value of C_c is in between.

At lower initial moisture content & dry density, the swelling potential is larger as compared to increased initial moisture content. The behavior of e-log P curve, starting from the early loading shows high rate of consolidation. The e-log P graph is, therefore, steeper which results in larger value of consolidation index (C_c).

On the other hand, the rate of consolidation depends on the permeability, void ratio effect (density and volume of void & particles in the samples). Near the OMC & MDD the voids available for moisture movement are at a minimum because higher densities result in closer particle spacing, therefore causing greater particle interaction. As a result, the sample compacted OMC & MDD requires more time & pressure for the final consolidation to take place and resulted in flatten e-log P curve become less steep. (Figure 5.2). For this reason, the value of consolidation index C_c obtained smaller.

Samples with high initial moisture content & low dry density has lower rate of consolidation as more time & pressure is required to dissipate the water for final consolidation to take place. As the result the e-log P curve has lesser slope as compared to samples with low initial moisture content & dry density hence, smaller value of C_c was obtained. (Figure 5.3 &.5.4)

High degree of expansion in expansive soil indicates high degree of disturbance in the soil mass. When subjected to overburden pressure, soil particles rearrange themselves to the new pressure & results in high rate of consolidation settlement. Therefore, the value of consolidation index (C_c) varies with degree of expansion, degree of disturbance, degree of saturation, permeability & void ratio effects, i.e. density of the soil & the volume of voids.

The test results of remolded samples which were prepared from same dry density & different moisture content has shown a different consolidation characteristics, i.e. different compression index. (Figure 5.4, Table 5.3) for instance, soil sample 7-1 and sample 7-3 had the same dry density of 1.13 g/cm^3, but different initial moisture content (dry side & wet side of the compaction curve). The corresponding compression indices (C_c) are 0.65 and 0.49, respectively. It can be easily observed that, the samples subjected to more swelling or disturbance show a greater value of compression index. Therefore, initial moisture content influences the swelling & consolidation characteristics of expansive soils.

The effect of swelling on consolidation characteristics can easily observed from the test results. Maximum swelling is observed on samples with lower initial moisture content & dry density & the resulted e-log P curve is steeper, thus the consolidation index C_c is obtained to be larger. On the other hand, for samples with lower dry density & large initial moisture content, smaller swelling is observed. During the consolidation, the behavior of the e-log P become flatten and as the result low consolidation index C_c is obtained**.** The swell-consolidation behavior of such soil is presented below in Figure 5.4.

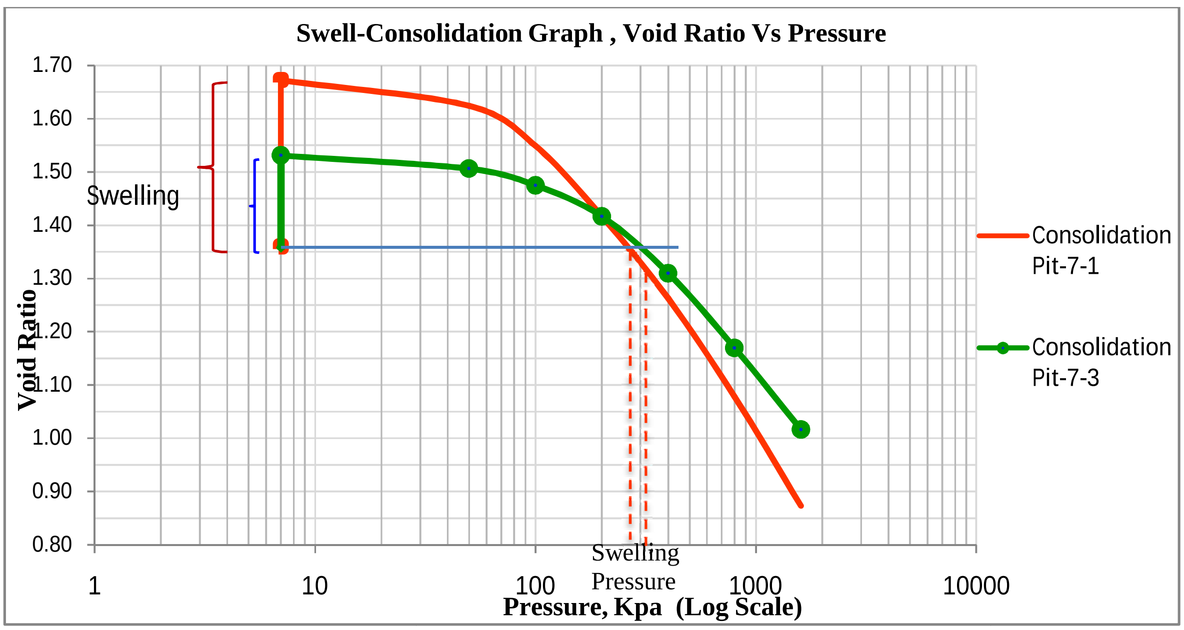


Figure 5.4 e-log P curve of remolded samples with the same (ρ_d = 1.13) & different, %w=27%, & wet side w=47 (dry & wet side)

As can be seen from the test results, those factors which affect the swelling characteristics of expansive soils have also affected its consolidation characteristics. Therefore, one of the factors anticipated to have an effect on consolidation characteristics of expansive soils is its swelling behavior which depends on the initial moisture content, amount of water absorbed by the soil mass, degree of disturbance & the density of the sample.

Swelling is a result of disturbance in internal stress equilibrium of the soil particle by change in moisture content. To attain new internal stress equilibrium, the soil particles start to swell and a new particle arrangement will take place in the soil mass. Therefore, swelling brings about a soil of different particle arrangement and different consolidation characteristics in expansive soils.

On the other hand, as can be seen from Table5.3, the samples with maximum swelling pressure at OMC-MDD shows lowest value of compressibility index C_c

while the lower value of the swelling pressure on the dry & wet side of the compaction curve shows lower value of C_c. Furthermore, the value of compressibility index C_c of dry side of the compaction curve is smaller as compared to that of wet side as presented in Table5.3.

Table 5.3 Initial condition & swell – consolidation characteristics of remolded soil samples

Sample No	Depth (m)	Sample Type	Initial Moisture content (%)	Initial dry density	Swelling Potential %	Swelling Pressure kPa	C_c	C_v m^2/year
6-1	1.5	Remolded	32.04	1.13	13.58	300	0.52	0.26
6-2	1.5	Remolded	44.00	1.19	10.24	560	0.35	0.45
6-3	1.5	Remolded	50.00	1.12	5.51	270	0.46	0.34
7-1	2.9	Remolded	27.00	1.13	13.24	280	0.65	0.47
7-2	2.9	Remolded	42.00	1.18	11.20	380	0.37	0.38
7-3	2.9	Remolded	47.00	1.13	7.31	300	0.49	0.33

6. Conclusion & Recommendation

6.1 Conclusion

The following conclusions are drawn based on this study:

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

2. 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.
3. 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

4. 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.

5. 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.

6. 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.

7. 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.

6.2 Recommendation

1. In this research work samples of soil were collected only from seven test pits at deferent depth, in order to classify the expansive soil of Galan Town based on its swelling property. Further in depth investigation on engineering property by increasing the size of sample is very important in the future.

2. In order to increase the level of accuracy of the obtained laboratory test results, to verify the relationship between consolidation & swelling and also, to develop an extra relationships, it is recommended to carry out an extensive laboratory testing on engineering property of expansive soils of Galan Town in future research programs with more number of samples.

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Appendix A

Laboratory test results of Atterberg Limit tests from TP-1 @ 1.5m & 2.8m depth, TP-2 @ 1.5m & 2.35m & TP- 3 @ 1.5m depth, TP-4 @ 1.5m, TP-5 @ 1.5m & 2.6m & TP-6 @ 1.5m & 2.6m depth &TP-7@ 1.5m & 2.9m

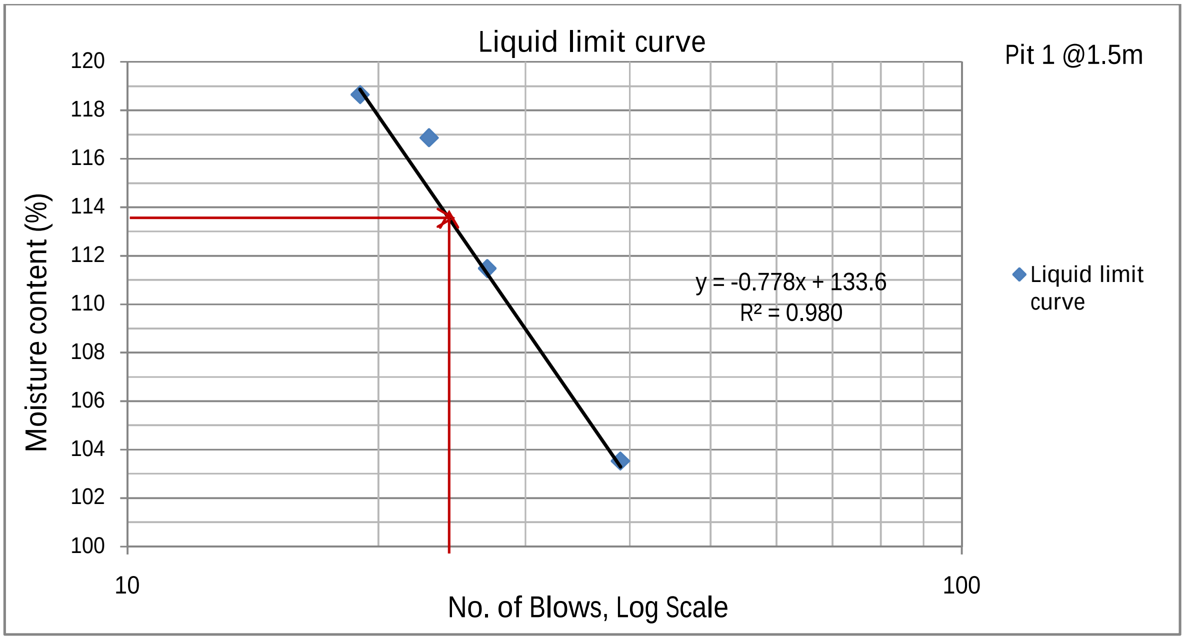

Figure A-1 Atterberg limit test results of TP-1 @ 1.5m, LL = 114%, PL = 35%, PI = 78%

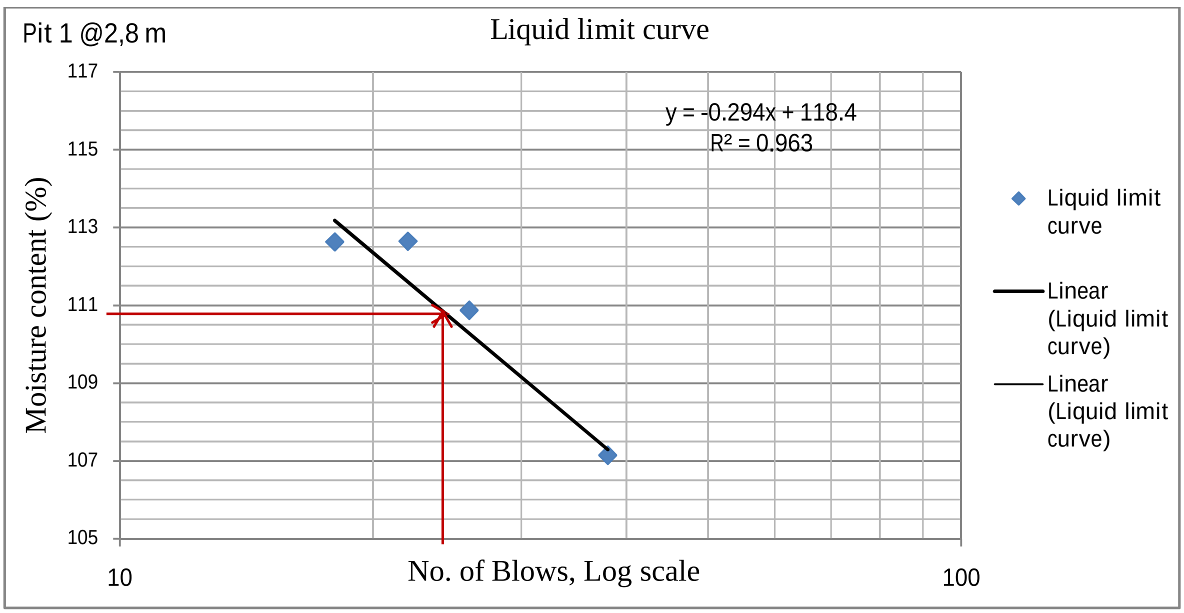

Figure A-2 Atterberg limit test results of TP-1 @ 2.8m, LL = 111%, PL = 38%, PI = 73%

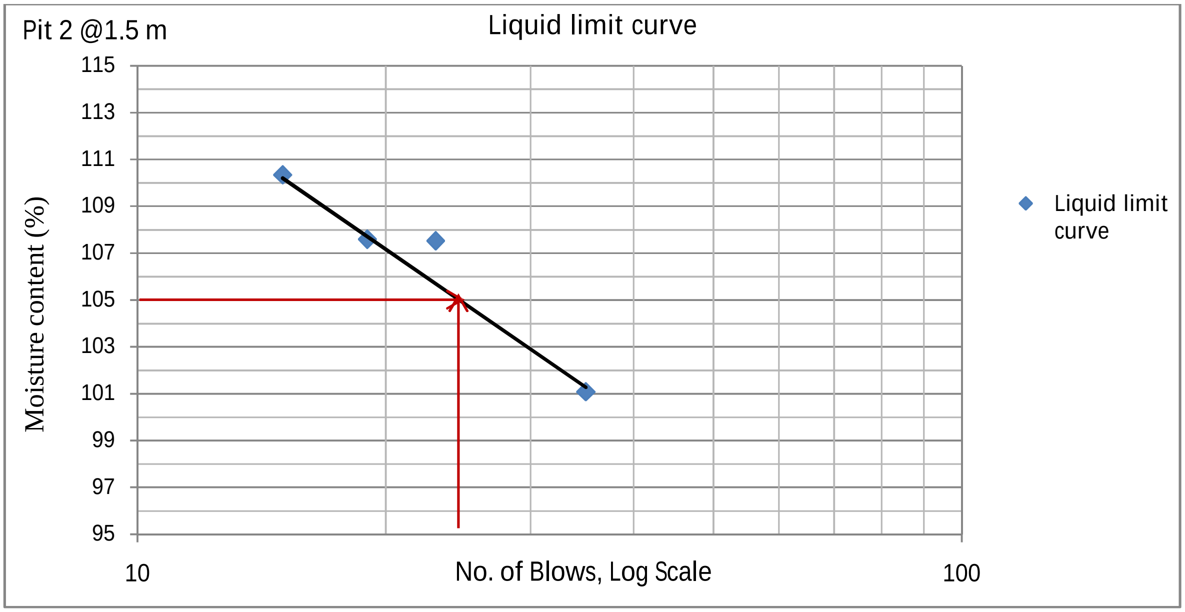

Figure A-3 Atterberg limit test results of TP-2 @ 1.5m, LL = 105%, PL = 32%, PI = 73%

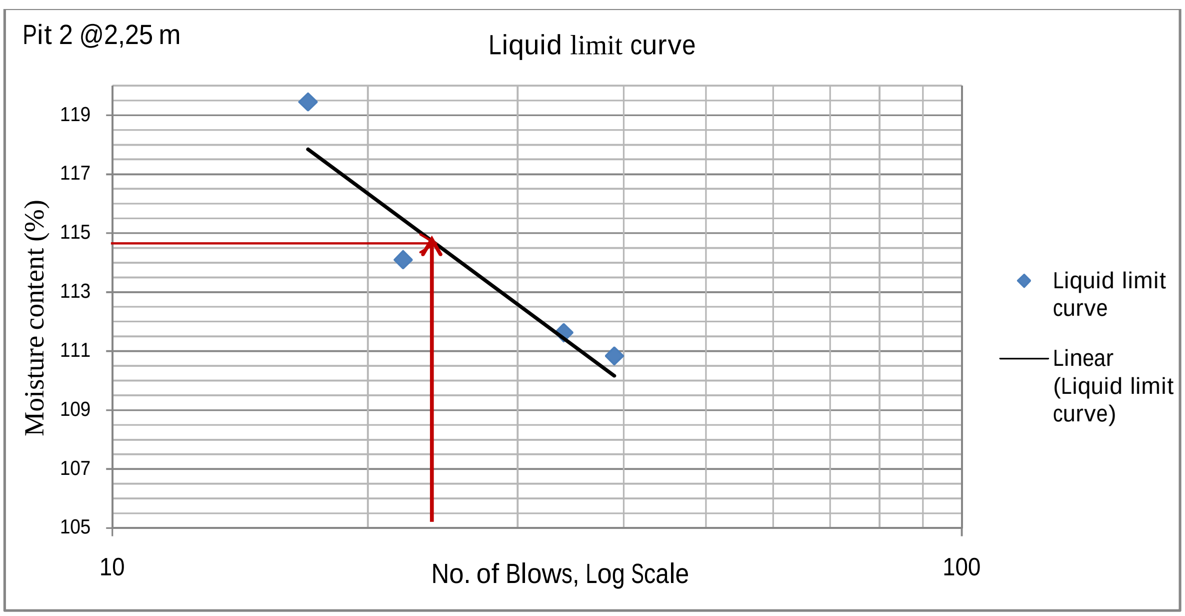

Figure A-4 Atterberg limit test results of TP-1 @ 2.8m, LL = 115%, PL = 29%, PI = 86%

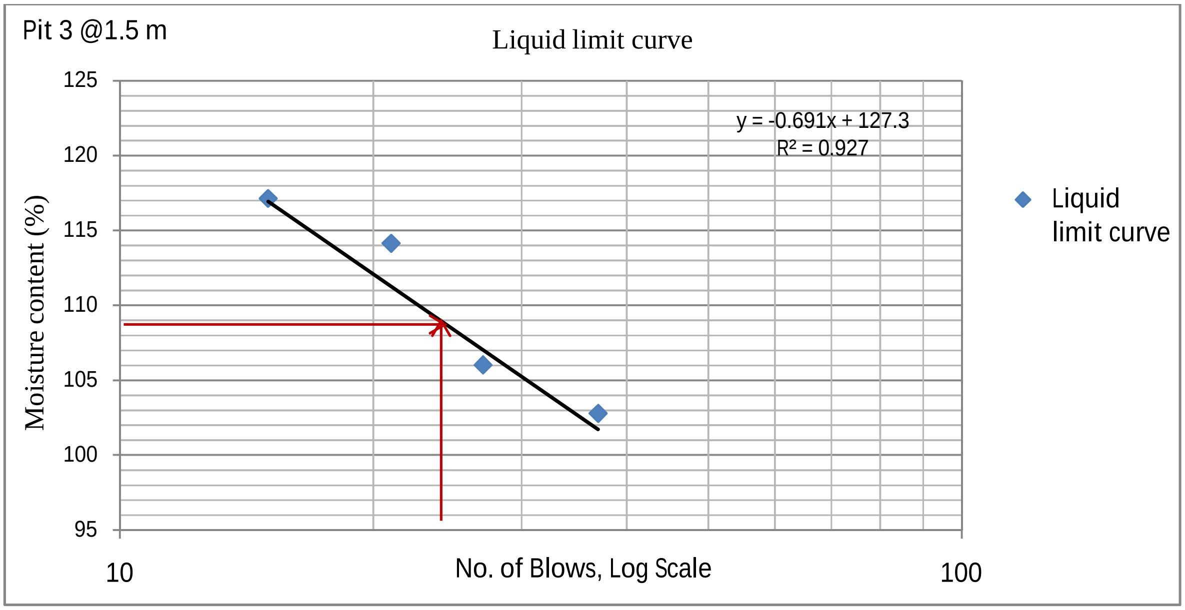

Figure A-5 Atterberg limit test results of TP-3 @ 1.5m, LL = 109%, PL = 30%, PI = 79%

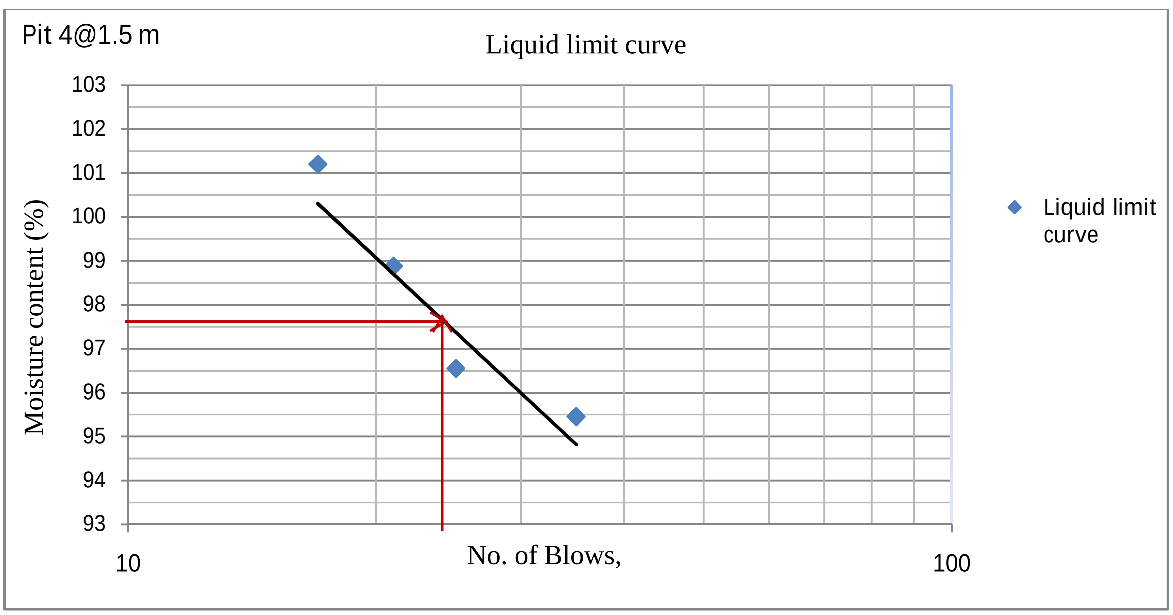

Figure A-6 Atterberg limit test results of TP-4 @ 1.5m, LL = 98%, PL = 36%, PI = 62%

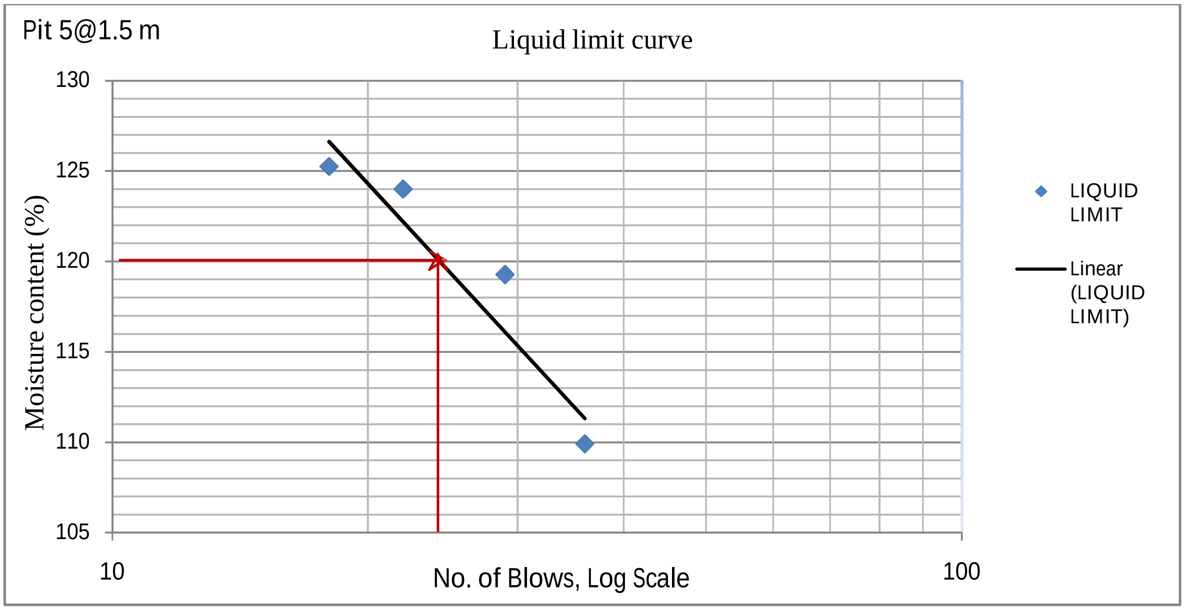

Figure A-7 Atterberg limit test results of TP-5 @ 1.5m, LL = 120%, PL = 31%, PI = 89%

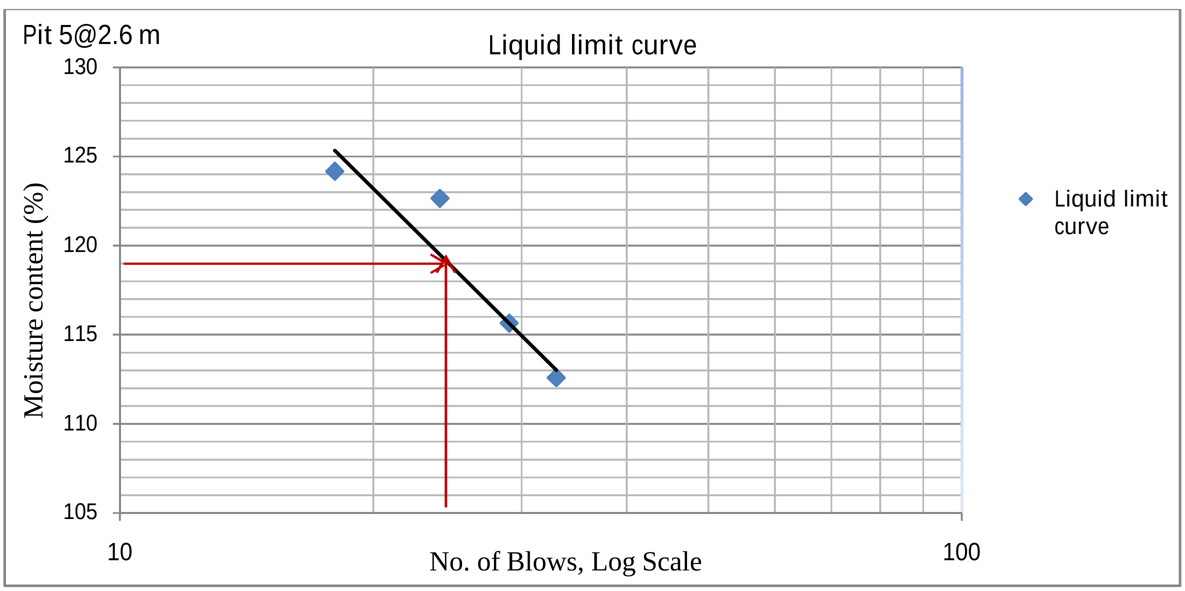

Figure A-8 Atterberg limit test results of TP-5 @ 2.6m, LL = 119%, PL = 29%, PI = 90%

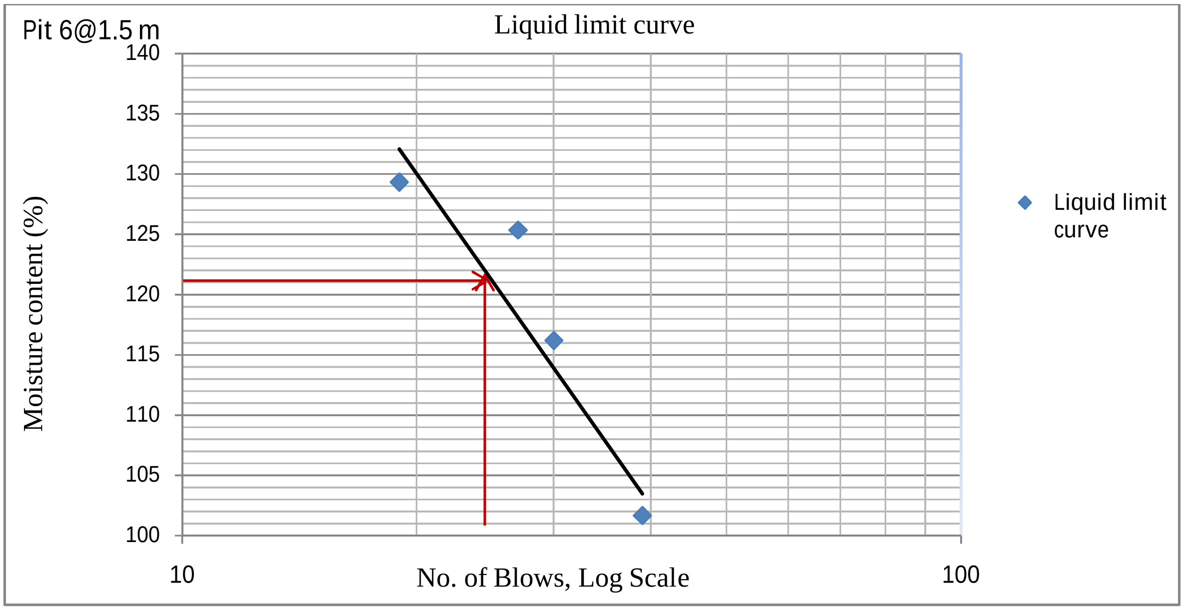

Figure A-9 Atterberg limit test results of TP-6 @ 1.5m, LL = 122%, PL = 32%, PI = 90%

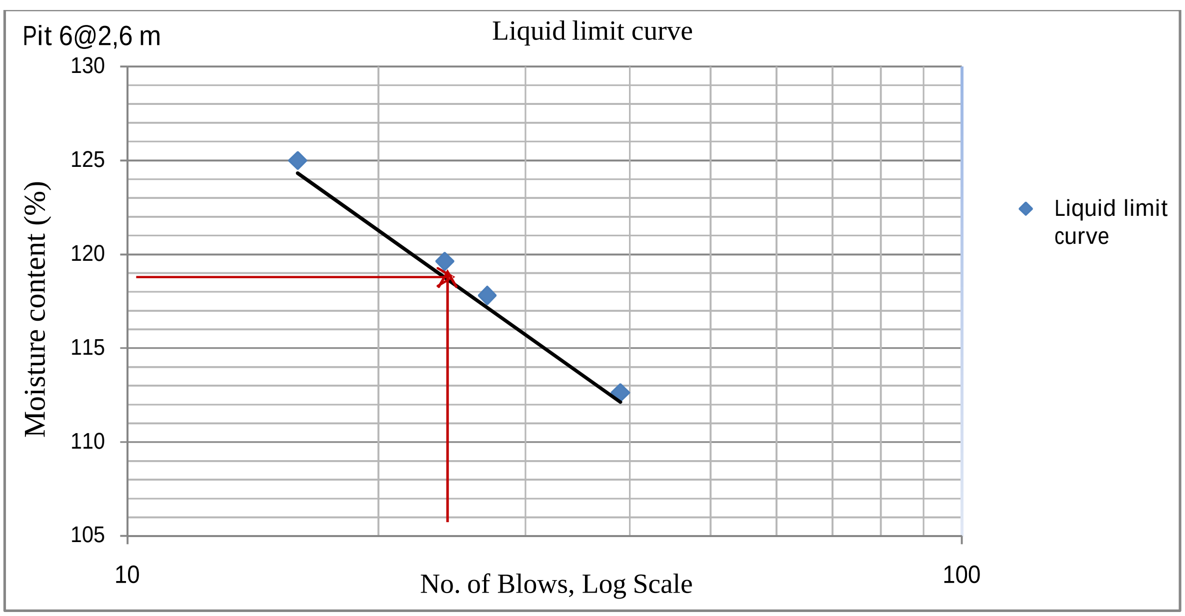

Figure A-10 Atterberg limit test results of TP-6 @ 2.6m, LL=119%, PL= 31%, PI = 88%

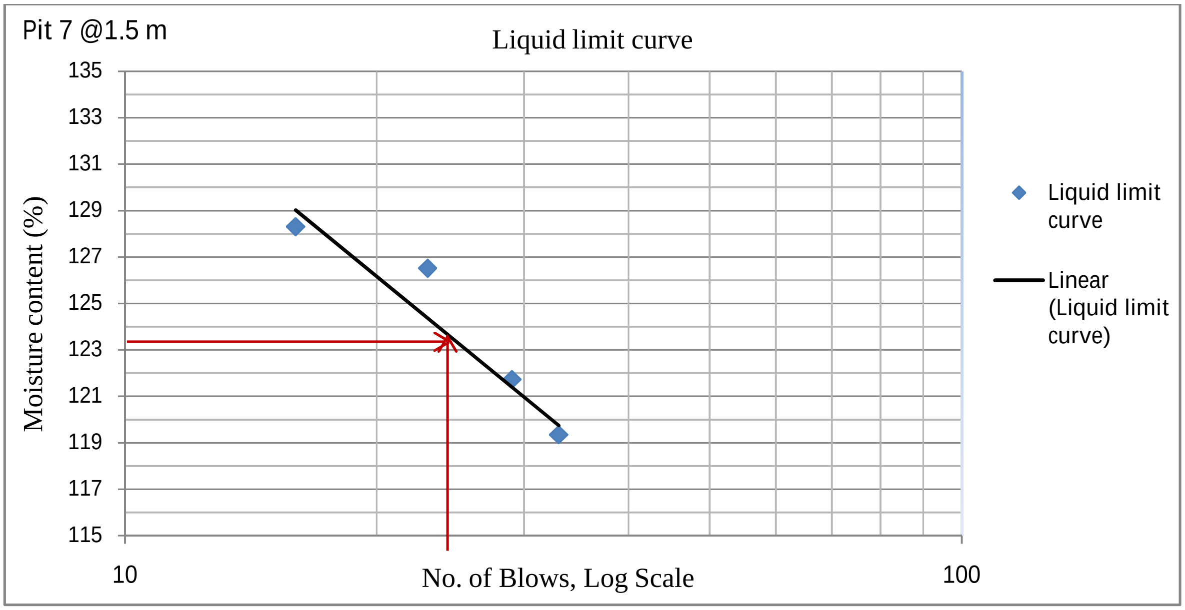

Figure A-11 Atterberg limit test results of TP-7 @ 1.5m, LL= 124%, PL= 30%, PI = 94%

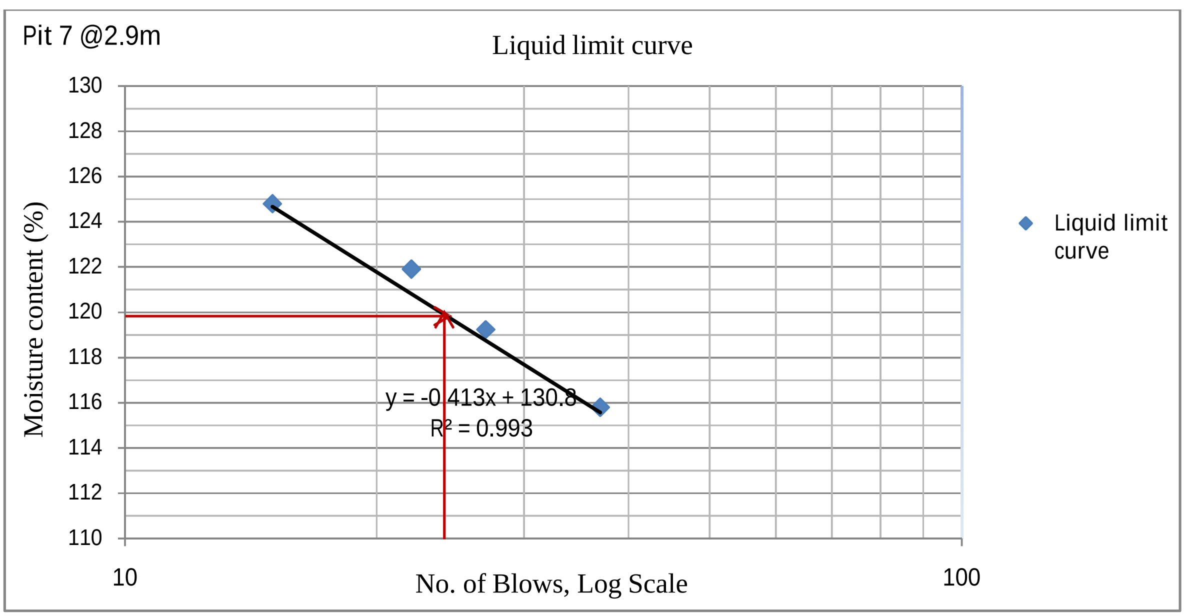

Figure A-12 Atterberg limit test results of TP-7 @ 2.9m, LL= 120%, PL= 30%, PI = 90%

Appendix B

Laboratory test results of Grain Size Distribution from TP-1 @ 1.5m & 2.8m depth, TP- 2 @ 1.5m & 2.35m & TP- 3 @ 1.5m depth, TP-4 @ 1.5m, TP-5 @ 1.5m & 2.6m & TP-6 @ 1.5m & 2.6m depth & TP-7@ 1.5m & 2.9m

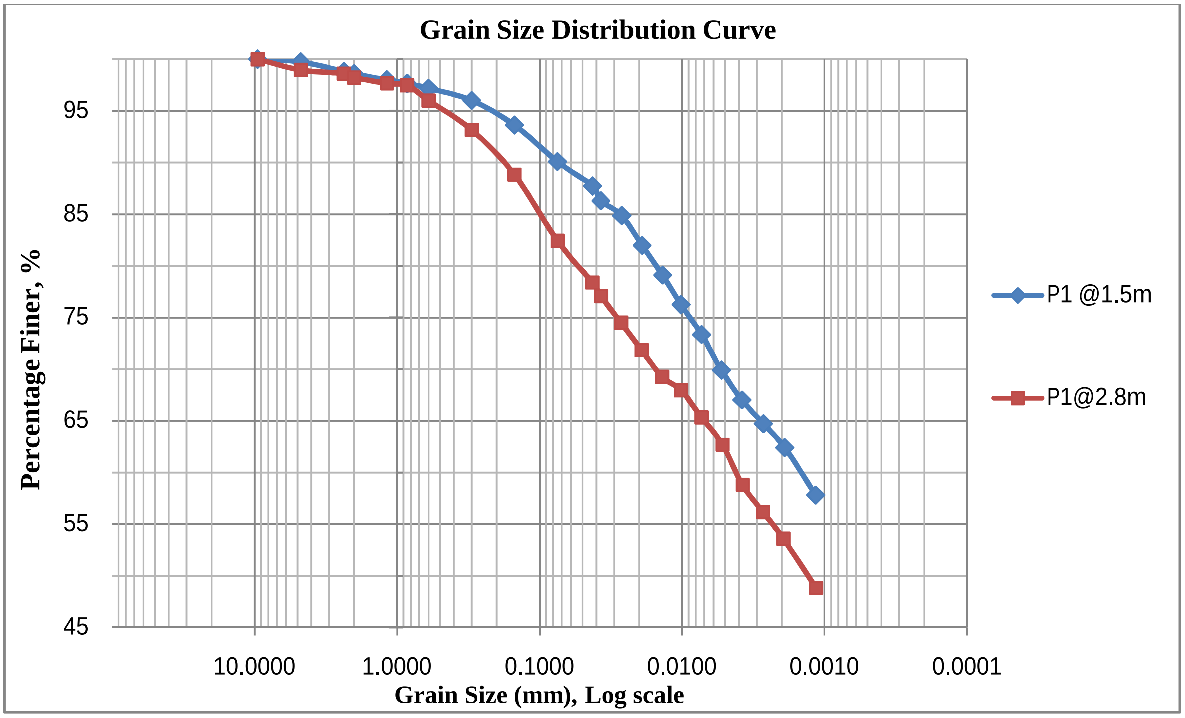

Figure B-1 Grain Size Distribution curve of TP-1 @ 1.5m & 2.8m depth

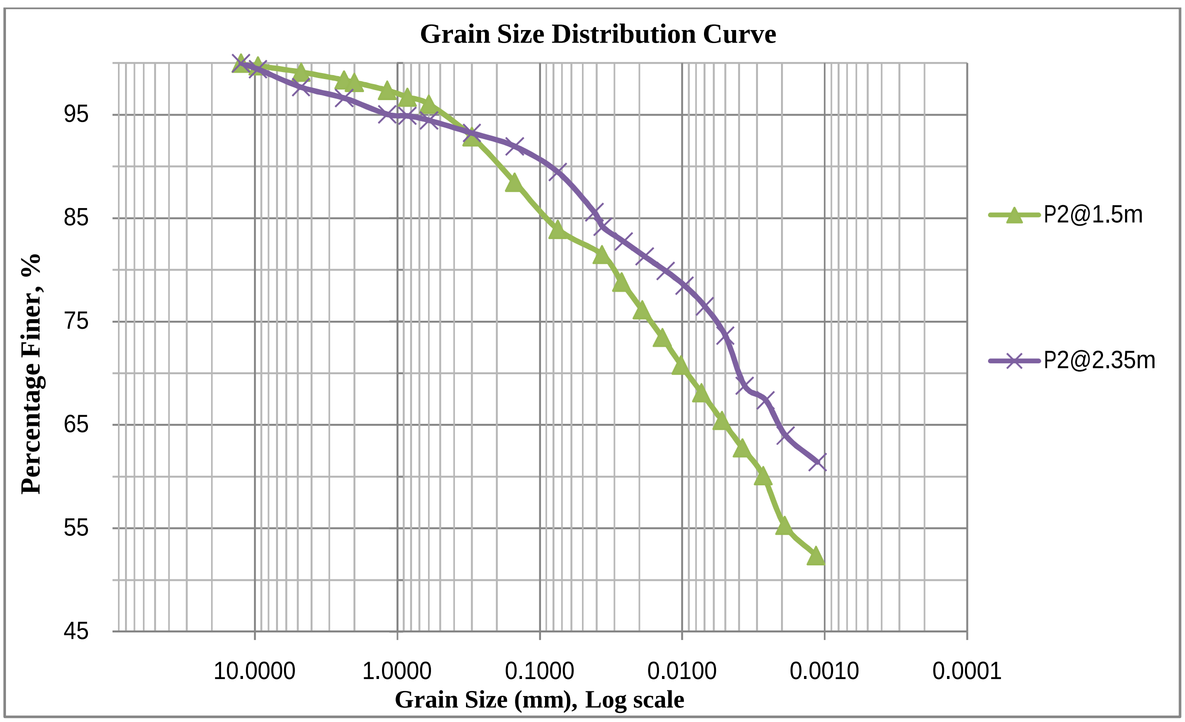

Figure B-2 Grain Size Distribution curve of TP-2 @ 1.5m & 2.35m depth

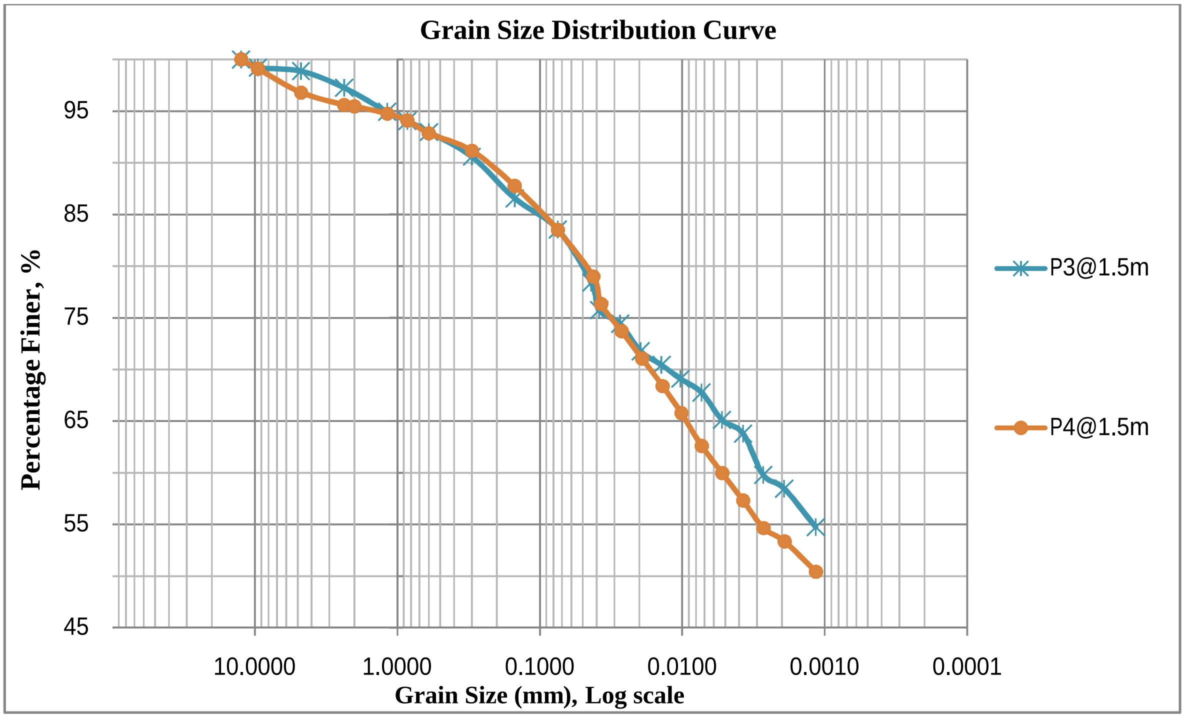

Figure B-3 Grain Size Distribution curve of TP-3 & TP-4 @ 1.5m depth

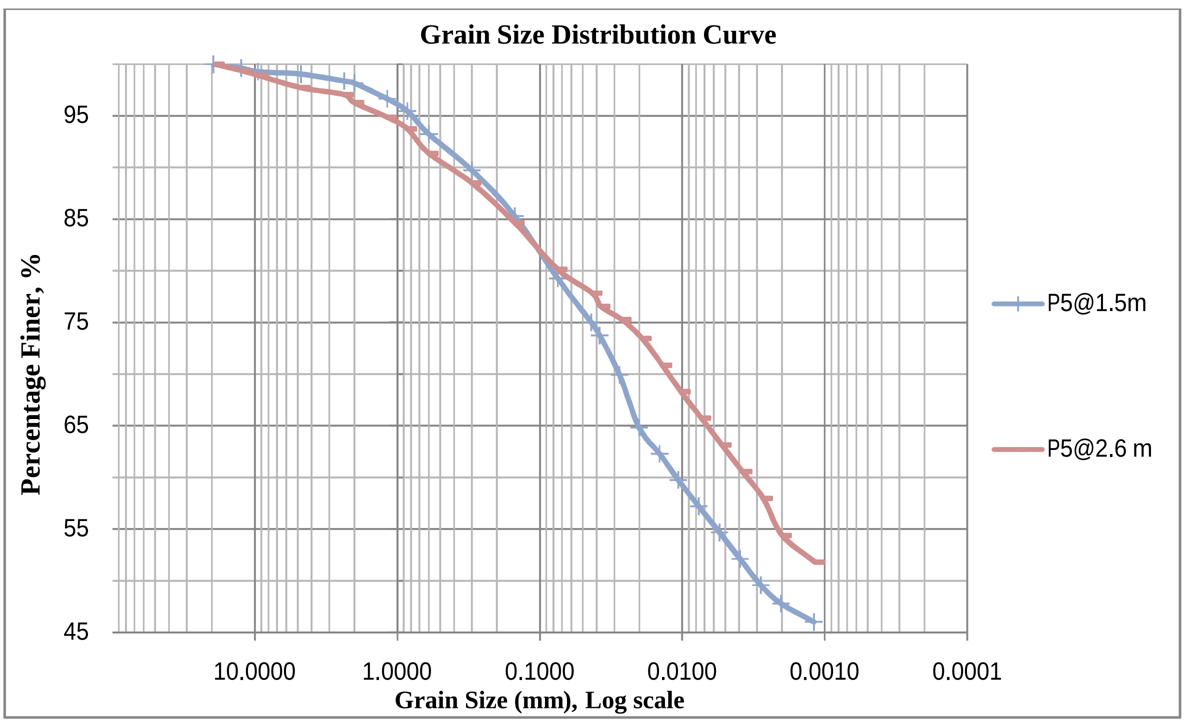

Figure B-4 Grain Size Distribution curve of TP-5 @ 1.5m & 2.6m depth

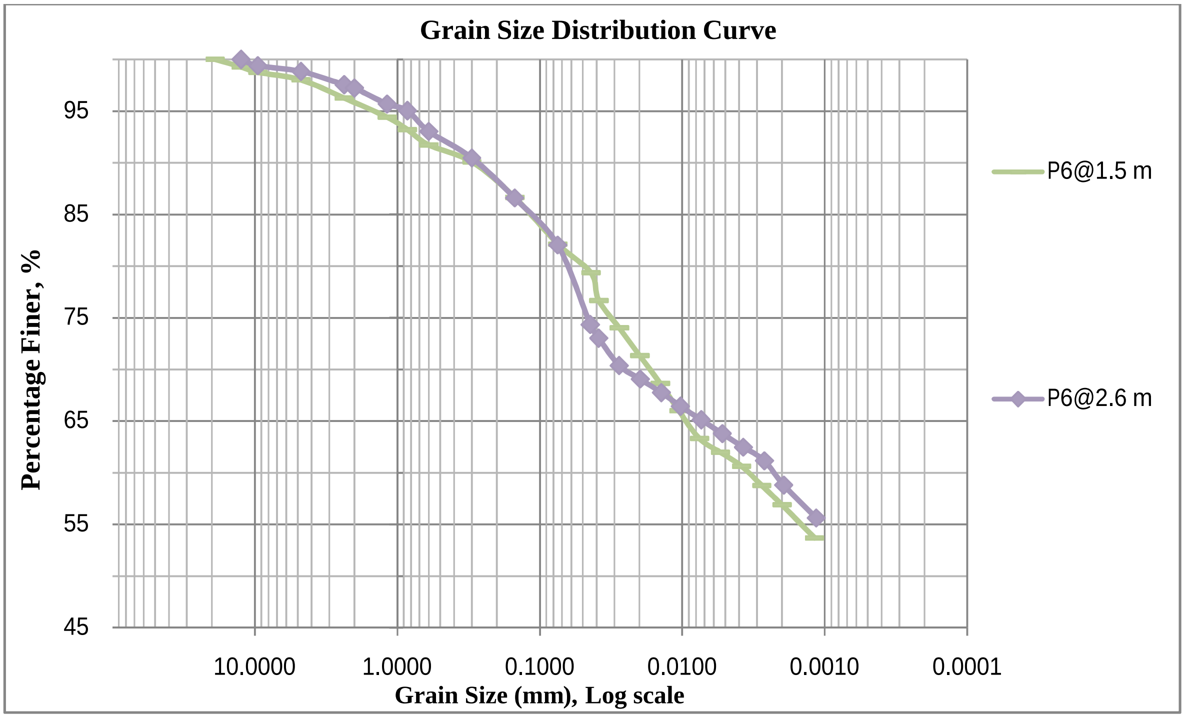

Figure B-5 Grain Size Distribution curve of TP-6 @ 1.5m & 2.6m depth

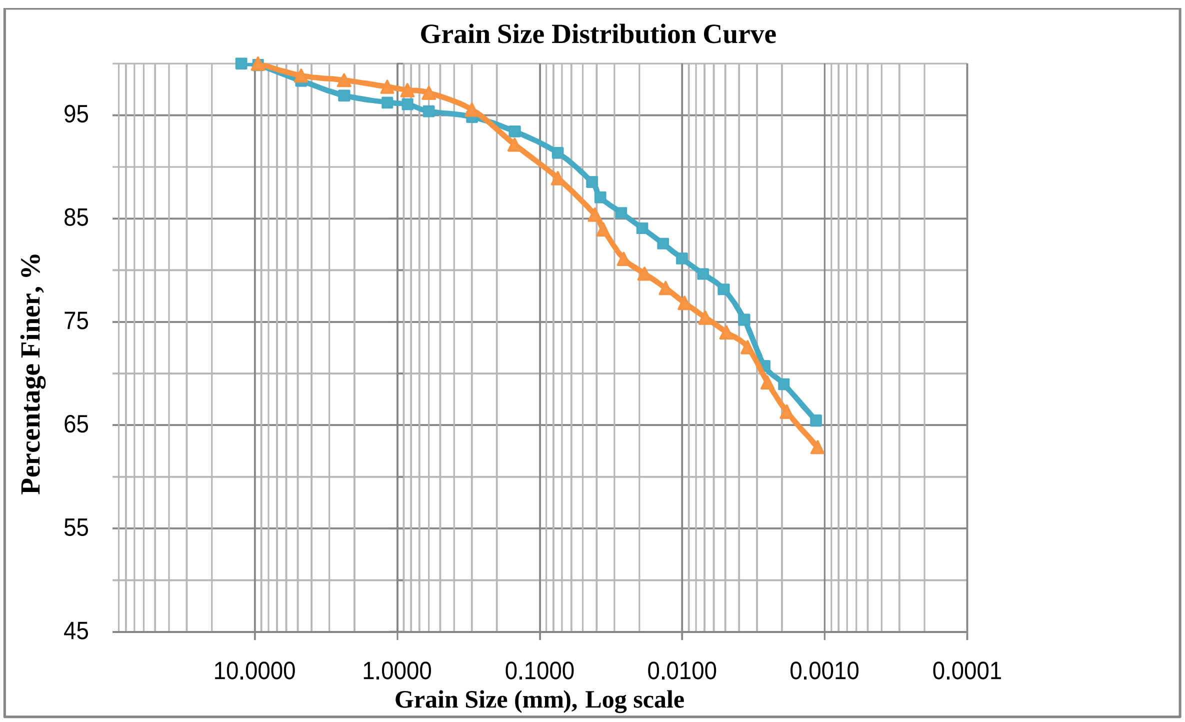

Figure B-6 Grain Size Distribution curve of TP-7 @ 1.5m & 2.9m depth

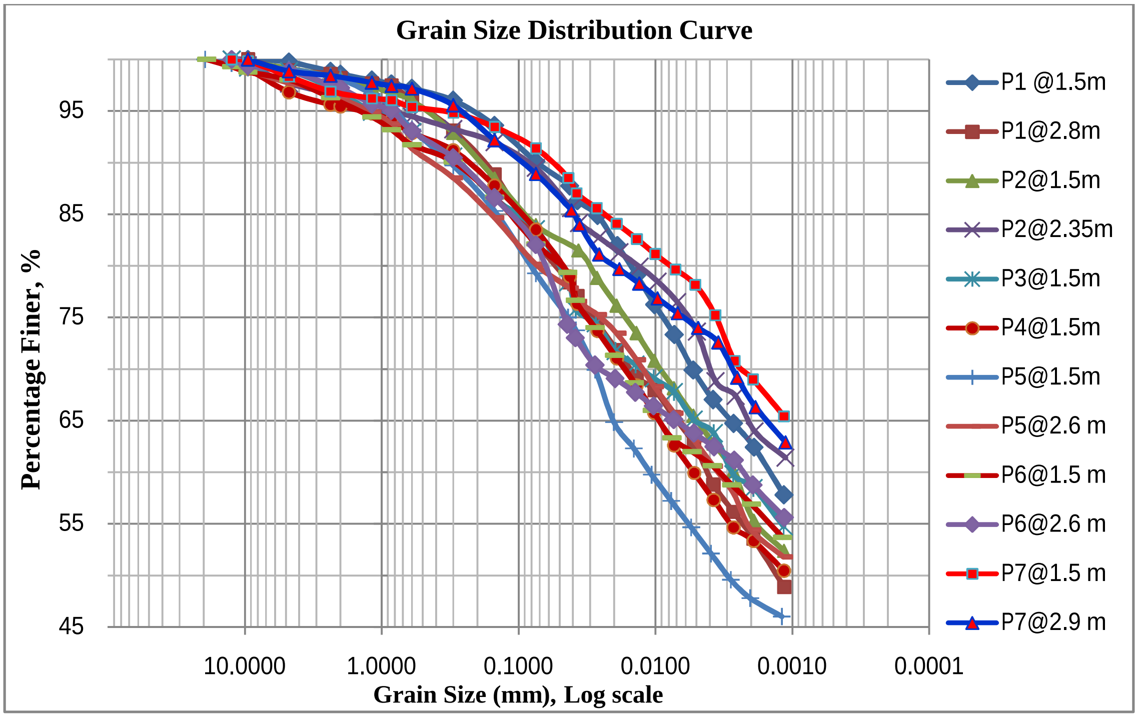

Figure B-7 Grain Size Distribution curve of all pits

Appendix C

Laboratory test results of undisturbed expansive soil samples prepared from Pit 1 @ 1.5m & 2.8m depth, pit 2 @ 1.5m & 2.35m & pit 3 @ 1.5m depth, which are sample 1,2,3,4 & 5 respectively.

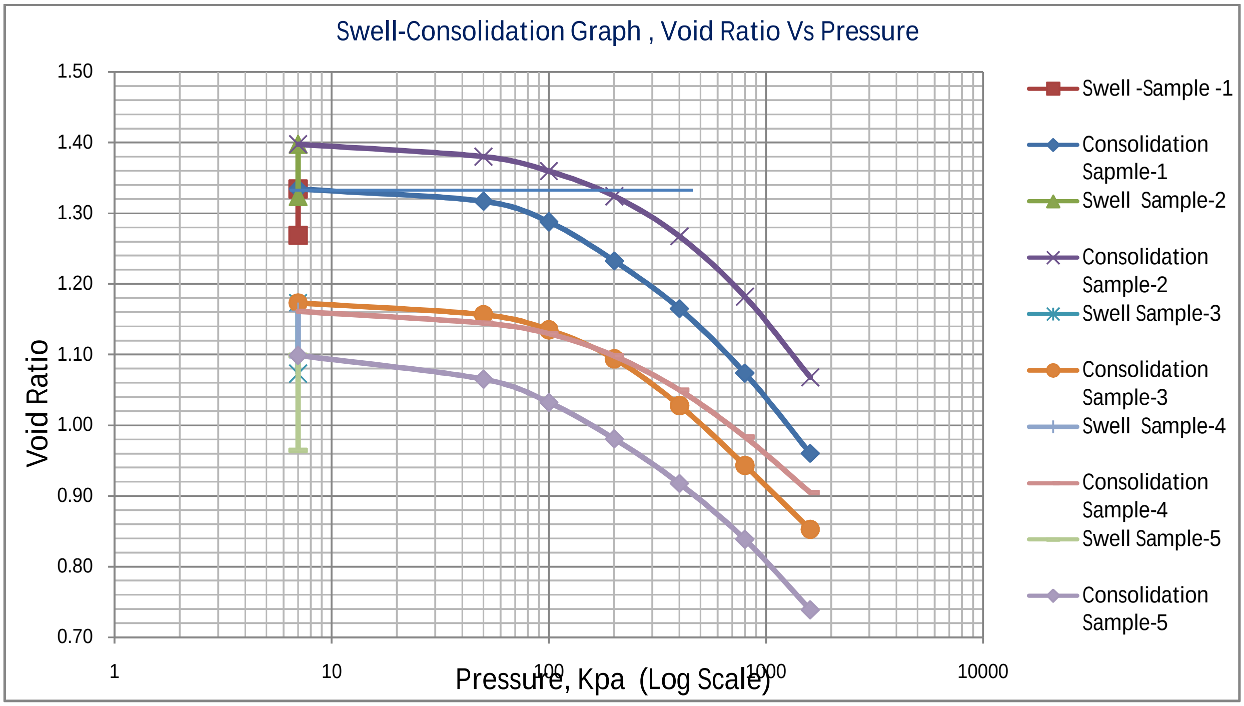

Figure C-1 Plot of e-log P of undisturbed soil of samples 1, 2, 3, 4, & 5 with initial condition (ρ_d =1.18, w=45.77%), (ρ_d =1.17, w=50.06%), (ρ_d =1.30, w=36.48%), (ρ_d =1.29, w=30.60%), & (ρ_d =1.37, w=30.66%), respectively.

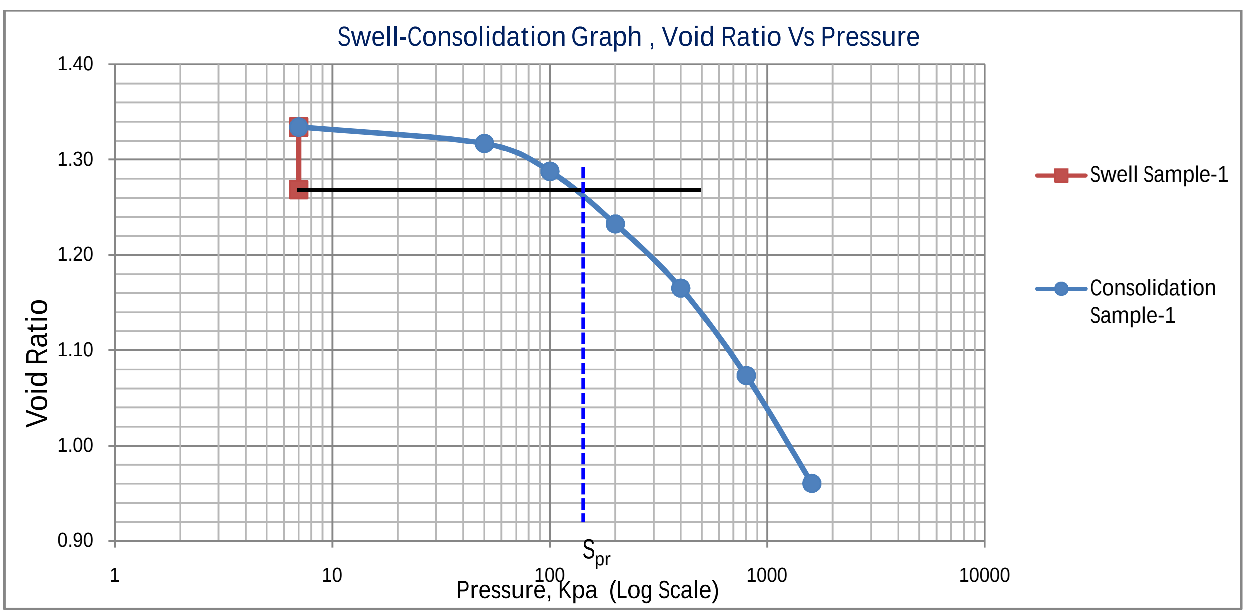

Figure C -2 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.18, w=45.77%) sample1,

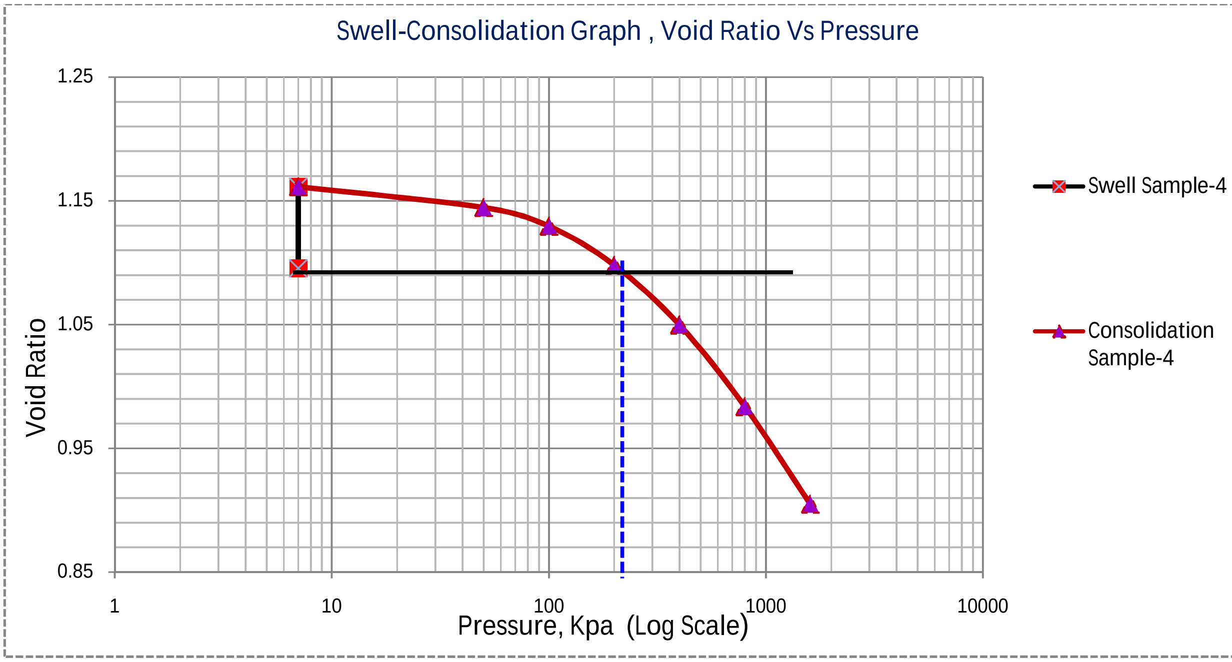

Figure C -3 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.29, w=30.60%) sample 4,

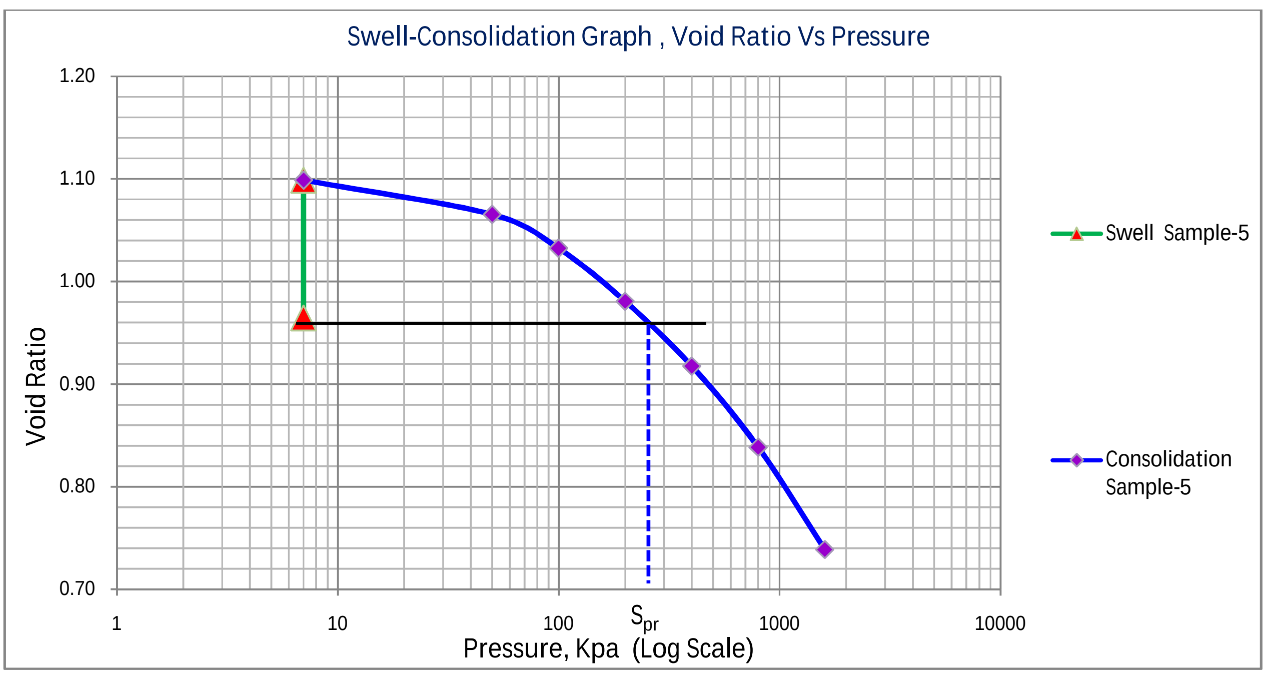

Figure C -4 Plot of Void ratio verses log Pressure of undisturbed samples & Determination of Swelling & Swelling Pressure, with initial condition (ρd = 1.37, w=30.66%) sample-5,

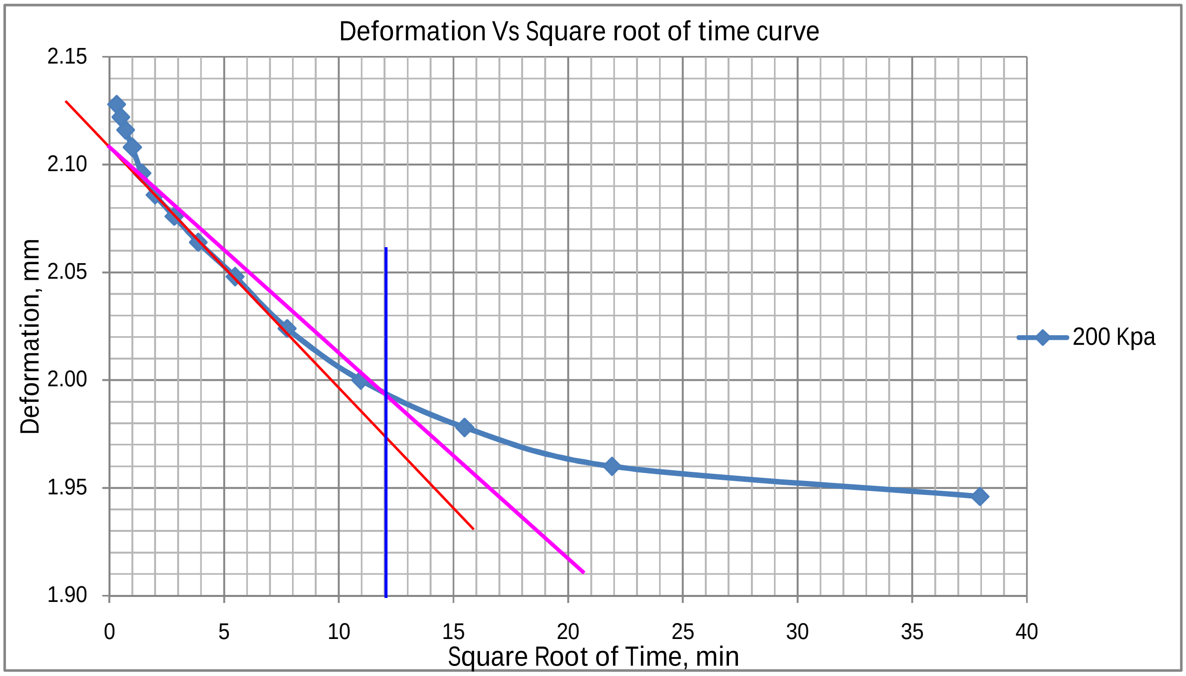

Figure C -5 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 1 (P=200KPa),

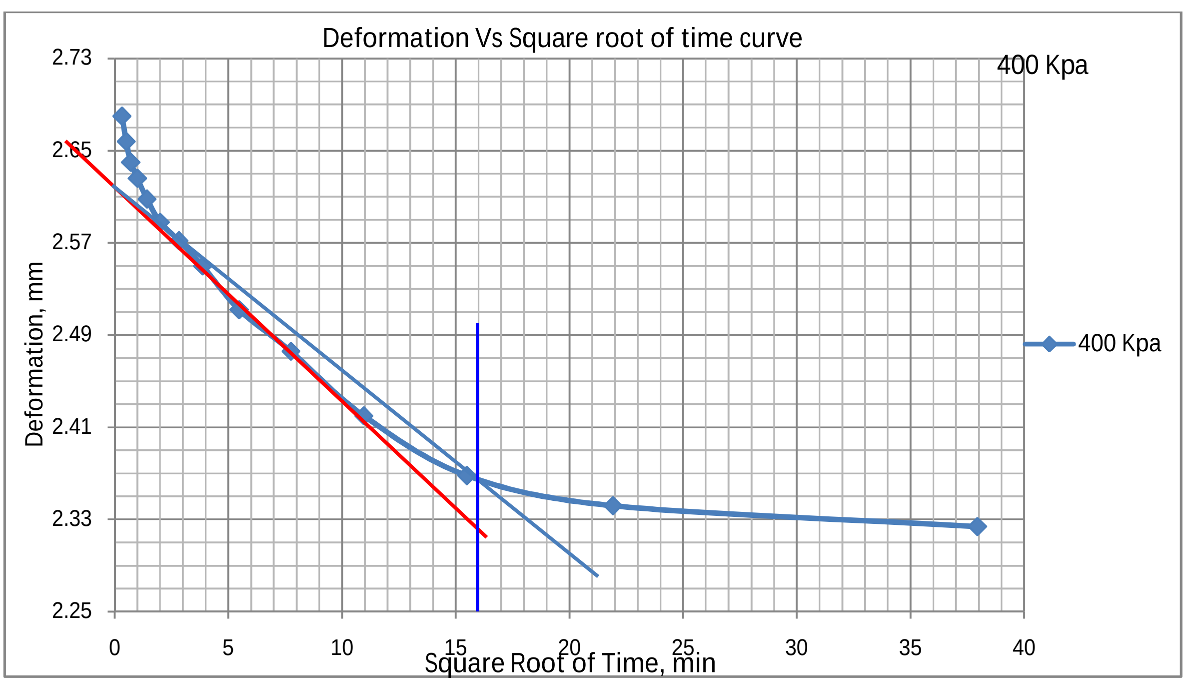

Figure C -6 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 1 (P=400kPa),

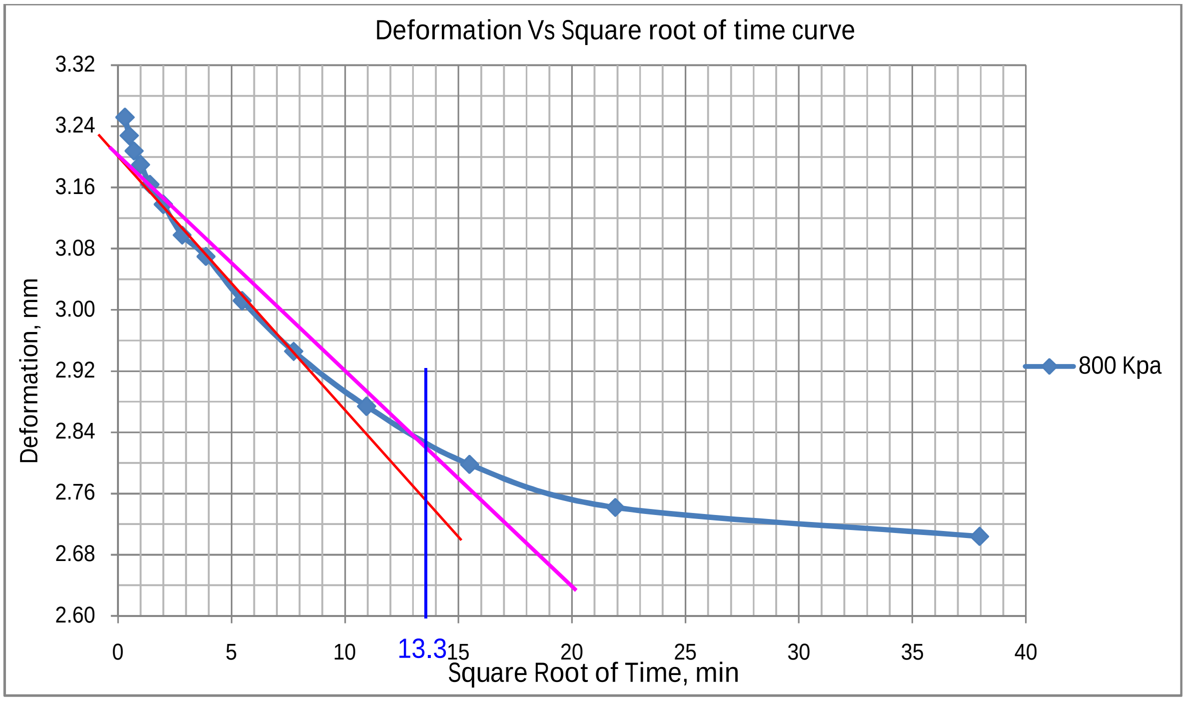

Figure C -7 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 1 (P=800kPa),

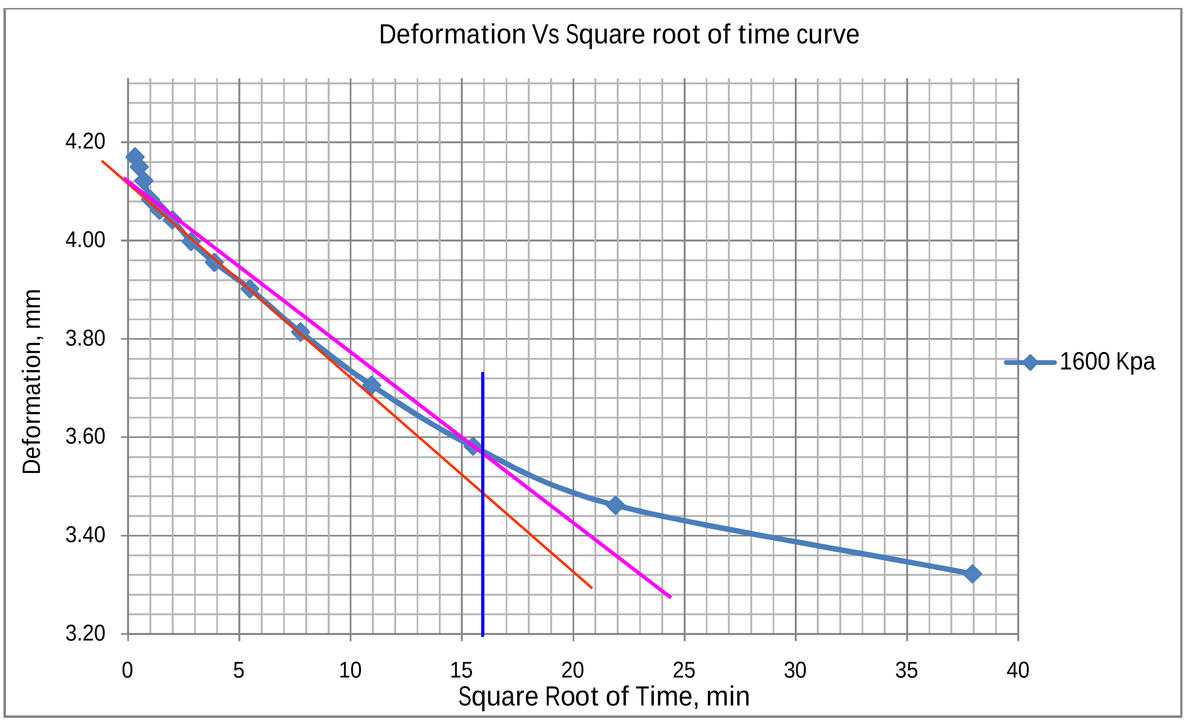

Figure C -8 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 1 (P=1600kPa),

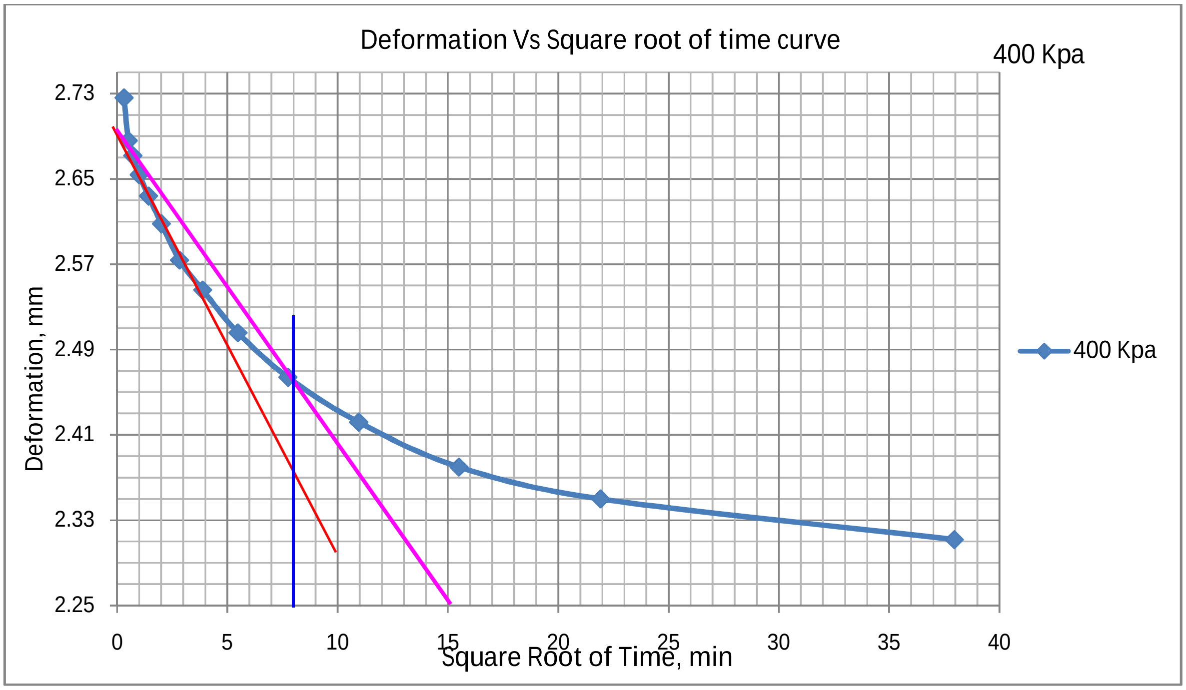

Figure C -9 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 2 (P=400kPa),

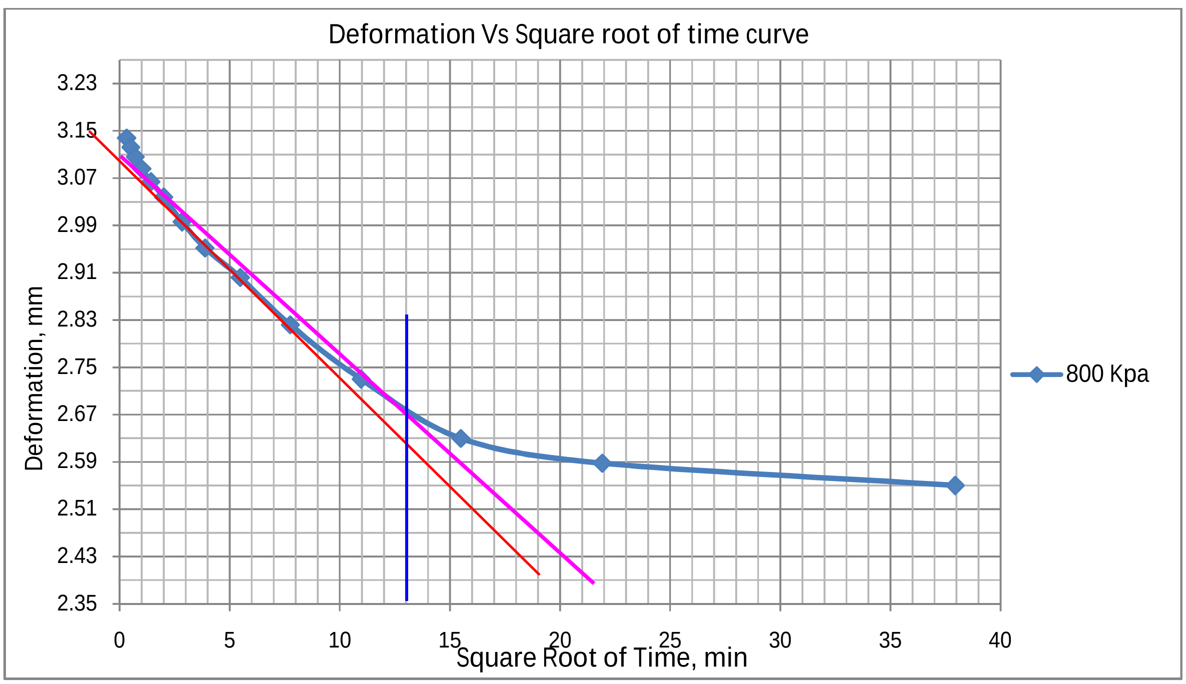

Figure C -10 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 2 (P=800kPa),

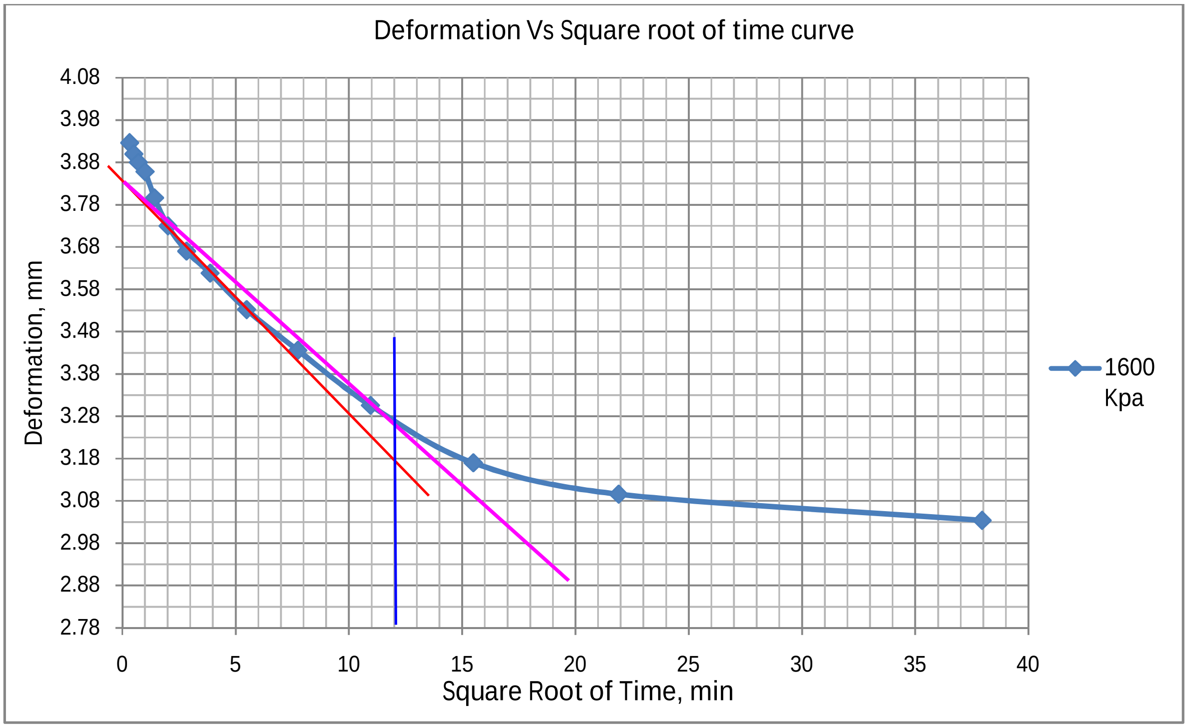

Figure C -11 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 2 (P=1600kPa),

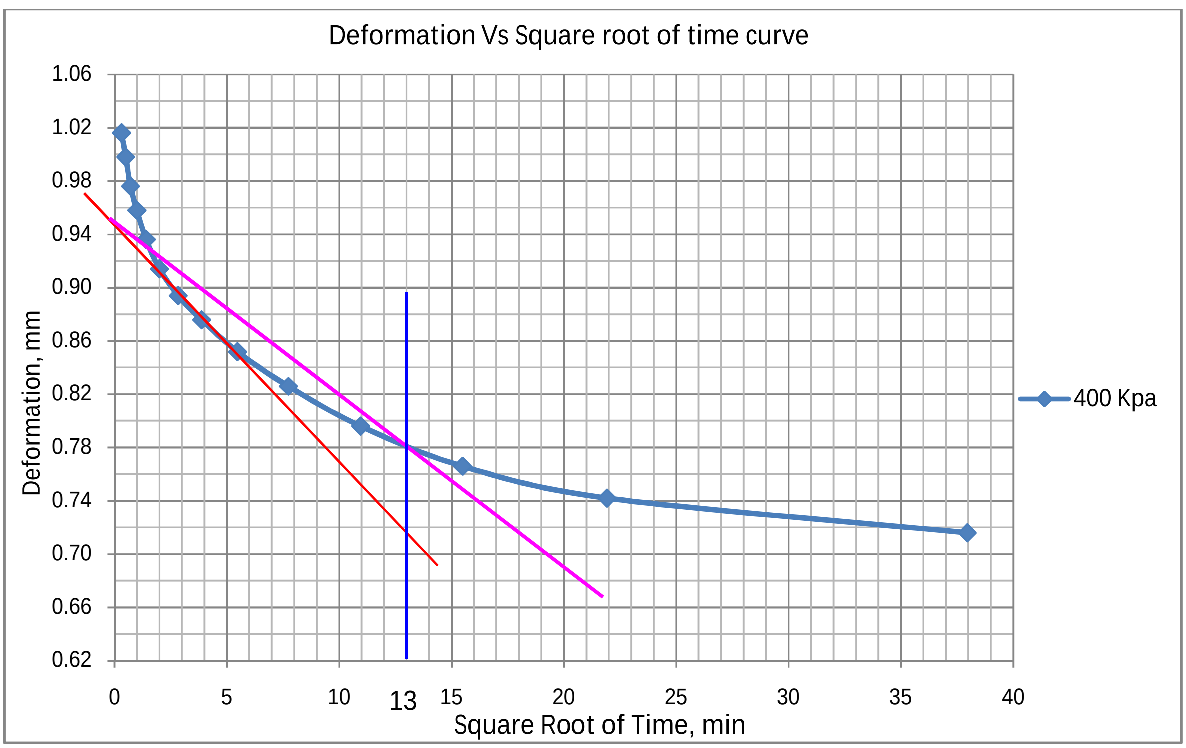

Figure C -12 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 4 (P=400kPa),

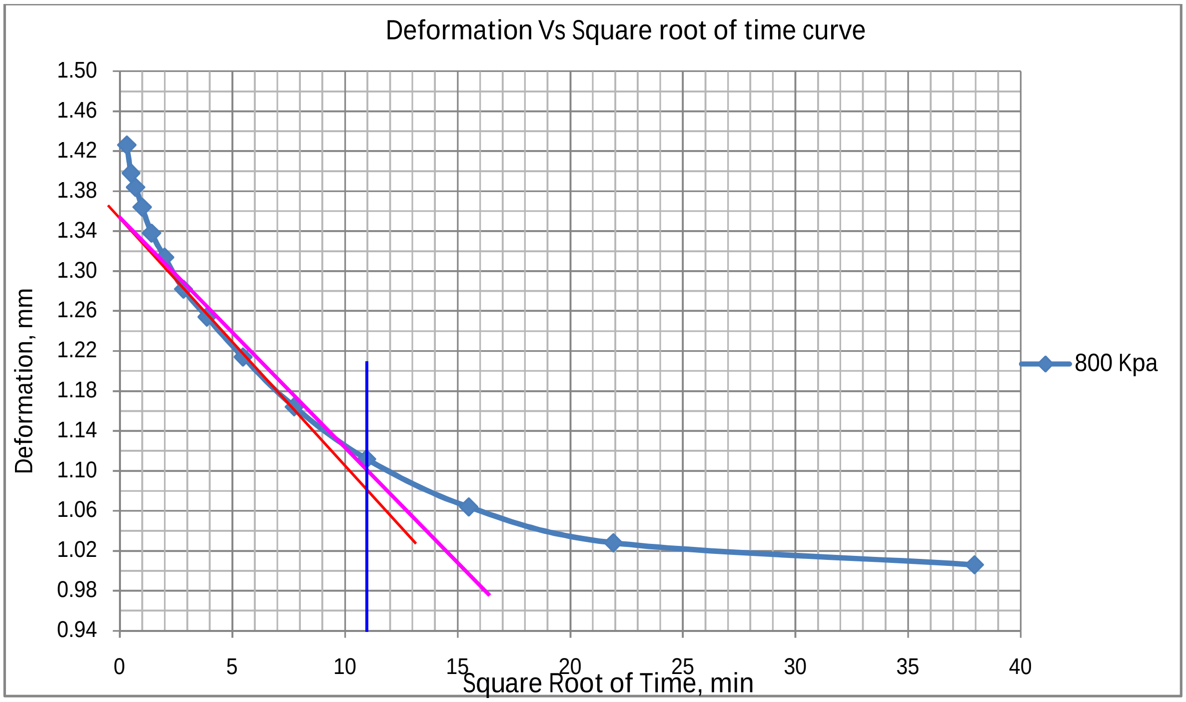

Figure C -13 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 4 (P=800kPa),

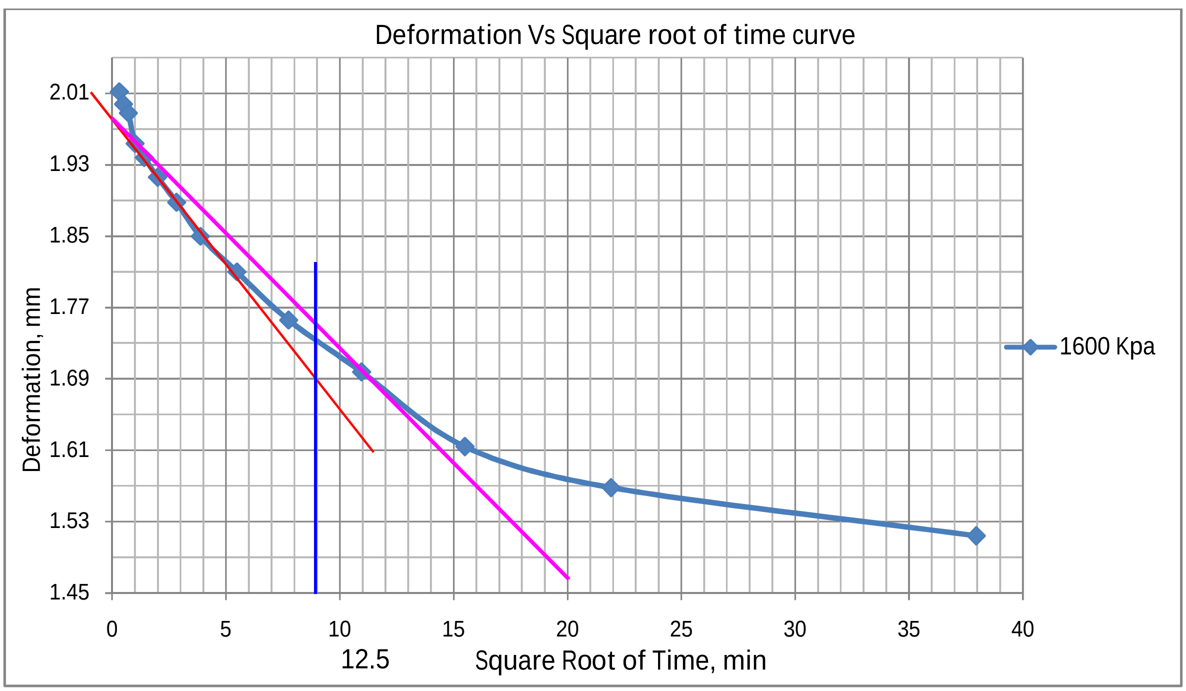

Figure C -14 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 4 (P=1600kPa),

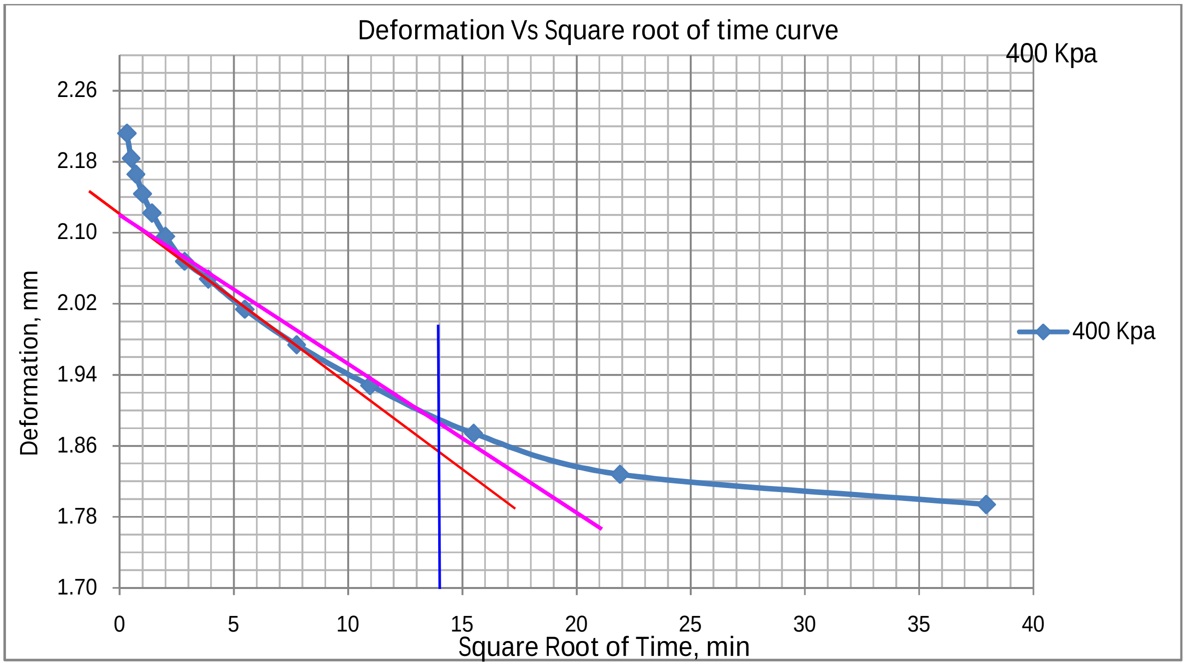

Figure C -15 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 5 (P=400kPa),

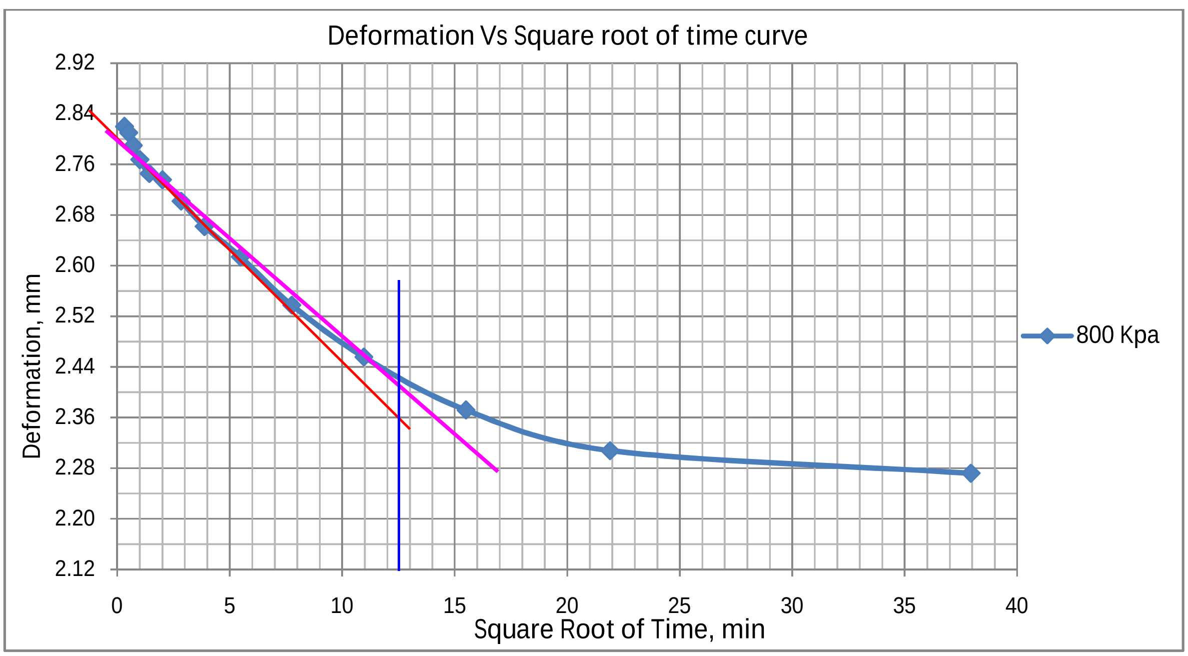

Figure C -16 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 5 (P=800kPa),

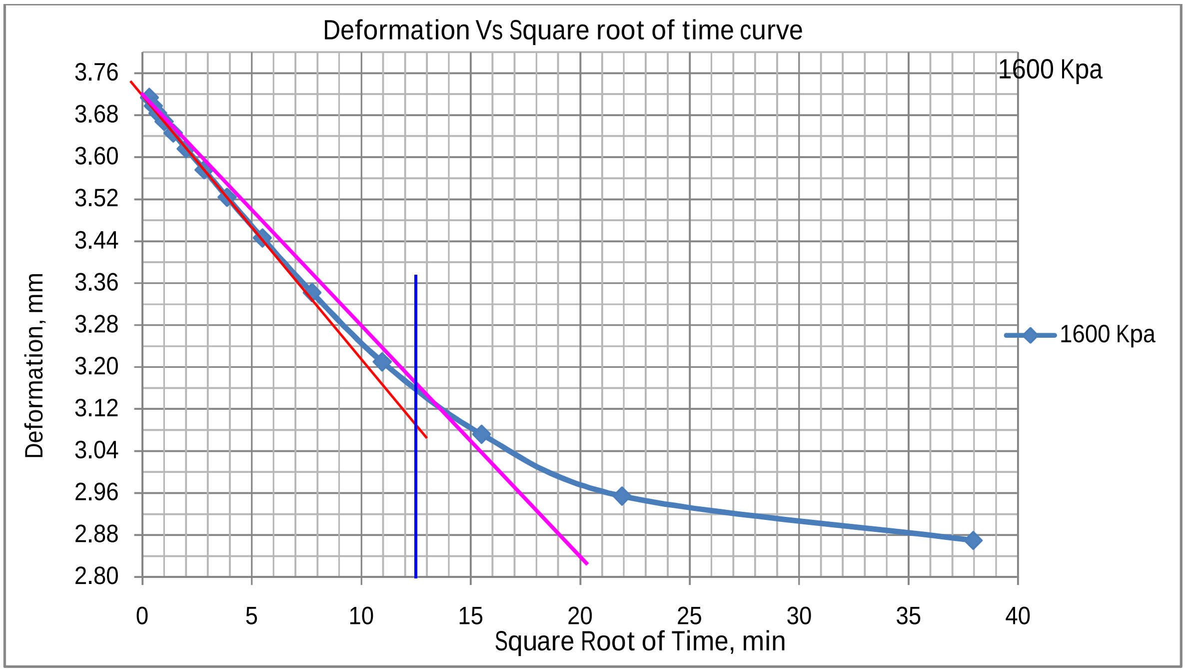

Figure C -17 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 5 (P=1600kPa),

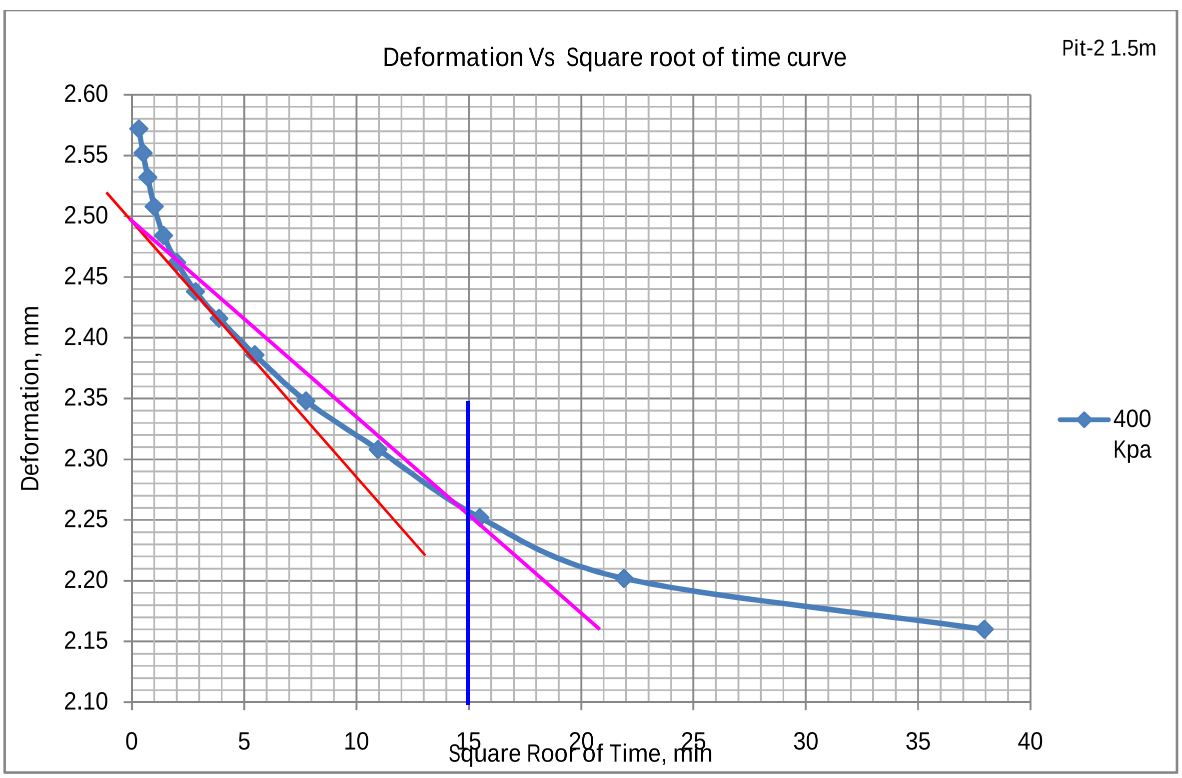

Figure C-18 Typical plots of Deformation versus Square root of time for undisturbed expansive soil sample of Galan Town, Sample-3 (P=400Presented)

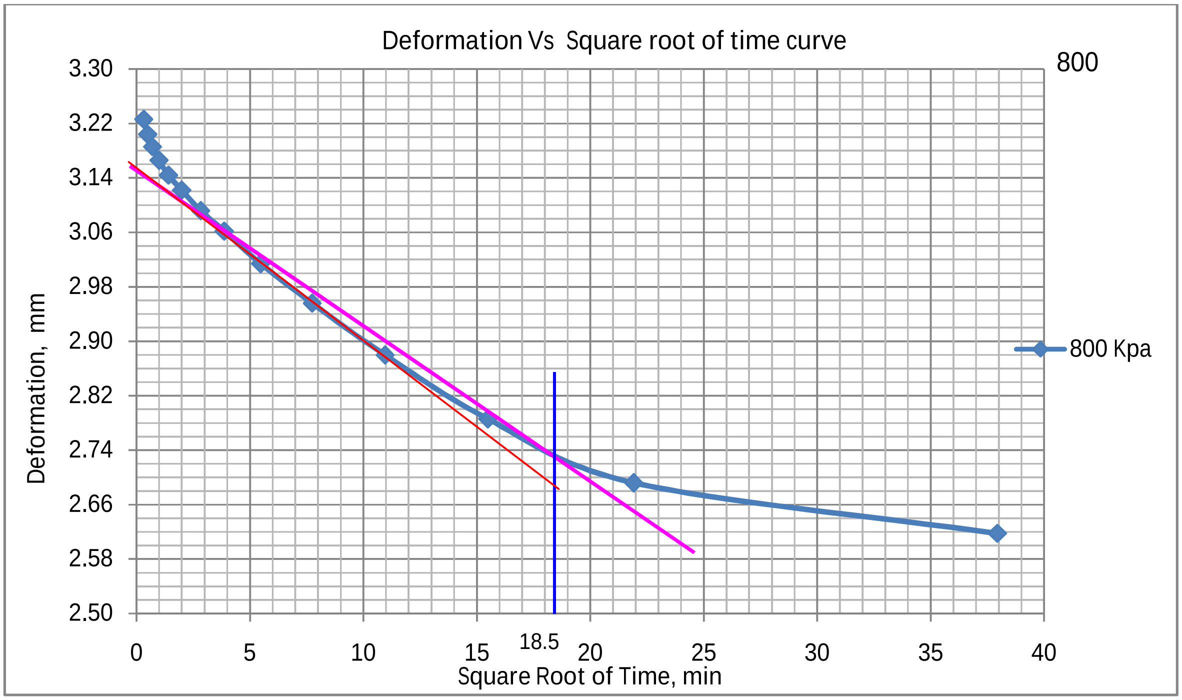

Figure C-19 Typical plots of Deformation versus Square root of time for undisturbed expansive soil sample of Galan Town, Sample-3 (P=800Presented)

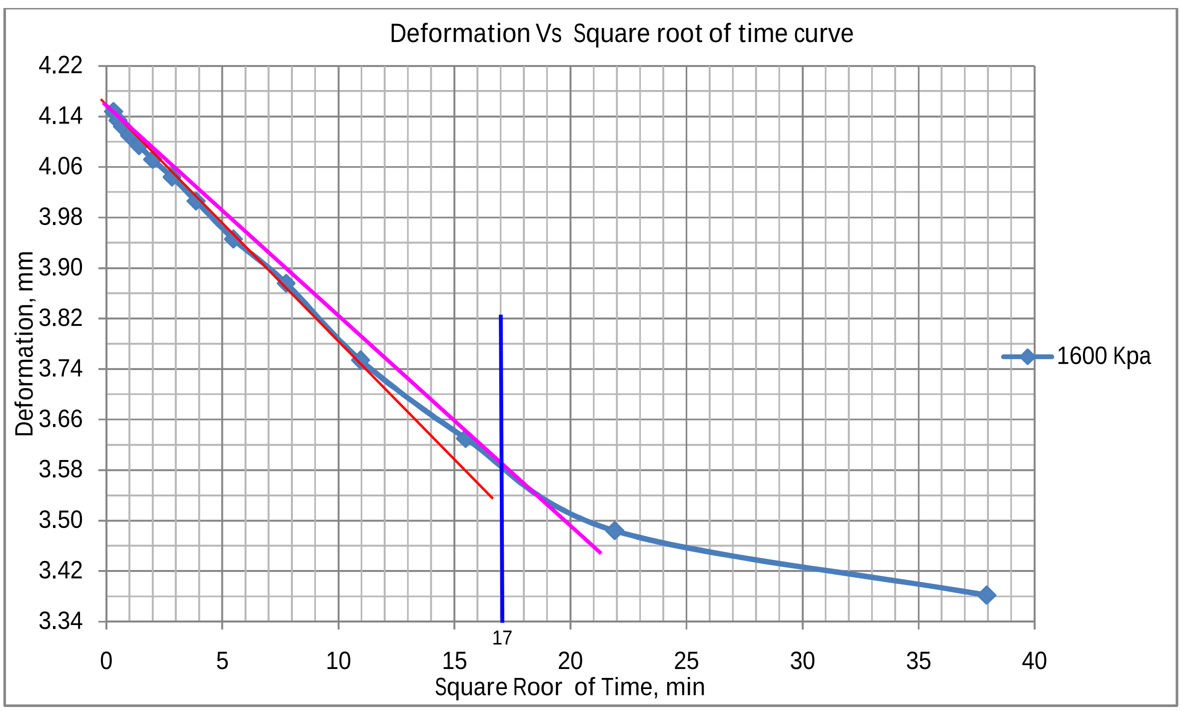

Figure C-20 Typical plots of Deformation versus Square root of time for undisturbed expansive soil sample of Galan Town, Sample-3 (P=1600Presented)

Appendix D

Laboratory test results of remolded expansive soil samples prepared from Pit 6 @ 1.5m depth, Sample 6-1, 6-2, 6-3

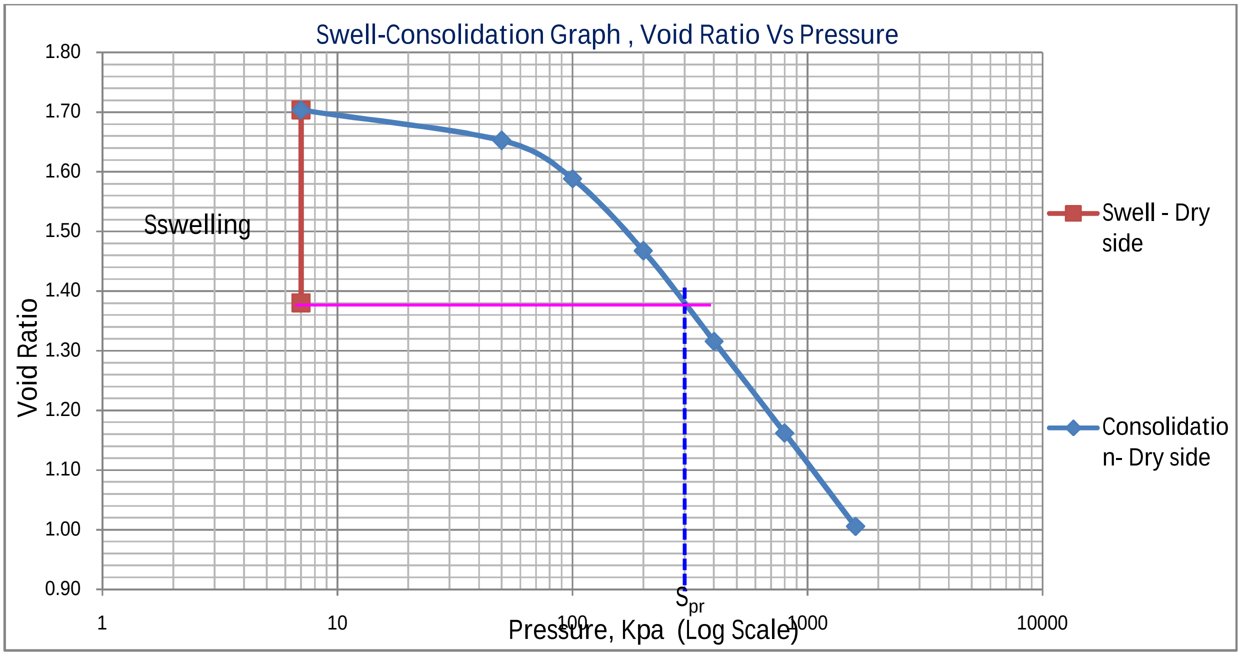

Figure D-1 Plot of e-log P of remolded samples at Dry side of the compaction curve with initial condition (ρ_d = 1.13, w=32%) sample 6-1,

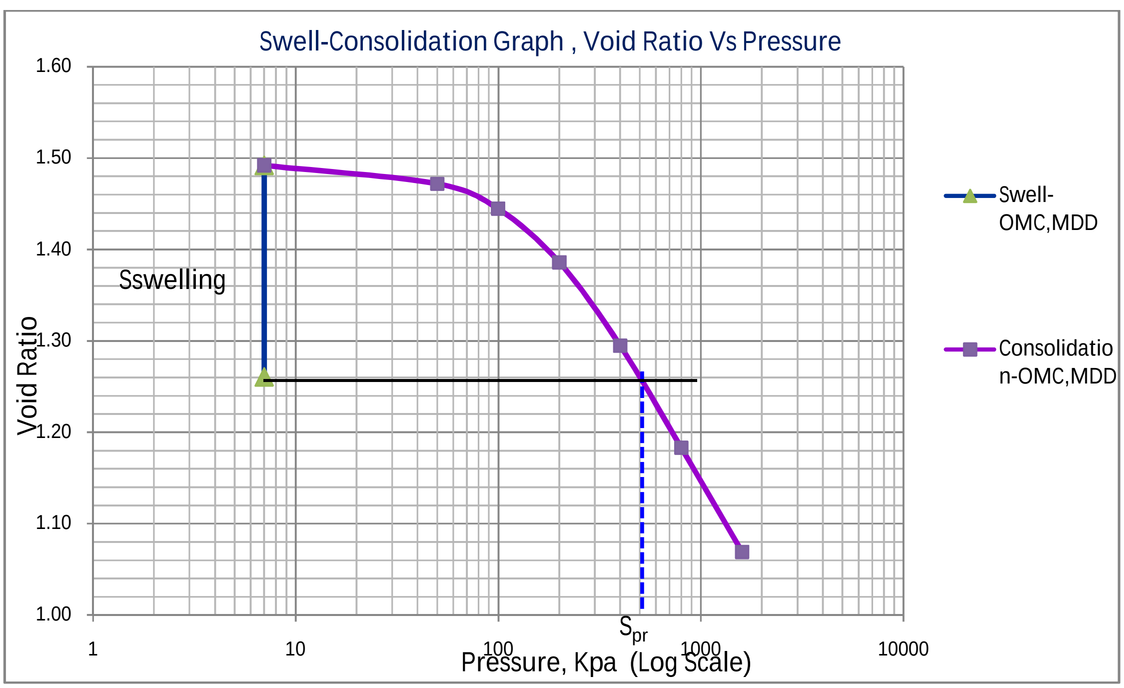

Figure D -2 Plot of e-log P of remolded sample at optimum moisture content & maximum dry density (OMC-MDD) with initial condition (ρ_d = 1.19 and w=44%), sample 6-2

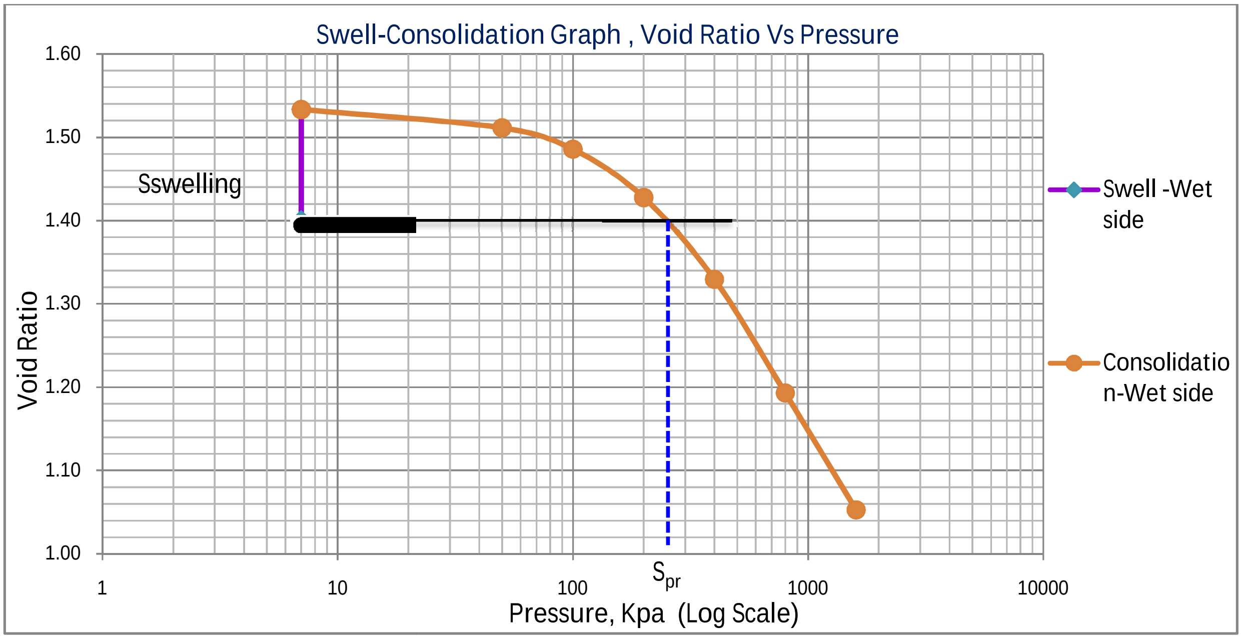

Figure D -3 Plot of e-log P of remolded sample at wet side of the compaction curve with initial condition (ρ_d = 1.12, w=50%), sample 6-3.

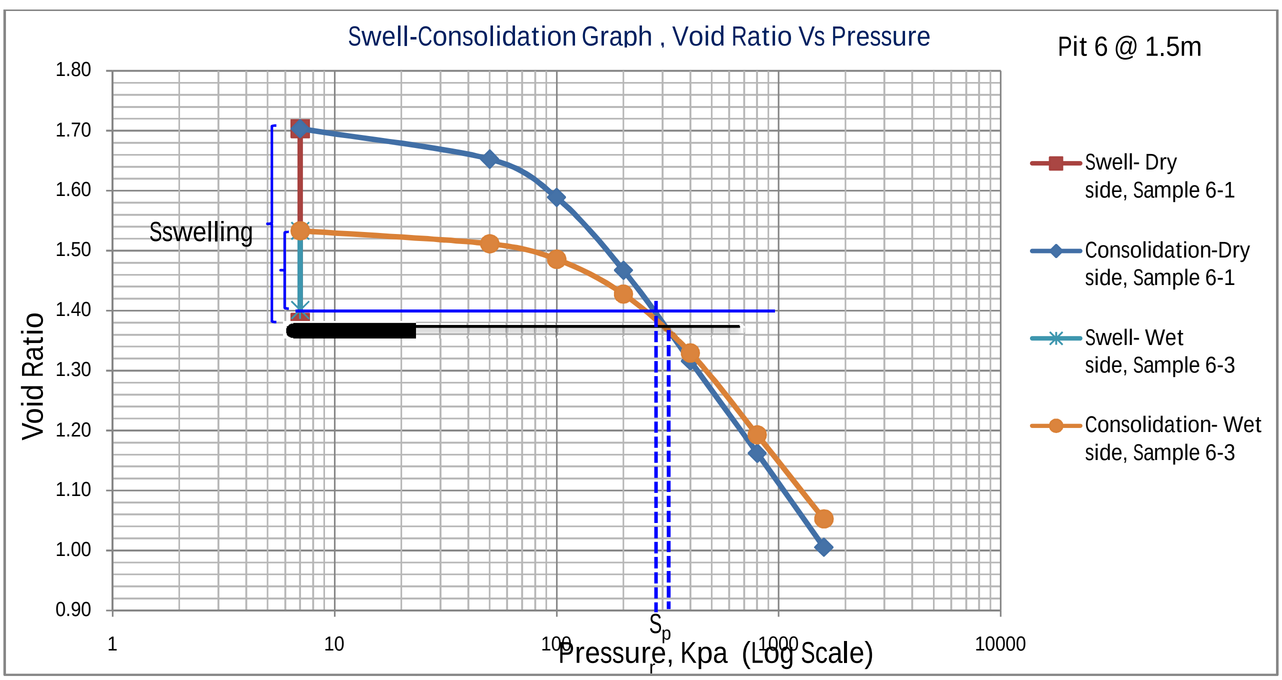

Figure D -4 Plot of e-log P of remolded sample at dry side of the compaction curve, sample 6-1 (ρ_d = 1.13, w=32%) & wet side of the compaction curve, sample 6-3 (ρ_d = 1.12, w=50%).

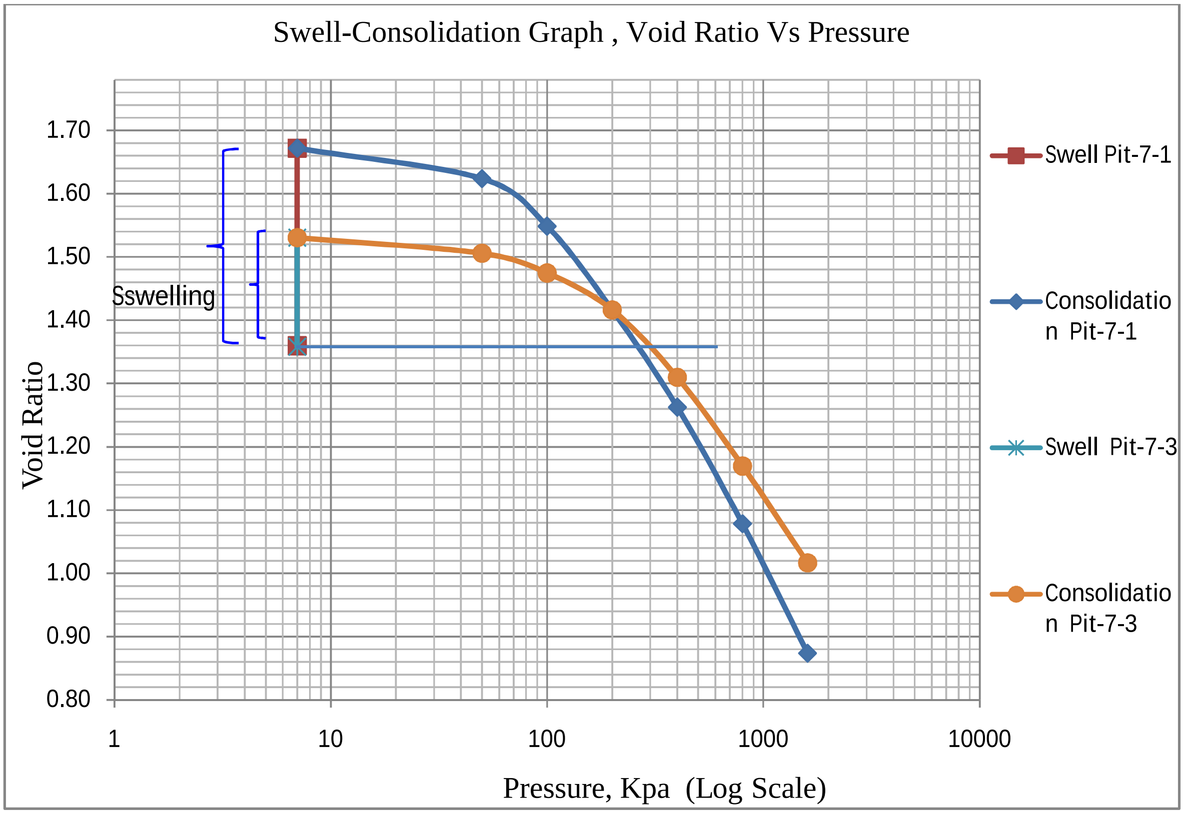

Figure D -5 Plot of e-log P of remolded samples with the Same dry density (ρ_d = 1.13) & different initial moisture content, (dry side, sample 7-1) w=27%, & (wet side sample 7-3) w=47%,

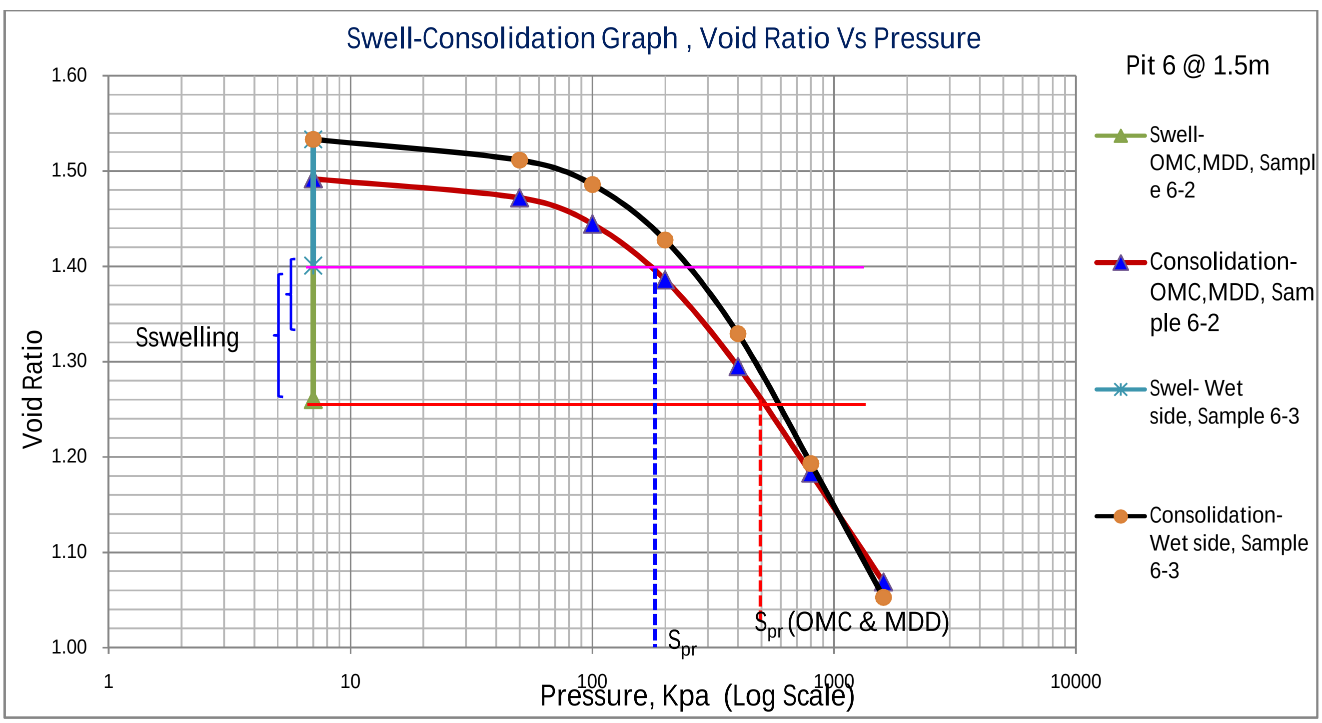

Figure D -6 Plot of e-log P of samples remolded at OMC & MDD, (ρd = 1.19, w = 44%). sample 6-2 and (wet side of the compaction curve) with initial condition of (ρd = 1.12, w=50%), sample 6-3

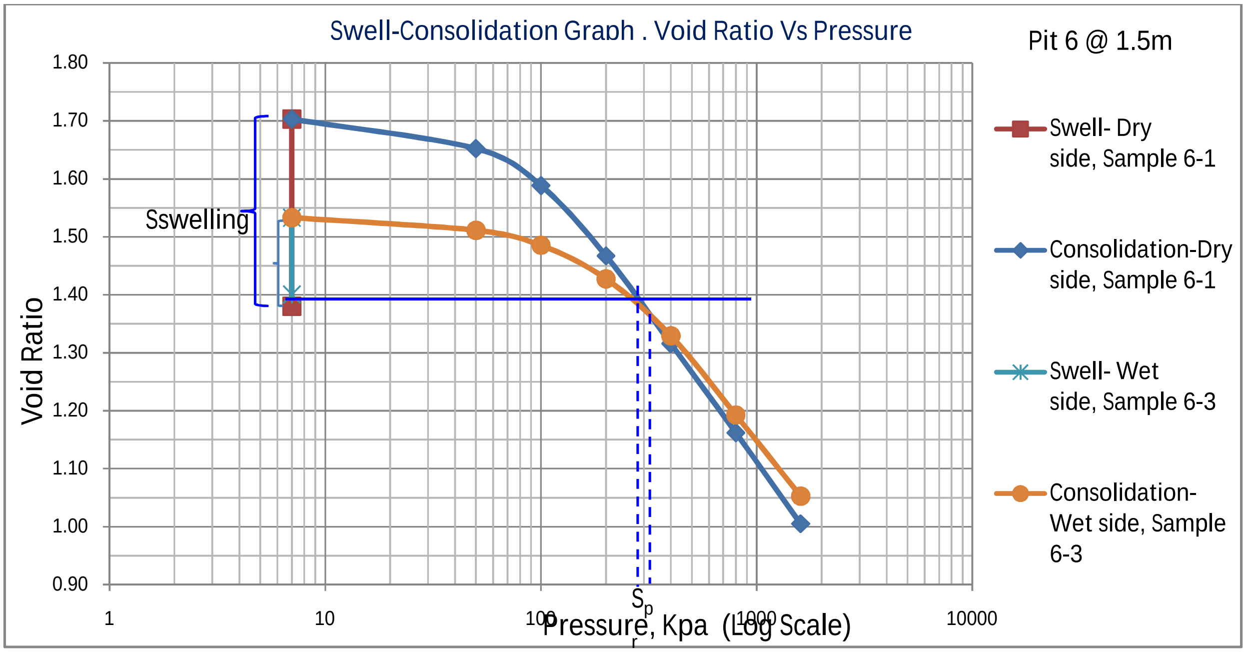

Figure D -7 Plot of e-log P of samples remolded at dry side of the compaction curve, (ρd = 1.13, w=32%), sample 6-1 & at wet side of the compaction curve (ρd = 1.12, w=50%). sample 6-3

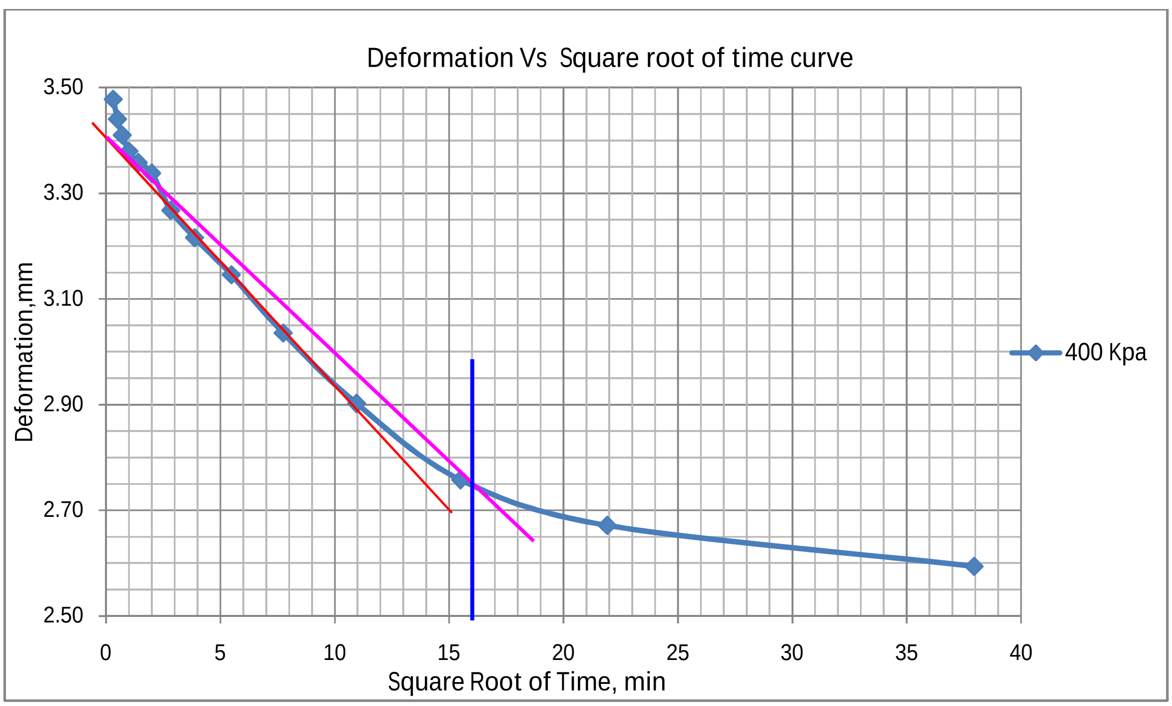

Figure D -8 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-1 (P=400kPa)

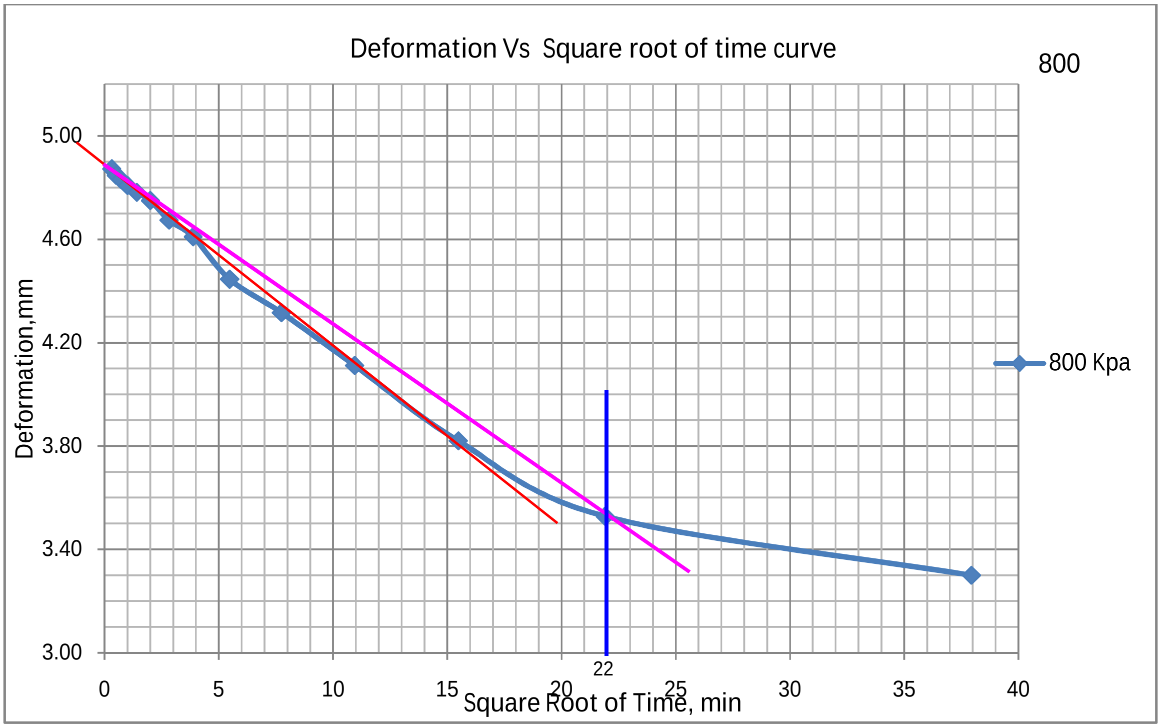

Figure D -9 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-1 (P=800kPa)

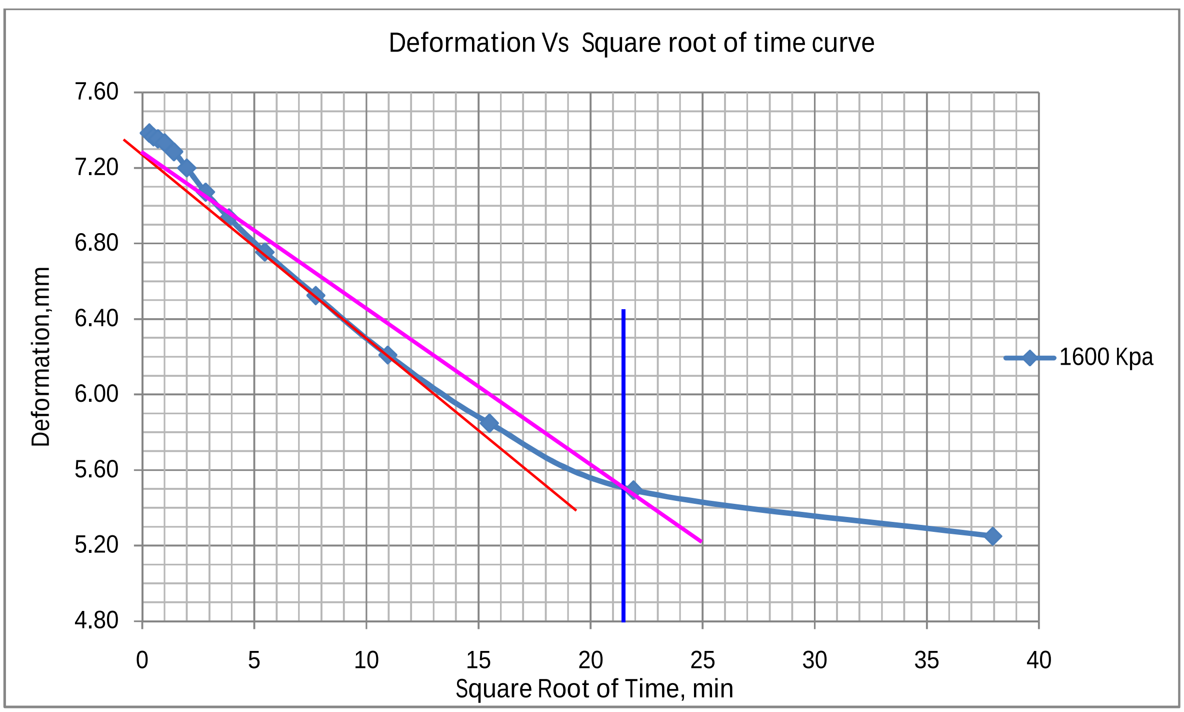

Figure D -10 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-1 (P=1600kPa)

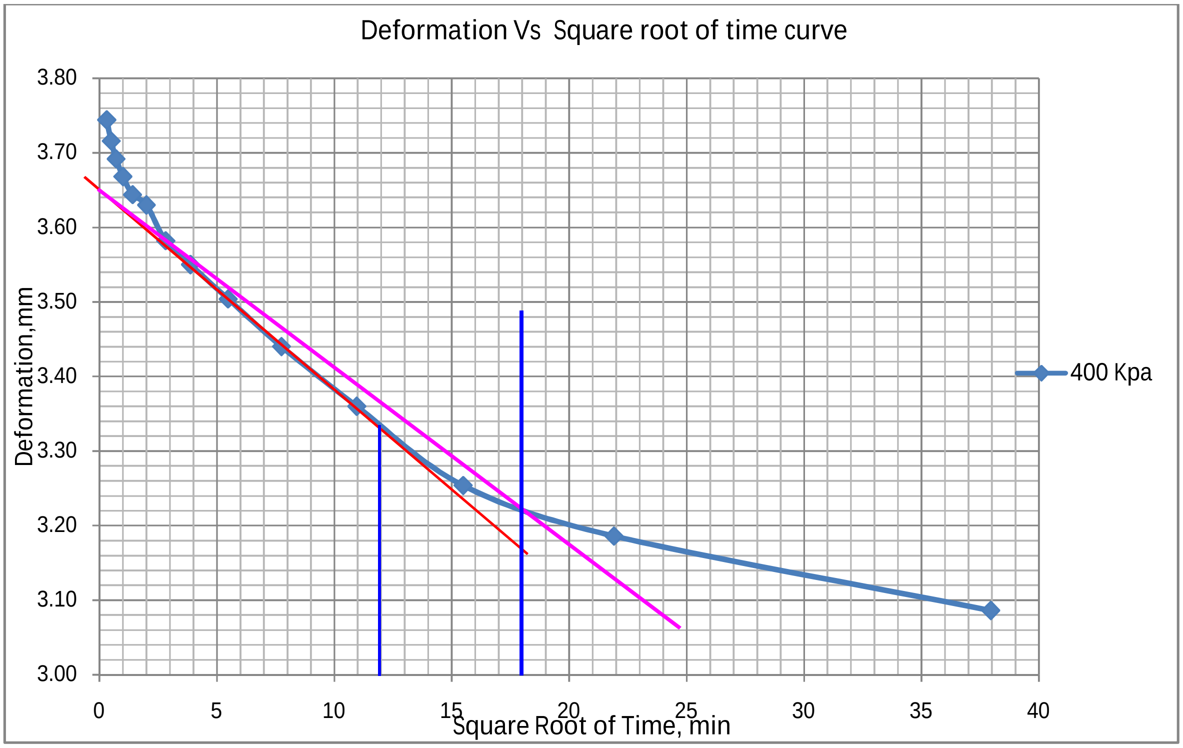

Figure D -11 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-2 (P=400kPa)

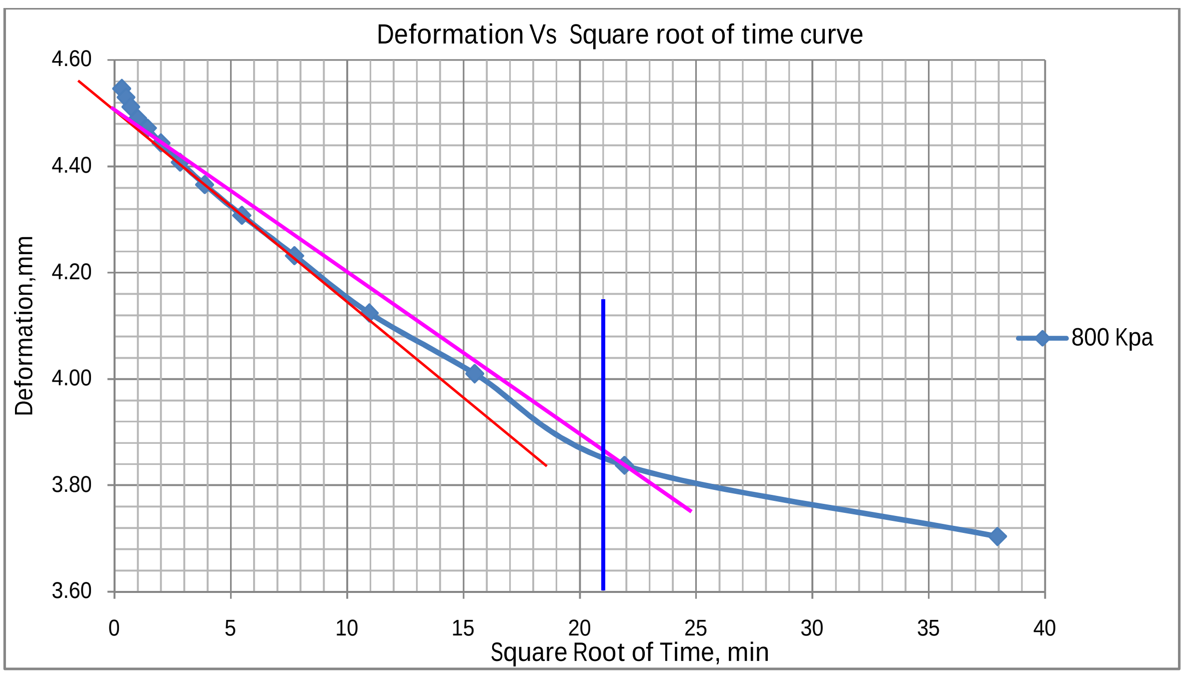

Figure D -12 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-2 (P=800kPa)

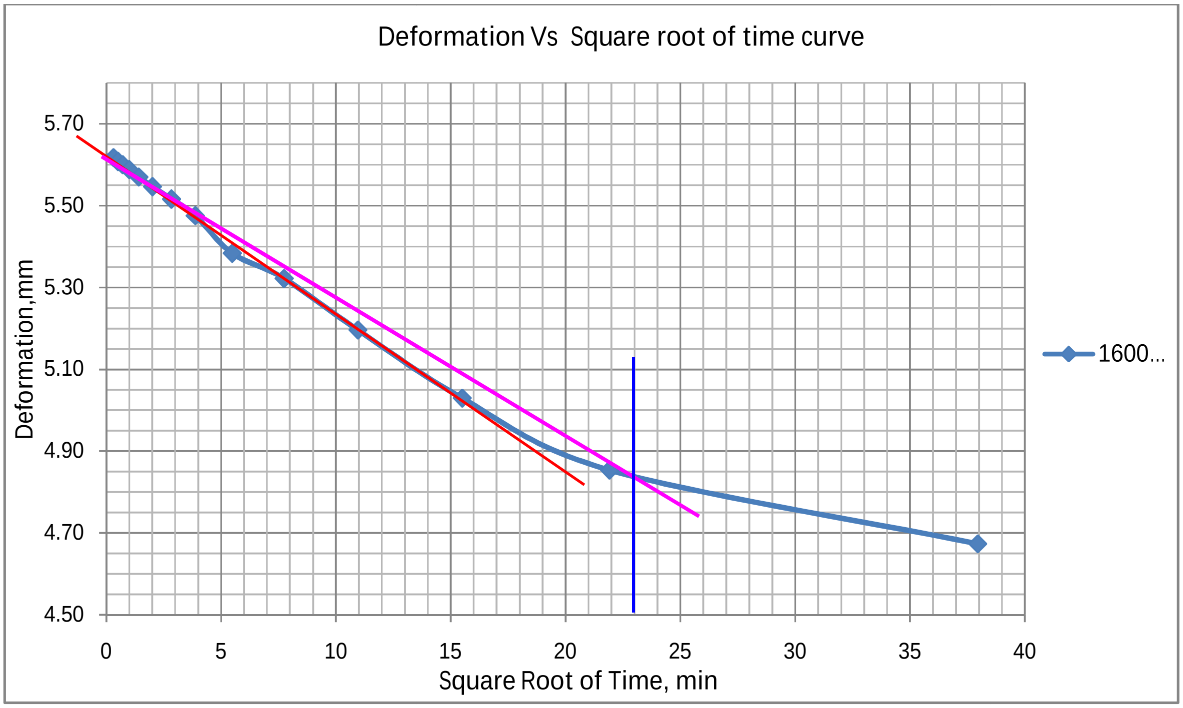

Figure D -13 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-2 (P=1600kPa)

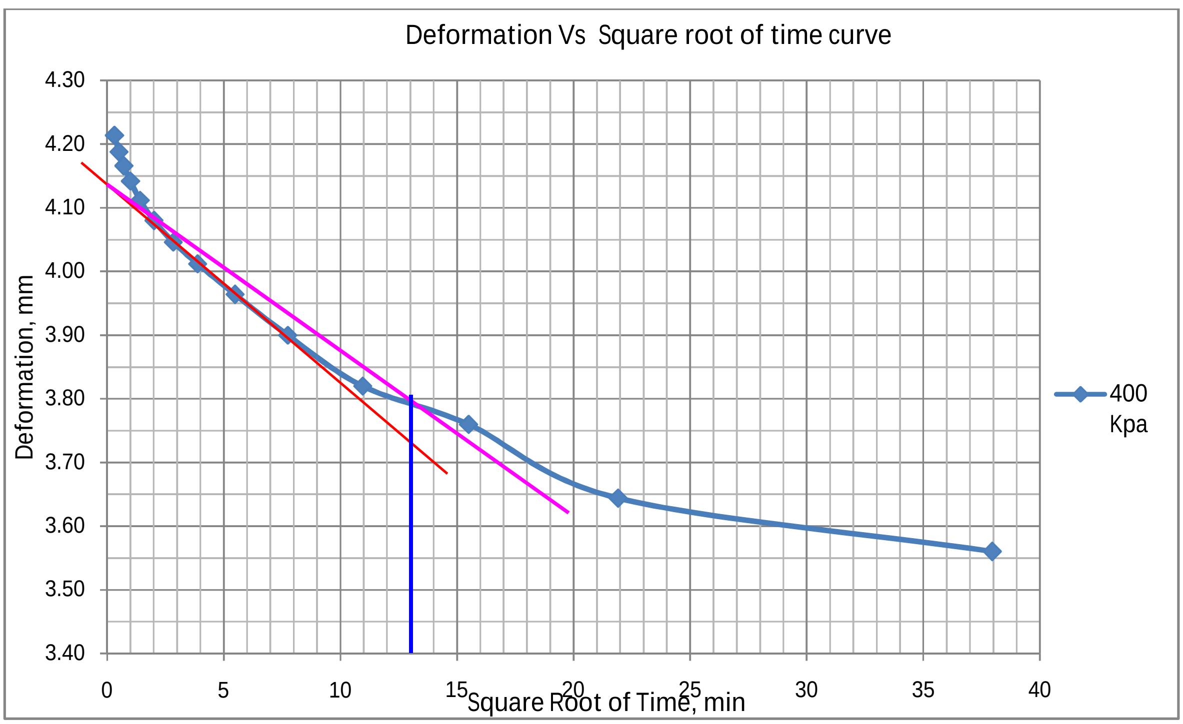

Figure D -14 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-3 (P=400kPa)

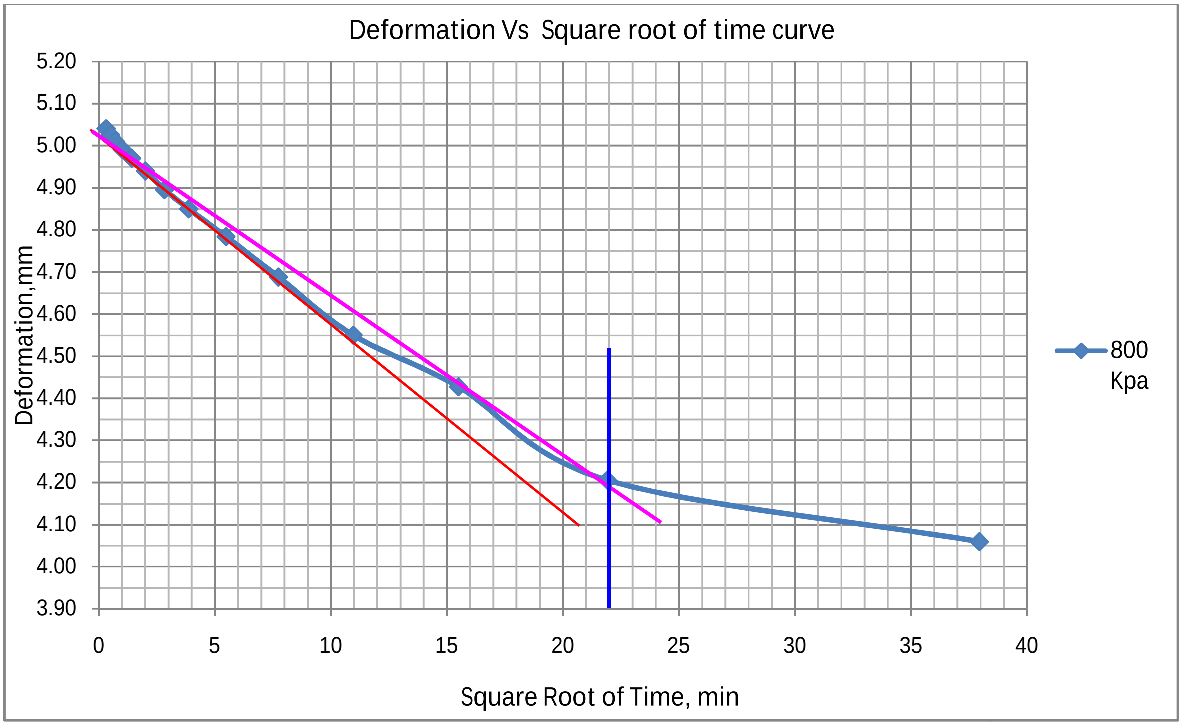

Figure D -15 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-3 (P=800kPa)

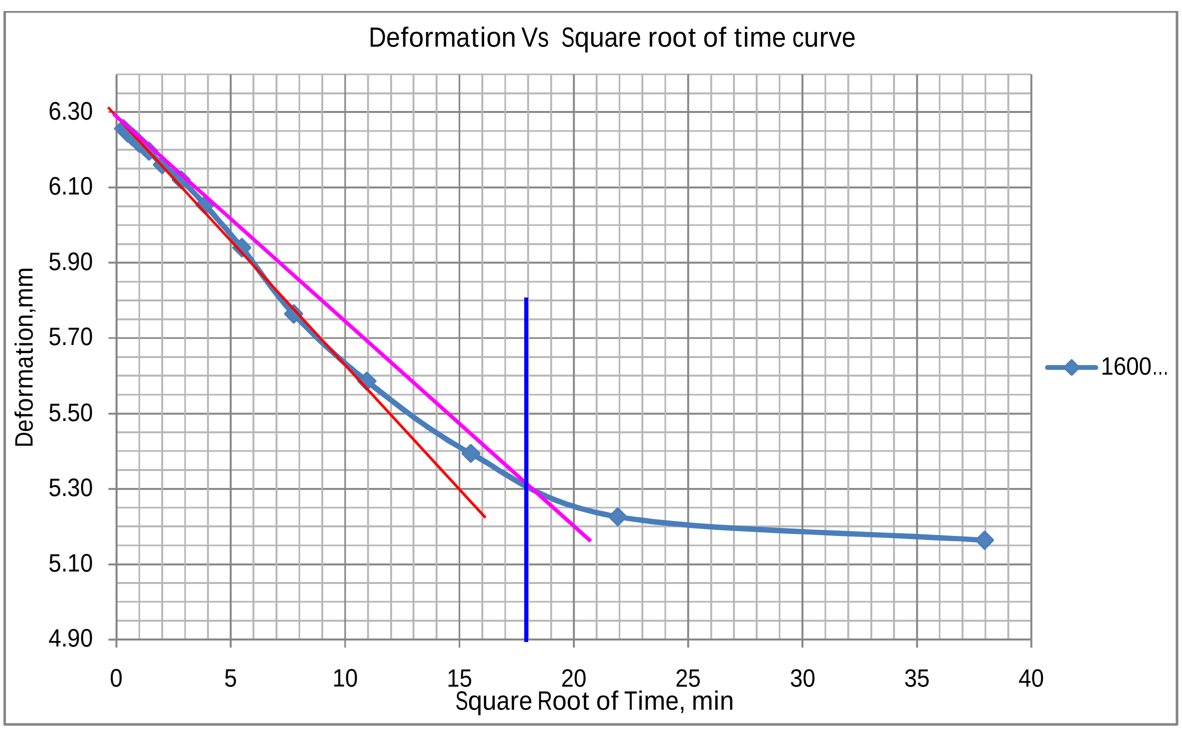

Figure D -16 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 6-3 (P=1600kPa)

Appendix E

Laboratory test results of remolded expansive soil samples prepared from Pit 7 @ 2.9m depth, Sample 7-1, 7-2, & 7-3

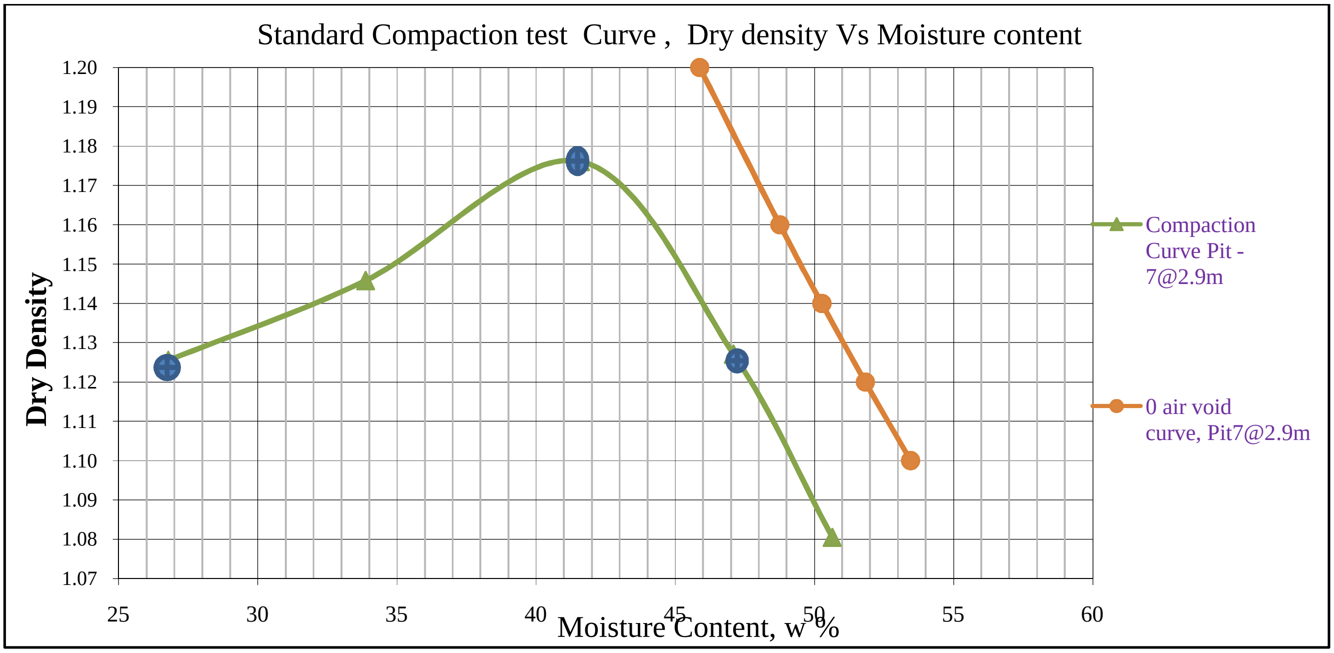

Figure E-1 Compaction curve obtained from Standard Compaction test conducted in the laboratory for remolding purpose, sample-12 (Pit 7 @2.9m depth)

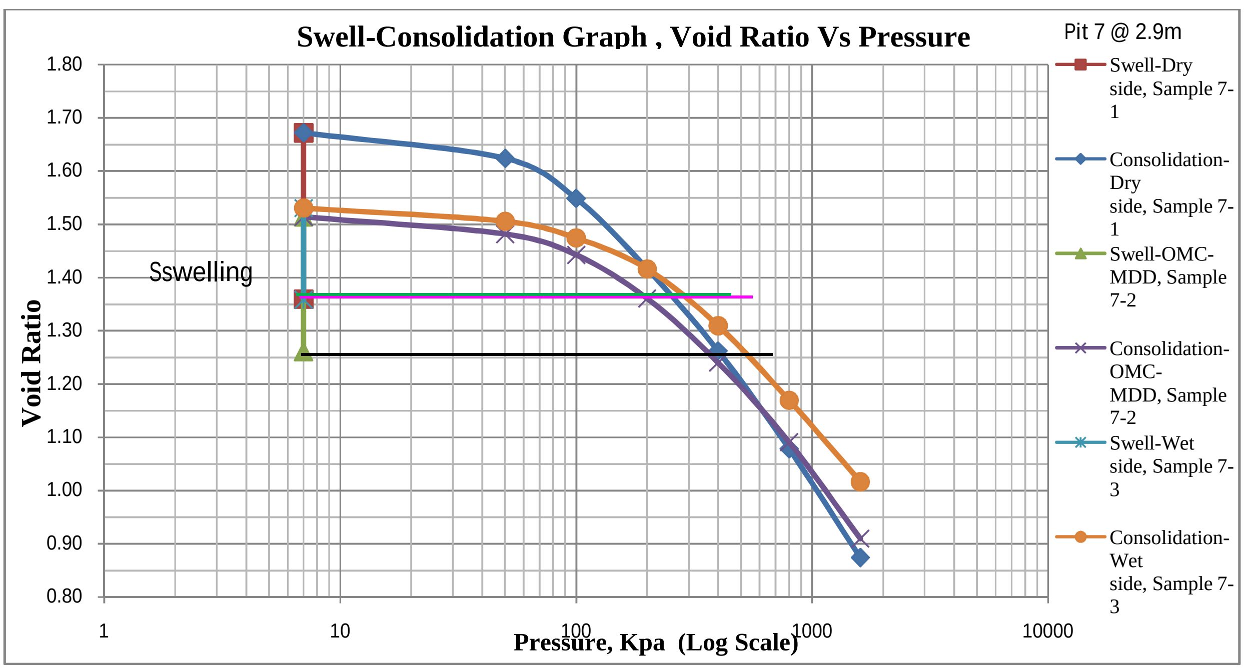

Figure E -2 Plot of e-log P of remolded samples of samples 7-1, 7-2 & 7-2 with initial condition (ρ_d =1.13, w=27%), (ρd =1.18, w=42%) & (ρd =1.13, w=47%) respectively.

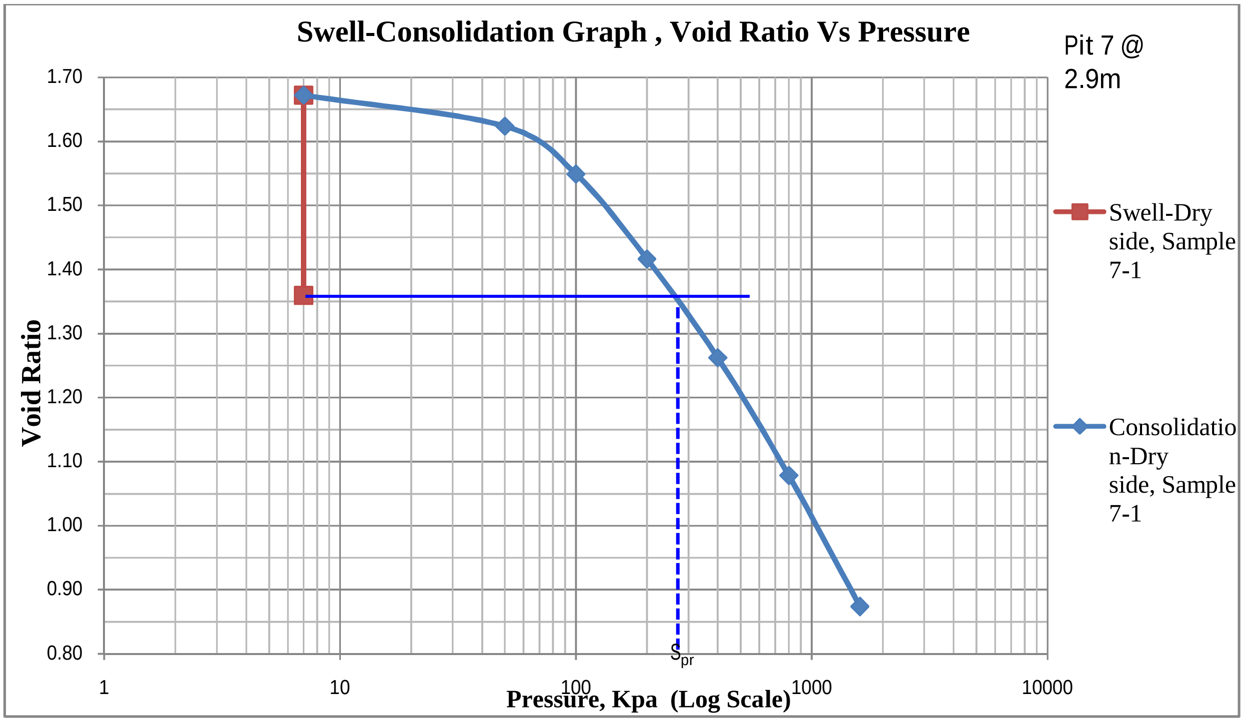

Figure E -3 Plot of e-log P of remolded samples, at dry side of the compaction curve with initial condition (ρd =1.13, w=27%), sample 7-1

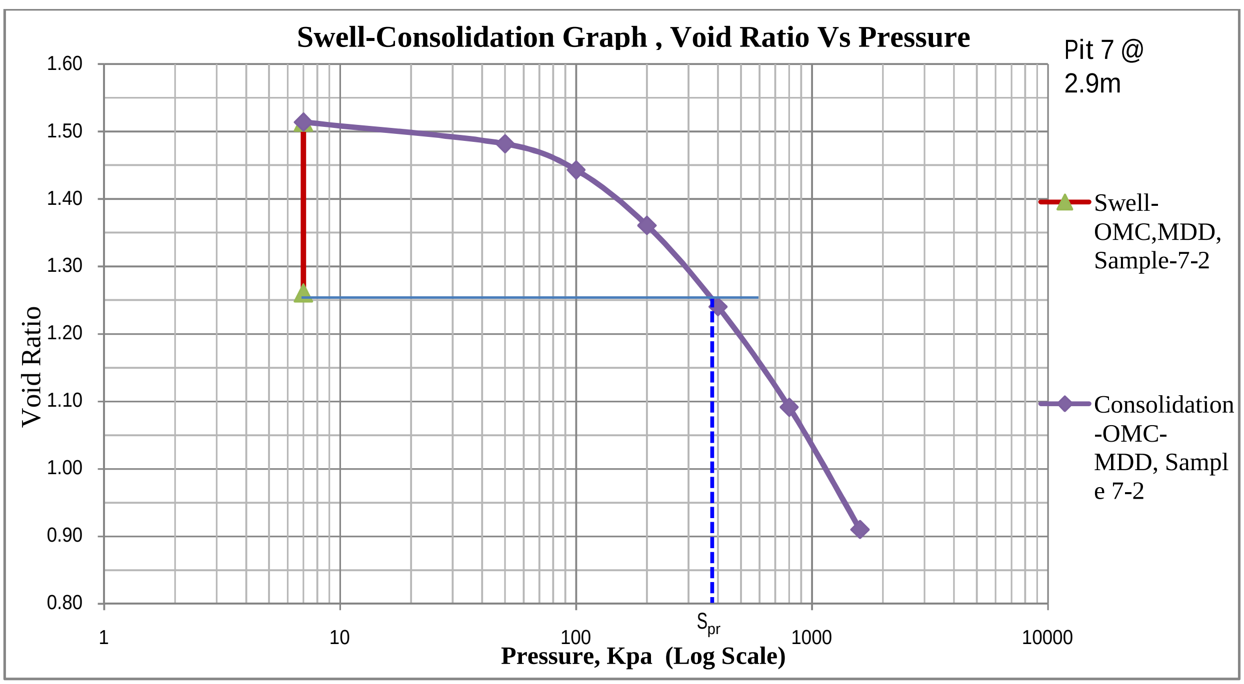

Figure E -4 Plot of e-log P of remolded samples, at OMC & MDD with initial condition (ρ_d =1.18, w=42%), sample 7-2

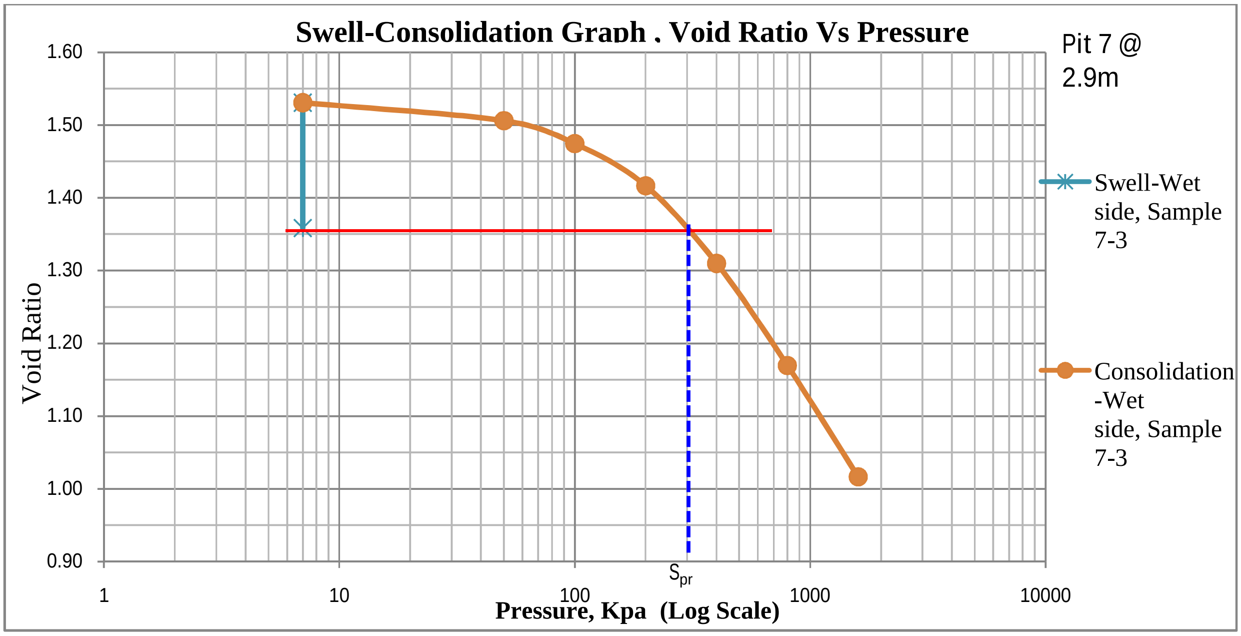

Figure E -5 Plot of e-log P of remolded samples,, at wet side of the compaction curve with initial condition (ρ_d =1.13, w=47%), sample 7-3

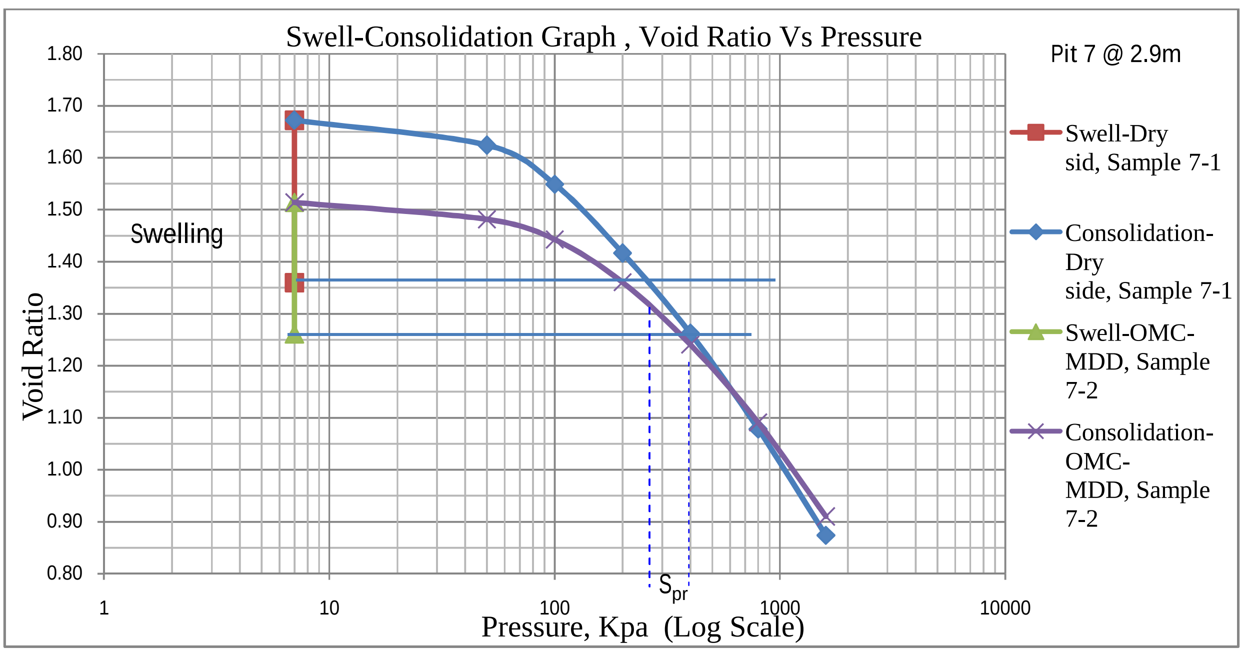

Figure E -6 Plot of e-log P of remolded samples, at dry side of the compaction curve & OMC-MDD, with initial condition (ρ_d = 1.13, w=27%), sample 7-1 & (ρ_d = 1.18, w=42%), sample 7-2 respectively.

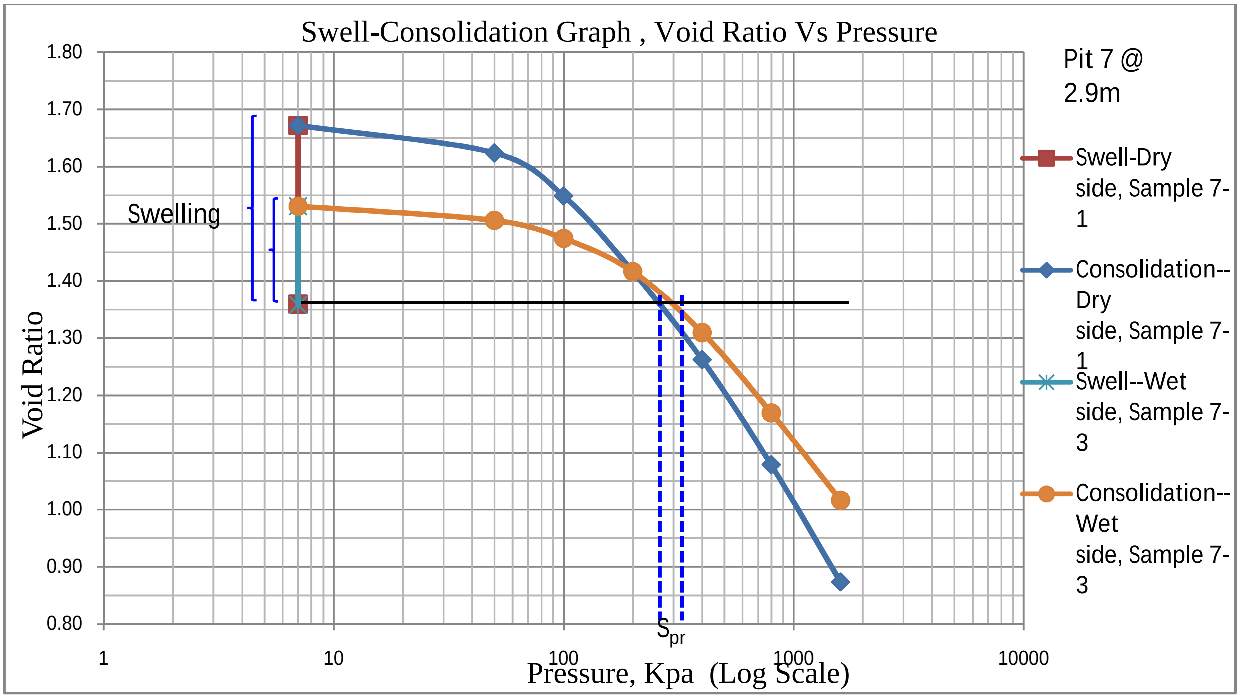

Figure E -7 Plot of e-log P of remolded samples, with the same dry density (ρ_d = 1.13) & different moisture content at initial condition of w=27%, sample 7-1 & w=47%, sample 7-3, respectively.

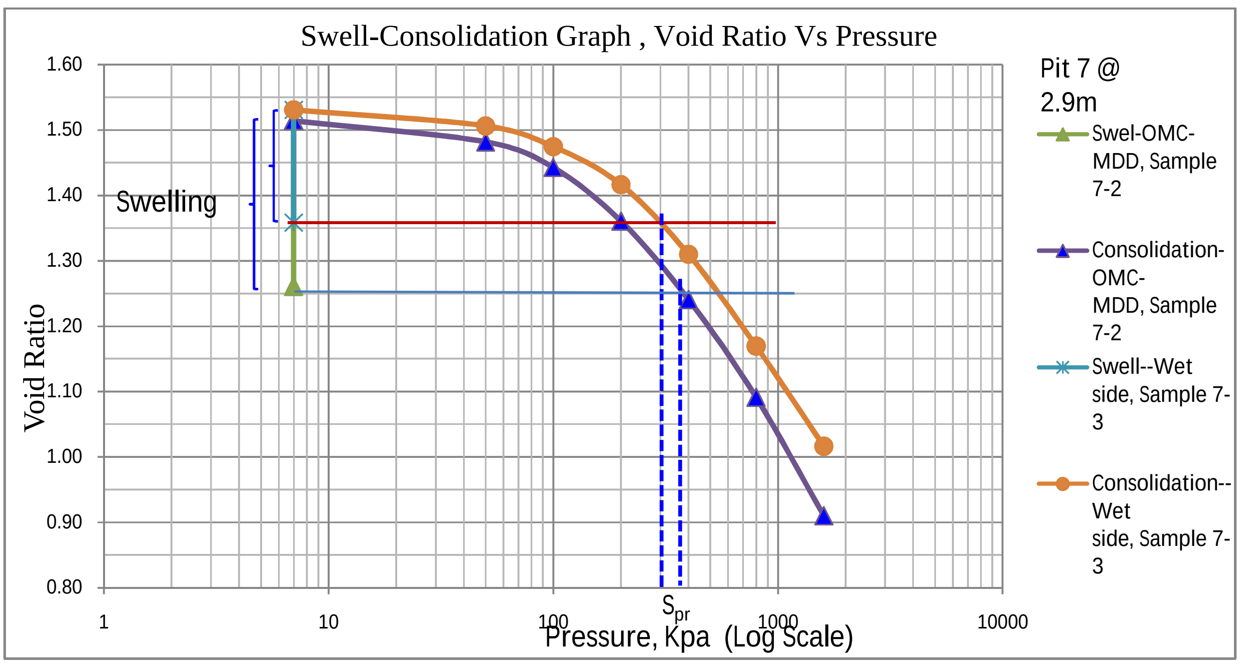

Figure E -8 Plot of e-log P of remolded samples, at OMC-MDD (sample 7-2) & wet side of the compaction curve, (sample 7-3) at initial condition of (ρd =1.18, w=42%) & (ρd =1.13, w=47%) respectively.

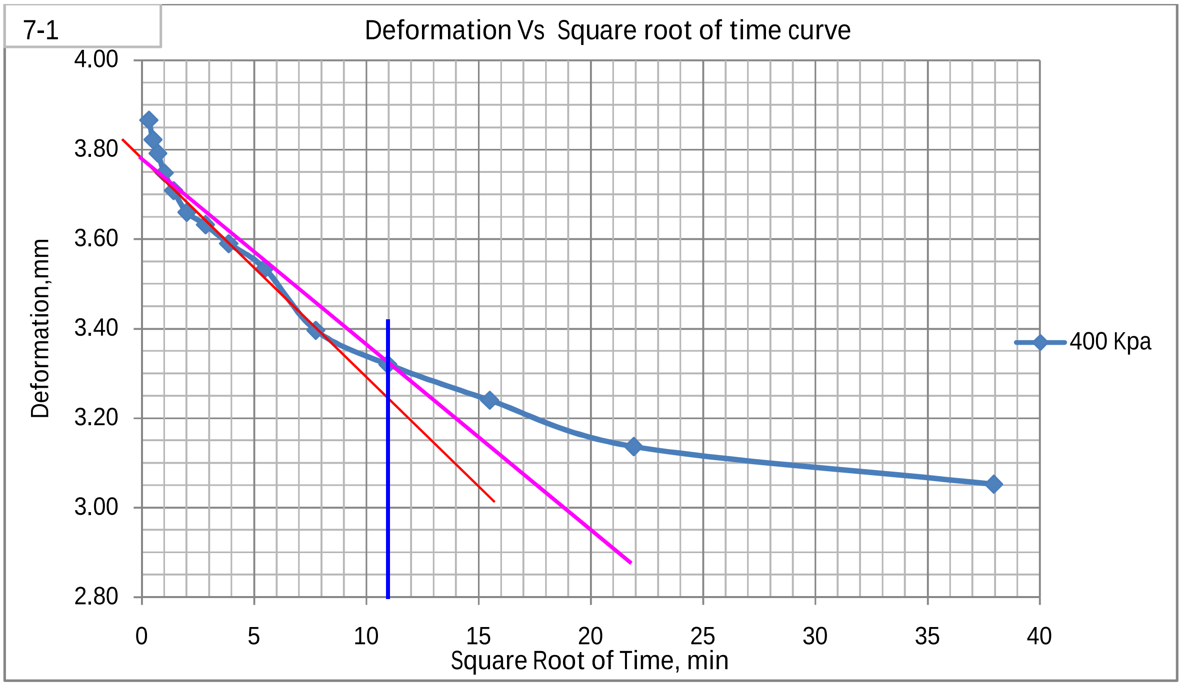

Figure E -9 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-1 (P=400kPa),

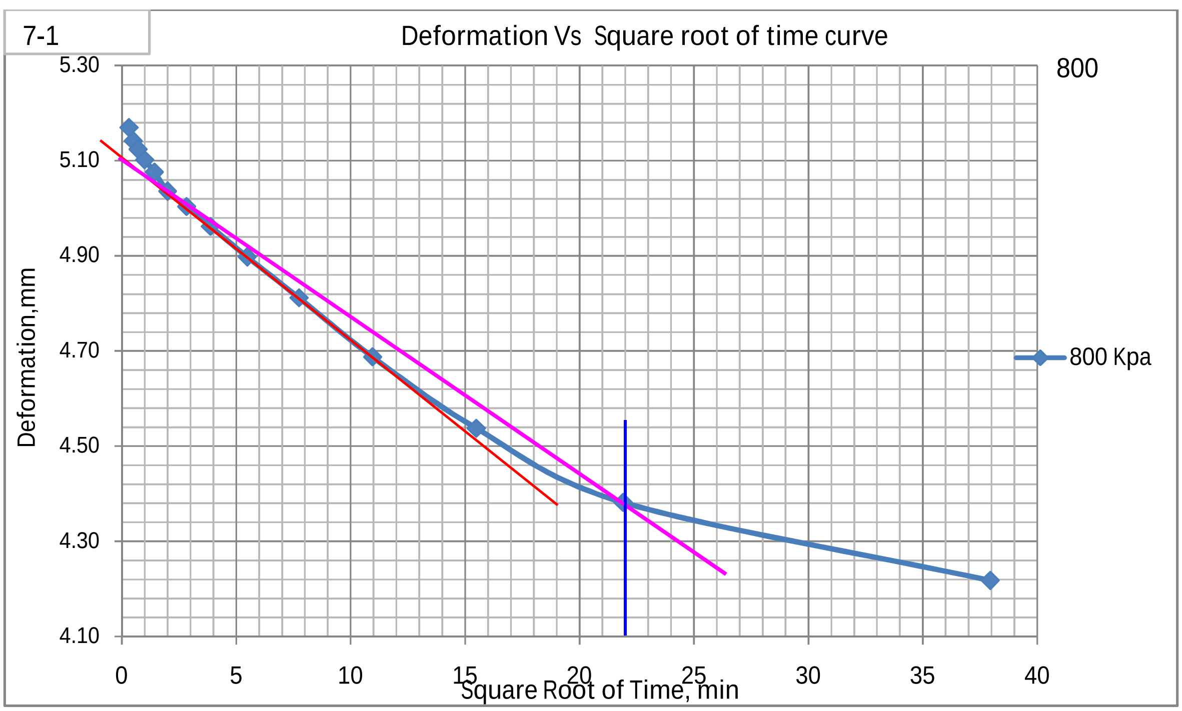

Figure E -10 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-1 (P=800kPa),

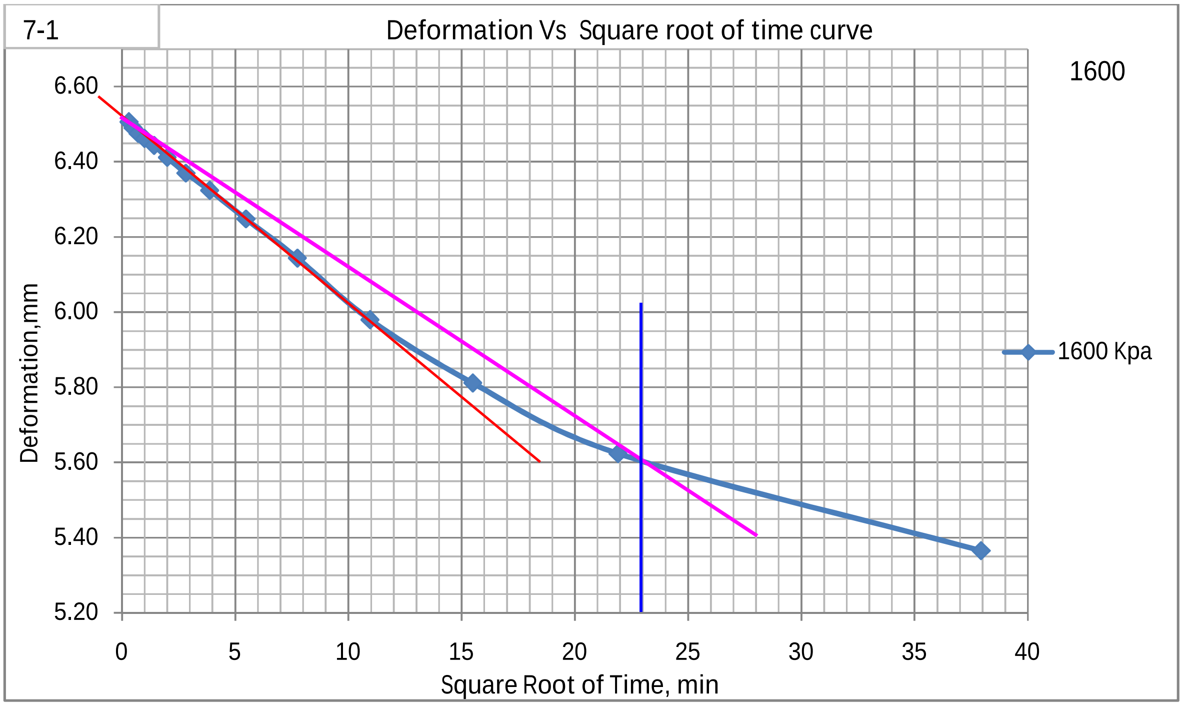

Figure E -11 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-1 (P=1600kPa),

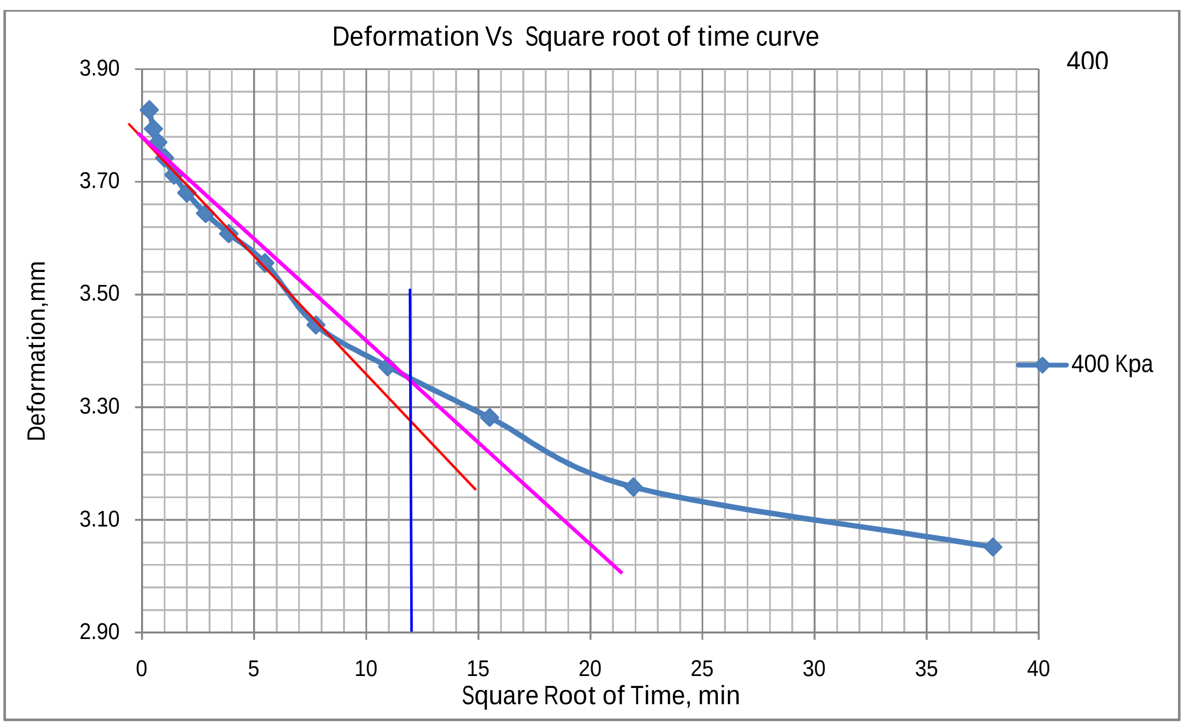

Figure E -12 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-2 (P=400kPa),

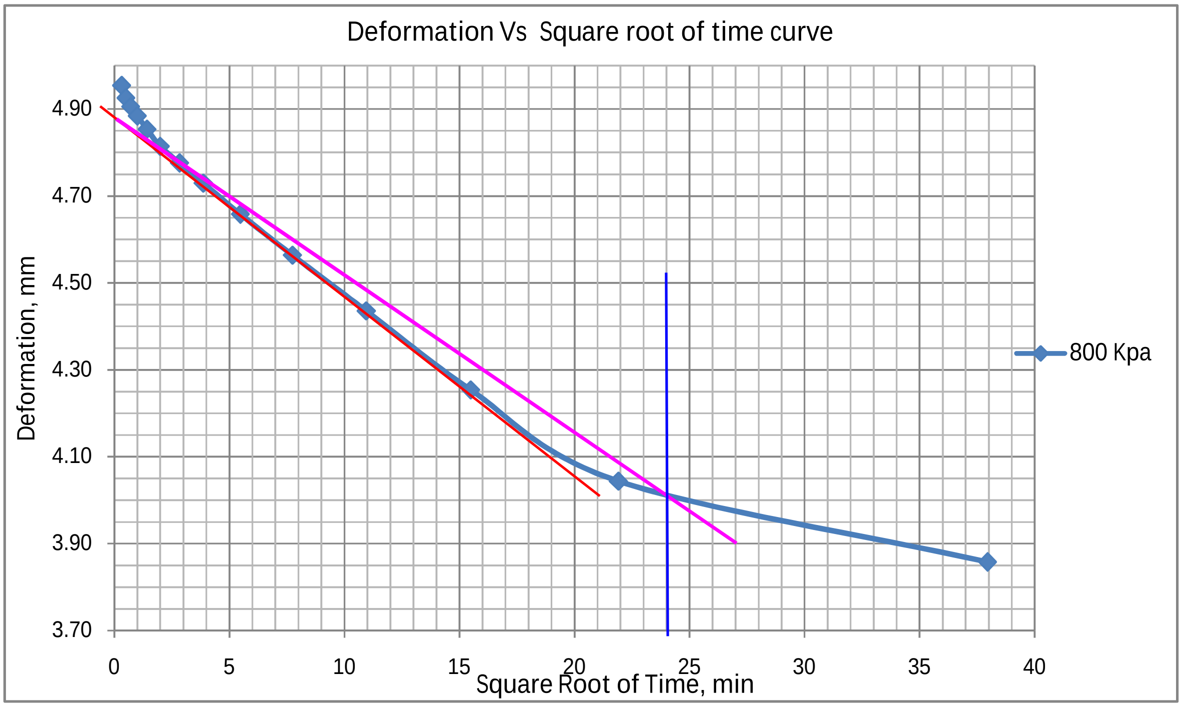

Figure E -13 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-2 (P=800kPa),

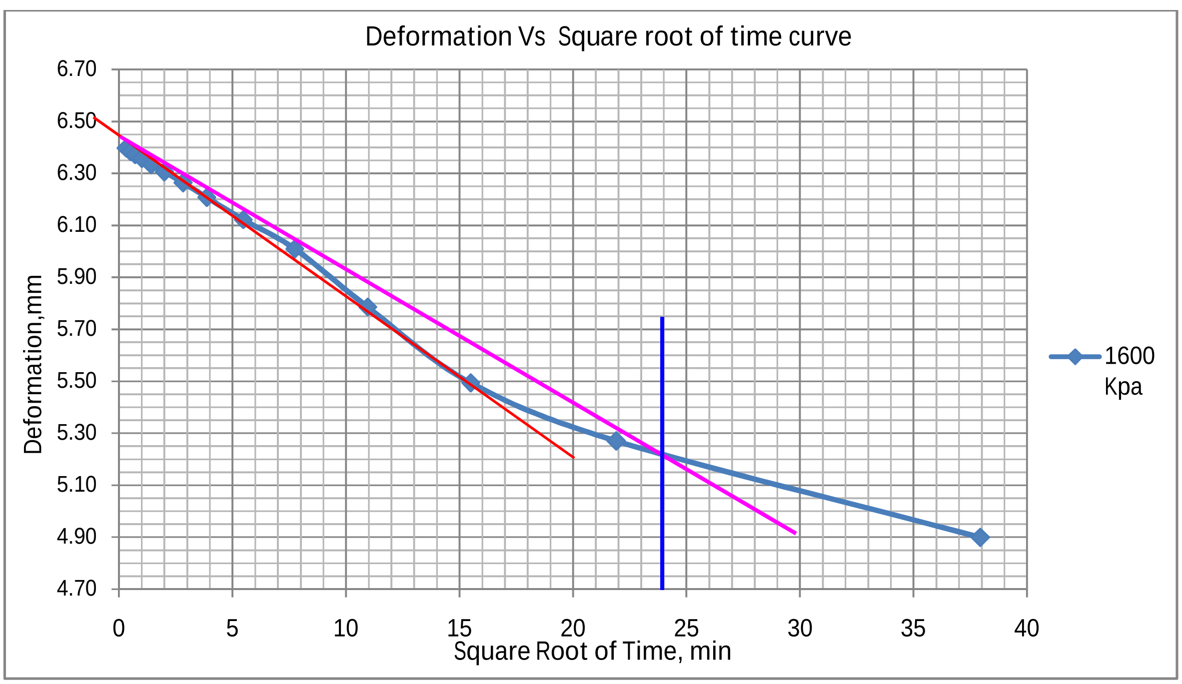

Figure E -14 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-2 (P=1600kPa),

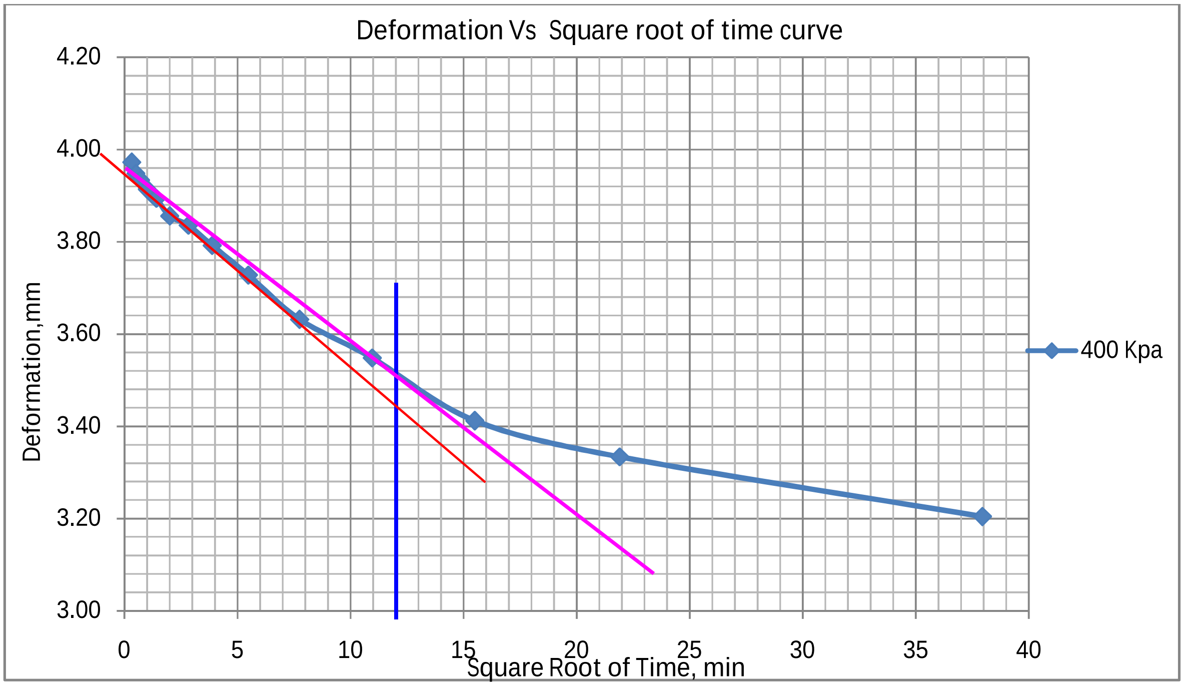

Figure E -15 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-3 (P=400kPa),

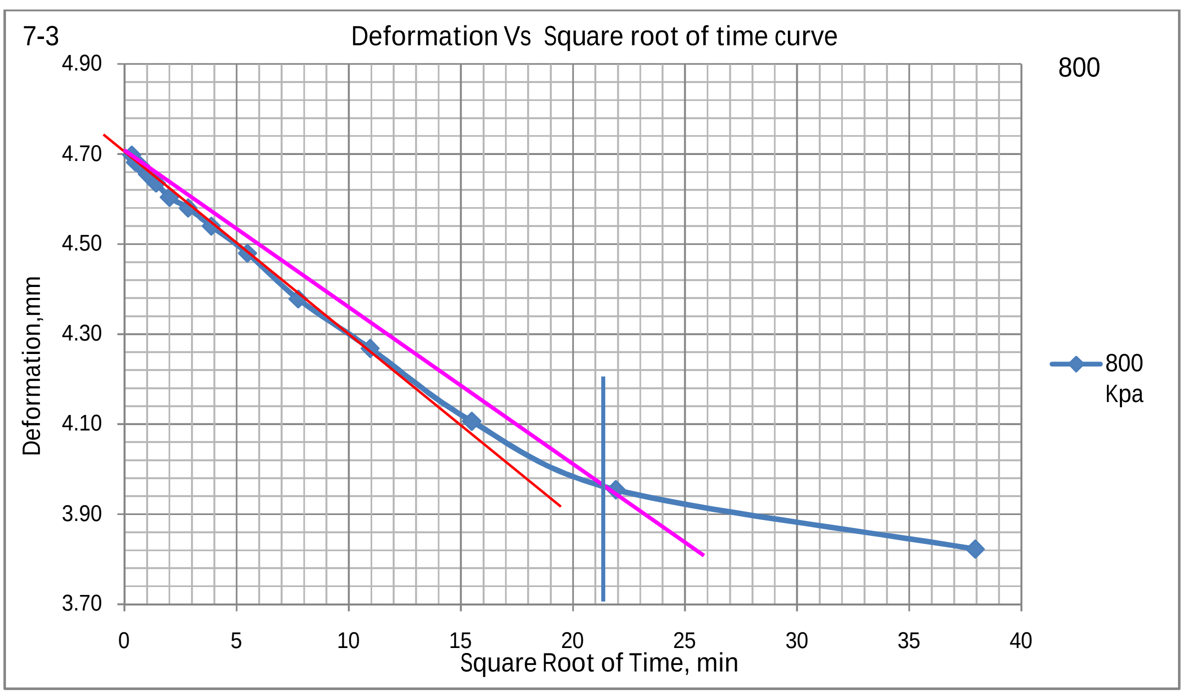

Figure E -16 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-3 (P=800kPa),

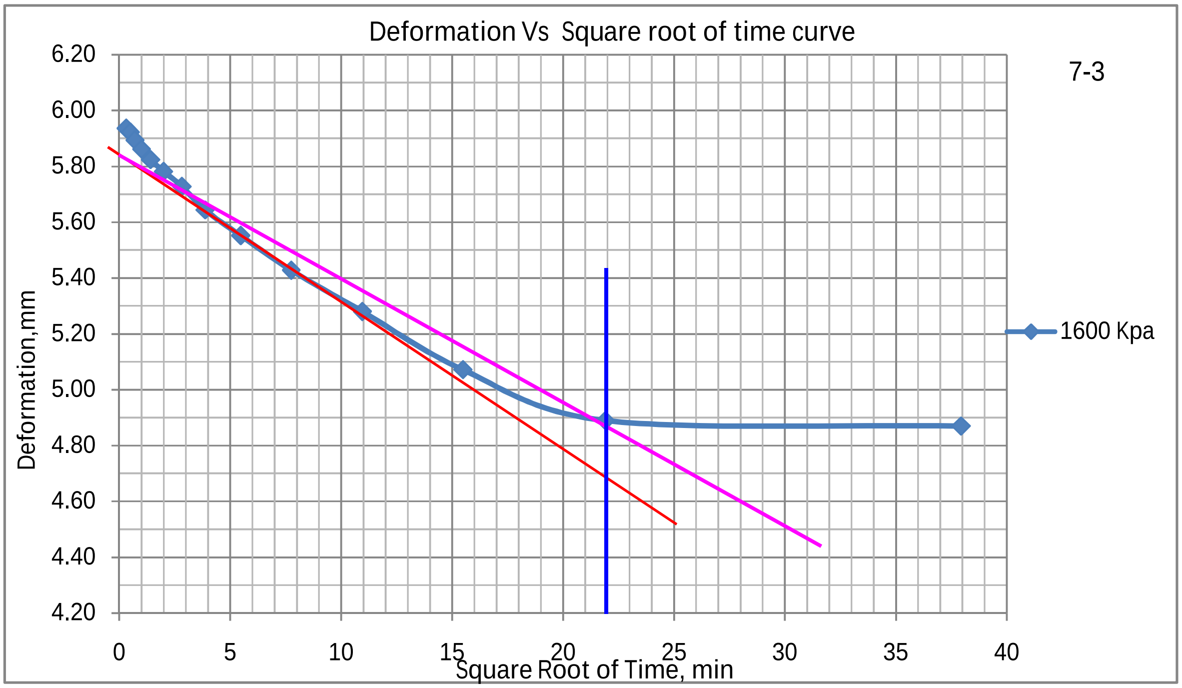

Figure E -17 Typical plots of Deformation versus Square root of time for remolded expansive soil sample of Galan town, Sample 7-3 (P=1600kPa),