EFFECT OF DUST RATIO ON THE CONSOLIDATION BEHAVIOUR OF COMPACTED LATERITIC SOILS

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CERTIFICATION

This is to certify that this project topic titled "Effect of Dust Ratio on the Consolidation Behaviour of Compacted Lateritic Soils was undertaken by Umennuihe Kingsley Kenechukwu with registration number (NAU/2017224011) in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

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APPROVAL PAGE

This research work "Effect of Dust Ratio on the Consolidation Properties of Compacted Lateritic Soils" has been assessed and approved by department of civil engineering Nnamdi Azikiwe University.

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DEDICATION

This work is dedicated to Almighty God who has been my ultimate source of strength, for the gift of life and also for guiding me through school. I would additionally like to dedicate this work to my parents Mr. & Mrs. Polycarp Umennuihe who have been of great support to me throughout my education up to this very moment.

ACKNOWLEDGEMENT

Special thanks go to Almighty God for giving me the strength to complete this work and also for His guidance and protection throughout my stay in Nnamdi Azikiwe University.

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I want to thank in a very special way, my project supervisor in the person of Engr. Mrs. Nkechinyere Ezema for her time and guidance in the accomplishment of this project. May the good lord enlarge your coast and also protect your family.

Special thanks go to the Head of Department of Civil Engineering in the person of Engr. Prof. C. A. Ezeagu. He has been more like a father to us all and I appreciate him so much.

I will also like to extend my heartfelt appreciation to all the staff (academic and non-academic) of the Department of Civil Engineering for their invaluable tutorship and professional guidance.

Finally, I will like to appreciate everyone who has in one way or the other contributed to making me a better person, may the Lord Almighty reward you all greatly

ABSTRACT

The study was undertaken to evaluate the effect of dust ratio on consolidation properties of compacted lateritic soils. Three laterite samples designated as A, B and C with dust ratios ranging between 0.2-0.4, 0.4-0.6 and 0.6-0.8 were used for the study. The laterite samples were subjected to various testing which include: sieve analysis test, specific gravity test, compaction test, atterberg limit test and consolidation test. Results obtained from sieve analysis test revealed that the samples were classified as A-2-6, A-7-5 and A-7-6 according to AASHTO Soil Classification System, SC (sand mixed with clay) and CH according to Unified Soil Classification System, the specific gravity of the samples were 2.66, 2.62 and 2.55, the liquid limit, plastic limit and plasticity index of the samples were 26.8%, 32.6%, 33.9% while the plastic limit was 15.83%, 17.57%, 20.14%, the plasticity index of the samples were 10.97%, 15.03% and 13.44%, the maximum dry unit weight of the samples were 19.55kN/m^3, 17.18kN/m^3 and 16.69kN/m^3, the optimum moisture content were 7.86%, 14.21% and 14.88%, the coefficient of consolidation of the samples were 0.079mm^2/min, 0.081mm^2/min and 0.083mm^2/min, the coefficient of volume compressibility of the samples was 0.0018mm^2/kpa, 0.0021mm^2/kpa and 0.0021mm^2/kpa while the compression index of the samples was 0.0022mm^2/kpa, 0.003mm^2/kpa and 0.001mm^2/kpa. The study therefore concluded that dust ratio has significant impact on consolidation properties of compacted lateritic soils as lateritic soils with high amount of dust ratio has high coefficient of consolidation and coefficient of volume compressibility.

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LIST OF SYMBOL& ABBREVIATION

DR- Dust Ratio

CBR—California Bearing Ratio

G_S – Specific Gravity

AASHTO – American Association of State Highway and Transportation Officials

USCS – Unified Soil Classification System

ASTM – American Society for Testing and Material

BSL – British Standard Light

BSH – British Standard Heavy

MDUW- Maximum Dry Unit Weight

OMC – Optimum Moisture Content

LL – Liquid Limit

PL – Plastic Limit

SL – Shrinkage Limit

PI – Plasticity Index

D_{10} – Particle Size such that 10% is finer than the Size

D_{30}- Particle Size such that 30% is finer than the Size

D_{60}- Particle Size such that 60% is finer than the Size

C_U – Coefficient of Uniformity

C_C – Coefficient of Curvature

SC – Clayey Sand

SM – Silty Sand

GM – Silty Gravel

GC—Clayey Gravel

GW—Well Graded Gravel

GP—Poorly Graded Gravel

SP—Poorly Graded Sand

SW—Well Graded Sand

CL – Inorganic Clay of Low Plasticity (lean clay)

CH—Inorganic Clay of High Plasticity (fat clay)

ML- Silt of low Plasticity

MH – Silt of High Plasticity

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CHAPTER ONE

INTRODUCTION

1.1 Background of Study

Lateritic soils are the most common reddish weathered pedogenic surface deposits occurring in tropical and subtropical regions of the world (Eberemu, 2014). They constitute the most common materials for the construction of earth dams, highway, embankments, airfields as well as foundation material to support structures in these areas (Eberemu, 2014).The consolidation properties of lateritic soils is of primary concerns to engineers engaged in the design and construction of foundations, embankments, bridge abutments earth dams and fills (Schiffman et al. 2013). Consolidation deal with respond of a soil system to imposed load and prediction of stresses and displacement of the loaded soil as a function of space and time.

The theory of consolidation since its introduction by Terzaghi in 2023 has formed the foundation of modern geotechnical engineering where the interaction of soil and water exists. Although consolidation is used for evaluating settlements, it has played a key role in the design and construction of civil engineering structures. The consolidation properties of laterite regarded as one of the properties which control the settlement of structure depends significantly on the dust ratio present within the soil.Latertic soils are composed of a variety of minerals, and different research has been done on the specific characteristics of the soil (Retnamony, et al. 2015).

In terms of fine fractions present in most lateritic soils, the values of exchangeable cations may change. The more the monovalent exchangeable cations in the soils, the higher the distribution of crystals, and as a result, the particles become smaller and their specific surface area rises (Tie Lan et al., 2014).Increase in specific surface area and fineness of soil expressed as dust ratio causes the changes in the hydraulic and mechanical properties of lateritic soils which include the liquid limit, plastic limit, plasticity index, compressibility, permeability and consolidation (Mitchell et al., 2012).

The chemical composition and morphological characteristics lateritic soils are influenced by the level of weathering of which the parent material has been subjected to. It is therefore almost impossible to execute any construction work in Nigeria without the use of lateritic soils. The consolidation characteristics of most residual laterite soils appear to depend upon the nature of the soil, the position of the sample in the profile and the characteristics of the material deposit.Evaluation of the consolidation characteristics of residual laterite and other tropically weathered clay on the basis of Terzaghi's theory of consolidation has been found to be useful in predicting the settlement of structures. It was reported that the decrease of the void ratio with applied normal pressure in residual clays follows the same law which governs the consolidation phenomenon for sedimentary clays

Several researchers have worked on the effect of dust ratio on different engineering properties of soils. Research done by Monkul and Ozden, (2013) and Wang et al. (2019) have shown that dust ratio have effect on the consolidation properties of lateritic soils. Abedi and Yarosbi, (2010) conducted a study on the effect of dust ratio on the compaction and consolidation characteristics of laterite and it was concluded that laterite with appreciable amount of dust ratio has significant effect on the consolidation properties of lateritic soils.

The consolidation properties of soils particularly lateritic soils is major parameter used in the design of foundations for buildings, roads, embankment, dams and other civil engineering structures. These properties of lateritic soils which determine settlement of structures is largely affected by the presence of dust ratio in the laterite samples as properties of laterite samples vary with depth and location. Therefore it is important to evaluate the effect of dust ratio on the consolidation properties of laterite as this will be valuable particularly in the design of foundations for civil engineering structures and also help to promote knowledge on factors affecting the consolidation properties of lateritic soils.

1.2 Statement of Problem

Lateritic soils are one of the most utilized materials for civil engineering applications. They are used at the sub-grade level of pavement, foundation of building, embankment, earth dams and other civil engineering structures. Laterites are products of nature and as a result, there are variability in the properties of the soil as properties of laterite varies with depth, location and mode of formation. There are cases whereby laterite obtained from a particular borrow pit may contain substantial amount of fines expressed as dust ratio than that obtained from another borrow pit. This variability in the composition of laterite significantly affect the engineering properties of the soil most especially the consolidation properties which is one of the major properties used in the foundation design of structures. Therefore it is pertinent to ascertain the effect of dust ratio on the engineering properties of laterities particularly the consolidation characteristics of lateritic soils as this will be valuable in the design of foundation for civil engineering structures.

1.3 Aim and Objectives

The aim of the study is to evaluate the effect of dust ratio on consolidation properties of compacted lateritic soils. The objectives of the work are:

1. To determine the dust ratio of the three soil samples collected at different locations.
2. To determine the index properties of the laterite samples collected at varying dust ratios.
3. To determine the compaction and consolidation characteristics of the samples collected at varying dust ratios.
4. To study and analyze the effect of dust ratio on the consolidation parameters of compacted lateritic samples.
5. To draw conclusion and make recommendation in the light of the findings obtained from the study.

1.4 Scope of Study

The study is basically centered on evaluating the impact of dust ratio on consolidation properties of compacted lateritic soils. Three samples of laterite with dust ratio ratios ranging from 0.2-0.4, 0.4-0.6 and 0.6 – 0.8 collected at different locations was subjected to several laboratory testing. This tests include: sieve analysis test, specific gravity test, atterberg limit test, (liquid and plastic limit), compaction and consolidation test. The effect of dust ratio on the consolidation properties of the lateritic soils will be evaluated and conclusion will be drawn.

1.5 Significance of Study

Key findings obtained on the effect of dust ratio on consolidation properties of compacted lateritic soils will be valuable in the following ways:

1 Provide information on how dust ratio influences the consolidation characteristics of lateritic soils which will be useful in the design of foundation for civil engineering structures.
2 Promote knowledge on some of the factors influencing the settlement of structures.
3 Serve as body of knowledge for subsequent studies.

CHAPTER TWO

LITERATURE REVIEW

2.1 Definition and Origin of Laterite

Lateritic soils are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of parent rocks under tropical and subtropical climatic conditions (Aginam, et al 2015).This weathering process primarily involves the continuous chemical alteration of minerals, the release of iron and aluminum oxides and the removal of bases and silica in the rocks. Lateritic soils are void or nearly void of bases primarily silicates, but may contain substantial amount of quartz and kaolinite (Alexander & cady, 2013). They are formed in hot, wet tropical regions with an annual rainfall of at least 1200mm and a daily temperature in excess of 25^{o}C and typically occur in humid tropical climate with 30^{o}N and 30^{o}S of the equator (Madu, 2010).They are also composed entirely of iron and aluminum oxide. They are reddish in colour and are the least soluble of rock weathering in tropical climate (Plummer, et al 2013). Laterite is also described as a product of in-situ weathering in igneous, sedimentary and metamorphic rocks commonly found under unsaturated conditions (Rhardjo, et al 2014). Lateritic soil is one of the most common and important material used in earth work engineering construction in the tropics and subtropics where it is in abundance.

The name laterite was coined by an English surgeon Francis Buchanan in 1807 in India from a Latin word "later" meaning brick. In the 19th century, He coined the term laterite when he wrote "What I have called indurate clay is one of the most valuable materials for building. It is diffused in immense masses without any appearance of stratification and is placed over the granite that forms the bases of Malayala. It is full of cavities and pores and contains a very large quantity of quartz in the form of yellow and red ochres In the masses, while excluded from the air It is so soft, that any iron instrument readily cut it, and it is dug up in square masses with a pick-axle, and immediately cut into shape wanted with a trowel or large knife. It very soon become as hard as brick, and resists the air and water much better than materials made from bricks. The most proper English name would be laterite, from lateritis, the appellation that may be given to it in sciences". Since then lot of researches have been carried out on laterite and a lot of terms referring to many soil types have been produced. There is a tendency to apply the term to any red soil and rocks in the tropics (Abebaw, 2014). Nearly all kind of rock can be deeply decomposed by action of high rainfall and elevated temperature. The percolating rainwater causes dissolution of primary rock material and a decrease of soluble elements such as sodium, potassium, calcium and magnesium. As a result, there remain a residual concentration of insoluble element predominantly iron and aluminum. In geosciences, only those weathered products that are most strongly altered geochemically and mineralogically are termed laterite.

2.2 Formation of Laterite

(Tuncer et al, 2012) described the genesis of laterite as the weathering process which involves leaching of silica, formation of colloidal oxide and precipitation of the oxide with increasing crystallinity and dehydration as the soil is weathered. The major processes of weathering are physical, chemical and biological process. The physical weathering is predominant in the dry climate while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature (Abebaw, 2014). As the disintegration of underlying rock occurs, the primary element are broken down by the process of physical and chemical weathering to simple ionic form. The silica and bases in the weathered material such as sodium, potassium, calcium and magnesium are washed out by the percolated rain water (verdose water), boxides and hydroxides of sesquioxide are accumulated thereby enriching the soil and giving the soil it's characteristic red colour. This process is termed laterization and it depends on the nature and extent of chemical weathering.

Laterization is the weathering process by which the rock is transformed into laterite. It is a gradual process which must be active for centuries. In tropical countries the "verdose water" is at high temperature and as a result they may contain more carbonic acids, alkaline, carbonates and organic matter. This element explains why rocks that are leached by verdose water are commonly found in tropical countries than in temperate ones. After weathering, dehydration occurs. Dehydration (either partial or complete) alters the composition and distribution of the sesquioxide rich material in a manner which is generally not reversible over wetting (Abebaw, 2014). It leads to the formation of strongly cemented soil with a unique granular soil structure. The topography and drainage of an area also influences the rate of weathering because to some extent, it determines the amount of water available for laterization to occur and the rate at which it moves through the weathering zone. The rate at which weathered material is eroded is also controlled.

Deep weathering cannot occur on steep slopes this is because the surface run-off on steep slopes is greater than the rate of infiltration thereby increasing the rate of erosion. Hence lateritic soils tend to be found on slopes (sometimes locally termed ridge gravel), to a lesser extent on uplands and rarely in poorly drained areas (Jiregna, 2012). The structure of Lateritic soil varies with the type of parent rock from which it was formed, the location (i.e where it was formed)and also the weathering process that lead to its formation. Studies in some lateritic soils shows that they have porous granular structure consisting of iron impregnated clayey material in minute spherical aggregation.The aggregation derives its strength from the film found within the micro-joints of the elementary clay particles, which in addition coats the particles. Thus the film found the micro joints of the elementary clay particles and as coatings over particles provides the strength of aggregation. Viewing carefully prepared thin sections of laterite under the optical microscope has shown that these soils contain rough materials with sizes tending from silt to fine sand spread throughout the soil with very finely-divided iron oxide, and a porous structure of peds or clay clusters which are usually not cemented by coatings of iron oxide but rather, they are weakly bonded. The surface of laterite soil initially exists as a gelatinous coating. After losing moisture, it becomes denser but retains its non-crystalline structure after which it crystallizes slowly into different forms, which gives them strongly cemented surfaces covered by iron oxides (Sergeyev et al, 2012). The structural development depends on the deposition of iron oxides at different stages of weathering process.

Lateritic soil chemistry and mineralogy as shown by studies greatly influence the geotechnical properties, and in certain circumstances, significantly affects the economic potential in the construction industry (Ogunsanwo, 1995). Studies by (Tuncer and Lohnes, 2014) also revealed that the degree of weathering is very well connected with the mineralogy of laterite, as the kaolinite content is high in the early stage of weathering and decrease with increase in weathering ,where as the amount of sesquioxide increases. The soil profile of laterite is defined as that in which laterite horizon exists or is capable of developing under favorable conditions (Ikiensinma, 2012). The alteration of rock by the processes of chemical weathering take place progressively through a series of events and stages which result in a profile of weathering. Lateritic gravels stand out as low humps in the terrain. They consist of gravel sized concretionary nodules in a matrix of silt and clay. They may take up an area of several hectares and a thickness of between 1 to 5m (Jiregna, 2012).

2.3 Review of Compaction and Consolidation Properties of Laterite

2.3.2 Compaction Characteristics

The compaction characteristics of lateritic soils are determined by their grading characteristics and plasticity of fines. Most lateritic soils contain a mixture of quartz and concretionary coarse particles, which may vary from very hard to very soft. The strength of these particles has major implications in terms of field and laboratory compaction results and their subsequent performance in civil engineering construction projects. Placement variables (moisture content, amount of compaction, and type of compaction efforts) also influence the compaction characteristics. Varying each of these placement variables has an effect on permeability, compressibility, strength and stress-strain characteristics of the soil.

2.3.2 Compressibility and Consolidation

When a soil mass is subjected to a compressive force, its volume decreases (Abebaw, 2014). The property of the soil due to which it decrease in volume occurs under compressive force is known as the compressibility of soil. The compression of soil can occur due to;

1. Compression of solid particles and water in the void
2. Compression and expulsion of air in the void.
3. Expulsion of water in the voids

The compression of saturated soil under a steady static pressure is known as consolidation. It is entirely due to expulsion of water from the voids. The consolidation characteristics of lateritic soils is generally moderate with the modulus of compressibility ranging between 1×10^{-3} to 1×10^{-2} sq. ft./ton.

2.4 Consolidation of Soils

Consolidation is a process by which soils decrease in volume (Mohammed, 2016). According to Karl Terzaghi consolidation is any process which involves decrease in water content of a saturated soil without replacement of water by air. In general it is the process in which reduction in volume takes place by expulsion of water under long term static loads (Mohammed, 2016). It occurs when stress is applied to a soil that causes the soil particles to pack together more tightly, therefore reducing its bulk volume. When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process (Mohammed, 2016). If the stress is reapplied, the soil will consolidate again along a recompression curve, defined by the recompression index (Mohammed, 2016). The soil which had its load removed is considered to be overconsolidated. The highest stress that it has been subjected to is termed the preconsolidation stress.

The over consolidation ratio or OCR is defined as the highest stress experienced divided by the current stress. A soil which is currently experiencing its highest stress is said to be normally consolidated and to have an OCR of one. Soil could be considered underconsolidated immediately after a new load is applied but before the excess pore water pressure has had time to dissipate(Mohammed, 2016)..

In case of fine grained soil on which a structure is to be built, high water content is not desired as the weight of the structure may cause sinking (consolidation settlement) of the structure in due time. Typically the permeability (ability of water to move through the soil voids) of fine grained soils is low, hence it takes a long time for consolidation process.

So two aspects of consolidation settlement are important:

The rate at which the consolidation is taking place and the total amount of consolidation.

It is very important to note that unlike settlement in sands and other coarse grained soil, consolidation settlement of fine grained soil does not occur immediately. Hence, it is common practice to ensure that the consolidation process is expedited and that most of the consolidation takes place during the various phases of construction (Mohammed, 2016).If the soil is such that it has never experienced pressure of the current magnitude in its entire history, it is called a normally loaded soil.

The soil is called pre-consolidated (or over-consolidated) if at any time in history, it has been subjected to a pressure equal to or greater than the current pressure applied to it. In case of normally consolidated soils, the consolidation will be greater than that for a pre-consolidated soil. That is because the pre-consolidated soil has previously experienced greater or equal pressure and has undergone at least some consolidation under that pressure (Mohammed, 2016). So a pre-consolidated soil is preferred over a normally consolidated soil.

Consolidation is divided into two, the primary consolidation which involves the reduction in volume due to expulsion of water from the voids (Mohammed, 2016). Expulsion of water from the voids depends on permeability of soil and it is therefore time dependent. The reduction in volume due to expulsion of water from the voids. Expulsion of water from the voids depends on permeability of soil and it is therefore time dependent.

When all the water is squeezed out of the voids and primary consolidation is completed, further reduction in volume of soil is called secondary consolidation(Mohammed, 2016). It may be due to plastic deformation of the soil particles or some other reasons. The value is however very small and commonly neglected.

2.5 Similarities and Contrast between Consolidation and Compaction

The terms "Compaction" and "Consolidation" are often interchangeably used. Compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air: there is no significant change in the volume of water in the soil, while on the other hand, consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water, the process is continued until the excess pore water pressure set up by increase in total stress has completely dissipated; the simplest case is that of one-dimensional consolidation in which a condition of zero lateral strain is implicit, (Craig, 2004). Compaction is an artificial process, which basically involves densification of the soil mass through reduction of air in voids of the soil mass while the later is a natural process of gradual reduction in volume of the soil mass (settlement) through expulsion of the excess pore water in the soil over a period of time. It should also be noted that compaction is not time dependent while time is a major factor for completion in consolidation process. In most times, not all in-situ soil geotechnical properties are directly suitable for civil engineering works

2.6 Theory of Consolidation

The theory of consolidation was originally developed byTerzaghi (1925) in a study of the delay in the deformationcaused by the slow expulsion of water through the pores in amaterial of low permeability under compressive loading, inthis case, a sample of clay. For the one-dimensional case, hedeveloped the mathematical description of the phenomenon,on the basis of Darcy's law for the flow of a fluid througha porous medium, and his own concept of the effectivestress. He realized that in a soft soil, such as clay, thedeformations are caused by the effective stresses, definedas the difference of the total stress and the pore pressure,where the latter must be considered to act over the entiresurface of a cross section.

The theory was generalized to three dimensions and moregeneral materials, including porous rock, by Biot (2015),and since then it has been applied to a large variety ofpractical problems. A further generalization, to dynamicproblems, was made by Jiregna, (2012) and Biot (2015). One of the results from this generalization wasthat, in general, there are two modes of compressive waves:one in which the particles and the fluid move in phase and another in which these two components move in oppositedirections. This last mode has been observed in laboratoryconditions, but it can be shown to be strongly damped..

The original theory had been restricted to elastic deformationbehavior of the porous medium, but this restrictionwas removed later, especially since the development of modern numerical methods. Computer models are nowavailable that include more realistic models of soil behavior, including plastic deformations and creep (Plummer, 2013).

2.7 Literature Review on Relevant Subject

Bolarinwa, etal. (2017) investigated the compaction and consolidation characteristics of lateritic soil of a selected site in ikole ekiti, southwest Nigeria.The investigation was carried out through laboratory tests on disturbed and undisturbed soil samples obtained from three borings (BH1, BH2, and BH3) of Holy Apostolic Nursery/Primary School, Ootunja, Ikole Local Government Area (L.G.A.) of Ekiti State Southwest, Nigeria. The soils are all lateritic and mostly fine-grained. Compaction tests indicate maximum dry densities of 2.05Mgm-3, 1.78Mgm-3, and

1.69Mgm-3 at optimum moisture contents of 14.3%, 20.7% and 19.6% for soil samples obtained from BH1, BH2 and BH3 respectively. Compression indices (Cc) obtained from oedometer tests are 0.04816, 0.03820 and 0.04318 while the calculated coefficients of volume compressibility (mv), are 1.308*10-4, 1.065*10-4 and 1.093*10-4 m2kN-1 for samples in BH1, BH2 and BH3 respectively. The unsoaked California Bearing Ratio (CBR) value at 2.5 mm penetration ranges from 42.10% to 92.40% and CBR value at 5.0 mm penetration ranges from 52.70 to 89.10% indicative of good materials for road sub-grade, sub-base and base courses.

Eberemu, (2011) evaluated the effect of dust ratio on Consolidation Properties of Compacted Lateritic Soil Treated with Rice Husk Ash. Specimens were prepared at three different moulding water contents (2% dry of optimum, optimum moisture content and 2% wet of optimum) and compacted using the British Standard Light compactive effort. Preliminary tests on soils showed improved index properties with an increase in liquid limit (LL), an increase in plastic limits (PL) with a resulting de-crease in plasticity index (PI). Preconsolidation pressure increased with RHA content, it also decreased be-fore increasing with increased moulding water content. Reductions in compression index (Cc) and Swell In-dex (Cs) with increased RHA content were recorded. Cc and Cs generally decreased before increasing with increased moulding water content. The coefficient of volume compressibility (Mv) decreased and increased with higher RHA content; they were also affected by the soil particle state with increasing pressure. The co-efficient of consolidation (Cv) showed no observable trend with increased RHA content but generally in-creased with higher consolidation pressure on the dry and wet side of optimum compacted states. The current study will therefore evaluate the impact of dust ratio on consolidation parameters of compacted laterite samples.

Amadi(2017),asserts that the effect of dust ratio on lateritic soil has been a subject of extensive research, highlighting its significant impact on various geotechnical properties and engineering behavior. The studies reviewed in this abstract demonstrate that higher dust ratios in laterite soil lead to decreased plasticity, reduced compaction efficiency, decreased permeability, weakened shear strength, and compromised engineering properties. Consequently, engineers and researchers should consider the dust ratio in their investigations and design approaches to ensure appropriate land use planning and effective soil stabilization strategies in laterite soil regions.

CHAPTER THREE

MATERIALS AND METHODS

This chapter presents the material, sampling procedure, preservation and methods employed to achieve the research goal. The samples were obtained in accordance with sampling procedure described in clause 5.2 of BS 812-102: 1984 while the test was conducted in accordance with BS 1377 (1990). Below is a description of sampling procedures, preparation and methods.

3.1 Collection and Preparation of Sample

Three natural reddish brown lateritic soil samples with dust ratios ranging between 0.2-0.4, 0.4-0.6 and 0.6-0.8 respectively were obtained at consistent depth from Efab Estate Amansea, old Enugu road Awka (N6^{o}15'0.2442" E7^{o}8'23.3088"), (N6^{o}14'59.0424" E7^{o}24.5184") and first bank awka timber market(N6^{o}12'39" E7^{o}4'4").The laterite sample was collected with the aid of a digger and a shovel at a depth of 300mm.The samples passed the entire physical test that could classify them as lateritic soils in that, it is reddish-brown in colour, fine grained in texture and could become hard during the dry season. These samples were collected in four cement bags each and were conveyed school laboratory for various laboratory testing. The in-situ moisture content of the sample on arrival was determined using oven-dried method before air-drying for a period of two weeks in an open area using corrugated roofing sheets (commonly known as zinc) so as to ensure complete and even dissipation of moisture from the samples. Upon drying, the sample was segregated by means of crushing. The crushing was done through the use of wooden mortar and pestle. Enough care was exercised to ensure that the individual particles were not crushed into smaller sizes. This was achieved by pressing the pestle on the sample agglomerate and not pounding the soil with the pestle.

3.2 Methods of Study

3.2.1 Particle Size Distribution (Sieve Analysis)

Sieve analysis is a procedure used to assess the particle size distribution of a granular material (sand, gravel). The size distribution is often of critical importance to the behaviour of the material during use. Sieve analysis can performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

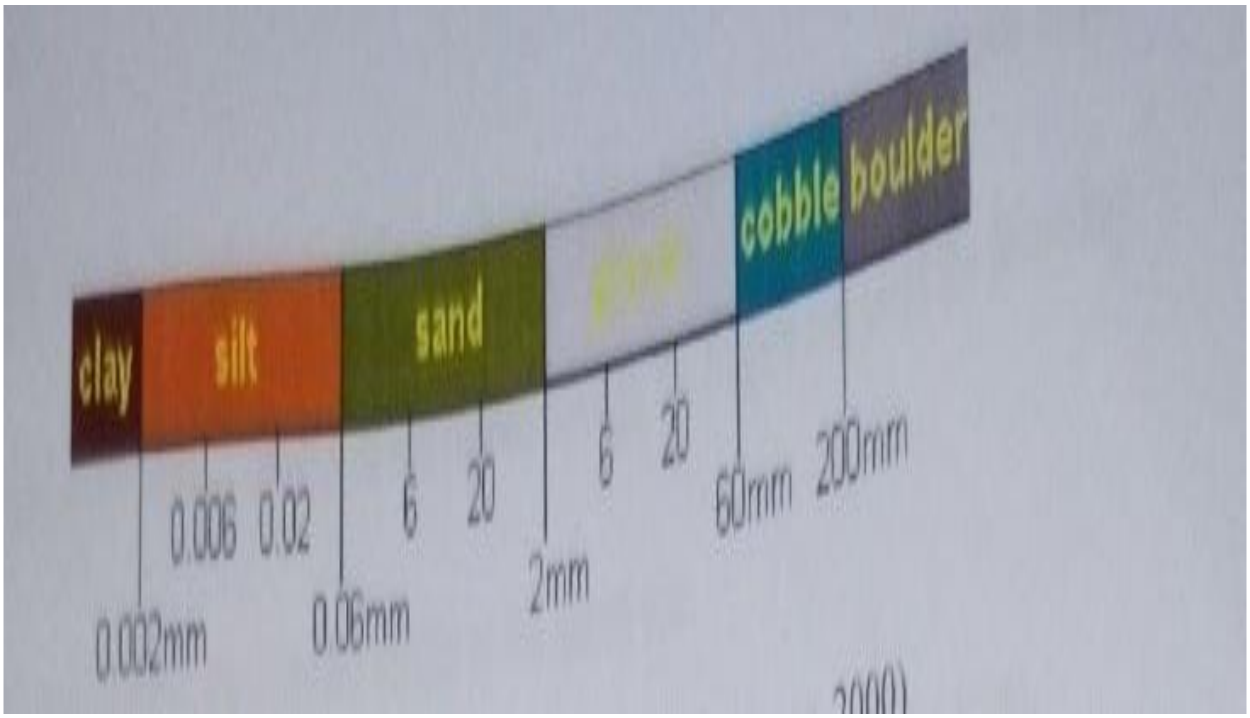

Figure 3.1 Ranges for grain Sizes of different Soil type (Atkinson, 2000).

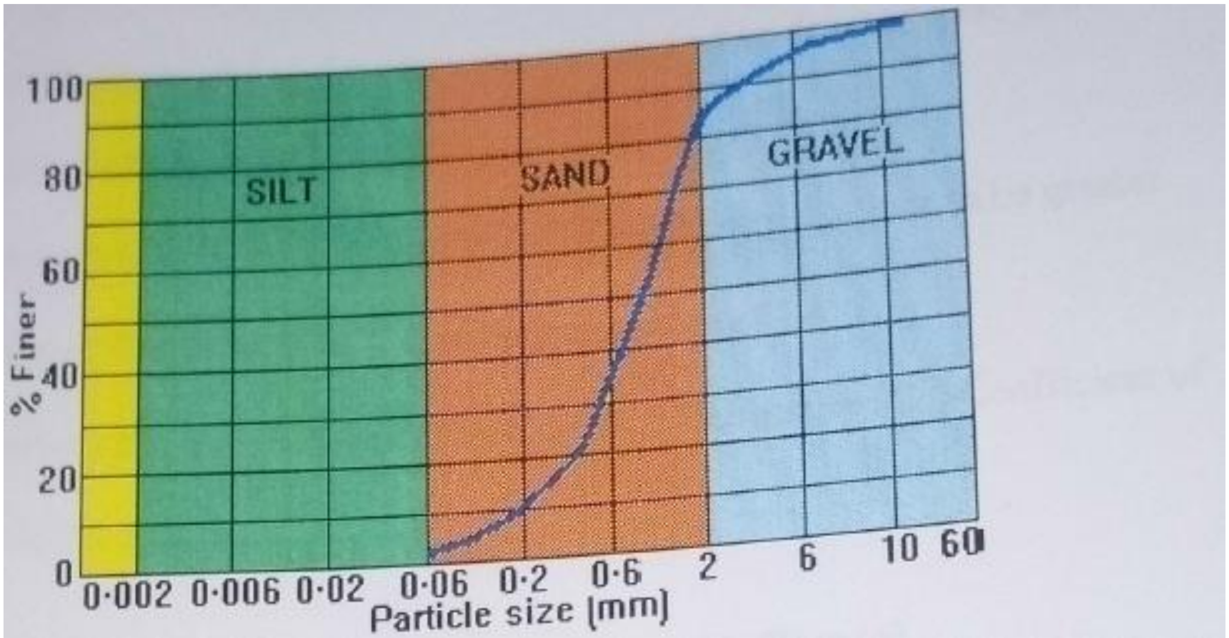

Figure 3.2 Grading Curve Ranges for Different Soil Types (Atkinson, 2000).

Soil posses a number of physical characteristics which can be used as aid to identify it sizes in the field. A handful of soil rubbed through the finger can yield the following:

1. Sand and other coarser particle are visible to the naked eye.
2. Silt particle becomes dusty and are easily brushed off.
3. Clay particle are greasy and sticky when wet and hard when dry and have to be scrapped or washed off hand and boot

For a soil to be well graded the value of coefficient of uniformity (Cu) has to be greater than 4 and 6 for gravel and sand respectively, while the Coefficient of Curvature (C_v) should be in the range of 1 to 3.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)

6. Thermostatically controlled Oven (With temperature of about $80^{O}C$-$110^{O}C$).
7. Masking tape for identification of sample.
8. Exercise book and pen for recording of result.
9. The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

Percentage retained (%) $= \frac{massofsoilretainedinthesieve(g)}{totalmassofsoilsample(g)} \times 100$

Cumulative percentage retained = $\sum Percentageretained$ (%)

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

Coefficient of Curvature $= \frac{D60}{D10}$

Coefficient of Uniformity $= \frac{(D30)2}{D10 \times D60}$

Where

D10= particle size such that 10% of the soil is finer than the size

D30= particle size such that 30% of the soil is finer than the size.

D60= particle size such that 60% of the soil is finer than the size.

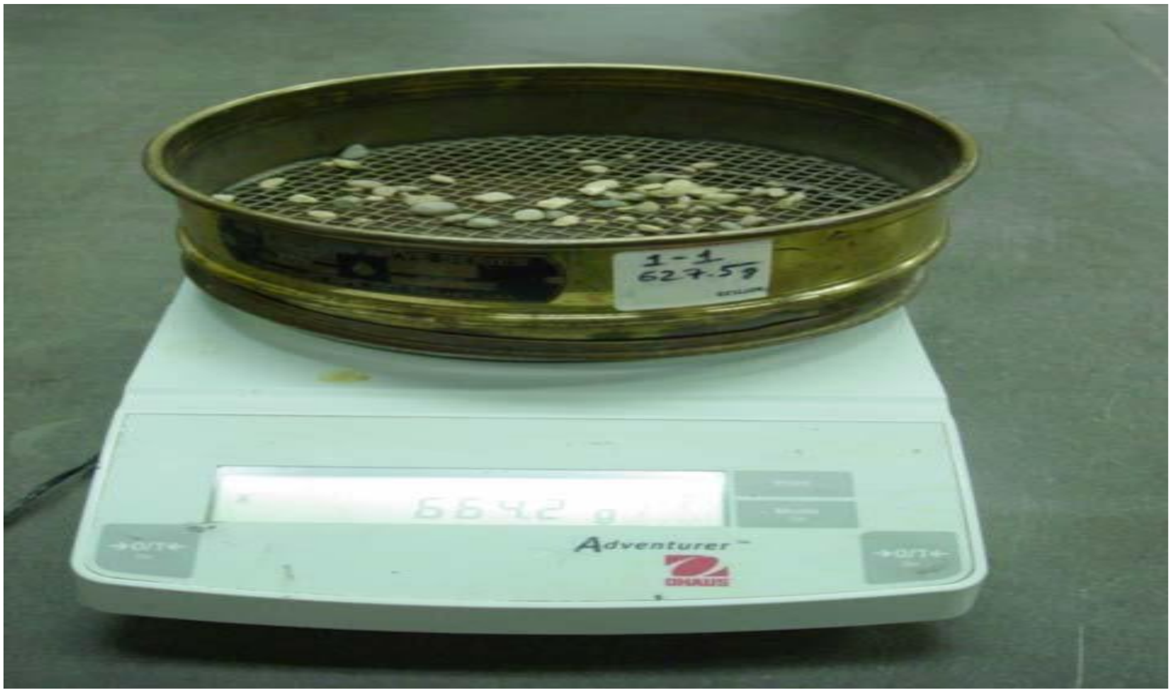

Figure 3.3 Apparatus for Particle Size Distribution Test (Sieve Analysis).

Figure 3.4 Apparatus for Particle Size Distribution Test (Sieve Analysis)

Test Procedure

1. The stack of sieves to be used for the experiment was properly cleaned using hand brush.
2. About 500g of air-dried soil sample was weighed with the aid of a weighing balance.

3. The weighed soil sample was poured into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.
4. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.
5. The sample was removed from the oven and the weight was determine (net weight) by deducting the weight of plate from the weight of plate and soil.
6. The stacks of sieve was arranged in the ascending order, placed in a mechanical sieve shaker, and thereafter the sample was poured and connected to the shaker for about 10-15 minute.
7. The sieve shaker was disconnected and the mass retained on each of the sieve sizes was determined.
8. The percentage retained, Cumulative percentage retained and Cumulative percentage finer was determined.
9. The graph of sieve Cumulative percentage finer against sieve sizes was plotted.
10. D10, D30 and D60 were determined from the plotted graph.
11. The Coefficient of Curvature and Coefficient of Uniformity was determined and used to classify the soil adopting the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

3.2.2 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined by (Braja, 2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

1. Density bottle of 50ml capacity and a stopper.
2. Desiccator containing anhydrous silica gel.

3. Thermostatically controlled oven with temperature of about 80-110^{O}C.
4. Weighing balance of 0.01g sensitivity.
5. Mantle heater.
6. Plastic wash bottle.
7. Distilled water.
8. Funnel
9. Thin glass rod for stirring.
10. 425um Sieve.
11. Dry piece of cloth for cleaning.
12. Masking tape for identification of sample.
13. Exercise book and pen for recording of result.

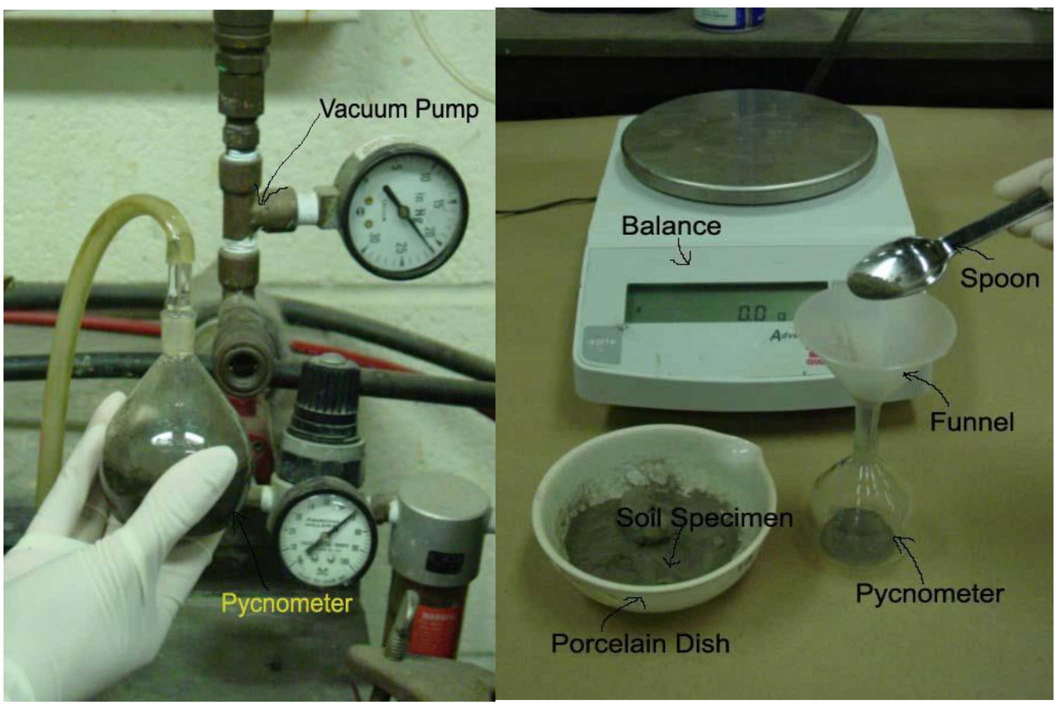

Figure 3.5 Apparatus used for Specific Gravity Test

Test Procedure

1. The density bottle properly cleaned and rinsed with distilled water, thereafter oven- dried and then cooled it in a desiccator so as to remove any moisture present.
2. The empty clean and dry density bottle was weighed and recorded as (M_1).
3. About 10-15g of soil passing through 425um sieve was placed inside the density bottle, weigh and the weight of density bottle +dry soil + stopper was recorded as (M_2).
4. Distilled water was added to fill about half to three-fourth of the density bottle, and then the sample was soaked for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).
5. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample, the sample was not allowed to boil over.
6. After agitation, the sample was allowed to cool at room temperature and then filled with distilled water up to the specified mark (at lower meniscus level), the exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and recorded as (M_3).
7. The density bottle was emptied, cleaned and rinsed with distilled water, then filled with distilled water up to the same mark. The exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle filled with distilled water + stopper was determined and recorded as (M_4).
8. The test procedure was repeated for two more trials and the average specific gravity value was obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result obtained is as follows:

Specific gravity (G_S) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

M_3= Weight of density bottle filled with water + wet soil + stopper.

M_4= Weight of density bottle filled with water + stopper

3.2.3 Atterberg Limit Test.

The behavior of soils especially fine grained soils differs considerably in the presence of water. Clay in the presence of water may almost take a liquid or can be quite hard. Consistency is the property of soil that offers resistance to deformation, it denote the degree of firmness of a soil and can be explained in terms of plasticity and stickiness of soil. Stickiness is the ability of soil especially fine grained soil to adhere to other materials while plasticity on the other hand is the ability of soils to undergo a change in shape under the action of an impressed force without a change in volume.

Stickiness of soils especially fine grained soils can be identified practically by mixing of an air-dried soil with a given quantity of water and then interposing the soil between the thumb and the fore finger (index finger), thereafter the following inference are made as it regards to the observation and this includes:

1. **Non-Sticky:** If the wet soil falls freely between the thumb and the forefinger without leaving any remain or without stretching.
2. **Slightly Sticky:** If the wet soil falls slowly with an infinitesimal traces of remains but without stretching.
3. **Sticky:** If the wet soil falls quite slowly with visible remains and apparent stretching.
4. **Very Sticky:** If the wet soil stretches between the thumb and the fore finger without falling.

The plasticity of soils can be identified practically by rolling a known weight of wet soil into a 3mm uniform diameter thread and the following inferences based on the observation are made and they are as follows:

1. **Non-Plastic:** If the wet soil cannot be rolled into thread.

2. **Slightly Plastic:** If the wet soil can be rolled into thread but crumbles easily under application of little pressure.
3. **Plastic:** If the wet soil can be rolled into 3mm thread but crumbles under intense application of pressure and cannot be reformed.
4. **Very Plastic:** If the wet soil can be rolled into 3mm diameter thread but crumbles under intense application of pressure and can be reformed.

The atterberg limit is a limit characterized by visible transition of soil (especially fine grained soils) from liquid-plastic-semi-solid-solid state consequent upon the variation of moisture content. This test was developed by Albert Atterberg a Swedish agricultural scientist in 1911. This test is divided into three limits namely:

1. Liquid Limit (LL)
2. Plastic Limit (PL)
3. Shrinkage Limit

3.2.3.1 Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Cassagrande type)
2. Grooving tool
3. Moisture content tins
4. Porcelain evaporating dish
5. Spatula or pellet knife
6. Thermostatically controlled oven
7. Weighing balance sensitive to 0.01g
8. Plastic wash bottle containing distilled water
9. Paper towels
10. Masking tape for identification of tin.
11. Exercise book and pen for recording of data
12. 425um Sieve
13. Airtight container

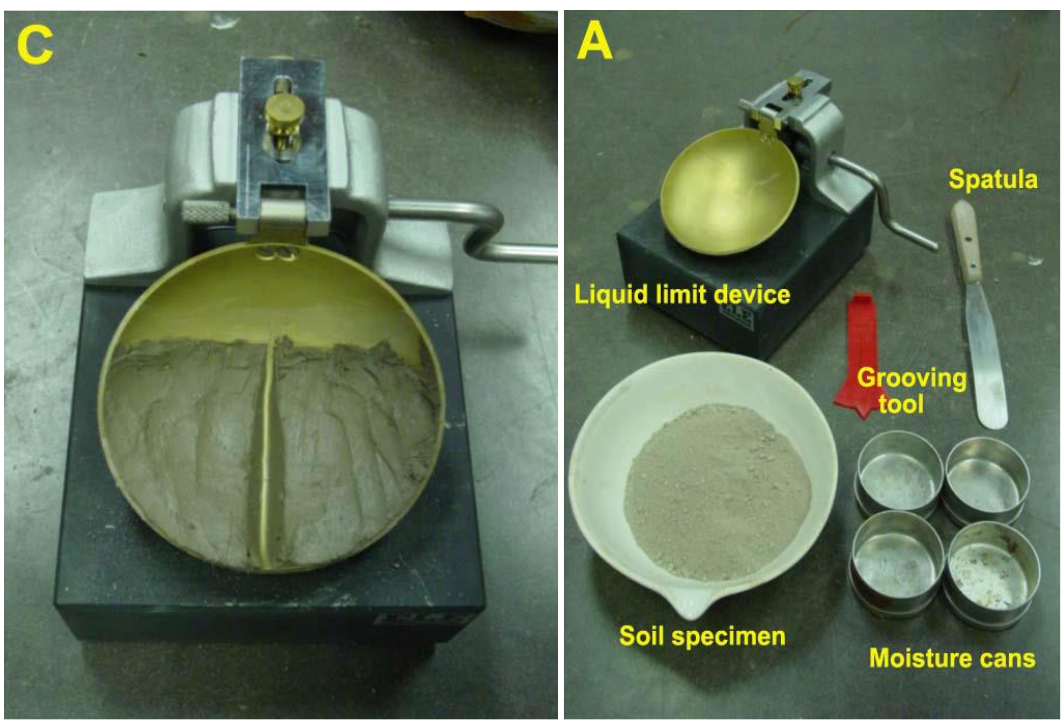

Figure 3.6 Apparatus for Atterberg Limit Test

Test Procedure

1. The sample was prepared by weighing about 150g of soil and passing it through 425um sieve, the sample was mixed with distilled water in a glass plate with the aid of pellet knife, during the mixing operation, coarse particle was removed by hand and mixed the sample was mixed to form a thick homogenous paste, thereafter, the mixed soil was placed in an airtight container and leave to mature for 24hrs.
2. The mass of four moisture content tins was determined and recorded as (W_1)
3. The matured sample was placed on an evaporating dish with little water added to it using the plastic squeeze bottle; the soil was properly mixed to ensure uniform distribution of moisture.
4. A portion of the paste (mixed soil) was placed on the liquid limit device and then the mixture was leveled so as to obtain a maximum depth of 1cm.
5. The grooving tool was used to cut a groove along the symmetrical axis of the cup holding the tool perpendicular to the cup.
6. The handle of the crank of the liquid limit device was rotated at the rate of 2 revolution per second and the no of blows required to close the groove at a distance of 13mm was counted. Closing of the groove should be as a result of plastic flow of the soil and not by sliding, if sliding occurs repeat the test.
7. About 10g of soil in the closed groove was taken and placed in the moisture content tins for moisture content determination, the sample was weighed and recorded as (W_2)
8. The rest of the soil in the cup was removed and paper towel was used to clean the cassagrande cup properly.
9. The water content of the soil was altered and the process was repeated to obtain the required no of blows in the range of 15-40 blows.
10. The graph of moisture content against the log of no of blows was plotted and the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content $= \frac{\text{Weightofwater}}{\text{weightofdrysoil}} \times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried

3.2.3.2 Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.2 is used to classify soil based on the ranges of it plasticity index.

Table 3.4: Plasticity Ratings for Fine grained Soil (Braja, M.Das, 2002).

Plasticity Index	Plasticity
0	Non-Plasticity
<7	Low Plasticity
7-17	Medium Plasticity
17-35	High Plasticity
>35	Very High Plasticity

1. The apparatus used for this experiment includes:

2. A smooth glass plate about 300mm square and 10mm thick.
3. A palette knife or spatula
4. A short length of 3mm metal rod
5. Moisture content tins
6. Plastic squeeze bottle
7. Weighing balance with 0.01g sensitivity
8. Veneer caliper
9. Masking tape for tin identification
10. Exercise book and pen for recording of result.

Test Procedure

1. The sample was prepared by the method described in the liquid limit using the sample passing 425um sieve.
2. The empty moisture content tins was identified, weighed and recorded as (W1).
3. About 20g of the prepared soil paste was placed on a porcelain evaporating dish and water was added using the plastic squeeze bottle, the soil was mix thoroughly until the paste is plastic enough to be rolled into a ball.
4. A portion of the ball was taken and rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward.
5. The rolling and remolding continued until the thread just start to crack at a distance of 3mm.
6. The small crumbed pieces was collected and placed in a moisture content tin a weighed and recorded as (W2).
7. The tin was placed in the oven at a constant temperature of 80-110^{O}C for a period of 16-24hrs.
8. After 24hrs, the tin was removed from the oven and the weight of the dry soil plus the tin was determined and recorded as (W_3).
9. The test procedure was repeated for at least two trials and take the average plastic limit value for all the trials.

The Computation for Plastic Limit is as follows:

Plastic limit = $\frac{Weight of water}{Weight of oven-dried soil} \times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

3.2.4 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil. According to (Shruthi, 2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing it dry density. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the dry density of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

In civil engineering practice soil compaction is essential for the following reasons:

1. Increasing the bearing strength of foundation
2. Provide stability to slope and foundation.
3. Prevention of undesirable settlement of structures
4. Reduction of water seepage from structure

The compaction methods to be adopted for this research are British Standard Light for natural laterite samples and laterite stabilized with sand, chipping dust and blend of sand and chipping dust.

Details of British Standard Compaction Process

Table 3.5: Details of Compaction Mould.

Type	**Diameter (mm)**	**Height (mm)**	**Volume(cm^3)**
British Standard	**105**	**115.5**	**1000**

Table 3.6: Details of Compaction Procedure.

Type of test	**Mould (cm^3)**	**Rammer(kg)**	**Drop (mm)**	**No of layers**	**Blow per layer**
BS light	**1000**	**2.5**	**300**	**3**	**27**
BS heavy	**1000**	**4.5**	**450**	**5**	**27**

The mechanical energy applied in each type of British Standard in term of work done is given as follows:

British Standard Light

Mechanical energy = $\frac{Weightoframmer \times nooflayers \times noofblows \times heightofdrop}{Volumeofmould}$

$= \frac{2.5g \times 3layers \times 27blows \times 300mm}{1000}$ **= 60.75kgm =60.75× 9.81Nm=596j**

Work done per unit volume of soil =$\frac{596}{1000}$ **=596kj/m^3**

The apparatus used for the test are as follows:

1. Compaction mould with a detachable base plate and removable extension collar.
2. Metal rammer (either 2.5kg or 4.5kg)
3. Measuring Cylinder 200ml or 500ml
4. Large Metal tray (600mm×600mm ×600mm)

5. Balance up to 10kg readable to 1g
6. Small tools such as palette knife, steel straight edge about 300mm long.
7. Drying oven temperature of 105-110^{O}C
8. Apparatus for moisture content determination

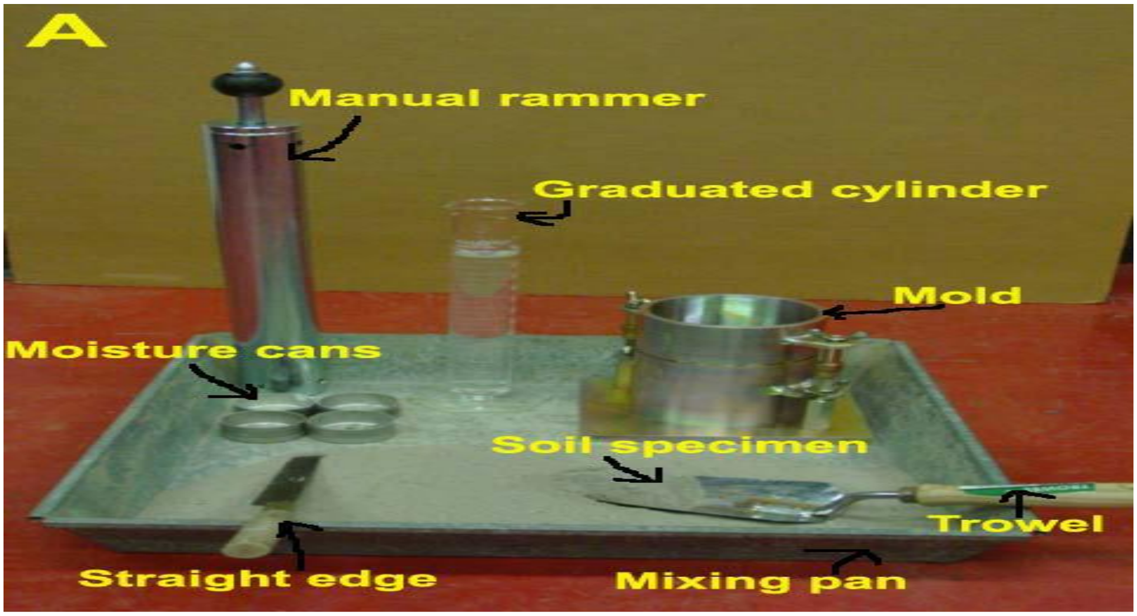

Figure 3.7:Apparatus employed for Compaction Test.

Test Procedure

1. The mould, extension collar and base plate was cleaned and dried. The dimension was measured and weigh to the nearest 1kg check if the rammer falls freely.
2. The internal surface of the mould was greased.
3. The extension collar was attached to the mould.
4. About 3kg of the soil sample was weighed on a weighing balance.
5. About 4% water was added to the soil sample, mixing it thoroughly and separating the soil into three layers for British Standard Light and five layers for British Standard Heavy.

6. The wet soil was poured into the mould and compacted thoroughly by applying the required no of blow using either a 2.5kg or 4.5kg rammer falling freely from a height of 300mm. The blow was distributed uniformly over the surface of the mould.
7. After completion of the compaction operation, the extension collar was removed and the top of the mould was carefully levelled by means of a straight edge.
8. The mould with the compacted soil to the nearest 1kg was weighed and recorded as W_2.
9. The moisture content of the representative sample of the specimen was determined and recorded as M.
10. The procedure was repeated and 8%, 12%, 16% and 20% of water was added and the value obtained was recorded.
11. The graph of dry density against moisture content was plotted and the maximum dry density (MDD) of the soil at the corresponding optimum moisture content (OMC) was determined.

The Computation of the result obtained is as follows:

Determination of Dry Density (P_d).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (M^3) = W_4

Bulk Density (kg/m^3) = $\frac{Wtofwetsoil\ (\text{kg})}{\text{Volofmould (m3)}} = \frac{\text{W2}-\text{W1}}{\text{W4}}$

Moisture Content (%) = $\frac{\text{mosturecontent(top)} + \text{moisturecontent (bottom)}}{2}$

Dry Density (kg/m^3) = $\frac{\text{Bulkdensity}}{1+\text{moisturecontent (\%)}} = \frac{\text{Pb}}{1+\text{w}/100}$

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W_1

Wt of tin + wet soil = W_2

Wet of wet soil (kg) = W_3 = W_2-W_1

Wt of tin + dry soil (kg) = W_4

Wt of dry soil (kg) = $W_5 = W_4 - W_1$

Wt of water (kg) = $W_6 = W_3 - W_5$

Moisture Content (%) = $\frac{\text{Wtofwater}}{\text{Wtofdrysoil}} \times 100 = \frac{W6}{W5} \times 100$

3.2.5 Consolidation Test

Consolidation test is used to determine the rate and magnitude of soil consolidation when the soil is restrained laterally and loaded axially. The Consolidation test is also referred to as Standard Oedometer test or One-dimensional compression test. This test is carried out on saturated soil specimens, especially in cohesive soils. The consolidation parameters obtained by this test are used to determine the consolidation settlement and time of consolidation for a given loading state (i.e. given height of embankment). These parameters are also used in design of "Ground Improvement measures", provided for construction of embankment on soft soils.

Test Apparatus

Consolidometers in fixed-ring or floating-ring models are required for testing soil samples with Consolidation Load Frames. Consolidometer Accessories include Consolidation Ring Porous Stones, Consolidation Cell, Dial Gauges, Loading Device, Equipment for measuring Initial Height of Test Specimen to an accuracy of 0.1 mm, and are compatible for use in testing soil consolidation.

Test Procedure

Preparation of Test Sample

1. Weigh the empty consolidation ring, designated W1

2. If the specimen is to be prepared from a tube sample, a representative sample for testing shall beextruded and cut off, care being taken to ensure that the two plane faces of the resulting soil disc areparallel to each other. The thickness of the disc of soil shall be somewhat greater than the

height ofthe consolidation ring.If the specimen is to be prepared from a block sample, a disc similar in size to that specified aboveshall be cut from the block with two parallel faces. The diameter of the disc shall be at least 10 mmgreater than the inside diameter of the consolidation ring. Care shall be taken to ensure that the soilstratum is oriented such that the laboratory test will load the soil in the same direction relative to thestratum as the applied force in the field.

3. Using the weighed consolidation ring as a template, the edges of the disc obtained in step 2 shall betrimmed carefully until the ring just slides over the soil. The last fraction of soil is pared away by thecutting edge of the ring as it is pushed down slowly and evenly over the sample with no unnaturalvoids against the inner face of the ring; this process is best done using a mechanical guide to preventtilting or horizontal movement of the ring. The top and bottom surfaces shall project above andbelow the edges of the ring to enable final trimming.Should an occasional small inclusion interfere with the trimming operation, it shall be removed, andthe cavity filled completely with material from the parings. Alternatively, if sufficient sample isavailable, it would be preferable to eventually extrude and discard the portion of the specimencontaining the inclusion from the ring, leaving a specimen free of such disturbed zones. If inclusions
are known to exist in a soil sample, a large diameter consolidation ring should be used, in order tominimize the relative effect of the disturbed zones. If excessive inclusions are encountered duringtrimming, the sample should be discarded. If no alternative exists, the tube sample shall be extrudeddirectly into a consolidation ring of equal diameter.

4. The soil sample thus obtained shall be trimmed flush with the top and bottom edges of the ring. Forsoft to medium soils, excess soil should be removed using a wire saw, and final trimming may bedone with a straight edge if necessary. For stiff soils a straight edge alone may be used for trimming.Excessive remoulding of the soil surface by the straight edge should be avoided. In the case of verysoft soils, special care should be taken so that the specimen may not fall out of, or slide inside thering during trimming.

5. A sample of soil similar to that in the ring, taken from the trimmings, shall be used for determiningmoisture content.

6. The thickness of the specimen (H) shall be measured and it shall be weighed immediately (W2)should the nature of the soil make satisfactory thickness determination difficult, the ring height maybe assumed as specimen height.

Assembly of Apparatus

1. The bottom porous stone shall be centered on the base of the consolidation cell. If soils sensitive tomoisture increase (swelling or collapsing soils) are being tested, the stone should be placed dry.When testing softer clays, the stone should be wet, and it may be covered by a wet filter paper. Nofilter paper shall be used for the stiffer and moisture sensitive soils.

2. The ring and specimen shall be placed centrally on the bottom porous stone, and the upper porousstone and then the loading cap shall be placed on top. The top stone shall be placed dry or wet, andwith or without filter paper.

3. The consolidometer shall be placed in position in the loading device and suitably adjusted. The dialgauge is then clamped into position for recording the relative movement between the base of theconsolidation cell and the loading cap. A seating pressure of 0.05 kgf/cm2shall be applied to thespecimen.

4. The consolidation cell shall be filled with water, preferably with distilled water. The type of water usedshall be noted in the data sheet

5. The specimen shall then be allowed to reach equilibrium for 24 hours.

Loading

1. For consolidation testing, it is generally desirable that the applied pressure at any loading stage bedouble than that at the preceding stage. The test may, therefore, be continued using a loading sequence which would successively apply stress of 0.1,0.2,0.4,0.8, 1.6, 3.2, etc, kgf/cm2 on the soilspecimen.

2. For each loading increment, after application of load, readings of the dial gauge shall be taken usinga time sequence such as 0, 0.25, 1, 2.25, 4,6.25,9, 12.25, 16, 20.25, 25, 36, 49, 64, 81, 100, 121,144, 169, 196, 225, etc, min, up to 24 hour(s) or 0, 1/4, 1/2, 1, 2, 4, 8, 15, 30, 60 min, and 2, 4, 8, 24 hour(s). These time sequences facilitate plotting of thickness or change of thickness ofspecimen against square root of time or against logarithm of timeThe loading Increment shall be left atleast until the slope of the characteristic linear secondarycompression portion of the thickness. versus log time plot is apparent, or until the end of primaryconsolidation is indicated on a square root of time plot. A period of 24 hours will usually be sufficient,but longer times may be required. If a period of 24 hours is seen to be sufficient, it is recommendedthat this commonly used load period be used for all load increments. In every case, the same loadincrement duration shall be used for all load increments during a consolidation test.

3. It is desirable that the final pressure be of the order of at least four times the pre-consolidation

pressure, and be greater than the maximum effective vertical pressure which will occur in situ due tothe overburden and the proposed construction.

4. On completion of the final loading stage, the specimen shall be unloaded by pressure decrements which decrease the load to one-fourth of the last load. Dial gauge readings may be taken as necessary during each stage of unloading. If desired, the time intervals used during the consolidation increments may be adopted; usually it is possible to proceed much more rapidly.

5. In order to minimize swell during disassembly, the last unloading stage should be to 0.05 kgf/cm2 which should remain on the specimen for 24 hours. On completion of this decrement, the water shall be siphoned out of the cell and the consolidometer shall be rapidly dismantled after the release of the final load. The specimen, preferably within the ring, shall be wiped free of water, weighed (W3), and thereafter placed in the oven for drying. If the ring is required for further testing, the specimen may carefully be removed from the ring in order to prevent loss of soil, and then weighed and dried.

6. Following drying, the specimen (plus ring) shall be reweighed (W4).

7. The porous stones shall be boiled clean after the test, in order to prevent clay from drying on them and reducing their permeability.

Determination of Coefficient of Compressibility Av

1. Transfer the final dial gauge reading for each pressure increment from Appendix-B to Col. 2 ofAppendix-A, recording it against the total applied pressure which is noted in Col. 1 of Appendix-A.

2. From the dry weight of the specimen, Ws, the volume of soil voids, Vs shall be obtained as:

$V_s = W_s / G_s \gamma_w$

Where:

G_s = specific gravity of the solid particles, and

γ_w = unit weight of water.

3. The equivalent height of soil solids can be determined as:

$H_s = V_s / A$

Where, A is area of specimen in cm2

4. From Col. 2 of Appendix-A, determine ΔH for each pressure increment and record it in Col. 3.

5. The height of specimen at the end of each pressure increment, H, can be determined by subtracting ΔH
of a particular increment from H of the specimen prior to application of that increment. This is to be
recorded in Col. 4 of Appendix-A.

6. Void ratio, e, is obtained as:

$e = (H / H_s) - 1$

and recorded in Col. 5 of Appendix-A.

7. Values of de and dσ obtained are recorded in Col. 6 and 7 of Appendix-A respectively.

8. The coefficient of compressibility, a_v, with units of inverse of units for stress shall be calculated as:

$a_v = de / d\sigma$

and recorded in Col. 8 of Appendix-A.

Determination Of Compression Index (Ce):

Plot the void ratio, e versus log σ. The slope of the straight line portion, that is, for the soil in the normally consolidated state, is designated as Ce. This can be directly obtained from the plot or calculated as

$C_e = de / \log(\sigma_2/\sigma_1)$

Where: where σ_2 and σ_1 are the successive values.

7. Presentation of Results

The results of a consolidation test are presented in the form of a set of curves showing the relationship of e versus and log σ, av versus log σ and Cv versus log σ. The value of Ce is also reported

CHAPTER FOUR

RESULTS AND DISCUSSION

This chapter presents key finding obtained from laboratory investigation of the effect of dust ratio on consolidation properties of compacted laterite soils. The findings are summarized in Table 4.1 below:

4.1 Results

Table 4.1: Index Properties of the Laterite Samples with Varying Percentages of Dust Ratio

Properties	Sample A	Sample B	Sample C
Specific Gravity	2.66	2.62	2.55
Percentage Passing Sieve No 200 (0.075mm)	16.18	43.28	63.17
Sand Content (%)	83.82	56.72	36.83
Fine Content (%)	16.18	43.28	63.17
AASHTO Soil Classification System	A-2-6	A-7-5	A-7-6
USCS Soil Classification System	SC	CH	CH
Liquid Limit (%)	26.8	32.6	33.9
Plastic Limit (%)	15.83	17.57	20.46
Plasticity Index (%)	10.97	15.03	13.44
Plasticity Rating	Medium Plasticity	Medium Plasticity	Medium Plasticity

4.2: Compaction and Consolidation Characteristics of Compacted Laterite Samples at Varying Percentages of Dust Ratio

MDUW (kN/m^3)	**19.55**	**17.18**	**16.69**
OMC (%)	**7.86**	**14.21**	**14.88**
Coefficient of Consolidation Cv (mm^2/min)	**0.079**	**0.081**	**0.083**
Coefficient of Volume Compressibility Mv (mm^2/kpa)	**0.0018**	**0.0021**	**0.0021**
Compression Index (mm^2/kpa)	**0.0022**	**0.003**	**0.001**

4.2 Discussion on Findings

4.2.1 Sieve Analysis Test

Figure 4.0 depicts the particle size distribution curve for the laterite samples obtained at varying percentages of dust ratio. Evaluation of the amount of fines and sand present in the different samples revealed that sample C contains significant amount of fines than sample A and B while sample A contains significant amount of sand than sample B and C. The percentage passing sieve No 200 (0.075mm) for sample A, B and C were 16.18%, 43.28% and 63.17% respectively and as a results, the sample were classified as A-2-6, A-7-5 and A-7-6 according to AASHTO Soil Classification System, SC (sand mixed with clay) and CH (clay of high plasticity) according to Unified Soil Classification System. The gradation of the sample could not be ascertained due to loss in some of the shape parameters (D10) of the samples during wet sieving.

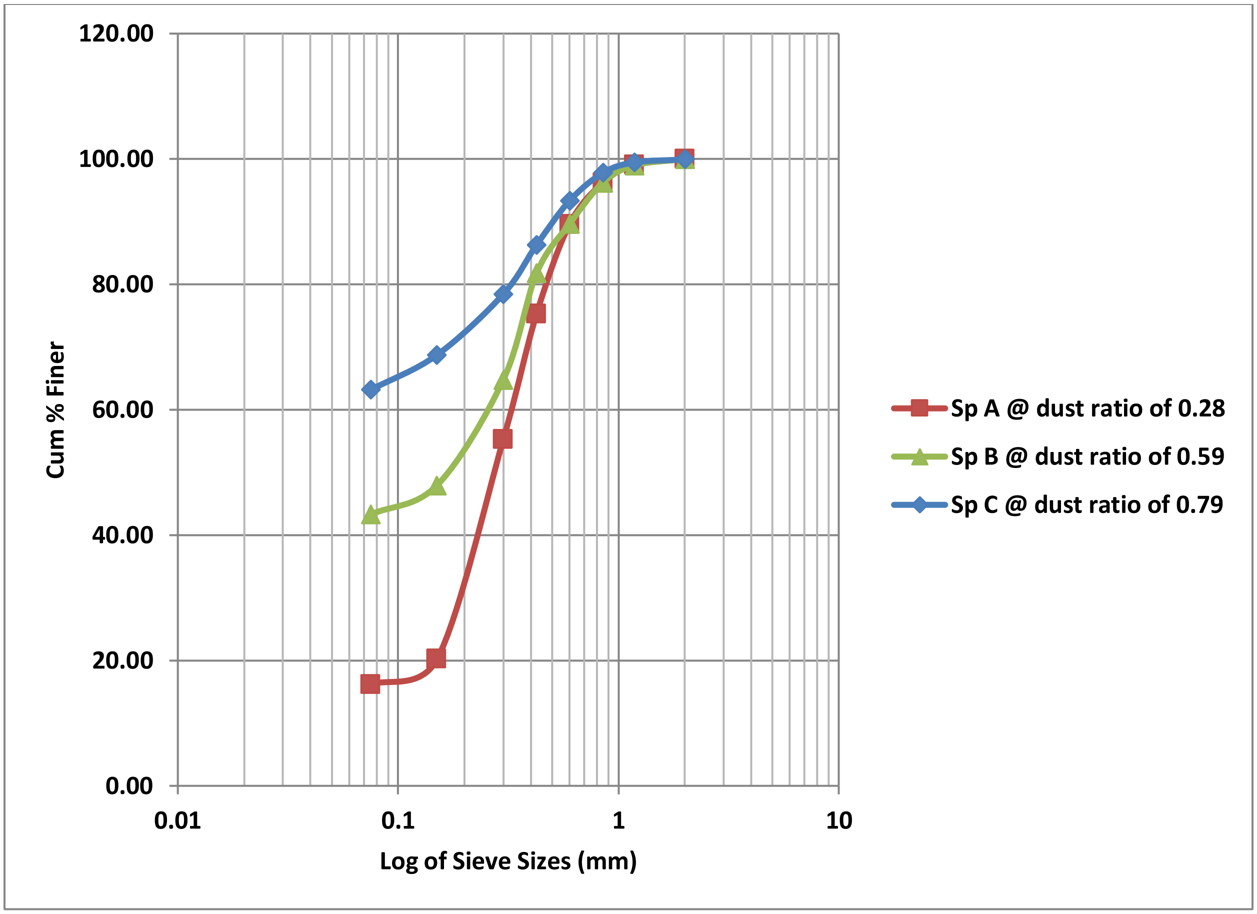

Figure 4.0: Particle Size Distribution Curve of the Different Laterite Samples with Varying Percentages of Dust Ratio

4.2.2 Specific Gravity

Figure 4.1 shows the specific gravity of Sample A, B, C and D respectively. The specific gravity of the samples was 2.66, 2.62 and 2.55. Sample A with a dust ratio of 0.28 yielded the peak specific gravity value while sample C with a dust ratio of 0.79 yielded the lowest specific gravity value. This implies that the specific gravity of soil decreased with increase in dust ratio. The range of specific gravity (2.55-2.62) obtained for the different laterite samples suggests that the presence of clay and silt in the samples which is of advantage particularly when used at the sub-grade level of pavement construction.

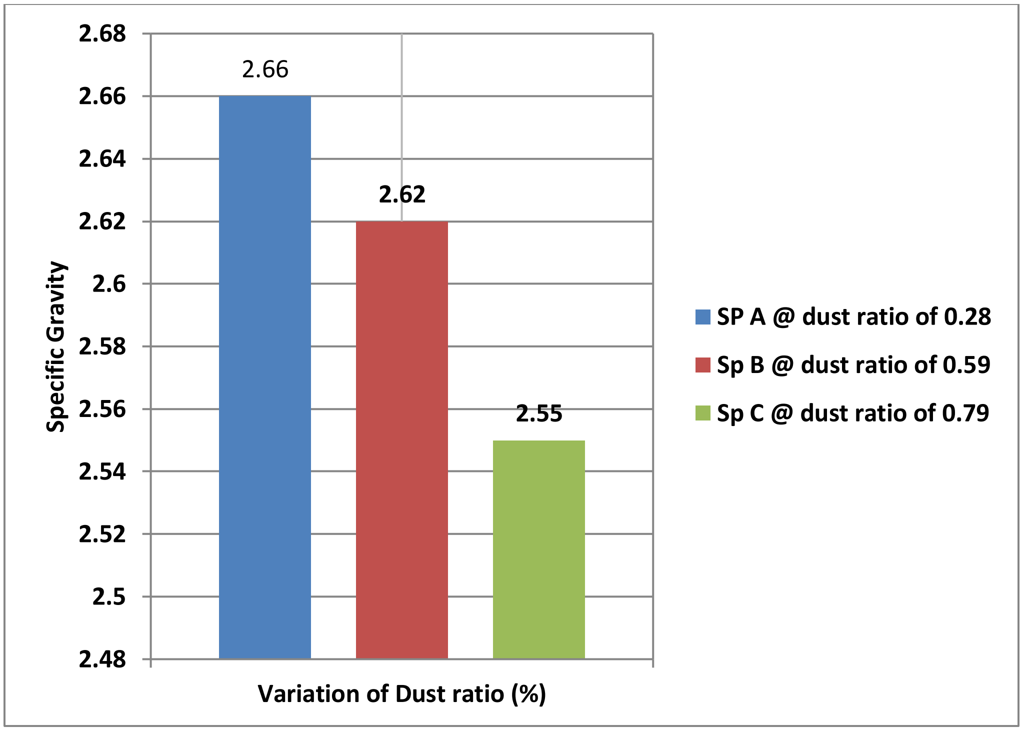

Figure 4.1: Specific Gravity Values of the Different Laterite Samples with Varying Percentages of Dust Ratio

4.2.3 Atterberg Limit Test

Table 4.1 depicts the liquid, plastic and plasticity index of the laterite samples obtained at varying percentages of dust ratio. The liquid and plastic limit of the samples was found to increase with increase in dust ratio while the plasticity index showed a trend of increasing and decreasing values. The increase in liquid and plastic limit of the samples could be attributed to the amount of fines present in the samples as samples containing significant amount of fines are usually characterized by high liquid and plastic limit values. These results indicate that the behaviour of lateritic soil is largely influenced by their dust ratios. Assessment of plasticity of the samples suggests that the sample were of medium plasticity.

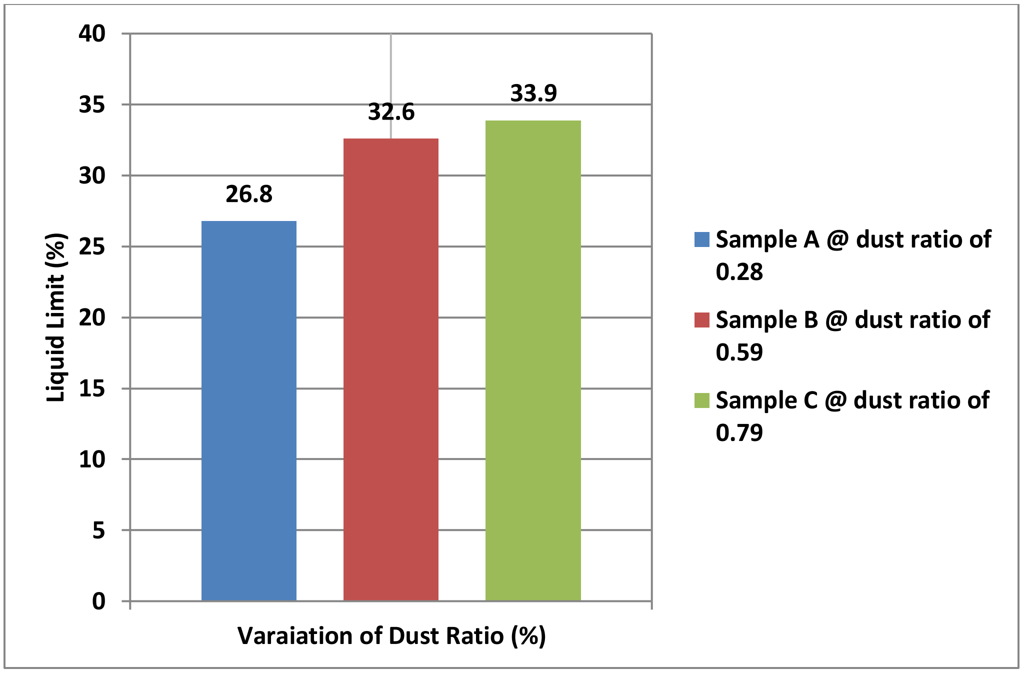

Figure 4.2: Liquid Limit Values of the Different Laterite Samples with Varying Percentages of Dust Ratio

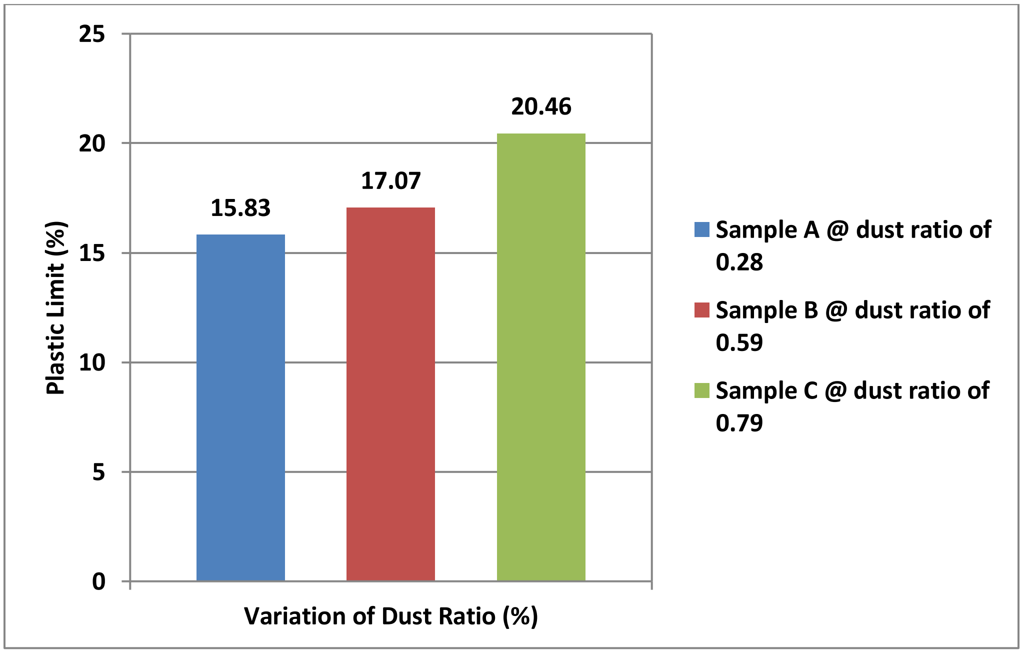

Figure 4.3: Plastic Limit Values of the Different Laterite Samples with Varying Percentages of Dust Ratio

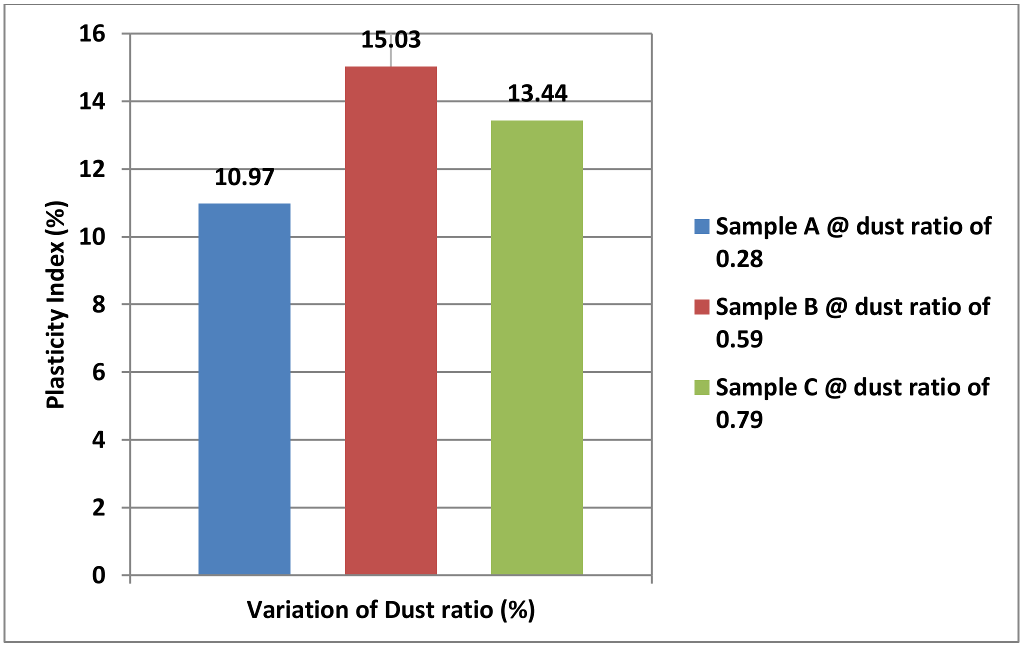

Figure 4.4: Plasticity Index Values of the Different Laterite Samples with Varying Percentages of Dust Ratio

4.2.4 Compaction Test

Figure 4.5 depicts the compaction curve for the different samples of laterite collected at varying percentages of dust ratio. The maximum dry unit weight of the samples were 19.55kN/m^3, 17.18kN/m^3 and 16.69kN/m^3 while the optimum moisture content were 7.86%, 14.21% and 14.88% respectively. The result showed that sample A with a dust ratio of 0.28 yielded the peak maximum dry unit weight while sample C with a dust ratio of 0.79 yielded the lowest maximum dry unit weight. In other words, it was observed that the maximum dry unit increased with decrease in dust ratio while the optimum moisture content increased with increase in dust ratio. Assessment of the optimum moisture content showed that sample A with optimum moisture content of 7.86% will require less water content to achieve maximum dry unit weight during field compaction while sample C with an optimum moisture content of 14.8% will require more water content to achieve maximum dry unit weight during field compaction. It was also observed that the optimum moisture content of the samples decreased with an increase in maximum dry unit weight of the samples, this agrees with Proctor (1933),Venkatramaiah (2006), Rowe (2000) and other concluded research works.

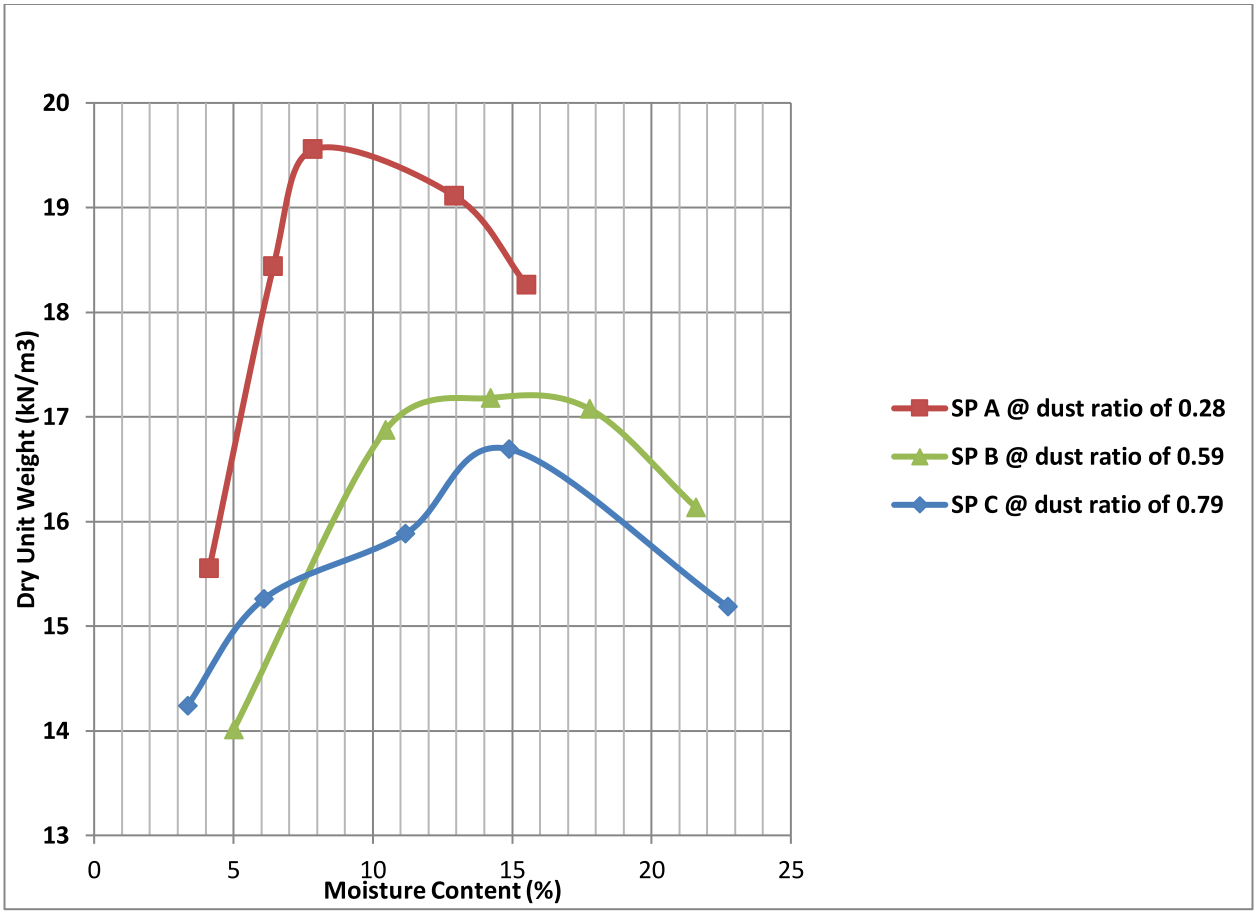

Figure 4.6: Compaction Curve for the three Different Samples with Varying Percentages of Dust Ratio

4.2.5 Consolidation Test

Table 4.1 shows the consolidation characteristics of compacted laterite soils obtained at varying percentages of dust ratio. The consolidation properties of the compacted laterite soils are dictated by parameters such as coefficient of consolidation, coefficient of volume compressibility and compression index. The coefficient of consolidation controls the rate of settlement of the compacted laterite soils while the coefficient of volume compressibility controls the magnitude of settlement of the compacted laterite soils. The compression index is also a parameter which controls the settlement of soils; they are used in foundation design (Kumar, 2015). Results obtained as depicted in Table 4.2 shows that the coefficient of consolidation of the compacted laterite samples increased with increase in dust ratio, this implies that compacted laterite soils containing significant amount of dust ratio will require more time to undergo settlement than compacted laterite sample with less amount of dust ratio. Assessment of the coefficient of volume compressibility of the compacted laterite samples shows that the volume compressibility of the samples also increased with increase in dust ratio, although the coefficient of volume compressibility for compacted laterite sample with a dust ratio of 0.59 and 0.79 were relatively the same. This result implies that compacted laterite samples containing significant amount of dust ratio will undergo substantial amount of settlement than compacted laterite sample with lesser amount of dust ratio. The increase in coefficient of consolidation and coefficient of volume compressibility could be attributed to the significant amount of fines present in the samples. Result obtained from compression index of the compacted laterite samples showed that the compression index of the samples decreased with increase in dust ratio. This finding is in agreement with the work of Mengwe, et al. (2016).

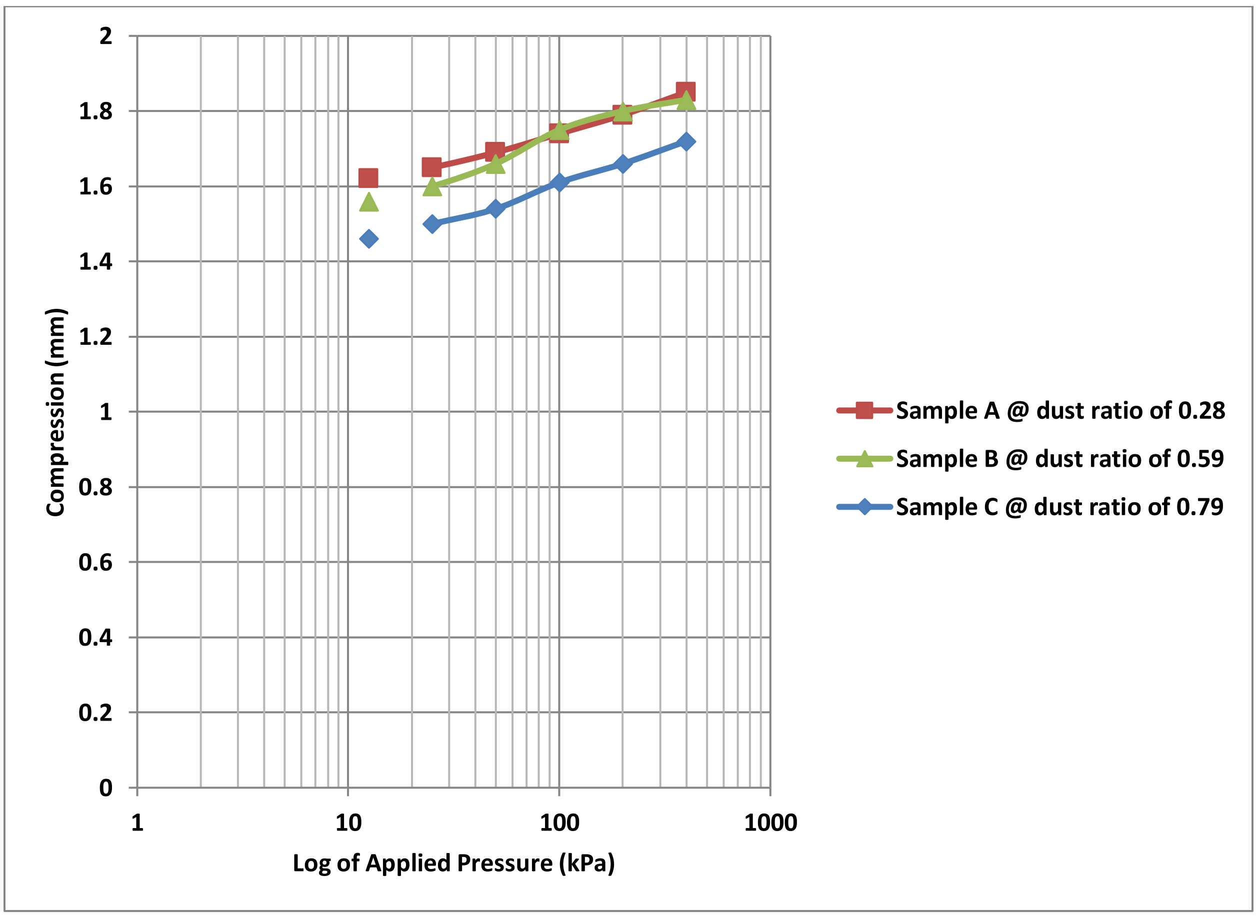

Figure 4.7: E-P Curve for three Different Samples with Varying Percentages of Dust Ratio

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusion in the light of the findings obtained from the study on the effect of dust ratio on consolidation properties of compacted laterite soils can be drawn:

1 Evaluation of the index properties of the laterite samples revealed that sample A, B and C with a dust ratio of 0.28, 0.59 and 0.79 were classified as A-2-6, A-7-5 and A-7-6 according to AASHTO Soil Classification System, SC and CH according to Unified Soil Classification System, the specific gravity of the samples were 2.66, 2.62 and 2.55 respectively while the liquid limit and plastic limit were 26.8%, 32.6%, 33.9%, 16.18%, 43.28% and 63.17%.

2 The specific gravity, liquid limit and plastic limit of the samples were found to increase with increase in dust ratio.

3 The maximum dry unit weight of the samples was found to increase with decrease in dust ratio while the optimum moisture content was found to increase with increase in dust ratio.

4 Assessment of consolidation characteristics of the compacted laterite samples showed that the coefficient of consolidation and coefficient of volume compressibility of the samples was found to increase with increase in dust ratio while the compression index was found to decrease with increase in dust ratio./

5 The study therefore adjudged that dust ratio of laterite has significant effect on the consolidation properties of compacted laterite soils.

5.2 Recommendation

From the findings obtained on assessment of the effect of dust ratio on consolidation characteristics of compacted laterite soils, the following recommendation can be made:

1 Lateritic soils containing significant amount of dust ratio should be subjected to treatment such as mechanical stabilization using granular materials like sand as this will help reduce the compressibility behaviour of such soils and also undue settlement of structures built on such soils.

2 It is also important to ensure that the samples are compacted at the optimum level during field compaction as this will also help to reduce the compressibility characteristics of the soil and settlement of structures supported by such soils.

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APPENDICES

APPENDIX A

Sieve Analysis Test

Table A1: Sieve Analysis Test Results for Sample A with Dust Ratio of 0.28

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum % Retained	Cum % Finer
2	0.01	0.00	0.002	99.998
1.18	4.95	0.99	0.99	99.008
0.85	14.04	2.81	3.80	96.2
0.6	33.1	6.62	10.42	89.58
0.425	71.5	14.30	24.72	75.28
0.03	100.3	20.06	44.78	55.22
0.15	175.18	35.04	79.82	20.184
0.075	20.02	4.00	83.82	16.18
Tray	2.13	2.25	86.07	13.928

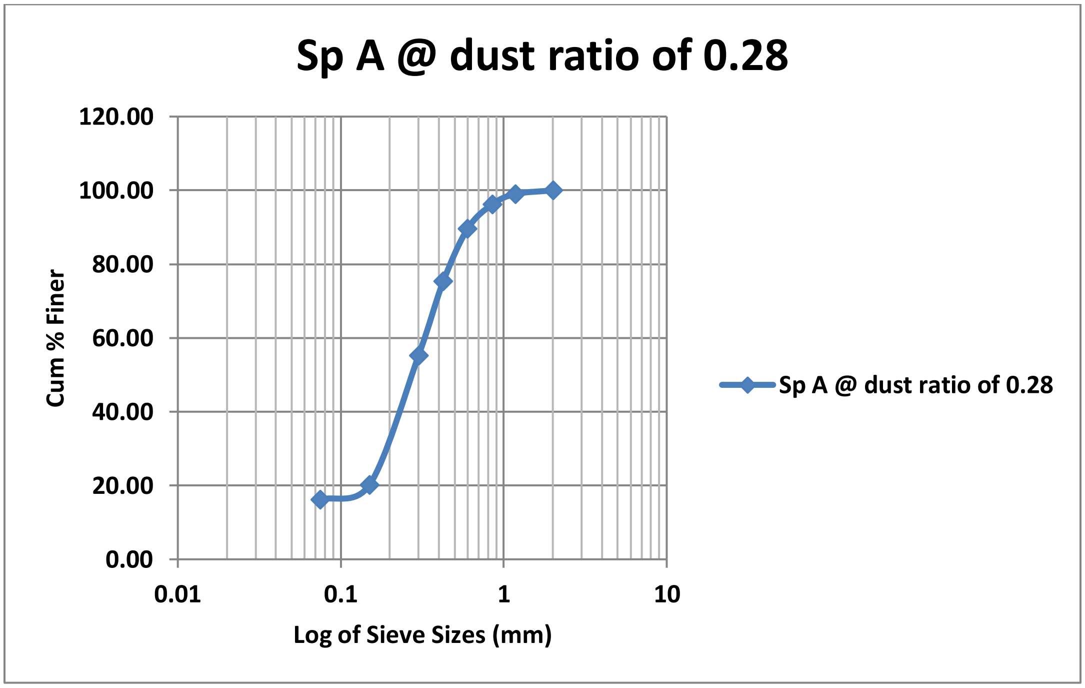


Figure A1: Particle Size Distribution Curve for Sample A

Table A2: Sieve Analysis Test Results for Sample B with Dust Ratio of 0.59

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum % Retained	Cum % Finer
2	0.03	0.006	0.006	99.994
1.18	5.04	1.008	1.014	98.986
0.85	13.78	2.756	3.77	96.23
0.6	32.95	6.59	10.36	89.64
0.425	39.34	7.868	18.228	81.772
0.03	85.14	17.028	35.256	64.744
0.15	84.09	16.818	52.074	47.926
0.075	23.21	4.642	56.716	43.284
Tray	0.68	0.136	56.852	43.148
Total	500			

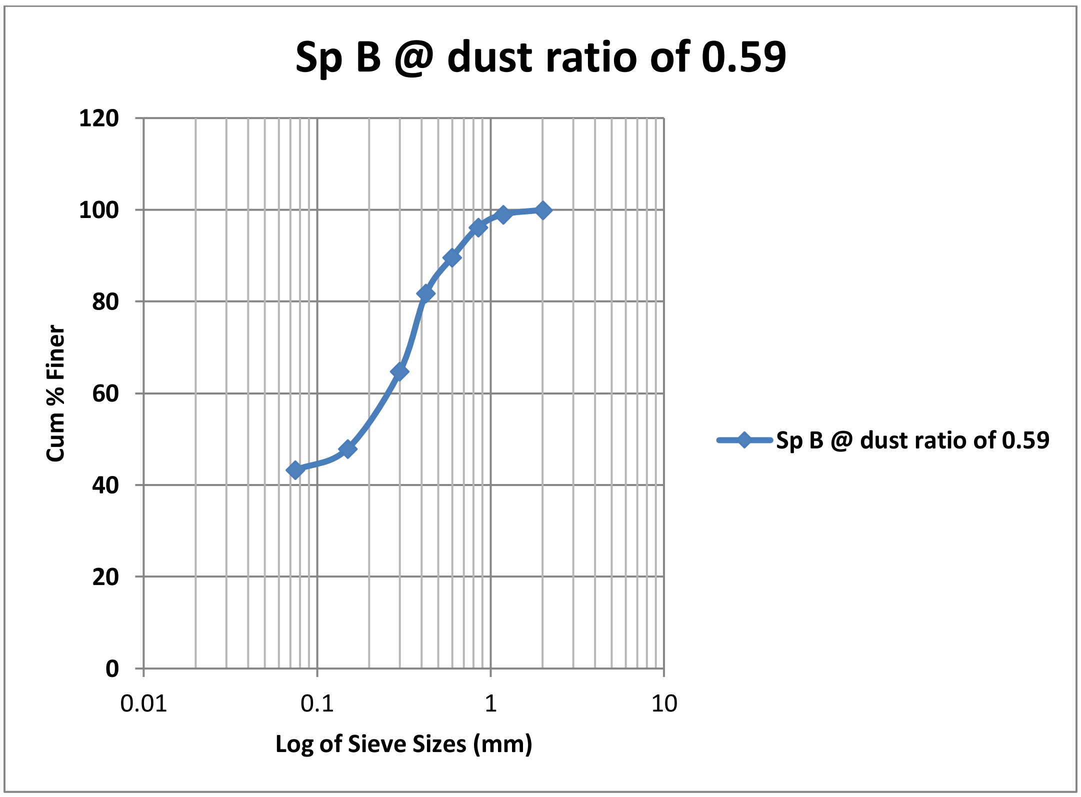


Figure A2: Particle Size Distribution Curve for Sample B

Table A3: Sieve Analysis Test Results for Sample C with Dust Ratio of 0.79

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum % Retained	Cum % Finer
2	0.39	0.078	0.078	99.922

1.18	2.34	0.468	0.546	99.454
0.85	8.41	1.682	2.228	97.772
0.6	22.2	4.44	6.668	93.332
0.425	35.19	7.038	13.706	86.294
0.03	39.32	7.864	21.57	78.43
0.15	48.52	9.704	31.274	68.726
0.075	27.8	5.56	36.834	63.166
Tray	7.76	1.552	38.386	61.614
Total	500	100	138.386	-38.386

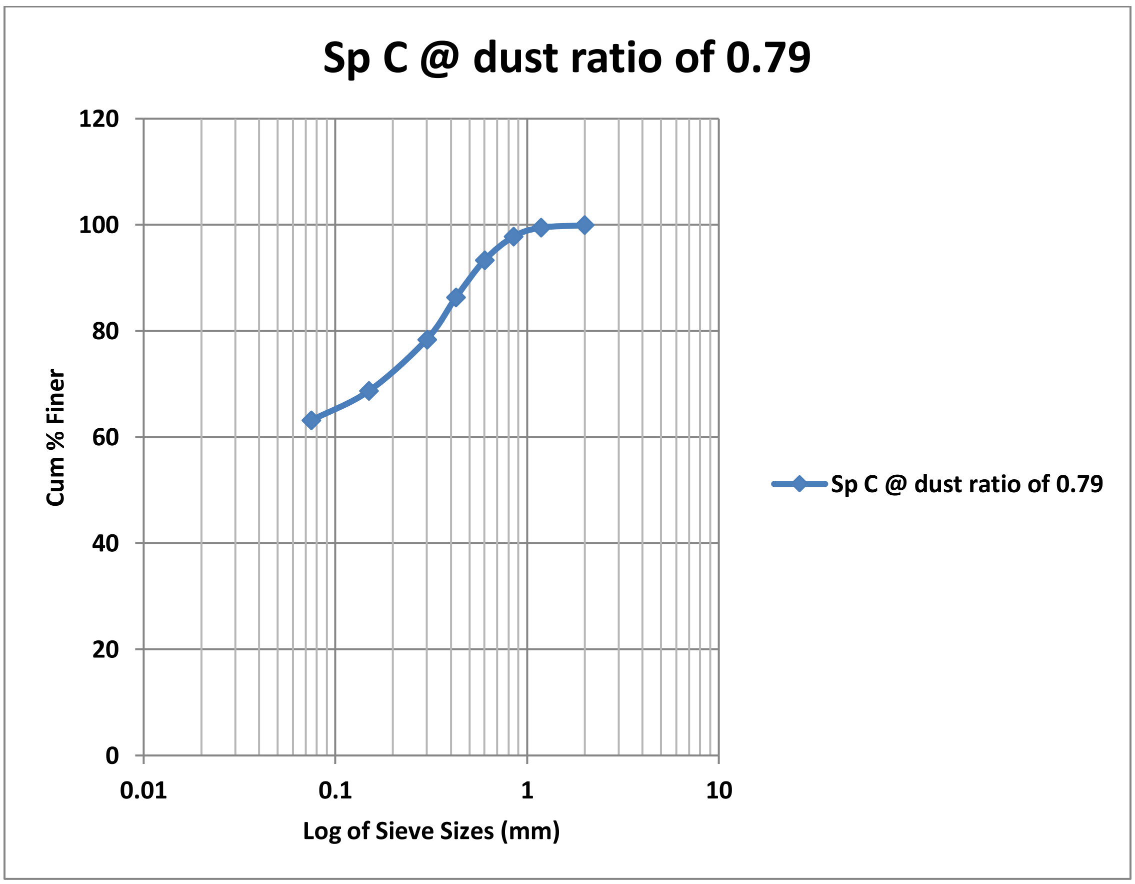


Figure A3: Particle Size Distribution Curve for Sample B

APPENDIX B

Compaction Test

Table B1: Dry Unit Weight Results for Sample A with Dust Ratio of 0.28

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.65	1.65	16.19	4.13	15.54
8	0.001	4	6	2	19.62	6.43	18.43
12	0.001	4	6.15	2.15	21.09	7.86	19.55
16	0.001	4	6.2	2.2	21.58	12.94	19.11
20	0.001	4	6.15	2.15	21.09	15.53	18.26

Table B1.1: Moisture Content Test Results (Top) for Sample A with Dust Ratio of 0.28

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.7	31.13	15.43	30.53	14.83	0.6	4.05
8	14.07	38.5	24.43	37.04	22.97	1.46	6.36
12	17.79	47.27	29.48	45.85	28.06	1.42	5.06
16	14.36	56.93	42.57	52.08	37.72	4.85	12.86
20	15.07	62.54	47.47	55.95	40.88	6.59	16.12

Table B1.2: Moisture Content Test Results (Bottom) for Sample A with Dust Ratio of 0.28

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	17.45	33.78	16.33	33.12	15.67	0.66	4.21
8	14.71	38.94	24.23	37.46	22.75	1.48	6.51
12	14.98	55.77	40.79	51.84	36.86	3.93	10.66
16	16.07	54.59	38.52	50.15	34.08	4.44	13.03
20	15.75	53.62	37.87	48.7	32.95	4.92	14.93

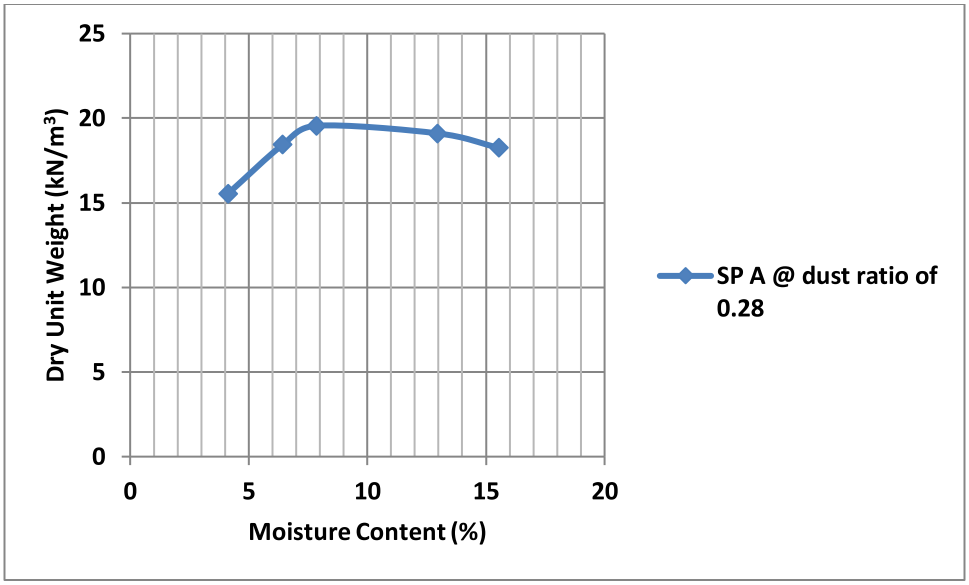


Figure B1: Compaction Curve for Sample A

Table B2: Dry Unit Weight Results for Sample B with Dust Ratio of 0.59

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.5	1.5	14.72	5.01	14.01
8	0.001	4	5.9	1.9	18.64	10.46	16.87
12	0.001	4	6	2	19.62	14.21	17.18
16	0.001	4	6.05	2.05	20.11	17.78	17.07
20	0.001	4	6	2	19.62	21.58	16.14

Table B2.1: Moisture Content Test Results for Sample B with Dust Ratio of 0.59

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.65	34.58	17.93	33.68	17.03	0.9	5.28
8	15.88	47.69	31.81	44.95	29.07	2.74	9.43
12	15.3	51	35.7	46.74	31.44	4.26	13.55
16	14.18	70.95	56.77	62.5	48.32	8.45	17.49

20	15.08	65.43	50.35	56.86	41.78	8.57	20.51

Table B2.2: Moisture Content Test Results (Bottom) for Sample B with Dust Ratio of 0.59

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.85	29.22	14.37	28.57	13.72	0.65	4.74
8	17.82	58.27	40.45	54.1	36.28	4.17	11.49
12	17.82	46.41	28.59	42.71	24.89	3.7	14.87
16	15.52	64.84	49.32	57.29	41.77	7.55	18.08
20	16.64	67.76	51.12	58.32	41.68	9.44	22.65

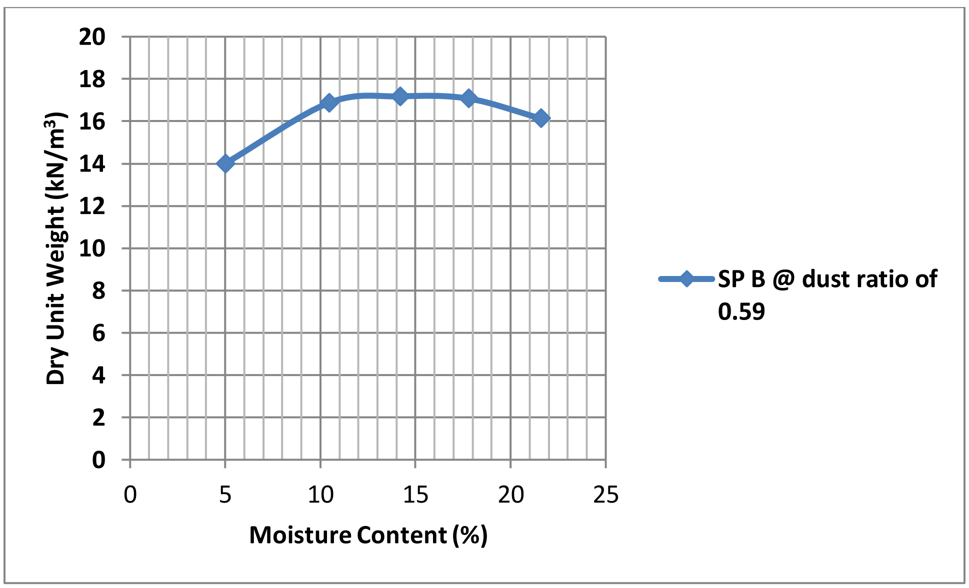


Figure B2: Compaction Curve for Sample B

Table B3: Dry Unit Weight Results for Sample C with Dust Ratio of 0.79

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)

4	**0.001**	**4**	**5.5**	**1.5**	**14.72**	**3.35**	**14.24**
8	**0.001**	**4**	**5.65**	**1.65**	**16.19**	**6.08**	**15.26**
12	**0.001**	**4**	**5.8**	**1.8**	**17.66**	**11.17**	**15.88**
16	**0.001**	**4**	**5.95**	**1.95**	**19.13**	**14.88**	**16.69**
20	**0.001**	**4**	**5.9**	**1.9**	**18.64**	**22.73**	**15.19**

Table B3.1: Moisture Content Test Results (Top) for Sample C with Dust Ratio of 0.79

Percentages of Water	**Wt of tin**	**Wt of tin + wet**	**Wt of wet Soil (g)**	**Wt of tin + dry Soil**	**Wt of dry**	**Wt of Water (g)**	**Moisture**
(%)	**(g)**	**Soil (g)**		**(g)**	**Soil (g)**		**Content (g)**
4	**14.71**	**28.34**	**13.63**	**27.95**	**13.24**	**0.39**	**2.95**
8	**15.03**	**40.04**	**25.01**	**39.04**	**24.01**	**1**	**4.16**
12	**14.31**	**46.22**	**31.91**	**43.06**	**28.75**	**3.16**	**10.99**
16	**17.82**	**65.65**	**47.83**	**59.34**	**41.52**	**6.31**	**15.20**
20	**13.56**	**63.34**	**49.78**	**52.78**	**39.22**	**10.56**	**26.93**

Table B3.2: Moisture Content Test Results (Bottom) for Sample C with Dust Ratio of 0.79

Percentages of Water	**Wt of tin**	**Wt of tin + wet**	**Wt of wet Soil (g)**	**Wt of tin + dry Soil**	**Wt of dry**	**Wt of Water (g)**	**Moisture**
(%)	**(g)**	**Soil (g)**		**(g)**	**Soil (g)**		**Content (g)**
4	**15.83**	**32.73**	**16.9**	**32.12**	**16.29**	**0.61**	**3.74**
8	**14.44**	**46.43**	**31.99**	**44.06**	**29.62**	**2.37**	**8.00**
12	**14.06**	**45.44**	**31.38**	**42.24**	**28.18**	**3.2**	**11.36**
16	**17.45**	**55.99**	**38.54**	**51.09**	**33.64**	**4.9**	**14.57**
20	**13.79**	**55.75**	**41.96**	**49.19**	**35.4**	**6.56**	**18.53**

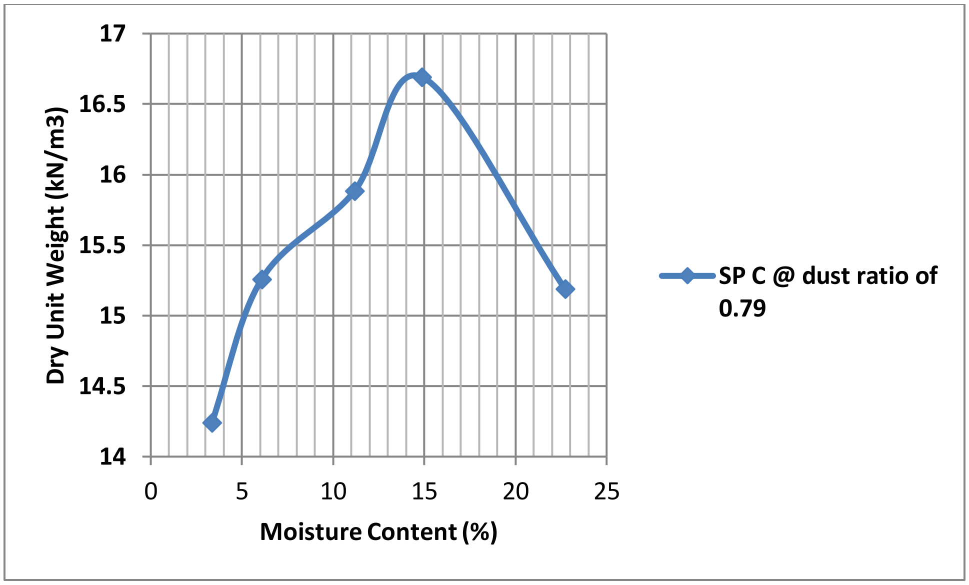


Figure B3: Compaction Curve for Sample C

APPENDIX C

Atterberg Limit Test

Table C1: Liquid Limit Test Results for Sample A

BLOWS	33	26	22	18	14
Wt of empty tin (g)	14.89	18.46	17.56	15.06	16.57
Wt of tin + wet soil (g)	45.05	50.57	47.44	40.93	49.21
Wt of wet soil (g)	30.16	32.11	29.88	25.87	32.64
Wt of tin +dry soil (g)	39.98	43.96	41.05	34.71	40.61
Wt of dry soil (g)	25.09	25.5	23.49	19.65	24.04
Wt of water (g)	5.07	6.61	6.39	6.22	8.6
Moisture Content (g)	20.21	25.92	27.20	31.65	35.77

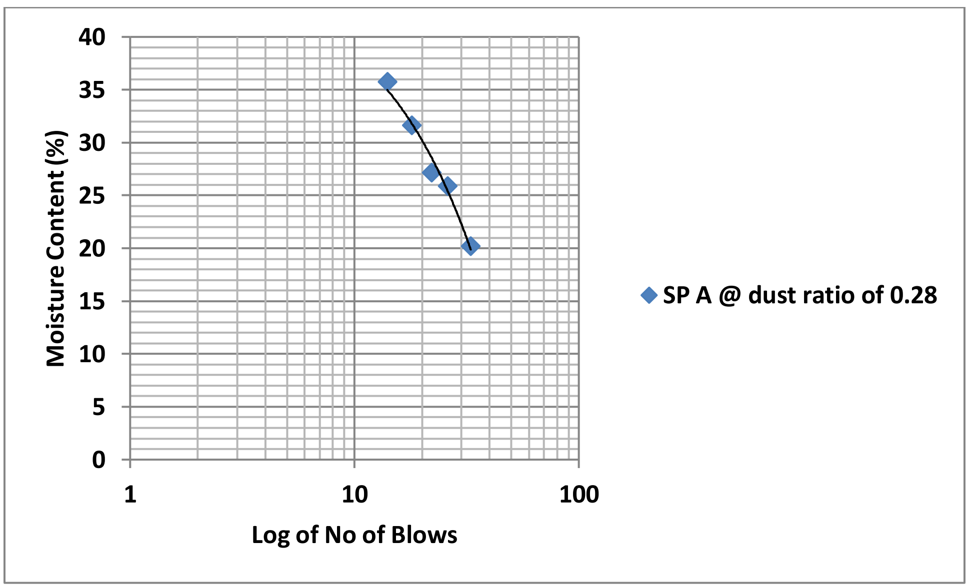


Figure C1: Liquid Limit Graph for Sample A

Table C2: Plastic Limit Results for Sample A

Sample A	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.63	15.42	16.88
Wt of tin + wet soil (g)	24.86	28.69	32.88

Wt of wet soil (g)	10.23	13.27	16
Wt of tin + dry soil (g)	23.49	27.03	30.47
Wt of dry soil (g)	8.86	11.61	13.59
Wt of water (g)	1.37	1.66	2.41
Plastic Limit (%)	15.46	14.30	17.73

Table C3: Liquid Limit Results for Sample B

BLOWS	33	28	23	17	13
Wt of empty tin (g)	14.18	16.42	17.16	15.49	16.04
Wt of tin + wet soil (g)	50.16	44.28	46.28	48.56	40.82
Wt of wet soil (g)	35.98	27.86	29.12	33.07	24.78
Wt of tin +dry soil (g)	42.68	37.69	38.59	39.45	33.67
Wt of dry soil (g)	28.5	21.27	21.43	23.96	17.63
Wt of water (g)	7.48	6.59	7.69	9.11	7.15
Moisture Content (g)	26.25	30.98	35.88	38.02	40.56

Figure C2: Liquid Limit Graph for Sample B

Table C4: Plastic Limit Results for Sample B

Sample B	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.62	14.7	13.64
Wt of tin + wet soil (g)	26.84	30.52	34.88
Wt of wet soil (g)	11.22	15.82	21.24
Wt of tin + dry soil (g)	25.02	28.44	31.61
Wt of dry soil (g)	9.4	13.74	17.97
Wt of water (g)	1.82	2.08	3.27
Plastic Limit (%)	19.36	15.14	18.20

Table C5: Liquid Limit Results for Sample C

BLOWS	33	26	22	18	14
Wt of empty tin (g)	14.82	15.64	16.48	14.88	15.21
Wt of tin + wet soil (g)	46.24	34.62	32.28	26.94	30.66
Wt of wet soil (g)	31.42	18.98	15.8	12.06	15.45
Wt of tin +dry soil (g)	39.14	29.78	27.78	23.25	25.67
Wt of dry soil (g)	24.32	14.14	11.3	8.37	10.46
Wt of water (g)	7.1	4.84	4.5	3.69	4.99
Moisture Content (g)	29.19	34.23	39.82	44.09	47.71

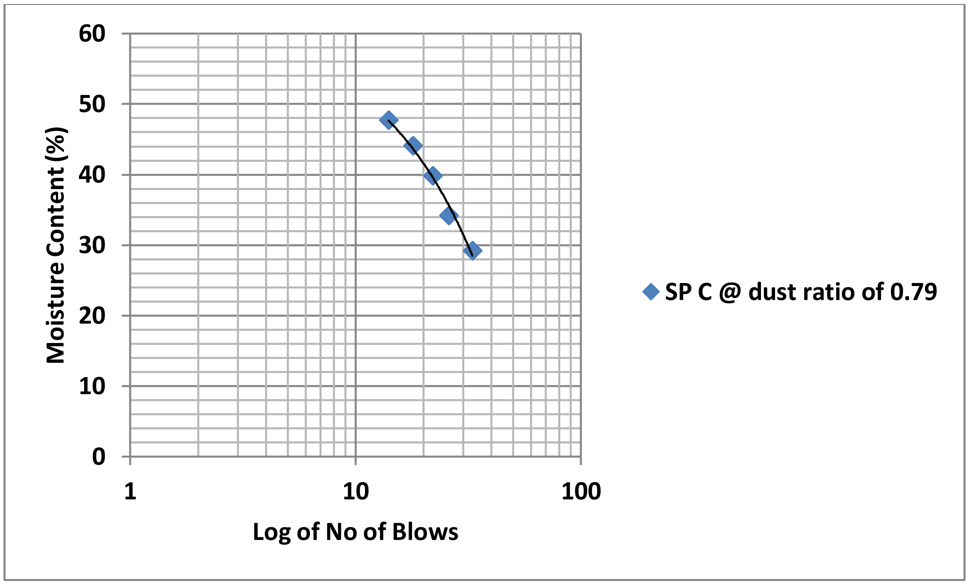


Figure C3: Liquid Limit Graph for Sample C

APPENDIX D

Table D1: Results of Compression against Time for Sample A @ dust ratio of 0.28

Time	1st day	2nd day	3rd day	4th day	5th day	6th day	7th day	8th day
0	1000	956.1	928.7	893.1	860.8	819.8	Unloading	saturation
15sec	978.2	950.2	922.1	884.2	846.8	808.2		
25sec	974.3	944.8	918.3	880.5	843.5	800.7		
36sec	970.2	942.3	912.2	878.3	840.1	798.5		
1min	968.1	941.1	908.5	875.4	838.3	796.4		
2.25min	967.2	940	907.2	874.5	836.1	795.3		
4min	966.3	939.3	906.4	873.2	835.3	794.1		
9min	965.4	938.1	905.1	872.1	834.2	792.8		
16min	964.1	937.4	904.8	871.3	833.5	790.4		
25min	963.7	936.3	903.2	870.4	832.4	788.6		
36min	962.8	935.4	902.1	869.3	830.8	786.2		
49min	962	934.1	901.2	868.5	828.4	784.7		
64min	961.3	933.5	900.4	867.2	826.6	782.6		
89min	960.2	932.8	898.5	866.1	825.1	780.3		
100min	959.4	932	897.1	865.4	824	778.2		
121min	958.8	931.4	896	864.2	823.2	777.1		
144min	957.8	930.2	895.4	863.5	822.4	776		
169min	957.2	929.1	894.2	862.1	821.5	775.2		
1440mm	956.1	928.7	893.1	860.8	819.3	773.8		

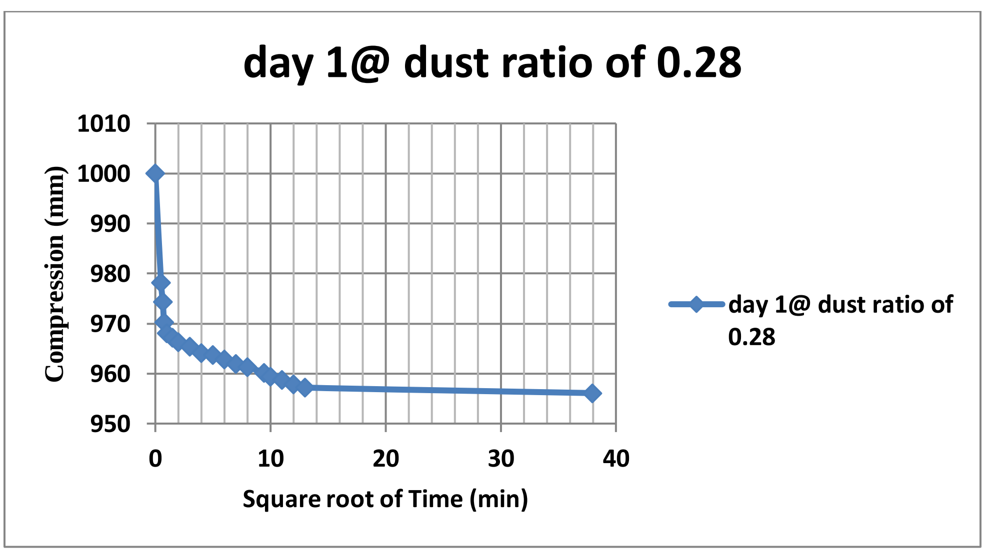


Figure D1: Compression Test Reading for Day 1

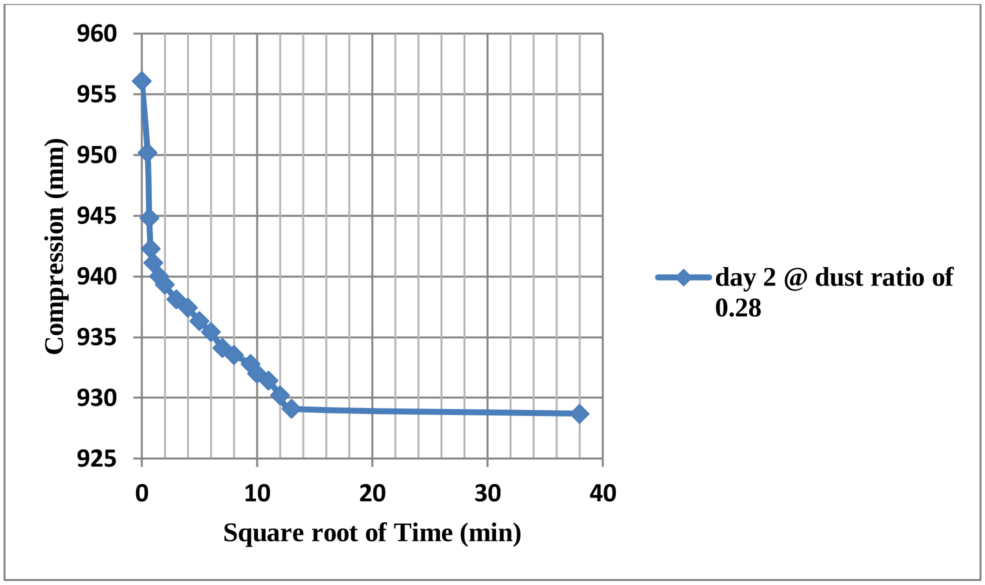


Figure D2: Compression Test Reading for Day 2

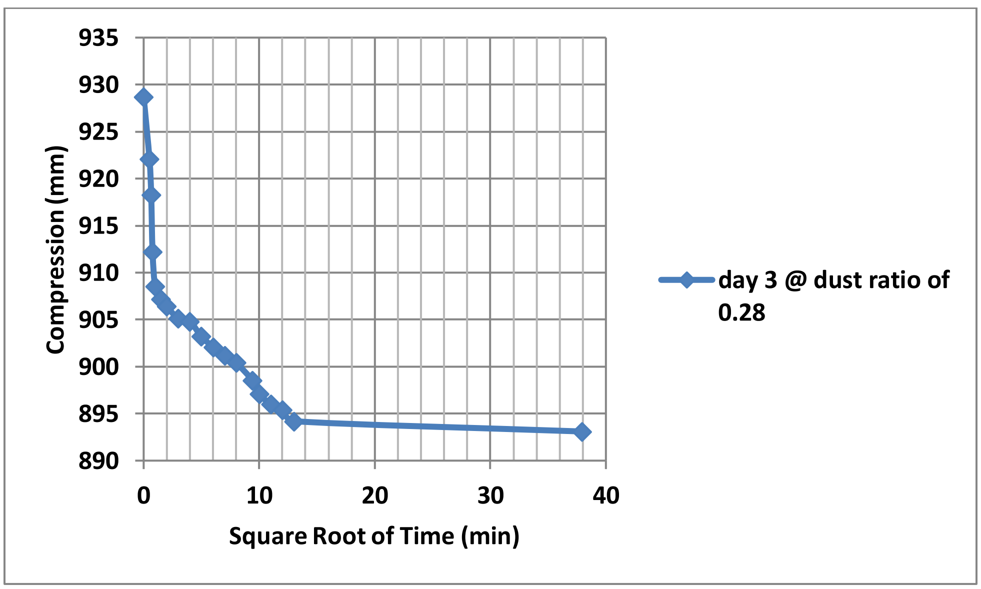


Figure D3: Compression Test Reading for Day 3

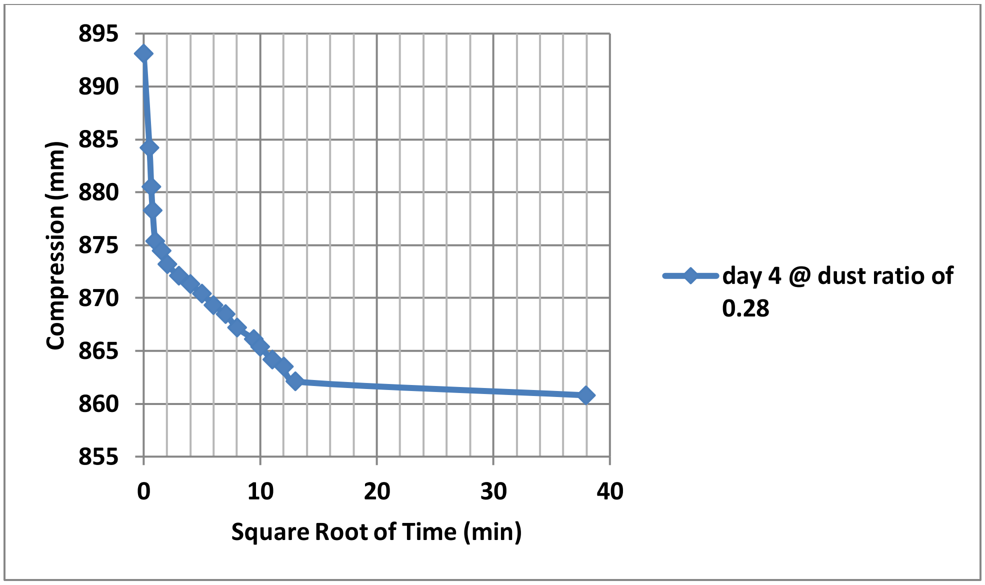


Figure D4: Compression Test Reading for Day 4

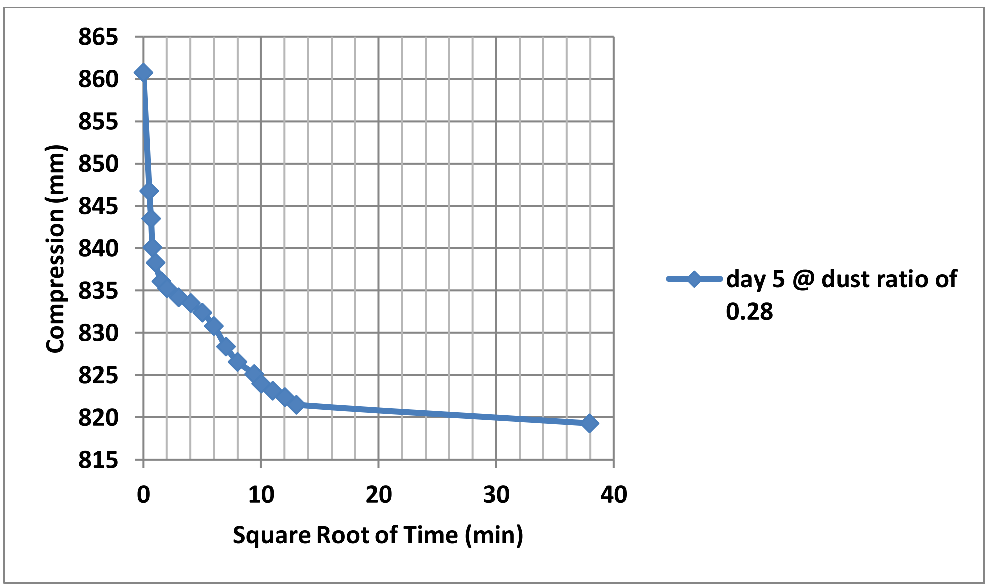


Figure D5: Compression Test Reading for Day 5

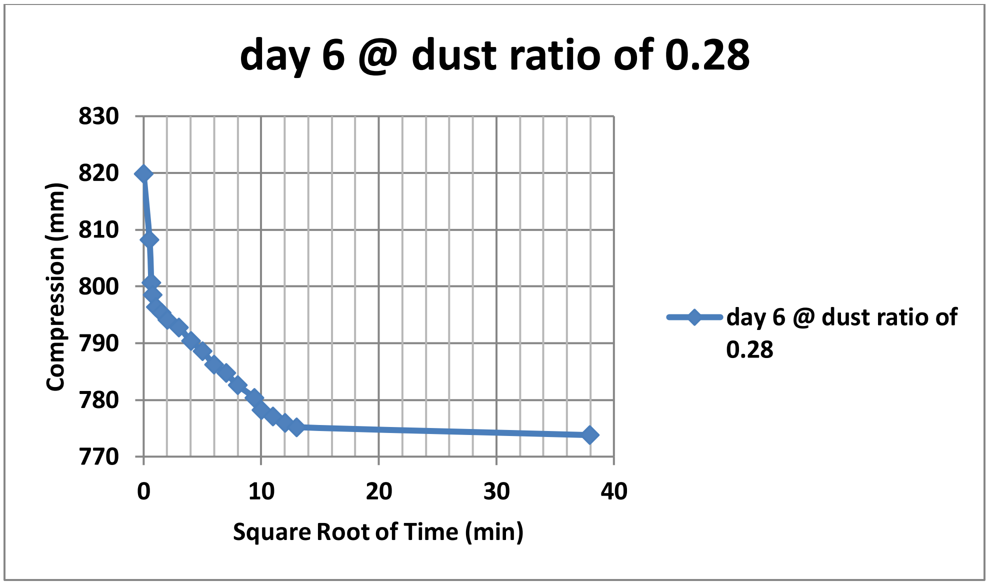


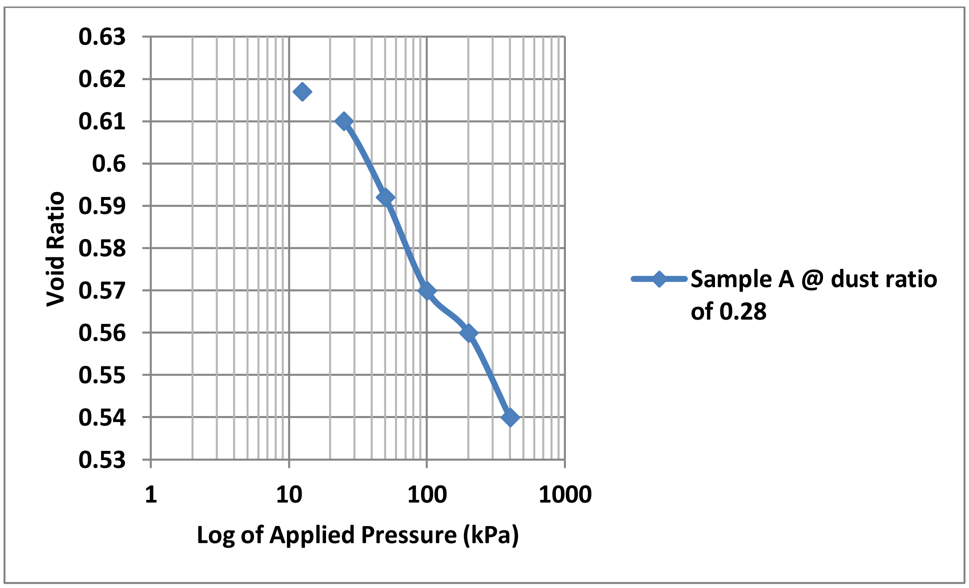
Figure D6: Compression Test Reading for Day 6

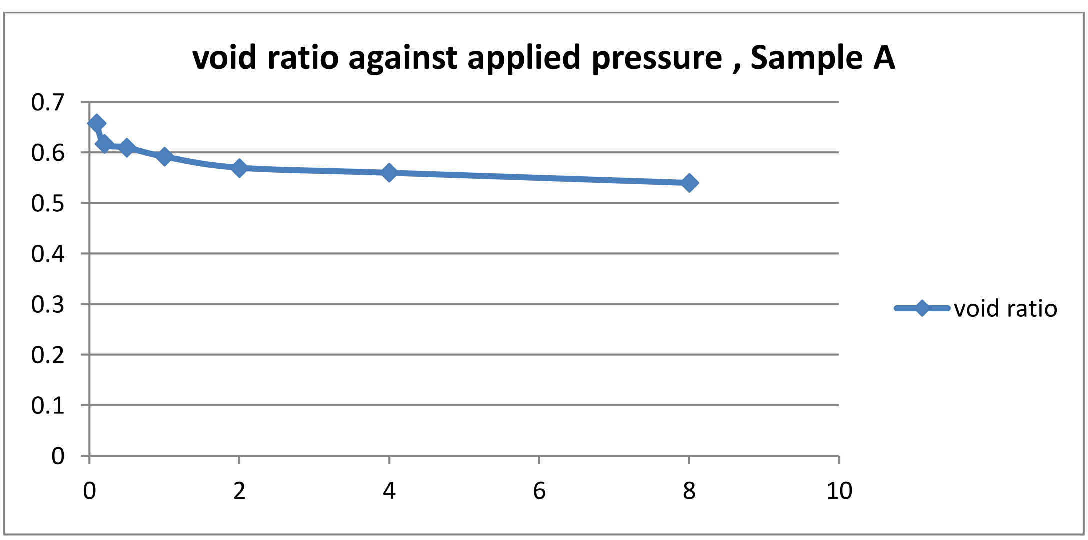
Table D2: Results of Compression against Time for Sample B @ dust ratio of 0.59

Time	1st day	2nd day	3rd day	4th day	5th day	6th day
0	1000	964.8	924.2	875.2	808.5	768.1
15sec	982.4	945.3	914.3	860.4	801.8	760.2
25sec	981.2	942.8	911.8	852.1	793.5	755.3
36sec	978	939.7	906.3	848.3	790.2	754.8
1min	977	937.2	901.5	835.8	783.7	752.1
2.25min	976	936.4	898.7	832.1	780.4	750.2
4min	974.2	935.6	894.2	829.6	778.3	749.3
9min	972	934.2	890.3	825.4	777.2	748.6
16min	971.8	933.8	888.2	821.3	776.8	747.1
25min	970.5	933.2	885.7	819.7	775.4	746.8
36min	969.2	932	884.8	818.3	774.8	746
49min	969	931.1	883.7	817.2	774	745.3
64min	968.4	930.5	882.3	816.8	773.2	744.1
89min	967	929.2	881.7	815.3	772.8	743.2
100min	966.8	928.3	880.1	814.7	771.4	742.1
121min	966.2	927.8	879.8	814	771	741.8
144min	965.3	927	877.2	811.2	770.3	740.3
169min	964.8	926.4	876.8	810.7	769.4	739.4
1440mm	960.2	924.2	875.2	808.5	768.1	737.1

Table 1.0 Determination of Void Ratio for Sample A

Days	Dial Reading	de	ΔH	H - ΔH	Void ratio (e)	Pressure (P)
day 0	1000	7.81	0.439	20	0.658	0
day 1	956.1	7.81	0.274	19.561	0.617	12.5
day 2	928.7	7.81	0.356	19.205	0.61	25
day 3	893.1	7.81	0.323	18.882	0.592	50
day 4	860.8	7.81	0.415	18.467	0.57	100
day 5	819.3	7.81	0.455	18.012	0.56	200
day 6	773.8	7.81	0	19.652	0.54	400





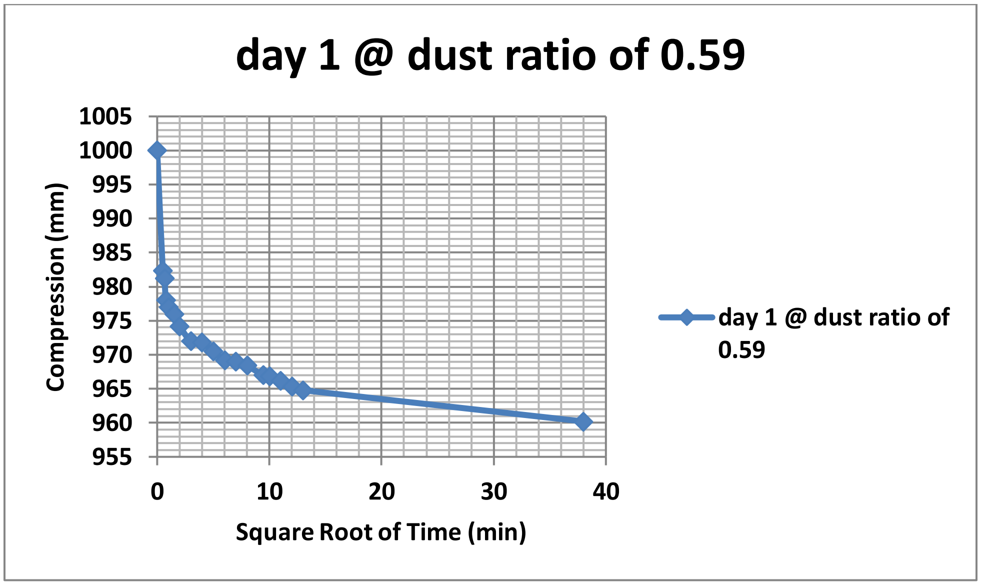


Figure D7: Compression Test Reading for Day 1

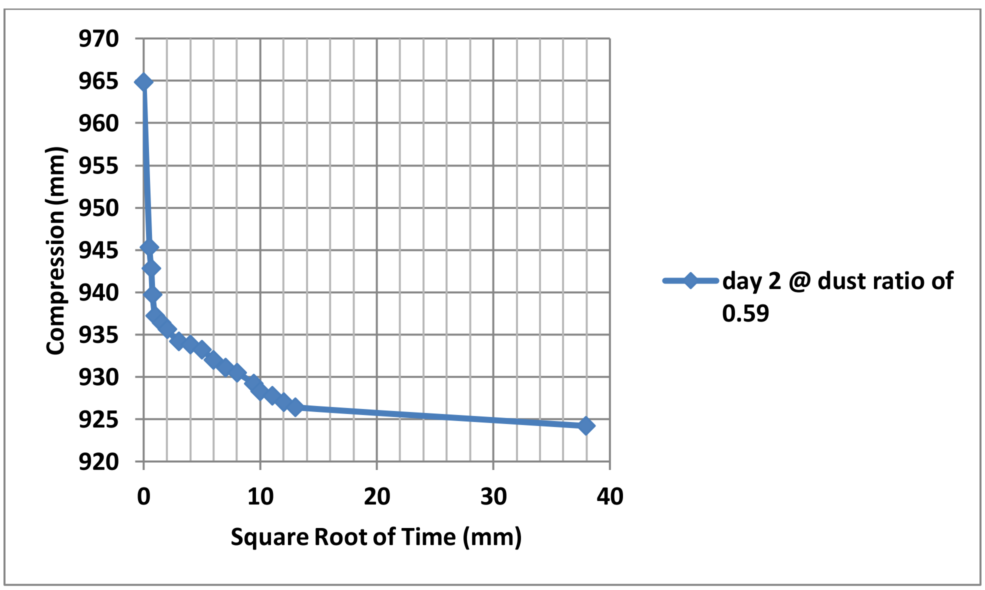


Figure D8: Compression Test Reading for Day 2

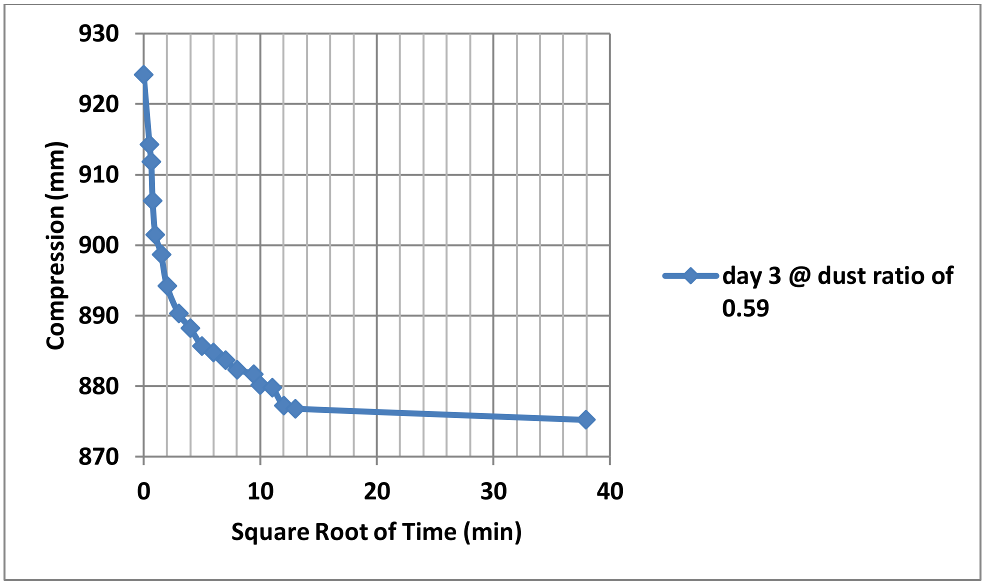


Figure D9: Compression Test Reading for Day 3

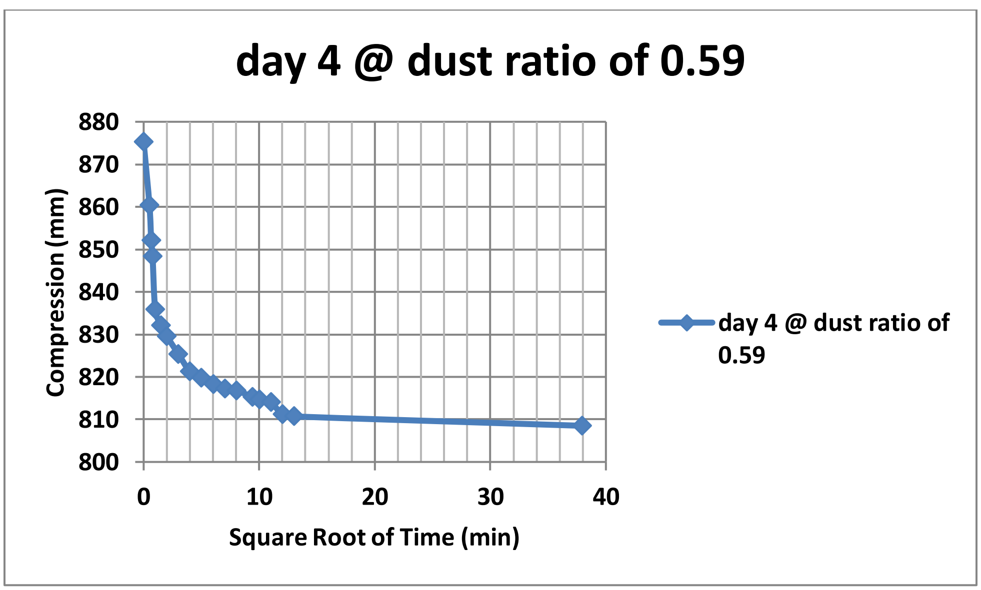


Figure D10: Compression Test Reading for Day 4

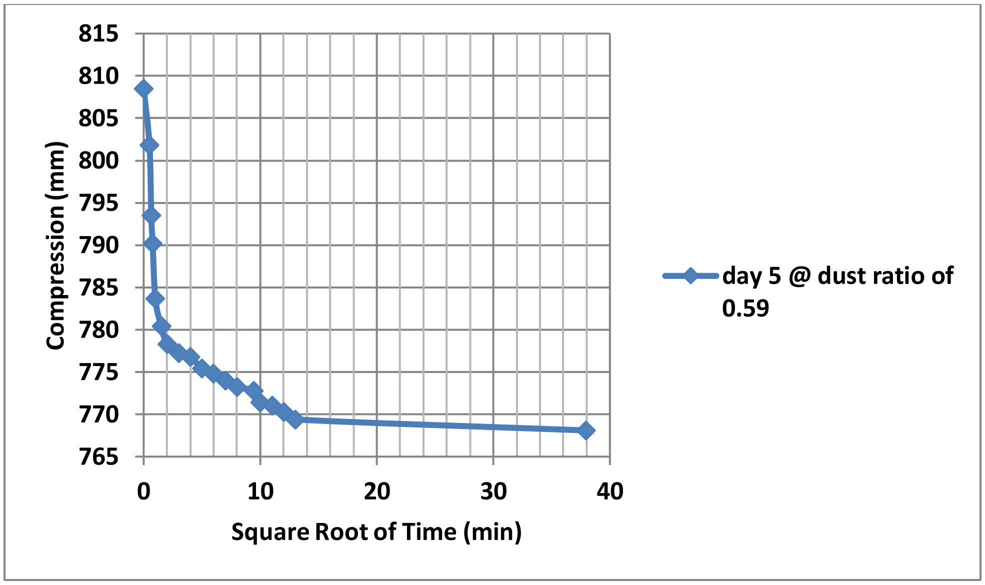


Figure D11: Compression Test Reading for Day 5

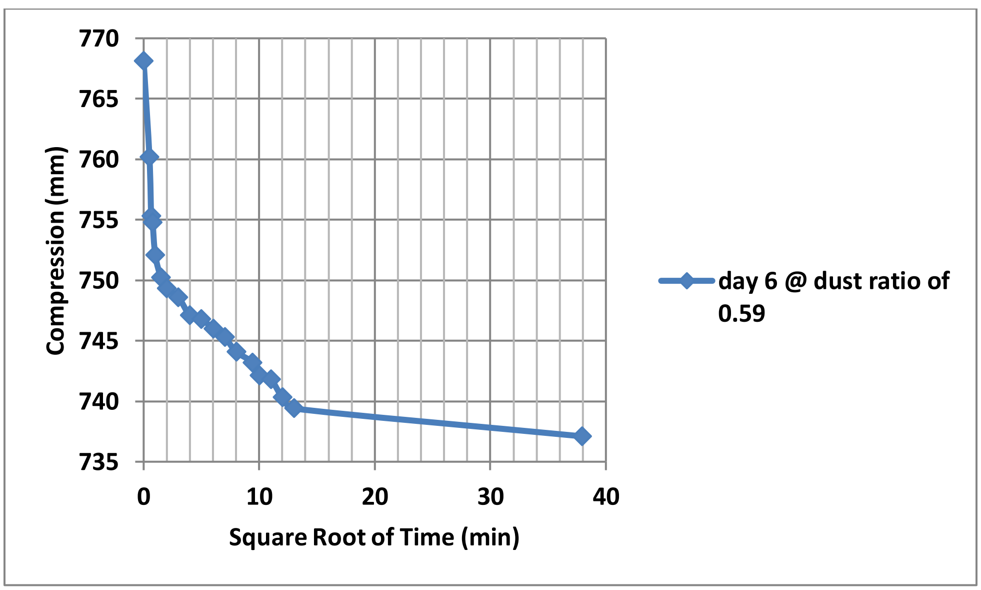


Figure D12: Compression Test Reading for Day 6

Table D3: Results of Compression against Time for Sample C @ dust ratio of 0.79

Time	1st day	2nd day	3rd day	4th day	5th day	6th day	7th day	8th day
0	1000	941.3	910.7	877.5	821.2	777.4	Unloading	saturation

15sec	970.4	933.8	903.4	858.5	812.4	758.3		
25sec	966.3	930.1	900.1	854.2	808.9	752.1		
36sec	962.1	928.7	898.3	850.3	804.3	748.2		
1min	960.4	926.2	897.3	847.1	800.5	746.8		
2.25min	958.3	925.1	895.2	845.2	796.8	745.2		
4min	955.2	924.3	893.4	840.9	794.7	744.1		
9min	953.8	923.2	892.1	838.7	792.4	743		
16min	952.4	922.4	890.4	836.1	791.3	742.8		
25min	951.3	921.1	888.5	834.8	790.1	741.1		
36min	950	920.2	887.1	832.7	788.2	740.2		
49min	948.7	919.3	886.2	830.6	786.3	739.3		
64min	947.2	918	885.4	829.1	784.8	738.2		
89min	946.1	917.1	884.2	828.3	785.6	737		
100min	945.8	916.4	883.1	827.4	783.7	736.1		
121min	944.2	915.3	882.4	825.1	782.1	735.3		
144min	943.5	914.2	881.1	824.2	781.4	734.1		
169min	942.1	912.8	880.4	823.1	779.2	732.8		
1440mm	941.3	910.7	877.5	821.2	777.4	730.2		

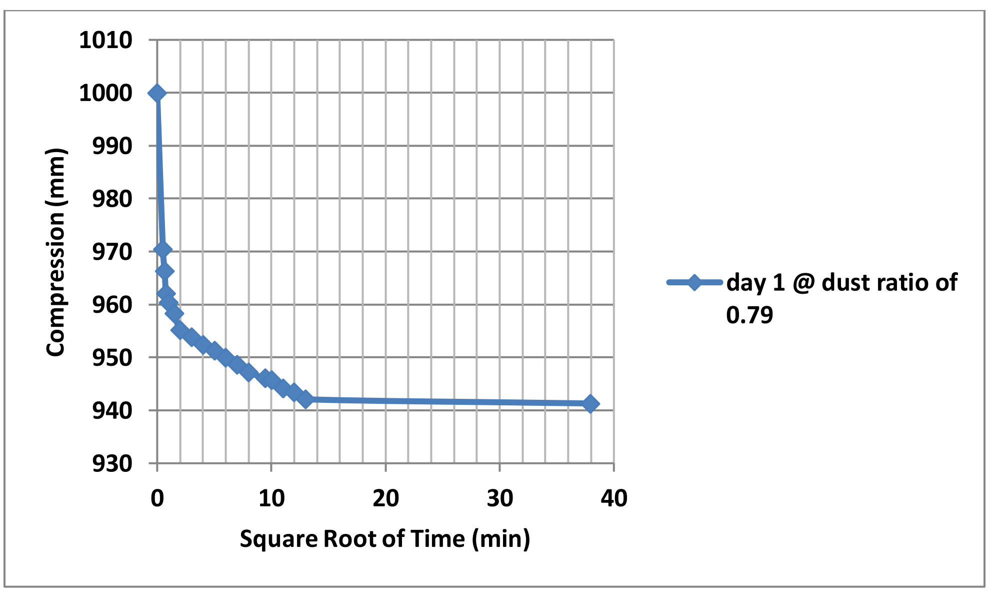


Figure D13: Compression Test Reading for Day 1

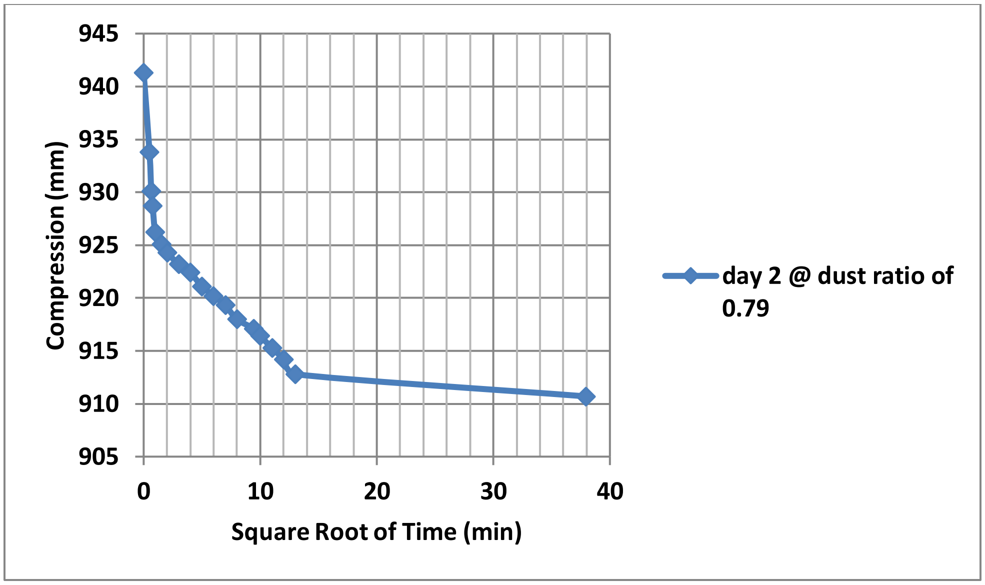


Figure D14: Compression Test Reading for Day 2

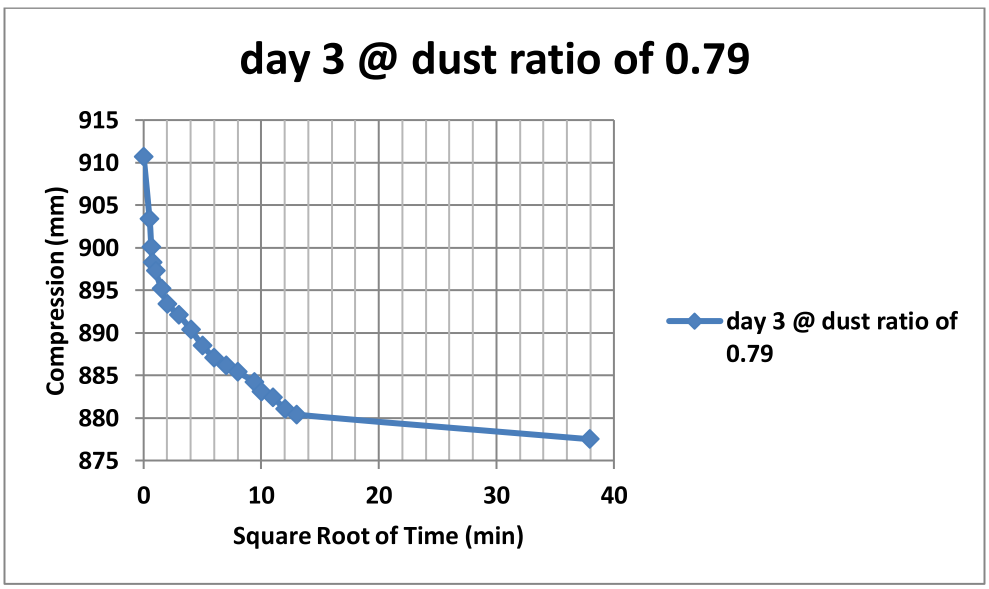


Figure D15: Compression Test Reading for Day 3

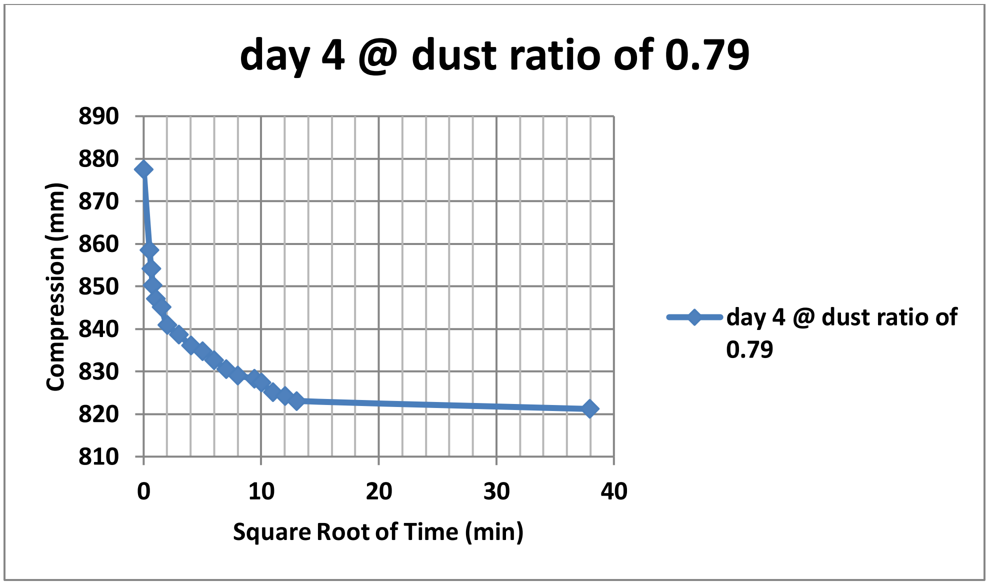


Figure D16: Compression Test Reading for Day 16

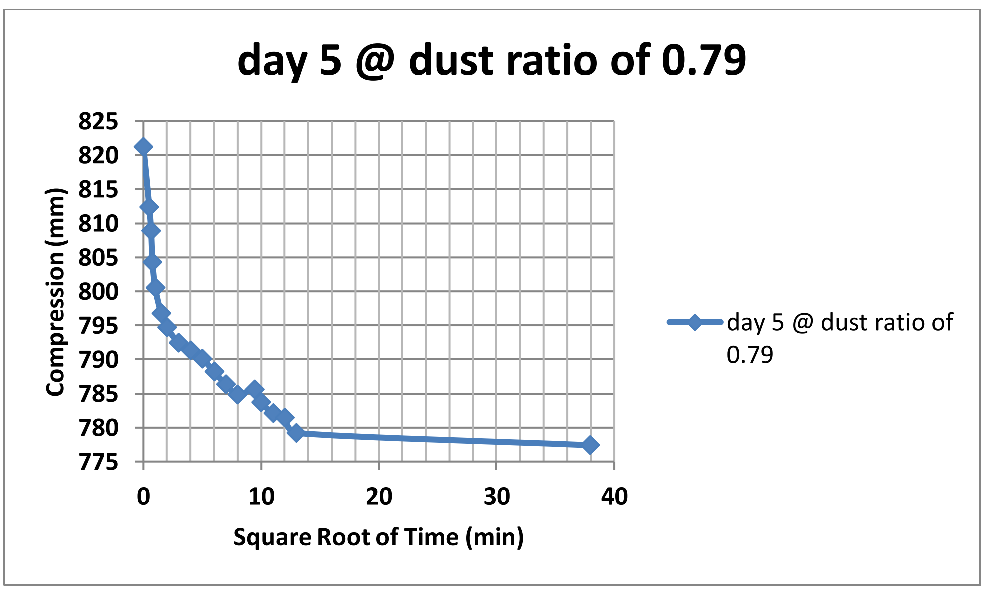


Figure D17: Compression Test Reading for Day 5

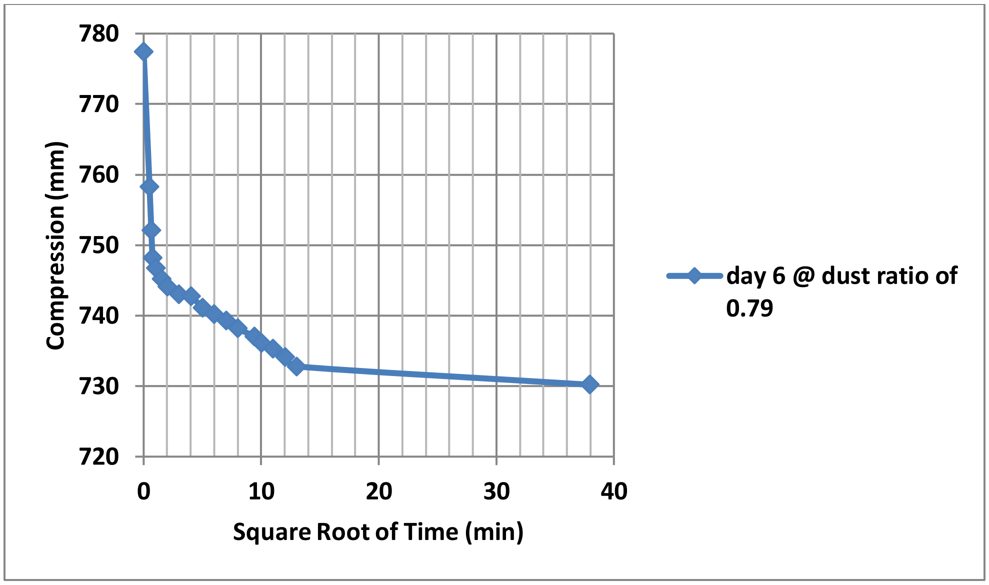


Figure D18: Compression Test Reading for Day 6

APPENDIX E

Specific Gravity Test

Table E1. Specific Gravity Result for sample A @ dust ration of 0.28

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.76	25.64	25.90
Wt of bottle + dry soil, W_2 (g).	34.74	35.63	35.90
Wt of bottle + soil + water, W_3 (g).	84.33	85.15	85.79
Wt of bottle + water, W_4 (g).	78.07	78.94	79.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample A

$$\text{Trial 1 } (G_{S1}) = \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.74-24.76)}{(34.74-24.76)-(84.33-78.07)} = 2.68$$

$$\text{Trial 2 } (G_{S2}) = \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.63-25.64)}{(35.63-25.64)-(85.15-78.94)} = 2.64$$

$$\text{Trial 3 } (G_{S3}) = \frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.90-25.90)}{(35.90-25.90)(85.79-79.56)} =2.65$$

$$\text{Specific Gravity} = \frac{(GS1+GS2+GS3)}{3} = \frac{(2.68+2.64+2.65)}{3} = 2.66$$

Table E2. Specific Gravity Result for sample B @ dust ratio of 0.59

Determinants	Trial 1	Trial 2	Trial 3
Wt of density	23.87	25.21	25.54

bottle, W_1 (g).			
Wt of bottle + dry soil, W_2 (g).	**33.86**	**35.21**	**35.54**
Wt of bottle + soil + water, W_3 (g).	**82.91**	**81.13**	**79.94**
Wt of bottle + water, W_4 (g).	**76.72**	**74.95**	**73.77**

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample B

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(33.86-23.87)}{(33.86-23.87)-(82.91-76.72)}$ **= 2.63**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.21-25.21)}{(35.21-25.21)-(81.13-74.95)}$ **= 2.62**

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.54-25.54)}{(35.54-25.54)(79.94-73.77)}$ **=2.61**

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.63+2.62+2.61)}{3}$ **= 2.62**

Table E3. Specific Gravity Result for sample C @ dust ratio of 0.79

Determinants	**Trial 1**	**Trial 2**	**Trial 3**
Wt of density bottle, W_1 (g).	**24.50**	**25.32**	**25.12**
Wt of bottle + dry soil, W_2 (g).	**34.48**	**35.31**	**35.10**
Wt of bottle + soil + water, W_3 (g).	**84.43**	**86.39**	**85.03**
Wt of bottle + water, W_4 (g).	**78.35**	**80.32**	**78.93**

The Specific gravity of the sample is calculated as follows:

Specific Gravity for Sample C

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.48-24.50)}{(34.48-24.50)-(84.43-78.35)}$ **= 2.56**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.31-25.32)}{(35.31-25.32)-(86.39-80.32)}$ **= 2.55**

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.10-25.12)}{(35.10-25.12)(85.03-78.93)}$ **=2.53**

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.56+2.55+2.53)}{3}$ **= 2.55**