
STABILIZATION OF LATERITIC SOIL USING ASPHALTIC EMULSION AND BAGASSE ASH AS BINDER

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ABSTRACT

This study is an experimental investigation that was undertaken to study the reaction a selected lateritic soil sample to stabilization with asphaltic emulsion and bagasse ash. This investigation is aimed at proffering a solution to the problem of road construction and maintenance and also building construction and maintenance with the existing and increasingly rampant situations of road failures, building failures and collapse and also failure of bridges. The undisturbed soil sample collected was mixed with 0, 4, 8, 12% asphaltic emulsion only, v then with 3, 6, 9, 12% asphaltic emulsion and 2, 4, 6, 8% bagasse ash simultaneously so as to determine the strength characteristics of the lateritic soil sample. And a series of tests were conducted on the various samples to determine their geotechnical properties and suitability in road and building constructions. From the various tests it was concluded that with the addition of the stabilization materials to the lateritic soil, there was a positive change in the geotechnical properties of the soil sample. The preliminary investigation carried out on the natural lateritic soil found in Landmark University Omu-Aran, Kwara State Nigeria shows that it falls under Group A-2-7(0) using AASHTO classification. The natural soil has a liquid limit value of 50, a plastic limit of 21.7% and a plasticity index value of 28.3%. The maximum dry density (MDD) of the soil was 1500kg/m³ and an Optimum Moisture Content (OMC) value of 23.7%. Unconfined compressive strength (UCS) values of 21.53 KN/m² for 28-days curing periods, respectively, were recorded for the natural soil. California Bearing Ratio (CBR) value of 8.89% was recorded for the natural lateritic soil. Peak UCS and CBR values of 40.48kN/m² and 41.9% were recorded for soil treated with 9% asphaltic emulsion 6% bagasse ash. The peak CBR value met the 20 – 30 % requirement for sub-base reported by Sadeeq J.A. et al (2015) for materials compacted at optimum moisture content.

Keywords: *Stablization, Laterite soils, Asphalt Emulsion, Bagasse Ash.*

INTRODUCTION

As far as any structure that is land based is concerned, the most important aspect is the foundation and has to be rigid and strong enough to carry and support the entire structure. The soil around the foundation plays the most vital role in the determination of the strength of the foundation of that structure. In other to determine the workability of a soil, an in depth knowledge of the soil i.e. its properties and factors that affect its behaviour should be

known. So, the stabilization of the soil aids in the achievement of some desired properties needed in the soil for the construction. Right from the on-set of construction works, enhancing the properties of the soil has been of great importance (Prashanth K. 2014).

Laterites are mostly used as building construction and road construction materials in various parts of the world. Taking into consideration the natural strength of laterites, the

harder ones can be used as subgrade materials for airfield and high way pavements meanwhile, the softer ones are used as good sub base course. (Oluyemi B.D. 2016). Lateritic soils as the years have gone by, has attracted increasing attention from soil Engineers as a result of how vastly distributed they are and their application in the field of Engineering. Although laterites are readily available in various countries, but their properties vary from one location to the other. As a result their performance cannot be predicted and also differs (Gidigasu 1976). These differences have resulted in some inadequacies in the use of these soils for construction works. A good number of these inadequacies can be remedied by the addition of some stabilization agents to improve the soil workability (Oluyemi B.D. etAyibiowu 2016). Lateritic soil is a soil type that is formed by the weathering and leaching of a wide variety of parent rocks. Exposed surfaces are blackish to reddish and also commonly lighter in colour (reddish, yellowish or sometimes brownish) and categorically soft when freshly broken but hardens when exposed.

Emulsion is the release of a little quantity of one liquid into another. Emulsions can be made by two liquid substances that cannot be mixed but most at times in emulsions one of the liquids is water (Prashanth K. 2014). Asphalt emulsion is an economical, energy conserving and

environmental friendly product used in paving and stabilization. An asphalt emulsion is liquid asphalt emulsified in water. And it comprises of asphalt, water and an emulsifying agent (Asphalt I. 2017). Asphalt emulsion can also be described as a mixture of liquid that is made up of asphalt cement, water and an agent of emulsification that is used for construction works and it has become widely accepted by engineers for the fact that it is of economic advantage and its performance (Coyne 1976, Oluyemi B.D. 2016). Asphalt emulsion helps in the conservation of energy in the sense that petroleum is not needed to soften it and there is no application of heat when used. It also contributes to the reduction of pollution atmospherically because there is almost no hydrocarbon emitted from it (Coyne and Ripple, 1975, Oluyemi B.D. 2016).

Bagasse-ash is an agricultural material gotten subsequently by pressing out the sweet juice in sugarcane and burning the remnants to ash. Bagasse is the fibrous remnant gotten from sugarcane after the removal of sugar juice at sugarcane mills or sugar producing factories (Osinubi and Stephen, 2005). The soil conditions and also the climatic conditions good for the manufacture of sugarcane are existent in the Northern part of Nigeria and for this reason; there is ample manufacture of sugar cane in that region. Following-up to the

above-mentioned is the immense generation of sugarcane residue waste which establishes disposal difficulties and therefore needs control. There is yet no sufficient responsiveness about how useful the sugarcane residue can be in the country; however a really small significance has been given to it. In other instances, the residue is being used as a source of primary fuel for both sugar mills and also in the production of paper. Nevertheless burning it to ash and implementing it as an admixture in the stabilization of soils because it has been discovered to be a good pozzolana, and also adds to its economic value (Okonkwo U.N., 2015).

Aim and Objectives

The aim of the study is to evaluate the stabilization of lateritic soil using asphaltic emulsion as the stabilizing agent and bagasse-ash as binder. The objectives of this study also include the following:

- (i) To determine stability of lateritic soils in response to loads applied on it and determine the properties that need to be altered to get the required properties for construction works.
- (ii) To investigate the strength, composition, gradation, bearing capacity and general stability of lateritic soils when asphaltic emulsion is used as a stabilizing agent.
- (iii) To determine whether asphalt emulsion can be used as a core stabilizing agent for the stabilization

of lateritic soils and also bagasse-ash as binder.

(iv) To examine the effect of bagasse-ash on the compaction and strength characteristic of the asphaltic emulsion stabilized lateritic soil.

(v) To compare the strength, composition, gradation, bearing capacity and general stability of lateritic soil with asphaltic emulsion used as stabilizing agent and when bagasse-ash is used as a binder along-side asphaltic emulsion for the stabilization of lateritic soil.

Scope of Study

Soils have peculiarities, they vary in properties. In other words, no two soils can be similar in all properties but can behave alike in some cases. For example, peculiarities of structure may play more important role in cement stabilization than the Atterberg limits. Lateritic soils with the same and similar plasticity index may have completely different behaviours in mixing operations. This study comprehensively covers experimental laboratory investigation on the response, strength, appropriateness and effectiveness of asphaltic emulsion as a stabilizing agent on lateritic soil. In this study, the following properties of the soil are examined before and after stabilization with asphaltic emulsion and also before and after addition of bagasse ash as a binder in the soil alongside asphaltic emulsion:

- (i) the soil gradation,

- (ii) shear strength,
- (iii) Durability,
- (iv) Plasticity index,
- (v) Workability

LITERATURE REVIEW

The term Laterite" came to be in academic literature over a century ago. This term was used first in 1807 to represent a construction material in the mountainous region of Malabar, India (Maignien R., 1966). The term „Laterite" may perhaps mean brick earth in some native languages but the name 'latérite' acquired its meaning from a Latin word "later" which means 'brick' and so connecting mainly but not completely to the use of these soils in block making (Prescott and Pendleton, 1952 in Gidigas, 1974). There have remained numerous opinions, criticism and learned discussion on the descriptions of "laterite" and "lateritic soils" by different authors and workers giving different explanations in relation to its physical nature, chemistry, origin and morphology (Oyelami, C.A. 2016). (Gidigas (1972) termed laterites as; "all the reddish, tropically weathered residual and non-residual soils together with laterite rocks". (Gidigas, 1974, Oyelami C.A. 2016) defined "laterite" as "soils in which a laterite horizon is originated, and "laterite soils" as can also be defined as those in which there is an undeveloped laterite horizon through which a real laterite horizon will progress if suitable circumstances are present

long enough". By inference, a lateritic soil can at this point be defined as that which has an immature laterite horizon capable of turning into a true laterite given the appropriate conditions and sufficient time. Lateritic soils are usually referred to as very weathered tropical or sub-tropical residual soils with changing amounts of particle sizes ranging from clay size to gravel, usually coated with sesquioxide rich concretions. It is occasionally discussed as brick earth with respect to its uses (Oyelami C.A. 2016). Laterite is an extremely weathered material rich in secondary oxides of iron, aluminium, or both. It is practically lacking bases and primary silicates, but may comprise of large volumes of quartz and kaolinite. It is either solid or liable to solidification on exposure to wetting and drying (Alexander and Cady, 1962).

Formation

Laterites are the results of severe and extensive long-term tropical rock weathering which is deepened by increased precipitation and eminent temperatures. Development of most of the laterites takes place in the Tertiary. For an accurate appreciative knowledge of laterite development we must pay attention to the chemical reactions amongst the rocks visible at the exterior and the permeated rain water. These reactions are most importantly measured by the mineral configuration of the rocks and their

physical properties (cleavage, porosity) which supports the admission of water. The second important reason for the establishment of laterites are the properties of the reacting water (dissolved constituents, temperature, acidity pH, redox potential Eh) which are themselves measured by the type of weather, plant life and the morphology of the landscape. Tropical and subtropical areas show generally a rather high yearly rainfall but its sequential dissemination differs intensely from countries with noticeable and long-term dry seasons to equatorial regions with an extra constant rainfall. Chemical weathering reduces during the dry seasons at least beyond the inconsistent water table. Aqueous suspension of minerals continues when a chemical stability is not attained i.e. when the dissolved elements are removed in the water. The chemical reactions are additionally well-ordered by the action of water which is equivalent to one in spontaneously moving water but reduced within minor pores in the soil. Constancy and reaction rate differ from mineral to mineral; e.g. quartz is more constant than feldspar. Minerals of similar class e.g. kaolinite can display dissimilar crystallinity which likewise controls their constancy. Resilient changes continue at the exterior of the parent rock however it is subordinate in the regolith on top of the rock. The chemical and mineralogical effects have revealed

that the main minerals are in general not completely dissolved but partly changed in secondary minerals which are extra steady under the severe weathering conditions. The components in the main rock minerals are free and indicate diverse reactions in the aqueous solution. The elements Na, K, Mg and Ca do not react with other elements and are detached in the permeating water. The major dissolution is principally stimulated by an increased acidity (lower pH) of the water. A high proportion of the dissolved Si is likewise detached but a different portion reacts with dissolved Al and produces the clay mineral kaolinite. The aluminium hydroxide gibbsite is produced if the concentration of dissolved Si is very small owing to a very strong drainage. Dissolved Fe is very reactive with hydroxyl ions and produces after oxidation goethite and hematite which results in the red-brown colour of laterites. Therefore the prevailing procedure of laterite development is the residual (or relative) enrichment of iron and recurrently of aluminium by subtraction of silica, alkalis and alkaline earths. This chemical change relates mineralogically with the development of goethite, hematite, kaolinite and gibbsite. These minerals combined with relicts of to some extent dissolved quartz form the bulk of laterites.

The conversion of rock into laterite continues over-all progressively as pointed out by the steady rise of iron

and fall of silica in laterite profiles above the parent rock. In other words, the original products of weathering cannot be called laterites. They also formed in temperate type of weather and are fundamentally kaolinized rocks still presenting the construction of the rock. They are known as saprolites in which iron is not as sturdily concentrated as in laterites. Certain saprolites show owing to finely dispersed hematite a deep-red colour and are occasionally mistakenly thought to be laterite. Saprolites as well as laterites are currently categorized as residual rocks which in their amount can be seen as an individual rock group.

An up-to-date laterite definition ought to encompass every products of concentrated tropical weathering independent of their properties and parent rocks, except products of weaker weathering which also ensue in temperate climate. Red-brown laterites on granites, granitic gneisses, clays and shale are usually solid or solidify after drying, whereas laterites on basalts are normally friable and indicate an concentrated reddish colour. Lateralization of alkaline rocks (nephelinites, phonolites) habitually fallouts in formation of extremely aluminous laterites (bauxites) with brighter colour. On ultramafic rocks (serpentinites etc.) forms very soft, yellow-brown Ni-bearing goethite (nickel limonite ore). Most weathering products are formed by

the same essential weathering process and are consequently diverse participants of a laterite family (Schellmann W. 1995).

Factors Affecting Lateritic Soil Formation

(1) Nature of the Parent Material/ Parent Rock

The parent rock material is the rock material that disintegrates into rock particles and might affect the nature of the soil in relation to the fertility, mineral configuration, depth, colour and the ultimate soil profile. Parent rock may perhaps be hard or it may be soft. Hard parent rock is generally impervious to weathering and can cause skeletal soils being formed. Alternatively, comparatively soft rocks are effortlessly disintegrated into soil particles and the outcome this is greater rate of soil formation. The parent rock arrangement may be categorized by connections or joints of weakness or may be just a block of massive rock. Well-jointed rocks are prone to weathering and effortlessly broken down to provide increase to soil particles. Then again, enormous rocks might not readily permit agents of weathering to infiltrate through them and consequently there is a reduced rate of weathering and soil formation. The colour of the parent rock also affects the degree to which soil is formed. This is due to bright coloured or shiny parent rock may be more impervious to weathering. On the other hand shady coloured rocks absorb heat, swell and well ahead, they contract

leading to the disintegration of rocks through the weathering process of thermal extension or exfoliation. This as a result means that by means of the dark coloured rock soil formation is more rapid. The mineral configuration of the parent rock defines the nature of the soil nutrients or soil fertility. Rocks of limestone nature will bring about increase to soils that are buoyant in lime. Furthermore mineral composition may define the nature and proportion of chemical disintegration of the rocks. Porosity or permeability of the parent rock may also regulate the degree of soil formation. Porous rocks permit in the agents of weathering such as moisture and air, which accelerate the chemical break down of rocks into soil. Alternatively non-permeable rocks may lead to the establishment of thin soils for the reason that the low rate of soil formation.

(2) Climate

Climate encourages soil formation through its part in weathering that indicates the formation of soil. In areas of substantial precipitation sufficient moisture is made available for the process of chemical weathering. Additionally, in the desert areas soil formation through physical weathering processes like exfoliation are common. High temperatures speed up chemical weathering resulting in high rate of soil formation compared to areas of subordinate temperatures where soil

formation through chemical weathering is restricted. In very cold provinces like mountain tops, the nature of soil formation is through physical weathering procedures like frost action or freeze and thaw. Climate also controls the nature of plant life and animal life that subsequently contribute to the soil formation through the accumulation of humus.

(3) Living Organisms/Biota

These comprise of bacteria, insects, mammals (animals), human beings and plants. Bacteria perform an essential part in the breakdown of rocks by complex processes. Organisms such as earthworms, termites also perform a key role in the breakdown of rocks into simple or reduced substances that make up soil. Rodents e.g. rats, moles, squirrels etc. physically breakdown rocks as they bore holes inside the ground. Man can also impact soil formation by activities like mining/Quarrying and digging. Consequently, masses of rock are physically weathered by man to bring about soil. Alternatively, plant roots physically disintegrate rocks as they propagate into the ground. Plant roots can also secrete substances that chemically decay rocks to form soil. Plant leaves or branches can fall down and decompose to produce humus that is added to the soil through the soil production processes of Humification and therefore mineralization. This is why in regions of impenetrable

vegetative cover the soils are rich in organic content while in desert areas or areas of limited vegetation cover the soils have restricted humus.

(4) Relief/Topography

Relief impacts soil formation through erosion and deposition. The nature of the relief influences the rate and nature of the soils formed. Steep slopes are easily eroded and this infers that the weathered materials on the steep slope soils are likely to be skeletal because of erosion. Nevertheless the speed of soil formation is high because erosion reveals the parent rock to additional weathering. Alternatively in the gentle slopes, soils are likely to be deep, mature and with a well-developed profile. In the lowlands or flatlands where precipitation is high leaching occurs and can bring about the formation of Laterite soils that are deprived when it comes to plant nutrients.

(5) Time

This talks about the period of the interaction between the soil forming processes and factors. Soil formation needs sufficient time, time is essential in that the type of the soils is determined by how long the processes and factors have been interacting. If a parent rock has been open to the weathering processes for a long while, soil formation will be far-reaching in comparison with a parent rock open for a relatively shorter time (Gayaza High School, 2015). 14

Properties of Lateritic Soils

The lateritic soils perform a lot like fine grained sands, gravels, and soft rocks. The lateritic soil characteristically has a permeable or porous look. Certain particles of lateritic soils are likely to crush effortlessly underneath impact, breaking-down into a soil material that is likely to be plastic. Lateritic soils can be self-hardening once opened to drying; or assuming the lateritic soils are not self-hardening, they might comprise of substantial volumes of hardened laterite rock or laterite gravels (Abebaw Z., 2005). The term lateritic soils are used for altogether reddish tropically weathered materials, notwithstanding the specifics of the rate of their weathering and nonetheless the occurrence or non-appearance of concretions. This usage has been explained as to carry no consequence with respect to the explanation or properties of laterites. The discrepancy used for engineering tenacities is based on variances in physical or engineering properties (Abebaw Z., 2005).

Soil Properties

The properties of soils is divided into

- (1) Physical properties
 - (i) Horizonation
 - (ii) Soil colour
 - (iii) Soil texture
 - (iv) Soilstructure
 - (v) Soil consistence
 - (vi) Bulkdensity
- (2) Chemical properties

- (i) Cation Exchange Capacity
- (ii) Soil Reaction (pH)

Physical Properties

- (i) Horizonation

Soil "horizons" are distinct strata that the soil profile is composed of. They are characteristically equivalent with the ground surface. In certain soils, they display indication of the activities of the soil forming processes.

O horizons: are controlled by mainly organic material. A selected are saturated with water for lengthy periods of time or used to be saturated but now are artificially drained; while the remaining have not ever been saturated.

A horizons: are mineral layers that are formed at the exterior or underneath the O horizon, they display elimination of all quite a number of the initial rock structure, and that display one or both of the following:

- a build-up of humified organic matter very well mixed together with the mineral portion and not controlled by properties distinctive to E or B horizons
- amendment as an effect of the activities of cultivation, pasturing, or similar kinds of disturbance

E horizons: are mineral layers that display the deterioration of silicate clay, iron, aluminum, humus, or certain amalgamation of these,

leaving a concentration of sand and silt particles. These horizons display elimination of all or a number of the original rock structure.

B horizons: are mineral layers that characteristically form below an A, E, or O horizon and are controlled by elimination of all or an amount of the original rock structure and display one or more of the following: 16

- illuvial concentration of silicate clay, iron, aluminum, humus, carbonate, gypsum, or silica, alone or in combination
- evidence of removal of carbonates
- residual concentration of sesquioxides
- coatings of sesquioxides that creates the horizon evidently inferior in value, higher in chroma, or redder in hue than superimposing horizons without obvious illuviation of iron.
- alteration that forms silicate clay or liberates oxides or both and that forms granular, blocky, or prismatic structure if volume changes accompany changes in moisture content; or brittleness

C horizons: are mineral layers that are not bedrock and are not really affected by pedogenic processes and have non-existent properties of O, A, E or B horizons. The material of C layers can either be similar or dissimilar from those which the overlying soil horizons presumably formed. The C horizon could have

been improved although there is no indication of pedogenesis.

R horizons: are stratum of hard bedrock.

Transitional horizons: are controlled by properties of one principal horizon, but have subsidiary properties of another. AB and B/C are illustrations of transitional horizon designations (Soil Survey Staff, 2003).

(ii) Soil Colour

In much ventilated or aerated soils, oxidized or ferric (Fe⁺³) iron compounds are accountable for the brown, yellow, and red colours noticeable in the soil. 17

Once iron is reduced to the ferrous (Fe⁺²) form, it turns out to be moveable, and can be removed from some parts of the soil. As soon as the iron is removed, a grey colour is left off, or the reduced iron colour perseveres in shades of green or blue. Upon aeration, reduced iron can be reoxidized and deposited again, at times in the same horizon, subsequently resulting in a variegated or mottled colour outline. These soil colour outlines brought about by saturation, called "redoximorphic features", can specify the period of the anaerobic state, extending from brown with a few mottles, to complete grey or "gleization" of the soil.

Soils that are majorly grey with brown or yellow mottles directly underneath the surface horizon are usually hydric.

Soil colour is stereotypically defined using specific forms of colour

reference chart, such as the Munsell Color Chart. Using the Munsell system, colour is defined in reference to the colour's "hue", "value", and "Chroma". Hue designates where in the colour spectrum the soil colour occurs, which for soils comprises of the colours yellow, red, blue, green, and grey. Value describes the precision of the colour. Chroma indicates the how strong the colour is. In a Munsell notation, the colour is written in the arrangement; hue-value-Chroma. The colour "5YR 4/3" is an example of a Munsell notation, where 5YR is the hue, 4 is the value, and 3 is the Chroma.

(iii) Soil Texture

Soil texture discusses to the fraction of the soil "separates" that composes the mineral constituents of soil. These separates are known as sand, silt, and clay. These soil separates have the following size varieties:

- Sand = <2 to 0.05 mm
- Silt = 0.05 to 0.002 mm
- 18
- Clay = <0.002 mm

Sand and silt are the "inactive" quota of the soil matrix, since they do not support the soil's capacity to hold in soil water or nutrients. These separates usually comprise of quartz or particular inactive mineral.

Subsequently, for its trivial size and sheet-like structure, clay has a big quantity of surface area per unit mass, and its surface charge attracts ions and water. For this reason, clay

is the "active" percentage of the soil matrix.

For every one mineral soils, the quantity of sand, silt, and clay all the time sum up to 100 per cent. These percentages are accumulated into soil texture "classes", which have been pre-set into a "textural triangle".

Soil texture can affect the quantity of pore space within a soil. Sand-sized soil particles are arranged together in a manner that produces large pores; nevertheless, generally there is a comparatively small volume of total pore space. Clay-sized soil particles are arranged together in such a way that produces small pores; though, generally there are additional pores present. As a result, a soil composed of clay-sized particles will have increased total pore space than would a soil composed of sand-sized particles. Therefore, clayey soils will in general have reduced bulk densities than sandy soils.

As a group, the soil separates of sand, silt, and clay are referred to as the "fine-earth fraction", and signify inorganic soil particles less than 2mm in diameter. Inorganic soil particles 2mm and bigger are called "rock fragments". When the organic matter composition of a soil exceeds 20 to 35% (on a dry weight basis) it is referred to as organic soil material, and the soil is called an organic soil. As this material is generally lacking

mineral soil material, they cannot be referred to in terms of soil texture. Nonetheless, the following "in lieu of" texture terms can be used to define organic soils:

"peat"; organic material in which the plant portions are still identifiable
"muck"; highly decomposed organic material in which no plant portions are detectable
"mucky peat"; disintegration is transitional between muck and peat

(iv) Soil Structure

The soil separates can be aggregated together into discrete structural units called "peds". These peds are structured into a reoccurring configuration that is referred to as soil structure. Amongst the peds are cracks termed "pores" through which soil air and water are directed. Soil structure is frequently referred to in terms of the shape of the separate peds that come about inside a soil horizon.

Types of Soil Structure

- Granular – they are roughly sphere-shaped, like grape nuts. Usually they are 1-10 mm in diameter. Very often in A horizons, where there are plant roots, microorganisms, and sticky produces of organic matter decay joins soil grains together into granular aggregates.
- Platy – they are flat peds that are positioned horizontally in the soil. Platy structure can be set up in A, B and C

horizons. It frequently takes place in an A horizon as the product of compaction.

- Blocky – they are coarsely cube-shaped, with additional or a smaller amount flat surfaces. If edges and corners continue to be sharp, they can be called angular blocky. If they are rounded, they can be called sub-angular blocky. Sizes generally vary from 5-50 mm across. Blocky structures are characteristics of B horizons, particularly those that have high clay composition. They can be formed by reoccurring expansion and contraction of clay minerals.

Prismatic – they are bigger, vertically extended blocks, frequently with five sides. Sizes are usually 10-100mm across. Prismatic structures generally occur in fragipans.

Columnar – the elements are comparable to prisms and are bounded by flat or slightly rounded vertical faces. The tops of columns, in divergence to those of prisms, are very distinct and normally rounded.

"Structureless" Soil Types:

Massive – they are compacted, coherent soil not disjointed into peds of any kind. Immense edifices in clayey soils typically have very slight pores, slow permeability, and reduced aeration.

Single grain – in certain very sandy soils, each grain performances

individually and there is no binding agent to clutch the grains together into peds. Permeability is fast, but fertility and water holding capability are low.

(v) Soil Consistency

Soil consistence can be defined as the simplicity with which a specific ped may be crushed by the fingers. Soil consistence, and its explanation, is determined by soil moisture content. Terms generally used to define consistence are:

Moist soil:

Loose – non-coherent when dry or moist; does not clutch together in a mass

Friable – while moist, can be crushed effortlessly under mild weight in the middle of the thumb and forefinger and can be constrained together into a lump 21

Firm – when moist, crushed under restrained pressure between thumb and forefinger, but opposition to the pressure is clearly evident

Wet soil

Plastic – when wet, freely distorted by enough pressure but can be pressed into a lump; will form a "wire" when it is rolled up between thumb and forefinger

Sticky – when wet, stick to to other material and has a tendency to stretch to a certain extent and pull away from each other rather than to pull free from other material

Dry Soil:

Soft – when dry, breaks-down into powder or separable grains under very trivial pressure

Hard – when it is dry, judiciously resilient to pressure; and can be fragmented with some effort between thumb and forefinger.

(vi) Bulk Density

Bulk density is the quantity of the bulk of a soil comparative to its volume. It can be conveyed as a unit of weight per volume, and is generally measured in units of grams per cubic centimetres (g/cc).

Bulk density is a pointer of the volume of pore space accessible inside separable soil horizons, as it is inversely proportional to pore space:

Pore space = $1 - \frac{\text{bulk density}}{\text{particle density}}$

For example, at a bulk density of 1.60 g/cc, pore space equals 0.40 or 40%.

At a bulk density of 1.06g/cc, pore space equals 0.60 or 60%. 22

The totaling of even a slight percentage of organic soil material to a mineral soil can disturb the bulk density of that soil. Liken the two soil samples below to each other:

Soil "A": 100% mineral soil material; bulk density = 1.33 g/cc

Soil "B": 95% mineral soil material and 5% organic soil material; bulk density = 1.26 g/cc

The alteration in bulk density re-counts to a change in "particle density" of mineral soil material as opposed to organic soil material. The average particle density of mineral

soil material is 2.65 g/cc, which estimates the density of quartz. Equally, the average particle density of organic soil material is 1.25 g/cc. Organic soil material in weight, weighs less than mineral soil material, so it will reduce the bulk density of a mineral soil when added, as it decreases the total weight of the soil (Natural Resources conservation Service, 2014).

Soil Chemical Properties

(i) Cation Exchange Capacity (Cec)
Certain plant nutrients and metals occur as positively charged ions, or "cations", in the soil surroundings. Between the more obvious cations found in soils are hydrogen (H⁺), aluminum (Al⁺³), calcium (Ca⁺²), magnesium (Mg⁺²), and potassium (K⁺). Almost all heavy metals also occur as cations in the soil setting. Clay and organic matter particles are principally negatively charged (anions), and also have the capacity to restrain cations from being "leached" or carried away. The riveted cations can be replaced by other cations in a speedy, reversible process called "cation exchange". The "cation exchange capacity", or "CEC", of a soil is a capacity of the extent of the negative charge per unit weight of soil, or the volume of cations a certain soil sample would hold in an interchangeable form. The more the clay and organic matter content, the more the CEC should be, even though diverse types of clay minerals and organic matter can be different

in CEC. Cation exchange is a significant instrument in soils for holding in and providing plant nutrients, and for absorbing impurities. It usually performs a vital part in the treatment of waste water in soils. Sandy soils with a reduced CEC are in general unsuited for septic systems because they possess low absorptive capacity and there is prospective for groundwater.

(ii) Soil Reaction (pH)

By explanation, "pH" is the amount of the energetic hydrogen ion (H^+) concentration. It is a signal of the acidity or alkalinity of a soil, and also recognised as "soil reaction". The pH scale varies from 0 to 14, with values under 7.0 acidic, and values exceeding 7.0 alkaline. A pH value of 7 is measured neutral, where H^+ and OH^- are equal, both at a concentration of 10^{-7} moles/liter. A pH of 4.0 is ten times additionally acidic compared to a pH of 5.0.

The very significant end product of pH in the soil is on ion solubility, which in return moves microbial and plant growth. A pH variety of 6.0 to 6.8 is idyllic for almost all crops for the reason that it corresponds with the best possible solubility of the most significant plant nutrients. A number of inconsequential elements (e.g., iron) and almost all weighty metals are more soluble at lower pH. This can result in pH managing essential in monitoring movement of weighty metals (and potential

groundwater contamination) in soil. In acidic soils, hydrogen and aluminium are the principal interchangeable cations. The second is soluble under acidic surroundings, and its reactivity with water (hydrolysis) produces 24 hydrogen ions. Calcium and magnesium are elementary cations; as their volumes escalate, the comparative volume of acidic cations will drop.

Elements affecting soil pH consist of the following: parent material, vegetation, and climate. A number of rocks and sediments bring about soils that are more acidic compared to the others: quartz-rich sandstone is acidic; limestone is alkaline. A number of vegetation types, principally conifers, bring about organic acids, which may add to reduced soil pH values. In humid regions for instance the eastern US, soils have a tendency of becoming more acidic as time goes by since rainfall leaches away basic cations and interchanges them with hydrogen. Adding a number of fertilizers to soil may in addition bring about hydrogen ions. Liming the soil enhances calcium, which substitutes replaceable and solution H^+ and increases soil pH. Lime necessities, or the total quantity of liming material necessary to increase the soil pH to a particular level, escalate with CEC. To reduce the soil pH, sulphur may be supplementary, which in turn can produce sulphuric acid (Brady, N.C. 1990, National Resource

Conservation Service, 2014, Natural Resources conservation Service, 2014).

Soil Stabilization

Soil Stabilization is the modification of soils to improve their physical properties. Stabilization can escalate the shear strength of a soil and/or regulate the shrink-swell properties of a soil, thus refining the load bearing capacity of a sub-grade to provide support for pavements and foundations (Mid-state reclamation, 2016). Soil Stabilization can be employed on roadways, parking areas, site development projects, airports and numerous additional conditions in which sub-soils are not appropriate for construction. Stabilization can be utilized in treating a wide variety of sub-grade materials, ranging from expansive clays to granular materials. This development is accomplished with the use of a wide variation of additives, lime inclusive, fly-ash, and Portland cement. Other material by-products used in stabilization comprise of lime-kiln dust (LKD) and cement-kiln dust (CKD) (Mid-state reclamation, 2016).

MATERIALS AND METHOD

Soil Sampling

The lateritic soils were sourced from the front of Mandate Lodge at Landmark University, Omu-Aran, Kwara State Nigeria. The method used for sample collection was the trial pit method. The pit was dug up to a depth of 1m by hand excavation

with the aid of a shovel and the lateritic soil sample was collected using a soil hugger. The soil samples collected were placed in cement bags.

Asphaltic Emulsion

The asphaltic emulsion was collected from a construction company in Omu-Aran Kwara State Nigeria.

Bagasse Ash

Bagasse ash was locally obtained from potatoes and sugar cane market at FarinYada, Zaria Road, Jos-Plateau State Nigeria. The sugar cane waste (bagasse) was collected, air-dried and burnt in a confined drum at temperature of 2800c to 3200c. The residue obtained after burning was the ash that was collected in a sack and transported to the Geotechnical/Highway Laboratory, Department of Civil Engineering, Landmark University Omu-Aran, Kwara State. The ash was then passed through sieve no. 200 (0.075mm). o.



The experimental design is as follows:

Asphaltic emulsion Stabilization

- (i) 100% lateritic soil + 0% asphaltic emulsion
- (ii) 96% lateritic soil + 4% asphaltic emulsion
- (iii) 92% lateritic soil + 8% asphaltic emulsion
- (iv) 88% lateritic soil + 12% asphaltic emulsion

Asphaltic emulsion – Bagasse ash Stabilization

- (i) 100% lateritic soil + 0% asphaltic emulsion + 0% bagasse ash
- (ii) 95% lateritic soil + 3% asphaltic emulsion + 2% bagasse ash
- (iii) 90% lateritic soil + 6% asphaltic emulsion + 4% bagasse ash
- (iv) 85% lateritic soil + 9% asphaltic emulsion + 6% bagasse ash
- (v) 80% lateritic soil + 12% asphaltic emulsion + 8% bagasse ash

Experimental Programme

Laboratory tests were performed on both the controlled soil sample and the stabilized soil sample to evaluate some of their physical characteristics and engineering properties. The laboratory test are carried out to determine the suitability of the lateritic soils for use as base and sub-base material making use of different codes for laboratory experiments, the following laboratory tests were

carried out on both the controlled sample and the stabilized sample

- (i) Particle Size Analysis by Sieve Analysis
- (ii) Moisture content test
- (iii) Atterberg Limit Test (liquid limit)
- (iv) Proctor Compaction test
- (v) California Bearing Ratio test 43
- (vi) Direct shear test
- (vii) Unconfined compressive strength test

Particle Size Analysis by Sieve Analysis

Sieve analysis was performed in order to determine the soil particle size distribution. Representative sample of approximately 500 g was used for the test after washing and oven-dried. The sample was washed using the BS 200 sieve and the fraction retained on the sieve was air dried and used for the sieve analysis. The sieving was done by mechanical method using an automatic shakers and a set of sieves.

Experimental Procedure

- (1) Write down the weight of each sieve as well as the bottom pan to be used in the analysis.
- (2) Record the weight of the given dry soil sample.
- (3) Make sure that all the sieves are clean, and assemble them in the ascending order of sieve numbers (#4 sieves at top and #200 sieve at bottom). Place the pan below #200 sieves. Carefully pour the soil sample into the top sieve and place the cap over it.

(4) Place the sieve stack in the mechanical shaker and shake for 10 minutes.



(5) Remove the stack from the shaker and carefully weigh and record the weight of each sieve with its retained soil. In addition, remember to weigh and record the weight of the bottom pan with its retained fine soil. Plate 3: Mechanical sieve shaker used in carrying out sieve analysis 45

Atterberg Limit Test (Liquid Limit)

This test determines the clay content in terms of liquid limit, plastic limit, plasticity index and shrinkage potential in order to estimate plasticity, strength and settlement characteristics of the soil sample. For the determination of liquid limit, the soil sample passing through 425 μm sieve, weighing 200 g was mixed with water to form a

thick homogeneous paste. The paste was collected inside the Cassangrade's apparatus cup with a groove created and the number of blows to close it was recorded. Similarly, for plastic limit determination, the soil sample weighing 200 g was taken from the material passing the 425 μm test sieve and then mixed with water till it became homogenous and plastic to be shaped to ball. The ball of soil was rolled on a glass plate until the thread cracks at approximately 3 mm diameter. The 3 mm diameter sample was placed in the oven at 105°C to determine the plastic limit.

Experimental Procedure

- (1) Take roughly 3/4 of the soil and place it into the porcelain dish. Assume that the soil was previously passed through a No. 40 sieve, air-dried, and then pulverized. Thoroughly mix the soil with a small amount of distilled water until it appears as a smooth uniform paste. Cover the dish with cellophane to prevent moisture from escaping.
- (2) Weigh four of the empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet.
- (3) Adjust the liquid limit apparatus by checking the height of drop of the cup. The point on the cup that comes in contact with the base should rise to a height of 10 mm. The block on the end of the grooving tool is 10 mm high and should be used as a gage. Practice using the cup and determine the correct rate to rotate the crank so

that the cup drops approximately two times per second.

(4) Place a portion of the previously mixed soil into the cup of the liquid limit apparatus at the point where the cup rests on the base. Squeeze the soil down to eliminate air pockets and spread it into the cup to a depth of about 10 mm at its deepest point. The soil pat should form an approximately horizontal surface.

(5) Use the grooving tool carefully cut a clean straight groove down the center of the cup. The tool should remain perpendicular to the surface of the cup as groove is being made. Use extreme care to prevent sliding the soil relative to the surface of the cup.

(6) Make sure that the base of the apparatus below the cup and the underside of the cup is clean of soil. Turn the crank of the apparatus at a rate of approximately two drops per second and count the number of drops, N ; it takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of 13 mm (1/2 in.). If the number of drops exceeds 50, then go directly to step eight and do not record the number of drops, otherwise, record the number of drops on the data sheet.

(7) Take a sample, using the spatula, from edge to edge of the soil pat. The sample should include the soil on both sides of where the groove came into contact. Place the soil into a moisture can cover it. Immediately weigh the moisture can containing the soil, record its mass, remove the

lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Place the soil remaining in the cup into the porcelain dish. Clean and dry the cup on the apparatus and the grooving tool.

(8) Remix the entire soil specimen in the porcelain dish. Add a small amount of distilled water to increase the water content so that the number of drops required closing the groove decrease.

(9) Repeat steps six, seven, and eight for at least two additional trials producing successively lower numbers of drops to close the groove. One of the trials shall be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

Moisture Content Test

Moisture content is defined as the ratio of the weight of the water in a soil specimen to the dry weight of the specimen. The moisture content of lateritic soil can be influenced by the mineralogy and formation environment.

Experimental Procedure

(1) Clean the containers with lid dry it and weigh it (W_1). "Make sure you

do this after you have tared the balance"

(2) Take a specimen of the sample in the container and weigh with lid (W_2).

(3) Keep the container in the oven with lid removed. Dry the specimen to constant weight maintaining the temperature between 1050 C to 1100 C for a period varying with the type of soil but usually 16 to 24 hours.

(4) Record the final constant weight (W_3) of the container with dried soil sample. Peat and other organic soils are to be dried at lower temperature (say 600 C) possibly for a longer period.

Compaction Test

The densification of soil with mechanical equipment thereby rearranging the soil particles which makes them more closely packed resulting in an increase of the ratio horizontal effective size to the vertical effective stress. The degree of compaction is measured in term of 48 its dry weight and it increasing the bearing capacity of road foundation, stability slopes, controls undesirable volume changes and curb undesirable settlement of structures. The mould is filled and compacted with soil in five layers via 25 blows of a 4.5 rammer.

Experimental Procedure

(1) Depending on the type of mold you are using obtain a sufficient quantity of air-dried soil in large mixing pan. For the 4-inch mold take approximately 4.5Kg, and for the 6-

inch mold take roughly 6.8kg. Pulverize the soil and run it through the # 4 sieve.

(2) Determine the weight of the soil sample as well as the weight of the compaction mold with its base (without the collar) by using the balance and record the weights.

(3) Compute the amount of initial water to add by the following method: Assume water content for the first test to be 8 percent. Compute water to add from the following equation: Water to add (in ml) = (soil mass in grams) 8/100. Where "water to add" and the "soil mass" are in grams. Remember that a gram of water is equal to approximately one milliliter of water.

(4) Measure out the water, add it to the soil, and then mix it thoroughly into the soil using the trowel until the soil gets a uniform colour.

(5) Assemble the compaction mould to the base, place some soil in the mold and compact the soil in the number of equal layers specified by the type of compaction method employed. The number of drops of the rammer per layer is also dependent upon the type of mold used. The drops should be applied at a uniform rate not exceeding around 1.5 seconds per drop, and the rammer should provide uniform coverage of the specimen surface. Try to avoid rebound of the rammer from the top of the guide sleeve.

(6) The soil should completely fill the cylinder and the last compacted layer must extend slightly above the collar joint. If the soil is below the collar

joint at the completion of the drops, the test point must be repeated. (Note: For the last layer, watch carefully, and add more soil after about 10 drops if it appears that the soil will be compacted below the collar joint.)

(7) Carefully remove the collar and trim off the compacted soil so that it is completely even with the top of the mould using the trowel. Replace small bits of soil that may fall out during the trimming process.

(8) Weigh the compacted soil while it's in the mould and to the base, and record the mass. Determine the wet mass of the soil by subtracting the weight of the mould and base.

(9) Remove the soil from the mould using a mechanical extruder and take soil moisture content samples from the top and bottom of the specimen. Fill the moisture cans with soil and determine the water content.

(10) Place the soil specimen in the large tray and break up the soil until it appears visually as if it will pass through the # 4 sieve, add 2 per cent more water based on the original sample mass, and re-mix as in step 4. Repeat steps 5 through 9 until, based on wet mass, a peak value is reached followed by two slightly lesser compacted soil masses.

California Bearing Ratio (CBR) Test

The California bearing ratio (CBR) test is a penetration test carried out to evaluate the mechanical strength

of a sub-base or base course material. It measures the shearing resistance, controlled density and moisture content. Both the soaked and unsoaked method of CBR was conducted to characterize the lateritic soil for use as a base or sub-base material. A portion of air-dried soil sample was mixed with about 5% of its weight of water. This was put 50 in CBR mould in 3 layers with each layer compacted with 55 blows using 2.5 kg hammer at drop of 450 mm (standard proctor test). The compacted soil and the mould was weighed and placed under CBR machine and a seating load of approximately 4.5 kg was applied. Load was recorded at penetration of 0.625, 1.0, 2.25, 6.25, 7.5, 10 and 12.5 mm.

Experimental Procedure

(1) Place the mould assembly with the surcharge weights on the penetration test machine.

(2) Seat the penetration piston at the center of the specimen with the smallest possible load, but in no case in excess of 4kg so that full contact of the piston on the sample is established.

(3) Set the stress and the strain dial gauge to read zero, apply the load on the piston so that the penetration rate is about 1.25mm/min.

(4) Record the load readings at the penetration of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4, 5.0, 7.5, 10 and 12.5mm. Note the maximum load and the corresponding penetration if it

occurs for a penetration less than 12.5mm.

(5) Detach the mould from the loading equipment. Take about 20 to 50g of soil from the top 3 cm layer and determine the moisture content.

Direct Shear Test:

Direct shear test is a test performed to determine the shearing strength of the soil using the direct shear apparatus. In many engineering problems such as design of foundation, retaining walls, slab bridges, pipes, sheet piling, the value of the angle of internal friction and cohesion of the soil involved are required for the design. Direct shear test is used to predict these parameters quickly. The laboratory report covers the laboratory procedures for determining these values for cohesionless soils. 51

Experimental Procedure

- (1) Check the inner dimension of the soil container.
- (2) Put the parts of the soil container together.
- (3) Calculate the volume of the container. Weigh the container.
- (4) Place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil.
- (5) Weigh the soil container, the difference of these two is the weight of the soil. Calculate the density of the soil.
- (6) Make the surface of the soil plane.

(7) Put the upper grating on stone and loading block on top of soil.

(8) Measure the thickness of soil specimen.

(9) Apply the desired normal load.

(10) Remove the shear pin.

(11) Attach the dial gauge which measures the change of volume.

(12) Record the initial reading of the dial gauge and calibration values.

(13) Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil.

(14) Start the motor. Take the reading of the shear force and record the reading.

(15) Take volume change readings till failure.

(16) Add 5 kg normal stress 0.5 kg/cm² and continue the experiment till failure

(17) Record carefully all the readings. Set the dial gauges zero, before starting the experiment.

Unconfined Compressive Strength Test

This test is usually carried out to determine the unconfined compressive strength (q_u) of the soil. The apparatus used to carry out this test includes: Unconfined compression testing machine (Triaxial Machine), Specimen preparation equipment, Sample extruder, Weighing Balance.

Experimental Procedure

(1) Remolded specimens are prepared in the laboratory depending on the proctors data at the required molding water content.

- (2) If testing undisturbed specimens retrieved from the ground by various sampling techniques, trim the samples into regular triaxial specimen dimensions (2.8" x 5.6")
- (3) There will be a significant variation in strength of undisturbed and remolded samples.
- (4) Measure the diameter and length of the specimen to be tested.
- (5) If curing the sample (treated soils), wrap the samples in a geotextile and then a zip bag. Place the sample in a humidity room maintained at a relative humidity of 90%.
- (6) Prior to testing, avoid any moisture loss in the sample, place on a triaxial base (acrylic). The ends of the sample are assumed to be frictionless.
- (7) The triaxial cell is placed above the sample and no confinement is applied.
- (8) The rate of strain is maintained at 1.2700 mm/min as per ASTM specifications.

- (9) The data acquisition system collects real time data and the test is stopped when there is a drop observed in the strain versus load plot.

RESULTS AND DISCUSSION

This study presents the results of the laboratory test carried out on bagasse ash and asphaltic emulsion treated lateritic soil and a discussion on their relevance to the practise. The laboratory tests carried out include moisture content, grain size analysis (sieve analysis), compaction test, Atterberg limits test, direct shear test, California bearing ratio test, unconfined compressive strength test.

Moisture Content Test

The results of the moisture content test carried out using the drying oven for the soil sample is as shown in Table 4.1.

Table 4.1: Natural moisture content of lateritic soil sample by the climatic condition

Sample No	Water Content (%)
1	12.00
2	19.05
3	17.86
Average	16.30

Atterberg Limits

The results of the Atterberg limit test using Casagrande method with different contents of asphaltic emulsion and bagasse ash for the soil

is as shown in Table 4.2. The tests were performed after the cooling of the soil sample after the addition of asphaltic emulsion and bagasse ash.

Table 4.2: Atterberg limits of the soil for different asphaltic emulsion and bagasse ash contents

Stabilizer Content	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Control Sample	50	21.7	28.3
4% Bitumen	42.5	24.8	17.67
8% Bitumen	40	29.2	10.76
12% Bitumen	48	32.1	15.87
3% Bitumen 2% Asphaltic Emulsion	42	32	10.02
6% Bitumen 4% Asphaltic Emulsion	43	31	12
9% Bitumen 6% Asphaltic Emulsion	48	31.2	16.8
12% Bitumen 8% Asphaltic Emulsion	50	33	16.97

According to Whitlow (1995), liquid limit less than 35% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity and greater than 90% extremely high plasticity of the natural soil. This shows that the natural soil sample is of intermediate

plasticity which is suitable for asphaltic emulsion stabilization because of its non-difficulty in mixing as stated by Oluyemi B.D. (2006). The plasticity indices of the soil samples decreased on addition of the different percentages of stabilizers. It was observed by Amu O.O (2011) that the reduction in plasticity indices of the soil samples is an indicator of soil improvement.

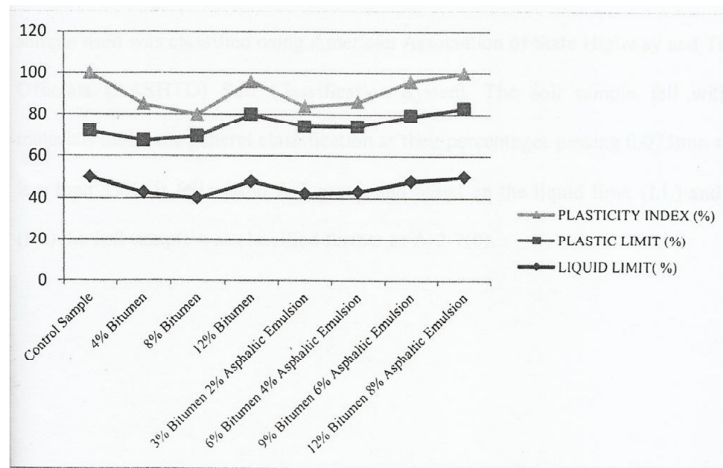


Figure 4.1: Variations of atterberg limits of Asphaltic Emulsion stabilized lateritic soil with bagasse ash

Particle Size Distribution

The grain size distribution of the soil is shown in Figure 4.2 below. It can be seen that the soil sample used was classified using American Association of State Highway and Transportation Officials (AASHTO) Soil Classification System. The soil sample fell within

granular materials under the general classification as their percentages passing 0.075mm sieve were all less than 35%. It fell within A-2 group and based on the liquid limit (LL) and plastic limit (PL) the soil sample was classified further as A-2-7(o).

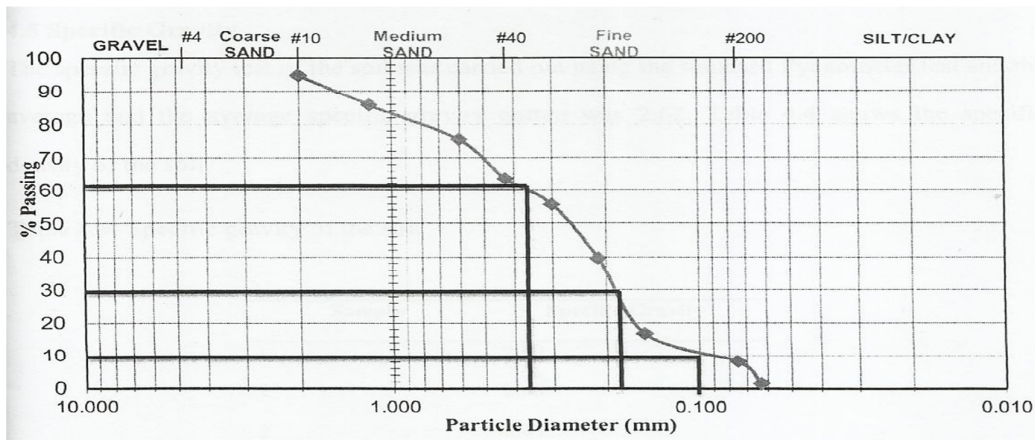


Figure 4.2: Grain size distribution of the soil

Table 4.3: Grain size distribution curve results

Grain Size Distribution Curve Results:

% Gravel:	0	D ₁₀ :	0.1	C _u :	3.8
% Sand:	60.5	D ₃₀ :	0.18	C _c :	0.85
% Fines:	39.5	D ₆₀ :	0.38		

Specific Gravity

The specific gravity test of the soil was carried out using the standard Pycnometer test and the average and the average specific gravity gotten was 2.62. Table 4.4 shows the specific density of the soil.

Table 4.4: Specific gravity of the soil

Sample	Specific Gravity
1	2.69
2	2.46
3	2.43
4	2.91
Average	2.62

Compaction Test

Standard proctor compaction test was carried out on the control soil sample and the stabilized soil sample to determine the optimum moisture content and the maximum dry density. A standard proctor mould of 100mm x 117mm diameter and height respectively was filled in three layers, each layer having 25 blows at a height of 300mm. Table 4.5 shows the optimum moisture content (OMC) and the maximum dry density (MDD).

Table 4.5: Optimum moisture content and maximum dry density for different asphaltic emulsion with bagasse ash content.

Stabilizer Content	Optimum Moisture (%)	Maximum Dry Density(Kg/M ³)
Control Sample	23.7	1500
4% Asphaltic Emulsion	9.5	1700
8% Asphaltic Emulsion	21.8	1940
12% Asphaltic Emulsion	29.5	1400
3% Asphaltic Emulsion & 2% Bagasse Ash	15	1600
6% Asphaltic Emulsion & 4% Bagasse Ash	27.5	1520
9% Asphaltic Emulsion & 6% Bagasse Ash	5.5	1670
12% Asphaltic Emulsion & 8% Bagasse Ash	15	1500

Stabilized soil sample there was an initial increase then a decrease from 27.5% to 5.5% with an increase in the bagasse ash content. The increase in OMC is most likely due to the additional water held within the flocculent soil structure due to excess water absorbed as a result of the porous property of bagasse ash. In addition, the maximum dry density (MDD) increased with an increase in the emulsion content which is a general indication of soil improvement as stated by Amu O.O. (2011). However there was a

decreased in the MDD with an increase in the asphaltic emulsion and bagasse ash content from 1520kg/m³ to 1670kg/m³ and from 1670kg/m³ to 1500kg/m³. Das (2000) was of the opinion that a change down in density might occur due to both the particle size and specific gravity of the soil and the stabilizers. The low MDD may be attributed to the dispersed structure of the soil in the presence of the polar organic liquid, which consequently leads to reduced MDD (Meegoda et al.1998).

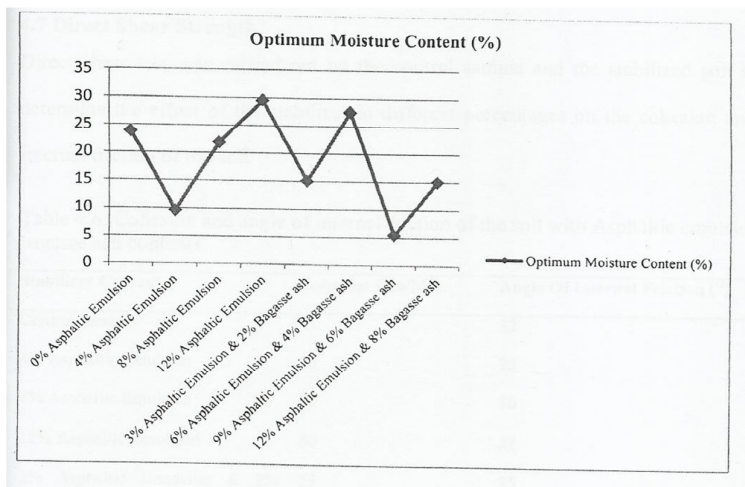
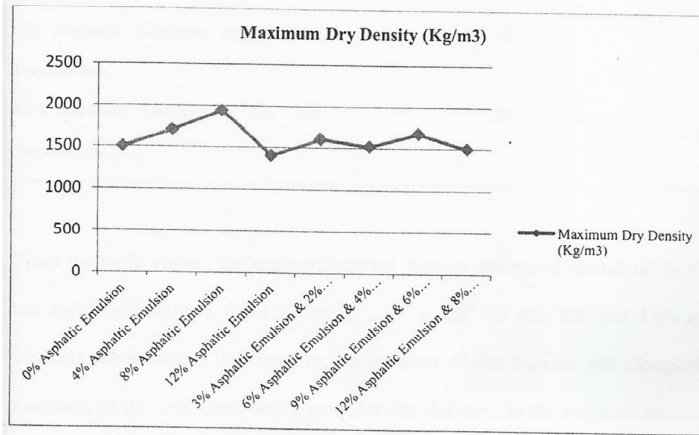


Figure 4.3: Variation of optimum moisture content of Asphaltic emulsion stabilization lateritic soil with bagasse ash.

Figure 4.3: Variations of optimum moisture content of Asphaltic emulsion stabilized lateritic soil with bagasse ash.



Direct Shear Strength

Direct shear test was carried out on the control sample and the stabilized soil samples to determine the effect

of the stabilizer in different percentages on the cohesion and angle of internal friction of the soil.

Table 4.6: Cohesion and angle of internal friction of the soil with Asphaltic emulsion and bagasse ash contents

Stabilizer Content	Cohesion (Kn/M ₃)	Angle Of Internal Friction (O)
Control Sample	41	32
4% Asphaltic Emulsion	63	32
8% Asphaltic Emulsion	69	30
12% Asphaltic Emulsion	86	28
3% Asphaltic Emulsion & 2% Bagasse Ash	53	35
6% Asphaltic Emulsion & 4% Bagasse Ash	180	25
9% Asphaltic Emulsion & 6% Bagasse Ash	210	18
12% Asphaltic Emulsion & 8% Bagasse Ash	190	29

From the table above, the angle of internal friction decreased considerably with increase of the asphaltic emulsion from 320 to 300, 300 to 280 for 4%, 8% and 12% of the emulsion contents respectively. Furthermore, on addition of the bagasse ash alongside the asphaltic emulsion to the soil, there was a considerable decrease in the angle of internal friction from 350 to 250, 250 to 180, and 180 to 290 for the variations of the asphaltic emulsion bagasse ash percentage mix.

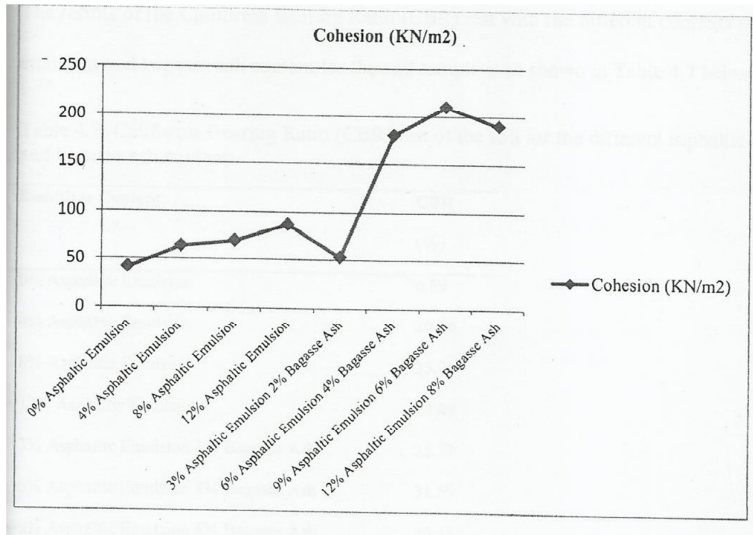


Figure 4:5 Variation of cohesion of asphaltic emulsion stabilized lateritic soil with bagasse ash

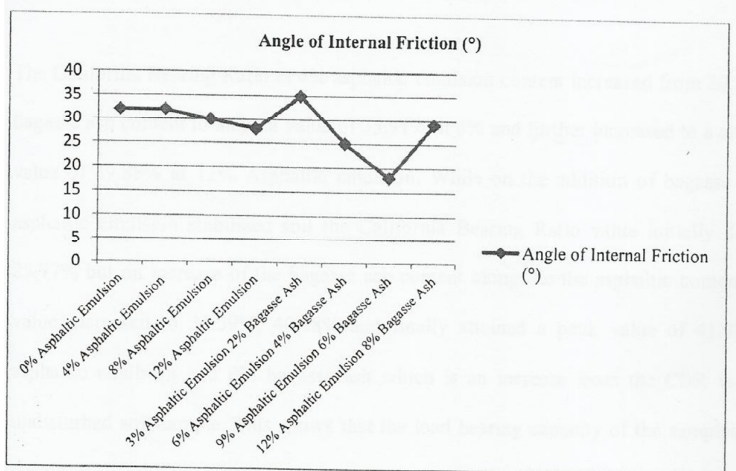


Figure 4:6: variation of angle of friction of asphaltic emulsion stabilized lateritic soil with bagasse ash

California Bearing Ratio Test

The results of the California Bearing Ratio (CBR) test with the different contents of asphaltic emulsion and

bagasse ash content for the soil sample is as shown in Table 4.7 below.

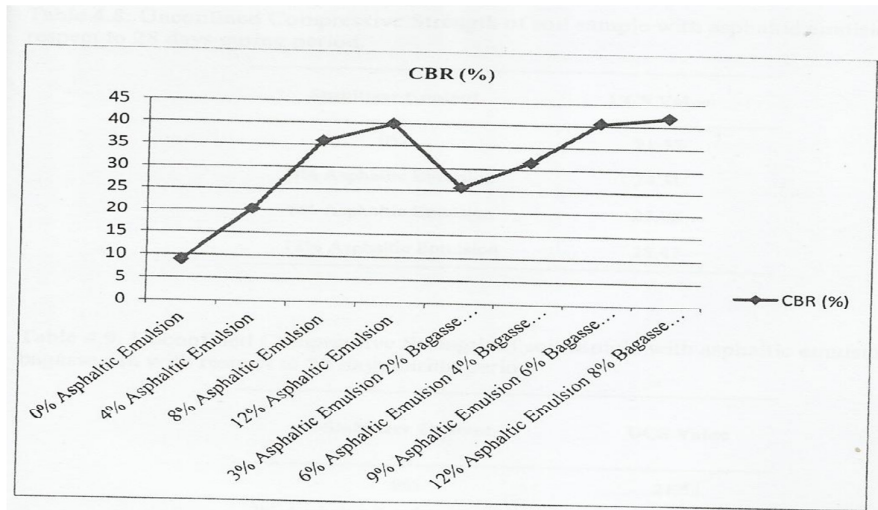
Table 4.7: California Bearing Ratio (CBR) test of the soil for the different asphaltic emulsion and bagasse ash content.

Stabilizer Content	CBR (%)
0% Asphaltic Emulsion	8.89
4% Asphaltic Emulsion	20.28
8% Asphaltic Emulsion	35.91
12% Asphaltic Emulsion	39.88
3% Asphaltic Emulsion 2% Bagasse Ash	25.77
6% Asphaltic Emulsion 4% Bagasse Ash	31.59
9% Asphaltic Emulsion 6% Bagasse Ash	40.48
12% Asphaltic Emulsion 8% Bagasse Ash	41.9

The California Bearing Ratio at 4% asphaltic emulsion content increased from 20.28% at 0% bagasse ash content to attain a value of 35.91% at 8% and further increased to attain a peak value of 39.88% at 12% Asphaltic emulsion. While on the addition of bagasse ash to the asphaltic emulsion stabilized soil the California Bearing Ratio value initially decreased to 25.77% but on increase of the bagasse ash content alongside the asphaltic content the CBR value increased to 31.59%, 40.48% and finally attained a peak value of 41.9% at 12% asphaltic emulsion and 8% bagasse ash which is an increase from the CBR value of the undisturbed soil sample. This shows that the load bearing capacity of the samples increased considerably with the bagasse ash.

For a base material that requires a CBR value of 100%, and for sub-base material requires a minimum

CBR value of 30% (Oluyemi B.D. 2006) which indicates that the soil sample can be used as sub-base material at 8% and 12% asphaltic emulsion stabilized soil but with bagasse ash, most of the percentage mixes can be used except for 3% asphaltic emulsion and 2% bagasse ash. The CBR value increased but with low percentage of increment to a maximum CBR value of 41.9% at 12% of the emulsion and 8% bagasse ash. This low CBR value is due to the presence of excessive amounts of clay which made mixing and compacting difficult. This phenomenon is in accordance with the work of Coyne and Ripple (1975); Oluyemi B.D. (2006) which shows that materials containing high amounts of clay are not suitable for treatment with asphalt due to difficulty in mixing and rolling and high amount of emulsions required to achieve water resistance.



Unconfined Compressive Strength

The variations of the unconfined compressive strength (UCS) of asphaltic emulsion stabilized

lateritic soil with bagasse ash content for 28 days curing period are shown in Table 4.8 below.

Table 4.8: Unconfined Compressive Strength of soil sample with asphaltic emulsion with respect to 28 days curing period.

Stabilizer Content	UCS Value
0%	21.53
4% Asphaltic Emulsion	24.31
8% Asphaltic Emulsion	27.08
12% Asphaltic Emulsion	28.47

Table 4.8: Unconfined Compressive Strength of soil sample with asphaltic emulsion with respect to 28 days curing period.

Stabilizer Content	UCS Value
0%	21.53
4% Asphaltic Emulsion	24.31
8% Asphaltic Emulsion	27.08
12% Asphaltic Emulsion	28.47

It is a special type of unconsolidated-undrained test that is commonly used for clay specimens where the confining pressure (σ_3) is 0 and the major principal stress (σ_1) is the unconfined compression strength (qu) (Amu. O. 2011). The sample

strength rose from 21.53 to 24.31KN/m², 27.08KN/m², 28.47KN/m² at 0%, 4%, 8%, 12% of asphaltic emulsion respectively this shows that more quality of emulsion would be required to stabilize this type of soil and hence would be

costly. But on addition of bagasse ash alongside asphaltic emulsion at different percentage mix the sample strength rose from 21.53KN/m² to 33.33KN/m², 44.44KN/m², 47.22KN/m², 43.06KN/m² for each percentage. It was observed by

Oluyemi B.D. (2006) that the results for the strength of all the samples that were tested indicates that age hardening of asphalt and bagasse ash increases the strength of the stabilized soil sample.

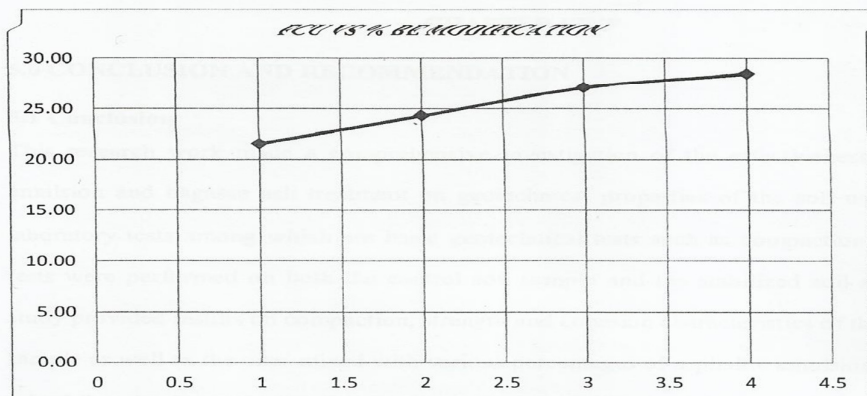


Figure 4.8: Variations of unconfined compressive strength of asphaltic emulsion stabilized lateritic soil (28-day curing period)

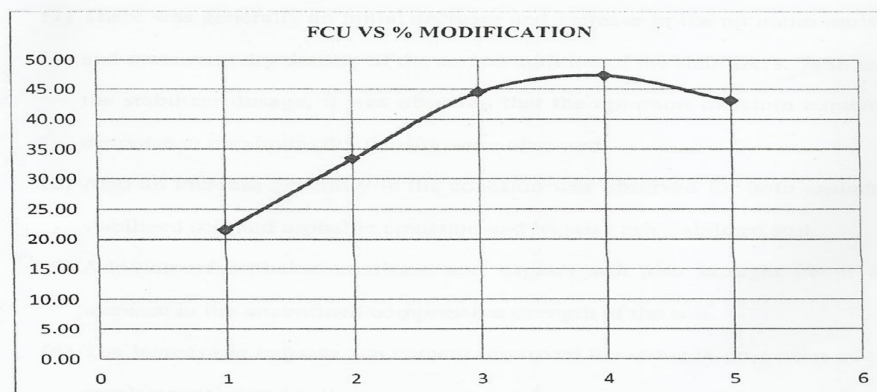


Figure 4.9: Variations of unconfined compressive strength of asphaltic emulsion stabilized lateritic soil with bagasse ash (28-day curing period).

RECOMMENDATION

From the result of this study, the following recommendations are made:

(1) Further study should be carried out on the stabilization of lateritic

soil using 9% asphaltic emulsion and 6% bagasse ash mix.

(2) Also further research works should be done on stabilizing of other soils using various admixtures such as rice husk ash, sugar cane straw ash, and palm kernel husk ash

and so on because they are all pozzolanic in nature.

(3) Proper geotechnical investigation should be carried out on any soil before it should be used as a construction material.

CONCLUSION

This research work made a comprehensive investigation of the effectiveness of asphaltic emulsion and bagasse ash treatment on geotechnical properties of the soil used. Different laboratory tests among which are basic geotechnical tests such as compaction and strength tests were performed on both the control soil sample and the stabilized soil samples. This study provided results on compaction, strength and cohesion characteristics of the natural soil sample as well as the ones mixed with various percentages of asphaltic emulsion and bagasse ash. After the investigation into the effects of asphaltic emulsion and bagasse ash on the properties of a reddish brown lateritic soil classified to be A-2-7(0) in the AASHTO rating the following conclusions were drawn:

(1) There was generally an initial decrease and increase in the optimum moisture content and maximum dry density of the soil on addition of the stabilizers. With an increase in the stabilizer dosage, it was observed that the optimum moisture content and also a decrease in maximum dry density were observed.

(2) Also an increase generally in the cohesion was observed for both asphaltic emulsion stabilized soil and asphaltic emulsion and bagasse ash stabilized soil.

(3) Addition of asphaltic emulsion and bagasse ash also brought about a significant increase in the unconfined compressive strength of the soil.

(4) The increase in bagasse ash content improved the strength properties of the asphaltic emulsion stabilized soil.

(5) Among all the variations of binder used, the 9% asphaltic emulsion and 6% bagasse ash produced the highest strength characteristics and was considered the most efficient.

Although asphaltic emulsion alone is a viable stabilization agent using bagasse ash as an admixture is more suitable for the improvement of the soil strength parameters.

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