CONSOLIDATION AND COMPACTION CHARACTERISTICS OF SOIL AT IKOLE CAMPUS OF FEDERAL UNIVERSITY OYE-EKITI, EKITI STATE.

EMATE VICTOR EJIROGHENE.
(CVE/11/0368)

A PROJECT SUBMITTED IN PARTIAL FULFILMENT OF THE REQUIREMENT FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG), IN CIVIL ENGINEERING.

FEDERAL UNIVERSITY OYE EKITI, EKITI-NIGERIA.

SEPTEMBER, 2016

ABSTRACT

Assessing the geotechnical parameters helps in improving the soil settlement which usually causes damage to the civil structure. Compaction is one of the most important and routine engineering techniques, performed to assure the safety and stability of soils. In this study, geotechnical investigations at the Ikole campus of Federal University Oye-Ekiti, Ekiti State are carried out for undisturbed and disturbed samples obtained at five (5) specified locations and at a depths of three (3) to five (5) metres below normal ground level in order to classify the soil and assess the consolidation and compaction characteristics of undisturbed and disturbed/remolded samples.

This research work was been carried out to determine the shear strength characteristics of soils in Ikole Campus of Federal University Oye Ekiti, Ekiti State. This research was also used as a medium of carrying out of classification of the soils in this study location. The study area falls within coordinates 7.7983 oN, 5.5145oE of and covers a land area of 538.550 hectares in Ikole Ekiti, Ekiti state.

Five different locations were considered in which trial pit method was used in taking samples of disturbed and undisturbed samples from depths of 1.5m and 3.0m. The coordinates of the locations are trial pit 1 (866971.98N, 610838.61E), trial pit 2 (867676.65N, 611093.21E), trial pit 3 (867224.80N, 610566.90E), trial pit 4 (867759.99N, 610610.02E) and trial pit 5 (867382.93N, 610810.38E). The tests carried out for the purpose of this research are natural moisture content, particle size distribution, specific gravity, Atterberg limit, compaction and consolidation. The results indicated that only point 3 soil is a lateritic soil while others are clayey, all the soils in the locations have low water content, locations with clayey content have close rate of settlements. Pits 1, 2, 4 and 5 soils are grouped into A-7-5 or A-7-6 classification i.e. clayey soil while pit 3 is classified to A-2-6 i.e. lateritic soil.

Keyword: Soil, Shear strength and Ikole Ekiti.

CERTIFICATION

This is to certify that this project was carried by EMATE VICTOR EJIROGHENE (CVE/11/0368) under my supervisor, in partial fulfillment of the requirement for the award of Bachelor of Engineering (B. Eng) Degree in Civil Engineering, Federal University Oye-Ekiti, Ekiti State, Nigeria.

....

Engr. BOLARINWA, A

Project Supervisor

....

Prof. ADEYERI, J.B.

Head of Department

ACKNOWLEDGEMENT

I express my most sincere acknowledgement to my supervisor Engr. Bolarinwa for his thorough supervision assistance, patience, encouragement, and his thorough understanding of the project work. I am also grateful for the knowledge impacted through numerous discussions.

I also thank my Head of Department, Prof. ADEYERI, J.B. and also my colleagues for their assistance in every way throughout the course of this work and other staffs in Civil Engineering Department who has helped me in one way or the other.

Table of Contents

LIST OF FIGURES

LIST OF TABLE

CHAPTER ONE

1.0 INTRODUCTION

1.1 General Background

Consolidation of soil is of great concern to engineers engaged in design and construction of foundations, embankments, bridge abutments, and earth dams and fills, Crawford (1986). The existing soil at a construction site may not always be totally suitable for supporting structures, Bowel (2007) adequate stabilization is therefore required if the soil is not suitable. Settlement of the subsoil causes damage to the structures due to stability problems which needs to be addressed before planning any engineering project, Osman (2006). Consolidation characteristics of a soil depend mainly on the clay contents and plasticity index, IJET (2012). Also consolidation characteristics of most soils appear to depend on the nature of the soil, the position of the sample in the profile and the characteristics of the material deposit, Gidigasu (1976). Settlement potential of soil increases with increase of these values. However, higher values of these characteristics the more undesirable they are for use under foundations and roads etc., IJET (2012). Clayey and silty soils have lower permeability and due to this reason, the settlement and consolidation take longer durations to occur. Consolidation deals with the response of a saturated soil to the imposed steady static pressure and predicts stresses and displacement of the loaded soil as a function of space and time, Lav & Ansal (2001). The time taken for settlement is thus a crucial factor that can influence the construction of embankments and sub-grades for roadways and rail-tracks, Townsend (1985). Although, consolidation is used for estimating settlements, it has also played key roles in the design and construction of civil engineering infrastructures, Schiffman *et al* (1984).

Most of the current design practices in geotechnical engineering are based on settlement and strength criteria. The recommendation for the allowable bearing capacity to be used for design is based on the minimum of either limiting the settlement to a tolerable amount, or the allowable bearing capacity, which considers soil strength, as computed, Taha, *et al.* (2000). Where shear strength is considered as one of the most important design parameters, also the settlement must not be exceed the allowable settlement. The safe design of structures therefore depends on what happened to the shear strength of soil in the field during the construction process and what is the total settlement occurred, Bowels (2007). Construction of most civil structures such as dams, highway, buildings etc. involves the use of compacted soils also, construction of concrete structure generally based on soil, is compacted before starting construction, Coko, *et al.* (2004). Soil compaction is one of the most important engineering techniques commonly performed in engineering projects such as highways, railway sub-grades, airfield pavements, earth dams, landfill, and foundations. The main aim of soil compaction is to improve engineering properties of soils such as increase density, reduction in compressibility leading to reduction in settlement, reduction in permeability, increase in shear strength, and increase in bearing capacity, Nagaraj (2000). Compaction is the process of mechanically pressing the soil particles together into a close state of contact with air being expelled from the soil, Imhoff *et al* (2004). In this process, both the number and size of voids in a given soil mass will be reduced, and therefore, the density of the soil increases, and the engineering property changes significantly, Atsbeha (2012). Compaction characteristics of soils are expressed in terms of Maximum dry density (ρ_{dmax}) and Optimum moisture content (OMC).

1.2 Aim and Objectives

The aim of this research is to determine the consolidation and compaction characteristics of soils in the Ikole campus of Federal University Oye-Ekiti, Ekiti State.

The objectives of this research are;

i. to determine the range of consolidation and compaction parameter of the soil around the campus

ii. to establish relevant relationships between consolidation characteristics and compaction characteristics of soils .

iii. to examine the validity of the degree of relation, and to draw appropriate conclusions on the relationships of each empirical equations.

iv. to evaluate the effect of the variation in particle-size distribution of compacted soil on the permeability and compressibility characteristics

1.3 Statement of Problem

Although so many research works have been carried out on soil in various locations in and around Ekiti state but there has not been any published or unpublished records so far on soils in Ikole-Ekiti L.G.A and its environs. This project research is however useful for soil classification at Ikole-Ekiti as it aims to provide information on the soil geotechnical properties. This will serve as a reference point for other subsequent works such as constructional activities and researches anticipated over time. This project concentrates on the consolidation and compaction characteristics which is highly crucial to construction of roads particularly. Compaction and consolidation of soil are very frequent topic that arises in majority of civil engineering projects. Determination of compaction and consolidation characteristics is very necessary for effective planning. .This study is therefore necessary for determination of the various parameters involved in predicting the soil characteristics and instrumental to making relevant recommendation that will inform the engineers in the future in case of construction works.

1.4 Scope of Study

For this study, the soil in the Ikole campus of Federal University Oye-Ekiti, Ekiti State was collected from trial pits of about three (3) to five (5) metres and taken to the laboratory for determination of the consolidation and compaction characteristics. These trial pits were dug in five different locations around the campus, see figure 1-3. Disturbed and undisturbed samples were collected from these trial pits. Compaction tests, oeodometer test and a few other classification tests were carried out on the soil samples to determine the consolidation and compaction characteristics. The data obtained were analyzed and plotted on their relevant graphs to determine some other useful parameters instrumental to predicting the consolidation and compaction characteristics by the use of application of software such as Microsoft spreadsheet and AutoCAD.

1.5 Study Area

The study area falls within coordinates (7.7983^{0}N, 5.5145^{0}E) and covers a land area of 538.550 hectares in Ikole Ekiti, Ekiti state. Ikole-Ekiti is underlain by rocks of the Crystalline Basement Complex (Figure 1.1), Talabi and Tijani (2011). The studied soils are underlain by migmatite gneiss, which is arguably the most extensive member of the Basement Complex rocks of Southwestern Nigeria, Rahaman (1976). Field study of the rock revealed mixture of felsic and mafic mineral components, Boesse and ocan (1992). The major minerals in the rock include biotite, hornblende, feldspar and quartz. The rocks are foliated with pegmatite and quartz veins intrusions on most of the outcrops, Talabi and Tijani (2011).

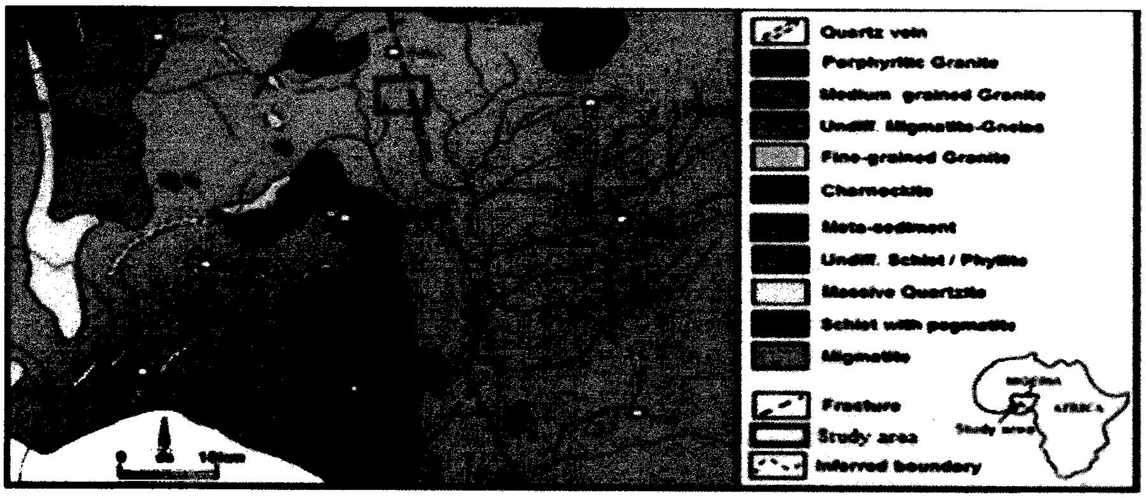


Figure 1.1: Geological Map of the Study Area (Talabi and Tijani, 2011).

The coordinates of the points where the soil samples were taken are presented in the tables below;

S/N	Trial pits	Coordinates in degree		Coordinates in metric (m)		Elevation (m)
		Northing	**Easting**	**Northing**	**Easting**	
1	TP1	7.801562^0	5.496712^0	866971.98	610838.61	553
2	TP2	7.808083^0	5.499003^0	867676.65	611093.21	539
3	TP3	7.803837^0	5.494267^0	867224.80	610566.90	561
4	TP4	7.808653^0	5.494655^0	867759.99	610610.02	551
5	TP5	7.805260^0	5.496458^0	867382.93	610810.38	568

Figure 1.2 Trial Pits and their Coordinates

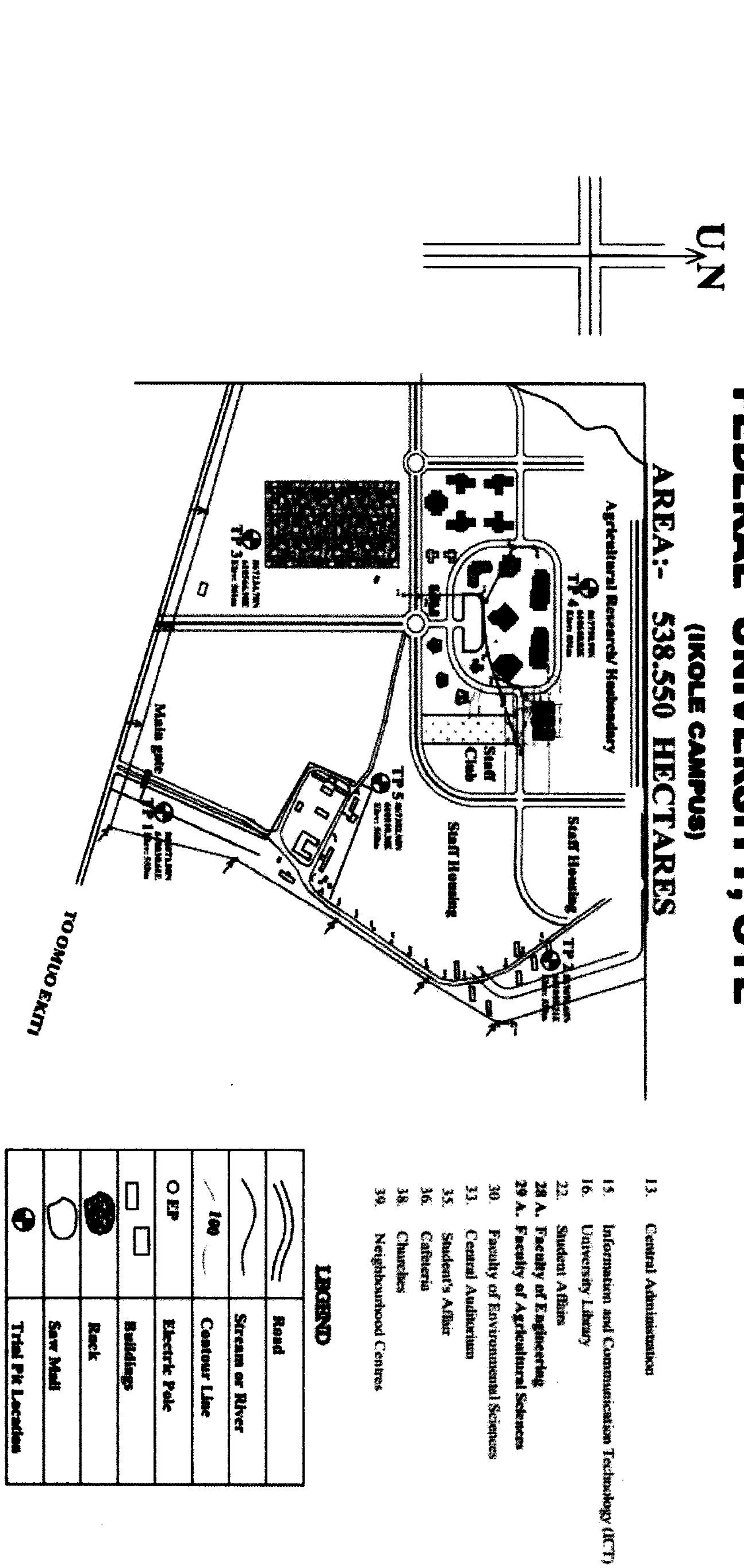


Fig. 1.1: Survey map showing study location and trial pit with coordinate

CHAPTER TWO

2.0 LITERATURE REVIEW

In Ekiti State numerous researches has been conducted on soil, its geotechnical properties and how to improve the soil geotechnical properties to become suitable for the consequent structure coming on it. To mention a few: comparative analyses of Ekiti State soil stabilized with different additives by Adeyemi and Joseph (2015), the study aimed at investigating the effects of three different additives - Sawdust Ash (SDA), Palm Kernel Shell Ash (PKSA) and Groundnut Shell Ash (GSA) on the geotechnical properties of Ido- Ekiti soil in Ido-Osi Local Government Area (LGA) of Ekiti State. Soil samples collected from the study area were subjected to various laboratory tests (i.e. Grain Size Analysis, Atterberg Limits and Compaction tests) in its treated and untreated state. The results of the tests carried out on the untreated soil sample indicated that the soil could be generally classified as Granular soil material and they are either Silty or Clayey Gravel and Sand with general Subgrade rating of excellent to good. However, further research work needs be done on this study in other to ascertain any other suitable hidden properties of the additives. Effects of locally available additives on geotechnical properties Of Ijero local government soils, Ekiti State, Nigeria by Adetoro and Ayeni (2015). This research work analyzed the effects of Palm Kernel Shell Ash (PKSA) and Sawdust Ash (SDA) additives on Geotechnical properties of Ekiti State soil and the study area is within Ijero Local Government Area. Four soil samples (i.e. A, B, C and D) were collected at some locations within the study area and subjected to the following geotechnical tests in the laboratory: Sieve analysis, Atterberg limits and Compaction. The additives were added to the soil samples at 0%, 2%, 4%, 6% and 8% proportions by soil weight. After the soil treatment (i.e. addition of additives to the soil samples), the LL, PI and MDD values increased as the quantities of additives increases on some soil samples. Thus this study proved that it is possible to use PKSA and SDA as cheap stabilizing agent. This will go a long way in reducing agricultural and industrial waste in the environment. However, in order to improve on the use of these additives, the additives should not be used for soil with extremely high content of clay, thus could be used for soil with very low content of clay. There is need for further study on these additives. Other research works carried out on soil in Ekiti state include; Moisture-Density Relationship of Selected Clay Soils in Ekiti State, Nigeria, Adekanmi and Adebayo (2016); Geophysical and Geotechnical Investigations of a proposed Site for Afijio Local Government Stadium Ilora, Southwestern Nigeria, Aladejana *et al.* (2015); Consolidation properties of compacted lateritic soil stabilized with tyre ash, Afolagboye and Talabi (2013); Geophysical investigation of Road failure in the basement complex areas of south western Nigeria, Oladapo *et al* (2008). Although so many research works have been carried out on soil in various locations in Ekiti state but there has not been any research so far on soils in Ikole-Ekiti L.G.A and its environs. This project research is however a breakthrough for Ikole-Ekiti as it aims to provide information on the soil geotechnical properties in Ikole campus of Federal University, Oye-Ekiti. This will serve as a reference point for other subsequent works such as constructional activities and researches anticipated over time.

2.1 Geology of Ekiti State

The southwestern Nigeria falls between latitude 700N and 1000N and longitude 200E and 700E which is made up of rocks which are mainly Precambrian in age, Rahaman (1976). Ekiti State b within the Precambrian Basement complex of southwestern Nigeria which lies to the rest of the West African Craton in the region of late Precambrian to early Paleozoic orogenesis, Rahaman (1988). The Nigeria basement complex extends westward and is continuous with the dahomeyan of the Dahomey – Togo - Ghana region to east and the south Mesozoic recent sediments of

Dahomey and Niger coastal basins over the basement complex, Rahaman (1976). In general, Ekiti State is underlain by metamorphic rocks of the Precambrian basement complex, the great majority of which are very ancient in age. These basement complex rocks show great variations in grain size and in mineral composition. The rocks are quartz gneisses and schists consisting essentially of quartz with small amounts of white micaceous minerals, Oyawoye (1992). In grain size and structure, the rocks vary from very coarse grained pegmatite to medium grained gneisses. The rocks are strongly foliated and they occur as outcrops especially in Efon Alaaye and Ikere-Ekiti areas, Oyawoye (1992). See figure 1.1 above.

2.2 Consolidation

2.2.1 Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. Since the surrounding layer soil strata are confined, the soils are unable to spread laterally. Hence there must be adjustment to the new pressure by vertical deformation, Nwaiwu & Nuhu (2006). The compression of the soil mass leads to the decrease in the volume of the mass, which in turn result in the settlement of the structure, built on the mass; Salas *et al* (1953).The vertical compression of the soil mass under increased pressures is thus made up of the following components:

- **i.** Deformation of the soil grai
- **ii.** Compression of water and air with in the voids
- **iii.** An escape of water and air from the voids

It is quite rational and acceptable to assume that the solid matter and the pore water relatively are incompressible under the loads encountered. The change in volume of the soil mass under imposed stresses must be only due to the escape of water and air. Generally, the volume change in a soil deposit can be divided in to three stages, Arora (1993).

a) Initial Consolidation:

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles; Mesfin (2005).

b) Primary Consolidation:

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as excess pore water, as water is almost incompressible as compared with solid particles, Mesfin (2005). A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the permeability of the soil and is, therefore, time dependent. The reduction in volume is called primary consolidation. In fine grained soils, the primary consolidation occurs over a long time: On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability, Strokova (2013). As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles.

c) Secondary Consolidation:

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, Józs (2003). This additional reduction in the volume is called secondary consolidation. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the adsorbed water to the new stress system. In most inorganic soil, it is generally small, Mesfin (2005).

2.2.2 Factors Affecting Consolidation Characteristics of Soil

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

2.2.2.1 Stress History

The maximum stress to which the soil is subjected in the past influence the consolidation characteristics of the soil in its insitu condition. In remolded soils, because it has lost its structural characteristics as compared with its structure in its natural condition, it is inferred that a remolded soil is unsuitable for evaluating its stress history, Jumikis (1984).

As to the stress history, the in-situ soil can be grouped in to two categories:

i. Normally Consolidated Soil

   A normally consolidated soil is one whose present effective overburden pressure on the in-situ prototype soil deposit is the maximum pressure to which the soil has ever been subjected at any time in the past history. In other words, the normally consolidated soil is one whose pre-consolidation pressure is equal to its present effective overburden pressure.

ii. Over-Consolidated Soil

Over-consolidated clay is one which has been completely consolidated under a large overburden pressure in the past that is larger than the present overburden pressure. The response of over-consolidated clays to applied loads is such that at early loading the soil shows relatively small decrease of void ratio with load up to the maximum effective stress to which the soil was subjected in the past. If the effective stress on the soil specimen is increased further, the decrease of void ratio with stress level will be larger.

2.2.2.2 Permeability

The expulsion of water from the voids of a saturated clay soil by an externally applied load in the consolidation process and the change in volume associated with such a process are essentially a hydraulic problem. Specifically, it is a problem of permeability of a soil to water. Therefore, the rate of consolidation depends on the permeability of the soil, Eberemu and Adrian (2011). The permeability of the soil by itself is a function of the soil type, size and shape of the soil particles (rounded, angular, or flaky), and thus, up on the size and geometry of voids. Also, the resistance is a function of the temperature of water (viscosity and surface tension effect), Jumikis (1984).

2.2.3 Theory of One-dimensional Consolidation

The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi (1943). The underlying assumptions in the derivation of the mathematical equation are the following:

i. The soil is homogeneous and isotropic

ii. The soil is fully saturated

iii. The soil particles and the water in the void are incompressible. The consolidation occurs due to expulsion of water from the void

iv. Darcy's law is valid throughout the consolidation process

v. The soil is laterally confined and consolidation takes place only in the axial direction. Drainage of water occurs in one direction.

The assumptions made by Terzaghi are not fully satisfied in actual field conditions. The results obtained from the use of the theory to practical problem are approximate. However, considering complexity of the problem, the theory gives reasonably accurate estimate of the time rate of settlement of a structure built on the soil. The standard one dimensional consolidation test is usually carried out on saturated specimen using an Odometer, Das (1997). 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 Puri & Nitish (2012). 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/cm^2, Das (1997). 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 is mainly the coefficient of consolidation, Cv, will be determined from the test. The coefficient of consolidation relates to how long it will take for an amount of consolidation to take place; Bowles (1984). The results of the odometer test are usually presented in the form of a dial reading- time plots.

2.2.3.1 Coefficient of Consolidation

A factor involved in characterizing the rate of consolidation of a soil is the one called the coefficient of consolidation, $C_v$expressed as

$$C_v = 0.848 \frac{(\frac{d}{2})^2}{t_{90}}$$

The coefficient of consolidation C_V as determined by Casagrande's semi logarithmic plot of dial gauge reading against the root of time.

2.3 Compaction

Discrete particles that form whole soil mass are not strongly bonded together, hence they can move freely with respect to one another once the disturbing energy is applied. However, it is not easy when compared to elements that accompany fluid. Thus, soil is inherently a particulate system Bose (2012). Generally when load is transmitted to soil, contact forces developed between adjacent particles. It can be said that deformation of soil mass is controlled by interactions between individual particles, especially sliding between particles. The inter-particle forces, in conjunction with the external forces at the time of formation of the soil and stress history, are responsible for the structure of a compacted soil, Jeng and Strohm (1976). Leroueil and Vaughan (1990) states that the effect of structure is as important in determining engineering behavior, as are the effects of porosity and stress history.

Existence of spaces (voids) among the soil particles, called pore spaces usually filled with air or water (with or without dissolved material). Thus, the soil is naturally a multiphase system as shown in fig. 2.1, that consist a mineral phase and fluid phase (both water and air) called pore fluid. In case of very tiny soil particles, the pore fluids may intrude between the Particles. Although particles are no longer in contact in the usual way, they still remain in close proximity cam transmit the load, also tangential force. The pore space between particles tends to increase or decrease as the transmitted compressive forces decrease or increase, Singh (2012).

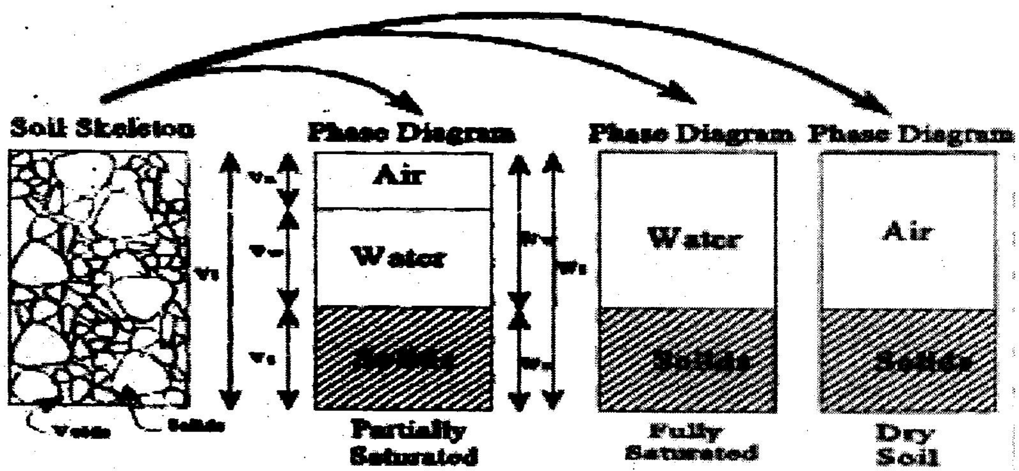


Figure 2.2: Weight Volume Relationship, Singh (2012).

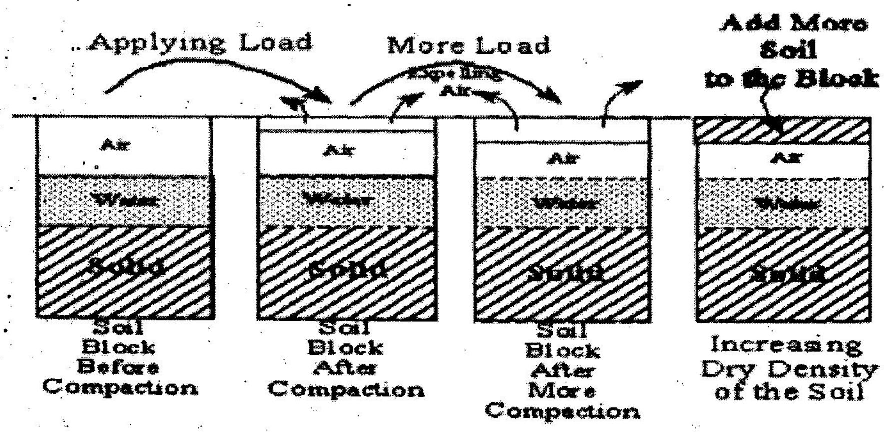


Figure 2.3: Mechanism of Soil Compaction Singh (2012).

Thus, soil is inherently multiphase, and the constituents of pore phase will influence the nature of the mineral surfaces and hence affect the processes of force transfer at particles contact. This phenomenon is known as chemical interaction. By reducing the air voids, more soil can be added to block. When moisture is added to block (water content, is increasing) the soil particles with result in adding more soil and, hence, the dry density will increase accordingly. This phenomenon as illustrated in fig 2.3

Soil can be perfectly dry (no water contents) and fully saturated or partially saturated (with both water and air). Water flows in soil from high energy to low point of energy and relate amount to pressure applied to soil. More permeable the soil, water movement get better for a given excess pressure. The flow also altering the magnitude of forces at the contact between particles and influences the compression and shear resistance of soil. Because the soil is multiphase system, it may in expectation that load given to soil mass will be carried in part by pore fluid. When load is applied to a soil, it suddenly changes; this change is carried jointly by pore fluid and mineral skeleton. Changes in pore pressure will cause water to move through soil; hence the properties of soil essentially change with time, Yesim (2004).

2.3.1 Purpose of Compaction

The compaction process increases the strength characteristics of soils, which in turn increases the bearing capacity of foundations, decreases the amount of excessive settlement of structures, and increases the stability of slopes of embankments. Generally, compaction is used as practical means of achieving the following characteristics of soils, Arora (2004):

i. **Reduce Excessive Settlement and Compressibility**

The primary advantage resulting from the compaction of soils used in embankments is that it reduces settlement that might be caused by consolidation of the soil within the body of the embankment. This is true because compaction and consolidation both bring about closer arrangement of soil particles. Densification by compaction prevents later consolidation and settlement of a structure, Military Soils Engineering (1997).

ii. **Increase Shear Strength**

The increase in density by compaction usually increases shearing resistance, Alemayehu and Mesfin (2009). This effect is highly desirable that it may allow the use of thinner pavement structure over a compacted sub-grade or the use of steeper side slopes for an embankment. For the same density, the highest strengths are frequently obtained by using greater compactive effort. Large-scale experiments have indicated that the unconfined compressive strength of clayey sand could be doubled by compaction, Military Soils Engineering (1997).

iii. **Reduce Permeability and Seepage**

When soil particles are forced together by compaction, both the number of voids contained in the soil mass and the size of the individual void spaces are reduced, Military Soils Engineering, (1997). This change in voids has an obvious effect on the movement of water through the soil. One effect is to reduce the permeability, thus reducing the seepage of water in earth dams, road embankments and water loss in reservoirs through deep percolation.

iv. **Optimizes Swelling and Shrinkage Characteristics**

Swelling characteristics is an important soil property. For expansive clay soils, the greater the density the greater the potential volume change due to swelling unless the soil is restrained, Amer *et al* (2006). An expansive clay soil should be compacted at moisture content at which swelling will not be excessive. Although the conditions corresponding to a minimum swell and minimum shrinkage may not be exactly the same, soils generally may be compacted so that these effects are minimized, Amer *et al* (2006).

2.3.2 Factors Affecting Compaction Characteristics

Compaction characteristics of soils depend up on many factors such as water content of the soils, amount of compaction energy, soil type, method of compaction, and admixtures, Terzaghi (1943).

2.3.2.1 Moisture Content in the Soil

The moisture content of a soil affects its dry density, Alemayehu and Mesfin (2009). A soil with very low water content is difficult to compress into close state of particles. This results in higher void ratio and hence lower dry density for the same compaction effort. On the other hand when the water content increases excessively, the soil grain tends to move apart and the total void ratio continues to increase whereas the dry density falls. However, if the moisture content of the soil is of some intermediate specific value, the water acts as lubricant causing the soil to soften and become more workable. In this case the soil grains are close packed thus lowering the void content and increasing the dry density, Alemayehu and Mesfin (2009). This specific value of moisture is called optimum water content and the corresponding dry density termed as maximum dry density.

2.3.2.2 Amount of Compaction Energy

The compactive effort is the amount of energy applied on the soil. With a soil of given moisture content, if the amount of compaction energy increases, the soils particles will be packed so that the dry unit weight increases. For a given compactive effort, there is only one moisture content which gives the maximum dry unit weight. If the compactive effort is increased the maximum dry unit weight also increases, but the optimum moisture content decreases, Roberts *et al* (2000).

2.3.2.2 Soil Type

The nature of a soil itself has a great effect on its response to a given compactive effort. Compaction characteristics of soils are divided in to three groups, Compaction of cohesionless soils, Compaction of sandy or silty soils with moderate cohesion, and compaction of clay, Terzaghi (1943).

In general, coarse grained soils can be compacted to higher dry density than fine grained soils, Arora (2004). In Coarse grained soils, when the amount of fines and the voids of the coarse grained soils are about the same highest dry density can be achieved, Arora (2004). In sand, the well graded sand attains higher dry density than poorly graded sand. Cohesive soils with high plasticity have, generally, low dry density and high optimum moisture content, Arora (2004).

2.3.3 Method of Soil Compaction

2.3.3.1 Laboratory Compaction Method

To attain the required maximum dry unit weight in the field, first appropriate tests are determined in the laboratory and this laboratory results must be confirmed in the field. The following tests are normally carried out in a laboratory, ASTM (1998).

A. **Standard Proctor Compaction Test (ASTM D-698)**

Proctor developed this test in connection with the construction of earth fill dams in California in 1933, Murphy (2007). It gives the standard specifications for conducting the test. A soil at a selected water content is placed in three layers into a mold of 101.6mm diameter, with each layer compacted by 25 blows of a 2.5 kg hammer dropped from a height of 305 mm, subjecting the soil to a total compactive effort of about $600KN/m^2$, so that the resulting dry unit weight at optimum water content is determined, Murphy (2007).

B. **Modified Proctor Compaction Test (ASTM D-1557)**

This test method covers laboratory compaction procedures used to determine the relationship between water content and dry unit weight of soils, compacted in 5-layers by 101.6mm diameter mold with a 4.5kg hammer dropped from a height of 457mm producing a compactive effort of $2{,}700KN/m^2$

2.3.3.2 Field Compaction Methods

Several methods are used for compaction of soils in the field. The choice of these methods depends up on the soil type, the maximum dry density required and economic considerations. The four major types of compaction processes currently in use by modern construction equipment are:

A. Impact:

Impact compaction involves dropping a weight on the soil during compaction, Aysen (2002). This compaction equipment subjects the soil to a series of blows until the desired density is reached. In order to effectively compact the soil with an impact, it must be placed in multiple lifts so that the stress of the blow is distributed through the entire lift. Another form of impact compaction is known as deep dynamic compaction, Garga and Madureira (1985). This type of compaction uses crane and very large mass to compact the soil to significant depth below the surface.

B. Manipulation:

Compaction performed by manipulation is accomplished by introducing kneading force to the soil during compaction. The construction equipment manipulates the soil over a series of passes until the desired level of compaction is achieved. The Proctor test does not accurately model the manipulation mechanisms of this type of compaction. The most common type of manipulation compaction test is typically referred to as the Miniature Harvard Compaction Test. Neither ASTM nor AASHTO currently has a recommended procedure for use of the Harvard Miniature Mold apparatus for compaction testing, Donaghe and Torrey (1994). It is commonly used for research purposes.

C. Pressure:

During pressure compaction, called static compaction, usually consolidation apparatus is used, in laboratory, to compress the soil into a ring of known volume, Donaghe and Torrey (1994). Static compaction is useful research method but the researcher must realize that in the field it may not have the same level of control.

D. Vibration:

Vibratory compaction is used to shake the soil into more dense state, Aysen (2002). The compaction equipment induces strong vibrations in the soil to the desired level of compaction. In general, modern compaction equipment typically incorporates more than one type of compaction mechanism at a time to accomplish compaction of the soil. Selection of the proper compaction method depends on the type of soil, the size of the project, final compaction requirements, rate of production, and economic factors, Blotz *et al* (1994).

The necessary compaction for sub-grades of roads, earth fills, and embankments may be obtained by mechanical means, Murphy (2007). Some of the equipments that are normally used for compaction are as follows;

i. Smooth wheel roller
ii. Rubber tired rollers
iii. Sheep foot rollers
iv. Vibratory rollers

The choice of roller for a given job depends on the type of soil to be compacted and percentage of compaction to be obtained. For cohesive soil Sheep's foot roller, or Rubber tired roller, and for cohesionless soils Rubber-tired roller or Vibratory roller are suggested, Murphy (2007).

2.3.4 Water Content Dry-Density Relationships

When some moisture is added to dry soil, the soil grains are surrounded by a film of adsorbed water. If more water is added, the film of water becomes thicker and the soil particle surrounded by this film of water slide over each other more easily. At this condition, when some specified compactive effort is applied, the soil particles becomes close together easily. The water in this process acts as lubricant and the soil particles become so closely packed together by the expulsion of air from the voids, Mittal and Shukla (2009).

If we continue to add still more water into the soil, the water occupies the space that could have been occupying by the soil particles during compaction. Thus, the soils are not dense under the given effort because the water hinders the soil grains from being close packed together. This condition leads to the conclusion that "there must be most appropriate water content that the water could provide maximum benefit of lubrication without occupying a space that could have been occupied by the soil grains with a given compaction effort", Horpibulsuk *et al* (2008).

Such moisture content at which the unit weight of compacted soil becomes maximum is called Optimum Moister Content (OMC) and the corresponding density is called Maximum Dry Density (ρ_{dmax}), Mittal and Shukla (2009). Most soils exhibit similar relationship between moisture content and dry density when subjected to a given compactive effort, Military Soils Engineering (1997). For each soil, maximum dry density develops at its OMC for a given compactive effort. Beyond OMC, the air content of most soils remains essentially the same even though the moisture content is increased, Military Soils Engineering (1997).

CHAPTER THREE

3.0 METHODOLOGY

3.1 Preamble

The practice of testing soil samples in the geotechnical laboratory plays important role in soil mechanics and civil engineering practices. It is important to carry out these geotechnical tests because it provides the engineer with relevant information which is critical and instrumental to making key decisions in the design and construction of civil works. Therefore, evaluation of materials by various geotechnical tests to determine their suitability is highly essential. This will ensure a satisfactorily performance when put into service for use.

3.2 Sampling of Materials

The samples were collected from trial pits dug at five locations around the campus. Two groups of samples were collected from each trial pit, the first group at a depth of 1.5m and the second group at a depth of 3.0m from each trial pits both disturbed and undisturbed samples were collected. The location for the trial pits was chosen at strategic points on the master plan of the university; the points chosen are such that it takes into consideration all the varieties of soil present on the campus. The plan indicates the developing area in which five different points were chosen by picking four edges and center of the proposed developing area. With trial pit 1 (TP1) being somewhere around the campus gate, TP2 being somewhere around the campus hostel, TP3 around former FADAMA, TP4 around Engineering faculty and TP5 was around the school market.

3.2.1 Disturbed Sampling

These are soil samples that their natural state and structure have been altered due to change in their physical appearance and are basically used for soil classification.

The disturbed soil samples were collected from the trial pits both at 1.5m and 3m depth with the use of hand angers, packed into the sack and labeled to avoid misinterpretation of results. Adequate quantity was taken ensuring it will be sufficient for all necessary experiments. While taking the disturbed sample, little quantity of the disturbed sample is also taken immediately from the trial pit and put into a small sealed polythene bag in order to avoid moisture loss. This soil sample is used in the laboratory for determining the moisture content of the soil.

3.2.2 Undisturbed Sampling

These are soil samples that their natural state and structure have not been altered. They are usually as a unit still compactive. They are usually obtainable in cohesive soils. The undisturbed soil samples were collected from the trial pits at both depths with the use of a shovel and small digger. This was achievable by using the digger to form a circular shape, digging gently round this circular shape until a satisfied depth of about 175mm is reach and then the shovel was used to form a chamfer between the sample and the soil all round and then gently lifting the sample until it cut off from the soil. This undisturbed sample was gently carried and lifted out of the pit to avoid breaking and was put immediately into an airtight polythene bag so as to maintain its moisture content. All the samples collected, both disturbed and undisturbed were taken and transported to the laboratory immediately after obtaining them from each location or trial pit.

3.3 Test Performed and Procedures

The laboratory test performed carried out in the course determining the consolidation and compaction characteristics are listed below; particle size distribution, specific gravity, atterberg limit (LL, PL and SL), compaction test, unconfined compression test, odometer test

3.3.1 Particle Size Distribution

This test is done to determine the particle size distribution of a soil sample. An oven-dried sample of the soil is weighed and passed through a set of BS sieves and shook thoroughly by using mechanical sieve shaker.

The weight of the each sieve is recorded and the percentage of sample retained and passing through each sieve is calculated. The percentage passed is plotted on the sand and gravel fraction of a semi-logarithmic chart.

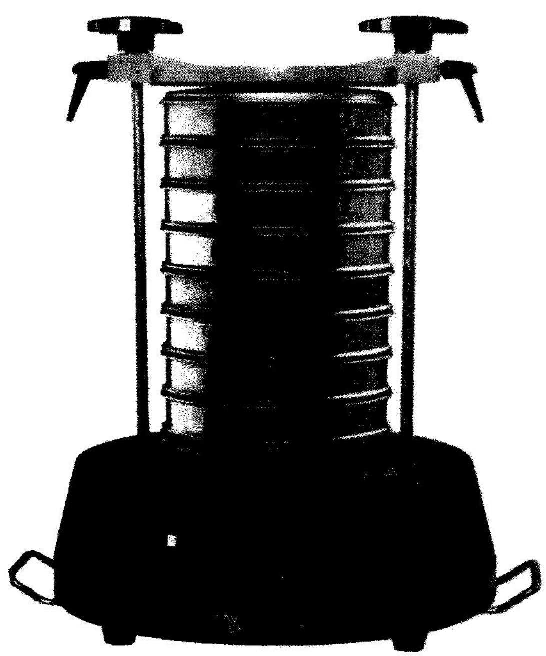


Figure 3.1- Mechanical Sieve Shaker

3.3.2 Specific Gravity

This is measured using standard density bottle. A known weight of oven-dried soil sample WS is put into a density bottle and is topped up with distilled water and ensured that all air or void from the sample is removed. The bottle is brought to a constant temperature, carefully wiped dry and weighed.

Specific gravity $G_S = \frac{weight\ of\ soil\ sample}{weight\ of\ eqaul\ volume\ of\ water}$

3.3.3 Atterberg Limit

3.3.3.1 Liquid Limit

Liquid limit (LL) is defined as the minimum moisture content at which the soil will flow under its own weight. It is determined by the standard Casagrande device apparatus. A simple of oven-dried soil all passing the 0.425mm sieve, is mixed with distilled water to a stiff consistency, a portion of it placed in the cup and leveled off parallel to the based. A groove is made through the center of this portion using the grooving tool. By turning the handle at two revolutions per second the cup is lifted 10mm and dropped on to the rubber base until the bottom of the groove has closed over a length of 10mm. The number of blows at which the groove has closed 10mm is recoded. This is repeated until two consecutive runs give the same number of blows for closure. At this stage the moisture contents of the soil in the cup is determined.

3.3.3.2 Plastic Limit

Plastic limit (PL) is defined as the minimum moisture content at which the soil can be rolled into a thread 3mm diameter without breaking up.

About 20g of the dried soil, all passing the 0.425mm sieve, is mixed with distilled water and molded into a ball. The ball is rolled by hand on a glass plate with sufficient pressure to form a thread. When the diameter of the resulting thread becomes 3mm the soil is kneaded together and then rolled out again. The process is continued until the thread crumbles when it is 3mm diameter, and at this stage the moisture content of the soil is determined. This whole procedure is carried out twice and the average value of moisture content taken as the plastic limit of the soil.

3.3.3.3 Shrinkage Limit

Shrinkage limit (SL) is defined as the maximum moisture content at which further loss of moisture does not cause a decrease in the volume of the soil. Mix a dried soil passing 0.425mm sieve to a consistency slightly above the expected liquid limit of the soil. Lightly coat the linear shrinkage mold with oil to prevent the soil sticking to the mold. The soils filled into the mould, ensured that no air is trapped and the whole sample is later dried. The soil bar is measured and recorded as as the original length of the mold is measured and recorded as L_0.

$$\text{Linear shrinkage L.S} = \frac{L_0 - L_f}{L_f} \times 100$$

3.3.4 Compaction Test (Proctor Test)

The standard proctor test is a method if finding the optimum moisture content for compaction of soil. A cylindrical mold 0.001 in volume is filled with a soil sample in three layers, each layer being compacted by 25 blows of a standard hammer, (weighing 2.5kg, height of drop 300mm each blow). The mould is then trimmed and weighed, hence giving the bulk density of the soil. The moisture content of the soil is then determined, and hence the dry density. The experiment is carried out with soil at different moisture contents and a graph of dry density against moisture content is plotted.

3.3.5 Odometer Test

An undisturbed sample of soil is retained in a 71mm diameter cutting ring. The initial moisture content of the soil and the initial void ratio is determined.

i. A load P1 is applied to the sample and the change in thickness (compression) of the sample was read at suitable intervals.
ii. A graph of time against compression is plotted.
iii. Next, load P1 is increased to P2 and plotting continued for further 24 hours. Then increase load to P3 and continue plotting.
iv. Finally, the pressure is released and allowed the sample to expand and take up water.
v. The final value of compression and final value of moisture are determined.

CHAPTER FOUR

4.0 RESULT AND DISCUSSION

This chapter present the results of various test carried out on the five locations used as a case study. Table, graphs and result were also discussed in this chapter.

The tests carried. Out include classification characteristics test (natural moisture content test, specific gravity test, particle size analysis and Atterberg limit) and mechanical behavior tests (compaction test, unconfined compression and odometer tests).

4.1 Natural Moisture Content

Results of the moisture content (MC)are shown in table 4.1. From the result some pits have high values of moisture content which shows the soil have high potential of water retention (clayey) while some of the pits were also low varying between 25.0% and 15.9% respectively.

Table 4.1: Results of Moisture Content of Soils.

	TP_1		TP_2		TP_3		TP_4		TP_5	
Depth(m)	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0
M.C(%).	16.7	21.3	21.3	22.6	15.9	25.0	22.2	22.7	21.5	25.0

4.2 Specific Gravity

The summary result of specific gravity test are shown on table 4.2.1 from the results it varies in value which good number of the pits have within 2.32 and 2.47 while pit 1, 2, and 5 at 3.0m fall within 2.52 and 2.58, having average specific gravity of 2.45.

Table 4.2: Results of Specific Gravity of Soils.

	TP_1		TP_2		TP_3		TP_4		TP_5	
Depth(m)	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0

G_s	16.7	21.3	21.3	22.6	15.9	25.0	22.2	22.7	21.5	25.0

4.3 Particle Size Analysis

The particle size distributions of samples are show in Table 4.3 while the graphs were shown Fig. 4.3.1..

Table 4.3: Particle Size Distribution Results

	TP_1		TP_2		TP_3		TP_4		TP_5	
Depth(m)	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0
Gravel	99.7	100	99.8	100	88.9	83.4	99.8	99.5	98.8	100
Sand	67.5	66.4	74.8	75.0	39.3	30.5	74.5	77.7	66.2	70.8
Clay	37.2	51.9	64.6	62.7	32.3	22.0	64.7	63.6	59.0	61.96

From the table above of the result analysis, over 75% of the soils are fine material passing through 0.075mm sieve.

All the testing points have a minimum of 60% fines, fraction ranging between 51.9% and 64.7%, i.e. > 35% except TP_3 at 1.5m and 3.0m that has 22.0% and 32.3% as passing through 0.075mm sieve. It was deduced that the soil material of pits 1, 2, 4 and 5 constituted of clayed silt soil with good percentage of sand and finer fraction while only TP_3 is a lateritic soil because it has less than 35% of particles that passed through sieve 0.0075mm. Complete tables and graphs are provided in appendix 3.

4.4 Atterberg Limit Test

Appendix four shows the result of the five trial pits tested, liquid limit (LL), plastic limit (PL) , shrinkage limit (SL) and plasticity index (PI), carried out on all the samples while the value of the (LL) were obtained from the graphs shown in appendix four also. It was observed that in all the samples due to the result obtained from the (LL), (PL), and (PI), pits 1, 2, 4 and 5 soils are grouped into A-7-5 or A-7-6 classification i.e. clayey soil while pit 3 is classified to A-2-6 i.e. lateritic soil according to AASHTO classification system (1978).

4.5 Compaction

The results and graphs are shown in appendix five. From the results obtained, the analysis is as follow, the (OMC) and (MDD) were derived from graphs.

$TP_1 - TP_5$ are with a high (OMC) and lower (MDD) except TP_3 with less (OMC) and high (MDD) respectively.

4.6 Odometer Test

The results for the odometer test at the various depth and trial pit are presented in the tables below; At five different pressures; 52, 104, 280, 416 and 832 kN/m2 dial guage for the settlement of the soil was obtained and the time taken to undergo the settlement was also recorded and a graph of dial guage against the root of time was plotted (Appendix 4) and the time at 90% consolidation was obtained from the various graphs. The time at 90% consolidation was used to estimate the rate of consolidation.

Table 4.4: Odometer Test Result

	TP_1		TP_2		TP_3		TP_4		TP_5	
Depth(m)	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0	1.5	3.0
Avg. C_v (cm^2/s)	2.76	5.00	5.12	3.79	3.31	10.94	5.43	5.75	6.44	6.03

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the result obtained from the tests carried out (particle size analysis, atterberg limit, and odometer test) I can draw the following conclusions;

1. Most of the soils obtained from the trial pits at various depths are of clayey material because greater than 35% of the soil passed through the 0.075mm sieve and only the soil obtained from trial pit 3 can be said to be a lateritic soil.
2. It can also be said that the soils from the trial pits have moderate moisture content. The moisture content ranges from 15.9% to 25%.
3. The soils from the various trial pits have relatively low rate of consolidation, with average coefficient of consolidation C_v from each trial pit ranging from 2.76 cm^2/s to 10.94 cm^2/s.
4. Soils from trial pits 1, 2, 4 and 5 soils are grouped into A-7-5 or A-7-6 classification i.e. clayey soil while pit 3 is classified to A-2-6 i.e. lateritic soil according to AASHTO classification system.

4.2 Recommendation

For future construction of buildings or road the area with low moisture contents and high rate of settlement should be carefully considered as these locations to be either stabilized or compacted before it can become suitable for use. When considering the choice of stabilization method to use, the dominant kind of soil (clayey) present on the campus should be taken into consideration as not all stabilizing agents are suitable for cohesive soil.

REFERNCES

Adekanmi JS, Adebayo VB. 2016. Moisture-Density Relationship of Selected Clay Soils in Ekiti State, Nigeria. EJGE vol. 21(16):5421 -5427.

Afolagboye OL, Talabi AO. 2013. Consolidation properties of compacted lateritic soil stabilized with tyre ash. J. Eng. Manuf. Technol.vol. 1 :36-44.

Aladejana JA, Talabi AO, Oke SA, Oyelami AC. 2015. Geophysical and Geotechnical Investigations of a proposed Site for Afijio Local Government Stadium Ilora, Southwestern Nigeria. IJSRP vol.5 (7):1 -7.

Adetoro AE, Samuel AI. 2015. Effects of locally available additives on geotechnical properties of Ijero local government soil, Ekiti state, Nigeria. JMEST vol. 2(11):2994-2997.

Adeyemi EA, Joseph OA. 2015. Comparative analyses of ekiti state soil stabilized with different additives. AJST vol.06 (12):2054-2058

Alemayehu, Teferra and Mesfin, Leikun 2009. Soil Mechanics. AAU press, 2nd ed. Addis Ababa.

Amer Ali, Al-Rawas, and Mattheus, F.A.G. 2006. Expansive Soils. Taylor & Francis, London, UK

Arora, K.R. 1993, „Soil mechanics and foundation engineering", Standard Publisher Distributors, New Delhi. Pp. 114-116.

ASTM D698-91, 1995, "Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort 12,400 $ft - lbf/ft^3$ 600 KN/m^2" *Annual Book of ASTM Standards*, Vol. 04.08, ASTM International, West Conshohocken, PA, pp. 69–76.

Atsbeha N. 2012. Prediction of compaction characteristics from atterberg limits for fine-grained soils, pp. 3-30.

Aysen, A. (2002). Soil mechanics. The University of Queensland, Australia. Pp. 219-225.

Boesse S, Ocan O. 1992. Geology and evolution of the Ife-Ilesha Schist belt, southwestern Nigeria. In Benin-Nigeria Geotraverse. International Meeting on the Proterozoic Geology and Tectonics of High Grade Terrain. IGCP 215, pp. 123-129.

Blotz, LR, Benson C & Boutwell, G. 1998. Estimating Optimum Water Content and Maximum Dry Unit Weight for Compacted Clays. *J. Geotech. Engrg.*, Vol. 124 (3): pp. 907-912.

Bowels J.E. 2007. Foundation engineering analysis and design" The McGraw-Hill Companies, Inc, pp. 65.

Crawford CB. 1986. State of the art: Evaluation and interpretation of soil consolidation tests, consolidation of soils: Testing and evaluation, ASTM STP 892. R.N Yong and F.C.

Das, B.M., 1997,„Advanced soil mechanics", Taylor & Francis, Philadelphia. Pp. 234-240.

Dessalegn Sissay (2003). Computation of Soil Compressibility using Tangent Modulus approach.

Donaghe, RT & Torrey VH. 1994. A Compaction Test Method for Soil-Rock Mixtures in Which Equipment Size Effect Are Minimized. *Geotech. Test. J.*, Vol. 17: pp. 363–370

Eberemu, Adrian O. 2011. Consolidation Properties of Compacted Lateritic Soil Treated with RHA", Geomaterials, 1, 70-78.

Gidigasu MD. 1976. Laterite soil engineering pedo-genesis and engineering principles. Amsterdam Elsevier Scientific, New York, p. 554.

Garga, V K. & Madureira, C T. 1985. Compaction Characteristics of River Terrace Gravel. *J.Geotech. Engrg.*, Vol. 111 (5): pp. 987–1007.

Horpibulsuk, S, Katkan W & Apichatvullop, A. 2008. An Approach for Assessment of Compaction Curves of Fine-Grained Soils at Various Energies Using a One Point Test. *Soils Found.*, Vol. 48: pp. 115–126. IACSIT International Journal of Engineering and Technology. 2012. Vol. 4 (5): pp. 15.

Imhoff S, Da Silva AP & Fallow D. 2004. Susceptibility to compaction, load support capacity, and soil compressibility of Hapludox. Soil Sci. Society of America J. 68: 17-24.

Jeng, Y.S., Strohm, W.E., 1 976. Prediction of the shear strength and compaction characteristics of compacted fine grained cohesive soils. United States Waterways Experiment Station, Soil and Pavement Laboratory, Vicksburg, Miss.

Józs V; Empirical Correlations of Overconsolidation Ratio, Coefficient of Earth Pressure at Rest and Undrained Strength. Second Conference of Junior Researchers in Civil Engineering, Budapest University of Technology and Economics, Budapest, Hungary, 2013: 88-92.

Jumikis, A.R, 1984, „Soil Mechanics", Robert E. Krieger Publishing Company, Florida. Pp. 110-116.

Lav MA & Ansal AM. 2001. Regression Analysis of Soil Compressibility", Tr Journal of Engineering and Environmental Sciences, Vol. 25:101-109.

Michael IO, Martins OO, John SO. 2008. Geophysical investigation of Road failure in the basement complex areas of south western Nigeria. Res. J. Applied Sci., vol. 3(2): 103-112. Military Soils Engineering, 1997. Field Manual 5-410, Department of the Army, Washington DC. Pp. 114-120.

Mittal S, Shukla JP. 2009. Soil Testing for Engineers, 5th ed. Khanna Publishers, New Delhi, India. Pp. 75-105

Murthy, V.N.S. 2007. Geotechnical Engineering, Principles and Practice of Soil Mechanics and Foundation Engineering. Marcel Dekker, INC, New York. Pp. 222-254

Nagaraj HB. 2000. Prediction of Engineering properties of Fine-Grained Soils from their index properties. PhD Thesis, Indian institute of Science, India, pp. 225.

Nigel RW. 2009. The State of Compaction: the Effect of Compaction on Soil Properties and Slope Fill Performance. Maunsell Geotechnical Services, Hong Kong, pp. 45-65.

Nwaiwu CMO, Nuhu I; Evaluation and Prediction of the Swelling Characteristics of Nigerian Black Clays. Geotechnical and Geological Engineering, 2006; 24(1): 45-56.

Oyawoye, M.O. (1964).The geology of the basement complex. Journal of Nigeria Mining, Geology and Metallurgy. Volume 1, pp. 87-102

Osman EAM. 2006. Research paper on New Reinforced Earth Mat foundation over Soft Clay, Dept. of Civil Engineering, Faculty of Engineering, University of Minia, Minia, Egypt, pp. 52-74.

Puri, Nitish. 2012. Compressibility Characteristics of Clay Treated with Industrial Waste. M.Tech Dissertation, NIT Kurukshetra, India.

Rahaman, M.A. (1976). Review of the basement geology of southwestern Nigeria, In: KOGBE, C.A. (ed.)Geology of Nigeria. Elizabethan Publishing Company. Lagos, 41 -58.

Rahaman, M.A. (1988). Recent Advances in study of the Basement Complex of Nigeria Precambrian geology of Nigeria. In: OLUYIDE, P.O. ET AL., (eds) Precambrian Geology of Nigeria. Geological Survey of Nigeria Publication, Kaduna, 11 -43.

Schiffman RL, Pane V & Gibson RE. 1984. The theory of one-dimensional consolidation of saturated clays IV. An Overview of non-linear finite strain sedimentation and consolidation. Proceedings, ASCE Symposium on Sedimentation/Consolidation Models, San Francisco, pp. 1-29.

Robert, Parsons L , Derek, H &Foster. 2000. Compaction and Settlement of Existing Embankments. University of Kansas, Lawrence, Kansas.

Singh, H.P. 2012.Improvement in CBR Value of Soil Reinforced with Coir fiber. 3rd International Conference on Natural Polymer (ICNP 2012), held at Mahatma Gandhi University Kottayam, Kerala, India. 26-27 October Paper No-118.

Strokova L; Effect of the Overconsolidation Ratio of Soils in Surface Settlements due to Tunneling. Sciences in Cold and Arid Regions, 2013; 5(5): 637-643.

Taha MR, Ahmed J & Asmirza S. 2000. One-Dimensional Consolidation of Kelang Clay, Pertanika Journal of Science & Technology, Vol. 8 (1): pp. 19-29.

Talabi AO & Tijani MN. 2011 . Assessment of groundwater quality in parts of the basement complex terrain of South-western Nigeria. GQ10: Groundwater quality management in a rapidly changing world (Proceedings of the 7th International Groundwater Quality Conference held in Zurich, Switzerland, 13–18 June 2010). IAHS Publ 342.

Terzaghi, K. 1943. Theoretical Soil Mechanics. John Wiley and Sons INK. London Uk. Pp. 213-218.

Yesim Gurtugand Sridharan, A. 2004. Compaction Behavior and Prediction of its Characteristics of Fine Grained Soils with Particular Reference to Compaction Energy. Journal of Soil and Foundation, Japanese geotechnical Society.

APPENDIX ONE

TRIAL PIT 1

	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	1.3	0.26	99.7	0	0	100
2.36	13.3	2.66	97.1	5.6	1.12	98.9
1.18	29.8	5.96	91.1	27.3	5.46	98.4
600	44.7	8.94	82.2	45.4	9.08	84.3
300	49.2	9.84	72.3	55.1	11.02	73.3
150	48.2	9.64	62.7	69.4	13.88	59.4
75	27.3	5.46	57.2	37.6	7.52	51.9

TRIAL PIT 2

	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	1.2	0.24	99.8	0	0	100
2.36	8.2	1.64	98.1	1.4	0.28	99.7
1.18	17.8	3.56	94.6	10.6	2.12	97.6
600	35.6	7.12	87.4	32.2	6.44	91.2
300	43.0	8.60	78.8	55.9	11.18	80.0
150	40.9	8.18	70.7	49.7	9.94	70.0
75	29.7	5.94	64.7	36.7	7.34	62.7

TRIAL PIT 3

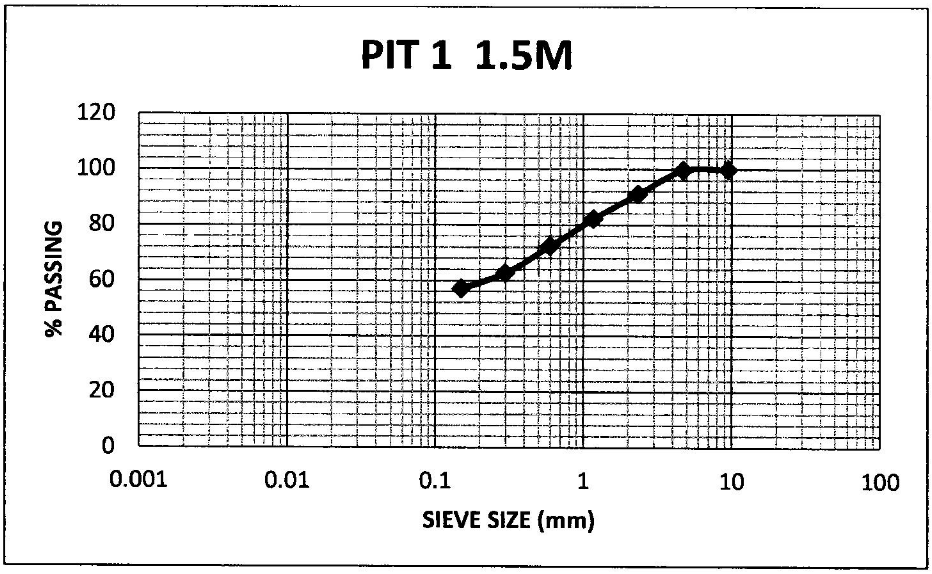
	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	55.5	11.1	88.9	83.0	16.60	83.4
4.75	89.4	17.88	71.0	70.9	14.18	69.2
2.36	43.9	8.78	62.2	37.3	7.46	61.8
1.18	35.1	7.02	55.2	44.0	8.80	53.0
600	33.1	6.62	48.6	47.6	9.52	43.4
300	33.1	6.62	42.0	46.0	9.20	34.2
150	27.6	5.52	36.5	37.5	7.50	26.7
75	20.9	4.18	32.3	23.7	4.74	22.0

TRIAL PIT 4

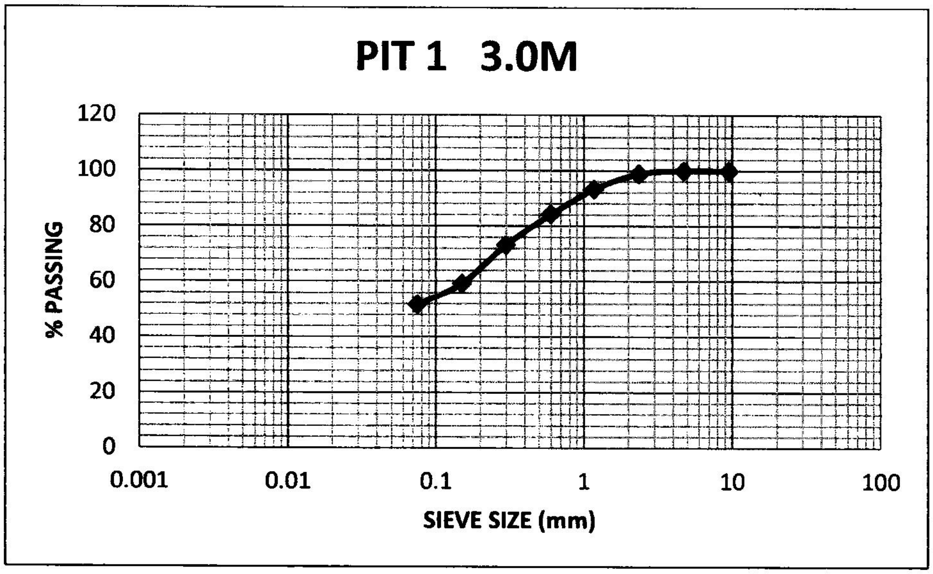
	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing(%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	2.6	0.52	99.5
4.75	1.1	0.22	99.8	4.8	0.96	98.5
2.36	8.9	1.78	98.0	8.1	1.62	96.9
1.18	19.1	3.82	94.2	16.4	3.28	93.6
600	35.5	7.1	87.1	32.6	6.52	87.1
300	42.7	8.54	78.5	40.0	8.00	79.1
150	40.2	8.04	70.5	44.6	8.92	70.2
75	29.1	5.82	64.7	33.0	6.6	63.6

TRIAL PIT 5

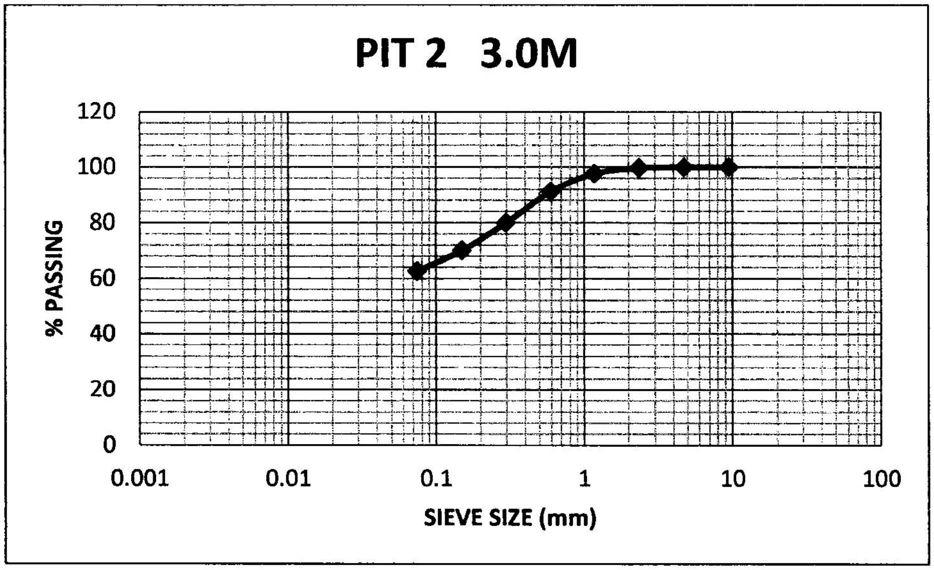
	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	6.0	1.2	98.8	0	0	100
2.36	22.9	4.58	99.2	8.6	1.72	98.3
1.18	36.7	7.34	86.9	27.9	5.58	92.7
600	44.9	8.98	77.9	45.9	9.18	83.5
300	42.3	8.46	69.4	43.8	8.76	74.76
150	32.4	6.48	63.0	39.4	7.88	66.9
75	19.7	3.94	59.0	24.6	4.92	61.96

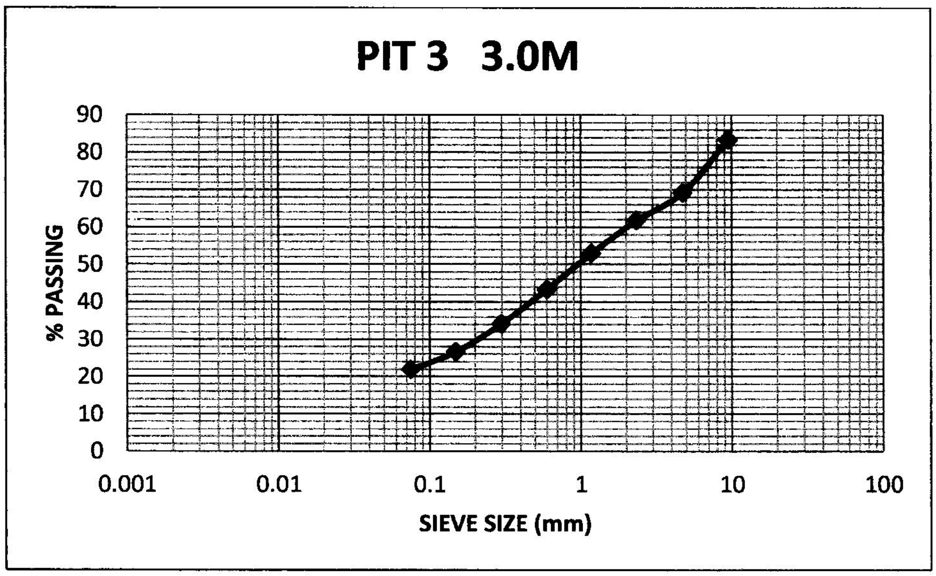


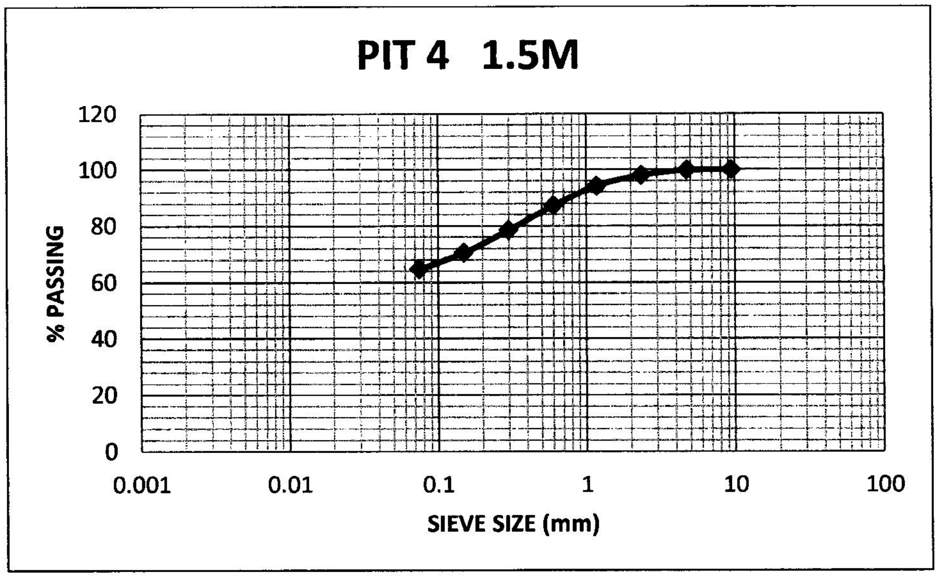
PIT 1 1.5M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

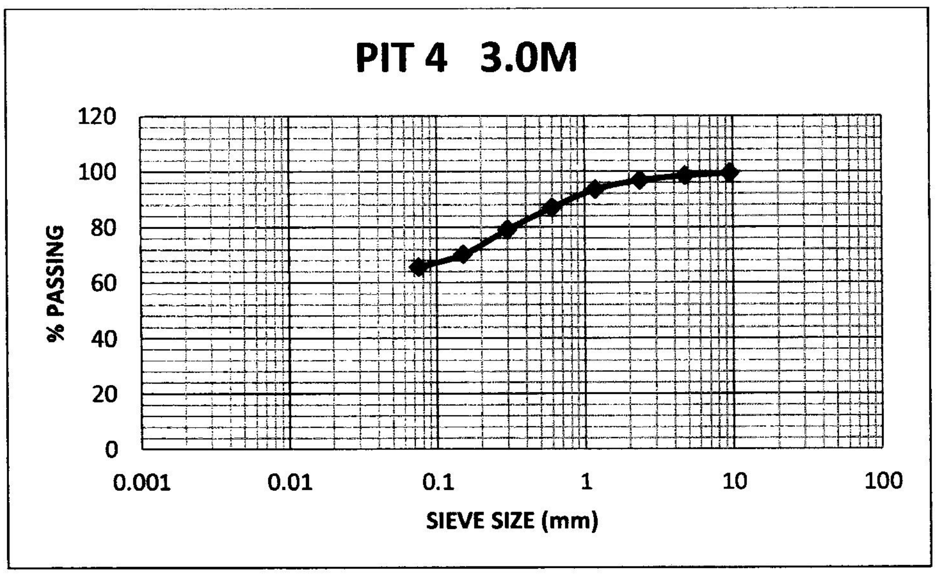


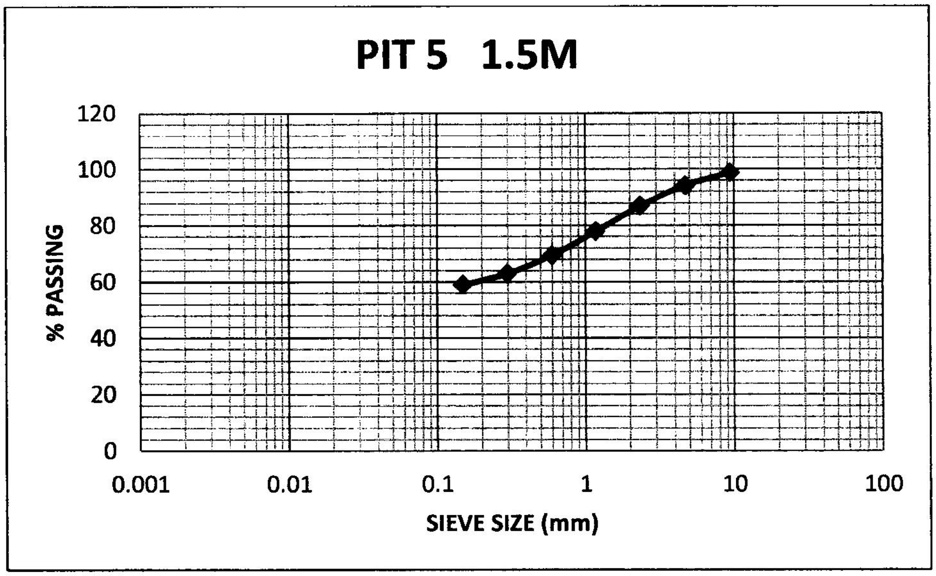
PIT 1 3.0M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

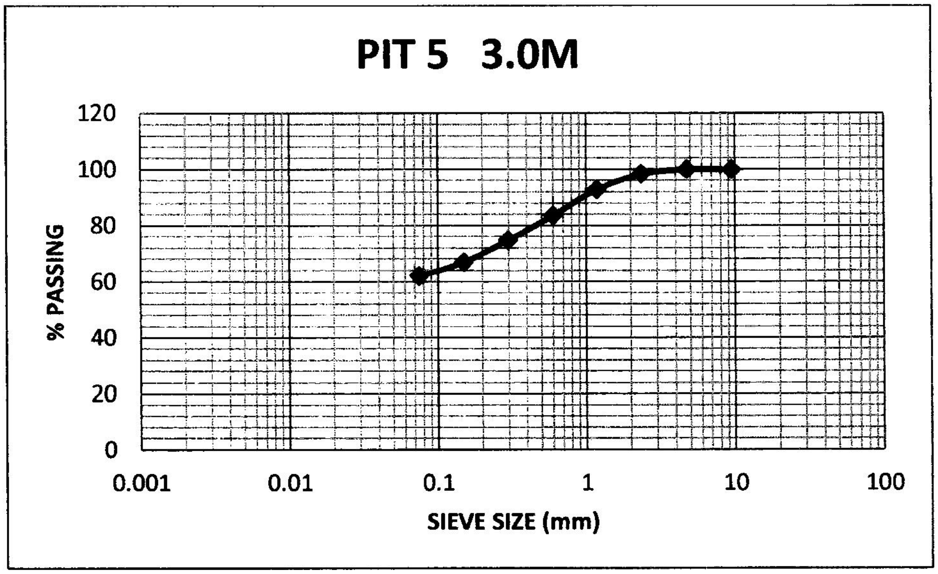

PIT 2 3.0M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


PIT 3 3.0M
% PASSING
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


PIT 4 1.5M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


PIT 4 3.0M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


PIT 5 1.5M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


PIT 5 3.0M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

APPENDIX TWO

SPECIFIC GRAVITY TEST

PIT (1) 3.0m

A. 45.0-18.8/(111.7-18.8)-(126.9-45.0) =26.2/11.0 =2.38

B. 47.1-23.9/(78.6-23.9)- (92.3-47.1)=23.2/9.5 = 2.44

Average= 2.41

PIT (2) 3.0m

A. 47.2-21.5/(72.8-21.5) –(87.4-47.2) = 25.7/11.1 = 2,32

B. 47.2-27.3/(88.7-27.3)- (102.4-51.2) = 23.9/10.0 =2.39

Average = 2.36

PIT (4) 1.5m

A. 47.0-20.8/(93.5-20.8)-(108.7-47.0) = 26.3/11 = 2.38

B.

PIT (4) 3.0m

A. 38.2-16.3/(93.6-20.9)-(106.1-47.2)=21.9/9.4 = 2.33

B. 47.2-20.9/(93.4-20.9)-(108.6-47.2) = 26.3/11.1 = 2.37

Average = 2.35

PIT (5) 1.5m

A. 41.1-15.9/(84.0-15.9)-(98.6-41.1) = 25.2/10.6 = 2.38

B 49.9-20.4/(80.8-20.4)-(97.8-49.9) = 29.5/12.3 = 2.36

Average = 2.37

PIT (5) 3.0m

A. 39.3-20.5/(72.8-20.5)-(84.0-39.5) = 18.8/7.6 = 2.47

B. 44.5-18.9/(75.5-18.9)-(90.8-44.5) = 25.7/10.4 = 2.47

Average =2.47

APPENDIX THREE

PIT 1 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
45	34	21	11		
H1	H2	H3	H4	H5	H6
8.3	18.2	12.9	11.0	9.4	16.1
25.9	43.9	41.1	39.2	19.2	28.7
20.2	35.2	31.0	28.7	16.9	25.9
5.7	8.7	10.1	10.5	2.3	2.8
11.9	17.0	18.1	17.7	7.5	12.6
47.9	51.2	55.8	59.3	30.7	22.2

Shrinkage Limit = 10.7%

PIT 1 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
48	36	21	12		
G1	G2	G3	G4	G5	G6
19.9	20.2	10.3	15.5	10.8	8.8
40.7	48.9	37.1	48.5	26.8	22.3
34.0	35.1	27.4	35.9	23.1	19.1
6.7	7.8	9.7	12.6	3.7	3.1
14.1	14.7	12.1	20.4	12.3	13.5
47.5	52.1	56.7	61.8	30.1	13.5

Shrinkage Limit = 10.7%

PIT 2 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
48	36	22	11		
F1	F2	F3	F4	F5	F6
14.1	11.2	15.8	22.0	20.2	15.5
34.2	32.7	38.7	56.2	37.2	32.7
28.6	26.2	31.4	44.3	33.1	28.6
5.6	6.5	7.3	11.9	4.1	4.1
14.5	15.2	15.6	22.3	12.9	13.1
38.6	42.8	46.8	53.4	31.8	31.3

Shrinkage Limit = 9.3%

PIT 2 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
45	32	21	10		
E1	E2	E3	E4	E5	E6
19.3	16.3	22.1	10.3	10.5	9.2
42.3	42.6	53.8	42.6	25.4	27.2
35.4	33.8	42.8	30.9	21.9	23.4
6.9	8.8	11.0	11.7	4.0	4.3
16.1	17.5	20.7	20.6	11.4	14.2
42.9	50.3	53.1	56.8	35.1	30.3

Shrinkage Limit = 8.6%

PIT 3 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
46	34	22	12		
A1	A2	A3	A4	A5	A6
10.4	14.9	10.7	13.0	14.2	22.3
33.0	37.2	39.2	43.8	30.1	41.2
26.3	30.1	29.9	33.2	26.6	36.8
6.7	7.1	9.3	10.6	3.5	4.4
15.9	15.2	19.2	20.2	12.4	14.5
42.1	46.7	48.4	52.3	28.2	30.3

Shrinkage Limit = 9.3%

PIT 3 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
44	32	21	11		
C1	C2	C3	C4	C5	C6
10.0	9.0	13.1	12.7	9.2	14.9
31.6	35.6	43.4	48.2	24.7	32.7
25.1	27.2	33.7	36.3	21.2	28.7
6.5	8.4	9.7	11.9	3.5	4.0
15.1	18.2	20.6	23.6	12.0	13.8
43.0	46.1	47.1	50.4	29.2	29.0

Shrinkage Limit = 7.9%

PIT 4 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
46	30	21	11		
J1	J2	J3	J4	J5	J6
7.2	9.1	8.8	18.6	19.6	18.6
32.7	31.9	36.5	49.9	38.8	34.1
24.7	24.0	26.5	38.2	34.2	30.3
8.5	7.7	10.0	11.7	4.6	3.8
17	14.7	17.7	19.6	14.6	11.7
				31.5	32.3

Shrinkage Limit = 10.0%

PIT 4 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
47	34	22	12		
I1	I2	I3	I4	I5	I6
9.8	12.2	8.8	10.4	9.5	10.7
30.9	37.6	34.6	40.2	23.4	21.2
24.6	28.7	25.1	28.7	19.8	18.5
6.3	8.9	9.5	11.5	3.6	2.7
14.8	19.2	16.3	18.3	10.3	7.8
42.6	46.4	58.3	62.8	35.0	34.6

Shrinkage Limit = 11.4%

PIT 5 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
48	36	23	13		
D1	D2	D3	D4	D5	D6
7.3	7.4	11.4	9.1	13.7	13.0
32.3	33.5	37.5	45.6	30.3	26.2
24.5	24.8	28.5	32.6	26.5	23.2
7.0	8.7	9.0	13.0	3.8	3.0
17.2	17.4	17.1	23.5	12.8	10.2
40.7	51.0	52.6	55.3	29.7	29.4

Shrinkage Limit = 9.3%

PIT 5 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
43	30	22	14		
B1	B2	B3	B4	B5	B5
18.0	11.6	9.0	9.5	10.4	8.2
5.6	32.3	35.1	39.5	24.7	21.1
5.8	7.2	9.4	11.3	3.3	3.0
11.8	13.5	16.7	18.3	11.0	9.9
49.2	53.3	56.3	61.7	30.0	30.0

Shrinkage Limit = 10.7%

COMPACTION TEST

PIT 2 (3.0m)

Weight of mould + soil wet	5600	5800	5900	59750
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1800	2000	2100	1950
Bulk density	1.80	2.00	2.10	1.95
Can no	AB	BC	EF	GF
Can weight	14.2	11.6	11.3	9.5
Weight of can + soil wet	48.5	47.5	53.9	50.1
Weight of can + soil dry	44.4	41.2	45.4	40.7
Weight of water	5.1	6.3	8.5	9.4
M.C	29.2	29.6	34.1	31.2
Dry density kg/m^3	17.5	21.3	25.0	30.1

PIT 5 (3.0M)

Weight of mould + soil wet	5500	5650	5850	5900	5850
Weight of empty mould	3800	3800	3800	3800	3800
Weight of wet soil	1700	1800	2050	2100	2050
Bulk density	1.70	1.80	2.50	2.10	2.50
Can no	AB1	BC1	CE1	DF1	GH1
Can weight	12.8	15.2	11.4	11.2	9.8
Weight of can + soil wet	65.6	71`.0	64.8	70.6	63.7
Weight of can + soil dry	59.9	63.6	56.0	60.3	51.9
Weight of water	5.7	7.4	8.8	10.3	11.8
M.c	12.1	15.5	19.7	21.0	28.0
Dry density kg/m^3	1.52	1.60	1.75	1.74	1.60

PIT 1 (3.0M)

Weight of mould + soil wet	5600	5750	5900	5800
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1800	1950	2100	2000
Bulk density	1.80	1.95	2.10	2.00
Can no	K1	K2	K3	K4
Can weight	27.4	22.1	13.9	11.7

Weight of can + soil wet	82.7	73.0	68.2	58.4
Weight of can + soil dry	77.6	65.7	59.4	48.9
Weight of water	6.1	7.3	8.8	9.5
M.c	12.2	16.7	19.2	22.5
Dry density kg/m^3	1.6	1.67	1.76	1.59

PIT 4 (3.0m)

Weight of mould + soil wet	5550	5700	5900	5800
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1750	1900	2100	2000
Bulk density	1.75	1.90	2.10	2.00
Can no	G1	G2	K3	K4
Can weight	15.4	9.7	13.9	11.7
Weight of can + soil wet	62.8	56.8	68.2	58.4
Weight of can + soil dry	56.5	49.0	59.4	48.9
Weight of water	6.3	7.3	8.8	9.5
M.c	15.3	16.7	19.2	22.5
Dry density kg/m^3	1.07	1.59	1.66	1.52

PIT 3 (1.5m)

Weight of mould + soil wet	4650	4850	5200	5150
Weight of empty mould	3100	3100	3100	3100
Weight of wet soil	1550	1700	2100	2050
Bulk density	1.55	1.70	2.10	2.05
Can no	H1	H2	H3	H4
Can weight	20.0	21.4	12.0	11.8
Weight of can + soil wet	93.7	70.4	58.2	56.6
Weight of can + soil dry	88.8	65.3	51.5	49.2
Weight of water	4.9	5.1	6.7	7.4
M.c	7.1	11.6	16.9	19.8
Dry density kg/m^3	1.45	1.52	1.79	1.71

PIT 3 (3.0m)

Weight of mould + soil wet	4800	5000	5250	5150
Weight of empty mould	3100	3100	3100	3100
Weight of wet soil	1700	1900	2150	2050
Bulk density	1.70	1.90	2.15	2.05
Can no	J1	J2	J3	J4
Can weight	11.2	10.3	11.7	9.5
Weight of can + soil wet	78.7	66.7	61.2	88.8
Weight of can + soil dry	71.9	59.6	52.9	74.2
Weight of water	6.8	7.1	8.3	14.6
M.C	11.2	14.4	19.9	22.6
Dry density kg/m^3.	1.53	1.66	1.79	1.67

PIT 5 (1.5m)

Weight of mould + soil wet	4850	5050	5200	5100
Weight of. empty mould	3100	3100	3100	3100
Weight of wet soil	1750	1950	2100	2000
Bulk density	1.75	1.95	2.10	2.00
Can no	I1	I2	I3	I4
Can weight	10.2	8.2	12.1	14.3
Weight of can + soil wet	63.3	54.0	55.7	74.5
Weight of can + soil dry	58.4	48.7	48.9	63.6
Weight of water	5.2	6.0	6.8	10.9
M.c	11.3	14.8	18.5	22.1
Dry density kg/m^3	1.57	1.69	1.77	1.64

APPENDIX FOUR

RAW CONSOLIDATION TEST RESULT FOR PIT 1 (1.5m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	54.0	90.5	148.0	210.5
1.0	0.5	50	83.5	137	199.5	295.0
4.0	1.0	52	86.0	139.5	202.5	300.0
9.00	3.0	52.5	87.0	141.0	203.5	301.5
16.00	4.0	53.0	87.5	142.0	206.0	302.5
25.00	5.0	53.5	88.5	142.5	207.0	303.5
36.00	6.0	53.5	89.0	143.5	207.5	---
64.00	8.0	54.0	89.5	144.0	208.5	305.0
81.00	9.0	54.0	90.0	144.5	210.0	305.5
100.00	10.0	---	90.5	145.0	210.5	305.5
121.00	11.0	---	---	145.0	---	---
1444	38.0	---	---	148.0	---	---

Table Showing the Consolidation Coefficients of Pit 1 (1.5m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	54.0	90.5	148.0	210.5
FINAL DIAL READING	54.0	90.5	148.0	210.5	305.5
INITIAL THICKNESS (mm)	20.000	19.460	19.095	18.520	17.895
CHANGE IN THICKNESS (mm)	0.540	0.365	0.575	0.625	0.950
FINAL THICKNESS (mm)	19.460	19.095	18.520	17.895	16.945
AVERAGE THICKNESS (mm)	19.730	19.278	18.808	18.208	17.420
t_{90}(min)	2.56	4.41	10.24	4	8.41
C_V (cm^2/sec)	5.370x10^{-3}	3.0x10^{-3}	1.22x10^{-3}	2.93x10^{-3}	1.27x10^{-3}
e	0.489	0.449	0.422	0.379	0.333

Average Coefficient of consolidation (C_v) = 2.76x10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 1 (3.0m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	25.0	41.0	68.0	107.0
1.0	0.5	22.0	39.0	63.0	100.0	184.0
4.0	1.0	24.5	40.0	65.0	103.0	187.0
9.00	3.0	25.0	40.5	65.5	104.0	189.0
16.00	4.0	25.0	41.0	65.5	105.0	190.5
25.00	5.0	---	41.0	66.0	105.5	191.5
36.00	6.0	---	---	66.0	105.5	192.0
64.00	8.0	---	---	66.5	106.0	193.0
81.00	9.0	---	---	---	106.5	194.0
100.00	10.0	---	---	---	107.0	194.0
121.00	11.0	---	---	---	---	194.5
1444	38.0	---	---	68.0	---	---

Table Showing the Consolidation Coefficients of Pit 1 (3.0m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	25.0	41.0	68.0	107.0
FINAL DIAL READING	25.0	41.0	68.0	107.0	194.5
INITIAL THICKNESS (mm)	20.000	19.750	19.590	19.320	18.930
CHANGE IN THICKNESS (mm)	0.250	0.160	0.270	0.390	0.875
FINAL THICKNESS (mm)	19.750	19.590	19.320	18.930	18.055
AVERAGE THICKNESS (mm)	19.875	19.670	19.455	19.125	18.493
t_{90}(min)	4.41	2.25	4.00	3.24	1.44
C_V (cm^2/sec)	3.16×10^{-3}	6.08×10^{-3}	3.34×10^{-3}	4.00×10^{-3}	8.39×10^{-3}
e	0.612	0.592	0.579	0.557	0.525

Average Coefficient of consolidation (C_v) = 5.00×10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 2 (1.5m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	38.0	64.0	99.0	141.0
1.0	0.5	35	57.0	93.0	134.0	183.0
4.0	1.0	37.5	58.5	95.0	136.5	185.5
9.00	3.0	38.0	59.0	96.0	137.5	189.0
16.00	4.0	38.0	59.5	96.5	138.5	189.5
25.00	5.0	---	59.5	97.0	139.0	190.0
36.00	6.0	---	60.0	97.5	139.5	192.0
64.00	8.0	---	60.5	98.0	140.5	192.0
81.00	9.0	---	60.5	98.5	141.0	192.5
100.00	10.0	---	---	99.0	141.0	193.0
121.00	11.0	---	---	---	---	193.0
144.00	12.0	---	---	---	---	193.5
1444	38.0	---	64.0	---	---	---

Table Showing the Consolidation Coefficients of Pit 2 (1.5m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	38.0	64.0	99.0	141.0
FINAL DIAL READING	38.0	64.0	99.0	141.0	193.5
INITIAL THICKNESS (mm)	20.000	19.620	19.360	19.010	18.590
CHANGE IN THICKNESS (mm)	0.380	0.260	0.350	0.420	0.525
FINAL THICKNESS (mm)	19.620	19.360	19.010	18.590	18.065
AVERAGE THICKNESS (mm)	19.810	19.490	19.185	18.800	18.328
t_{90}(min)	2.25	9.00	4.00	4.41	1.00
C_V (cm^2/sec)	6.16×10^{-3}	1.49×10^{-3}	3.25×10^{-3}	2.83×10^{-3}	11.87×10^{-3}
$\ddot{e}$	0.657	0.626	0.604	0.575	0.540

Average Coefficient of consolidation (C_v) = 5.12×10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 2 (3.0m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	16.5	37.5	69.5	105.5
1.0	0.5	15.0	35.5	64.5	99.5	124.0
4.0	1.0	15.5	36.0	66.5	101.5	127.0
9.00	3.0	16.0	36.5	67.0	102.5	128.5
16.00	4.0	16.5	37.0	67.5	103.0	129.5
25.00	5.0	16.5	37.0	68.0	103.5	130.5
36.00	6.0	16.5	37.5	69.0	104.0	131.0
64.00	8.0	---	37.5	69.0	104.5	---
81.00	9.0	---	---	---	105.0	---
100.00	10.0	---	---	---	105.0	---
484.00	22.0	---	---	---	105.5	139.5
529.00	23.0	---	---	---	105.5	---
1225.00	35.0	---	---	69.5	---	141.0

Table Showing the Consolidation Coefficients of Pit 2 (3.0m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	16.5	37.5	69.5	105.5
FINAL DIAL READING	16.5	37.5	69.5	105.5	141.0
INITIAL THICKNESS (mm)	20.000	19.835	19.625	19.305	18.945
CHANGE IN THICKNESS (mm)	0.165	0.210	0.320	0.360	0.355
FINAL THICKNESS (mm)	19.835	19.625	19.305	18.945	18.590
AVERAGE THICKNESS (mm)	19.918	19.730	19.465	19.125	18.768
t_{90}(min)	0.903	1.00	4.00	3.24	1.00
C_V (cm^2/sec)	15.52×10^{-3}	1.02×10^{-3}	1.02×10^{-3}	1.34×10^{-3}	1.07×10^{-3}
E	0.541	0.528	0.512	0.487	0.460

Average Coefficient of consolidation (C_v) = 3.79×10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT OF PIT 3 (1.5m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m²	104 KN/m²	208 KN/m²	416 KN/m²	832 KN/m²
0	0	---	38	66	86	115
1.0	1.0	28	54	72	99	126
4.00	2.0	31	59.5	75	107	134
9.00	3.0	33	62	78	111	139
16.00	4.0	35	63	81	112.5	141.5
25.00	5.0	35.5	64	82	113	143
36.00	6.0	36	64.5	83	113	144
64.00	8.0	36.5	---	---	---	146
81.00	9.0	---	---	---	---	146.5
100.00	10.0	38.0	---	---	---	147
121.00	11.0	38.0	---	---	---	147
1024.0	32.0	---	66	86	115	150

Table Showing the Consolidation Coefficients of Pit 3 (1.5m) Soil Sample

PRESSURE (KN/m²)	52	104	208	416	832
INITIAL DIAL READING	**0**	38	66	86	115
FINAL DIAL READING	**38**	66	86	115	150
INITIAL THICKNESS (mm)	**20.00**	19.62	19.34	19.14	18.85
CHANGE IN THICKNESS (mm)	**0.38**	0.28	0.20	0.29	0.35
FINAL THICKNESS (mm)	**19.62**	19.34	19.14	18.85	18.50
AVERAGE THICKNESS (mm)	**19.81**	19.48	19.24	19.00	18.68
t_{90}(min)	**1.96**	25	1.21	17.64	3.0
C_V (cm²/sec)	**6.7×10^{-3}**	2.42×10^{-3}	3.62×10^{-3}	1.94×10^{-3}	1.87×10^{-3}
e	**0.488**	0.460	0.439	0.424	0.403

Average Coefficient of consolidation (C_v) = $3.31.00\times10^{-3}$ cm²/sec

RAW CONSOLIDATION TEST RESULT OF PIT 3 (3.0m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m²	104 KN/m²	208 KN/m²	416 KN/m²	832 KN/m²
0	0	---	54	103	157	190
1.0	1.0	40	94	139	175	203
4.00	2.0	48	100	147	181.5	208.5
9.00	3.0	50	100	150	184.5	211
16.00	4.0	51	---	151	186	212
25.00	5.0	51.5	---	152	187	213
36.00	6.0	52	---	153	187.5	213.5
64.00	8.0	---	---	---	188	214
81.00	9.0	---	---	---	---	214
100.00	10.0	---	---	---	---	---
121.00	11.0	---	---	---	---	---
1024.0	32.0	54	103	157	190	216.5

Table Showing the Consolidation Coefficients of Pit 3 (3.0m) Soil Sample

PRESSURE (KN/m²)	52	104	208	416	832
INITIAL DIAL READING	0	54	103	157	190
FINAL DIAL READING	54	103	157	190	216.5
INITIAL THICKNESS (mm)	20	19.46	18.97	18.43	18.10
CHANGE IN THICKNESS (mm)	0.54	0.49	0.54	0.33	0.27
FINAL THICKNESS (mm)	19.46	18.97	18.43	18.10	17.83
AVERAGE THICKNESS (mm)	19.73	19.22	18.70	18.27	17.97
t_{90}	3.80	2.16	11.49	25.00	8.53
C_V (cm²/sec)	$3.62x10^{-3}$	$6.04x10^{-3}$	$1.08x10^{-3}$	$3.57x10^{-3}$	$1.34x10^{-3}$
E	0.566	0.524	0.486	0.443	0.417

Average coefficient of consolidation (C_v) = $10.94x10^{-3}$cm²/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 4 (1.5m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	32.5	56.0	85.0	138.0
1.0	0.5	30	50.0	81.0	131.0	179.0
4.0	1.0	31.5	51.0	82.5	132.5	183.0
9.00	3.0	32.0	51.5	83.5	134.5	184.5
16.00	4.0	32.0	52.0	84.0	135.5	185.0
25.00	5.0	32.5	52.5	84.0	136.0	185.5
36.00	6.0	---	52.5	84.5	136.5	187.0
64.00	8.0	---	---	85.0	137.5	188.0
81.00	9.0	---	---	85.0	138.0	188.0
100.00	10.0	---	---	---	138.0	188.5
121.00	11.0	---	---	---	---	188.5
144.00	12.0	---	---	---	---	---
1444	38.0	---	56.0	---	---	---

Table Showing the Consolidation Coefficients of Pit 4 (1.5m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	32.5	56.0	85.0	138.0
FINAL DIAL READING	32.5	56.0	85.0	138.0	188.5
INITIAL THICKNESS (mm)	20.000	19.675	19.440	19.150	18.620
CHANGE IN THICKNESS (mm)	0.325	0.235	0.290	0.530	0.505
FINAL THICKNESS (mm)	19.675	19.440	19.150	18.620	18.115
AVERAGE THICKNESS (mm)	19.838	19.558	19.295	18.885	18.368
t_{90}(min)	1.21.	3.0	4.41	9	1
C_V (cm^2/sec)	7.70×10^{-3}	5.49×10^{-3}	2.57×10^{-3}	6.34×10^{-3}	5.07×10^{-3}
e	0.618	0.592	0.573	0.549	0.507

Average Coefficient of consolidation (C_v) = 5.43×10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 4 (3.0m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	76.5	113.5	174.5	228.0
1.0	0.5	70.0	107.0	165.0	218.5	283.0
4.0	1.0	72.0	109.0	167.5	222.5	287.0
9.00	3.0	73.0	110.0	169.5	224.0	289.0
16.00	4.0	73.5	111.0	170.0	225.0	290.0
25.00	5.0	74.0	111.5	171.0	225.5	291.0
36.00	6.0	74.5	112.0	171.5	225.5	291.5
64.00	8.0	76.0	113.0	172.0	227.0	292.0
81.00	9.0	76.5	113.5	172.0	227.5	293.0
100.00	10.0	76.5	---	172.5	227.5	293.0
121.00	11.0	---	---	---	228.0	293.5
144.00	12.0	---	---	---	---	293.5
1444	38.0	---	---	174.5	---	---

Table Showing the Consolidation Coefficients of Pit 4 (3.0m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	76.5	113.5	174.5	228.0
FINAL DIAL READING	76.5	113.5	174.5	228.0	293.5
INITIAL THICKNESS (mm)	20.000	19.235	18.865	18.255	17.720
CHANGE IN THICKNESS (mm)	0.765	0.370	0.610	0.535	0.655
FINAL THICKNESS (mm)	19.235	18.865	18.255	17.720	17.065
AVERAGE THICKNESS (mm)	19.618	19.050	18.560	17.988	17.393
t_{90}(min)	4.00	2.56	2.25	9.22	9
C_V (cm^2/sec)	4.23x10^{-3}	7.04x10^{-3}	8.09x10^{-3}	2.34x10^{-3}	7.07x10^{-3}
e	0.528	0.469	0.441	0.395	0.354

Average Coefficient of consolidation (C_v) = 5.75x10^{-3} cm^2/sec

RAW CONSOLIDATION TEST RESULT FOR PIT 5 (1.5m) SOIL SAMPLE

Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m^2	104 KN/m^2	208 KN/m^2	416 KN/m^2	832 KN/m^2
0	0	---	21.5	45.5	92.0	135.0
1.0	0.5	18.5	40.0	82.0	127.0	188.0
4.0	1.0	19.5	42.0	84.0	130.0	193.0
9.00	3.0	20.0	43.0	84.5	131.5	194.0
16.00	4.0	20.5	43.5	86.0	133.0	195.5
25.00	5.0	21.0	44.0	86.5	134.0	196.0
36.00	6.0	21.5	44.0	87.0	134.5	197.0
64.00	8.0	21.5	45.0	87.5	135.0	198.0
81.00	9.0	---	45.5	88.5	---	198.5
100.00	10.0	---	45.5	89.5	---	199.0
121.00	11.0	---	---	89.5	---	199.5
196.00	14.0	---	---	---	---	199.5
1444	38.0	---	---	92.0	---	---

Table Showing the Consolidation Coefficients of Pit 5 (1.5m) Soil Sample

PRESSURE (KN/m^2)	52	104	208	416	832
INITIAL DIAL READING	0	21.5	45.5	92.0	135.0
FINAL DIAL READING	21.5	45.5	92.0	135.0	199.5
INITIAL THICKNESS (mm)	20.000	19.785	19.545	19.080	18.650
CHANGE IN THICKNESS (mm)	0.215	0.240	0.465	0.430	0.645
FINAL THICKNESS (mm)	19.785	19.545	19.080	18.650	18.005
AVERAGE THICKNESS (mm)	19.893	19.665	19.313	18.865	18.328
t_{90}(min)	5.32	4.41	2.55	6.45	9
C_V (cm^2/sec)	2.70x10^{-3}	3.05x10^{-3}	11.02x10^{-3}	4.34x10^{-3}	11.07x10^{-3}
e	0.462	0.446	0.429	0.395	0.363

Average Coefficient of consolidation (C_v) = 6.44x10^{-3} cm^2/sec

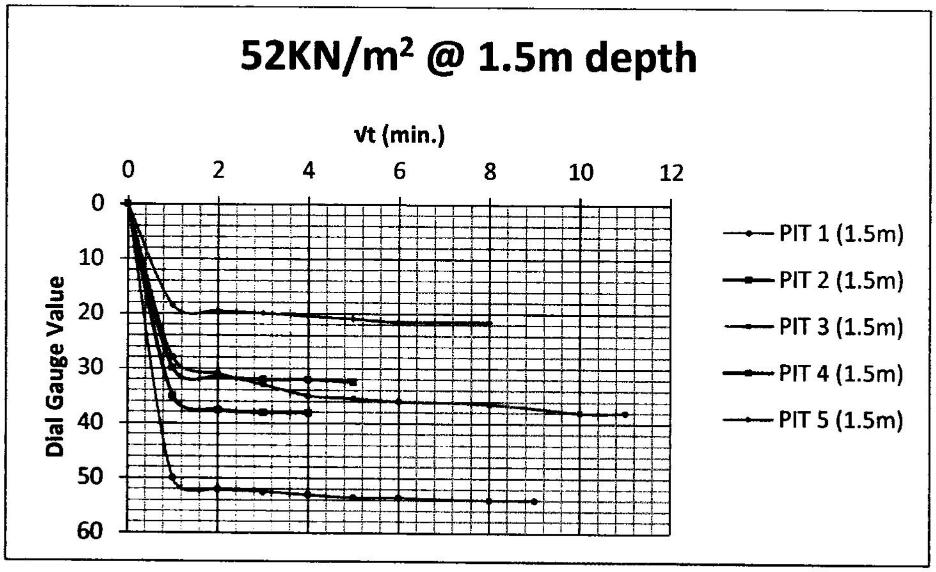
RAW CONSOLIDATION TEST RESULT FOR PIT 5 (3.0m) SOIL SAMPLE

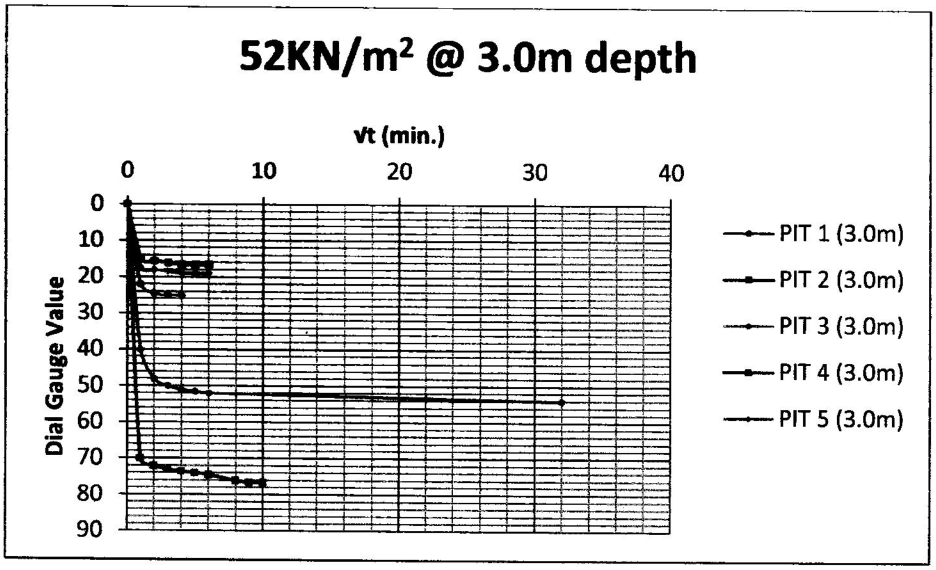
Elapsed time (Min.)	$\sqrt{t}$	DIAL READINGS				
		52 KN/m²	104 KN/m²	208 KN/m²	416 KN/m²	832 KN/m²
0	0	---	19.0	38.0	73.5	156.5
1.0	0.5	17.5	30.5	66.0	146.0	205.5
4.0	1.0	18.0	31.5	67.0	148.0	207.5
9.00	3.0	18.5	32.0	67.0	149.5	209.5
16.00	4.0	19.0	32.0	68.0	149.5	210.5
25.00	5.0	19.0	32.5	68.5	---	211.5
36.00	6.0	19.0	33.0	68.5	---	212.0
64.00	8.0	---	34.0	---	---	212.5
81.00	9.0	---	34.5	---	151.0	212.5
100.00	10.0	---	---	---	151.0	213.0
169.00	13.0	---	---	71.0	---	---
225.00	15.0	---	---	---	---	214.0
1089.0	33.0	---	---	73.5	---	---
1444.0	38.0	---	---	---	156.5	216.0
1600.0	40	---	38.0	---	---	---

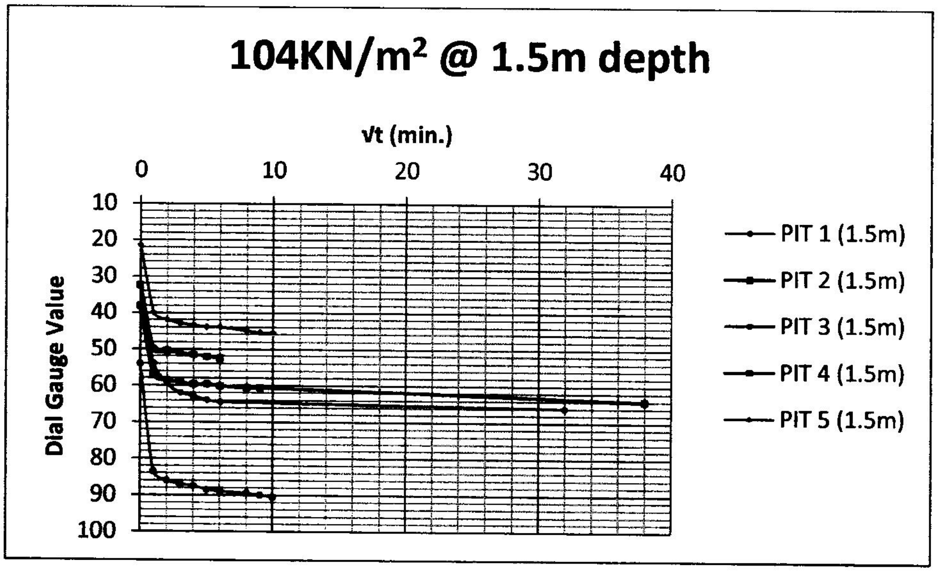
Table Showing the Consolidation Coefficients of Pit 5 (3.0m) Soil Sample

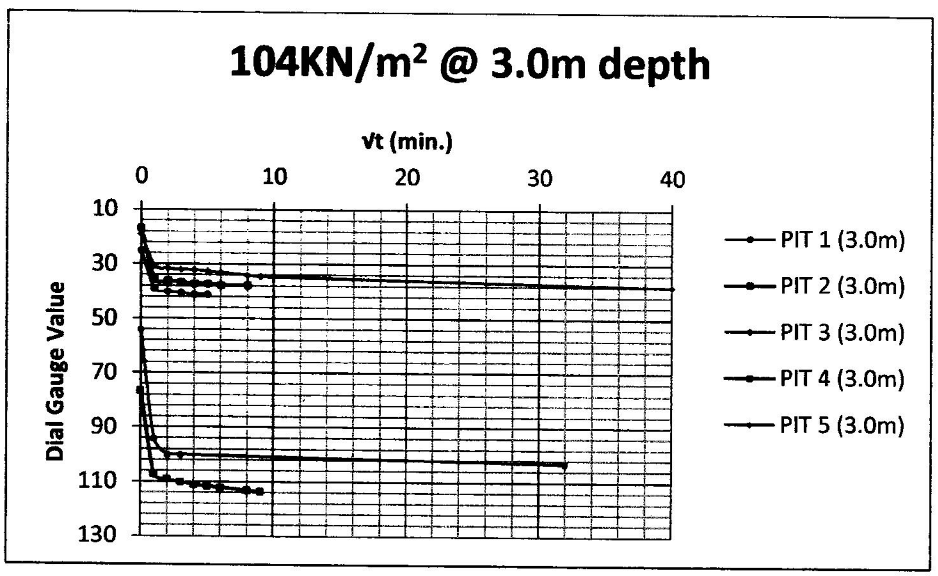
PRESSURE (KN/m²)	52	104	208	416	832
INITIAL DIAL READING	0	19.0	38.0	73.5	156.5
FINAL DIAL READING	19.0	38.0	73.5	156.5	216.0
INITIAL THICKNESS (mm)	20.000	19.810	19.620	19.265	18.435
CHANGE IN THICKNESS (mm)	0.190	0.190	0.355	0.830	0.595
FINAL THICKNESS (mm)	19.810	19.620	19.265	18.435	17.840
AVERAGE THICKNESS (mm)	19.905	19.715	19.443	18.850	18.138
t_{90}(min)	2.57	3.59	7.71	9.0	25
C_V (cm²/sec)	$5.17x10^{-3}$	$3.32x10^{-3}$	$3.67x10^{-3}$	$6.8x10^{-3}$	$11.17x10^{-3}$
e	0.608	0.592	0.577	0.549	0.482

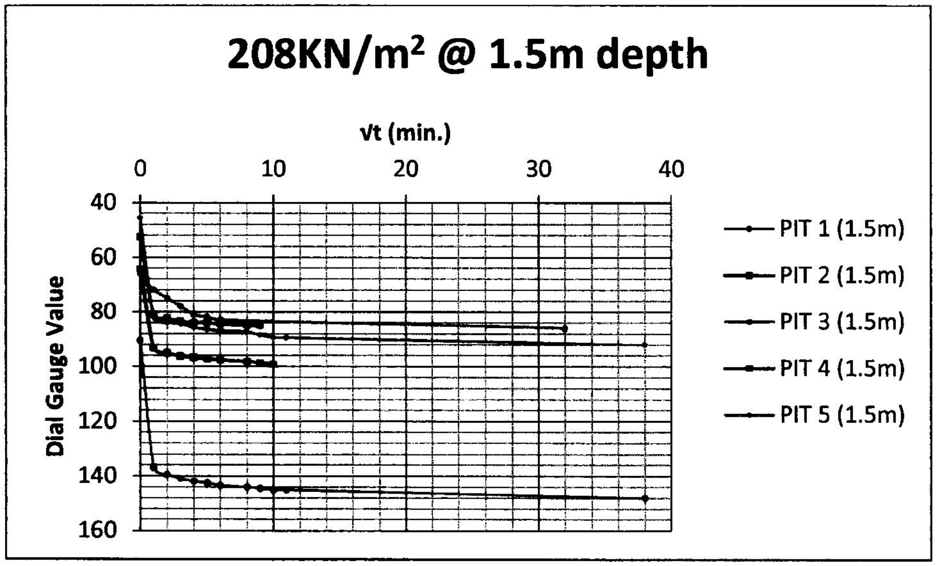
Average Coefficient of consolidation (C_v) = $6.03x10^{-3}$ cm²/sec

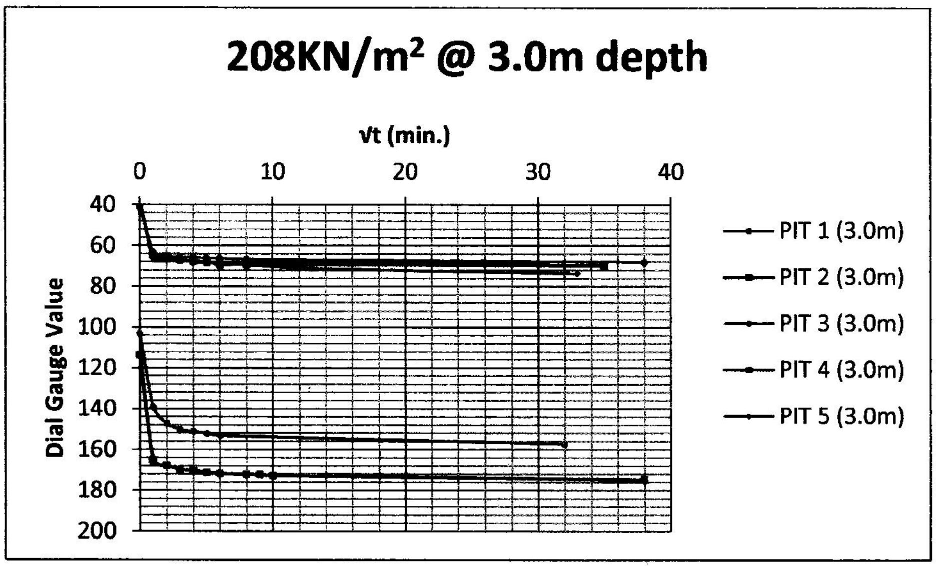

52KN/m² @ 1.5m depth
√t (min.)
0
2
4
6
8
10
12
Dial Gauge Value
0
10
20
30
40
50
60
PIT 1 (1.5m)
PIT 2 (1.5m)
PIT 3 (1.5m)
PIT 4 (1.5m)
PIT 5 (1.5m)

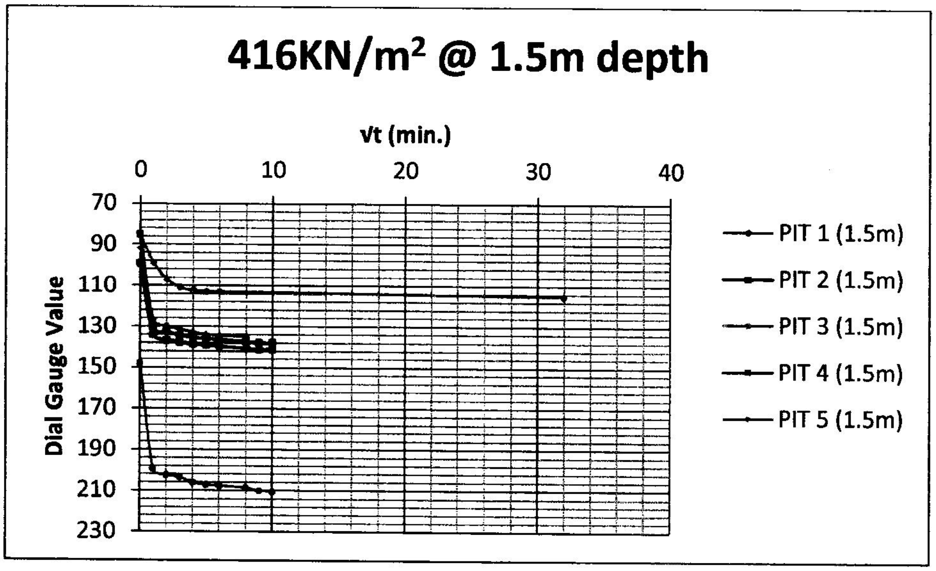

52KN/m² @ 3.0m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
0
10
20
30
40
50
60
70
80
90
PIT 1 (3.0m)
PIT 2 (3.0m)
PIT 3 (3.0m)
PIT 4 (3.0m)
PIT 5 (3.0m)

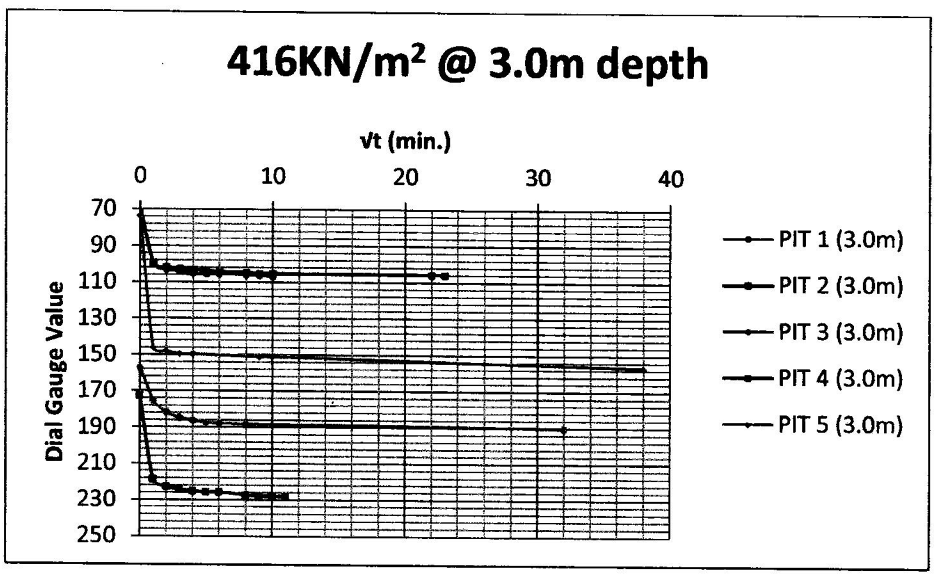

104KN/m² @ 1.5m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
10
20
30
40
50
60
70
80
90
100
PIT 1 (1.5m)
PIT 2 (1.5m)
PIT 3 (1.5m)
PIT 4 (1.5m)
PIT 5 (1.5m)

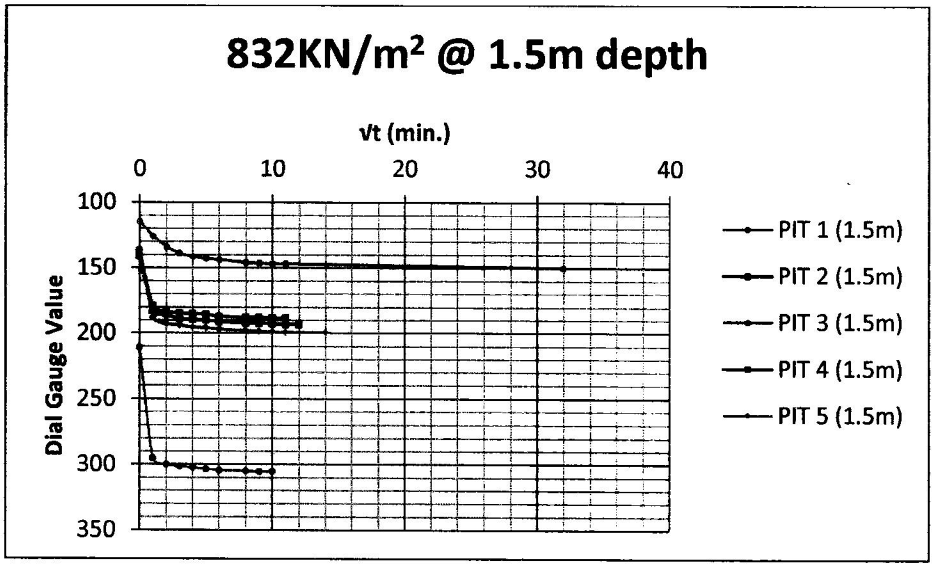

104KN/m² @ 3.0m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
10
30
50
70
90
110
130
PIT 1 (3.0m)
PIT 2 (3.0m)
PIT 3 (3.0m)
PIT 4 (3.0m)
PIT 5 (3.0m)


208KN/m² @ 1.5m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
40
60
80
100
120
140
160
PIT 1 (1.5m)
PIT 2 (1.5m)
PIT 3 (1.5m)
PIT 4 (1.5m)
PIT 5 (1.5m)

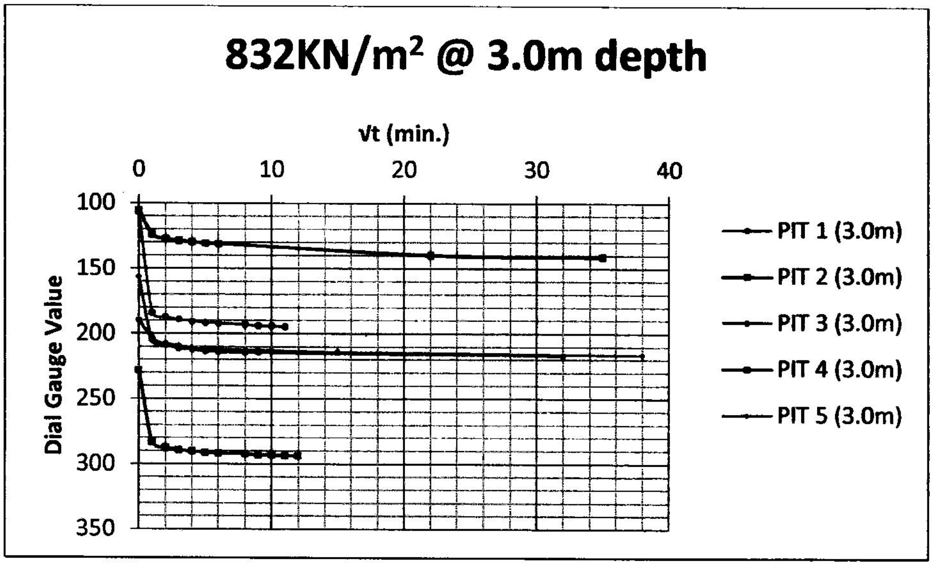

208KN/m² @ 3.0m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
40
60
80
100
120
140
160
180
200
PIT 1 (3.0m)
PIT 2 (3.0m)
PIT 3 (3.0m)
PIT 4 (3.0m)
PIT 5 (3.0m)


416KN/m² @ 1.5m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
70
90
110
130
150
170
190
210
230
PIT 1 (1.5m)
PIT 2 (1.5m)
PIT 3 (1.5m)
PIT 4 (1.5m)
PIT 5 (1.5m)


416KN/m² @ 3.0m depth
√t (min.)
0
10
20
30
40
Dial Gauge Value
70
90
110
130
150
170
190
210
230
250
PIT 1 (3.0m)
PIT 2 (3.0m)
PIT 3 (3.0m)
PIT 4 (3.0m)
PIT 5 (3.0m)



832KN/m² @ 1.5m depth
√t (min.)
0
10
20
30
40
100
150
200
250
300
350
Dial Gauge Value
PIT 1 (1.5m)
PIT 2 (1.5m)
PIT 3 (1.5m)
PIT 4 (1.5m)
PIT 5 (1.5m)



832KN/m² @ 3.0m depth
√t (min.)
0
10
20
30
40
100
150
200
250
300
350
Dial Gauge Value
PIT 1 (3.0m)
PIT 2 (3.0m)
PIT 3 (3.0m)
PIT 4 (3.0m)
PIT 5 (3.0m)