GEOTECHNICAL PROPERTIES OF LATERITIC SOIL – BENTONITE MIXTURE TREATED WITH BAMBOO LEAF ASH

BY

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SUBMITTED TO

THE DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

NNAMDI AZIKIWE UNIVERSITY AWKA.

IN PARTIAL FUFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG) DEGREE IN CIVIL ENGINEERING

MAY, 2023

CERTIFICATION

This is to certify that the project topic titled “Geotechnical Properties of Lateritic Soil-Bentonite Mixtures Treated with Bamboo Leaf Ash” was done by **Maduka Ebuka Johnpeter** with registration number (NAU/2017224013) in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

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APPROVAL PAGE

This research work “Geotechnical Properties of Lateritic Soils-Bentonite Mixtures Treated with Bamboo Leaf Ash” 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 for the gift of life and also for guiding me through school. I also want to dedicate this work to my parents Mr and Mrs Maduka who served as a real source of inspiration toward my academic pursuit.

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.

Also I will like to express my profound gratitude to my parents; Mr and Mrs Maduka for their moral support, constant prayers throughout my stay in school.

I want to thank in a very special way, my project supervisor in the person of Engr. Prof. C. M. O. Nwaiwu for his time and guidance in the accomplishment of this project. May the lord rain blessing from heaven for him and his family and may he 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 investigated the geotechnical properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash. Sodium bentonite was used for the study and was added to lateritic soil in an increasing order of 5%, 10%, 15% and 20% by weight of lateritic soil. The mixture (laterite + bentonite) was treated with bamboo leaf ash. Bamboo leaf ash was added to lateritic soil-bentonite mixtures in an increasing order of 4%, 8%, 12% and 16% by weight of lateritic soil. The mixtures were subjected to geotechnical testing. The tests include: sieve analysis test, specific gravity test, compaction test, Atterberg limits (liquid and plastic) test, UCS test, hydraulic conductivity test and volumetric shrinkage test. Results obtained from sieve analysis test revealed that the lateritic soil was classified as A-7-6 according to AASHTO Soil Classification System and CH (clay of high plasticity) according to Unified Soil Classification System. The specific gravity of lateritic soil increased on addition of bentonite while for lateritic soil-bentonite mixtures treated with bamboo leaf ash the specific gravity was found to decrease. Volumetric shrinkage strain test was conducted after drying for 5 days, the VSS increased with higher compaction water content. The maximum dry unit weight of lateritic soil increased marginally on addition of bentonite to lateritic soil while for lateritic soil-bentonite mixtures treated with bamboo leaf ash, an increasing and decreasing trend in maximum dry unit weight was observed. The results showed improvement in the plasticity index (PI) and UCS values of the treated soil. The PI decreased from 30.58% to 19.62% and UCS values increased from 103.86kN/m^2 to 245.15kN/m^2. Improvements observed in the strength properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash were above the minimum values specified for landfill liner construction materials. The study therefore recommends the use of lateritic soil-bentonite mixtures treated with bamboo leaf ash for use as material in the construction of landfill liners.

TABLE OF CONTENTS

Contents

CHAPTER ONE: INTRODUCTION

CHAPTER TWO: LITERATURE REVIEW

CHAPTER ONE

INTRODUCTION

1.1 Background of Study

During the last decade, the global demand for indigenous lateritic soil has continued to increase (Amadi and Eberemu, 2013). This growing demand has generated interest in the use of this red tropical soil as a road construction and land fill material especially in developing countries like Nigeria. There have been innumerable cases of pavement failures and waste disposal problems in Nigeria, which have been largely attributed to the deficiencies in properties of laterite obtained at different location and at different depth (Amadi and Eberemu, 2013). Hence modification of certain undesirable properties of laterite became necessary as the only cost effective means for civil engineering application. Some of the benefits associated with soil modification include: increased stability and strength of the soil, reduced compressibility and void ratio. Among various alternative of soil modification, the only economically viable means is the use of suitable material known as stabilizer.

Soil stabilization is any treatment whether technical or compactive, applied to improve the engineering properties of the soil such as strength, waterproofing property and durability Ayobami, et al., (2018). It was also described as the treatment of natural soil to improve its engineering properties (Garber and Hoel, 2000). In general, soil stabilization is the process of creating or improving certain desired properties in a soil material so as to render it stabled and useful for a specific purpose (Olugbenga and Adetuberu, 2013). Soil stabilization may be broadly defined as the alteration or preservation of one or more soil properties to improve the engineering characteristics and performance of a soil. There are three purposes for soil stabilization which includes; strength improvement, dust control and soil waterproofing. Engineers are responsible for selecting or specifying the correct stabilizing material, method, technique, and quantity of material required. Many procedures have been outlined for making correct decisions in selection but most of them are not precise. Soils vary throughout the world, and the engineering properties of soils are equally variable. The key to success in soil stabilization is soil testing (Olugbenga and Adetuberu, 2013). The method of soil stabilization selected should be verified in the laboratory before construction and preferably before specifying or ordering materials (Olugbenga and Adetuberu, 2013).

To provide long term containment and isolation of wastes from the environment required in engineered landfills, the design of waste cells in waste disposal facilities should ensure minimum fluid flow through or into the repository systems over the designed lifetime and prevent leachate migration out of the repository. Such designs typically consist of a compacted clay layer, geomembrane, geosynthetic clay layer, and/or a combination of these in a multi barrier system (Rowe and Quigley, 1995: Shackelford and Nelson, 1996). Laterite when compared to active clay soils presents attractive option because of its greater shear strength properties, chemical resistance, better workability and availability within economic haul distance in tropical latitudes where they occur in abundance (BRRI, 1971: Charman, 1988: Frempong and Yanful, 2008: Osinubi and Nwaiwu, 2008). During Laterization, sesquioxides cause a physical cementation of fine particles into coarser aggregation resulting in a granular structure Townsend, et al., (1971). The consequences of these processes on the engineering characteristics of lateritic soils include low plasticity, high permeability, low swelling potential. This properties makes lateritic soil unfit for use as a soil liner and slurry walls (Amadi and Eberemu, 2013). Therefore, for lateritic soil to satisfy the hydraulic requirements and self sealing function, a combination with bentonite becomes necessary.

In other to better modify this lateritic soil-bentonite mixtures for civil engineering application (road construction, slurry walls and landfills) and also proffer solution to problems associated to disposal of waste, this study will therefore evaluate the properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash.

1.2 Statement of Problem

In Nigeria, laterite is the most common and effective material used for building, highway pavement, earth dam construction and landfills. Lateritic soil ranges in performance from excellent to poor based on their shear strength, reduced compressibility and low permeability. A poor performing laterite will posses low shear strength, high void ratio, high compressibility (swelling and shrinkage), high permeability and reduced soil density. Pavement distresses and landfill problems have been recorded mainly due to the deficiencies in properties of laterite obtained at different locations.

Disposal of waste is a challenge for most developing countries (Nigeria inclusive) mainly due to the increasing generation of waste, the high costs associated to its management and the lack of understanding over a diversity of factors that affect the different stages of waste management Naman, (2015). Research on the effect of bamboo leaf ash on geotechnical properties of lateritic soil-bentonite mixtures was only done to a certain extent (as separate entity and not collective) and knowledge gap is one of the problems hampering detailed research on the use of bamboo leaf ash as an additive for modifying certain properties of lateritic soil-bentonite mixtures. Therefore this study shows how lateritic soil-bentonite mixtures stabilized with bamboo leaf ash will be beneficial to geotechnical studies and projects.

1.3 Aim and Objectives

The aim of the study is to evaluate how bamboo leaf ash affects the mixture of lateritic soil and bentonite while the objectives include:

1. Evaluation of the effect of bamboo leaf ash on geotechnical properties of lateritic soil-bentonite mixtures.
2. Assessment of the suitability of treated lateritic soil-bentonite mixtures as construction materials for landfill liners.

1.4 Scope of Study

The study is centered on identifying the index properties of natural laterite samples and to a considerable extent, lateritic soil-bentonite mixtures. Laboratory testing to evaluate the effect of bamboo leaf ash on lateritic soil-bentonite mixtures will be conducted. These tests include: particle size distribution test, specific gravity test, Atterberg (liquid limit and plastic limit), hydraulic conductivity, compaction and volumetric shrinkage strain test. Results obtained from volumetric shrinkage strain and compaction test will be used as supporting data to make recommendation on the use of lateritic soil-bentonite mixtures treated with bamboo leaf ash as a material for landfill liners and slurry walls.

1.5 Justification of Study

Lateritic soil is one of the widely used construction materials in Nigeria, this is due to its vast abundance, relative cheapness, chemical resistance, better workability and availability within economic haul distance (BRRI, 1971: Charman, 1988: Frempong and Yanful, 2008: Osinubi and Nwaiwu, 2008). The red tropical soil which is in abundant supply in Nigeria pose a unique challenge as there are instances where this soil may contain substantial amount of clay minerals that it strength and stability cannot be guaranteed under load especially in the presence of moisture. There are also instances where lateritic soils obtained at different location and at different depth exhibit varying geotechnical properties such as low permeability, high compressibility and low swelling potentials.

One of the essential for use of material as slurry walls and landfill liners is that such materials must possess low permeability, low compressibility and high shear strength. Most lateritic soils obtained from different locations and at different depths falls below this requirement and as a result, the need to modify it properties for better civil engineering application becomes necessary.

This study shows ways in which the properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash can be modified for better civil engineering applications.

1.6 Significance of Study

This study tends to evaluate the effect of bamboo leaf ash on lateritic soil-bentonite mixtures and the findings obtained from the study will be significant in the following ways:

1 Proffer solution to waste disposal problems resulting from large waste generation in urban areas.
2 Modify certain properties of laterite-bentonite mixture for better use as slurry walls and landfill liner materials.
3 Encourage high rate of construction activities in area of civil engineering.
4 Serve as body of knowledge for field application and further 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 (Hamilton, 1984).The aggregation derives its strength from the film found within the micro-joints of the elementary clay particles, which in addition coats the particles (Gidigasu, 1988).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 Properties of Laterite

2.3.1 Chemical Properties

(Mallet, 1983) was perhaps the first to introduce the chemical concept for establishing the ferruginous and aluminum nature of lateritic soils. (Fermor, 2010) defined various forms of lateritic soils on the basis of the relative contents of the so-called lateritic constituents (Iron, Aluminum, Titanium, and Manganese) in relation to silica. Also, (Lacroix, 2014) divided laterite into:-true laterite, silicate laterite, and lateritic clays depending on the relative content of the hydroixides. There are other several attempts by the researchers to classify laterite in terms of their chemical compositions, but (Fox, 1996) has demonstrated that such classification are inadequate, other than in relations to deposits that may be exploited for their mineral content, classification based on chemical composition cannot be used to distinguish between indurate and softer formations.

The high content of the sesquioxides of iron or aluminum relative to other components is a feature of laterite. These essential components are mixed in variable proportions. Some laterite may contain more than 80% of Fe_2O_3 and little of Al_2O_3, While others may contain up to 60% of Al_2O_3 and a little of Fe_2O_3. Although alkali and alkaline bases are almost entirely absent in most cases, this is not an absolute criterion. In particular, some ferruginous tropical soils may contain significant amounts of alkaline bases. Combined silica content is low in sesquioxides. This combined silica is predominantly in the form of Kaolinite, the characteristic clay mineral of most tropical formation.

2.3.2 Physical Properties

The physical properties of residual soils, commonly known as the index properties, vary from region to region due to their heterogeneous nature and highly variable degree of weathering controlled by regional climate and topographic conditions, and the nature of bedrock, (Nnadi, 1988). It also varies with the depth of the soil and can be determined by simple laboratory tests. Studies on the effect of weathering on the physical properties of lateritic soil by (Tuncer et al, 1977 and Rahardjo, 2004) have revealed the following;

1 Pore-size distribution varies with the degree of weathering.
2 Higher pore volume and larger range of pore-size distribution indicates advancement in the weathering stage.
3 Soil classification and Atterberg limits do not show any correlation to weathering.
4 High specific gravity is a good indication of advanced degree of weathering.
5 Soil aggregation increases with increasing weathering.
6 Position in the topographical site, and depth of soil in the profile.
7 Genesis and pedological factors (parent material, climate, vegetation, period of time in which the processes have operated).

2.3.3 Plasticity

Textural lateritic soils are very variable and may contain all fractions sizes; boulders, cobbles, gravel, sand, silt, and clay as well as concretionary rocks. The interaction of the soil particles at the micro scale is reflected in the atterberg limits of the soil at micro scale level. Knowledge of the atterberg limits may provide the following information:-

1. A basis for identification and classification of a given soil texture.
2. Strength and compressibility characteristics swell potential of the soil or the water holding capacity.

Atterberg limit depends on:

1. The clay content**:** plasticity increases with increase in clay content (Piaskowski, 1993).

2. Nature of soil minerals**:** only minerals with sheet-like or plate-like structures exhibit plasticity. This is attributed to the high specific surface areas and hence the increased contact in the shaped particles.
3. Chemical composition of the soil environment**:** the absorptive capacity of the colloidal surface of the actions and water molecules decrease as the ratio of silica to sesquioxides decreases (Baver, 1980).
4. Nature of exchangeable actions**:** this has a considerable influence upon the soil plasticity (Hough, 1989).

Pre-test preparation, degree of molding and time of mixing, dry and re-wetting, and irreversible changes may affect the plasticity of soil. Drying drives off absorbed water, which is not completely regained, on re-wetting (Fookes, 1997).Studies on the relationship between the natural moisture content, liquid limits and plastic limits of laterite have shown that generally the natural moisture contents is less than the plastic limit in normal lateritic soils (Vargas, 1993). However, the lateritic soil from high rain fall areas may have moisture contents as high as the liquid limit (Hirashima, 1979).

2.3.4 Particle Size Distribution

Consequently great importance has also been accorded to particle-size distribution in dealing with lateritic soils. Recent studies have revealed that lateritic soils are strikingly different from temperate zone soils in terms of genesis and structure. Their concretionary structure as compared to the dispersed temperate zone soils has necessitated modifications to mechanical or grading tests (Remillion, 1997). Consistent reports of variations in the particle-size distribution with methods of pretreatment and testing have been widely reported on laterite soils. Schofield (1957) found out that wet sieving increased the silt and clay fraction from 7 to 20% as compared to the dry sieving. It has been found that sodium hexametaphosphate generally gives better dispersion of the fine fractions. It was also found, for example, that using sodium oxalate on a halloysitic clay from Kenya gave between 20 and 30% clay fraction, while the sodium hexametaphosphate gave as high as between 40 to 50% clay fraction for the same soil (Quinones, 1963).

Another factor which has been found to affect the sedimentation test is the method of drying. Oven-dried lateritic soils were found to give the least amount of clay fraction, as compared to air-dried (Mohr and Mazhar, 1969). The decrease in the clay content was accompanied by an increase in silt and sand fraction contents as a result of the cementation and coagulation of the clay particles by free iron oxide into clusters (Terzaghi, 1958). The variation in the grading of lateritic gravels with the method of manipulation is also widely reported (Novais-Ferreira and Correia 1965 and Nascimento et al., 1998). In the study of the particle-size distribution of lateritic soils, three sources of confusion were noted. The first confusion arises from the belief by some authors, e.g. Bawa (1987), opined that lateritic soils represent a group of materials that can be defined within a specific range of particle-size distribution. The second source of confusion seems to arise out of attempts by some authors to confine the word laterite to concretionary lateritic gravels. The third source of confusion arises out of the attachment of unnecessary importance to the soil colour. (Nascimento et al. 1998) have suggested an interesting lithological classification of lateritic soils as follows:

Lateritic clays <0.002 mm
Lateritic silts =0.002 - 0.06 mm
Lateritic sands ~0.06- 2 mm
Lateritic gravel =2 - 60 mm
Laterite stones and cuirasse ≥ 60 mm

Studies on lateritic gravels by de Graft-Johnson et al. (1969) among others have shown that the grading, though important for identification purposes, cannot alone form the basis for grouping lateritic gravels in terms of mechanical properties. The strength of the aggregates was found to be an important factor. Studies of lateritic aggregates in Nigeria, has also established that the strength of the aggregates is mainly a function of the degree of maturity of the lateritic concretionary particles and the predominant sesquioxide in the aggregates.

2.3.5 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. 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.

2.3.6 Shear Strength Characteristics

Shear strength is a term used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain. The shear strength of a lateritic soil is a function of the friction and interlocking of particles (soil angle of internal friction) and possibly cementation or bonding at particle contact relative to total and effective stress. Due to cohesion, particulate materials may expand or contract in volume as it is subject to shear strains. If soil expands in volume, the density of particles will decrease and the strength will decrease likewise the shear strength.

The cohesion is attributable to the resultant of inter particle forces which are mainly associated with the clay-size particle of soils and will vary with the particle size and the distance separating them. The angle of internal friction included the effect of interlocking. The interlocking effect is affected to some degree by the shape of particles and the grain–size distribution. The two parameters cohesion (**c**) and angle of friction (ø) depends on the grading, particle shape and void ratio factors of the soil. Cohesion also depends on degree of saturation, while angle of internal friction does not (Gidigasu, 1976).

The shear strength characteristics of lateritic soils have been found to depend significantly on the parent materials, and the degree of weathering which in turn depends on the position of the sample in the soil profile and compositional factors as well as the pretest preparation of the samples (Lohnes, 1988).

2.3.7 Compressibility and Consolidation

When a soil mass is subjected to a compressive force, its volume decreases. 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.

2.3.8 Specific Gravity

The available data indicate that specific gravities vary not only with the textural soil groups but also within different fractions. In the first place lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (De Graft-Johnson and Bhatia, 1969). For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction. While alumina is concentrated in the silt and clay fractions (Nascimento et al., 1989; Novais-Ferreira and Correia, 1985). It is common to find specific gravities reported for the gravel and fines separately. The average of the two values can be assumed to be more representative of the specific gravity for the whole soil.

2.4 Stabilization

Soil stabilization is not new, but man has sought to accomplish it by various means almost since the first roads were built, but it is only in recent years that scientific methods has been applied to soil stabilization (Olugbenga and Adetuberu, 2013). Soil stabilization maybe defined as the process of blending and mixing materials with a soil so as to improve certain properties of the soil. A stabilized material may be considered as a combination of binder soil and aggregates preferably obtained at or near the site of stabilization manipulated and treated with or without admixtures, and compacted so that it will remain in its compacted state without detrimental change in shape or volume under applied force or exposure to weather.

Stabilization signifies improvement in both strength and durability which are related to performance. Increase in strength may be expressed quantitatively in terms of compressive strength, shearing strength, or some measure of bearing value or load deflection to indicate the load bearing quality (Olugbenga and Adetuberu, 2013). Stabilization is a method of processing available materials for the production of low cost roads and other civil engineering projects. In this type of project, design and construction, emphasis is definitely placed on the effective utilization of local materials, with a view to decrease construction cost. In some areas, naturally occurring aggregates and soil aggregate combinations exists which requires minimum processing for successful stabilization (Olugbenga and Adetuberu, 2013). While in other places, the natural soils are of unfavorable character and require modification through the use of suitable components such as gravels, crushed stones, geosynthetics, natural fibers or clay binder. While in other areas, admixtures like bituminous materials, lime or Portland cement must be used for effective stabilization. The type and degree of stabilization is dependent on the availability and cost of the required materials.

2.4.1 Methods of Soil Stabilization

In road construction projects, soil or gravelly material is used as the road main body in pavement layers. To have required strength against tensile stresses and strains spectrum, the soil used for constructing pavement should have special specification. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller, 2014). The method can be achieved in two ways, namely:

1. In-situ soil stabilization
2. Ex-situ soil stabilization

Stabilization is not necessary a magic wand by which every soil properties can be improved for better. The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Sherwood, 1993: Altabba and Evans, 2015). Some stabilization technique includes mechanical and chemical stabilization.

2.4.1.1 Mechanical Stabilization

Mechanical Stabilization is the process of improving the properties of the soil by changing its gradation moisture (Onyelowe and Chibuzor, 2012). This process includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration techniques and sometime blasting. The stability of the soil in this method relies on the inherent properties of the soil material moisture (Onyelowe and Chibuzor, 2012). Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

2.4.1.2 Chemical Stabilization

In order to improve the properties of expansive soil, a combination of chemical stabilizers such as cement, fly ash, and lime with chloride or individually can be used. About replacing soil particles to meet more stable soil structure, there are two main methods. Firstly, increasing the particle size by cementation to produce an increment in shear strength, reduction in plasticity index, and reduction in expansion potential. Secondly, improve the compaction and physical properties of the soil by using absorption and chemical binding of moisture (Onyelowe and Chibuzor, 2012).

2.4.1.3 Types of Additives used in Soil Stabilization

There are many additives that have been used to improve the engineering properties of expansive soil. These additives can be classified as waste materials such as dust, agricultural wastes, synthetic wastes, and organic wastes to enhance the economic cost.

Table 2.1: Additives Employed in Soil Stabilization (Salem, 2018).

Industrial Solid Waste	**Agricultural Solid Waste**	**Domestic Solid Waste**	**Mineral Solid Waste**
Fly Ash	Rice Husk Ash	Incinerator Ash	Quarry Dust, Stone Dust or Chipping Dust
Cement Kiln Ash	Bagasse Ash	Waste Tire	Marble Dust
Silica Fume	Groundnut Shell Ash	Egg Shell Powder	Limestone Dust
Copper Slag	Plantain Peel Ash	Grain Storage Dust	Granite Dust
Granulated Blast Furnace Slag	Banana Leaf Ash	Glass Cullet	Mine Tailings
Phosphogypsum	Concob Ash		Baryte

Ceramic Dust	Guinea Corn Ash		
Brick Dust	Bamboo Leaf Ash		

2.5 Bentonite

Bentonite is a geological extract found freely in its natural state. It is formed out of volcanic ash, it is a form of clay that consist of primary minerals called montmorillonite which is a dual layered, dual dimensional mineral that contains aluminium and silicate. This minerals gives bentonite a layer of cards that look like crystallize packets called platelets.(Knight,1997) also suggested the name "bentonite" for a clay-like material with soapy properties from fort Benton, Montana USA. (Hewelt, 1997) showed that this clay was formed by in-situ alteration of volcanic ash most often in the presence of water. There are different type of bentonite and are named according to the dominant element found in them, such as Potassium (K), Sodium (Na), Calcium (Ca) and Aluminium (AL). Sodium bentonite expands when wet absorbing as much as several times it dry mass in water. Due to it colloidal properties, it is often used in drilling during oil exploration. (Salem, 2018) stated that some of the physical characteristics of bentonite are based on characteristics of smectite minerals, high swelling, large cation exchange capacity, low hydraulic conductivity and large specific surface area.

2.5.1 Characteristics of Bentonite

2.5.1.1 Compaction Characteristics

Compaction of betonies can be done using the British Standard Light or Heavy compaction method. The British Standard Light method involves the use of 2.5kg rammer with an equivalent numbers of blows while the British Standard Heavy involves the use of 4.5kg rammer with a considerably higher energy than the former. Due to the high swelling potential property of bentonite, it is recommended to use the Light compaction method. Many studies have been done and concluded that heavy compaction method may lead to higher swelling potential, the swelling potential increases with the increase in dry unit weight which would not be advisable for landfills. The swelling potential of bentonite reduces with a decrease in water content. Therefore less dry unit weight and high water content is recommended and can be obtained through the Light compaction method.

2.5.1.2 Strength Properties

Shear strength of bentonite was investigated using the direct shear technique as specified by standard Australia AS1289.6.2.2 (1998). Hassan et al, (2016) performed standard compaction test on bentonite clay in accordance with the Standard Australia (2013) by subjecting clay samples to 596kj/m^3 compaction effort. In this study specimen prepared from bentonite were subjected to normal stress of 50kpa, 100kpa, and 200kpa after different curing periods and changes in their shear strength parameters including the shear stress, internal friction angle and cohesion. For the purpose of this study, direct shear test was performed by using a shear box that shears the sample under a specified shearing rate. This was conducted with the aid of a Geocomp Shear Trac-11 automated direct shear device. The device uses a horizontal and vertical displacement transducer to assist in the control of the horizontal and vertical loading system. The shear box employed for the study has an internal size of 63.5mm × 63.5mm ×24mm. Normal stress was then applied on each sample to consolidate the samples. On the individual parametric scale sample cohesion tended to exhibit a uniform trend of increasing values with increasing curing periods. Three levels of normal stress 50kpa, 100kpa and 200kpa and very slow horizontal strain rate was applied for testing each composition and curing day combination. The failure of the sample was marked by shearing or splitting of the sample along the shearing plane. Initially, the shear strength of pure bentonite was explored under increasing curing periods to obtain the benchmark result to assess the effect of both controlling parameters. The shear strength of bentonite was evaluated in relationship of their shear strength parameters such as angle of friction, cohesion and normal stress. The result obtained indicate that the trend was steeper during the initial period of curing and normally tended to gradually rise after 7 days of curing, whereas the highest cohesion values were observed at 28 days curing period. Therefore, the cohesion value of bentonite sample ranges from 55.60kpa to 58.90kpa.

2.5.1.3 Permeability Characteristics

Bentonite powder was fully mixed with de-aired water to prepare slurry having initial water content of 1.5 times of liquid limit (LL). The slurry was mixed with quartz and poured directly into the consolidometer ring of model JIS A-1217. The inner diameter of the consolidometer ring was 60mm with a sample height of about 20mm. The applied pressure for slurry samples are 4.90, 9.81, 19.62, 39.24, 78.48, 156.96, 313.92, 627.84 and 1255.68kpa respectively. A series of consolidation tests were conducted by incrementing the load at 24hr interval. The coefficient of permeability of the bentonite slurry was measured from the consolidation tests. The permeability test by falling head method of the slurry was carried out at the end of each load increment. Direct permeability test as well as a standard incremental loading consolidation test was conducted for the same sample. The outcome indicate that the coefficient of permeability of the slurry and compacted bentonite have distinct variation at the same void ratio. It is depicted that the coefficient of permeability varied about 10 times among the direct and indirect test methods. Not only is the permeability of bentonite dependent on its void ratio, but for any void ratio, it would also be a function of the geometrical arrangement of the particles in the soil that is of soil fabric. The permeability of the soil with a particular fabric and for a particular permeant, it is a function of the effective stress. As the effective stress increases; void ratio decreases and therefore permeability decreases. The important factors affecting the permeability values of bentonite are the type of permeameter, effective stress, hydraulic gradient, size of the specimen, type and chemistry of the permeant and termination criteria. These findings were summarized by Benson et al, (1994), Daniel, (2013) and Shakelford, et al., (2015).

2.5.1.4 Swelling Characteristics

The swelling behaviour of expansive clays exposed to water are due to two mechanism which are crystalline swelling and osmotic swelling (Madsen and Muller-Vonmoos, 2013). This bentonite is produced when expansive clay mineral is allowed to saturate under controlled volume conditions. Both mechanisms are influenced by the breakup of montmorillonite particles and by the de-mixing of cations. According to yong, (1999), the difference between these two mechanisms is only due to the hydration structure of water.

The crystalline swelling is developed as a result of the hydration of ion when montmorillonite absorbs water. Madsen and Muller-Vonmoos (2013), noticed that in the case of unconfined condition, montmorillonite volume may double its original volume while in the case of confined condition, it can be more than 100Mpa only due to crystalline swelling. When the swelling of the mineral is restricted, the crystalline swelling pressure of very high degree will develop reaching several thousand of kpa. The second mechanism, which appears when montmorillonite absorbs water considerably, is termed osmotic phenomena. It occurs because of the interactions between diffuse double layers and vanderwalls attraction.

When bentonite absorbs water under unrestrained boundary condition, they exhibit swelling pressures. Insertion of water molecules within the interlayer and inter-particle pores causes montmorillonite clay to swell. However if volume change is not allowed, the specimen will exert swelling pressure equivalent to the net repulsive force exerted between the clay platelets (Bolts, 1996, Tripathy et al, 2012, Schanz and Tripathy 2014). The magnitude of swelling pressure depends on the surface area, available exchangeable cation, temperature, initial dry density and initial water content (Villar, 2015).

2.6 Lateritic Soil-Bentonite Mixtures

Rasayan, (2017) conducted an experimental investigation on the hydraulic conductivity of laterite mixed with bentonite. The minimum requirement of a liner for landfill is that it should have a hydraulic conductivity less than 10^{-7}cm/sec. Bentonite content in the mixture is viewed as 0%, 10%, 20% and 30% which is added to the dry weight of the lateritic soil. The solute concentration is de-ionized water, 5ml of $K_2Cr_2O_7$, 0.5ml of $CaCl_2$ and 0.5ml of NaCl. Hydraulic Conductivity value decreased appreciably from 7.18×10^{-7}cm/s to $1.894\times10\text{-}^{8}$cm/sec when permeated with de-ionized water with 0-30% variation of bentonite. When 5ml of $K_2Cr_2O_7$ is used as solute,K value decreases with increase in bentonite content as the precipitate formed clogs the pores in the sample. Similar result was obtained with 0.5ml of $CaCl_2$ and 0.5ml of NaCl. Of all the mixtures, 20-80% lateritic soil-bentonite mixture is found is adjudged as the most effective combination.

2.6.1 Characteristics of Lateritic Soil-Bentonite Mixtures

2.6.1.1 Strength Characteristics

Shear strength parameters (cohesion and angle of internal friction) of lateritic soil-bentonite mixtures were assessed by carrying out series of consolidated undrained (CU) test. Specimens were molded at optimum water content and 85% of maximum dry unit weight. The bentonite content was varied from 3%, 5%, 7% and 9% respectively. 0.2mm/min was adopted as the shearing rate of the specimen based on the full consolidation time (Head, 1986).

The undrained shear strength of the natural soil and lateritic soil-bentonite mixtures was assessed by means of unconfined compression test. Compacted lateritic soil-bentonite mixtures has great potential to be used as barrier facilities provided that some technical requirements are satisfied such as hydraulic conductivity, compatibility after disposal, swelling and cracking. On the other hand the bentonite addition to lateritic soil lowered the hydraulic conductivity and as a result, investigation on the changes in geotechnical properties of lateritic soil induced by the bentonite addition are of substantial significance. This test conducted suggests that on the variation of bentonite from 3%-6%-9% in the presence of moisture, the undrained shear strength of lateritic soil-bentonite mixtures decreases proportionately.

2.6.1.2 Permeability Characteristics

Permeability is defined as the property of a porous material which permits the passage of water through its interconnecting voids. Permeability is involved in the problem of flow of water through soils, such as seepage under dams. The squeezing out of water from the soil is achieved by the application of load and drainage of sub-grade and backfill, computation of losses from canals. A-7-6 lateritic soil was treated using British Standard Light compaction energy with up to 8% bentonite content (By dry weigth of soil). Effect of bentonite on lateritic soil were investigated with respect to unconfined compressive strength and coefficient of permeability. The unconfined compressive strength increased with lateritic soil-bentonite mixtures at specified bentonite content to a maximum value of 8%. The coefficient of permeability of the specimen decreases with an increase in bentonite content corresponding to a maximum value of 5%, beyond this point the permeability rises slightly.

2.6.1.3 Swelling and Compressibility

Understanding the characteristics of lateritic soil-bentonite mixtures lead to increasing the confidence level before such material is utilized in the field. The outcome of this study can provide insight into the swelling and compressibility behavior of lateritic soil-bentonite mixtures (Villar, 2015). A simple swell and compressive laboratory test have been conducted for the purpose of this study. The result of this study indicated that the existence of bentonite, follows a hyperbolic curve model (Villar, 2015). Amount and size of non-swelling fraction affected the swelling and compressibility of the mixture. The swelling behavior of expansive soil often causes unfavorable problems such as differential settlement of structure (Villar, 2015). Recently, expansive soils are attracting greater attention. This soil is often designed as mixtures possessing characteristics such as low shrinkage and swelling properties, low hydraulic conductivity and low strength

2.6.1.4 Compaction Characteristics

The compaction characteristics of lateritic soil-bentonite mixtures were determined through laboratory test. This test were carried out to evaluate the effectiveness of compaction on lateritic soil-bentonite mixtures with the percentage of bentonite been varied between 5%, 10% and 15%. Compaction was conducted to obtain optimum moisture content (OMC) and maximum dry unit weight (MDUW).

Different compaction efforts were used as recommended by Daniel and Benson (1990) such as Reduced British Standard Light (RBSL), British Standard Light (BSL) and British Standard Heavy (BSH). The BSH and BSL are in accordance with the British Standard of modified and standard proctor compaction (BS1377). The laboratory result shows that the maximum dry unit weight value was slightly low for the entire sample of lateritic soil mixed with bentonite at all compaction energy. However, high value of maximum dry unit weight (MDUW) was obtained with increase in compaction energy. Therefore soil compaction efforts play an important role for the workability of the mixture.

2.7 Bamboo Leaf Ash (BLA)

Bamboo is one of the oldest building materials used by mankind (Abdulatif et al., 2014). The bamboo culm (stem) has been made into different products ranging from domestic household products to industrial applications, examples are found in food containers, skewers, chopsticks, handicrafts, toys, furniture, flooring, pulp and paper, boats, charcoal, musical instruments and weapons. In Asia, bamboo is quite common for bridges, scaffolding and housing, but it is usually a temporary exterior structural material, while in many overpopulated region of the tropics, certain bamboos supply the one suitable material that is sufficiently cheap and plentiful to meet the extensive need for economical housing (McClure, 2017). Bamboo shoots are important sources of food and delicacy in Asia and in addition to its more common applications, bamboo has other uses from skyscraper, scaffolding and phonograph needles to slide rules, skins of airplanes and diesel fuels (Farelly, 1984). Extracts from various parts of the plant have been used for skin ointment, medicine for asthma, eyewash, potions for lovers and poisons for rivals. Bamboo ashes are used to polish jewels and manufacture electrical batteries. (Ernesto et al., 2013). Bamboo has been used in bicycles, dirigibles, windmills, scales, retaining walls, ropes, cables and filament in the first light bulb. Indeed, bamboo has many applications beyond imagination. Its uses are broad and plentiful. (Lee, et al., 2013). Massive plantation of bamboo provides an increasingly important source of raw material for pulp and paper industry in China (Hammett et al., 2015).

The chemical composition of bamboo is similar to that of wood. The main constituents of bamboo culms are cellulose, hemicelluloses and lignin, which amount to over 90% of the total mass. The minor constituents of bamboo are resins, tannins, waxes and inorganic salts. Compared with wood, however, bamboo has higher alkaline extractives, ash and silica contents. Yusoff et al (2016) studied the chemical composition of one, two, and three years old bamboo (Gigantochloa scortechinii). The results indicated that the holocellulose content did not vary much among different ages of bamboo. Alpha-cellulose, lignin, extractives, pentosan, ash and silica content increased with increasing age of bamboo. Bamboo contains other organic composition in addition to cellulose and lignin. It contains about 2-6% starch, 2% deoxidized saccharide, 2-4% fat, and 0.8-6% protein. The carbohydrate content of bamboo plays an important role in its durability and service life. Durability of bamboo against mould, fungal and borers attack is strongly associated with its chemical composition. Bamboo is known to be susceptible to fungal and insect attack. The natural durability of bamboo varies between 1 and 36 months depending on the species and climatic condition. The presence of large amounts of starch makes bamboo highly susceptible to attack by staining fungi and powder-post beetles (Mathew and Nair, 2014). The ash content of bamboo is made up of inorganic minerals, primarily silica, calcium, and potassium. Manganese and magnesium are two other common minerals. Silica content is the highest in the epidermis, with very little in the nodes. Higher ash content in some bamboo species can adversely affect the processing machinery. Bamboo leaf fired in an open atmosphere and then heated at 600ºC for 2 h in a furnace was found to be an amorphous material containing amorphous silica.

Table 2.1: Chemical Composition of Bamboo Leaf Ash (Olugbenga and Adetuberu, 2010).

BLA	**SiO2**	**AL2O3**	**Fe2O3**	**CaO**	**MgO**	**K2O**	**Na2O**	**TiO2**	**SO3**	**LOI**
	75.9	**4.13**	**1.22**	**7.47**	**1.85**	**5.62**	**0.21**	**0.20**	**1.06**	**-**

2.7.1 Review of Previous Works on Soil Stabilization Using Bamboo Leaf Ash (BLA)

Various studies have been conducted in the area of soil stabilization using bamboo leaf ash. Ayobami, et al., (2018) conducted a research on sustainability in road construction using bamboo leaf ash to improve the index properties of lateritic soil. The lateritic soil was stabilized with increasing percentage of bamboo leaf ash at 0%, 2%, 4%, 6%, 8%, 10%, and 12%. The index properties, Compaction and CBR, of the soil samples with bamboo leaf ash were evaluated. Response Surface Analysis was used to model the mathematical relationship between the atterberg limit and the CBR of the BLA stabilized soil sample. The plasticity index of the un-stabilized soil sample was 14.01 upon the addition of 16% BLA, the plasticity index reduced to 10.73 which showed an improvement in the soil index properties. The CBR increased from 26.38% to 30.2% at 0% and 8% respectively which signifies an improved strength. From the Response Surface Analysis, the highest plasticity index achievable with bamboo leaf ash stabilization is 27.18. The model equation showed that the plasticity index and plasticity limit have a positive relationship with the CBR.

In another study, Olugbenga and Adetuberu, (2010) investigated the stabilization of lateritic soil using bamboo leaf ash for highway construction. The results showed that the addition of bamboo leaf ash improved the strengths of the samples, Optimum moisture contents reduced to 20.20, 19.60 and 9.32% at 8, 4 and 6% bamboo leaf ash additions in the samples. Maximum Dry Unit Weight (MDUW) increased to 1400, 1676 and 1941 kg/m^3 respectively at 8%, 2% and 4% bamboo leaf ash additions in the samples. The un-soaked CBR and shear strength of the samples increased. It was therefore concluded that bamboo leaf ash has a good potential for stabilizing lateritic soils in highway construction.

Lorliam, et al., (2013) conducted an experimental study on effect of bamboo leaf ash on cement stabilization of Markudi shale for use as flexible pavement construction material. Classification test, compaction, consistency, california bearing ratio and unconfined compressive strength tests, were conducted on Makurdi shale specimen treated with cement and bamboo leaf ash in combined incremental order of 2% up to 14% cement, and 4% up to 20% bamboo leaf ash by dry weight of soil sample respectively. Results obtained suggested that Makurdi shale can be classified as an A-7-6, CH and high swell potential soil by the AASHTO, USCS and NBRRI classification systems respectively. The plasticity index (PI) decreased from 39.4% for untreated Makurdi shale to 4.7% at 14% cement + 20% bamboo leaf ash. Maximum dry unit weight (MDUW) of untreated shale increased from 1.49Mg/m^3 to a peak value of 1.80Mg/m^3 at 0% BLA + 14% cement. While, the optimum moisture content (OMC) of shale increased from 14.5% to 33.1% at 14% cement content + 20% bamboo leaf ash. Maximum soaked CBR and 7 day UCS values of 80% and 1783.9 kN/m^2 was obtained at 14% cement content + 20% bamboo leaf ash. Based on the results of the different tests, the use of Makurdi shale treated with 14% cement content + 20 % bamboo leaf ash were recommended for use as sub-base materials in flexible pavement.

The current study will therefore bridge knowledge gap on extensive use of bamboo leaf ash (BLA) for soil stabilization by seeking ways in which lateritic soil-bentonite mixtures can be modified using bamboo leaf ash for use as landfill liners and slurry walls.

CHAPTER THREE

MATERIALS AND METHODS

These section present relevant materials and methods employed to accomplish the research goal. The method of collection, storage and preparation of samples before testing were clearly stated. All laboratory testing such as Particle Size Distribution (Sieve Analysis), Specific Gravity, Atterberg (Plastic and Liquid Limit) test, Compaction were conducted at laboratory unit of Department of Civil Engineering, Nnamdi Azikiwe University, Awka Anambra State. Below are detailed description of Materials and Methods adopted in obtaining results.

3.1 Collection and Preparation of Sample

3.1.1 Laterite

The laterite samples designated as LAT was procured from Enugu Agidi in Anambra State. The choice of sites is justified by the fact that it is a burrow pits from where construction companies obtain their materials for road construction. The laterite sample obtained from Enugu Agidi designated as LAT was collected with the aid of a digger and a shovel at a depth of 5m from the highest level of the burrow pit. The samples passed all the 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 to the school laboratory for various laboratory testing. The in-situ moisture content of the sample 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 (i.e. zero moisture content).

3.1.2 Sodium Bentonite

The sodium bentonite, designated as BN is a clay material with soapy properties which has high swelling properties was procured from Onitsha market, in Anambra State from borehole materials vendors and the material upon arrival was tested for it free swell. The sodium bentonite will be admixed with laterite and both mixtures were treated with bamboo leaf ash.

3.1.3 Bamboo Leaf Ash (BLA)

Bamboo leaf designated as BLA was obtained locally from different bamboo plantations located at Awka in Anambra State. The bamboo leaf was collected in large quantities with the aid of a cutlass and a large bowl. This bamboo leaf was thereafter conveyed to Metallurgical Institute in Onitsha,Anambra State where it was air-dried and burnt at a controlled temperature. The ash was collected in sacks and conveyed to the soil mechanics research laboratory in Nnamdi Azikiwe University. The ash was then passed through the BS No 200 sieve (75µm) to meet the requirements of ASTM class N pozzolans (ASTM D4318-10e1) as reported by Head in (1994). The laterite-bentonite mixture was treated with the ash by varying the percentage replacement so as to study its effect on the geotechnical properties of laterite-bentonite mixtures.

3.2 Methods

The lists of civil engineering test to be conducted are as follows:

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.

The grain size analysis is widely used in classification of soils. The data obtained from the grain distribution curve is used in the design of filters for earth dams and to determine the suitability of soil for road construction, air field etc. Information obtained from grain size analysis can be used to predict soil water movement although permeability test are more generally used. Soil gradation is very important to geotechnical engineering; it is an indication of other engineering properties such as shear strength, compressibility and hydraulic conductivity. In a design, the gradation of the in-situ- soil help in the selection of filler material for the construction of highway embankment and it also controls the design and ground water drainage of site. A poorly graded soil (one with predominantly one-sized particle) will have better drainage property than the well graded soil (soil with varieties of particle sizes) because of the relatively higher magnitude of void present. A well graded can be easily compacted more than a poorly graded soil. However most Engineering project may have gradation requirement that must be satisfied before the soil is to be used is accepted for construction work. When options for ground remediation technique are to be considered the soil gradation is a controlling factor.

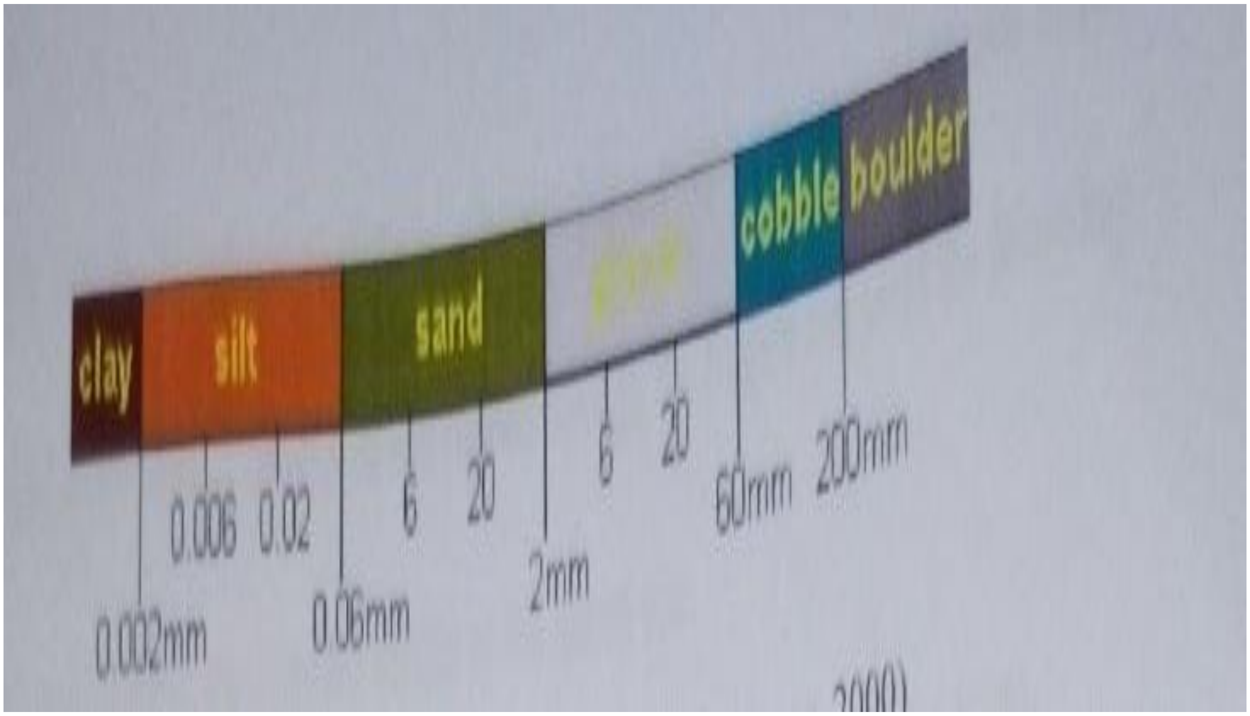


Figure 3.1 Ranges for grain Sizes of different Soil type (Atkinson, 2000).

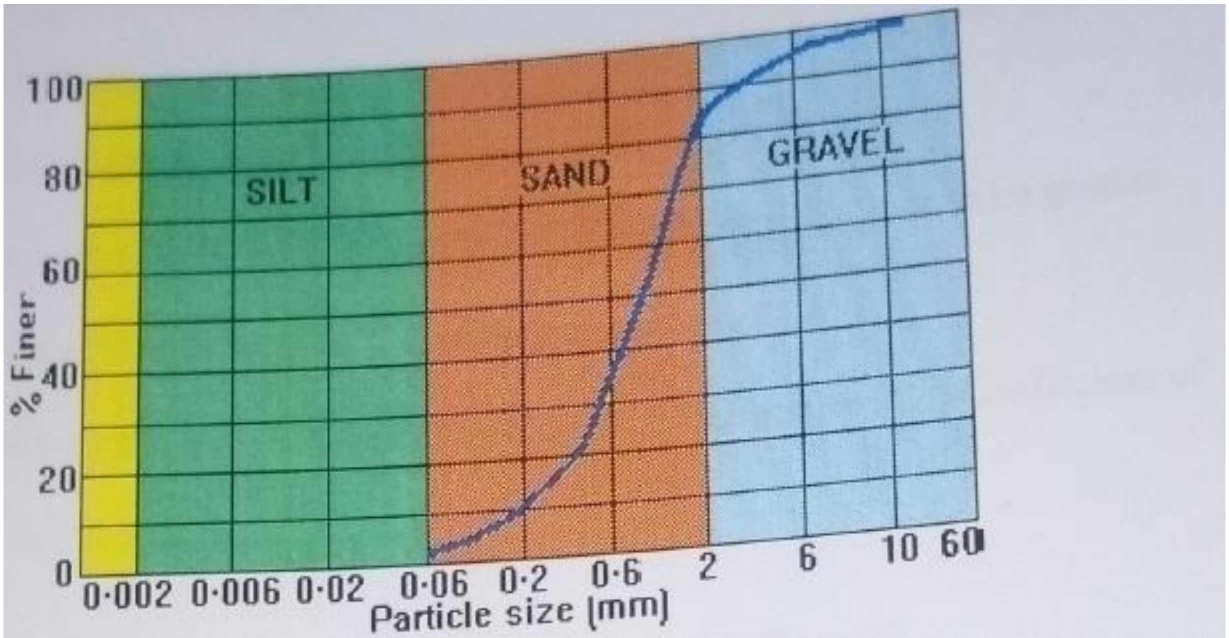


Figure 3.2 Grading Curve Ranges for Different Soil Types (Atkinson, 2000)

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 (%) =$\frac{mass\ of\ soil\ retained\ in\ the\ sieve(g)}{total\ mass\ of\ soil\ sample(g)} \times 100$

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

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.

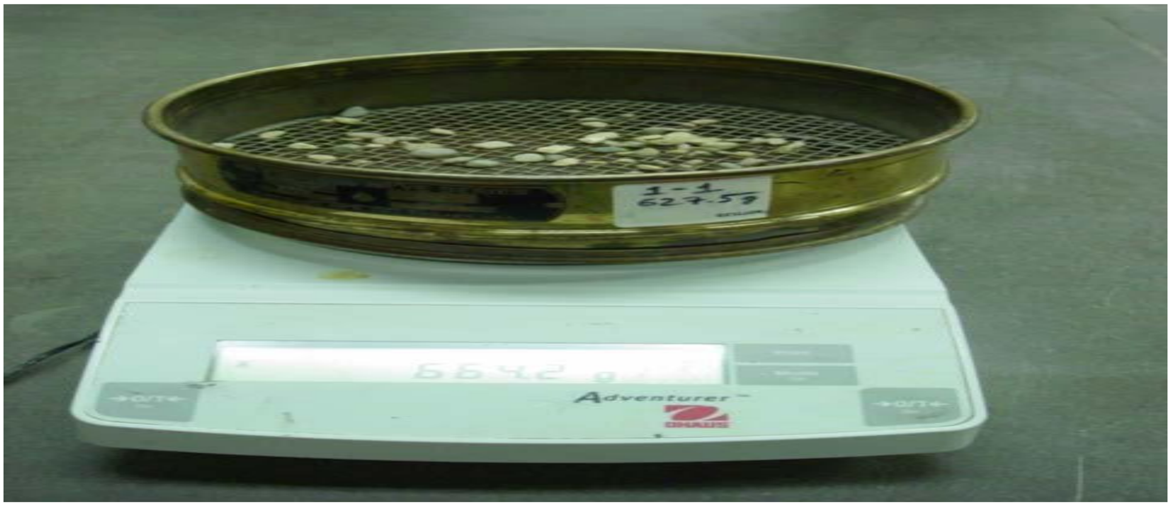


Figure 3.3 Apparatus for Particle Size Distribution Test (Sieve Analysis).

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

   Thin glass rod for stirring.
9. 425um Sieve.
10. Dry piece of cloth for cleaning.
11. Masking tape for identification of sample.
12. Exercise book and pen for recording of result.

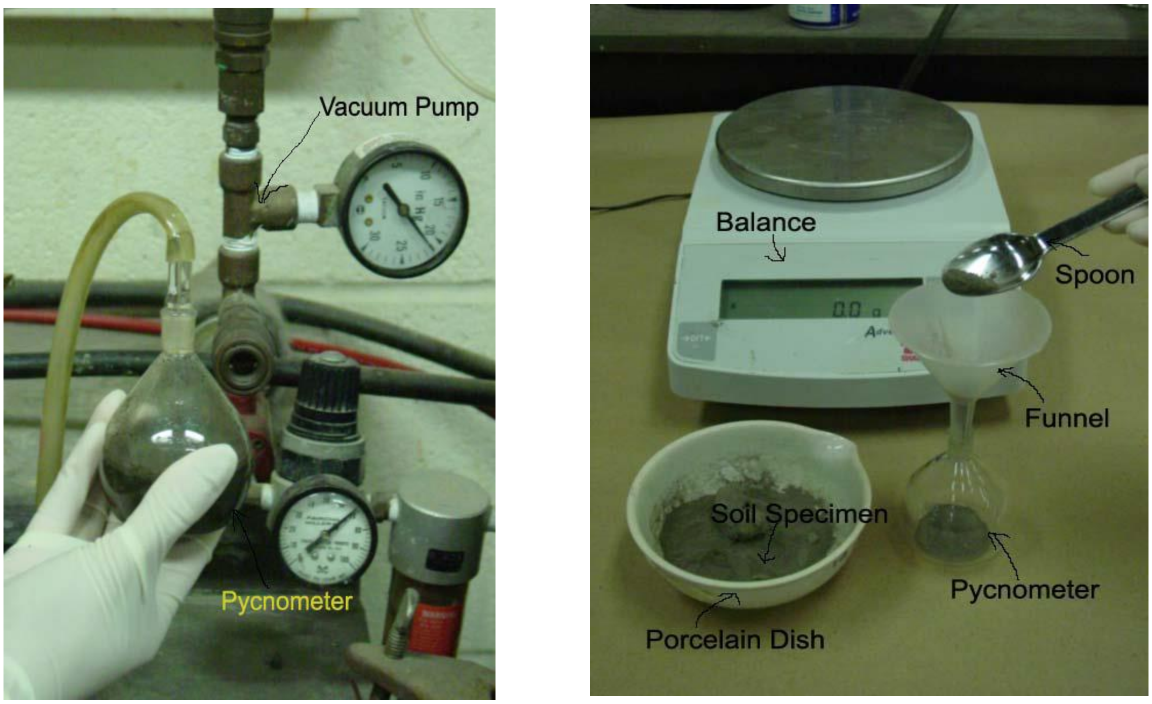


Figure 3.5 Apparatus used for Specific Gravity Test

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 are 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.

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 (SL)

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

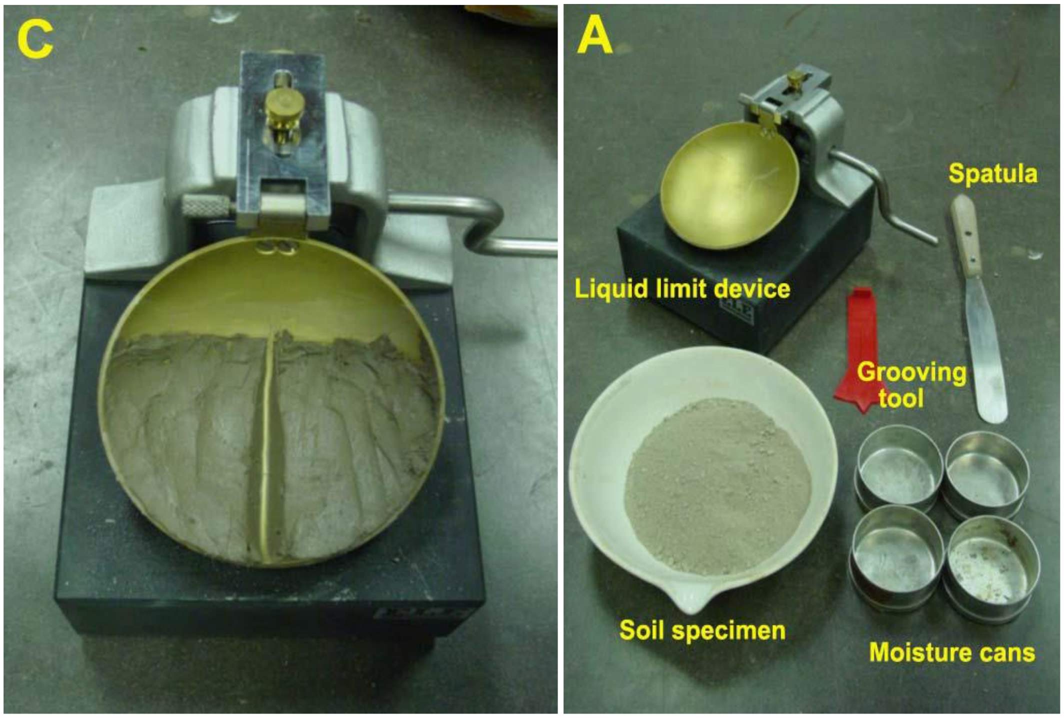


Figure 3.6 Apparatus for Atterberg Limit Test.

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 = $\frac{Weight\ of\ water}{weight\ of\ dry\ soil}\times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried soil.

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: 3.1 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:

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

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.

The compaction methods to be adopted for this research are:

1. British Standard Light and British Standard Heavy for the natural samples of laterite.

2. British Standard Light and British Standard Heavy for laterite – bentonite mixture treated with bamboo leaf ash.

Details of British Standard Compaction Process

Table 3.2 Details of Compaction Mould.

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

Table 3.3 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 = $\frac{Weight\ of\ rammer \times no\ of\ layers \times no\ of\ blows \times height\ of\ drop}{Volume\ of\ mould}$

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

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

British Standard Heavy

Mechanical energy $=\frac{4.5\times 5\times 27\times 450}{1000}$ **=2652j**

1. The apparatus used for the test are as follows:

2. Compaction mould with a detachable base plate and removable extension collar.
3. Metal rammer (either 2.5kg or 4.5kg)
4. Measuring Cylinder 200ml or 500ml
5. Large Metal tray (600mm×600mm ×600mm)
6. Balance up to 10kg readable to 1g
7. Small tools such as palette knife, steel straight edge about 300mm long.
8. Drying oven temperature of 105-110^{O}C
9. Apparatus for moisture content determination

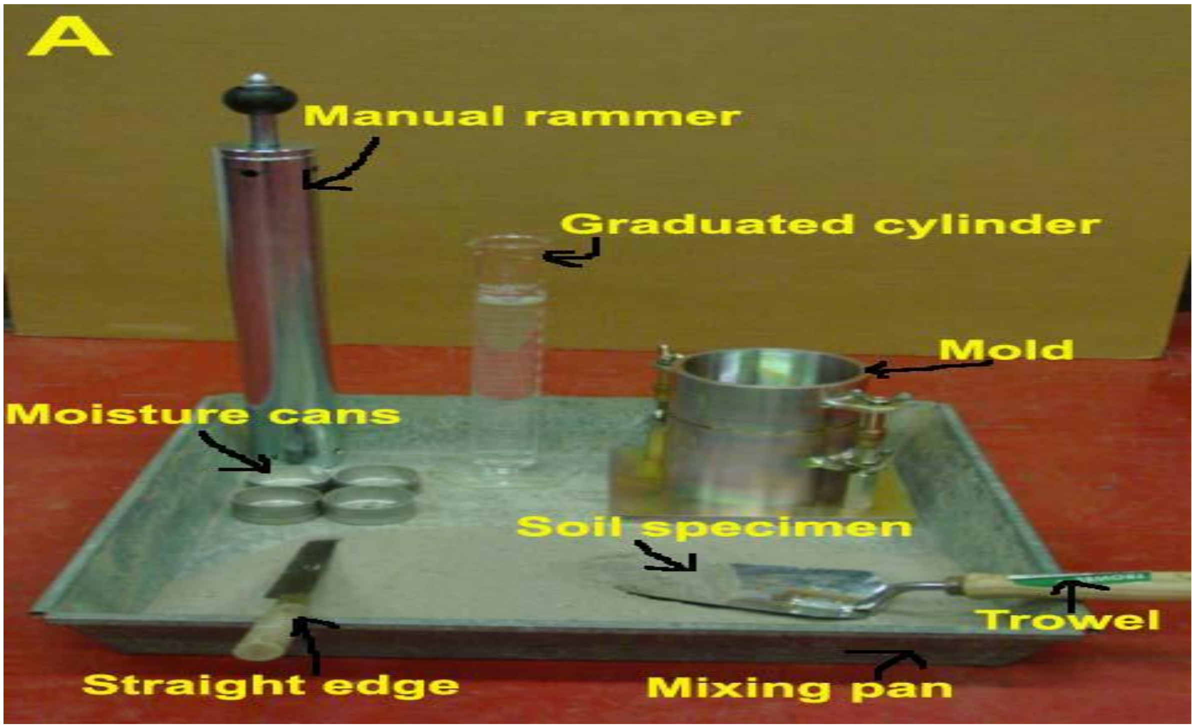


Figure 3.7 Apparatus employed for Compaction Test.

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) = $\frac{Wt\ of\ wet\ soil\ (kg)}{Vol\ of\ mould\ (m3)} = \frac{W2-W1}{W4}$

Moisture Content (%) = $\frac{mosture\ content\ (top) + moisture\ content\ (bottom)}{2}$

Dry Density (kg/m^3) = $\frac{Bulk\ density}{1+moisture\ content\ (\%)} = \frac{Pb}{1+w/100}$

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 (%) = $\frac{Wt\ of\ water}{Wt\ of\ dry\ soil} \times 100 = \frac{W6}{W5} \times 100$

3.2.5 Hydraulic Conductivity Test (Falling head permeability test)

The falling head permeability test is a common laboratory testing method used to determine the permeability of fine grained soils with intermediate and low permeability such as silts and clays. This testing method can be applied to an undisturbed sample. The falling head permeability test involves flow of water through a relatively short soil sample connected to a standpipe which provides the water head and also allows measuring the volume of water passing through the sample. The diameter of the standpipe depends on the permeability of the tested soil. The test can be carried out in a Falling Head permeability cell or in an oedometer cell.

Apparatus

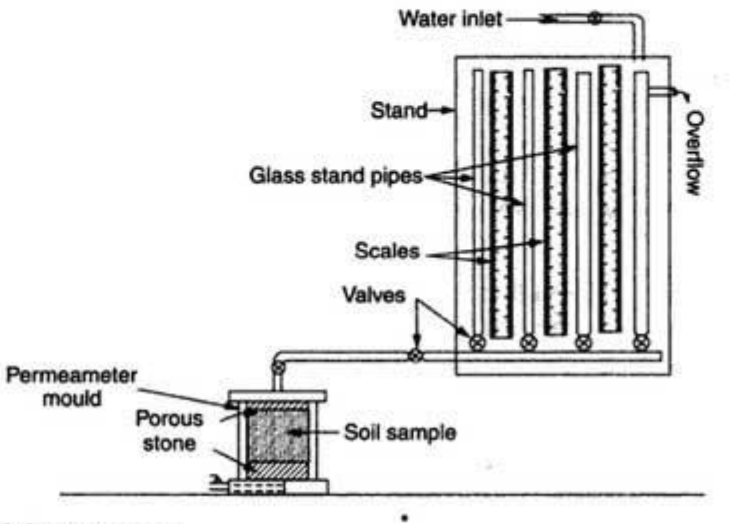
1. Permeameter mold of 10.5 cm diameter and 12.17 cm in height.

2. Compaction equipment – suitable dynamic or static compaction equipment.

3. Drainage base – with a porous disc of 12 mm thickness and dummy plate of 12 mm thickness to suit the mold, provided with water inlet/outlet connection.

4. Drainage cap – with a porous disc, 12 mm thick, and water inlet/outlet connection to the constant head tank.

5. Set of standpipes – glass standpipes of diameter 5 to 20 mm, suitably mounted on a stand (below Fig.3.8).



Permeability test set-up

Figure 3.8 Falling Head Permeability Test Setup

6. Miscellaneous apparatus – IS sieves, mixing tray, graduated cylinder, metric scale, stopwatch, 75-gauge wire, thermometer, and source of water.

Specimen preparation:

This testing method can be applied to an undisturbed sample, compacted sample or prepared sample in the laboratory by placing it inside the permeameter cell. Before placing the soil sample in the permeameter, measure the inside diameter (D) and height (L) of the permeameter cell. Place porous stone or filter paper in the bottom of the cell, then put the soil sample. Add a filter paper on the top of the soil sample and assembly the top part of the permeameter device. Saturate the soil sample by flowing water through it. It is important that the sample be fully saturated; otherwise, the falling head test will give erroneous results. Then apply a vacuum or remove any entrapped air within the sample.

Test procedure:

1. Determine the standpipe area (a). Note that the diameter of the standpipe depends on the permeability of the tested soil.
2. Locate hl and h2 on the standpipe. Then fill it with distilled water.
3. Allow water to flow down through the sample and observe the water level in the standpipe. As soon as it reaches the level h1,start the timer clock.
4. When the level of water in the standpipe reaches h2, stop the clock and

record the time required for the water in the standpipe to drop from h1 to h2.

5. Refill the standpipe and repeat the test two to three times. Use the same h1and h2 values and obtain the corresponding elapsed times. Record the temperature of water (T) for each run.

Calculation:

Calculate coefficient of permeability, k. as follows:

Where:

$$k = \frac{2.303 a L}{A t} \log_{10} \frac{h1}{h2}$$

k = coefficient of permeability (hydraulic conductivity) (m/s).

a = the inside area of the standpipe

L = Length of the sample.

A =the inside area of specimen

t = elapsed time of test (s).

h1 = the elevation of water in the standpipe at time t=0.

h2 = the elevation I water in the standpipe at time equal to t.

3.2.6 Unconfined Compressive Strength Test

Unconfined compressive strength (qu): Unconfined compressive strength (qu) is the load per unit area at which an unconfined cylindrical specimen of soil will fail in simple compression test. *Sensitivity* : sensitivity is defined as the ratio of unconfined compression strength of undisturbed soil sample to the unconfined compression strength of remolded sample at constant moisture content.Soil consistency can be easily known from the value of unconfined compressive strength. Sample should always be extracted along the same direction in which it entered the tube in the field. Where no maximum stress is observed, stress at 20% strain will give the unconfined compression test. The main objective of the test is to determine the unconfined compressive strength (qu) of a cohesive soil. The test is suitable for saturated non-fissured cohesive soil, for which angle of shearing resistance may be assumed to be zero. The undrained shear strength can be taken as half the unconfined compressive strength. i.e. Cu=(qu /2). In this simple test, a cylindrical specimen of soil is subjected to simple vertical compressive stress, till the sample fails either due to shear along a diagonal plane or by lateral bulging. The maximum load at which the failure takes place, when divided by cross-sectional area of the sample will give the value of unconfined compressive strength of the soil.

Equipments

Compression machine, Dial Gauge, Proving Ring, Split mould of internal diameter 40 mm and length 80 mm, Sampling tube of internal diameter 30 mm and length 200 mm, Metal Plate , Moving Plate, Vernier caliper, Balance, Sampling extractor

Apparatus employed for Unconfined Compressive Strength Test.

Test Procedure

Procurement of sampling tube with undisturbed sample from the site. Coat the inside of the split mould with a thin layer of grease or oil to prevent adhesion of the soil. Extrude the specimen from the sampling tube to the split mould with the help of sample extractor and knife. Trim the two ends of the mould sample. Remove the sample from mould by splitting it in two parts and weigh it. Also measure the length and diameter of the specimen. Place the specimen on the bottom of the machine. Raise the bottom plate of the machine to make contact of the specimen with the upper plate. Adjust the strain dial gauge and proving ring dial gauge to read zero. Apply the compression load by raising the bottom of the machine to produce axial strain at a rate of 1.25 mm/min. Record the strain and proving ring dial gauges reading every after 30 sec. Compress the specimen till it fails or 20% vertical deformation is reached whichever is earliest. Measure the failure angle from horizontal and note the load at failure. Determine the water content of the specimen. The graph shows the sample deformation along the horizontal scale and the extension of the spring along the vertical scale. Two curves obtained The extension of the spring and corresponding deformation in the sample length (ΔL) can be read from the graph. The stiffness of the spring multiplied by spring extension will give the total load Pf applied on the sample.

3.2.7 Volumetric shrinkage Test

Volumetric shrinkage is the decrease in volume (expressed as a percentage of the soil mass when dried) of a soil mass when the water content is reduced from a given percentage to the shrinkage limit.

The apparatus used for the test are as follows:

1. Split mould of internal diameter 40 mm and length 80 mm.
2. Metal rammer
3. Measuring Cylinder 200ml or 500ml
4. Large Metal tray (600mm×600mm ×600mm)
5. Weighing Balance
6. Small tools such as palette knife, steel straight edge about 300mm.
7. Vernier caliper.

Test procedure.

Coat the inside of the split mould with a thin layer of grease or oil to prevent adhesion of the soil. 200g of soil sample was weighed out using a weighing balance, after which the weighed sample was placed in a mixing tray. It was mixed thoroughly with water at optimum moisture content before been shared into three layers for British Standard Light (BSL) compaction. After completing the compaction, extrude the specimen from the split mould with the help of sample extractor and knife. Trim the two ends of the mould sample. Remove the sample from mould by splitting it in two parts and weigh it. After the extrusion, the diameter and height of the extruded soil were taken using a vernier caliper. The extruded cylindrical specimens were air dried openly for 5 days.

Formula used in computation of volumetric shrinkage

Volume of specimen = $\pi r^2 H$ (cm3)

Radius = r (cm)

Height of specimen = H (cm)

Volumetric shrinkage (%) = $\frac{\text{Initial volume} - \text{Final volume}}{\text{Initial volume}}$

CHAPTER FOUR

RESULTS AND DISCUSSION

This chapter presents key findings obtained from experimental investigation of the effect of bamboo leaf on lateritic soil-bentonite mixtures, these findings are presented below:

4.1 Results

4.1.1 Specific Gravity Test

Specific Gravity Results for Laterite Stabilized with Bentonite and Bamboo Leaf Ash:

Figure 4.0-4.5 shows the specific gravity results for lateritic soils bentonite mixtures treated with bamboo leaf ash. From the results for lateritic soil-bentonite mixtures, it was observed that addition of bentonite to lateritic soil increased the specific gravity of laterite from its natural value of 2.64 to 2.66 at 5% bentonite content, beyond 5% bentonite content, the specific gravity of laterite decreased. The decrease in specific gravity of lateritic soil could be attributed to the low specific gravity value of bentonite. For lateritic soil-bentonite mixtures treated with bamboo leaf ash, it was observed that the specific gravity of lateritic soil-bentonite mixtures decreased with increasing percentages of bamboo leaf ash.

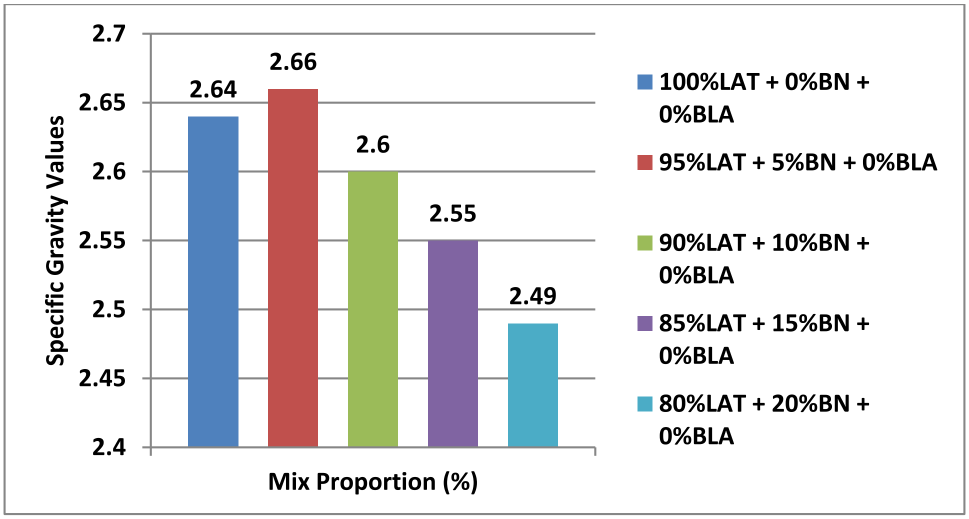


Figure 4.0: Specific Gravity Test for Lateritic Soil-Bentonite Mixtures at 0% Bamboo leaf Ash

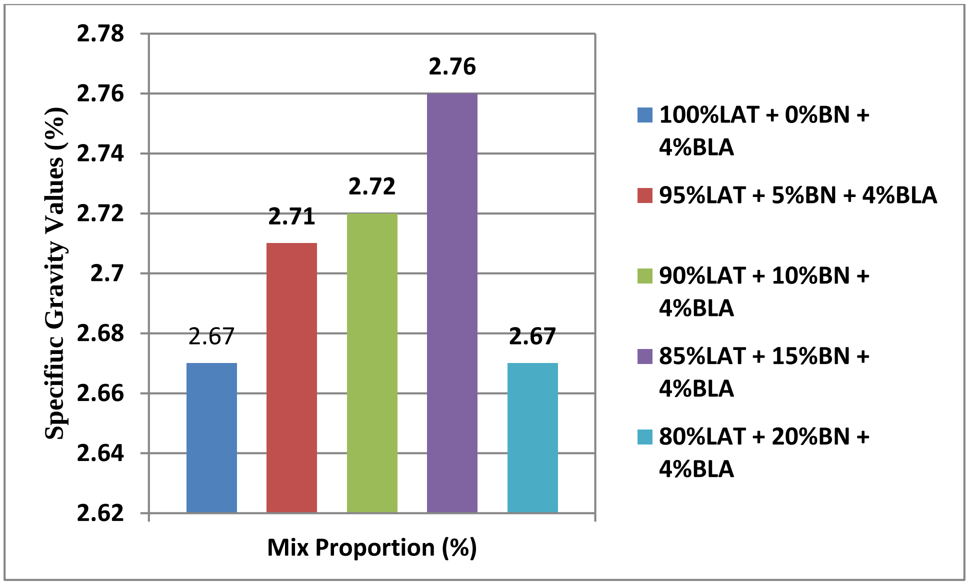


Figure 4.1: Specific Gravity Test for Lateritic Soil-Bentonite Mixtures at 4% Bamboo leaf Ash

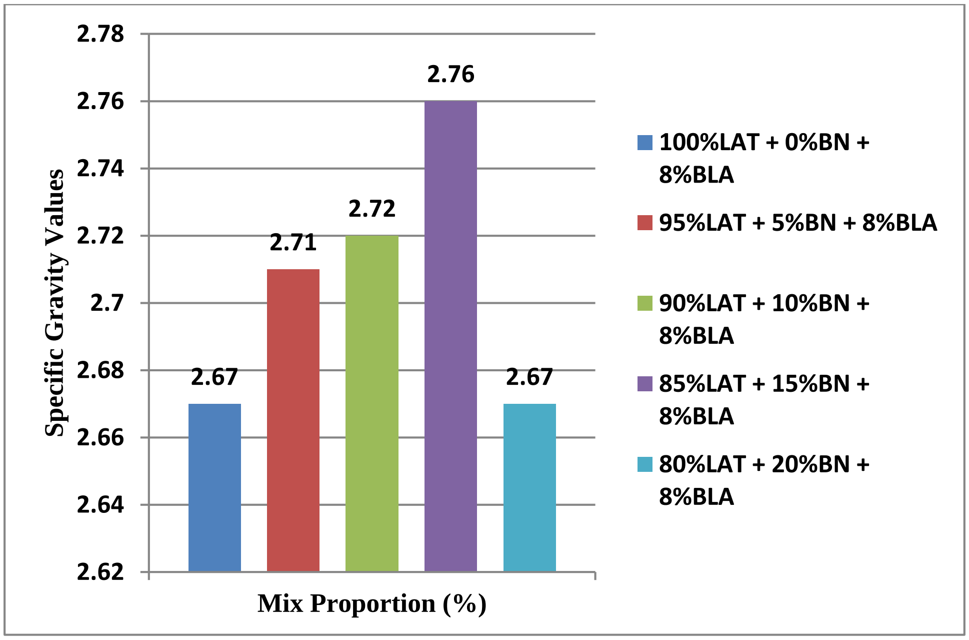


Figure 4.2: Specific Gravity Test for Lateritic Soil-Bentonite Mixtures at 8% Bamboo leaf Ash

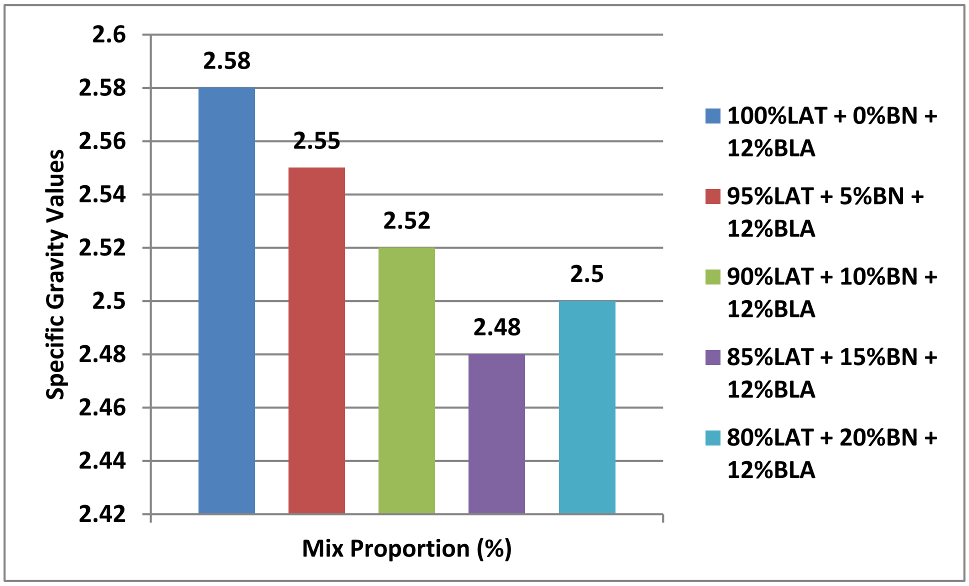


Figure 4.3: Specific Gravity Test for Lateritic Soil-Bentonite Mixtures at 12% Bamboo leaf Ash

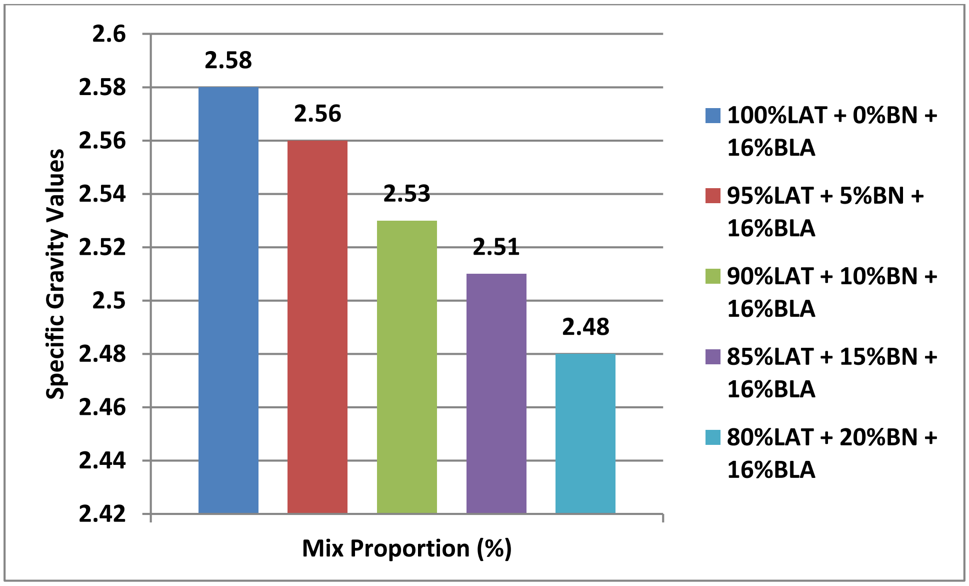


Figure 4.4: Specific Gravity Test for Lateritic Soil-Bentonite Mixtures at 16% Bamboo leaf Ash

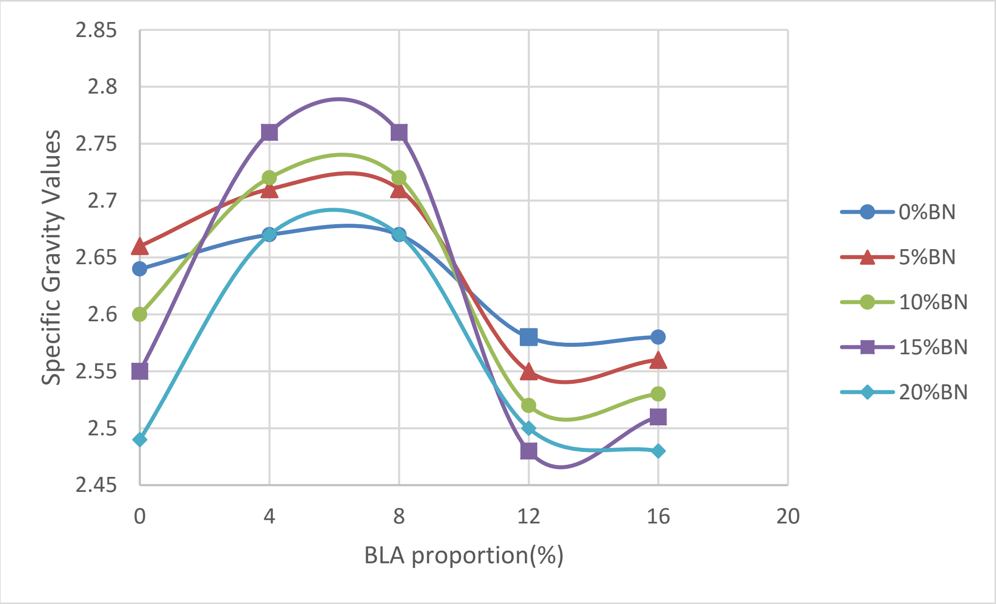


Figure 4.5: Specific Gravity graph for Lateritic Soil-Bentonite Mixtures at varying percentages of Bamboo Leaf Ash

4.1.2 Sieve Analysis Test

Sieve Analysis Test Results for the Laterite:

Figure 4.6 shows the particle size distribution test results for the natural lateritic soils, the passing through sieve No 200 was 54.54%, coefficient of uniformity and coefficient of curvature was 0 and as a results the laterite sample is classified as A-7-6 according to AASHTO Soil Classification System and CH (clay of high plasticity) according to Unified Soil Classification. The lateritic soil sample is characterized by high content as the amount of fines present was 54.54% while the amount of sand present was 46.46%. The gradation characteristics of the lateritic soil sample cannot be ascertained due to loss in shape parameter (D10 and D30).

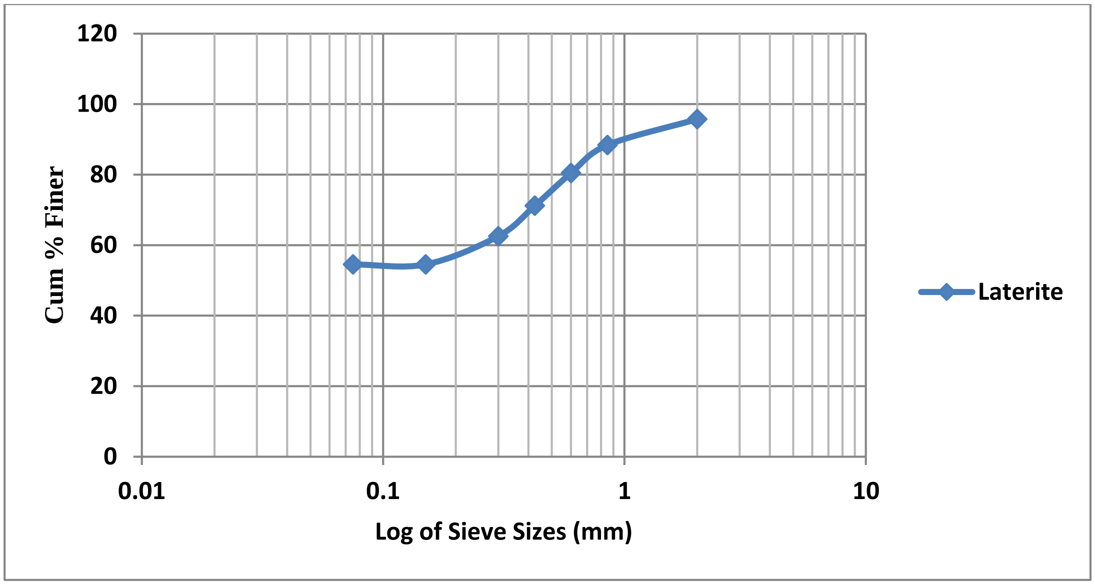


Figure 4.6: Particle Size Distribution Curve for Lateritic Soil

4.1.3 Atterberg Limit Test

Liquid Limit Results for Lateritic Soil-Bentonite Mixtures Treated with Bamboo Leaf Ash:

Figure 4.7- 4.12 depicts the liquid limit results for lateritic soil-bentonite mixtures stabilized with bamboo leaf ash. From the findings, it was observed that consistent addition of bentonite to lateritic soil from 5% to 20% by weight of lateritic soil increased the liquid limit of lateritic soil while for lateritic soil-bentonite mixtures treated with bamboo leaf ash; it was observed that the liquid limit of lateritic soil-bentonite mixtures decreased with increase in bamboo leaf ash. The decrease in the liquid limit of lateritic soil on addition of bamboo leaf ash could be attributed to the formation of cementitious compounds between the $Ca(OH)_2$ present in the soil the pozzolans in the bamboo leaf ash.

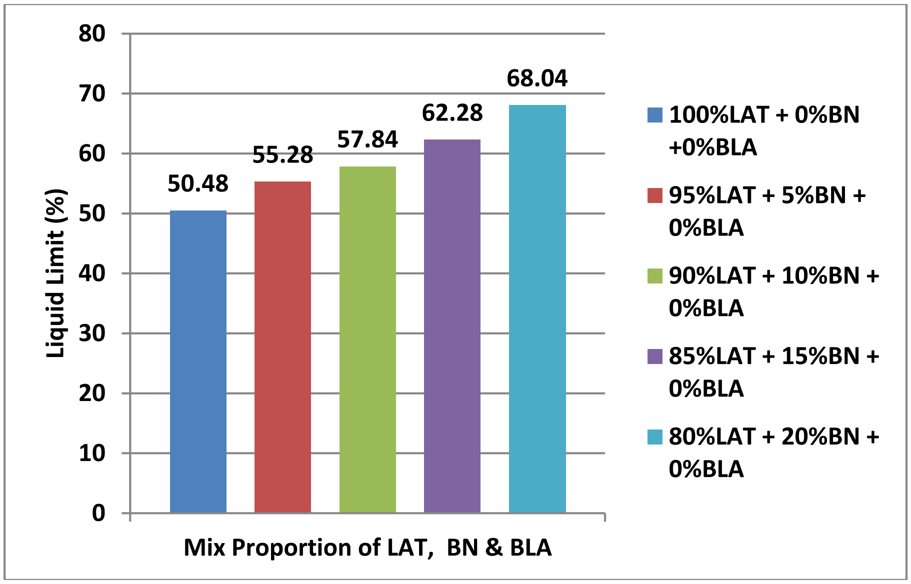


Figure 4.7: Liquid Limit Values for Lateritic Soil-Bentonite Mixtures at 0% BLA

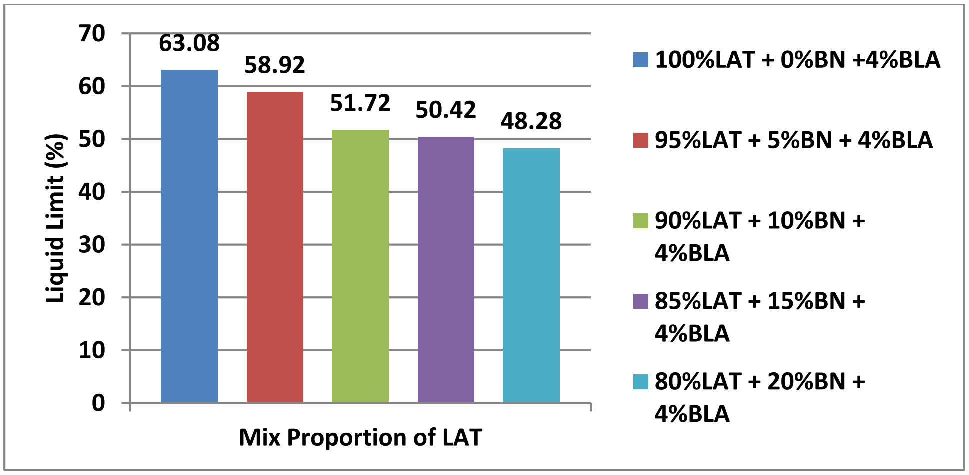


Figure 4.8: Liquid Limit Values for Lateritic Soil-Bentonite Mixtures at 4% BLA

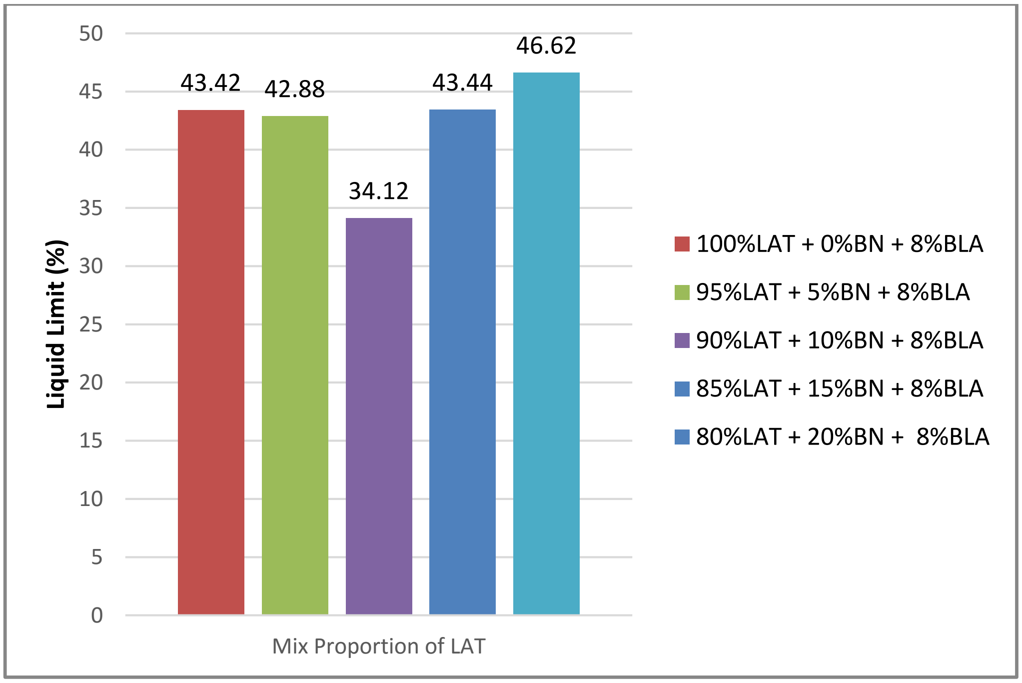


Figure 4.9: Liquid Limit Values for Lateritic Soil-Bentonite Mixtures at 8% BLA

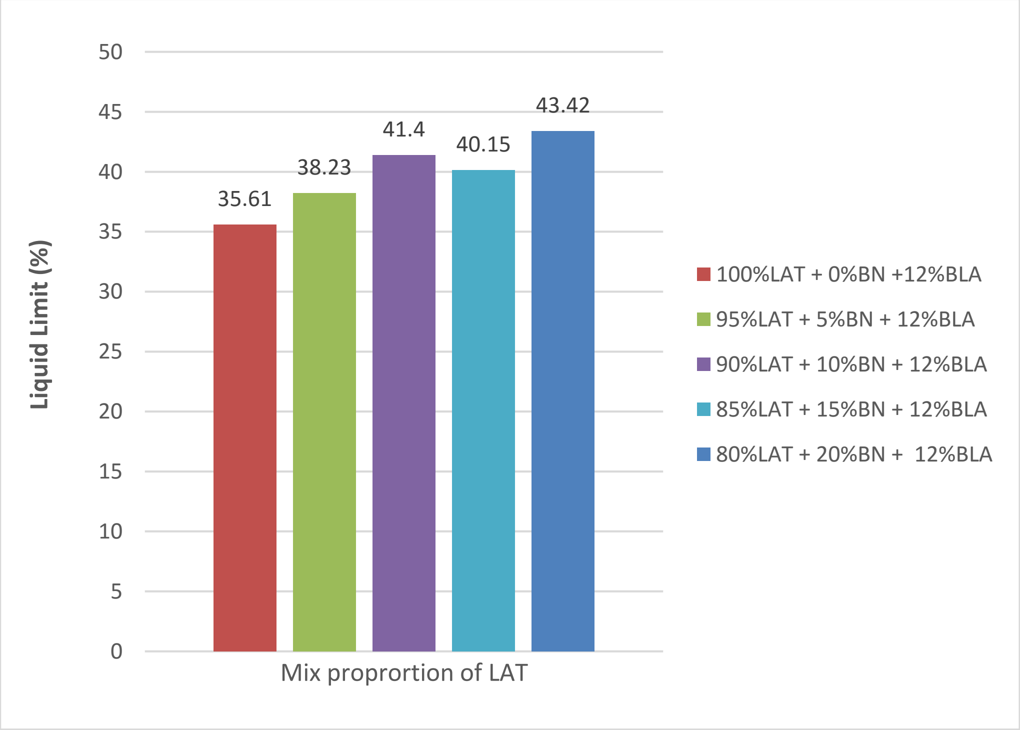


Figure 4.10: Liquid Limit Values for Lateritic Soil-Bentonite Mixtures at 12% BLA

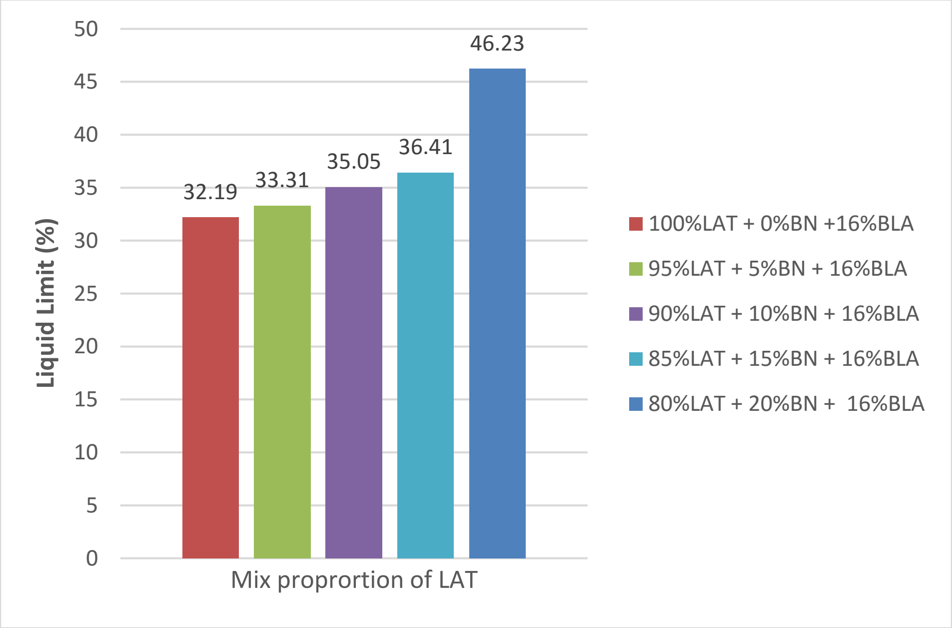


Figure 4.11: Liquid Limit Values for Lateritic Soil-Bentonite Mixtures at 16% BLA

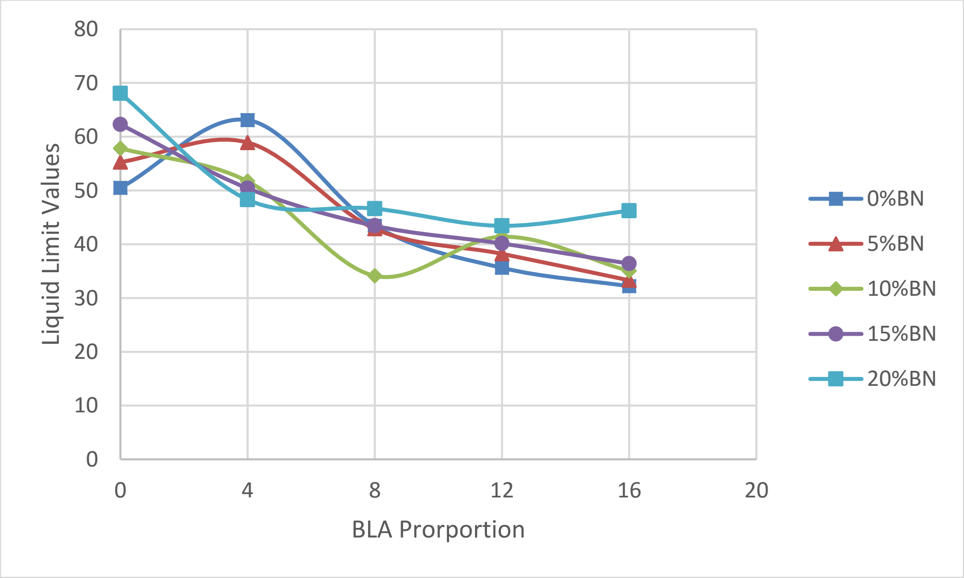


Figure 4.12: Liquid Limit graph for Lateritic Soil-Bentonite Mixtures at varying percentages of Bamboo Leaf Ash

Plastic Limit Results for Laterite- Bentonite Mixture treated with Bamboo Leaf Ash:

Figure 4.13 - 4.18 show the plastic limit results for lateritic soil-bentonite mixtures stabilized with bamboo leaf ash. It was observed that the plastic limit of lateritic soil increased consistent on addition of bentonite to lateritic soil. The increase in plastic limit of lateritic soil could be attributed to the expansive nature of bentonite. For lateritic soil-bentonite mixtures treated with bamboo leaf ash, it was observed that the plastic limit of lateritic soil-bentonite mixtures decreased on addition of bamboo leaf ash to the mixture.

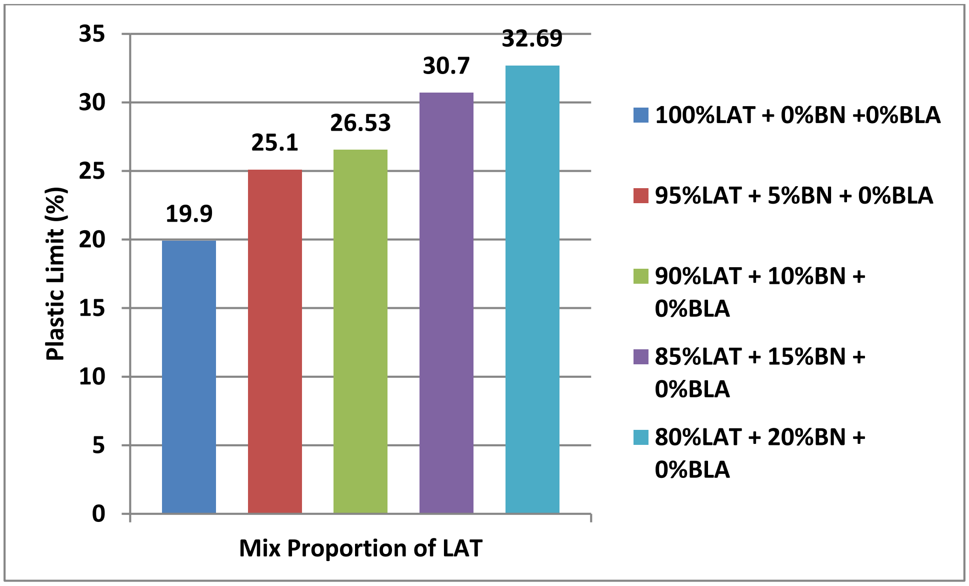


Figure 4.13: Plastic Limit Values for Lateritic Soil-Bentonite Mixtures at 0% BLA

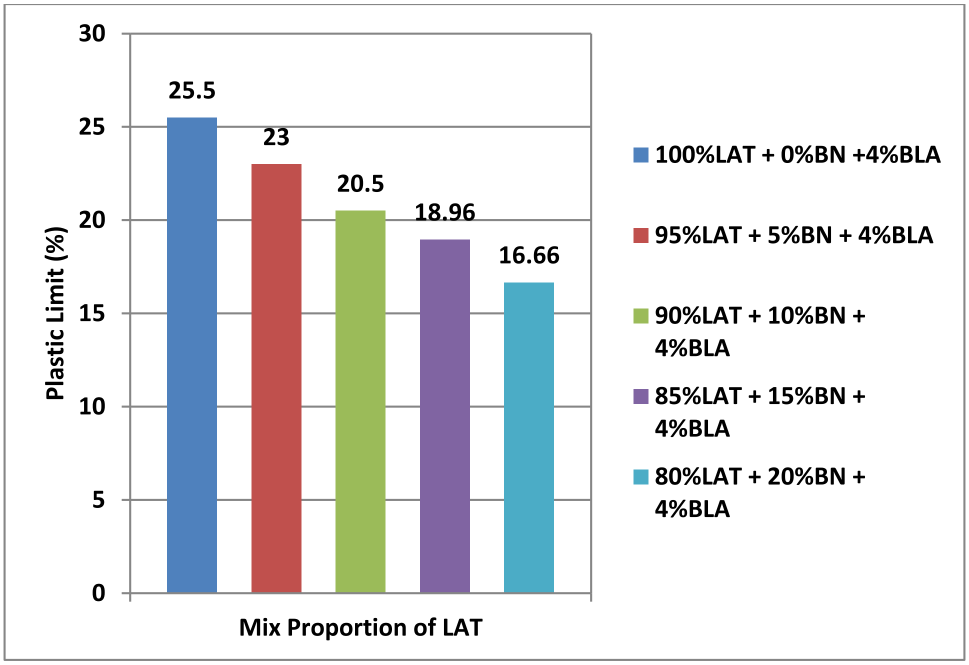


Figure 4.14: Plastic Limit Values for Lateritic Soil-Bentonite Mixtures at 4% BLA

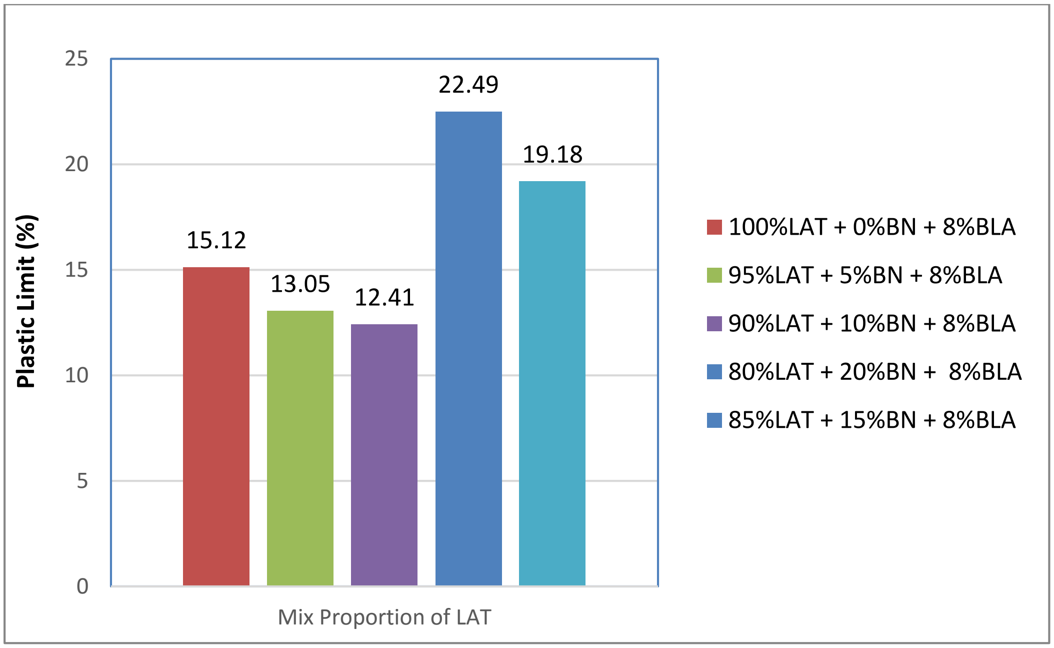


Figure 4.15: Plastic Limit Values for Lateritic Soil-Bentonite Mixtures at 8% BLA

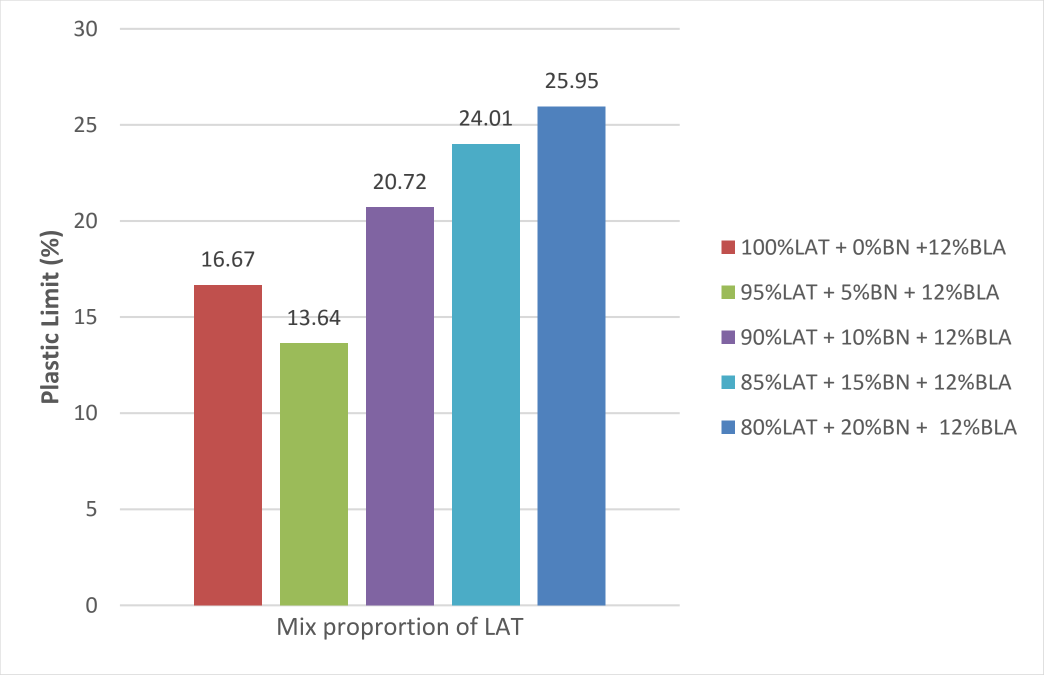


Figure 4.16: Plastic Limit Values for Lateritic Soil-Bentonite Mixtures at 12% BLA

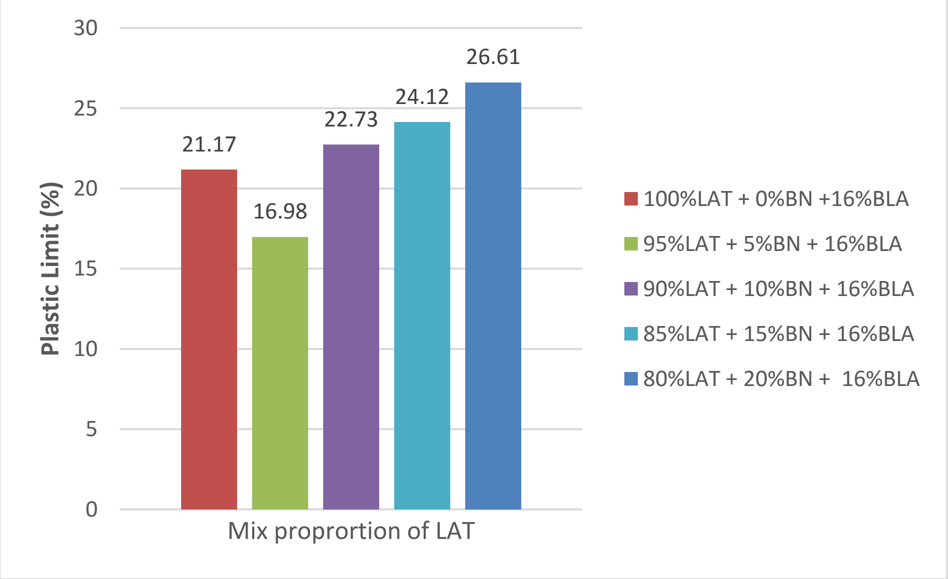


Figure 4.17: Plastic Limit Values for Lateritic Soil-Bentonite Mixtures at 16% BLA

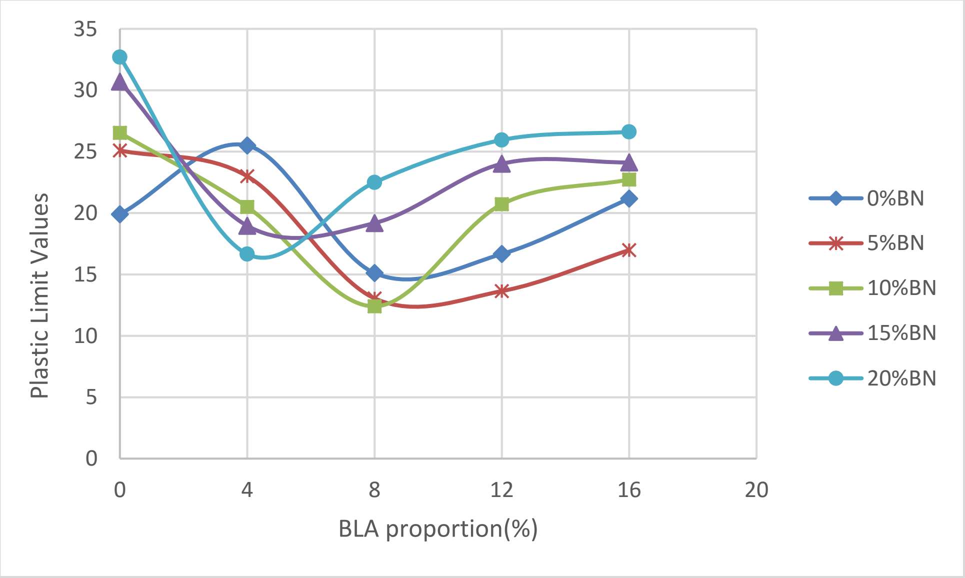


Figure 4.18: Plastic Limit Graph for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

Plasticity Index Results for Laterite- Bentonite Mixture treated with Bamboo Leaf Ash:

Figure 4.19 - 4.24 shows the plasticity index results for lateritic soil-bentonite mixtures treated with bamboo leaf ash. It was observed that the plasticity index of lateritic soil increased on addition of bentonite while for lateritic soil-bentonite mixtures treated with bamboo leaf ash, the plasticity index was found to decrease. The decrease in the liquid limit of laterite on addition of bamboo leaf ash could be attributed to the formation of cementitious compounds between the $Ca(OH)_2$ present in the soil the pozzolans in the bamboo leaf ash.

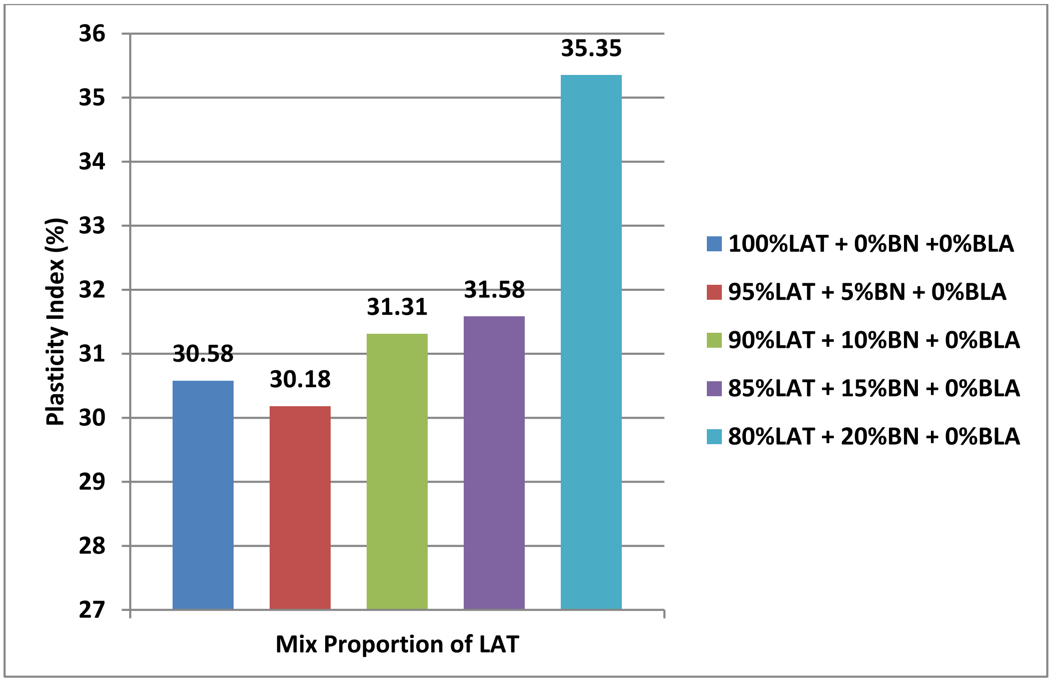


Figure 4.19: Plasticity index Values for Lateritic Soil-Bentonite Mixtures at 0% BLA

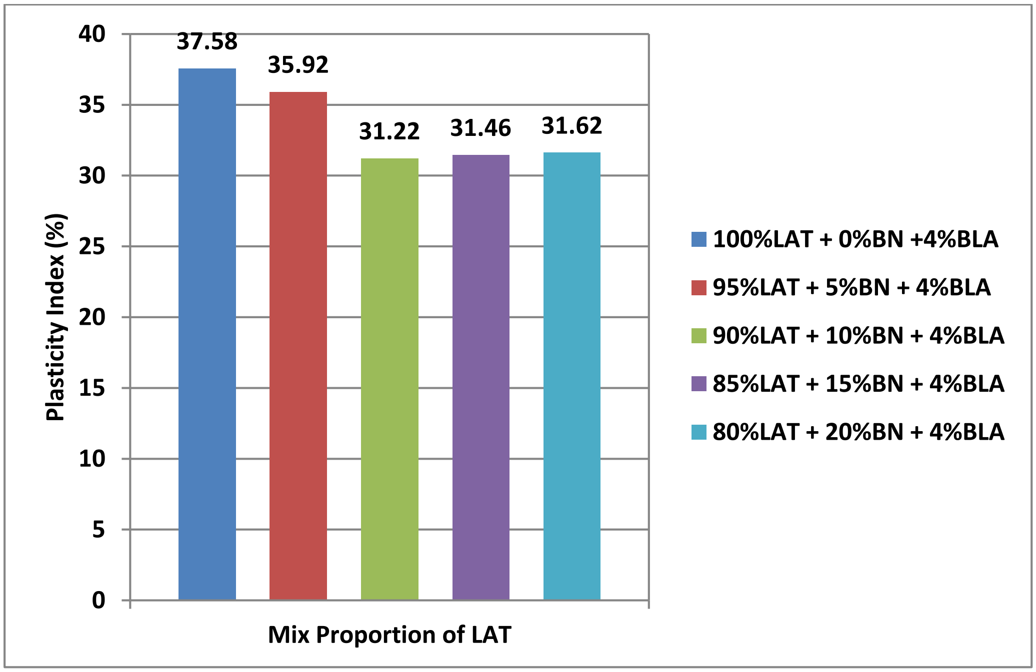


Figure 4.20: Plasticity index Values for Lateritic Soil-Bentonite Mixtures at 4% BLA

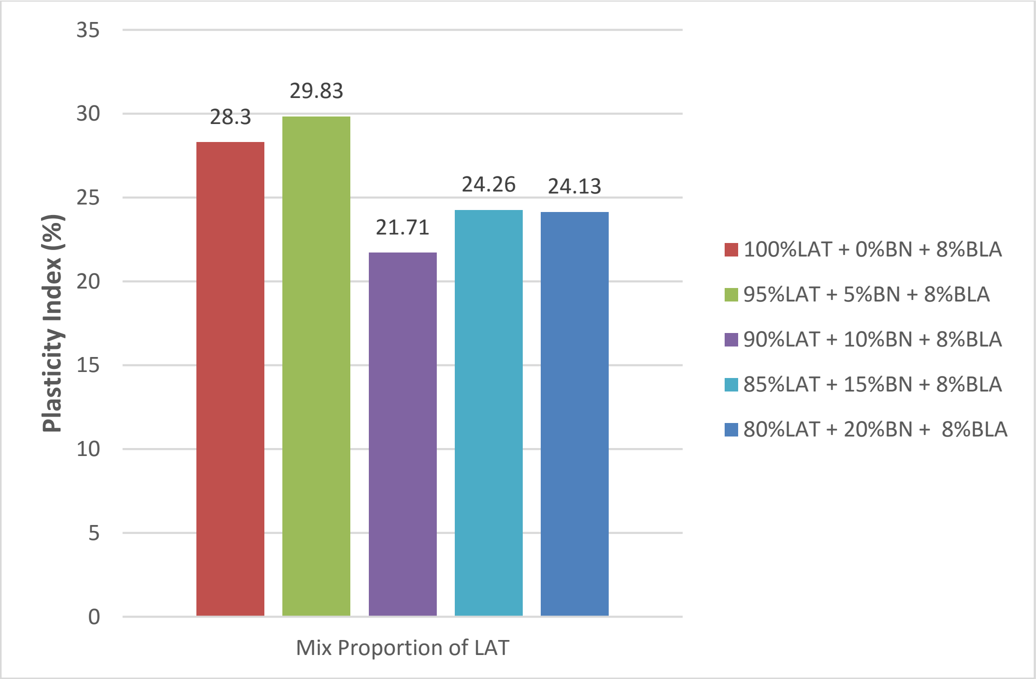


Figure 4.21: Plasticity index Values for Lateritic Soil-Bentonite Mixtures at 8% BLA

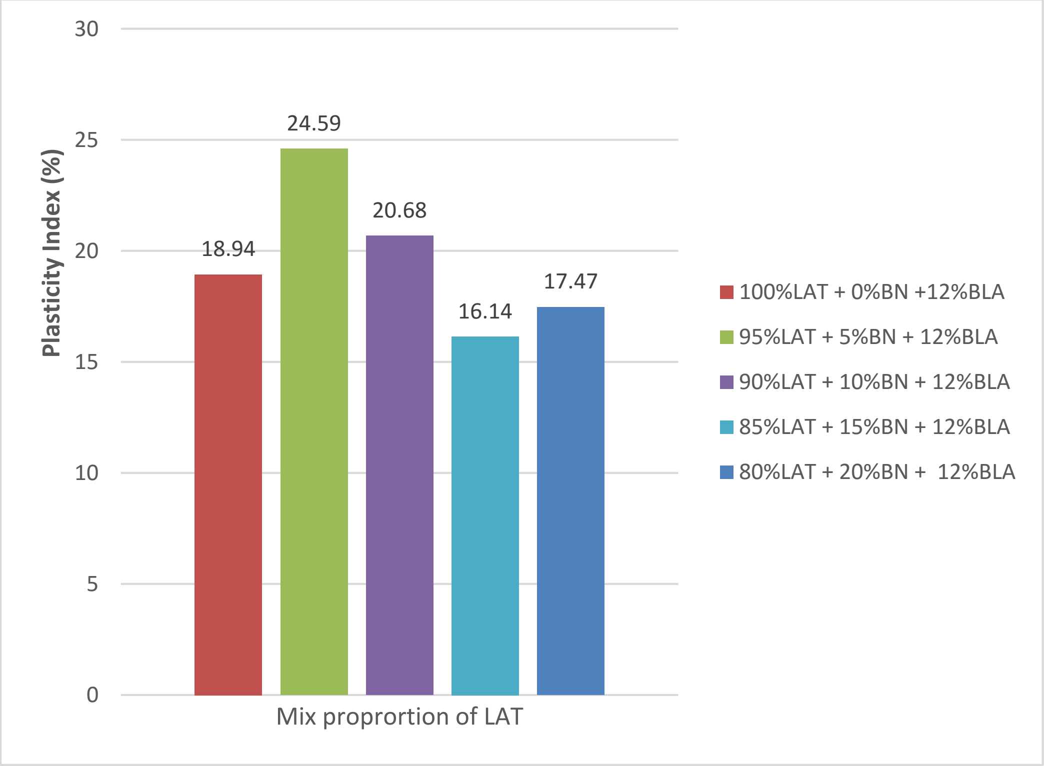


Figure 4.22: Plasticity index Values for Lateritic Soil-Bentonite Mixtures at 12% BLA

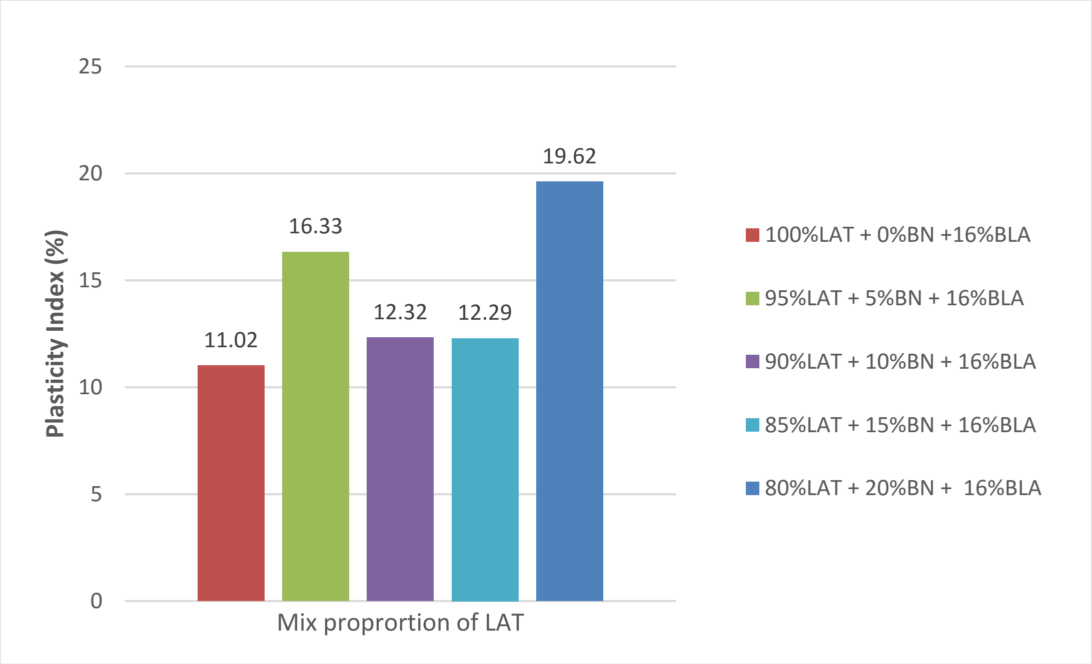


Figure 4.23: Plasticity index Values for Lateritic Soil-Bentonite Mixtures at 16% BLA

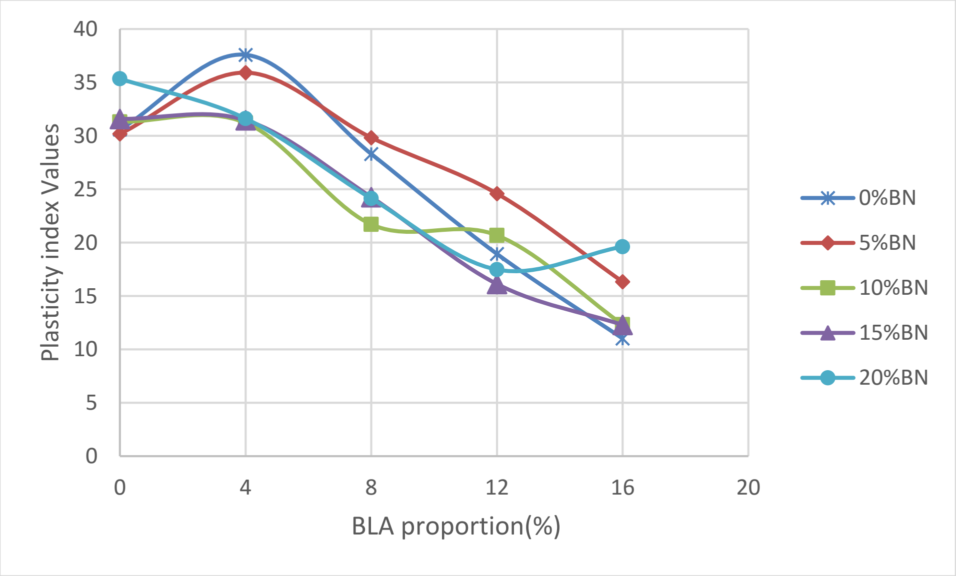


Figure 4.24: Plasticity Index Graph for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash.

4.14 Compaction Test

Maximum Dry Unit Weight and Optimum Moisture Content Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash:

Figure 4.25 shows the values of maximum dry unit weight of lateritic soil stabilized with bentonite and lateritic soil-bentonite mixtures treated with bamboo leaf ash. For lateritic soil-bentonite mixtures, the maximum dry unit weight of lateritic soil increased from 5% addition of bentonite to 15% addition of bentonite beyond 15% bentonite content, a decrease in maximum dry unit weight of lateritic soil was observed while for lateritic soil-bentonite mixtures treated with bamboo leaf ash, the maximum dry unit weight was found to increase with increase in bentonite content but decreased with increasing percentages of bamboo leaf ash. The decrease in maximum dry unit weight as the bamboo leaf ash increase could be attributed to the low density of bamboo leaf ash. Works indicative of these findings are the work of Taha and Taha, (2015).

Evaluation of the optimum moisture content obtained for lateritic soil-bentonite mixtures and lateritic soil-bentonite mixtures treated with bamboo leaf ash revealed that on consistent addition of bentonite to lateritic soil from 5% to 20% by weight of lateritic soil, the optimum moisture content of lateritic soil decreased from its natural value of 15% up to 15% bentonite content, beyond 15% bentonite content, the optimum moisture content decreased. While for lateritic soil-bentonite mixtures treated with bamboo leaf ash, the mixture showed a decreasing and increasing trend in optimum moisture content. The lowest optimum dry unit weight was obtained for lateritic soil-bentonite mixtures to bamboo leaf ash mixture at 15% and 8% respectively; it implies less water content will be required to achieve maximum dry unit weight during field compaction.

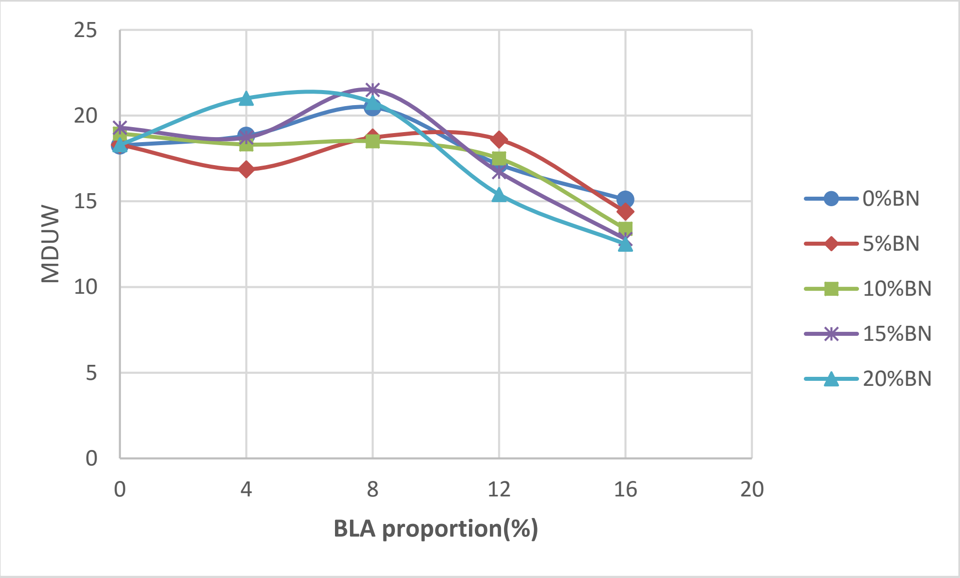


Figure 4.25: Maximum Dry Unit Weight Graph for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

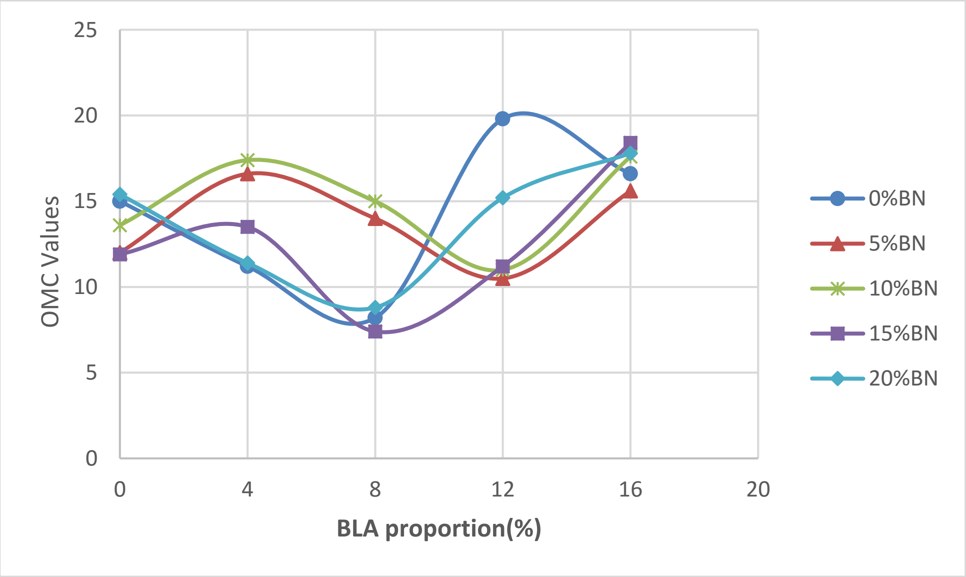


Figure 4.26: Optimum Moisture Content Graph for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

4.1.5 Hydraulic Conductivity Test

Hydraulic Conductivity Test Results for Laterite- Bentonite Mixture treated with Bamboo Leaf Ash:

Figure 4.27 shows the hydraulic conductivity of lateritic soil-bentonite mixtures and lateritic soil-bentonite mixtures treated with bamboo leaf ash. There were no significant changes observed from hydraulic conductivity values of the different mix proportions of lateritic soil-bentonite mixtures treated with bamboo leaf ash. Nevertheless, the hydraulic conductivity values of all the mix proportions were less than 10^{-7} m/s which meets the requirements of landfill for inert waste according to the EU Landfill Directive (1999/31/EC).

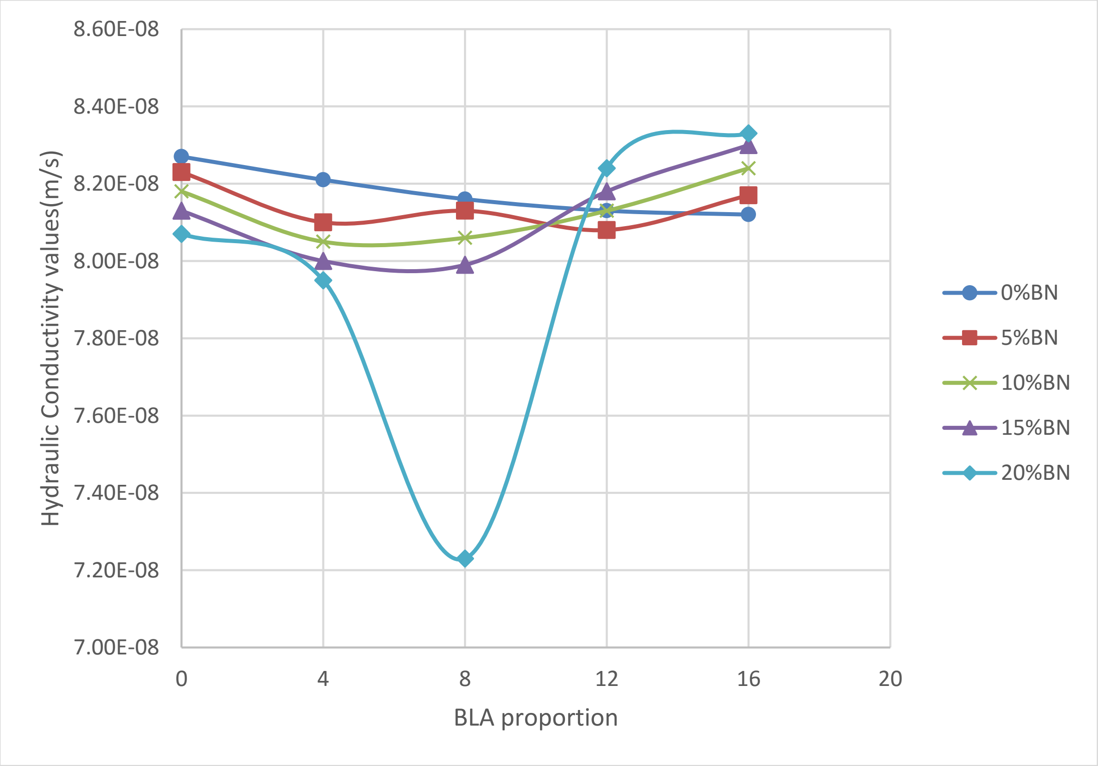


Figure 4.27: Hydraulic Conductivity of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

4.1.6 Unconfined Compressive Strength Test

Unconfined Compressive Strength Test Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash:

Figure 4.28 shows the Unconfined Compressive Strength test results of lateritic soil-bentonite mixtures and lateritic soil-bentonite mixtures treated with bamboo leaf ash, it was observed that there was a general increase in the Unconfined Compressive Strength (UCS) values of lateritic soil-bentonite mixtures treated with bamboo leaf ash from 103.86kN/m^2 to 245.15kN/m^2. The UCS value of untreated lateritic soil increased from 103.86kN/m2 to 200.59kN/m^2 on addition of bentonite only while it increased from 103.86kN/m^2 to 130.12kN/m^2 on addition of bamboo leaf ash only.

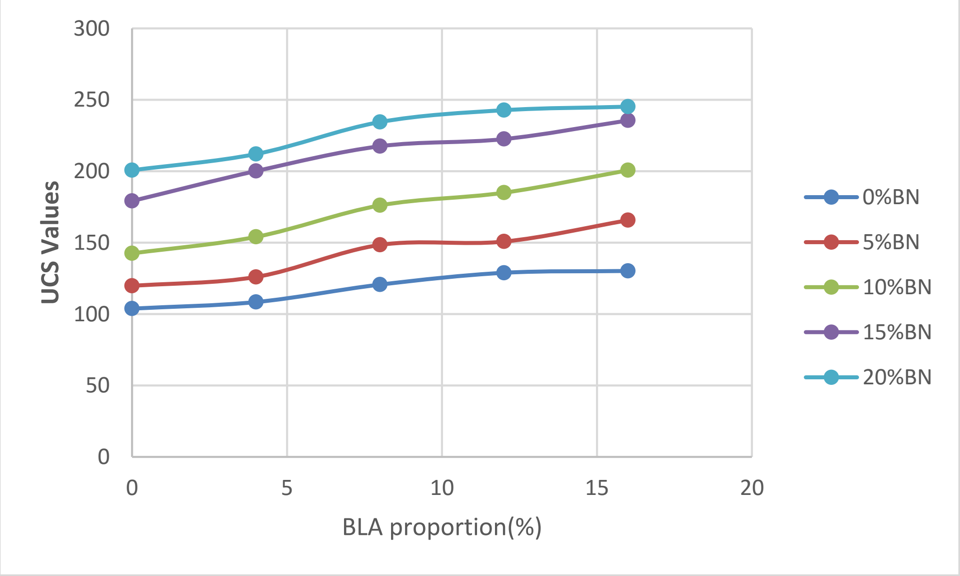


Figure 4.28: Unconfined Compressive Strength for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

4.1.7 Volumetric shrinkage Test

Volumetric shrinkage Test Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash:

Figure 4.29 shows the values of volumetric shrinkage of laterite stabilized with bentonite and lateritic soil-bentonite mixtures treated with bamboo leaf ash. It was observed that the volumetric shrinkage strain increased generally for specimen with higher moulding water content and it decreased with specimen compacted with lower water content. There was no steady trend observed on the addition of bamboo leaf ash but on addition of 16% bamboo leaf ash, 3 specimens exceeded the maximum permissible VSS value of 4%.

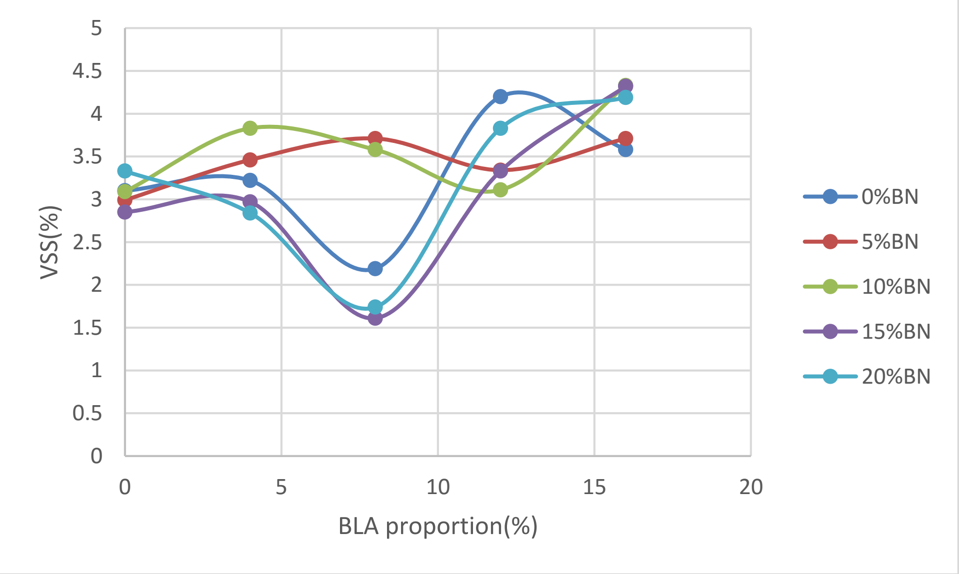


Figure 4.29: Volumetric shrinkage for Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

Statistical Analysis

Table 4.1: Anova: Two-Factor Without Replication for Specific Gravity of Lateritic Soil-Bentonite Mixtures at varying percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	0.160216	4	0.040054	24.8551	1.08E-06	3.006917
Columns	0.017336	4	0.004334	2.68942	0.068914	3.006917
Error	0.025784	16	0.001612			

Total	0.203336	24				

Table 4.1 shows the Anova: Two-Factor Without Replication results for specific gravity of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the specific gravity of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

Table 4.2: Anova: Two-Factor Without Replication for Liquid Limit of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1882.937	4	470.7343	16.37293	1.64E-05	3.006917
Columns	125.7219	4	31.43047	1.093204	0.393242	3.006917
Error	460.0123	16	28.75077			
Total	2468.672	24				

Table 4.2 shows the Anova: Two-Factor Without Replication results for liquid limit of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the specific gravity of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

Table 4.3: Anova: Two-Factor Without Replication for Plastic Limit of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	291.0102	4	72.75255	5.033982	0.008062	3.006917
Columns	145.1162	4	36.27905	2.510264	0.082956	3.006917
Error	231.2366	16	14.45229			
Total	667.363	24				

Table 4.3 shows the Anova: Two-Factor Without Replication results for plastic limit of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the plastic limit of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

Table 4.4: Anova: Two-Factor Without Replication for Plasticity Index of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	1318.348	4	329.587	40.22026	3.71E-08	3.006917
Columns	59.81342	4	14.95335	1.824792	0.173412	3.006917
Error	131.1128	16	8.194551			
Total	1509.274	24				

Table 4.4 shows the Anova: Two-Factor Without Replication results for plasticity index of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the plasticity index of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

Table 4.5: Anova: Two-Factor Without Replication for Maximum Dry Unit Weight of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*Df*	*MS*	*F*	*P-value*	*F crit*
Rows	120.2123	4	30.05306	18.85004	6.7E-06	3.006917
Columns	1.452136	4	0.363034	0.227704	0.918854	3.006917
Error	25.50918	16	1.594324			
Total	147.1736	24				

Table 4.5 shows the Anova: Two-Factor Without Replication results for maximum dry unit weight of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the maximum dry unit weight of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

Table 4.6: Anova: Two-Factor Without Replication for Optimum Moisture Content of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Rows	107.2936	4	26.8234	2.934438	0.053752	3.006917
Columns	15.6816	4	3.9204	0.428886	0.785689	3.006917
Error	146.2544	16	9.1409			
Total	269.2296	24				

Table 4.6 shows the Anova: Two-Factor Without Replication results for optimum moisture content of lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of both bentonite and bamboo leaf ash to the mixture there was no significant effect.

Table 4.7: Anova: Two-Factor Without Replication for Hydraulic Conductivity of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	SS	Df	MS	F	P-value	F crit
Rows	3.08E-17	4	7.71E-18	2.145797	0.12214	3.006917
Columns	1.38E-17	4	3.44E-18	0.956934	0.457467	3.006917
Error	5.75E-17	16	3.59E-18			
Total	1.02E-16	24				

Table 4.7 shows the Anova: Two-Factor Without Replication results for hydraulic conductivity of lateritic soils bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of both bentonite and bamboo leaf ash to the mixture there was no significant effect on the hydraulic conductivity.

Table 4.8: Anova: Two-Factor Without Replication for Unconfined Compressive Strength of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*Df*	*MS*	*F*	*P-value*	*F crit*
Rows	7205.447	4	1801.362	64.13161	1.2E-09	3.006917
Columns	41438.93	4	10359.73	368.8245	1.57E-15	3.006917
Error	449.4163	16	28.08852			
Total	49093.79	24				

Table 4.9 shows the Anova: Two-Factor Without Replication results for unconfined compressive strength of lateritic soils bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of both bentonite and bamboo leaf ash to the mixture has significant effect on the unconfined compressive strength.

Table 4.10: Anova: Two-Factor Without Replication for Volumetric shrinkage of Lateritic Soil-Bentonite Mixtures at Varying Percentages of Bamboo Leaf Ash

ANOVA						
Source of Variation	*SS*	*Df*	*MS*	*F*	*P-value*	*F crit*
Rows	5.93868	4	1.48467	4.636515	0.01121	3.006917
Columns	0.99252	4	0.24813	0.774892	0.557409	3.006917
Error	5.1234	16	0.320213			
Total	12.0546	24				

Table 4.10 shows the Anova: Two-Factor Without Replication results for volumetric shrinkage of lateritic soils bentonite mixtures treated with bamboo leaf ash. From the results, it was observed that on the addition of bentonite to lateritic soil it has significant effects on the volumetric shrinkage of the soil while on the addition of bamboo leaf ash to the mixture has no significant effects.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the findings obtained on geotechnical properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash, the following conclusion can be drawn:

1 Particle size distribution test for lateritic soil revealed that the particle size distribution test results for the natural lateritic soil, the passing through sieve No 200 was 54.54%, coefficient of uniformity and coefficient of curvature was 0 and as a results the lateritic soil sample is classified as A-7-6 according to AASHTO Soil Classification System and CH (clay of high plasticity) according to Unified Soil Classification.

2 Specific gravity test results for the specific gravity results for lateritic soil-bentonite mixtures treated with bamboo leaf ash. From the results for lateritic soil-bentonite mixtures, it was observed that addition of bentonite to lateritic soil increased the specific gravity of lateritic soil from its natural value of 2.64 to 2.66 at 5% bentonite content, beyond 5% bentonite content, the specific gravity of lateritic soil decreased.

3 From atterberg limit test conducted on the lateritic soil-bentonite mixtures and lateritic soil-bentonite mixtures treated with bamboo leaf ash, it was observed that addition of bentonite to lateritic soil from 5% to 20% by weight of lateritic soil increased the liquid limit and plastic limit of lateritic while for lateritic soil-bentonite mixtures treated with bamboo leaf ash; it was observed that the liquid limit and plastic limit of lateritic soil-bentonite mixtures decreased with increase in bamboo leaf ash. Similarly, plasticity index (PI) decreased on addition of bamboo leaf ash. The reduction in plasticity index shows improvement in the swelling properties of the soil.

4 Compaction test results revealed that for lateritic soil-bentonite mixtures the maximum dry unit weight of the soil increased generally on addition of 0-8% bamboo leaf ash, while it decreased gradually from 8-16% bamboo leaf ash addition.

5 There were no significant changes observed from hydraulic conductivity values of the different mix proportions of lateritic soil-bentonite mixtures treated with bamboo leaf ash. Nevertheless, the hydraulic conductivity values of all the mix proportions were less than

10^{-7} m/s which meets the requirements of landfill for inert waste according to the EU Landfill Directive (1999/31/EC).

6 The treatment of lateritic soil-bentonite mixtures with bamboo leaf ash yielded improvement in the UCS test, it was observed that there was a general increase in the unconfined compressive strength (UCS) values of lateritic soil-bentonite mixtures treated with bamboo leaf ash from 103.86kN/m^2 to 245.15kN/m^2. The mix ratios with 20% bentonite met the minimum unconfined compressive strength requirement of greater than or equal to 200kN/m^2 for landfill liner construction.

7 The volumetric shrinkage test results of the specimens show that the volumetric shrinkage strain varies directly to the moulding water content.

5.2 Recommendations

From the findings obtained on geotechnical properties of lateritic soil-bentonite mixtures treated with bamboo leaf ash, the following recommendation can be made:

1 Lateritic soil-bentonite mixtures treated with bamboo leaf ash can be used as materials for landfill liner construction.

2 In order to get the minimum requirements for landfill liner, high bentonite and bamboo leaf ash content is required to get effective results.

3 Lateritic soil-bentonite mixtures treated with bamboo leaf ash can be used at the sub-grade level of pavement construction, the suitability of lateritic soil-bentonite mixtures for use at the sub-grade of foundation is due to their low liquid limit and plasticity index.

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APPENDICES

APPENDIX A

Specific Gravity Test

Table A1. Specific Gravity Result for 100%LAT + 0%BN + 0%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.52	25.94	25.16
Wt of bottle + dry soil, W_2 (g).	34.52	35.93	35.15
Wt of bottle + soil + water, W_3 (g).	84.92	82.19	86.24
Wt of bottle + water, W_4 (g).	78.71	75.94	80.05

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100% LAT + 0%BN + 0% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.52-24.52)}{(34.52-24.52)-(84.92-78.71)} = 2.63$

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.93-25.94)}{(35.93-25.94)-(82.19-75.94)} = 2.67$

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.15-25.16)}{(35.15-25.16)(86.24-80.05)} = 2.63$

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.63+2.67+2.63)}{3} = 2.64$

Table A2. Specific Gravity Result for 95%LAT + 5%BN + 0%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.16	26.34	24.92
Wt of bottle + dry soil, W_2 (g).	35.16	36.33	34.92
Wt of bottle + soil + water, W_3 (g).	79.48	82.44	84.63
Wt of bottle + water, W_4 (g).	73.24	76.18	78.4

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 95% LAT + 5%BN + 0%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.16-25.16)}{(35.16-25.16)-(79.48-73.24)}$ = 2.66

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.33-26.34)}{(36.33-26.34)-(82.44-76.18)}$ = 2.68

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.92-24.92)}{(34.92-24.92)(84.63-78.4)}$ = 2.65

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

Table A3. Specific Gravity Result for 90%LAT + 10%BN + 0%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.14	25.88	25.26
Wt of bottle + dry soil, W_2 (g).	36.14	35.87	35.25
Wt of bottle + soil + water, W_3 (g).	86.14	84.26	82.14

Wt of bottle + water, W_4 (g).	79.96	78.13	76.02

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 85% LAT + 15%BN + 0% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)-(86.14-79.96)}$ = 2.62

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.87-25.88)}{(35.87-25.88)-(84.26-78.13)}$ = 2.59

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.25-25.26)}{(35.25-25.26)(82.14-76.02)}$ = 2.58

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.62+2.59+2.58)}{3}$ = 2.60

Table A4. Specific Gravity Result for 85%LAT + 15%BN + 0%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.42	25.48	24.82
Wt of bottle + dry soil, W_2 (g).	36.41	35.46	34.81
Wt of bottle + soil + water, W_3 (g).	82.44	78.84	84.38
Wt of bottle + water, W_4 (g).	76.40	72.77	78.28

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80% LAT + 20%BN + 0% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.41-26.42)}{(36.41-26.42)-(82.44-76.4)}$ = 2.53

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.46-25.48)}{(35.46-25.48)-(78.84-72.77)}$ = 2.55

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(34.81-24.82)}{(34.81-24.82)(84.38-78.28)}$ = 2.57

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.53+2.55+2.57)}{3}$ = 2.55

Table A5. Specific Gravity Result for 80%LAT + 20%BN + 0%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.16	26.42	24.73
Wt of bottle + dry soil, W_2 (g).	35.16	36.41	34.73
Wt of bottle + soil + water, W_3 (g).	84.83	82.11	87.24
Wt of bottle + water, W_4 (g).	78.81	76.15	81.29

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 60% LAT + 40%BN

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.16-25.16)}{(35.16-25.16)-(84.83-78.81)}$ = 2.51

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(36.41-26.42)}{(36.41-26.42)-(82.11-76.15)}$ = 2.48

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(34.73-24.73)}{(34.73-24.73)(87.24-81.29)}$ = 2.47

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.51+2.48+2.47)}{3}$ = 2.49

Table A6: Specific Gravity Result for 100%LAT + 0%BN + 4% BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.48	24.78	26.14
Wt of bottle + dry soil, W_2 (g).	35.47	34.78	36.14
Wt of bottle + soil + water, W_3 (g).	80.48	78.24	82.18
Wt of bottle + water, W_4 (g).	74.26	71.99	75.9

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100%LAT + 0%BN + 4%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.47-25.48)}{(35.47-25.48)-(80.48-74.26)}$ = 2.65

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.78-24.78)}{(34.78-24.78)-(78.24-71.99)}$ = 2.67

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)(82.18-75.9)}$ = 2.69

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.65+2.67+2.69)}{3}$ = 2.67

Table A7: Specific Gravity Result for 95%LAT + 5%BN + 4%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.72	25.48	26.28
Wt of bottle + dry soil, W_2 (g).	34.71	35.46	36.27

Wt of bottle + soil + water, W_3 (g).	82.64	86.32	78.88
Wt of bottle + water, W_4 (g).	76.36	80.02	72.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 95% LAT + 5%BN + 4%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.71-24.72)}{(34.71-24.72)-(82.64-76.36)}$ = 2.69

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.46-25.48)}{(35.46-25.48)-(86.32-80.02)}$ = 2.71

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(36.27-26.28)}{(36.27-26.28)(78.88-72.56)}$ = 2.72

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.69+2.71+2.72)}{3}$ = 2.71

Table A8: Specific Gravity Result for 90%LAT + 10%BN + 4%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.54	26.28	24.48
Wt of bottle + dry soil, W_2 (g).	35.54	36.26	34.47
Wt of bottle + soil + water, W_3 (g).	81.11	83.19	82.09
Wt of bottle + water, W_4 (g).	74.8	76.92	75.71

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90% LAT + 10%BN + 4% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.54-25.54)}{(35.54-25.54)-(81.11-74.8)}$ = 2.71

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.26-26.28)}{(36.26-26.28)-(83.19-76.92)}$ = 2.69

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.47-24.48)}{(34.47-24.48)(82.09-75.71)}$ = 2.76

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.71+2.69+2.76)}{3}$ = 2.72

Table A9: Specific Gravity Result for 85%LAT + 15%BN + 4%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.42	25.83	24.91
Wt of bottle + dry soil, W_2 (g).	36.42	35.82	34.91
Wt of bottle + soil + water, W_3 (g).	83.33	86.78	85.14
Wt of bottle + water, W_4 (g).	76.99	80.41	78.74

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 85% LAT + 15%BN + 4% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.42-26.42)}{(36.42-26.42)-(83.33-76.99)}$ = 2.73

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.82-25.83)}{(35.82-25.83)-(86.78-80.41)}$ = 2.76

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.91-24.91)}{(34.91-24.91)-(85.14-78.74)}$ = 2.78

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.73+2.76+2.78)}{3}$ = 2.76

Table A10: Specific Gravity Result for 80%LAT + 20%BN + 4%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.14	25.28	24.82
Wt of bottle + dry soil, W_2 (g).	36.14	35.26	34.80
Wt of bottle + soil + water, W_3 (g).	88.11	87.64	86.41
Wt of bottle + water, W_4 (g).	81.88	81.41	80.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80%LAT + 20%BN + 4% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)-(88.11-81.88)}$ = 2.65

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.26-25.28)}{(35.26-25.28)-(87.64-81.41)}$ = 2.66

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.80-24.82)}{(34.80-24.82)-(86.41-80.14)}$ = 2.69

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.65+2.66+2.69)}{3}$ = 2.67

Table A11: Specific Gravity Result for 100% LAT + 0%BN + 8%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.48	24.78	26.14
Wt of bottle + dry soil, W_2 (g).	35.47	34.78	36.14

Wt of bottle + soil + water, W_3 (g).	80.48	78.24	82.18
Wt of bottle + water, W_4 (g).	74.26	71.99	75.9

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100%LAT + 0%BN + 8%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.47-25.48)}{(35.47-25.48)-(80.48-74.26)}$ = 2.65

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.78-24.78)}{(34.78-24.78)-(78.24-71.99)}$ = 2.67

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)(82.18-75.9)}$ = 2.69

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.65+2.67+2.69)}{3}$ = 2.67

Table A12: Specific Gravity Result for 95%LAT + 5%BN + 8% BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.72	25.48	26.28
Wt of bottle + dry soil, W_2 (g).	34.71	35.46	36.27
Wt of bottle + soil + water, W_3 (g).	82.64	86.32	78.88
Wt of bottle + water, W_4 (g).	76.36	80.02	72.56

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 95% LAT + 5% BN + 8% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.71-24.72)}{(34.71-24.72)-(82.64-76.36)}$ = 2.69

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.46-25.48)}{(35.46-25.48)-(86.32-80.02)}$ = 2.71

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(36.27-26.28)}{(36.27-26.28)(78.88-72.56)}$ = 2.72

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.69+2.71+2.72)}{3}$ = 2.71

Table A13: Specific Gravity Result for 90%LAT + 10%BN + 8%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.54	26.28	24.48
Wt of bottle + dry soil, W_2 (g).	35.54	36.26	34.47
Wt of bottle + soil + water, W_3 (g).	81.11	83.19	82.09
Wt of bottle + water, W_4 (g).	74.8	76.92	75.71

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90% LAT + 10% BN + 8% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.54-25.54)}{(35.54-25.54)-(81.11-74.8)}$ = 2.71

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.26-26.28)}{(36.26-26.28)-(83.19-76.92)}$ = 2.69

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.47-24.48)}{(34.47-24.48)(82.09-75.71)}$ = 2.76

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.71+2.69+2.76)}{3}$ = 2.72

Table A14: Specific Gravity Result for 85%LAT + 15%BN + 8% BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.42	25.83	24.91
Wt of bottle + dry soil, W_2 (g).	36.42	35.82	34.91
Wt of bottle + soil + water, W_3 (g).	83.33	86.78	85.14
Wt of bottle + water, W_4 (g).	76.99	80.41	78.74

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 85% LAT + 15% BN + 8% BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.42-26.42)}{(36.42-26.42)-(83.33-76.99)}$ = 2.73

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.82-25.83)}{(35.82-25.83)-(86.78-80.41)}$ = 2.76

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.91-24.91)}{(34.91-24.91)-(85.14-78.74)}$ = 2.78

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.73+2.76+2.78)}{3}$ = 2.76

Table A15: Specific Gravity Result for 80% LAT + 20%BN + 8%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	26.14	25.28	24.82
Wt of bottle + dry soil, W_2 (g).	36.14	35.26	34.80

Wt of bottle + soil + water, W_3 (g).	88.11	87.64	86.41
Wt of bottle + water, W_4 (g).	81.88	81.41	80.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80% LAT + 20%BN + 8%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.14-26.14)}{(36.14-26.14)-(88.11-81.88)}$ = 2.65

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.26-25.28)}{(35.26-25.28)-(87.64-81.41)}$ = 2.66

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.80-24.82)}{(34.80-24.82)-(86.41-80.14)}$ = 2.69

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.65+2.66+2.69)}{3}$ = 2.67

Table A16: Specific Gravity Result for 100%LAT + 0%BN + 12%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.12	24.86	25.42
Wt of bottle + dry soil, W_2 (g).	35.11	35.85	35.42
Wt of bottle + soil + water, W_3 (g).	80.09	82.75	79.71
Wt of bottle + water, W_4 (g).	73.96	76.63	73.60

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100%LAT + 0%BN + 12%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.11-25.12)}{(35.11-25.12)-(80.09-73.96)}$ = 2.59

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.85-24.86)}{(35.85-24.86)-(82.75-76.63)}$ = 2.58

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.42-25.42)}{(35.42-25.42)(79.71-73.60)}$ =2.57

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.59+2.58+2.57)}{3}$ = 2.58

Table A17: Specific Gravity Result for 95%LAT + 5%BN + 12%BLA

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 95%LAT + 5%BN + 12%BLA

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

Table A18: Specific Gravity Result for 90%LAT + 10%BN + 12%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.41	24.70	24.11
Wt of bottle + dry soil, W_2 (g).	35.41	34.68	34.11
Wt of bottle + soil + water, W_3 (g).	81.32	83.41	78.88
Wt of bottle + water, W_4 (g).	75.25	77.37	72.88

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90%LAT + 10%BN + 12%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.41-25.41)}{(35.41-25.41)-(81.32-75.25)}$ = 2.54

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(34.68-24.70)}{(34.68-24.70)-(83.41-77.37)}$ = 2.53

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(34.11-24.11)}{(34.11-24.11)(78.88-72.88)}$ =2.50

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.54+2.53+2.50)}{3}$ = 2.52

Table A19: Specific Gravity Result for 85%LAT + 15%BN + 12%BLA.

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.44	24.69	25.12
Wt of bottle + dry soil, W_2 (g).	34.43	34.68	35.12

Wt of bottle + soil + water, W_3 (g).	82.51	84.65	81.07
Wt of bottle + water, W_4 (g).	76.54	78.75	75.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 85%LAT + 15%BN + 12%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.43-24.44)}{(34.43-24.44)-(84.65-78.75)}$ = 2.49

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.68-24.69)}{(34.68-24.69)-(84.65-78.75)}$ = 2.50

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.12-25.12)}{(35.12-25.12)(81.07-75.14)}$ =2.46

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.49+2.50+2.46)}{3}$ = 2.48

Table A20: Specific Gravity Result for 80%LAT + 20%BN + 12%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.81	25.38	25.65
Wt of bottle + dry soil, W_2 (g).	35.80	35.37	35.65
Wt of bottle + soil + water, W_3 (g).	80.02	83.31	79.37
Wt of bottle + water, W_4 (g).	74.01	77.35	73.34

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80%LAT + 20%BN + 12%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.80-25.81)}{(35.80-25.81)-(80.02-74.01)}$ = 2.51

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.37-25.38)}{(35.37-25.38)-(83.31-77.35)}$ = 2.48

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(35.65-25.65)}{(35.65-25.65)(79.37-73.34)}$ =2.52

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.51+2.48+2.52)}{3}$ = 2.50

Table A21: Specific Gravity Result for 100%LAT + 0%BN + 16%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.12	24.86	25.42
Wt of bottle + dry soil, W_2 (g).	35.11	35.85	35.42
Wt of bottle + soil + water, W_3 (g).	80.09	82.75	79.71
Wt of bottle + water, W_4 (g).	73.96	76.63	73.60

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 100%LAT + 0%BN + 16%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.11-25.12)}{(35.11-25.12)-(80.09-73.96)}$ = 2.59

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.85-24.86)}{(35.85-24.86)-(82.75-76.63)}$ = 2.58

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(35.42-25.42)}{(35.42-25.42)(79.71-73.60)}$ =2.57

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.59+2.58+2.57)}{3}$ = 2.58

Table A22: Specific Gravity Result for 95%LAT + 5%BN + 16%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.42	24.38	24.62
Wt of bottle + dry soil, W_2 (g).	35.41	34.38	34.62
Wt of bottle + soil + water, W_3 (g).	80.14	83.32	84.16
Wt of bottle + water, W_4 (g).	74.08	77.23	78.05

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 95%LAT + 5%BN + 16%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.41-25.42)}{(35.41-25.42)-(80.14-74.08)}$ = 2.54

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.38-24.38)}{(34.38-24.38)-(83.32-77.23)}$ = 2.56

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.62-24.62)}{(34.62-24.62)-(84.16-78.05)}$ = 2.57

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.54+2.56+2.57)}{3}$ = 2.56

Table A23: Specific Gravity Result for 90%LAT + 10%BN + 16%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.90	25.11	24.82
Wt of bottle + dry soil, W_2 (g).	34.89	35.11	34.79

Wt of bottle + soil + water, W_3 (g).	83.68	78.30	80.42
Wt of bottle + water, W_4 (g).	77.64	72.22	74.42

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 90%LAT + 10%BN + 16%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.89-24.90)}{(35.89-24.90)-(83.68-77.64)}$ = 2.53

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.11-25.11)}{(35.11-25.11)-(78.30-72.22)}$ = 2.55

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.79-24.82)}{(34.79-24.82)(80.42-74.42)}$ =2.51

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.53+2.55+2.51)}{3}$ = 2.53

Table A24: Specific Gravity Result for 85%LAT + 15%BN + 16%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.81	25.13	25.42
Wt of bottle + dry soil, W_2 (g).	34.79	35.12	35.42
Wt of bottle + soil + water, W_3 (g).	84.13	82.11	78.84
Wt of bottle + water, W_4 (g).	78.14	76.08	72.84

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 85%LAT + 15%BN + 16%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(34.79-24.81)}{(34.79-24.81)-(84.13-78.14)}$ = 2.50

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(35.12-25.13)}{(35.12-25.13)-(82.11-76.08)}$ = 2.52

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(35.42-25.42)}{(35.42-25.42)(78.84-72.84)}$ = 2.5

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.50+2.52+2.50)}{3}$ = 2.51

Table A25: Specific Gravity Result for 80%LAT + 20%BN + 16%BLA

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.44	24.69	25.12
Wt of bottle + dry soil, W_2 (g).	34.43	34.68	35.12
Wt of bottle + soil + water, W_3 (g).	82.51	84.65	81.07
Wt of bottle + water, W_4 (g).	76.54	78.75	75.14

The Specific gravity of the sample is calculated as follows:

Specific Gravity for 80%LAT + 20%BN + 16%BLA

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(34.43-24.44)}{(34.43-24.44)-(84.65-78.75)}$ = 2.49

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)}$ = $\frac{(34.68-24.69)}{(34.68-24.69)-(84.65-78.75)}$ = 2.50

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)}$ = $\frac{(35.12-25.12)}{(35.12-25.12)(81.07-75.14)}$ =2.46

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3}$ = $\frac{(2.49+2.50+2.46)}{3}$ = 2.48

Table A26: Hydraulic Conductivity Test Results for 100-0-0

OMC = 15%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2.}$	127.63	20.24	20.38	20.42
H3	113.5	21.12	22.18	23.16
Total		41.36	42.56	43.58

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{60x86.6x41.36}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.5 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{60x86.6x42.56}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.26 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.58}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.07 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.27 x 10^{-8}m/s

Table A27: Hydraulic Conductivity Test Results for 95-5-0

OMC = 12%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.34	20.48	20.54
H3	113.5	21.24	22.21	23.35
Total		41.58	42.69	43.89

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x41.58}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.46 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x42.69}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.24 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x43.89}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.01 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

$K = 8.23 \times 10^{-8}$m/s

Table A28: Hydraulic Conductivity Test Results for 90-10-0

OMC = 13.6%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.42	20.56	20.64
H3	113.5	21.38	22.41	23.48
Total		41.80	42.97	44.12

$K = \frac{2.303 x a x L x}{\Delta t A}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

$A = 86.6cm^2$

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x41.80}$ x [Log10$\frac{143.5}{113.5}$]

$K1 = 8.4 \times 10^{-8}$m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.97}$ x [Log10$\frac{143.5}{113.5}$]

$K2 = 8.18 \times 10^{-8}$m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x44.12}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.97 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.18 x 10^{-8}m/s

Table A29: Hydraulic Conductivity Test Results for 85-15-0

OMC = 11.9%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.48	20.61	20.72
H3	113.5	21.62	22.68	23.61
Total		42.10	43.29	44.33

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.10}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.35 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x43.29}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.12 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.33}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.93 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.13 x 10^{-8}m/s

Table A30: Hydraulic Conductivity Test Results for 80-20-0

OMC = 15.4%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.60	20.74	20.84
H3	113.5	21.78	22.81	23.94
Total		42.38	43.55	44.78

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x42.38}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.3 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x43.55}$ x [Log10$\frac{143.5}{113.5}$]

$K2 = 8.07 \times 10^{-8}$m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x44.78}$ x [Log10$\frac{143.5}{113.5}$]

$K3 = 7.85 \times 10^{-8}$m/s

$K = \sqrt[3]{K1K2K3}$

$K = 8.07 \times 10^{-8}$m/s

Table A31: Hydraulic Conductivity Test Results for 100-0-4

OMC = 11.2%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.35	20.52	20.56
H3	113.5	21.34	22.45	23.28
Total		41.69	42.97	43.84

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x41.69}$ x [Log10$\frac{143.5}{113.5}$]

$K1 = 8.43 \times 10^{-8}$m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.97}$ x [Log10$\frac{143.5}{113.5}$]

$K2 = 8.18 \times 10^{-8}$m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.84}$ x [Log10$\frac{143.5}{113.5}$]

$K3 = 8.02 \times 10^{-8}$m/s

$K = \sqrt[3]{K1K2K3}$

$K = 8.21 \times 10^{-8}$m/s

Table A32: Hydraulic Conductivity Test Results for 95-5-4

OMC = 11.6%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.58	20.67	20.71
H3	113.5	21.79	22.72	23.51
Total		42.37	43.39	44.22

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.37}$ x [Log10$\frac{143.5}{113.5}$]

$K1 = 8.26 \times 10^{-8}$m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x43.39}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.10 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.22}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.95 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 8.10 x 10^{-8}m/s

Table A33: Hydraulic Conductivity Test Results for 90-10-4

OMC = 17.4%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.76	20.88	20.84
H3	113.5	21.85	22.92	23.75
Total		42.61	43.8	44.59

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x42.61}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.25 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x43.80}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.02 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.59}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.89 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 8.05 x 10^{-8}m/s

Table A34: Hydraulic Conductivity Test Results for 85-15-4

OMC = 13.5%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.92	20.97	20.91
H3	113.5	22.1	23.08	23.84
Total		43.02	44.05	44.75

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x43.02}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.17 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x44.05}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 7.98 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.75}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.86 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 8.0 x 10^{-8}m/s

Table A35: Hydraulic Conductivity Test Results for 80-20-4

OMC = 11.4%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	21.06	21.15	21.18
H3	113.5	22.18	23.21	23.92
Total		43.24	44.36	45.1

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x43.24}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.13 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x44.36}$ x [Log10$\frac{143.5}{113.5}$]

$K2 = 7.93 \times 10^{-8}$m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x45.10}$ x [Log10$\frac{143.5}{113.5}$]

$K3 = 7.80 \times 10^{-8}$m/s

$K = \sqrt[3]{K1K2K3}$

$K = 7.95 \times 10^{-8}$m/s

Table A36: Hydraulic Conductivity Test Results for 100-0-8

OMC = 8.2%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.38	20.45	20.42
H3	113.5	22.42	22.28	23.14
Total		42.80	42.73	43.56

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.80}$ x [Log10$\frac{143.5}{113.5}$]

$K1 = 8.22 \times 10^{-8}$m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x44.36}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.23 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x42.73}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.07 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.16 x 10^{-8}m/s

Table A37: Hydraulic Conductivity Test Results for 95-5-8

OMC = 14%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.45	20.56	20.52
H3	113.5	22.53	22.41	23.28
Total		42.98	42.97	43.8

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.98}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.18 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.97}$ x [$Log10\frac{143.5}{113.5}$]

K2 = 8.18 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.80}$ x [$Log10\frac{143.5}{113.5}$]

K3 = 8.02 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.13 x 10^{-8}m/s

Table A38: Hydraulic Conductivity Test Results for 90-10-8

OMC = 15.0%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.62	20.73	20.74
H3	113.5	22.71	22.58	23.51
Total		43.33	43.31	44.25

$K = \frac{2.303xaxLx}{\Delta tA}$ x [$Log10\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x43.33}$ x [$Log10\frac{143.5}{113.5}$]

K1 = 8.12 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x43.31}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.12 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.25}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.95 x 10^{-8}m/s

$$K = \sqrt[3]{K1K2K3}$$

K = 8.06 x 10^{-8}m/s

Table A39: Hydraulic Conductivity Test Results for 85-15-8

OMC = 7.4%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.78	20.92	20.91
H3	113.5	22.85	22.79	23.79
Total		43.63	43.71	44.70

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^{2}

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x43.63}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.06 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x43.71}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.04 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.70}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.87 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 7.99 x 10^{-8}m/s

Table A40: Hydraulic Conductivity Test Results for 80-20-8

OMC = 8.8%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.92	21.08	21.12
H3	113.5	23.05	22.94	23.93
Total		43.97	44.02	45.05

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x43.97}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.0 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x44.02}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 7.99 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x45.05}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.80 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 7.23 x 10^{-8}m/s

Table A41: Hydraulic Conductivity Test Results for 100-0-12

OMC = 19.8%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.44	20.53	20.61
H3	113.5	22.51	22.40	23.24
Total		42.95	42.93	43.85

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x42.95}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.19 x 10^{-8}m/s

For T2

$K2 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 42.93}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.19 x 10^{-8}m/s

For T3

$K3 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 43.85}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.02 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.13 x 10^{-8}m/s

Table A42: Hydraulic Conductivity Test Results for 95-5-12

OMC = 10.5%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.68	20.68	20.75
H3	113.5	22.59	22.51	23.38
Total		43.27	43.19	44.13

$K = \frac{2.303 x a x L x}{\Delta t A}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^{2}

For T1

$K1 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 43.27}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.13 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x43.19}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.14 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x44.13}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 7.97 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.08 x 10^{-8}m/s

Table A43: Hydraulic Conductivity Test Results for 90-10-12

OMC = 11.0%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.54	20.52	20.61
H3	113.5	22.41	22.38	23.30
Total		42.95	42.9	43.91

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^{2}

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.95}$ x [Log10$\frac{143.5}{113.5}$]

$K1 = 8.20 \times 10^{-8}$m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.9}$ x [Log10$\frac{143.5}{113.5}$]

$K2 = 8.14 \times 10^{-8}$m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.91}$ x [Log10$\frac{143.5}{113.5}$]

$K3 = 8.01 \times 10^{-8}$m/s

$$K = \sqrt[3]{K1K2K3}$$

$K = 8.13 \times 10^{-8}$m/s

Table A44: Hydraulic Conductivity Test Results for 85-15-12

OMC = 11.2%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.32	20.41	20.48
H3	113.5	22.28	22.30	23.12
Total		42.6	42.71	43.6

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.6}$ x $[Log10\frac{143.5}{113.5}]$

K1 = 8.25 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.71}$ x $[Log10\frac{143.5}{113.5}]$

K2 = 8.23 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.91}$ x $[Log10\frac{143.5}{113.5}]$

K3 = 8.12 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.18 x 10^{-8}m/s

Table A45: Hydraulic Conductivity Test Results for 80-20-12

OMC = 15.2%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.14	20.28	20.24
H3	113.5	22.08	22.17	23.02
Total		42.22	42.45	43.26

$K = \frac{2.303xaxLx}{\Delta tA}$ x $[Log10\frac{h1}{h2}]$

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 42.22}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.33 x 10^{-8}m/s

For T2

$K2 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 42.45}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.28 x 10^{-8}m/s

For T3

$K3 = \frac{2.303 x 0.64 x 12.17}{86.6 x 60 x 43.26}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.12 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.24 x 10^{-8}m/s

Table A46: Hydraulic Conductivity Test Results for 100-0-16

OMC = 16.6%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.42	20.51	20.68
H3	113.5	22.58	22.42	23.26
Total		43.0	42.93	43.94

$K = \frac{2.303 x a x L x}{\Delta t A}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x43.0}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.17 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x42.93}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.19 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x43.94}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.0 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.12 x 10^{-8}m/s

Table A47: Hydraulic Conductivity Test Results for 95-5-16

OMC = 15.6%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	20.28	20.43	20.52
H3	113.5	22.41	22.28	23.14
Total		42.69	42.71	43.66

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x42.69}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.24 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.71}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.23 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.66}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.05 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.17 x 10^{-8}m/s

Table A48: Hydraulic Conductivity Test Results for 90-10-16

OMC = 17.6%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	20.04	20.25	20.38
H3	113.5	22.22	22.14	23.05
Total		42.26	42.39	43.43

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x42.26}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.32 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x42.39}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.29 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x44.43}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.10 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 8.24 x 10^{-8}m/s

Table A49: Hydraulic Conductivity Test Results for 85-15-16

OMC = 18.4%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}$.	127.63	19.92	20.11	20.14
H3	113.5	22.04	22.02	22.95
Total		41.96	42.13	43.09

K = $\frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

$K1 = \frac{2.303x0.64x12.17}{86.6x60x41.96}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.38 x 10^{-8}m/s

For T2

$K2 = \frac{2.303x0.64x12.17}{86.6x60x42.13}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.35 x 10^{-8}m/s

For T3

$K3 = \frac{2.303x0.64x12.17}{86.6x60x43.09}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.16 x 10^{-8}m/s

$K = \sqrt[3]{K1K2K3}$

K = 8.30 x 10^{-8}m/s

Table A50: Hydraulic Conductivity Test Results for 80-20-16

OMC = 17.8%

	Height (cm)	T1 (min)	T2 (min)	T3 (min)
H1	143.5	0	0	0
$\sqrt{HIH2}.$	127.63	19.86	20.01	20.03
H3	113.5	21.96	21.94	22.82
Total		41.86	41.95	42.85

$K = \frac{2.303xaxLx}{\Delta tA}$ x [Log10$\frac{h1}{h2}$]

Where:

a = 0.64

L = 12.17

A = 86.6cm^2

For T1

K1 = $\frac{2.303x0.64x12.17}{86.6x60x41.86}$ x [Log10$\frac{143.5}{113.5}$]

K1 = 8.40 x 10^{-8}m/s

For T2

K2 = $\frac{2.303x0.64x12.17}{86.6x60x41.95}$ x [Log10$\frac{143.5}{113.5}$]

K2 = 8.38 x 10^{-8}m/s

For T3

K3 = $\frac{2.303x0.64x12.17}{86.6x60x42.85}$ x [Log10$\frac{143.5}{113.5}$]

K3 = 8.21 x 10^{-8}m/s

K = $\sqrt[3]{K1K2K3}$

K = 8.33 x 10^{-8}m/s

Table A51: Unconfined Compressive Strength Test Result for 100LAT + 0%BN + 0%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	4.8931182	3.2620788	17.941434
0.25	9.7739884	8.1449903	21.176975
0.375	13.015654	11.388697	22.777394
0.5	19.498984	16.249154	32.498307
0.625	24.34311	19.474488	34.080354
0.75	25.933323	19.449992	35.658319
0.875	30.757035	21.044287	45.326157
1	32.334999	24.251249	58.202999
1.125	33.908881	30.679464	62.973636
1.25	38.704014	33.866012	66.119358
1.5	45.040368	35.38886	69.169136
1.75	52.948561	38.508044	75.411587
2	56.01467	46.412155	76.820119
2.25	62.257122	51.082766	84.605832
2.5	66.874657	55.728881	87.573956
3	72.868064	60.195357	91.877124
3.5	74.068378	63.036918	94.555376
4	79.955634	64.278059	98.768725
4.5	84.217975	68.622054	99.813896
5	91.534177	72.917056	105.49702
5.5	97.225463	80.249589	106.48503
6	104.38652	85.965372	98.246139
6.5	108.41207	90.0889	85.508109
7	101.75726	92.644672	74.419491
7.5	77.040585	96.678381	
8	66.107109	69.111978	
		62.759294	

Table A52: Unconfined Compressive Strength Test Result for 95LAT + 5%BN + 0%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	17.94143	17.94143	19.57247
0.25	26.06397	21.17697	19.54798
0.375	29.28522	29.28522	22.77739
0.5	34.12322	35.74814	32.49831
0.625	37.3261	38.94898	35.70323
0.75	42.14165	45.38331	37.27915
0.875	43.70737	56.6577	42.08857
1	48.5025	66.28675	48.5025
1.125	54.90009	69.43247	50.05597
1.25	58.05602	74.18269	56.44335
1.5	67.56055	83.6464	62.7348
1.75	68.99358	85.0386	68.99358
2	75.2197	88.02305	80.02096
2.25	84.60583	90.99118	87.7985
2.5	89.16621	93.94297	97.12748
3	91.87712	98.21348	106.1339
3.5	100.8591	99.28315	122.922
4	108.1753	105.0398	123.8528
4.5	116.9694	109.1714	112.2906
5	119.4599	110.1513	91.53418
5.5	97.22546	115.7446	63.27371
6	89.03556	115.1322	
6.5	77.87346	105.3582	
7		94.16344	
7.5			
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8.5			
9			
9.5			

Table A53: Unconfined Compressive Strength Test Result for 90LAT + 10%BN + 0%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	3.262079	24.46559	19.57247
0.25	9.773988	37.46696	21.17697
0.375	21.15044	42.30087	30.91218
0.5	21.1239	47.12255	38.99797
0.625	35.70323	51.93197	45.44047
0.75	47.00415	53.48748	53.48748
0.875	53.42011	58.27649	63.13286
1	59.81975	63.05325	74.3705
1.125	67.81776	67.81776	79.12072
1.25	72.57003	69.34469	91.92203
1.5	73.99489	77.21206	101.3408
1.75	78.62059	83.4341	104.2926
2	86.42263	104.0272	108.8285
2.25	97.37652	111.7436	113.3399
2.5	106.681	121.0113	116.2345
3	117.2225	129.8952	118.8066
3.5	122.922	140.2571	124.4979
4	130.1239	145.8015	126.9884
4.5	132.5653	141.9229	129.4461
5	136.5256	139.6284	131.8713
5.5	140.4368	134.2637	134.2637
6	130.4832	130.4832	141.2288
6.5	90.0889		131.316
7	57.71307		104.7948
7.5			78.55118
8			
8.5			
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Table A54: Unconfined Compressive Strength Test Result for 85%LAT + 15%BN + 0%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	17.94143	21.20351	3.262079
0.25	35.83796	26.06397	11.40299
0.375	40.67392	29.28522	24.40435
0.5	50.37238	35.74814	25.99865
0.625	58.42346	38.94898	29.21173
0.75	68.07497	45.38331	37.27915
0.875	77.70198	55.0389	55.0389
1	85.68775	88.92125	64.67
1.125	108.1855	98.49723	67.81776
1.25	111.274	122.5627	87.08403
1.5	120.6438	127.0782	90.08074
1.75	129.9647	134.7782	96.27011
2	132.8348	147.2386	100.8264
2.25	142.0739	148.4593	102.1655
2.5	151.2641	152.8564	105.0887
3	159.9929	156.8247	128.3112
3.5	162.3201	159.1682	156.0164
4	164.6145	159.9113	169.3178
4.5	179.3531	162.1976	180.9127
5	181.5169	162.8998	179.9655
5.5	160.4992	168.2155	166.6722
6	130.4832	175.0009	
6.5	94.66969	172.5431	
7		160.9891	
7.5		138.9752	
8			
8.5			
			147.3692

Table A55: Unconfined Compressive Strength Test Result for 80%LAT + 20%BN + 0%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	19.57247	8.155197	19.57247
0.25	24.43497	13.03198	27.69297
0.375	29.28522	22.77739	37.42
0.5	34.12322	34.12322	47.12255
0.625	45.44047	37.3261	50.30909
0.75	53.48748	40.52082	53.48748
0.875	69.60803	46.94495	56.6577
1	71.137	64.67	66.28675
1.125	77.50601	71.04718	67.81776
1.25	80.63336	75.79536	74.18269
1.5	85.25498	82.03781	88.47215
1.75	94.66561	86.6431	91.45661
2	102.4268	89.62347	100.8264
2.25	105.3582	95.78019	108.5509
2.5	108.2733	100.312	113.05
3	152.0725	104.5498	133.0634
3.5	170.1997	111.8905	152.8645
4	175.5888	133.2594	159.9113
4.5	184.0319	137.2441	182.4723
5	187.7226	161.3484	200.134
5.5	191.3644	186.7346	206.797
6	191.887	202.6327	207.2379
6.5	172.5431	196.974	190.8663
7	168.5829	192.8832	183.7706
7.5	164.6554	175.2296	157.1024
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8.5			
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9.5			
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Table A56: Unconfined Compressive Strength Test Result for 100%LAT + 0%BN + 4%BLA

	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	8.1551971	6.5241577	3.2620788
0.25	17.918979	13.031985	11.402986
0.375	21.150437	16.269567	21.150437
0.5	27.623561	19.498984	22.748815
0.625	32.45748	25.965984	27.588858
0.75	37.279151	34.037486	30.795821
0.875	40.469783	37.2322	32.375826
1	40.418749	42.035499	35.568499
1.125	46.82655	50.055967	37.138298
1.25	51.605352	59.668688	37.091347
1.5	57.909044	62.734798	40.214614
1.75	62.575572	65.784576	46.530554
2	65.617185	70.418443	48.012575
2.25	70.238804	71.83514	51.082766
2.5	73.243672	74.835926	54.136627
3	76.03624	83.956682	58.611269
3.5	78.796147	86.675762	64.612841
4	81.523392	87.794422	72.116847
4.5	84.217975	88.896751	82.658383
5	86.879897	96.188457	89.98275
5.5	94.138941	108.02829	94.138941
6	96.711043	112.062	105.92162
6.5	97.723553	100.77741	111.46593
7	101.75726	94.163437	107.83232
7.5	90.635983	77.040585	93.657182
8	85.638755		84.136321
8.5	64.253563		
9			
9.5			
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Table A57: Unconfined Compressive Strength Test Result for 95LAT + 5%BN + 4%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	17.94143	11.41728	6.524158
0.25	34.20896	21.17697	13.03198
0.375	39.04696	24.40435	24.40435
0.5	43.87271	29.24848	34.12322
0.625	48.68622	34.08035	37.3261
0.75	51.86665	37.27915	40.52082
0.875	59.89528	45.32616	45.32616
1	61.4365	51.736	48.5025
1.125	66.20305	54.90009	50.05597
1.25	72.57003	59.66869	53.21802
1.5	73.99489	65.95197	54.69187
1.75	77.01609	68.99358	57.76207
2	84.82221	75.2197	62.41635
2.25	92.58751	84.60583	65.44979
2.5	97.12748	92.35072	68.46691
3	102.9657	102.9657	72.86806
3.5	107.1628	115.0424	85.09984
4	112.8785	119.1496	95.63321
4.5	123.2078	121.6482	115.4098
5	121.0113	125.6656	121.0113
5.5	103.3985	114.2013	128.0907
6	81.36008	90.57066	128.9481
6.5		79.40039	125.2083
7			124.5387
7.5			117.8268
8			
8.5			
9			

Table A58: Unconfined Compressive Strength Test Result for 90%LAT + 10%BN + 4%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	16.31039	21.20351	8.155197
0.25	26.06397	30.95096	17.91898
0.375	35.79305	32.53913	34.16609
0.5	37.37305	34.12322	37.37305
0.625	40.57185	38.94898	45.44047
0.75	51.86665	45.38331	50.24581
0.875	61.51407	53.42011	58.27649
1	72.75375	53.35275	61.4365
1.125	77.50601	56.5148	64.58834
1.25	79.0207	61.28136	70.95736
1.5	85.25498	67.56055	75.60347
1.75	86.6431	73.80709	81.82959
2	91.22389	80.02096	89.62347
2.25	102.1655	83.0095	92.58751
2.5	106.681	92.35072	101.9042
3	115.6384	104.5498	109.3021
3.5	122.922	124.4979	111.8905
4	128.5561	133.2594	120.7173
4.5	132.5653	141.9229	131.0057
5	141.1798	152.0398	138.077
5.5	152.7829	162.0424	143.5233
6	147.3692	165.7904	138.1586
6.5	132.843	163.3816	132.843
7	110.8699	159.4703	110.8699
7.5		151.06	
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8.5			
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9.5			
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Table A59: Unconfined Compressive Strength Test Result for 85%LAT + 15%BN + 4%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	21.20351	17.94143	17.94143
0.25	26.06397	19.54798	37.46696
0.375	32.53913	35.79305	42.30087
0.5	37.37305	40.62288	47.12255
0.625	38.94898	48.68622	50.30909
0.75	42.14165	53.48748	55.10831
0.875	51.80132	58.27649	59.89528
1	59.81975	63.05325	67.9035
1.125	72.66189	64.58834	72.66189
1.25	90.30937	72.57003	82.24603
1.5	99.73224	82.03781	85.25498
1.75	117.1286	101.0836	93.06111
2	129.634	108.8285	99.22599
2.25	137.2849	119.7252	121.3216
2.5	146.4873	130.5648	132.1571
3	152.0725	169.4975	156.8247
3.5	163.896	182.8071	168.6238
4	184.9954	192.8342	184.9954
4.5	194.949	199.6278	196.5086
5	200.134	201.6855	187.7226
5.5	185.1914	203.7105	182.1048
6	175.0009	194.9572	159.65
6.5		186.2855	
7		174.658	
7.5			
8			
8.5			
9			
9.5			
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Table A60: Unconfined Compressive Strength Test Result for 80%LAT + 20%BN + 4%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	9.786236	22.83455	19.57247
0.25	17.91898	35.83796	27.69297
0.375	22.77739	43.92783	34.16609
0.5	27.62356	56.87204	38.99797
0.625	37.3261	76.27508	45.44047
0.75	42.14165	85.90413	48.62498
0.875	53.42011	95.50869	50.18253
1	59.81975	100.2385	53.35275
1.125	66.20305	119.4884	56.5148
1.25	74.18269	125.788	62.89402
1.5	82.03781	131.9039	65.95197
1.75	94.66561	154.0322	75.41159
2	105.6277	168.044	83.2218
2.25	118.1289	174.0007	87.7985
2.5	135.3416	186.2937	95.53522
3	153.6566	196.427	106.1339
3.5	167.0478	201.7181	118.1942
4	181.8599	208.5118	136.3949
4.5	191.8298	213.6641	149.7208
5	198.5826	214.0969	169.1055
5.5	208.3403	214.5133	177.4751
6	213.3783	214.9134	196.4923
6.5	207.6626	210.7164	207.6626
7	188.3269	208.0708	192.8832
7.5	161.6342	200.9098	158.613
8			147.2386
8.5			
9			
9.5			
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Table A61: Unconfined Compressive Strength Test Result for 100LAT + 0%BN + 8%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	8.155197	6.524158	17.94143
0.25	14.66098	14.66098	26.06397
0.375	26.03131	19.52348	37.42
0.5	35.74814	19.49898	42.2478
0.625	40.57185	21.09736	45.44047
0.75	47.00415	24.31249	53.48748
0.875	53.42011	29.13824	56.6577
1	59.81975	33.95175	59.81975
1.125	64.58834	40.36772	62.97364
1.25	66.11936	45.15468	66.11936
1.5	69.16914	49.86612	69.16914
1.75	81.82959	54.55306	73.80709
2	91.22389	60.81593	80.02096
2.25	102.1655	63.85346	83.0095
2.5	108.2733	68.46691	90.75846
3	114.0544	74.45215	96.62939
3.5	118.1942	80.37207	99.28315
4	122.2851	90.92994	106.6075
4.5	126.327	95.13512	115.4098
5	113.2542	99.29131	116.357
5.5	91.05242	104.9418	121.9176
6		113.5971	108.9918
6.5		103.8313	99.25048
7		80.49455	77.45702
7.5		67.97699	
8			
8.5			
9			
9.5			

Table A62: Unconfined Compressive Strength Test Result for 95LAT + 5%BN + 8%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	6.524158	16.31039	21.20351
0.25	13.03198	19.54798	29.32197
0.375	24.40435	26.03131	34.16609
0.5	27.62356	30.87339	38.99797
0.625	32.45748	35.70323	50.30909
0.75	35.65832	38.89998	53.48748
0.875	37.2322	43.70737	56.6577
1	40.41875	48.5025	67.9035
1.125	50.05597	50.05597	72.66189
1.25	54.83069	58.05602	75.79536
1.5	67.56055	59.51763	80.42923
1.75	72.20258	65.78458	85.0386
2	76.82012	75.2197	92.82431
2.25	81.41316	83.0095	105.3582
2.5	93.94297	89.16621	114.6423
3	107.718	101.3817	123.5589
3.5	118.1942	107.1628	135.5294
4	123.8528	114.4463	142.6659
4.5	127.8866	126.327	149.7208
5	136.5256	138.077	139.6284
5.5	138.8935	146.6098	120.3744
6	141.2288	148.9043	101.3163
6.5	146.5853	148.1123	
7	144.2827	145.8015	
7.5	138.9752	143.507	
8	133.7167		
8.5			
9			
9.5			
10			

Table A63: Unconfined Compressive Strength Test Result for 90LAT + 10%BN + 8%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	22.83455	17.94143	19.57247
0.25	29.32197	26.06397	24.43497
0.375	34.16609	35.79305	35.79305
0.5	37.37305	38.99797	40.62288
0.625	43.8176	43.8176	47.06335
0.75	50.24581	53.48748	50.24581
0.875	53.42011	56.6577	55.0389
1	58.203	61.4365	67.9035
1.125	67.81776	71.04718	75.8913
1.25	75.79536	79.0207	82.24603
1.5	85.25498	83.6464	88.47215
1.75	93.06111	101.0836	96.27011
2	102.4268	115.2302	99.22599
2.25	105.3582	126.1106	103.7619
2.5	117.8268	130.5648	116.2345
3	125.143	144.152	129.8952
3.5	133.9534	154.4404	148.1368
4	139.5304	163.0468	155.208
4.5	141.9229	166.8764	162.1976
5	148.937	181.5169	164.4512
5.5	155.8694	186.7346	171.302
6	159.65	182.6764	178.0711
6.5	163.3816	177.1239	166.4354
7	153.3953	171.6205	141.2452
7.5	134.4434		113.295
8	109.6777		
8.5			
9			
9.5			
10			

Table A64: Unconfined Compressive Strength Test Result for 85%LAT + 15%BN + 8%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	34.25183	29.35871	27.72767
0.25	45.61195	42.35395	40.72495
0.375	53.68957	55.31653	45.55479
0.5	60.12187	61.74678	55.24712
0.625	68.16071	71.40646	60.04634
0.75	81.04163	79.4208	63.21247
0.875	89.03352	87.41473	67.98923
1	93.7715	97.005	74.3705
1.125	101.7266	100.1119	85.57956
1.25	106.436	109.6614	91.92203
1.5	114.2095	117.4267	98.12366
1.75	118.7331	125.1511	109.1061
2	124.8327	137.636	120.0314
2.25	129.3033	145.2666	130.8996
2.5	135.3416	160.8176	138.5261
3	139.3998	180.5861	150.4884
3.5	148.1368	193.8385	162.3201
4	167.7501	213.215	178.7244
4.5	193.3894	219.9025	201.1874
5	212.5455	221.854	207.8912
5.5	216.0566	212.9701	214.5133
6	211.8432	207.2379	196.4923
6.5	189.3394	189.3394	161.8546
7	160.9891		145.8015
7.5			
8			
8.5			
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9.5			
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Table A65: Unconfined Compressive Strength Test Result for 80%LAT + 20%BN + 8%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	34.25183	30.98975	26.09663
0.25	47.24094	43.98295	39.09595
0.375	52.06261	56.94348	55.31653
0.5	58.49695	69.87136	60.12187
0.625	69.78358	97.37244	66.53783
0.75	77.79997	106.975	69.6958
0.875	80.93957	119.7906	82.55836
1	92.15475	127.7232	87.3045
1.125	100.1119	137.2502	93.6531
1.25	106.436	141.9147	98.3727
1.5	110.9923	146.3812	101.3408
1.75	117.1286	152.4277	129.9647
2	136.0356	158.4415	142.4373
2.25	158.0373	166.019	150.0556
2.5	162.4099	171.9634	156.0409
3	194.8429	193.2588	167.9134
3.5	203.2941	214.3255	181.2311
4	213.215	233.5959	194.4019
4.5	224.5813	235.4984	215.2237
5	242.0226	232.714	218.7512
5.5	229.9459	211.4268	223.7729
6	205.7029	188.8168	225.6591
6.5	149.6392		221.4049
7			211.1084
7.5			190.3356
8			
8.5			
9			
9.5			
10			

Table A66: Unconfined Compressive Strength Test Result for 100%LAT + 0%BN + 12%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	8.155197	1.631039	4.893118
0.25	11.40299	4.886994	8.14499
0.375	17.89652	4.88087	16.26957
0.5	21.1239	8.124577	21.1239
0.625	27.58886	12.98299	22.72024
0.75	35.65832	19.44999	25.93332
0.875	38.85099	24.28187	30.75703
1	42.0355	29.1015	35.5685
1.125	46.82655	30.67946	41.98242
1.25	49.99268	37.09135	45.15468
1.5	53.08329	40.21461	49.86612
1.75	56.15756	46.53055	54.55306
2	67.2176	48.01257	57.61509
2.25	71.83514	52.6791	59.06445
2.5	81.20494	57.32113	62.0979
3	85.54077	64.94762	68.1158
3.5	91.40353	72.49246	78.79615
4	101.9042	81.52339	84.65891
4.5	113.8502	84.21798	90.45634
5	117.9084	91.53418	103.9456
5.5	126.5474	100.312	109.5716
6	135.0884	110.5269	118.2024
6.5	116.0467	117.5736	122.1544
7	97.20097	127.5763	123.02
7.5	77.04059	123.8692	123.8692
8		111.1801	118.6923
8.5		100.116	107.5874
9			
9.5			
10			

Table A67: Unconfined Compressive Strength Test Result for 95%LAT + 5%BN + 12%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	14.67935	19.57247	17.94143
0.25	27.69297	21.17697	24.43497
0.375	43.92783	29.28522	34.16609
0.5	50.37238	34.12322	37.37305
0.625	61.66921	40.57185	42.19472
0.75	66.45414	47.00415	50.24581
0.875	69.60803	55.0389	59.89528
1	75.98725	59.81975	69.52025
1.125	85.57956	67.81776	74.2766
1.25	90.30937	80.63336	79.0207
1.5	94.90649	82.03781	85.25498
1.75	102.6881	89.8521	102.6881
2	107.2281	99.22599	113.6298
2.25	114.9362	103.7619	118.1289
2.5	119.419	106.681	122.6035
3	125.143	114.0544	131.4793
3.5	137.1053	126.0738	138.6812
4	142.6659	128.5561	142.6659
4.5	145.0421	131.0057	148.1613
5	150.4884	138.077	148.937
5.5	152.7829	145.0666	149.6963
6	145.8341	148.9043	150.4394
6.5	123.6814	134.3699	146.5853
7	112.3886	113.9074	135.1701
7.5		98.18898	122.3586
8			
8.5			
9			
9.5			
10			

Table A68: Unconfined Compressive Strength Test Result for 90%LAT + 10%BN + 12%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	19.57247	17.94143	21.20351
0.25	19.54798	27.69297	34.20896
0.375	22.77739	37.42	39.04696
0.5	25.99865	47.12255	51.99729
0.625	35.70323	50.30909	56.80059
0.75	42.14165	56.72914	64.83331
0.875	53.42011	61.51407	69.60803
1	74.3705	66.28675	77.604
1.125	80.73543	83.96485	92.03839
1.25	87.08403	103.2107	98.3727
1.5	107.7752	114.2095	106.1666
1.75	113.9196	123.5466	110.7106
2	121.6319	137.636	116.8306
2.25	134.0923	145.2666	122.9179
2.5	140.1183	157.6331	128.9726
3	153.6566	166.3293	145.7361
3.5	162.3201	170.1997	167.0478
4	174.0211	180.2921	181.8599
4.5	179.3531	185.5915	162.1976
5	183.0684	187.7226	148.937
5.5	185.1914	182.1048	129.634
6	181.1413	164.2553	
6.5	166.4354	151.1661	
7	153.3953		
7.5			
8			
8.5			
9			
9.5			
10			

Table A69: Unconfined Compressive Strength Test Result for 85%LAT + 15%BN + 12%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	24.46559	19.57247	14.67935
0.25	42.35395	35.83796	37.46696
0.375	47.18174	55.31653	45.55479
0.5	51.99729	58.49695	61.74678
0.625	53.55484	64.91496	71.40646
0.75	76.17914	74.5583	79.4208
0.875	79.32077	85.79594	87.41473
1	82.45425	98.62175	97.005
1.125	93.6531	100.1119	101.7266
1.25	99.98537	104.8234	108.0487
1.5	110.9923	114.2095	114.2095
1.75	117.1286	120.3376	117.1286
2	129.634	123.2323	126.4331
2.25	137.2849	130.8996	134.0923
2.5	154.4486	136.9338	148.0796
3	159.9929	140.9839	169.4975
3.5	178.0793	146.5608	174.9274
4	186.5631	166.1823	195.9697
4.5	193.3894	182.4723	216.7833
5	206.3398	189.2741	218.7512
5.5	226.8594	194.4509	216.0566
6	242.5452	201.0976	210.3081
6.5	201.5548	206.1356	200.0279
7	183.7706	203.5145	
7.5	178.2508	194.8674	
8		186.3019	
8.5			
9			
9.5			
10			

Table A70: Unconfined Compressive Strength Test Result for 80%LAT + 20%BN + 12%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	24.46559	34.25183	27.72767
0.25	29.32197	42.35395	50.49894
0.375	35.79305	52.06261	63.45131
0.5	56.87204	56.87204	74.74611
0.625	66.53783	61.66921	84.38945
0.75	76.17914	71.31664	95.62913
0.875	85.79594	89.03352	105.2214
1	100.2385	95.38825	114.7892
1.125	111.4149	100.1119	125.9473
1.25	124.1754	104.8234	135.464
1.5	136.7297	117.4267	149.5984
1.75	146.0097	121.9421	155.6367
2	171.2448	129.634	161.6423
2.25	181.9824	132.4959	172.4043
2.5	200.624	152.8564	184.7014
3	210.6837	159.9929	196.427
3.5	230.0847	178.0793	201.7181
4	236.7314	195.9697	206.944
4.5	244.856	198.0682	216.7833
5	245.1254	218.7512	223.4054
5.5	248.4651	229.9459	226.8594
6	248.6855	239.475	234.8697
6.5	250.4166	233.6204	238.2012
7	241.4837	220.2209	224.7772
7.5	228.1006	197.8886	199.3992
8	213.3457		169.7751
8.5			
9			
9.5			
10			

Table A71: Unconfined Compressive Strength Test Result for 100%LAT + 0%BN + 16%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	17.94143	13.04832	14.67935
0.25	19.54798	27.69297	24.43497
0.375	24.40435	35.79305	29.28522
0.5	37.37305	40.62288	34.12322
0.625	45.44047	43.8176	38.94898
0.75	58.34998	47.00415	40.52082
0.875	64.75165	51.80132	45.32616
1	67.9035	53.35275	50.11925
1.125	75.8913	56.5148	54.90009
1.25	87.08403	62.89402	59.66869
1.5	91.68932	65.95197	65.95197
1.75	97.87461	70.59808	68.99358
2	104.0272	81.62138	75.2197
2.25	110.1472	84.60583	83.0095
2.5	114.6423	93.94297	89.16621
3	117.2225	104.5498	95.0453
3.5	121.3461	111.8905	99.28315
4	122.2851	117.5818	106.6075
4.5	131.0057	121.6482	118.529
5	134.9741	124.1141	122.5627
5.5	109.5716	128.0907	104.9418
6	98.24614	128.9481	93.64085
6.5	79.40039	132.843	88.56197
7		107.8323	
7.5		81.57238	
8		63.10224	
8.5			
9			
9.5			
10			

Table A72: Unconfined Compressive Strength Test Result for 95%LAT + 5%BN + 16%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	26.09663	30.98975	19.57247
0.25	43.98295	37.46696	34.20896
0.375	50.43566	47.18174	39.04696
0.5	55.24712	50.37238	47.12255
0.625	61.66921	53.55484	51.93197
0.75	69.6958	58.34998	56.72914
0.875	76.08319	61.51407	66.37044
1	80.8375	67.9035	74.3705
1.125	83.96485	72.66189	80.73543
1.25	90.30937	77.40803	85.47136
1.5	98.12366	82.03781	91.68932
1.75	105.8971	89.8521	102.6881
2	108.8285	100.8264	107.2281
2.25	113.3399	108.5509	114.9362
2.5	116.2345	113.05	125.788
3	118.8066	121.9748	136.2316
3.5	129.2257	129.2257	143.409
4	137.9627	136.3949	155.208
4.5	151.2804	145.0421	157.5188
5	152.0398	156.6941	161.3484
5.5	157.4127	163.5857	168.2155
6	159.65	165.7904	168.8606
6.5	151.1661	166.4354	171.0162
7	138.2076	160.9891	165.5454
7.5	126.8904	154.0812	152.5706
8		145.7361	138.224
8.5			
9			
9.5			
10			

Table A73: Unconfined Compressive Strength Test Result for 90%LAT + 10%BN + 16%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	27.72767	34.25183	21.20351
0.25	34.20896	42.35395	29.32197
0.375	40.67392	52.06261	35.79305
0.5	53.62221	61.74678	45.49763
0.625	56.80059	73.02933	50.30909
0.75	61.59164	79.4208	55.10831
0.875	67.98923	84.17715	66.37044
1	71.137	92.15475	66.28675
1.125	75.8913	98.49723	90.42368
1.25	85.47136	108.0487	99.98537
1.5	93.2979	115.8181	104.558
1.75	99.47911	126.7556	113.9196
2	105.6277	134.4352	123.2323
2.25	113.3399	140.4776	127.7069
2.5	125.788	146.4873	132.1571
3	134.6475	164.7452	144.152
3.5	137.1053	185.9589	154.4404
4	145.8015	199.1052	158.3435
4.5	151.2804	205.8662	166.8764
5	159.797	210.994	178.4141
5.5	171.302	202.1672	188.2779
6	182.6764	191.887	193.4221
6.5	193.9202	172.5431	196.974
7	186.8081		183.7706
7.5	178.2508		176.7402
8	156.2532		154.7507
8.5			
9			
9.5			
10			

Table A74: Unconfined Compressive Strength Test Result for 85%LAT + 15%BN + 16%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	29.35871	21.20351	30.98975
0.25	34.20896	27.69297	37.46696
0.375	43.92783	40.67392	55.31653
0.5	68.24645	45.49763	60.12187
0.625	77.89795	53.55484	71.40646
0.75	87.52496	66.45414	74.5583
0.875	101.9839	76.08319	79.32077
1	114.7892	84.071	85.68775
1.125	122.7179	90.42368	93.6531
1.25	137.0767	98.3727	98.3727
1.5	159.2499	119.0353	106.1666
1.75	168.4727	129.9647	115.5241
2	185.6486	152.0398	140.8369
2.25	193.1567	162.8263	148.4593
2.5	211.7697	183.1092	165.5944
3	236.0292	196.427	175.8338
3.5	247.4199	225.357	182.8071
4	253.9767	230.4604	189.6987
4.5	257.3327	235.4984	193.3894
5	237.3683	238.9197	197.0312
5.5	222.2296	234.5757	202.1672
6	201.0976	219.5187	210.3081
6.5		192.3932	186.2855
7			174.658
7.5			160.1236
8			
8.5			
9			
9.5			
10			

Table A75: Unconfined Compressive Strength Test Result for 80%LAT + 20%BN + 16%BLA

%Strain	Axial Stress Trial1	Axial Stress Trial2	Axial Stress Trial3
0	0	0	0
0.125	17.94143	27.72767	24.46559
0.25	29.32197	35.83796	42.35395
0.375	34.16609	39.04696	50.43566
0.5	38.99797	45.49763	53.62221
0.625	47.06335	53.55484	68.16071
0.75	51.86665	59.97081	82.66247
0.875	58.27649	66.37044	89.03352
1	69.52025	88.92125	92.15475
1.125	77.50601	101.7266	106.5708
1.25	88.6967	130.626	111.274
1.5	115.8181	143.164	131.9039
1.75	123.5466	155.6367	139.5917
2	129.634	163.2428	145.6381
2.25	140.4776	183.5787	153.2483
2.5	148.0796	191.0704	167.1866
3	172.6656	198.011	177.4179
3.5	179.6552	219.0533	185.9589
4	189.6987	224.1893	194.4019
4.5	212.1045	244.856	204.3066
5	214.0969	249.7797	226.5083
5.5	220.6864	256.1814	242.292
6	227.1942	253.2908	248.6855
6.5	230.5665	248.8897	236.6742
7	221.7397	244.5212	226.296
7.5	209.9734		212.9946
8	198.3213		
8.5			
9			
9.5			
10			

Compaction Test

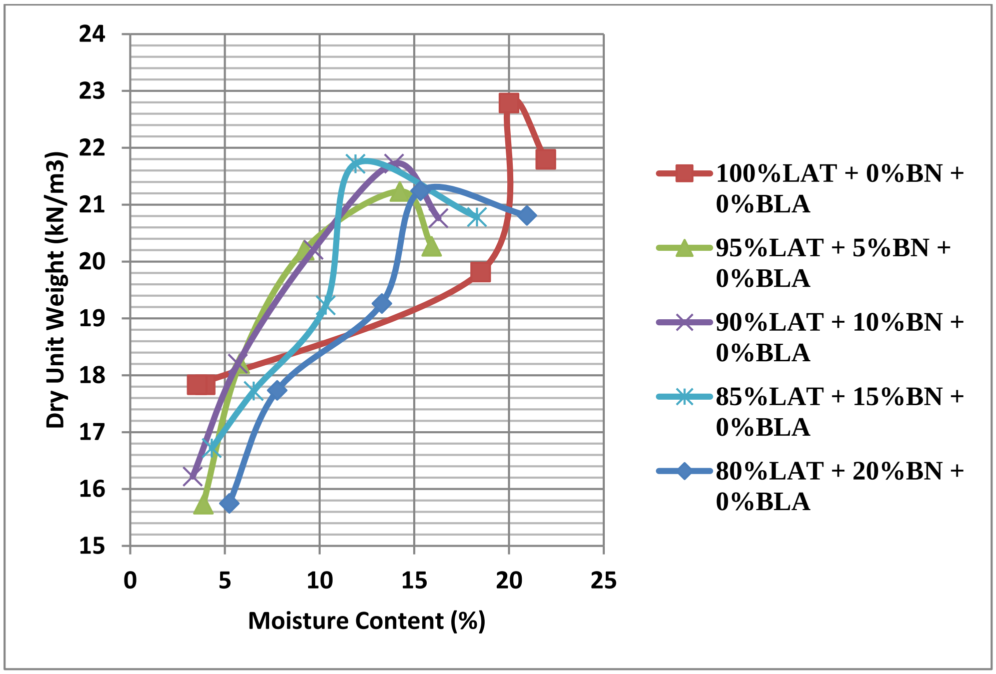

Figure A1: Compaction curve for Lateritic Soil Bentonite Mixture at 0% BLA

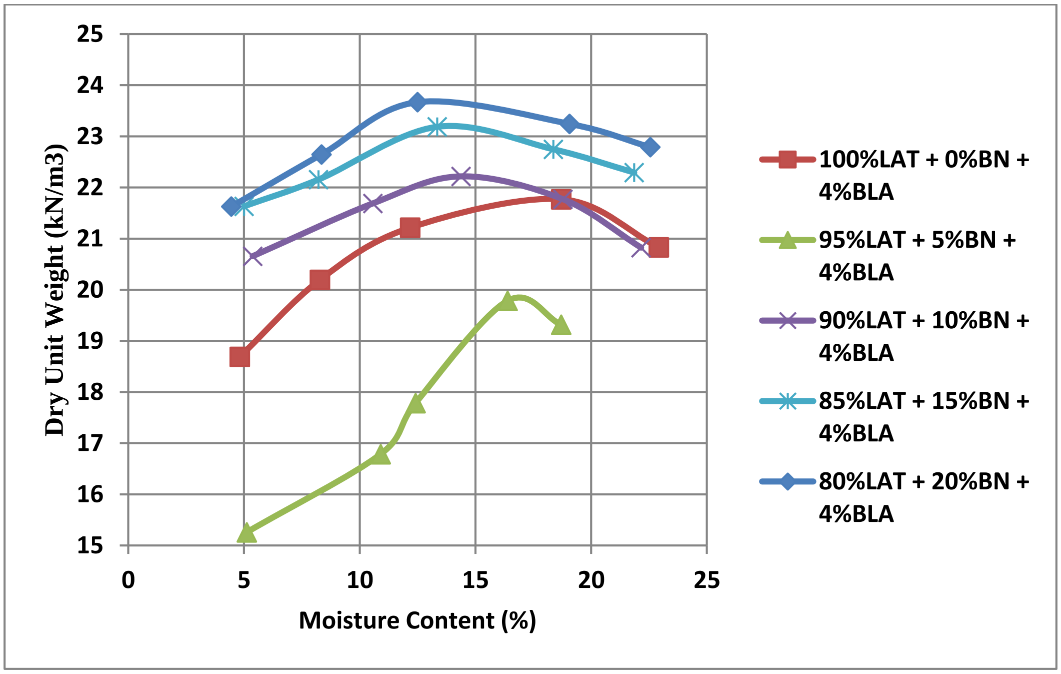

Figure A2: Compaction curve for Lateritic Soil Bentonite Mixture at 4% BLA

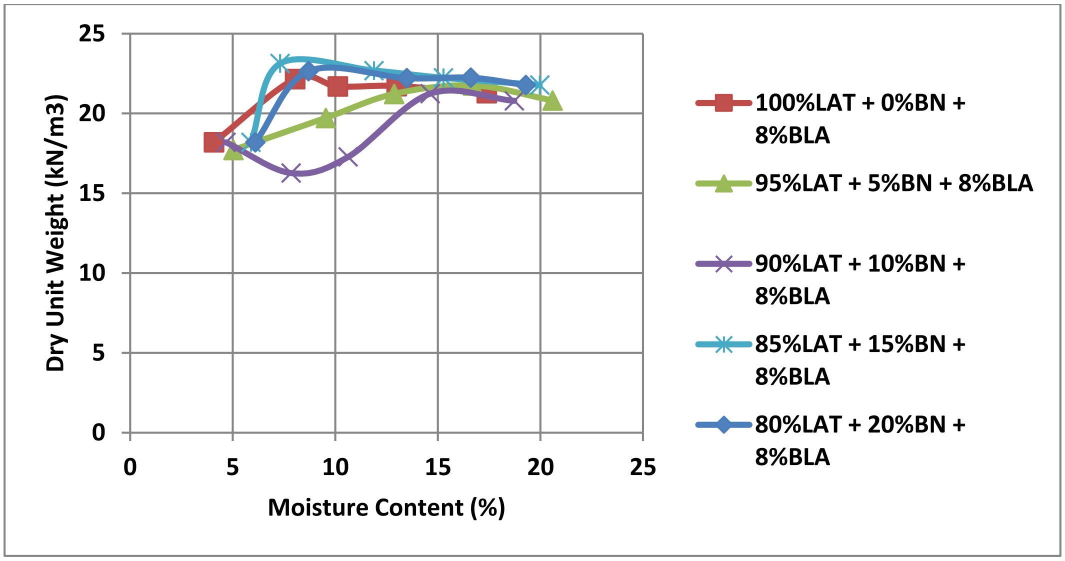

Figure A3: Compaction curve for Lateritic Soil Bentonite Mixture at 8% BLA

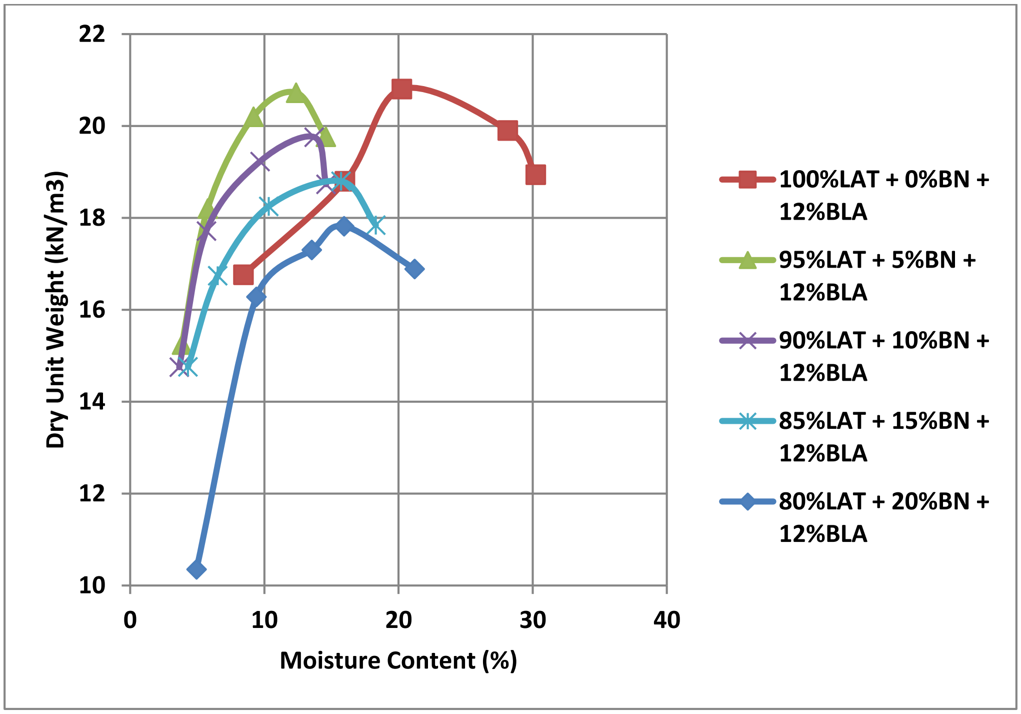

Figure A4: Compaction curve for Lateritic Soil Bentonite Mixture at 12% BLA

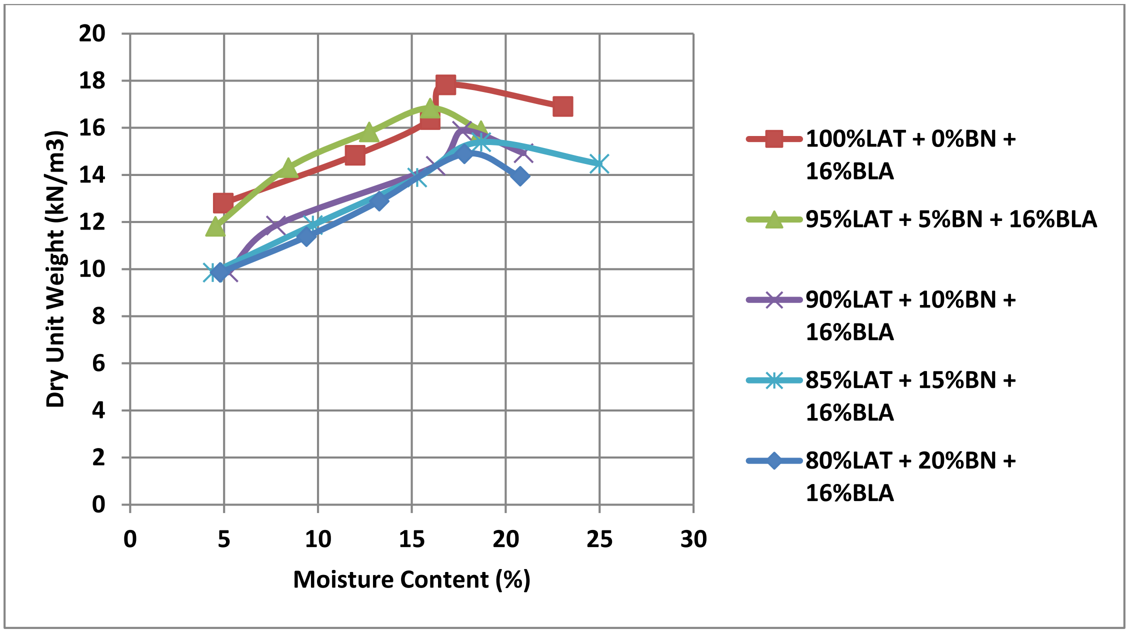

Figure A5: Compaction curve for Lateritic Soil Bentonite Mixture at 16% BLA

Specific Gravity Test

Table A76: Specific Gravity Results for Lateritic Soil Stabilized with Bentonite and Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	2.64	2.66	2.60	2.55	2.49
4	2.67	2.71	2.72	2.76	2.67
8	2.67	2.71	2.72	2.76	2.67
12	2.58	2.55	2.52	2.48	2.50
16	2.58	2.56	2.53	2.51	2.48

Sieve Analysis Test

Table A77: Sieve Analysis Test Results for the Lateritic Soil

Sieve Sizes (mm)	Mass Retained (g)	% Mass Retained	Cum % Retained	Cum % Finer
2	21.3	4.26	4.26	95.74
0.85	36.7	7.34	11.6	88.4
0.6	39.8	7.96	19.56	80.44
0.425	46.3	9.26	28.82	71.18
0.3	43.2	8.64	37.46	62.54
0.15	40	8	45.46	54.54
0.075	0	0	45.46	54.54
Tray	272.7	54.54		
Total	500			

Table A78: Liquid Limit Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	50.48	55.28	57.84	62.28	68.04
4	63.08	58.92	51.72	50.42	48.28
8	43.42	42.88	34.12	43.44	46.62
12	35.61	38.23	41.4	40.15	43.42
16	32.19	33.31	35.05	36.41	46.23

Table A79: Plastic Limit Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	19.9	25.1	26.53	30.7	32.69
4	25.5	23	20.5	18.96	16.66
8	15.12	13.05	12.41	19.18	22.49
12	16.67	13.64	20.72	24.01	25.95
16	21.17	16.98	22.73	24.12	26.61

Table A80: Plasticity Index Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	30.58	30.18	31.31	31.58	35.35
4	37.58	35.92	31.22	31.46	31.62
8	28.3	29.83	21.71	24.26	24.13
12	18.94	24.59	20.68	16.12	17.47
16	11.02	16.33	12.32	12.29	19.62

Compaction Test

Table A81: Maximum Dry Unit Weight for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	18.25	18.3	18.95	19.29	18.3
4	18.82	16.86	18.32	18.72	21.01
8	20.48	18.72	18.50	21.5	20.77
12	17.15	18.6	17.5	16.7	15.4
16	15.1	14.4	13.4	12.8	12.5

Table A82: Optimum Moisture Content Results for Laterite- Bentonite Mixture treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0	5	10	15	20
0	15.0	12.0	13.6	11.9	15.4
4	11.2	16.6	17.4	13.5	11.4

8	8.2	14	15	7.4	8.8
12	19.8	10.5	11.0	11.2	15.2
16	16.6	15.6	17.6	18.4	17.8

Table A83: Hydraulic Conductivity Test Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

MIX PROPORTIONS	HYDRAULIC CONDUCTIVITY (m/s)
100%LAT+0%BN+0%BLA	8.27 x 10^{-8}
100%LAT+0%BN+4%BLA	8.21 x 10^{-8}
100%LAT+0%BN+8%BLA	8.16 x 10^{-8}
100%LAT+0%BN+12%BLA	8.13 x 10^{-8}
100%LAT+0%BN+16%BLA	8.12 x 10^{-8}
95%LAT+5%BN+0%BLA	8.23 x 10^{-8}
95%LAT+5%BN+4%BLA	8.1 x 10^{-8}
95%LAT+5%BN+8%BLA	8.13 x 10^{-8}
95%LAT+5%BN+12%BLA	8.08 x 10^{-8}
95%LAT+5%BN+16%BLA	8.17 x 10^{-8}
90%LAT+10%BN+0%BLA	8.18 x 10^{-8}
90%LAT+10%BN+4%BLA	8.05 x 10^{-8}
90%LAT+10%BN+8%BLA	8.06 x 10^{-8}
90%LAT+10%BN+12%BLA	8.13 x 10^{-8}
90%LAT+10%BN+16%BLA	8.24 x 10^{-8}
85%LAT+15%BN+0%BLA	8.13 x 10^{-8}
85%LAT+15%BN+4%BLA	8.0 x 10^{-8}
85%LAT+15%BN+8%BLA	7.99 x 10^{-8}
85%LAT+15%BN+12%BLA	8.18 x 10^{-8}
85%LAT+15%BN+16%BLA	8.3 x 10^{-8}
80%LAT+20%BN+0%BLA	8.07 x 10^{-8}
80%LAT+20%BN+4%BLA	7.95 x 10^{-8}
80%LAT+20%BN+8%BLA	7.23 x 10^{-8}
80%LAT+20%BN+12%BLA	8.24 x 10^{-8}
80%LAT+20%BN+16%BLA	8.33 x 10^{-8}

Table A84: Unconfined Compressive Strength Test Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

MIX PROPORTIONS	AXIAL STRESS(KN/m^2)
100%LAT+0%BN+0%BLA	103.86
100%LAT+0%BN+4%BLA	108.43
100%LAT+0%BN+8%BLA	120.61
100%LAT+0%BN+12%BLA	128.85
100%LAT+0%BN+16%BLA	130.12
95%LAT+5%BN+0%BLA	119.68
95%LAT+5%BN+4%BLA	125.94
95%LAT+5%BN+8%BLA	148.40
95%LAT+5%BN+12%BLA	150.74
95%LAT+5%BN+16%BLA	165.70
90%LAT+10%BN+0%BLA	142.49
90%LAT+10%BN+4%BLA	154.03
90%LAT+10%BN+8%BLA	176.06
90%LAT+10%BN+12%BLA	184.92
90%LAT+10%BN+16%BLA	200.62
85%LAT+15%BN+0%BLA	179.14
85%LAT+15%BN+4%BLA	200.11
85%LAT+15%BN+8%BLA	217.48
85%LAT+15%BN+12%BLA	222.48
85%LAT+15%BN+16%BLA	235.52
80%LAT+20%BN+0%BLA	200.59
80%LAT+20%BN+4%BLA	211.98
80%LAT+20%BN+8%BLA	234.39
80%LAT+20%BN+12%BLA	242.69
80%LAT+20%BN+16%BLA	245.15

Table A85: Volumetric shrinkage Test Results for Lateritic Soil-Bentonite Mixtures treated with Bamboo Leaf Ash

Bentonite/Bamboo Leaf Ash	0%BN	5%BN	10%BN	15%BN	20%BN
0	3.1	2.99	3.09	2.85	3.33

4	3.22	3.46	3.83	2.97	2.84
8	2.19	3.71	3.58	1.61	1.74
12	4.2	3.34	3.11	3.33	3.83
16	3.58	3.71	4.33	4.32	4.19