BITUMINOUS STABILIZATION OF LATERITE MODIFIED WITH LIME FOR USE AS PAVEMENT MATERIALS

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MARCH, 2017

Declaration

I declare that the work in this project dissertation entitled **"Bituminous Stabilization of Laterites Modified with Lime for use as Pavement Material"** has been carried out by me in the Department of Civil Engineering. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this project dissertation was previously presented for another degree or diploma at this or any other institution.

_____ _____

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Certification

Thisproject dissertationentitled **"Bituminous Stabilizationof Laterites Modified with Limefor use as Pavement Material"**by **CyrilChibuzoOkeh**meets the regulation governing the award of the degree of Master of Science (Civil Engineering) of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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Dedication

This research work is dedicated to God Almighty who gave me the strength to complete this work. This research work is also dedicated to my Father, Celestine ChibuezeOkeh and to my late Mother, Theresa UzoamakaOkeh.

Acknowledgements

All praises be to almighty God who made it possible for me to accomplish the work.

My utmost gratitude goes to my parents who made sure I finished this work in this difficult time, may God fill back their pockets million times more. I wish to also acknowledge the support and guidance I got from my parents, brothers and sister for their prayers, advice and encouragement.

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Abstract

Marginal laterites A-7-6(14), improved with hydrated lime was stabilized using bituminous emulsion with a view to improving the strength properties of the soil. The preliminary investigation conducted on the natural lateritic soil showed that the soil belongs to the CH group using the Unified Soil Classification System or A-7-6(14) soil group using the AASHTO soil classification system. The MDD decreased with higher additive blend as it decreased from 1.67 to 1.49 mg/m^3 at 6 % lime with 6 % bitumen emulsion treatment. The OMC, on the other hand, decreased with higher bitumen content but increases with higher lime content. The decreases may be due to the effect of breakdown of the flocculated aggregates and elimination of large pores. The UCS values showed a sharp increase from 235 to 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion with 6 % lime, 6 % bitumen emulsion with 8 % lime and 6 % bitumen emulsion with 10 % lime respectively. The UCS values obtained fell short of 1710 kN/m^2 specified by TRRL as a criterion for adequate stabilization using OPC. The values of 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion with 6 % lime, 6 % bitumen emulsion with 8 % lime and 6 % bitumen emulsion with 10 % lime respectively on the other hand meet the requirement of 687–1373 kN/m^2. The CBR value increased with higher additive content to a peak value of 47 % at 8 % bitumen emulsion with 10 % lime. The low value of CBR recorded is as a result of the ingress of water into the specimen which weakened it and reduces its strength. The CBR value obtained does not meet the 80 % value for base material but meets the 30 % requirement for sub base materials. The resistance to loss in strength increase from 14.5 for the natural soil to a peak value of 61.25 % at 6 % bitumen emulsion with 2 % lime content. The peak resistance to loss in strength value of 61.25 % fell short of the acceptable 80 %. Statistical analysis was carried out on results obtained from the tests conducted to determine significance difference on the various soil-lime-bitumen mixtures using a two way Analysis of Variance (ANOVA) with the Microsoft Excel Analysis Tool Pak Software Package.The results also showed that the stabilized soil did not meet the requirements of the Nigerian General Specifications of not more than 35% passing sieve No. 200,maximum plasticity index (PI) of 30% and liquid limit (LL) of a maximum of 50% when used as a subgrade material in road construction. However, from the test results, laterite treated with 6% bitumen emulsion plus 6% lime is recommended for use as sub-base treatment for light traffic roads

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CHAPTER ONE

INTRODUCTION

1.1 Preamble

Developments in the country have awakened the sense of economical resource management in the populace. People are being inspired to go back and take a closer look at the resources which they have earlier condemned, so as to find ways through which they could utilize these materials. This is due to increased competition for available materials as multiple uses of such resources are being discovered, and the increasing cost of acquiring these suitable materials increases. Research work is on-going, with the aim of putting waste materials into full use. Soil is a very important component in the construction of road pavements. When a section of a road is to be filled with soil, the material is either obtained from other cut sections along the road or from a borrow site where the suitable material is present, if the material within the vicinity of the road work does not meet the required standards. The major cost incurred by "importing" soil material is that of hauling the material from the borrow site to the construction site. This cost, in terms of financial resources, and time,could however be avoided by simply improving the properties of the non-suitable material that is readily available, that will not require long haulage.

This soil can be improved upon through stabilization or modification when some of the following stabilizing agentsor additives such as Portland cement, quicklime/hydrated lime, fly ash, bitumen, calcium chloride are added. Bitumen emulsion is a fine dispersion of one fluid (in the form of minute droplets) in another liquid in which it is notsoluble.Bitumen emulsions which will be considered in this research work is used in cold laid plant mixes and road mixes (mixed in place) for several purposes including the construction of highway pavement surfaces and bases

and in surface treatment thereby, making the soil stronger and resistance to water and frost (Guyer, 2011).

The advantage to be derived from usage of bitumen emulsion in soil stabilization is that it adds cohesive strength to soil. The bituminous material is admixed in order to waterproof the soil and maintain its existing strength. Here emphasis is on impeding the entry of water by adding sufficient bituminous material to wrap soil particles or agglomerates of particles in thin bituminous films or membranes, and plug the soil-void channels (Williams, 1968).

Lime helps in the reduction of soil plasticity (by reducing the soil water content, it becomes more rigid). It also increases the strength and workability of the soil and reduces the soil ability to swell. Lime is used as a soil treatment for a variety of reasons, e.g. to expedite construction on weak clay subgrades, or to improve the engineering properties of plastic sands, plastic gravels, and reactive clays. Whilst lime is mainly used in sub base and subgrade capping layers in pavements (Ola, 1977)

1.2 Statement of the Problem

Laterites found in different locations are not normally suitable for use as road construction/building material in their natural state, hence the need for treatment to make them fit for use in road work.Such treatment may not be achieved with the singular use of conventional stabilizing agent hence the need for a modifier, and because of the effect of climate change that may give rise to more rainfall, it is good to use bitumen, instead of the conventional stabilizing agent which have less water proofing potential.

1.3 Aim and Objectives of the study

1.3.1 Aim

The aim of the study is to ascertain the suitability of laterite treated with bitumen emulsion and modified with lime for use as pavement materials.

1.3.2 Objectives

i. To determine the effect of bitumen emulsion on the Atterberg's limits value of laterite modified with lime
ii. To determine the compaction characteristics of lime modified laterite treated with bitumen emulsion.
iii. To determine the strength indices of lime modified laterite treated with bitumen emulsion.
iv. To determine the suitability of lime modified laterite treated with bitumen emulsion for use in road work, using the different strength indices.

1.4 Justification for the Study

The efficacy of lime and portland cement in stabilization of soils has been highlighted by some researchers (Umar and Osinubi, 2003; Umar and Elinwa, 2005), and hydrated lime and quicklime have been found to be most effective agents in reducing the swelling properties of soils. This study attempts to improve the strength of lateritic soil using bitumen emulsion and lime. Bitumen emulsion as a binder substance waterproofs, impart cohesion and stiffness on the soil while lime, when mixed/added in moderate amount to the soil, increasethe bearing capacity and reduce shrinkage on soils with high plasticity index. The study is different from others because past researchers have used only lime or bitumen to improve soil strength.

1.5 Scope of Work

The research was limited to the use of bitumen emulsion and lime in the stabilization of laterite for use as road pavement material. The optimum percentages from (0,2,4,6,8 and 10%) of the bitumen emulsion and lime for the stabilization of the laterite soil was determined,in line with the Nigerian General Specification (1997) as well as AASHTO (1986) and ASTM (1992) requirements for transportation materials.

CHAPTER TWO

LITERATURE REVIEW

2.1 Laterite Formation and Problems associated with the use of Laterite in Road Work.

The term Laterite is derived from the Latin word ―later, meaning brick. It was first used in 1807 by Buchanan to describe a red iron-rich material found in the southern parts of India. Laterites are widely distributed throughout the world in the regions with high rainfall, but especially in the inter-tropical regions of Africa, Australia, India, South-East Asia and South America, where they generally occur just below the surface of grasslands or forest clearings. Their extension indicates that conditions were favorable for their formation at some point in time in the history of the world, but not necessarily simultaneously in all regions (Lemougna*et al*., 2011).

Laterites contribute to the general economy of the regions where they are found. Their scope is very wide and includes civil engineering, agronomic, mining research (iron, aluminum and manganese) deposits. There is no need to emphasize the importance oflaterites for various construction purposes. Laterite crusts were originally widely used for the construction of monuments and dwellings. Certain African megaliths like ―Tazunu‖ , located in the northwest of the Central African Republic, are of lateritic origin, in addition to rock minerals(Lemougna *et al*., 2011).

The use of indurated laterites as a building material has been, and is still very common in Africa.Civil engineeringstudies of these materials are now in progress, with focus on their use in road and earth dam construction.(Lemougna*et al.,* 2011).

Lateritic soils refer to highly weathered soils formed from materials with lower concentrations of oxides or hydroxides of iron and aluminum(Amu *et al.,* 2011).

Laterites are soil types rich in iron oxides and aluminum oxides, formed in hot and wet tropical areas. Nearly all laterites are rusty-red because of iron oxides. They develop by intensive and long-lasting weathering of the underlying parent rock. Tropical weathering (laterization) is a prolonged process of chemicalweathering which produces a wide variety in the thickness, grade, chemistry and ore mineralogy of the resulting soils. The majority of the land area containing laterites is between the tropics of Cancer and Capricorn (Ogunribido, 2012).

Laterites are a source of aluminumore; the ore exists largely in clay minerals and the hydroxides, gibbsite, boehmite, and diaspore, which resembles the composition of bauxite. In Northern Ireland they once provided a major source of iron and aluminum ores. Laterite ores also were the early major source of nickel (Ogunribido, 2012).

Laterization is the process whereby rocks are converted to laterite by enrichment of iron oxide and depletion of silica. Laterization is favored by a warm climate with alternating wet and dry seasons (Ushie and Anike, 2011).

Laterites are formed from the leaching of parent sedimentary rocks (sandstones, clays, lime stones); metamorphic rocks (schists, gneisses, migmatites); igneous rocks (granites, basalts, gabbros, peridotites); and mineralized proto-ores;which leaves the more insoluble ions, predominantly iron oxide and aluminum oxide. The mechanism of leaching involves acid dissolving the host minerallattice, followed by hydrolysis and precipitation of insoluble oxides and sulfates of iron, aluminum and silica under the high temperature conditions of a humid sub-tropical monsoonclimate (Hill *et al.*, 2000).

An essential feature for the formation of laterite is the repetition of wet and dry seasons. Rocks are leached by percolating rain water during the wet season; the resulting solution containing the leached ions is brought to the surface by capillary

action during the dry season. These ions form soluble salt compounds which dry on the surface; these salts are washed away during the next wet season. Laterite formation is favoured in low topographical reliefs of gentle crests and plateaus which prevents erosion of the surface cover. The reaction zone where rocks are in contact with water from the lowest to highest water table levels, is progressively depleted of the easily leached ions of sodium, potassium, calcium and magnesium. A solution of these ions can have the correct pH to preferentially dissolve silicon oxide rather than the aluminum oxides and iron oxides (Yamaguchi, 2004).

2.1.1 Structure, composition and properties of lateritic soils

Laterites vary greatly in structure, but can be reduced to the following three structural patterns:

(a) The indurated elements form a continuous, coherent skeleton;

(b) The indurated elements are free concretions or nodules in an earthy matrix;

(c) The indurated elements cement pre-existing materials.

These structural patterns exhibit great variability in relation to the shape and size of the elements involved and the degree of induration. The degree of hardness ranges from products that are practically unconsolidated and scarcely coherent to the hardest blocks which can be broken only with a hammer. Induration is an empirical criterion, as it is impossible to give quantitative expressions to any character related to the mechanical properties of the material. The usual definition of induration is a state in which the hard brittle consistency of the medium is not affected by humidity. Induration, which involves the precipitation of goethite in a reticular network, is influenced by composition and the extent of crystallization of the components in the soil: the higher the sesquioxide content, the greater the induration. In other words,

hardness increases as the iron content increases; the hardest laterites are also the least hydrated (Amu*et al.,* 2011).

Laterites vary in color, but are usually brightly colored. The shades most frequently encountered are pink, ochre, red and brown; however, some occurrences are mottled and streaked with violet, and others exhibit green marbling. A single sample may exhibit a whole range of colors merging more or less perceptibly into one another in a variety of patterns and forms. Laterites owe their color to iron oxides in various states of hydration and sometimes also to manganese. Their mineralogy generally involves quartz, kaolinite, hematite, goethite, and sometime maghemite. Kaolinite is always present with iron oxides. The physical properties of lateritic soil vary according to the mineralogical composition and particle size distribution of the soil. The granulometry can vary from very fine to gravel according to its origin, thus influencing geotechnical properties such as plasticity and compressive strength. One of the main advantages of lateritic material is that it does not readily swell with water. This makes it an excellent packing material particularly when it is not too sandy (Lemougna*et al.,* 2011).

The problems associated with the use of laterite is that laterites did not conform with any accepted specifications but performed equally well when compared with adjoining sections of road using stone or other stabilized material as a base. Foundation problem (i.e. if a thin laterite layer overlies a soft material), the presence of warm holes in laterite need to be examined carefully in the continuous softening effect with depth.Laterite shows variability in properties depending on the stage of weathering (Shroff*et al.,* 2003).

Unfortunately, laterites have not been used to their fullest extent in the upper (base and sub-base) layers of low volume paved roads (LVPRs) in the African region for a number of reasons including:

I. The variability in their engineering properties and their failure to meet traditional specifications. For example, these materials commonly exhibit gaps in the grading curve (e.g. in the sand coarse fraction); high plasticity indices (PIs 15-20) and soaked CBR values lower than the minimum of 80 per cent normally specified.

II. Lack of awareness of the more appropriate specifications that were first developed by the Portuguese in the 1950s and 1960s in countries such as Angola and Mozambique and subsequently adapted for use in other countries, notably Brazil and Australia

In view of the above, the use of neat (untreated) laterites for the construction of low volume sealed roads (LVSRs) in some African countries has been limited as the road authorities continue to use much tighter, restrictive standards that greatly suppress the use of this type of material. As a result, other more expensive options are adopted such as hauling over long distances other natural gravels which meet the traditional specifications; stabilizing laterites with cement and lime or using crushed stone for the base (Ugbe, 2011).

Soil stabilization is a means of modifying physical and chemical characteristics of a given soil with a view to improve its engineering properties. High water content and low workability of some soil used as fill materials pose difficulty for construction project. Most of the time, additives such as rock flour, cement, lime, asphalt and polymer stabilizers are used to improve their engineering properties. The choice and

the effectiveness of an additive depend on the type of soil and its field conditions (Balasigam and Farid, 2008).

Bajeh (1994), reported that engineering properties of soil can be modified by the use of chemical stabilizers. In the recent past, cement treatment were widely used for the improvement of the mechanical properties of soils, but due to the high cost of Portland cement, other low cost additives such as rock flour, lime, rice husk are now been considered and their effectiveness has to be determined. Lateritic soil, are often used as imported fill material for the prepared sub grade in many road projects (Okunnade, 2010, Amadi, 2010.)

Stabilization of soil can be justified when employed because it is more economical to overcome a deficiency in a readily available material than to bring in one that fully complies with the requirements of specification for the soil. It has been regarded as a last resort for upgrading substandard materials where no economic alternativeis available. A continual reference to economy here denotes a careful consideration of all costs that would be incurred by importation (not readily available) of a compliant soil and comparing this to the cost of improving the properties of an unstable but readily available soil. Interest in the art of soil stabilization grew with a better appreciation of the cyclic loading effects of heavy traffic which creates a need for stronger pavements that often cannot be provided by realistic thickness of unbound granular materials, and the availability of purpose built in-situ stabilization equipment that improves homogeneity of mix. Although road construction has been the major area of application of soil stabilization techniques, they have also been applied in soil foundation strengthening, although to a limited extent (Amu *et al.*, 2011).

2.2 Soil Stabilization.

Stabilization processes are very complex because many parameters come into play. The knowledge of soil properties can help to better consider what changes, the economic studies (cost and time), as well as production and construction techniques to use. The simplest process consists of taking soil and drying it in open-air. It is the pisel technique, rammed earth, adobe, and brick dried in the sun, widely used in the majority of African countries. More elaborate processes can include heat treatment, or mixing soil with ordinary Portland cement, lime, etc. The different methods of soil stabilization as reported by Lemougna*et al.* (2011) are as follows;

2.2.1 Mechanical stabilization

Mechanical stabilization consists of compacting the soil to affect its resistance, compressibility, permeability and porosity. The soil is mechanically treated so that maximum air can be eliminated and this contributes to an increase in its density. With mechanical stabilization, the particle size distribution constituting the material is not affected, but its structure is changed because the particles are redistributed. Mechanical stabilization is widely used in road construction and requires a prior analysis of the soil to determine the optimum water content for better soil compressibility (lemougna*et al.*, 2011).

Fine grained cohesive soils are compacted in the field by using common compaction equipment like hand–operated tampers, sheep's foot rollers, rubber–tyred rollers and other types of specialty equipment. Considerable compaction can also be obtained by proper routing of the hauling equipment over the loose soil. The objective of mechanical compaction is the improvement of the engineering properties of the soil mass (O' Flaherty, 2002).

The several advantages obtained through compaction are as follows;

i. Reduction in settlement due to reduced void ratio

ii. Increase in soil strength

iii. Reduction in shrinkage.

2.2.2 Physical stabilization

Physical stabilization consists of modifying the properties of soil by intervening with its texture (granulometry treatment, heat (dehydration or freezing) or electric (electrosmosis) treatments that lead to the drainage of the soil and thus confer new structural properties to it). Physical stabilization may also involve the introduction of synthetic fibers or fibers originating from plants, animals and minerals into the soil. This method is used when there are reasons not to affect the particle size distribution of the soil or if the material is sensitive to movements induced by factors such as water action, thermal expansion, etc. These movements can then be countered by a frame made of fibers. The armature acts at a macroscopic level (on grain aggregation), and not at the level of individual grains (lemougna*et al.*, 2011).

2.2.3 Chemical stabilization

Chemical stabilization consists of adding other materials to the soil or chemicals that alter its properties, either by a physico-chemical reaction between particles and the added materials or by creating a matrix that binds or coats the particles. The physico-chemical reaction can cause the formation of a new material made from a pozzolanic reaction between clay and limestone, for example.The most often used additives for soil stabilization are cement, lime or a cement/lime mixture(lemougna*et al.*, 2011).

2.2.3.1 Lime stabilization:

The two primary types of lime used in construction today are quicklime (calcium oxide) and hydrated lime (calcium hydroxide). Heating limestone at elevated

temperatures produces quicklime and the addition of water to quicklime produces hydrated lime.

The use of lime for stabilization dates back to early times. Its principal use is in highway construction and maintenance, airfields construction, building foundation, rail round beds and under hydraulic conditions where the soil is partly or wholly submerged (e.g. irrigation canals, reservoirs, levees and dams). Lime stabilization has been used successfully on many projects to minimize swelling and improve soil plasticity and workability. Generally, from 3 to 8% by weight of hydrated lime is added to the top several inches of the soil. It is also used as a follow-up treatment over ponded area to add strength to the surface, provide a working table for equipment and prevent evaporation by capillary rise from the treated layer below (Bell,1966)

Lime has been considered to be the most appropriate for the stabilization of highly clayed soils having fine contents in excess of 25% because it makes the soil more friable, less plastic and hence easier to work. The reactions of lime with soils result in strength gain mainly from chemical reactions between the lime, clay-grade minerals and amorphous constituents in the soil.

Improvement in the soil properties for soil stabilization has been attributed to the soil–lime reaction (Clare and Cruchley, 1960). The cation exchange, the pozzolanic reactions and carbonation are the mechanisms used in explaining the chemical changes occurring in the soil properties for soil–lime mixes (O'Flaherty, 2002). Cation exchange is believed to be mainly responsible for the change in plasticity of the soil, and magnitude of this change is affected by the soil clay mineralogy. Montmorillonite soils show the greater change, the effect in illite-chlorite is intermediate and effect in kaolinite soil is less (Ola, 1977).

When introducing lime into soil for stabilization, $Ca^{2+}$is partly absorbed on the surface of clay particles in replacement of monovalent cations such as Na^{+} and K^{+}.Theamount of cations is no longer availableforpozzolanic reactions. The amount of lime Ca^{2+}absorbed depends on the cation exchange capacity of the treated soil. The more the exchange capacity, the higher the amount of Ca^{2+}absorbed by the soil. All the absorbed Ca^{2+}required to satisfy the affinity of soil for lime is called the Lime Fixation Point (LFP). The lime in excess of the LFP is involved in the process of cementing. The reactions between the lime, silica and alumina-free, contributing to the formation of new minerals such as CSH (calcium silicate hydrates), CAH (calcium aluminate hydrates) and CASH (calcium aluminosilicate hydrates), are primarily responsible for the consolidation. These reactions are more or less slow and depend on various factors such as mineralogy, grain sizes of the treated soil, as well as curing time and temperature (Lemougna*et al.*, 2011)

mechanism of lime stabilization: When lime is admixed with a moist clay soil, some reactions take placesuch as: (1) cation exchange (2) pozzolanic reaction (3) carbonation.

Cation exchange: takes place immediately and causes the individual clay particles to change from a state of mutual repulsion to a state of mutual attraction, typically due to excess Ca^{2+} replacing dissimilar cations from the exchange complex of the soil. This has the immediate positive effect of promoting flocculation of the particles and a change in soil texture, i.e. with clay particles 'clumping' together into larger-sized aggregates or lumps – thereby improving the soil's gradation, permeability and handling properties(O'Flaherty, 2002).

***Pozzolanic reaction*:** the amount of lime added to a clay soilmustbe in excess of that required by the soil for cation exchange and flocculation/aggregation. The pozzolanic

reaction, which is affected by the clay content, the type of clay mineral, and the curing temperature, occurs more slowly than the cation exchange reaction, and results in the formation of cementitious products that have long-term effects on the strength, volume stability, and (in colder climates) resistance to frost action of the stabilized soil (O'Flaherty, 2002).

Carbonation: the carbonation effect means thatlime needs to be protected whilst in storage and in shipment prior to field use, andthat lime stabilization is best carried out in non-industrial locales. Long intensivemixing and processing during construction should also be avoided if high strengthgain is a primary objective, and compaction should occur as soon as possible after the addition of lime(O'Flaherty, 2002).

2.2.3.2 Cement stabilization:

Stabilization of soils with cement usually involves the mixing of predetermined quantities of the additive with pulverized soil particles; and the mixture is termed 'soil-cement'. Soil-cement mixtures have been used for road sub base or base course. Stabilization of soil with ordinary Portland cement (OPC) produces hardened materials which are capable of bearing loads for engineering purposes. TRRL (1977) recommended a minimum of 15% for soil fraction passing 0.425mm sieve and plasticity index greater or equal to 10. Generally, gravels require about 10% by dry weight of cement, sand requires about 7–10%, silt about 12 to 15% and clays, 12 to 20% by dry weight of cement (Gillot, 1987).

Hydration is the first phase of the cement stabilization reaction. Cement hydration produces a cementitious compound; a product of cement and water. It generates bonding between the reaction products (Calcium–silicate hydrates and aluminium hydrates) and the soil particles. It results in agglomeration and flocculation of the clay particles due to exchange of ions at the surface of the particles which

manifest in an early strength development, immediate swell, shrinkage and plasticity reduction

In highway construction, Portland cement is normally added to soils at optimum moisture content (OMC) and allowed to cure while the soil – cementmixture hardens by process of hydration. Yoder and Wictzak (1975), Nelson and Miller (1992), Indraratna et al., (1995) reported decrease in liquid limit, plastic limit and swelling potentials of clays when treated with cement. However, researchers such as Ingles and Metcalf (1972) and Ibrahim (1983) reported that cement stabilization may not be effective for soils of high plastic limit (of about 20%) or liquid limit in excess of about 45 to 50% and high organic matter as well as montmorillonite–rich soil.

Kedzi (1979) identified two types of reaction for soil-cement mixtures. The first being the hydration of cement while the second is the reaction between the free lime product of hydration of cement and alumina of the clay fraction of the soil. The primary reaction results in the formation of calcium silicate hydrates (CSH), which are cementing substances and are responsible for the initial strength development, while the secondary reaction is pozzolanic in nature, and it is responsible for the time dependent gain in strength. The amount of reduction of maximum dry density (MDD) is dependent on the rate of hydration which is expected to decrease with increase in both time and cement content. O'Glesby and Hicks (1982) attributed the effects of OPC in the strength properties of soil as (i) the surface action of cement quickly produces flocculation and reduces the moisture affinity of clay soils; and (ii) the reaction which is time dependent, promotes the cementation of the soil cement mix. When OPC hydrates, it liberates calcium silicates and aluminum ions into the water and these subsequently combine to form hydrated calcium silicate, which constitute the matrix of the hardened cements (Neville and Brooks, 1994).

2.2.4 Bitumen stabilization

Penetration grade bitumen*:* bitumen used for road construction is viscous liquid or semi-solid materials, consisting essentially of hydrocarbons and their derivatives, which are soluble in trichloroethylene. Whilst bitumen occurs naturally, the predominant majority of those used in road works are the penetration-grade products of the fractional distillation of petroleum at refineries. Penetration grade bitumen are primarily designated by the number of 0.1mm units that a special needle penetrates the bitumen under standard loading conditions, with lower penetration depths being associated with harder bitumen (O'Flaherty, 2002).

Cutback bitumen*:* they are slow curing bitumen, medium curing bitumen and rapid-curing bitumen. They are used mainly in cold laid plant mixes, road mixes (mixed-in-place), and as surface treatment (O'Flaherty, 2002).

Bitumen emulsion:they are produced by breaking asphalt cement, usually of 100 to 250 penetration range, into minute particles and dispersing them in water with an emulsifier. These minute particles have like electrical charges and therefore do not coalesce. They remain in suspension in the liquid phase as long as the water does not evaporate or the emulsifier does not break. Bitumen emulsion consist of asphalt, which makes up about 55 to 70 percent by weight, water, and an emulsifying agent, which in some cases also may contain a stabilizer (O'Flaherty, 2002).

Soil stabilization with bitumen can be done with either of two additives. Depending upon project conditions, the choice is made between using an asphalt emulsion or using foamed bitumen as the additive. For example, extremely wet soil conditions might dictate the use of foamed bitumen rather than a standard asphalt emulsion to compensate for the high field moisture content. Emulsion might be chosen for projects where high performance emulsions are readily available. Therefore, cost

and product availability are practical considerations for stabilization with Bitumen, just as with other construction activities.

It is not uncommon for certain materials or project conditions to require that more than a single additive be utilized to achieve the required strength and stability for the new base design. For example, when using foamed bitumen, it is often the practice to also utilize cement or lime to help increase the compressive strength of the base material, or to assist in drying out materials that have field moisture contents greatly exceeding optimum moisture content (OMC).

When more than a single additive is selected, the first additive to be applied may be added in the dry state or as slurry. When added in the bulk state, it is not uncommon for the agent to be spread unto the surface of the material and then either bladed, disked, scarified or otherwise distributed prior to final stabilization. As slurry, it may be applied to a truck distributor, or mixed directly into the soil through an additive system incorporated into the stabilizers mixing chamber. There are also many projects where the in-situ material field moisture content is not sufficient for proper activation of the chemical reaction of the cement or lime. For these projects, it is necessary to wet the treated soil prior to mixing. Dry spreading of cement or lime in arid regions is often discouraged due to the creation of dust during the process (Kowalski and Starry, 2007).

Mechanism of bitumen emulsion stabilization: The chemical and physical influence of the soil/aggregate with which the emulsion comes in contact is thought to have only a limited effect on the rate of breakdown of anionic emulsions. However, with cationic emulsions the influence of the soil aggregate is of greater importance. When brought into contact with the surface of the soil/aggregate, the positively charged particles of the binder attach itself to the soil/aggregate surface which usually carries a weak negative charged. Although the bond formed in this manner between the soil/aggregate

and the binder is immediate, the deposition of the negatively charged emulsion particles will rapidly neutralize the negative charge on the on the soil/aggregate surface. This inhibits further deposition of binder from the emulsion and the unbroken emulsion remaining (of higher water content than the original emulsion) must depend on the process of evaporation (Osinubi, 2001).

However, as evaporation continues the particles of the binder becomes more closely packed and two features occur. First is the waterproofing effect the binder has on the soil thereby protecting existing strength from the effect of moisture changes. Waterproofing preserves the natural shear strength which the soil has in a dry and compacted condition. Secondly the binder acts as a cementing agent for the soil particles on the basis of adhesion between the binder and soil particles, when the binder exceeds the optimum, the film coatings about the particles become too thick, then contact between the particles are prevented and the strength is lowered. The two features described above generally occur together with either predominating depending on the nature of the soil(Osinubi, 2001).

2.2.5 Modification and conventional stabilizing agents

The process of reducing plasticity and improving the texture of a soil is called soil modification. Monovalent cations such as sodium andpotassium are commonly found in expansive clay soiland these cations can be exchanged with cations of higher valences, such as calcium, which are found in lime, fly ash and Portland cement. This ion exchange process takes place quite rapidly, often within a few hours. The calcium cations replace the sodium cations around the clay particles decreasing the size of the bound water layer and enabling the clay particle to flocculate.

The flocculation creates a reduction in plasticity, an increase in shear strength of the clay soil andan improvement in texture from a cohesive material to more granular,

sand - like soil. The change in the structure causes a decrease in the moisture sensitivity and increases the workability and constructability of the soil. Soil stabilization includes all the effects of modification with an additional long - term strength gain. Soil conditions and mineralogical properties have a greater role for soil stabilization than modification. The magnitude of soil stabilization is usually measured by the increase in strength as determined from unconfined compression testing (Milburn and Parsons, 2004).

The stabilizing agents or additives generally accepted following their principles method and behavior are explained below

***Lime*:** The two primary types of lime used in construction today are quicklime (calcium oxide) and hydrated lime (calcium hydroxide). Heating limestone at elevated temperatures produces quicklime and the addition of water to quicklime produces hydrated lime. Equation (2.1) shows the reaction that occurs when limestone is heated to produce quicklime with carbon dioxide produced as a by - product.

$$CaCO_3 + Heat \rightarrow CaO + CO_2 \quad (2.1)$$

Equation (2.2) shows that the addition of water to quicklime produces hydrated lime along with heat as a by - product.

$$CaO + H_2O \rightarrow Ca(OH)_2 + Heat \quad (2.2)$$

Hydrated lime has a higher atomic weight of 74.1, compared to 56.1 for quicklime, because it has one additional oxygen and two additional hydrogen atoms. Since hydrated lime has a higher atomic weight than quicklime, 30% more hydrated lime is required to introduce the same amount of calcium.For soil stabilization with lime, soil conditions and mineralogical properties have a significant effect on the long - term strength gain. Introduction of calcium hydroxide increases the pH, causing the silica and alumina in the clay particles to become soluble and interact with the calcium in a

pozzolanic reaction. A pozzolanic reaction between silica or alumina in the clay particles and calcium from the lime can form a cemented structure that increases the strength ofthe stabilized soil. Residual calcium must remain in the system to combine with the availablesilica or alumina and to keep the pH high enough to maintain the pozzolanic reaction.

The treatment of pavement subgrades with lime can significantly improve the engineering properties of a wide range of soils. There are many recommendations for evaluations of soils for lime treatment. For examples, soils that should be considered for lime treatment include soils with a PI that exceeds 10 and have more than 25 percent passing the #200 sieve(Milburn and Parsons, 2004).

***Portland Cement*:** Portland cement is a multimineralic compound made up of oxides of calcium, silica, alumina andiron. When cement is mixed with water, cementing compounds of calcium - silicate- hydrate (C- S- H) and calcium - aluminate- hydrate (C - A- H) are formed and excess calcium hydroxide is released. Some calcium is therefore available to react with the clay particle early in the modification process when the water is added, and additional calcium becomes available later as it forms during cement hydration. The total amount of calcium hydroxide generated from cement is approximately 31% by weight. The hydrates help to stabilize flocculated clay particles through cementation. The hydration reactions and strength increasesoccur for the most part between 24 hours and 28 days, although the cement will continue to hydrate at decreasing rates as long as free moisture is present. Pozzolanic reactions also occur from interaction between the calcium hydroxide and the clay minerals due to the alkalineenvironment, which dramatically increases the solubility of silica and alumina in clay minerals (Milburn and Parsons, 2004).

Portland cement can be used either to modify and improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability. Cement can be used effectively as a stabilizer for a wide range of materials; the soil should have a PI less than 30. For coarse-grained soils, the amount passing the No. 4 sieve should be greater than 45 percent. The amount of cement used depends on whether the soil is to be modified or stabilized Portland cement can be used either to modify or improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability (Milburn and Parsons, 2004).

The main reaction in a soil/cement mixture comes from the hydration of the two anhydrous calcium silicates (3CaO. SiO_2 (C_3S) and 2CaO. SiO_2 (C_2S)), the major constituents of cement, which form two new compounds: calcium hydroxide (hydrated lime called portlandite) and calcium silicate hydrate CSH), the main binder of concrete. The reaction is presented as equation(2.3).

$$\text{cement} + H_20 \rightarrow CSH + Ca\,(OH)_2 \qquad (2.3)$$

Unlike lime, the mineralogy and granulometry of cement treated soils have little influence on the reaction since the cement powder contains in itself everything it needs to react and form cementitiousproducts. Cement will create physical links between particles, increasing the soil strength; meanwhile lime needs silica and alumina from clay particles to develop pozzolanic reactions. Generally, the hydration reactions of cements are faster than those of lime, but in both cases, the final strengthresultsfromthe formation of CSH. Other materials such as gypsum, asphalt and bitumen can be used also (Lemougna*et al.*, 2011).

***Class C fly-ash*:** Class C fly ash is an industrial by product generated at coal - fired electricity generating powerplants that contains silica, alumina, and calcium - based minerals. Upon exposure to water, these calcium compounds hydrate and produce

cementitious products similar to the products formed during the hydration of Portland cement. Free lime that is present or generated as a part of these reactions may also react with available pozzolans. The rate of hydration for fly ash is much more rapid than for Portland cement. It is, therefore, desirable to mix and compact fly ash as quickly as is practical, and a maximum delay time between mixing and compaction is often included in compaction specifications (Milburn and Parsons, 2004).

The hydration properties of fly ash are dependent on a number of factors including the coal source, boiler design and the type of ash collection system. The coal source governs the amount and type of inorganic matter present in the coal, thereby dictating the composition of the fly ash. Coals sources from eastern Nigeria have low calcium contents and the fly ash that is produced from these coals contains only a small amount of calcium. This fly ash does not exhibit self-cementing characteristics, however the addition of lime causes apozzolanic reaction producing the cementitious products. Coals sources from western Nigeriacontain higher amounts of calcium compounds and the ashes produced through the combustion of these coals typically contain 20 to 35 percent calciumoxide. These ashes are generally classified as Class C fly ashes. The amount of calcium oxide that is contained in the fly ash is much lower than that of lime, and much of it is combined with silicates and aluminates, so the fly ash has less of an effect on the plasticity than lime does while coal sources dictate the chemical composition of aparticular fly ash, boiler design and operation have a major influence on the hydration characteristics of a specific fly ash,particularly the rate at which hydration occurs. During combustion the inorganic matter is fusedand is transported from the combustion chamber suspended in the exhaust gases. Where rapidcooling of the fused particles occurs, the fly ash particles are generally noncrystalline. A more gradual cooling of the fused particles can result in

crystallization of some compounds before the particles solidify. It is not known whether the crystallinity of the ash may be attributed to the coal source, boiler design or other factors, however the properties of a given ash source arereported to be relatively constant. Compaction time after mixing is critical to achieving the maximum density and maximum potential strength. When compaction is delayed, hydration products begin to bond particles in a loose state and disruption of these aggregations is required to densify the material. Therefore, a portion of the compactive energy is utilized in overcoming the cementation and themaximum densities are reduced. Additionally, if thecementitious products formed prior to compaction must be disrupted to achieve effective compaction, the strength that is developed will be lower because the fly ash hasalready started reacting with the stabilized material(Milburn and Parsons, 2004).

***Bitumen*:** Bitumen used for road construction is viscous liquid or semi-solid materials, consisting essentially of hydrocarbons and their derivatives, which are soluble in trichloroethylene1. Whilst bitumen occur naturally (e.g. in lake asphalts containing mineral materials), the predominant majority of those used in roadwork are the penetration-grade products of the fractional distillation of petroleum at refineries. It might be noted here that the term ‘asphalt’ is used in the American technical literature to describe what is termed ‘bitumen’ in the UK. (and inEurope, following agreement by the ComitéEuropéen de Normalisation) the term ‘asphalt’ is reserved for materials containing a mixture of bitumen and mineral matter, e.g. lake asphalt or hot rolled asphalt. Bitumen that are produced artificially from petroleum crudes (usually naphthenicand asphaltic-base crudes) are known as refinery bitumen. Not all petroleumcrudes are suitable for the production of road bitumen; those used in the UK are mainly derived from Middle East and South American sources, as most of the North Sea crudes contain little or no bitumen. Bitumen is obtained by a refinery distillation

process, which involves condensation in a fractionating column. The first distillation is normally carried out at oilfield refineries where the crude is heated, at atmospheric pressure, to not greater than 350°C to remove naphtha, gasoline and kerosene fractions. The 'topped' oil is then shipped to a destination refinery where it is heated, at reduced pressure, to collect the heavier diesel and lubricating oils; without the reduced pressure, chemical changes, i.e. cracking, would impart undesirable properties to the bitumen residue. The residue is then treated to produce a wide range of penetration-grade bitumen that, depending upon the amount of distillate removed, range in consistency from semi-solid to semi-fluid at room temperature. In practice, it is common for a refinery to prepare and stock large quantities of bitumen at two extremes and to blend these to obtain intermediate grades (O'Flaherty, 2002). Bitumen are termed visco - elastic material in that

a) at temperatures above about 100°C they exhibit the properties of a viscous material
b) at temperatures below about minus 10°C they behave as an elastic material
c) at temperatures in between they behave as a material with viscous and elastic properties, with the predominating property at any given time depending upon the temperature and rate of load application.

Specifications with regard to the design and construction of a bituminous pavement are of little value if the properties of the binder used are not adequately controlled.Asphalts and tars are bituminous materials which are used for stabilization of soil, generally for pavement construction. Bituminous materials when added to a soil, imparts both cohesion and reduced water absorption (O'Flaherty, 2002)

2.3 Use of Bitumen Emulsion and Lime Stabilization.

Bitumen emulsions generally are classified as anionic, cationic, or non-ionic. Thefirst two types have electrical charges surrounding the particles, whereas the thirdtype is neutral. Classification as anionic or cationic is based on the electrical chargesthat surround the asphalt particles. Emulsions containing negatively charged particlesof asphalt are classified as anionic, and those having positively charged particles ofasphalt are classified as cationic. The anionic and cationic asphalts generally are usedin highway maintenance and construction, although it is likely that the non-ionic maybe used more frequently in the future as emulsion technology advances.

Each of these categories is further divided into three subgroups based on how rapidly the asphalt emulsion returns to the state of the original asphalt cement. These subgroups are rapid-setting (RS), medium-setting (MS), and slow-setting (SS).A cationic emulsion is identified by placing the letter "C" in front of the emulsion type.
No letter is placed in front of anionic and non-ionic emulsions for example, CRS-2 denotes a cationic emulsion, and RS-2 denotes either an anionic or non-ionic emulsion.
Bitumen emulsions are used in cold-laid plant mixes and road mixes (mixedin-place) for several purposes, including the construction of highway pavement surfaces and bases and in surface treatments. Note, however, that since anionic emulsions contain negative charges, they are more effective in treating aggregates containing electropositive charges (such as limestone), whereas cationic emulsions are more effective with electronegative aggregates (such as those containing a high percentage of siliceous material). Also note that ordinary emulsions must be protected during very cold spells because they will break down if frozen. Three grades of high-float,medium-setting anionic emulsions designated as HFMS have been developed and are used mainly in cold and hot plant mixes and coarse aggregate seal coats. These

highfloatemulsions have one significant property: They can be laid at relatively thicker films without a high probability of runoff (Garber andHoel, 2009).

2.3.1 Advantages to be derived from the combined use of bitumen emulsion and lime in soil stabilization and the applications.

In soil stabilization, there are advantages considered in the combination of bitumen emulsion and lime in stabilizing the soil in other to serve as pavement material. They function as working platform for the project, waterproofs the soil, improves soil strength, reduce soil volume change due to temperature or moisture, improves soil workability, reduces dust in work environment, upgrade marginal materials, improves durability, dries wet soil, conserve aggregate materials, reduces cost and conserves energy. The disadvantages only will be when you use the additives in excess against what is specified by Nigerian general specifications (1997).

2.3.2 Applications

Soil stabilization is used in many sectors of the construction industry. Roads, parking lots, airport runways, building sites, landfills and soil remediation all use some form of soil stabilization. Other Application includes waterway management, mining and agriculture (Huffman, 2006).

2.3.3 Importance of soil stabilization

Throughout Nigeria, agencies and contractors have experienced recurring pavement failures actually due to weakness of the sub-grade soil or sub-base material beneath the pavement. Many regions of the western hemisphere include geographies where expansive soils are encountered. Other places are plagued by periods of heavy rainfall and high ground water tables that can weaken the supportive layers beneath pavements. In some cases, poor construction techniques and inattention to details during construction creates further difficulties.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials

3.1.1 Laterites

Samples of laterite were taken from borrow pitat a depth of1.0m, to avoid organic matter, at Maitama ExtensionDistrict(MED), Abuja.Various tests were performed in accordance with the British Standard (BS 1377(1990))and American Society of Testing and Materials (ASTM 1992).

3.1.2 Bitumen emulsion

The bitumen emulsion used for this research was obtained from Kakatar Construction Company, Abuja. The bitumen emulsion was classified as cationic because it has droplets which carry a positive charge

3.1.3 Lime (hydrated lime)

The lime used for this research is a Delta Lime which was obtained from Delta Steel Company Limited, Ovwian-Aladja, Delta State, Nigeria. A typical oxide composition of lime and laterite is given in Table 4.2 and 4.3

3.2 Methods

3.2.1 Natural moisture content

The natural moisture content of the soil as obtained from the site was determined in accordance with BS 1377 (1990) Part 2. Three weighing containers cleaned and weighed to the nearest 0.01g (M_1). The sample as freshly collected was crumbled and placed loosely in the containers and the containers with the samples were weighed together to the nearest 0.01g as M_2. The containers were then placed in the oven and dried at 105 -110^{o}C for 24 hours. The containers and the samples were then removed and weighed dry to the nearest 0.01g as M_3. The natural moisture content (as

collected from the site) is calculated as the average of the three oven dried samples using Equation 3.1.

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \quad (3.1)$$

$$w = \frac{9.23}{49.76} \times 100 = 18.55$$

Where;

W = moisture content (%).

M_1 = weight of empty container (g)

M_2 = weight of container + wet soil (g)

M_3 = weight of container + dry soil (g)

The result of the natural moisture content of the naturalsoilis shown in Table 3.1 below

Table 3.1: Natural Moisture Content of the Natural Soil

CONTAINER NO	P2	P5	P9
Wt of container + wet soil, M2	76.94	87	85.37
Wt of container + dry soil, M3	67.71	76.4	74.93
Wt of container M1	17.95	18.46	18.24
Dry wt of soil M3 – M1	49.76	57.94	56.69
Wt of moisture, M2 – M3	9.23	10.6	10.44
Moisture content	18.55	18.29	18.41
Average Moisture content		18.42	

3.2.2 Determination of specific gravity

The determination of specific gravity was carried out according to BS 1377 (1990) for fine – grained soils. The density bottle and the stopper were weighed to the nearest 0.001g (M_1). The air dried soil was transferred into the density bottle, and the bottle, content and the cover were weighed as M_2. Water was then added just enough to cover the soil, the solution was gently stirred to remove any bubble. The bottle was then completely filled up and covered. The covered bottle was then wiped dry and the whole weighed to the nearest 0.001g (as M_3). The bottle was subsequently emptied and filled completely with water, wiped dry and weighed to the nearest 0.001g (M_4).To obtain a more accurate results, three density bottles were used and their average taken. The specific gravity is calculated using Equation 3.2.

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \qquad (3.2)$$

$$G_s = \frac{57.1}{23.9} = 2.39$$

Where;

G_s = specific gravity.

M_1= mass of empty density bottle (g)

M_2 = mass of density bottle + dry soil (g)

M_3 = density bottle + soil + water (g)

M_3 = mass of density bottle filled with water (g)

The resultof the specific gravity of the natural soil is shown in Table 3.2 below

Table 3.2: Specific Gravity of the NaturalSoil

BOTTLE NO	Y1	Y2	Y3
Wt of bottle + water (full), M4	683.9	346.5	352.4
Wt of bottle +soil + water, M3	717.1	369.7	385.7
Wt of bottle + soil, M2	241.3	137	160
Wt of bottle, M1	184.2	105.1	105.8
Wt of water, (M4 – M1)	499.7	248.4	248.6
Wt of water added to the soil, (M3-M2)	475.5	232.7	225.7
Wt of water displaced by the soil, W = (M4 – M1) – (M3 – M2)	23.9	15.7	22.9
Specific gravity of the soil, (M2 – M1)/ (M4 – M1) – (M3 – M2)	2.39	2.03	2.36

3.2.3 Particle size distribution of the natural soil.

Sieve analysis was carried out in accordance with BS 1377(1990) Part 2. Wet sieving was conducted by measuring 200 g of the soil sample and soaking it for 24 hours. The sample was then washed through BS No 200 sieve. The particles retained were then dried in the oven for 24 hours and dry sieving was carried out on the dried sample to obtain the particle size distribution. The percentage passing each sieve size was calculated and plotted on a semi-log graph; percentages passing against sieve sizes. The results of the soil sieve analysis and the hydrometer analysis of the soil are shown in Table 3.3 and 3.4 below;

Table 3.3: Sieve analysis of the natural soil

B.S SIEVE SIZE	WEIGHT RETAINED	WEIGHT PASSING	% PASSING
2.4mm (7)	1.94	198.06	99.03
1.4mm (10)	3.13	194.93	97.465
0.6mm (600)	21.82	173.11	86.555
0.425mm (425)	7	166.11	83.055
0.3mm (300)	15.45	150.66	75.33
0.212mm (212)	12.69	137.97	68.985
0.15mm (150)	18.2	119.77	59.885
0.075mm (75)	7.23	112.54	56.27

Table 3.4: Hydrometeranalysis of the natural soil

TIME	H.R.R	TEMP	FACTOR F	H=17.67-0.176R	COR. T R	PART. SIZE	K	% PASSING
0.17		26.2						
0.25								
0.5								
1								
2								
4	53	29	0.04168	8.342	52.8	0.06019	1.704225	44.99155
8	51	29	0.04168	8.694	50.8	0.04345	1.704225	43.28732
15	44	29	0.04168	9.926	43.8	0.03391	1.704225	37.32254
30	35	30	0.04121	11.51	34.8	0.02553	1.704225	29.65352
60	30	30	0.04121	12.39	29.8	0.01873	1.704225	25.39296
120	25	30	0.04121	13.27	24.8	0.0137	1.704225	21.13239
240	21	30	0.04121	13.974	20.8	0.00994	1.704225	17.72394
360	18	30	0.04121	14.502	17.8	0.00827	1.704225	15.16761
720	15	29	0.04168	15.03	14.8	0.00602	1.704225	12.61127
1440	12	29	0.04168	15.558	11.8	0.00433	1.704225	10.05493

3.2.4 Atterberg limits

The test includes the determination of the liquid limits, plastic limits and the plasticity index for the natural soil and the stabilized soils. They were also conducted in accordance with Test 1(A) BS 1377 (1990) Part 2 for the natural soil

3.2.4.1 Liquid limit: The soil sample for liquid limit was air dried and 200 g of the material passing through BS No 4 sieve (425 μm aperture) was obtained and thoroughly mixed with water to form a homogeneous paste on a flat glass plate. A portion of the soil water mixture was then placed in the cup of the casagrande apparatus, leveled off parallel to the base and divided by drawing the grooving tool along the diameter through the centre of the hinge. The cup was then lifted up and dropped by turning the crank until the two parts of the soil come into contact at the bottom of the groove. The number of blows at which that occurred was recorded and a little quantity of the soil was taken and its moisture content was determined as in section 3.2.1. The test was performed for well–spaced out moisture content from the drier to the wetter states. The values of the moisture content (determined) and the corresponding number of blows was then plotted on a semi–logarithmic graph and the liquid limit was determined as the moisture content corresponding to 25 blows.

The liquid limit result for soil-lime-bitumen mixesis shown in Table 3.5 below;

Table 3.5: Liquid Limit Test Result for Soil-Lime-Bitumen mixtures

BITUMEN	LIME CONTENT, %

CONTENT, %	0	2	4	6	8	10
0	45	48	46	48	47	50
2	52	46	43	52	43	47
4	47	44	41	47	40	44
6	40	40	40	46	38	42
8	36	40	38	43	38	40
10	36	36	32	40	40	38

3.2.4.2Plastic limit: A portion of the soil mixes used for the liquid limit test was retained for the determination of plastic limit. The ball of the soil/soil admixtures was moulded between the fingers and rolled between the palms of the hand until it dried sufficiently (even though the soil was already relatively drier than the ones used for liquid limit). The sample was then divided into approximately four equal parts. Each of the parts was rolled into a thread between the first finger and the thumb. The thread was then rolled between the tip of the fingers of one hand and the glass. This continued until the diameter of the thread was reduced to about 3 mm in five to ten forward and backward movements of the hand. The movement continued until the thread sheared both longitudinally and transversely. The crumbled soil was then put in the moisture container and the moisture content determined.

The plastic limitresult for soil-lime-bitumen mixes is shown in Table 3.6 below;

Table 3.6: Plastic Limit Test Result for Soil-Lime-Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	22	24	25	27	32	33
2	24	25	25	27	33	30
4	25	27	26	29	32	33
6	25	27	27	30	32	32
8	27	28	29	31	33	31
10	28	28	29	32	33	30

3.2.4.3 Plasticity index: The plasticity index (PI) of the soil lime bitumen mix is the difference between the liquid limit (LL) of the natural/various mixes of the soil and their corresponding plastic limit (PL). The plasticity index of the samples was calculated using Equation 3.3:

$$PI = LL - PL \quad (3.3)$$

Where:

PI = Plasticity index (%)

LL = Liquid limit (%)

PL = Plastic limit (%)

$$PI = LL - PL$$

$$PI = 48 - 24$$

$$PI = 24$$

The plasticity index result for soil-lime-bitumen mixes is shown in Table 3.7 below;

Table 3.7: Plasticity Index Test Result for Soil-Lime-Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	27	24	21	21	15	17
2	28	21	18	25	10	28
4	22	17	15	18	8	11
6	15	13	13	16	6	10
8	9	12	9	12	5	9
10	8	8	3	8	7	8

3.2.5 Compaction characteristics

3.2.5.1 Maximum dry density: The compaction tests were carried out for the natural soil and the stabilized soils (in different percentages); all according to BS 1377 (1990) Part 4, using the standard proctor in accordance with the Nigerian General Specification (1997).

Three kg of the soil/soil-admixtures sample was mixed thoroughly with 8 % of water (and the water was added at 8 % for each of the compaction). The sample was then compacted into the 1000 cm^3 (of mass m1); with the standard proctor in three layers of approximately equal mass with each layer receiving 27 blows of 2.5 kg rammer falling through a height of 300 mm. The blows were uniformly distributed over the surface of each layer. The collar was then removed and the compacted sample leveled off at the top of the mould with a straight edge. The mould containing the leveled sample was then weighed to the nearest 1 g, $M_{2.}$

Two small samples were then taken from the compacted soil for the determination of moisture content. The sample was then removed from the mould, crushed and addition water added (8 %) and the same procedure was repeated until

minimum of five set of samples were taken for moisture content determination. The bulk density in Mg/m^3 was later calculated for each compacted layer using Equation 3.4

$$\rho = \frac{M_2 - M_1}{1000} \tag{3.4}$$

The dry density was also calculated using the equation 3.5.

$$\rho_d = 100\rho/(100 + w) \tag{3.5}$$

Where;

w = moisture content of each compacted layer.

The values of the dry densities as obtained from equation (3.5) were plotted against their respective moisture contents and the maximum dry density (MDD) was deduced as the maximum point on the resultant curves.

The result of the maximum dry densityof the soil-lime-bitumen mixesis shown in Table 3.8 below;

Table 3.8: Maximum Dry Density Test Result for Soil-Lime-Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	1.67	1.65	1.62	1.58	1.6	1.61
2	1.6	1.59	1.56	1.54	1.55	1.57
4	1.58	1.56	1.54	1.51	1.53	1.55
6	1.52	1.53	1.52	1.49	1.51	1.53
8	1.55	1.51	1.5	1.5	1.49	1.5
10	1.55	1.53	1.52	1.53	1.5	1.52

3.2.5.2 Optimum moisture content: The corresponding values of moisture contents at maximum dry densities (MDD), deduced from the graph of dry density against moisture contents, gives the optimum moisture content (OMC).

The result of the Optimum moisture content of the soil-lime-bitumen mixes is shown in Table 3.9 below;

Table 3.9: Optimum Moisture Content Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	13.2	13.5	13.7	14.1	14.5	14.3
2	13	13.4	13.5	13.8	14.4	14
4	12.8	13.2	13.4	13.5	14.2	13.8
6	12.6	13	13.2	13.3	14	13.6
8	12.3	12.8	13	13.3	14	13.4
10	12.5	13.1	13.1	13.5	14.3	13.5

3.2.6 Strength characteristics

3.2.6.1 Unconfined compressive strength

The unconfined compressive strength (UCS) tests were performed on the soil samples according to BS 1377; 1990 Part 7 using the standard proctor. The natural soil sample/the stabilized soil samples were compacted in 1000 cm^3 moulds at their respective OMC. The samples were extruded from the moulds and trimmed into a cylindrical specimen of 38.1 mm diameter and 76.2 mm length. The three cylindrical specimens from the mould were cured for 7 days, second for 14 days and the third for 28 days. At the elapsed day of curing, the specimens were then placed centrally on the lower platen of a compression testing machine and a compressive force was applied to

the specimen with a strain control at 0.10% mm. Record was taken simultaneously of the axial deformation and the axial force at regular interval until failure of the sample occurs. The UCS of the sample was determined at the point on the stress–strain curve at which failure occurred.There were equally other cylindrical specimensfrom the mould that was cured for 7 days and soaked for 7 days. The UCS was calculated from the Equation 3.6

$$\text{Compressive strength} = \frac{\text{Failure load}}{\text{Surface Area of Specimen}} \quad (3.6)$$

$$\text{Compressive strength} = \frac{[R \times C_r \times (100 - E\%) \times 1000 kN/m_2]}{(100 \times A_o)}$$

Where;

$E\% = L/L_o$

E% = Strain percent

L = Amount of compression at any stage

R = Load ring reading at Strain E

C_r = mean calibration of load ring

L_o = Initial length of the specimen

A_o = Initial cross sectional area

σ = Compressive stress at strain E.

The results of the unconfined compressive strength test of thesoil-lime-bitumen mixesare shown in Table 3.10 to 3.13 below;

Table 3.10: Unconfined Compressive Strength (7 days curing) Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	235	276	371	812	782	691
2	240	342	528	875	975	709
4	254	300	552	938	882	838
6	294	313	631	985	999	1076
8	274	342	607	792	892	787
10	313	303	533	740	665	659

Table 3.11: Unconfined Compressive Strength (14 days curing) Test Result for Soil-Lime–Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	270	286	323	582	640	779
2	298	310	367	600	789	968
4	330	496	591	716	938	1005
6	371	431	616	782	1027	1125
8	358	454	594	750	835	936
10	313	405	542	696	738	887

Table 3.12: Unconfined Compressive Strength (28 days curing) Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	305	325	475	642	884	1032
2	323	403	654	779	984	1125
4	354	476	676	879	1055	1135
6	464	571	771	955	1356	1839
8	335	513	705	802	1023	1225
10	315	494	714	755	890	1076

Table 3.13: Unconfined Compressive Strength (7 days curing + 7 day soaking) Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	39	87	117	235	284	303
2	59	150	264	343	352	371
4	97	189	298	313	333	382
6	132	264	264	362	440	587
8	151	223	254	303	357	411
10	120	175	205	266	288	301



Plate 1: Crushed samples of UCS showing different pattern of failure



Plate ll: Crushed samples inside triaxial cell

3.2.6.2 California bearing ratio

The California bearing ratio (CBR) tests were carried out in accordance with BS 1377(1990) for the natural and treated soils. The CBR is expressed by the force exerted by the plunger and the depth of its penetration into the specimen; it is aimed at determining the relationship between force and penetration.

Five kg of the soil sample/soil-admixture sample were mixed at their respective optimum moisture contents in 2360 cm^3mould, at the standard proctor compactive energies. The compaction was in three layers each receiving 62 blows from the 2.5 kg rammer.The base plates were removed (after compaction) and the compacted specimens placed in sealed plastic bags for curing (for 6 days) and after the 6^{th} day the specimens were immersed in water for 24 hours before testing according to the specification by the Nigerian General Specifications (1997). The base plates were later replaced and the specimens transferred to the CBR testing machine and positioned on the lower plate of the machine. The plunger was then made to penetrate the specimen at a rate 1.3 mm/min until the specimen failed. The mould was then inverted, base plate removed and the procedure repeated for the base of the specimens.

From the values of the penetration and force recorded, a curve of force against penetration was obtained. The CBR value was calculated at penetration 2.5 mm or 5.0 mm; the greater of the two values and as their means where the value are within 10% of each other. The CBR was calculated using Equation 3.7.

$$\text{CBR} = \frac{\text{Measured load}}{\text{Standard load}} \times 100\% \qquad (3.7)$$

Standard load = 13.24 kN of 2.5 mm penetration

= 19.96 kN of 5.0 mm penetration

$$CBR = \frac{0.26}{13.24} \times 100 = 2$$

$$CBR = \frac{0.40}{19.96} \times 100 = 2$$

The California bearing ratio test result ofthe soil-lime-bitumen mixtures is shown Table 3.14 below;

Table 3.14: California Bearing Ratio (Soaked) Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	2	5	9	17	21	25
2	3	8	13	20	24	27
4	5	13	22	23	25	29
6	7	16	24	24	26	42
8	9	10	15	23	34	47
10	8	8	17	23	27	37

3.2.8 Durability assessment

The durability assessment (under adverse field conditions) of the soil sample was determined by resistance to loss in strength when immersed in water. It was expressed as the ratio of UCS of the specimen/wax cured for 7 days and de-waxed top and bottom soaked for another 7 days to the UCS of the specimen cured for 14 days. The resistance to loss in strength was calculated using Equation 3.8.

$$\text{Resistance to loss in strength}(\%) = \frac{\text{UCS}(7 \text{ day cured} + 7 \text{ day soaked})}{\text{UCS}(14 \text{ day cured})} \times 100\% \quad (3.8)$$

$$\text{for } (0\,\%\,\text{bitumen with } 0\,\%\,\text{lime}) = \frac{(39)}{(270)} \times 100\% \quad = 14.44\ \%$$

The results of the resistant to loss in strength test for of soil-lime-bitumen mixes is shown in Table 3.15 below;

Table 3.15: Resistance to Loss in Strength Test Result for Soil-Lime –Bitumen mixtures

BITUMEN CONTENT, %	LIME CONTENT, %					
	0	2	4	6	8	10
0	14.44	30.42	36.22	40.38	44.38	38.9
2	19.8	48.39	71.93	57.17	44.61	38.33
4	29.39	38.1	50.42	43.72	35.5	38.01
6	35.58	61.25	42.86	46.29	42.84	52.18
8	42.18	49.12	42.76	40.4	42.75	43.91
10	39.02	43.21	39.03	38.22	39.04	33.93

3.3 Methods of Analysis

The two methods of analysis used in this research work are:

i. Graphical method using Microsoft excel software package
ii. Two way analysis of variance (ANOVA) without replication using the microsoft excel software package.

3.3.1 Graphical method using microsoft excel software package

In the graphical method using microsoft excel software package, graphs were used to show the relationship between two quantities. An independent variable usually plotted on the X-axis and a dependent variable on the Y-axis. The graphs were plotted using the microsoft excel package

3.3.2 Two way analysis of variance (ANOVA) without replication using the microsoft excel software package.

In this method,statistical analysis was carried out on results obtained using analysis of variance (ANOVA) with the Microsoft Excel Analysis Tool Pack Software Package to determine the levels of significance of bituminous stabilization of laterite modified with lime for use as pavement materials.

CHAPTER FOUR

ANALYSISAND DISCUSSION OF RESULTS

4.1 Properties of Materials used in this Study

4.1.1 Natural soil

Preliminary test conducted on the natural soil revealed that the soil has high moisture content of 18.4%, because the sample was collected in rainy season. The index properties are summarized in Table 4.1. The particle size distribution curve of the natural soil is shown in Figure 4.1. The soil belongs to the CH group using the Unified Soil Classification System (ASTM, 1992) or A-7-6(14) soilusing the AASHTO soil classification system (AASHTO, 1986). The soil is reddish brown in colour with a liquid limit of 44.4 %, plastic limit of 21.6 % and plasticity index of 22.7 %. The Soaked CBR value was 2 % while the UCS value of the soil was found to be 235 kN/m^2. The soil was found to be of low plasticity and fell below the standard recommendation for most construction works especially highway construction.

Table 4.1: Properties of the Natural Soil

Property	Quantity
Percentage passing BS No 200 sieve	68.5
Natural Moisture Content, %	18.4
Liquid Limit, %	44.4
Plastic Limit, %	21.6
Plasticity Index, %	22.7
Specific Gravity	2.26
AASHTO Classification	A-7-6 (14)
USCS	CL
Maximum Dry Density, Mg/m^3	1.67
Optimum Moisture Content, %	13.2
Unconfined Compressive Strength, kN/m^2	235
California Bearing Ratio, %	2
Colour	Reddish Brown

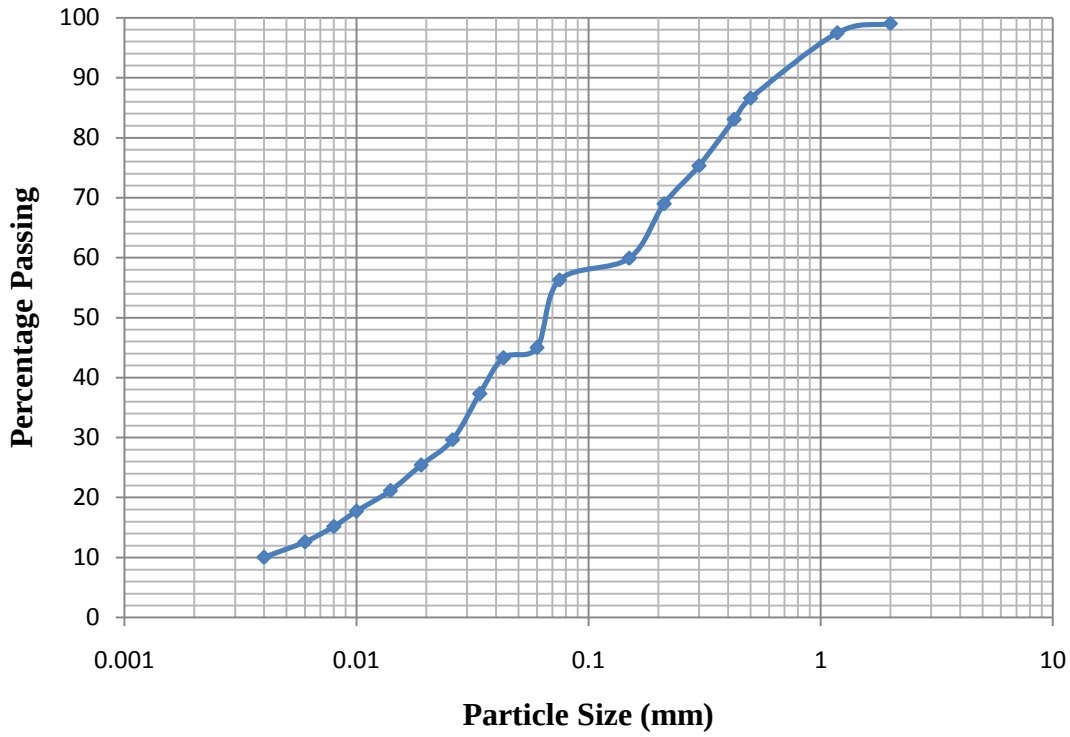


Figure 4.1: Particle size distribution curve of the natural soil used in this work

4.1.2 Additives

The oxide composition of lime was determined at Nigerian geological survey in Kaduna, using the method of XRF. The test on the bitumen emulsion was performed in accordance with B.S. 434 (1984).Thebitumen emulsion used consists of three basic ingredients which are bitumen, water, and emulsifiers. Some physical and chemical principles apply in the formulation and production ofbitumen emulsion to make a stable dispersion of the bitumen cement in water, stable enough for pumping, prolonged storage, and mixing.Upon curing, the residual bitumen retains all of the adhesion, durability, and water-resistance of the bitumen cement from which it was produced. The calculated compound composition of lime and the summary result of the properties of the bitumen used as compared with specifications are shown in Table 4.2 and Table 4.3 below.

Table 4.2: Oxide composition of the hydrated lime and laterite

Oxide	Concentration (%)
CaO	69.14
V_2O_3	0.002
TiO_2	0.05
MnO	0.07
Fe_2O_3	0.288
ZrO_2	0.001
BaO	0.11
NiO	0.059
SrO	0.705
CrO_2	0.01
Loss on ignition	29.36

Table 4.3:Results of the properties of bitumen emulsion used in comparison with specification.

Test Carried Out	**Results**	**Specifications**
Test on bitumen emulsion		(O'Flaherty, 2002)
Viscosity, Eurol at 25^{o}C (sec)	100	100min
Storage stability test 24 hr (%)	0.5	1 max
Sieve test (%)	0.08	0.10 max
Residue by distillation (%)	65	65 min
Test on residue for distillation:		
Penetration at 25^{o}C, 100g, 5 sec	140	100 – 200
Ductility at 25^{o}C, 5cm/mm, cm	45	40 min
Solubility in trichloroethylene, %	98.0	97.5 min

4.2 Atterberg's Limits of the Stabilized Soil

4.2.1 Liquid limit

The variation of liquid limit of laterite – bitumen emulsion mixture with lime content is shown in Figure 4.2.

Liquid limit of laterite decreased with lime and bitumen emulsion content. This decrease can be attributed to the agglomeration and flocculation of the soil particles, as a resultofcation exchange at the surface of the soil particles as reported by Moses and Afolayan(2011). Figure 4.2 shows that the liquid limit of the natural laterite decreased from 50 % to a value of 32 % when treated with 4 % lime with 10 % bitumen emulsion. According to Nigerian general specification (1977), liquid limit should not be greater than 35% for base and sub-base. Detailed test results are given in Table 3.5 of chapter 3

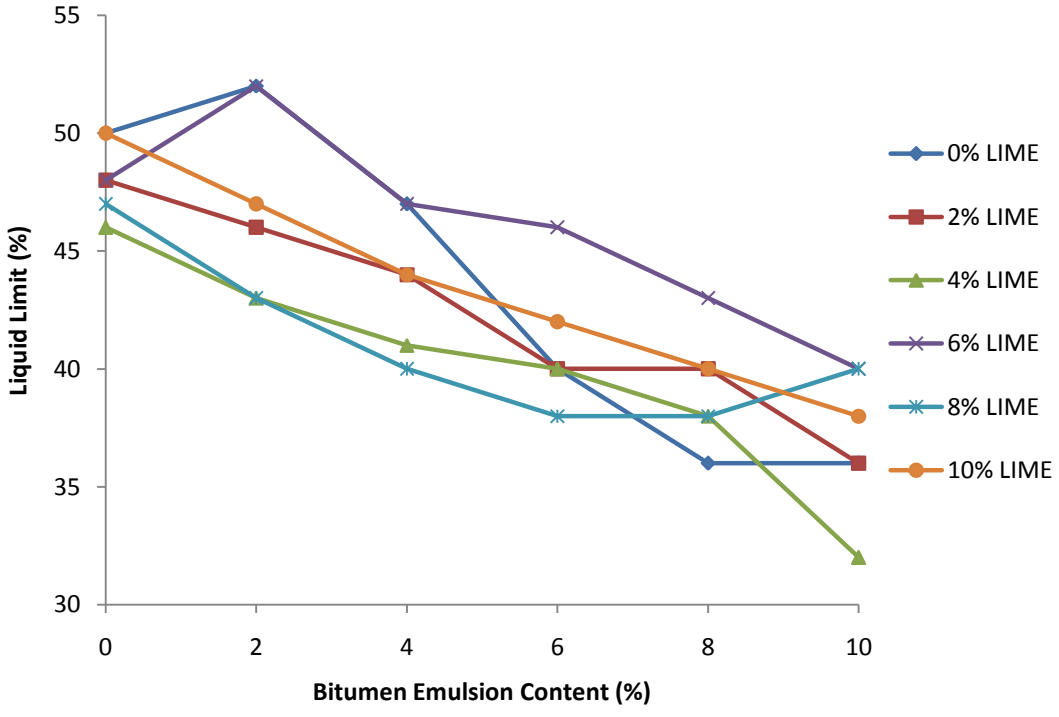


Figure 4.2: Variation of liquid limit of soil - lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the liquid limit result (see Table A.14 of the appendix)shows that the effects of Lime and Bitumen on lateritic soil

were statistically significant (F_{CAL}=25.5>F_{CRIT} = 2.6) for Bitumen emulsion and (F_{CAL}=5.79 >F_{CRIT} = 2.6) for Lime. The effect of Bitumenemulsion on the liquid limitresult was more pronounced than that of Lime (α = 5% and $p < 0.05$). Detailed test results are given in Table A.3 of the appendix.

4.2.2 Plastic limit

The variation of plastic limit of laterite with bitumen emulsion andlime content is shown in Figure 4.3. Plastic limit generally increases with increase in admixture which is probably due to the higher release rate of Ca^{2+}, $Si^{2+}$and Al^{2+} cations with increased lime plus Bitumen emulsion (Ramzi*et. al*., 2001). The decrease in 10 % lime with 10% bitumen emulsion is as a result of lime and bitumen emulsion occupying greater percentage in the soil matrix, as a result lead to decrease in plasticity index. Detailed test results are given in Table 3.6 of chapter 3

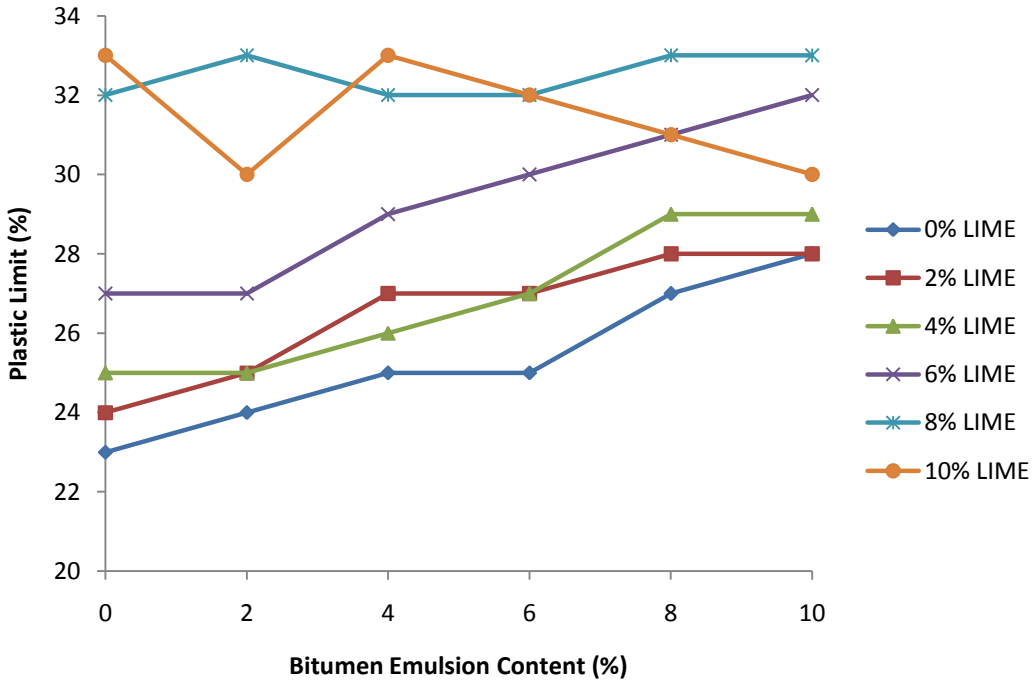


Figure 4.3: Variation of plastic limit of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the plastic limit result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on

lateritic soil were statistically significant (F_{CAL}=5.08>F_{CRIT} = 2.6) for Bitumen emulsion and (F_{CAL}=32.01>F_{CRIT} = 2.6) for Lime. The effect of Lime on the plastic limit was more pronounced than that of Bitumen emulsion (α = 5% and $p < 0.05$). Detailed test results are given in Table A.4 of the appendix.

4.2.3 Plasticity index

The variation of plasticityindex limit of laterite with bitumen emulsion andlime content is shown in Figure 4.4.The plasticityindex decreases with increase in the admixtures and the decrease can be associated with the agglomeration and flocculation of the soil particles which is as a result of exchange of ions at the surface of the soil particles (Moses and Afolayan, 2011). The plasticity index of laterite decrease from natural value of 27% to a value of 3% at 4 % lime with 10% bitumen emulsion. Detailed test results are given in Table 3.7 of chapter 3

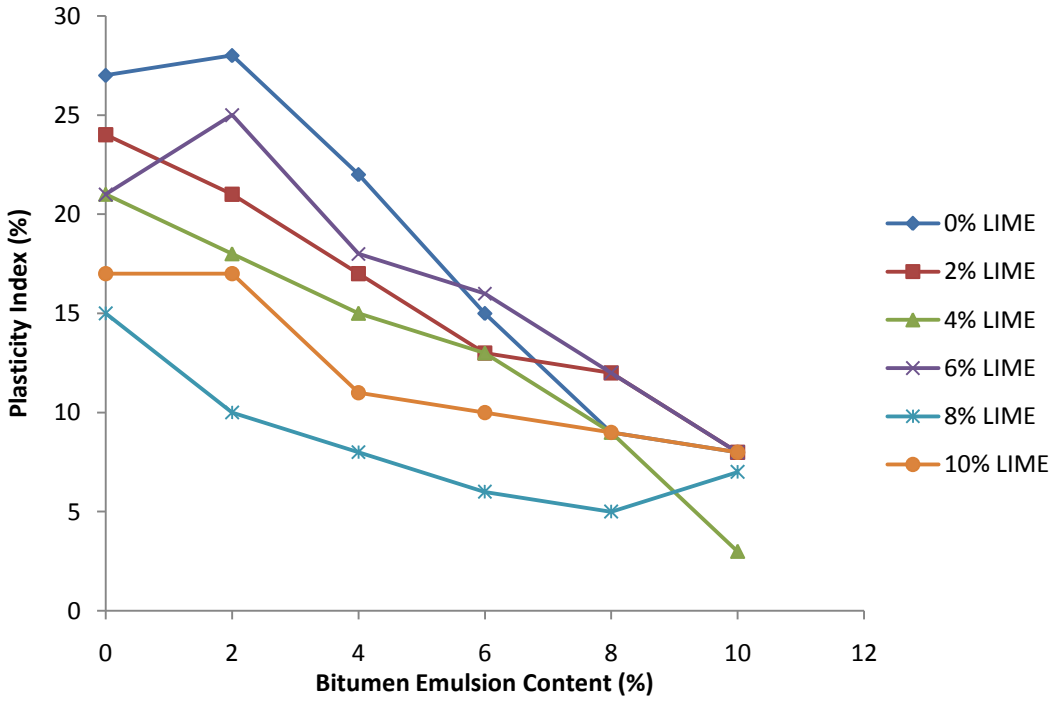


Figure 4.4: Variation of plasticityindex of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the plasticityindex result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on

lateritic soil were statistically significant (F_{CAL}= 27.55 >F_{CRIT}= 2.6) for Bitumen emulsion and (F_{CAL}=11.07>F_{CRIT} = 2.6) for Lime. The effect of Bitumen emulsion in the plasticity index result was more pronounced than that of Lime (α =5 % and p < 0.05). Detailed test results are given in Table A.5 of the appendix.

4.3 Compaction Characteristic of the Stabilized Soil

4.3.1 Maximum dry density

The variation of MDD of soil-lime mixture with bitumen content is presented in Figure 4.5. Generally, MDD values decrease with higher lime content and also decrease as the bitumen content increases. The decrease in the value of the MDD is as a result of the increase in lime / bitumen particles coating the soil to form large aggregates which then occupy larger spaces within the soil. The MDD value decreased from 1.67 to 1.49 $Mg/m^3$when laterite was treated at6 % Lime with 6% Bitumen emulsion. The sharp drop for 0 % lime with 10% bitumen emulsion is as a result of flocculation. The following researchers Hausmann (1990), Stephen (2006), Osinubi et al., (2007) and Sani (2012) carried out a similar research. Detailed test results are given in Table 3.8 of chapter 3

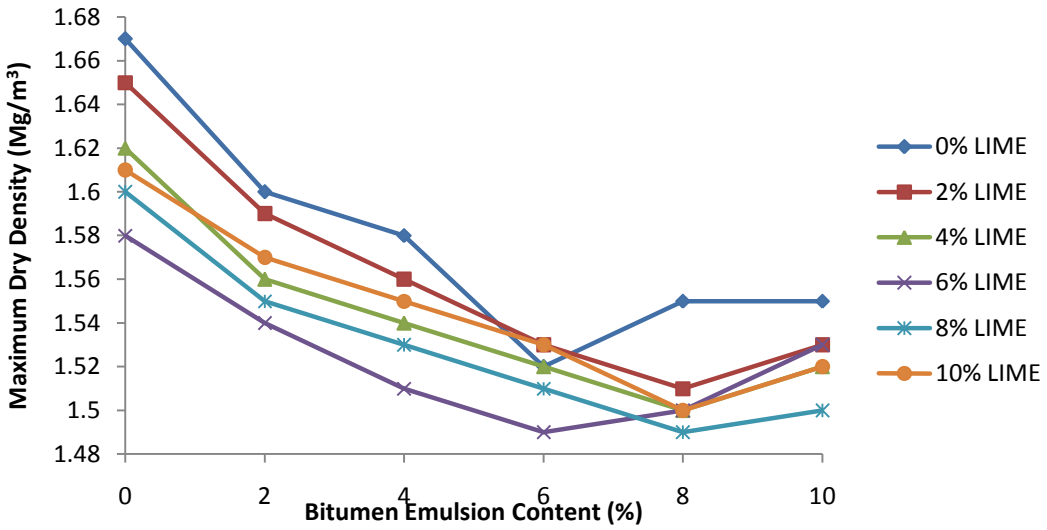


Figure 4.5: Variation of maximum dry density of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the MDD result(see Table A.14 of the appendix)shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}= 67.1 >F_{CRIT} = 2.6) for Bitumen and (F_{CAL}= 14.9 >F_{CRIT} = 2.6) for Lime. The effect of Bitumen on the MDD result was more pronounced than that of Lime (α =5 % and $p < 0.05$). Detailed test results are given in Table A.6 of the appendix.

4.3.2 Optimum moisture content

The OMC value decreased as the bitumen value increased conversely, it increased as the lime content was increased. This is as a result of bitumen occupying greater fraction of the soil matrix. The increase in OMC as a result of increase in lime content is as a result of more water needed to dissociate Ca^{2+} and OH^- ions to supply Ca^{2+} for the cation exchange reaction. The OMC value decreased from 13.2 % for the natural soil to a value of 12.3 % at 0% lime with 8% bitumen emulsion.Liquid limit of laterite decreased with lime and bitumen emulsion content. This decrease can be attributed to the agglomeration and flocculation of the soil particles, as a resultofcation exchange at the surface of the soil particles as reported by Moses and Afolayan(2011). Figure 4.2 shows that the liquid limit of the natural laterite decreased from 50 % to a value of 32 % when treated with 4 % lime with 10 % bitumen emulsion. According to Nigerian general specification (1977), liquid limit should not be greater than 35% for base and sub-base. Detailed test results are given in Table 3.9 of chapter 3

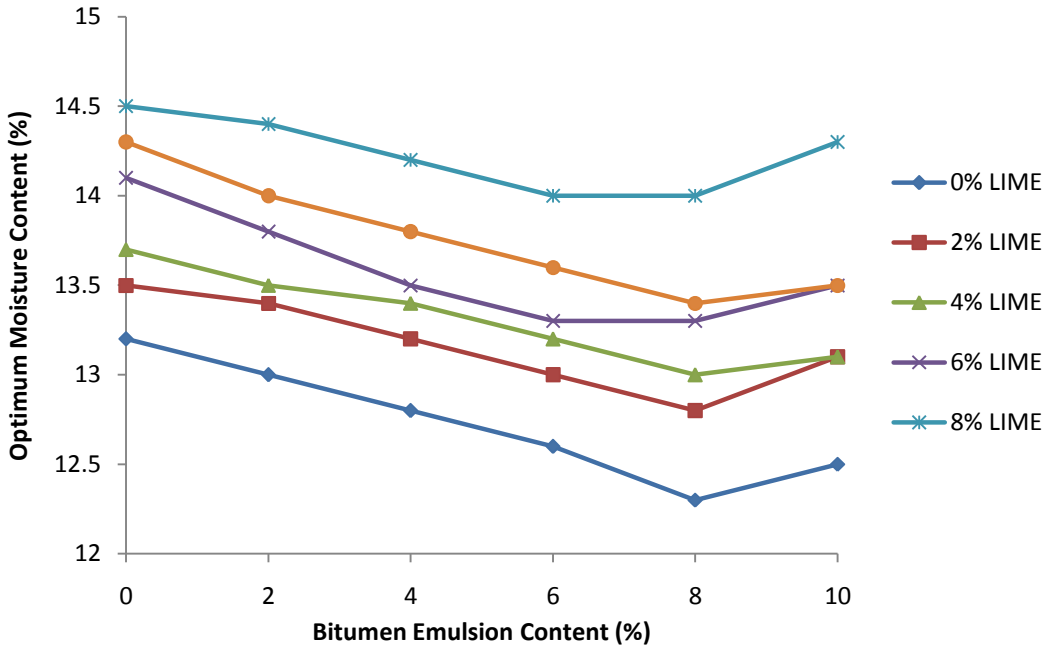


Figure 4.6: Variation of optimum moisture content of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the OMC result (see Table A.14 of the appendix)shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}= 54.8 > F_{CRIT} = 2.6) for Bitumen and (F_{CAL}=191.7 >F_{CRIT}= 2.6) for Lime. The effect of Lime in the OMC result was more pronounced than that of Bitumen (α =5 % and $p < 0.05$). Detailed test results are given in Table A.7 of the appendix.

4.4 Strength Characteristics

4.4.1 Unconfined compressive strength

The variation of unconfined compressive strength of laterite treated with bitumen emulsion and lime is as presented in Figure 4.7 – 4.10.

4.4.1.1 7 Day curing period: The variation of UCS of soil-lime-bitumen mixture cured for 7 days is shown in Figure 4.7.

The UCS values showed a sharp increase from 235 to 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion with 6 % lime, 6 % bitumen emulsion with 8 % lime and 6 %

bitumen emulsion with 10 % lime respectively. Subsequent addition of bitumen content did not increase the UCS value, rather there was slight decreased to values of 740, 665 and 659 kN/m^{2}respectively at 10 % bitumen emulsion with 6 % lime, 10 % bitumen emulsion with 8 % lime and 10 % bitumen emulsion with 10 % lime. The decrease is as a result of bitumen occupying greater fraction of the soil matrix which leads to a decrease in strength and less formation of some compounds.

The increase in the UCS value was primarily due to the formation of various compounds such as calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH) and micro fabric changes, which are responsible for strength development. Although the UCS values obtained fell short of 1710 kN/m^2. specified by TRRL (1977) as a criterion for adequate stabilization using OPC. The values of 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion with 6 % lime, 6 % bitumen emulsion with 8 % lime and 6 % bitumen emulsion with 10 % lime respectively on the other hand meet the requirement of 687–1373 kN/m^{2}for sub-base as specified byIngles and Metcalf (1972).Liquid limit of laterite decreased with lime and bitumen emulsion content. This decrease can be attributed to the agglomeration and flocculation of the soil particles, as a resultofcation exchange at the surface of the soil particles as reported by Moses and Afolayan(2011). Figure 4.2 shows that the liquid limit of the natural laterite decreased from 50 % to a value of 32 % when treated with 4 % lime with 10 % bitumen emulsion. According to Nigerian general specification (1977), liquid limit should not be greater than 35% for base and sub-base. Detailed test results are given in Table 3.10 of chapter 3

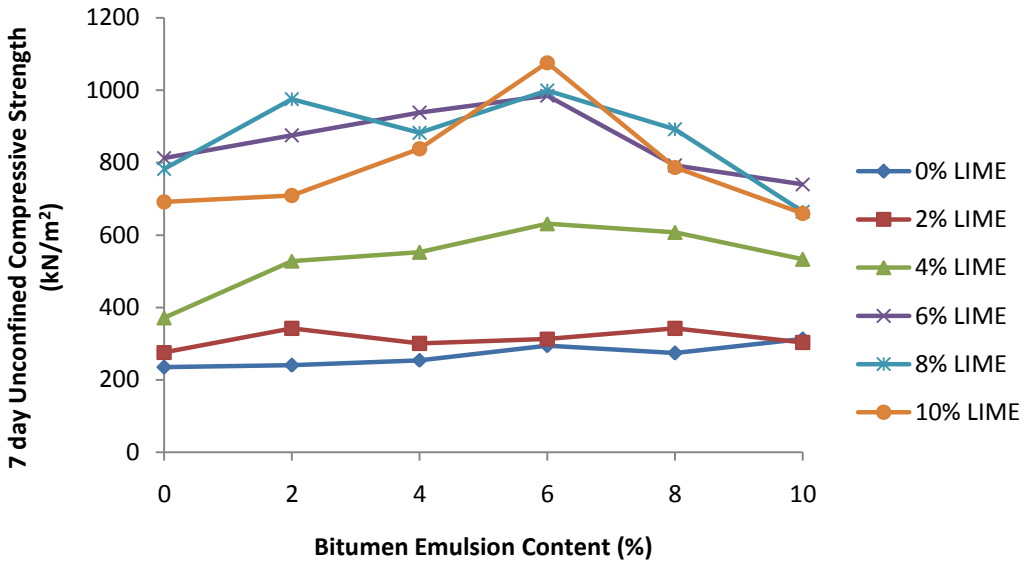


Figure. 4.7: Variation of UCS (7 day curing) of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the 7 days UCS result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}= 4.9> F_{CRIT} = 2.6) for Bitumen and (F_{CAL}= 76.4>F_{CRIT} = 2.6) for Lime. The effect of Lime in the 7 days UCS result was more pronounced than that of Bitumen (α = 5% and $p < 0.05$). Detailed test results are given in Table A.8 of the Appendix.

4.4.1.2 14 Days curing period: The variation of UCS values (14 days curing) for soil-lime mixtures with bitumen contents is shown in Fig. 4.8.

There was an increase in the 14 day UCS value from 270 to 1125 kN/m^{2}at 6 % bitumen emulsion plus 10 % lime. The UCS values of laterite treatment decreases beyond 6 % bitumen emulsion. Detailed test results are given in Table 3.11 of chapter 3

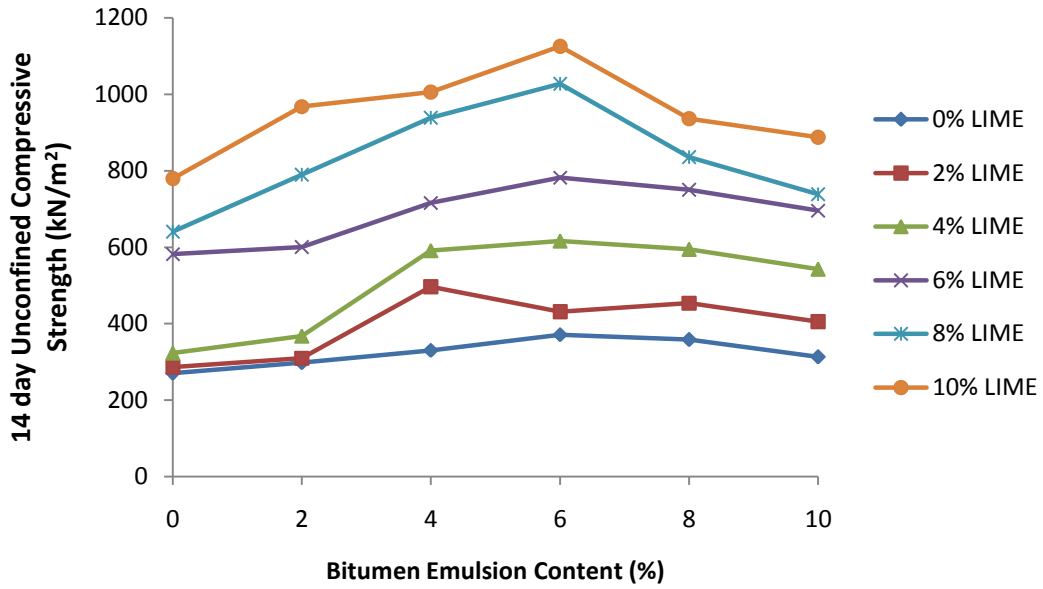


Figure 4.8: Variation of UCS (14 day curing) of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the 14 days UCS result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}= 15.2 >F_{CRIT} = 2.6) for Bitumen and (F_{CAL}= 116.6 >F_{CRIT} = 2.6) for Lime. The effect of Lime on the 14 days UCS value was more pronounced than that of Bitumen (α = 5% and $p < 0.05$). Detailed test results are given in Table A.9 of the appendix.

4.3.1.3 28 Days curing period: The variation of UCS values (28 days curing) for soil-lime mixtures with bitumen contents is shown in Figure 4.9. The same trend of increase in UCS values recorded for specimens cured for 7 and 14 days was also observed for 28 days curing period except that the UCS values increases with curing period. A peak value of 1839 kN/m^2 was recorded when laterite was treated with 6% bitumen emulsion with 10% lime, as against value of 305 kN/m^{2}obtained for the natural laterite. At 6% bitumen emulsion with all the percentages of the lime, it is where we have the optimal blend and decreased beyond. Detailed test results are given in Table 3.12 of chapter 3

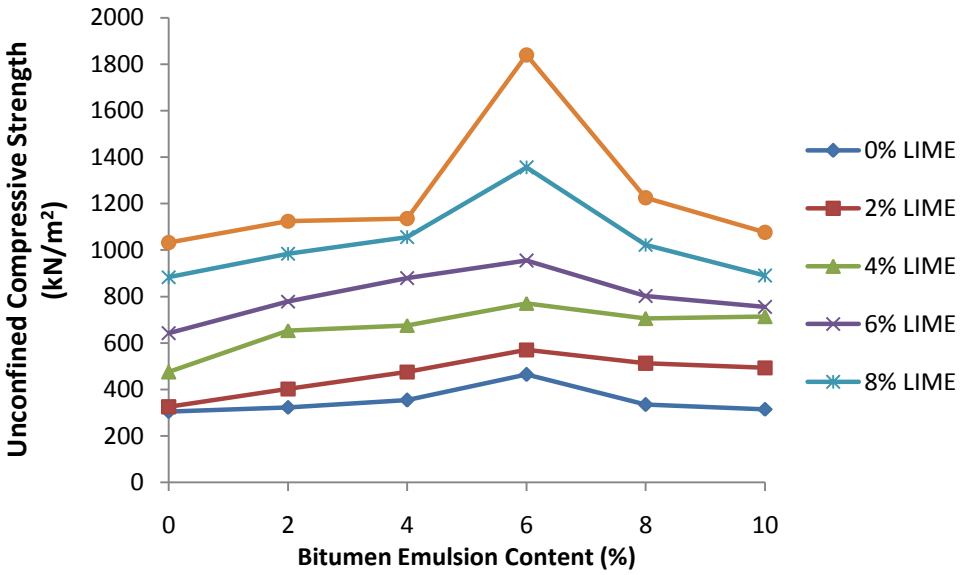


Figure 4.9: Variation of UCS (28 day curing) of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the 28 days UCS result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}=8.8>F_{CRIT} = 2.6) for Bitumen and (F_{CAL}=61.6 > F_{CRIT}= 2.6) for Lime. The effect of Lime on the 28 days UCS result was more pronounced than that of Bitumen (α =5% and p < 0.05). Detailed test results are given in Table A.10 of the appendix.

4.4.1.4 7 Days curing and 7 days soaking: The variation of UCS values (7 days curing, 7 days soaking) of soil-lime mixtures with bitumen content is shown in Figure 4.10. The UCS value increases with increase in the admixture and the value increase from a natural value of 39 to a peak value of 587 kN/m^{2}but addition of admixture beyond 6 % bitumen emulsion did not have any impact on the UCS value because greater percentage of the admixture occupied the soil matrix which leads to decrease in strength. Detailed test results are given in Table 3.13 of chapter 3

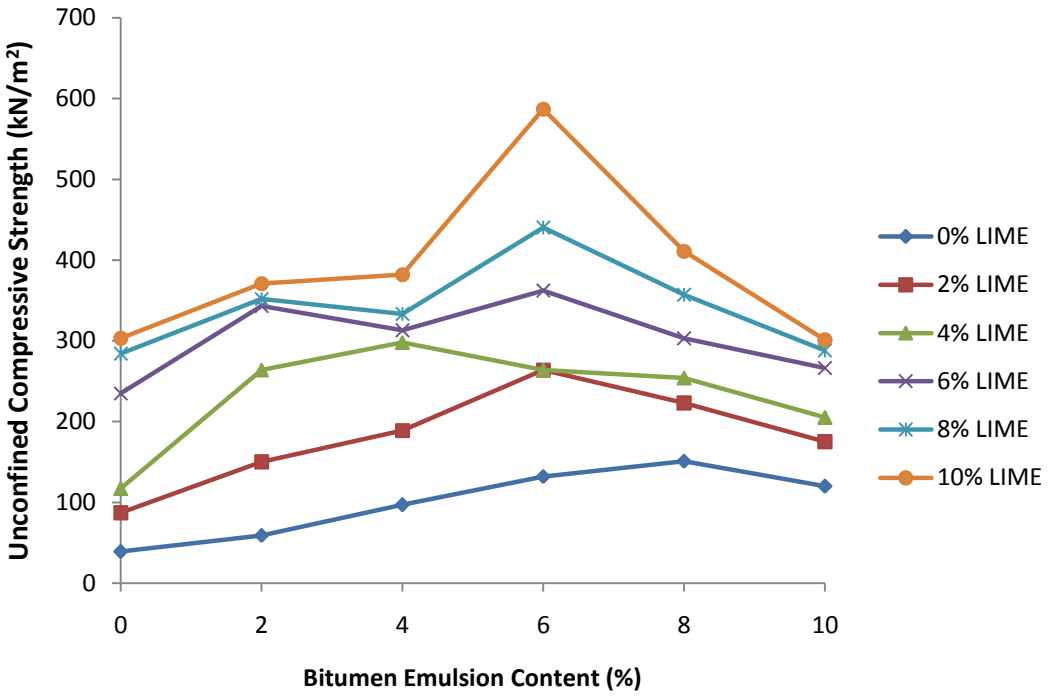


Figure 4.10: Variation of UCS (7 day curing and 7 day soaking) of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the 7 day curing and 7 day soaking UCS result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}= 11.4 >F_{CRIT}= 2.6) for Bitumen and (F_{CAL}= 44.1 >F_{CRIT} = 2.6) for Lime. The effect of Lime in the 28 days UCS result was more pronounced than that of Bitumen (α = 5% and $p < 0.05$). Detailed test results are given in Table A.11 of the appendix.

4.4.2 California bearing ratio (CBR) of the stabilized soil

4.4.2.1 Soaked CBR

The variation of CBR (soaked for 24 hours) of laterite treated with lime and bitumen emulsion isshown in Figure 4.11. The CBR value increased with additive content to a peak value of 47% at 8% bitumen emulsion with 10% lime. The low value of CBR recorded is as a result of the ingress of water into the specimen which weakened it and reduced its strength. The CBR value obtained does not satisfy the 80

% value for base material as recorded by the Nigeria General Specification (1997) but meets the 30 % requirement for sub-base materials specified by the Nigeria General Specification (1997). Detailed test results are given in Table 3.14 of chapter 3

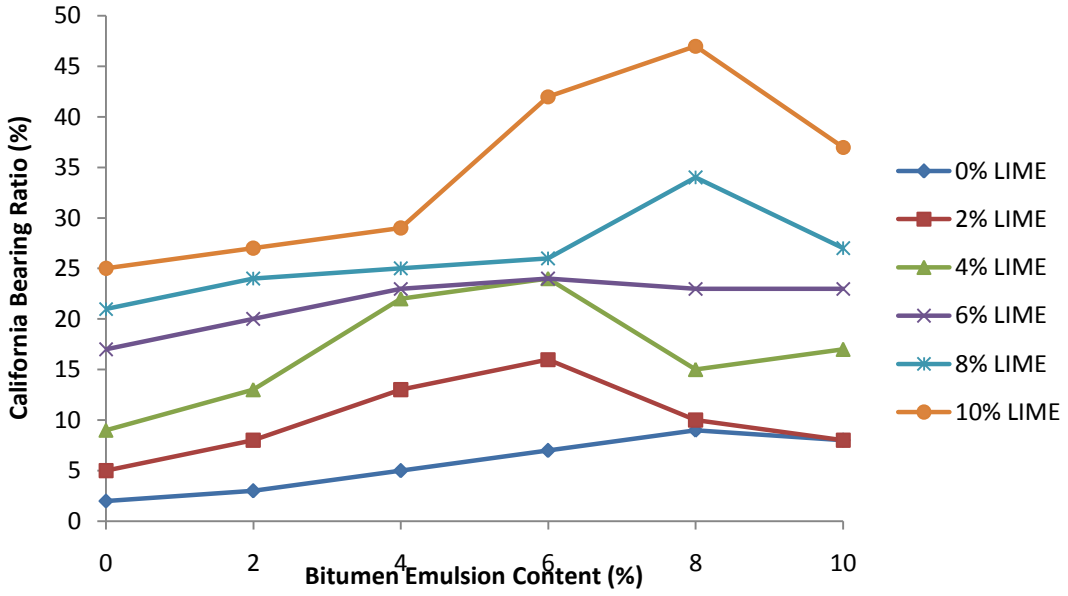


Figure 4.11: Variation of CBR (soaked) of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the CBR result (see Table A.14 of the appendix) shows that the effects of Lime and Bitumen on lateritic soil were statistically significant (F_{CAL}=7.2>F_{CRIT} = 2.6) for Bitumen and (F_{CAL}=51.4>F_{CRIT}= 2.6) for Lime. The effect of Lime in the CBR result was more pronounced than that of Bitumen (α =5 % and $p < 0.05$). Detailed test results are given in Table A.12 of the appendix.

4.5 Durability Assessment of the Stabilized Soil

Durability assessment of soil samples involves the simulation of some of the worst conditions that could be obtained in the field. It was evaluated by the immersion of specimens in water to determine resistance to loss in strength which is more acceptable for tropical regions like Nigeria (Ola, 1974). The resistance to loss in

strength determined as the ratio of the unconfined compressive strength of specimen wax-cured for 7 days, de-waxed top and bottom to allow absorption of water and later immersed in water for 7 days, to those cured for 14 days. Conventionally, an allowable 20 % loss in strength is recommended for a specimen cured for 7 day andimmersed in water for 7days (Ola, 1974; Osinubi, 1998, 1999).

The variation of resistance to loss in strength of soil-lime mixtures with bitumen contents is shown in Figure 4.12. The resistance to loss in strength increase from 14.5 for the naturallaterite to a peak value of 61.25 % at 6 % bitumen emulsion with 2 % lime content. The peak resistance to loss in strength value of 61.25 % fell short of the acceptable 80 % specified by Ola, (1974) and Osinubi, (1998, 1999). Detailed test results are given in Table 3.15 of chapter 3

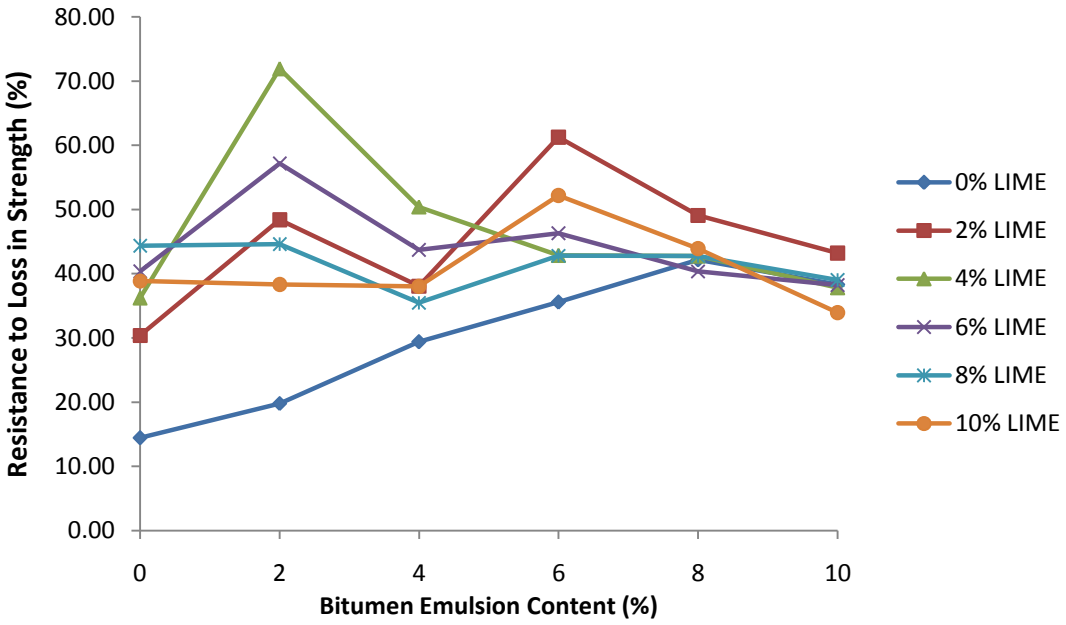


Figure 4.12: Variation of resistance to loss in strength of soil – lime mixtures with bitumen content

The two – way analysis of variance (ANOVA) test on the Resistance to loss in strength result (see Table A.14 of the appendix) shows that the effects of Lime on lateritic soil was statistically significant while that for Bitumen was not statistically

significance (F_{CAL}= 2.2 <F_{CRIT} = 2.6) for Bitumen and (F_{CAL}= 3.2 >F_{CRIT} = 2.6) for Lime. The effect of Lime in the Resistance to loss in strength result was more pronounced than that of Bitumen (α =5% and $p < 0.05$). Detailed test results are given in Table A.13 of the Appendix.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In an effort to improve the suitability of lateritic soil as a pavement material, the air dried samples were treated with Lime/Bitumen in stepped concentration of 0, 2, 4, 6, 8, 10 % by dry weight of the soil. The preliminary investigation conducted on the natural lateritic soil showed that the soil belongs to the CH group using the Unified Soil Classification System (ASTM, 1992) or A-7-6(14) soil group using the AASHTO soil classification system (AASHTO, 1986).

The MDD decreased with higher additive blend as it decreased from 1.67 to 1.49 mg/m^3 at 6 % Lime with 6 % Bitumen treatment. The OMC, on the other hand, decreased with higher bitumen content but increases with higher lime content. The decreases may be due to the effect of breakdown of the flocculated aggregates and elimination of large pores. The increase in OMC is as a result of increase in lime content is as a result more water needed to dissociate Ca^{2+} and OH^- ions to supply Ca^{2+} for the cation exchange reaction. The OMC value decreases from 13.2 % for the natural soil to a value of 12.3 % at 0% lime with 8% bitumen emulsion. This is in comformity with same trend reported by Hausmann (1990), Stephen (2006), Osinubi *et al.*(2007a) and Sani (2012).

The UCS values showed a sharp increase from 235 to 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion with 6 % lime, 6% bitumen emulsion with 8% lime and 6% bitumen emulsion with 10% lime respectively. The UCS values obtained fell short of 1710 kN/m^2 specified by TRRL (1977) as a criterion for adequate stabilization using OPC. The values of 985, 999 and 1078 kN/m^2 at 6 % bitumen emulsion plus 6 % lime, 6 % bitumen emulsion plus 8 % lime and 6 % bitumen emulsion plus 10 % lime

respectively on the other hand meet the requirement of 687–1373 kN/m^{2}for sub-base as specified byIngles and Metcalf (1972).

The 28 days curing period UCS values obtained showed that the Lime/Bitumen blend had a long time advantage in terms of strength gain. There were tremendous increases in the values of UCS from their natural states. A peak value of 1839 kN/m^2 was recorded at 6 % bitumen emulsion with 10 % lime from a natural value of 305 kN/m^2.

The CBR value increased with higher additive content to a peak value of 47 % at 8 % bitumen emulsion with 10 % lime. The low value of CBR recorded is as a result of the ingress of water into the specimen which weakened it and reduced its strength. The CBR value obtained did not meet the 80 % value for Base material as recorded by the Nigeria General Specification (1997), but meets the 30 % requirement for sub-base materials specified by the Nigeria General Specification (1997).

The resistance to loss in strength increased from 14.5 for the natural to a peak value of 61.25 % at 6 % bitumen emulsion with 2 % lime content. The peak resistance to loss in strength value of 61.25 % fell short of the acceptable 80 % specified by Ola (1974) and Osinubi(1998, 1999).

Based on the test results presented, the stabilized soil using bitumen emulsion (MS2) and hydrated lime, meet the specification for mainly sub-base. Therefore, bitumen emulsion stabilization and lime modifications may be employed to improve A-7-6 laterite as only fill or sub-base material on very lightly trafficked roads with restriction on the wheel loads.

5.2 Recommendation

It is recommended that 6 % bitumen emulsion with 6 % lime be used as sub-base treatment for light traffic pavement roads.

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APPENDIX

Table A.1: Atterberg Limit of the Natural Soil

TYPE OF TEST	**LL**	**LL**	**LL**	**LL**		**PL**	**PL**
CONTAINER No	15	2	5	30		1	22
Wt of WET SOIL + CONTAINER - W1,g	28.2	21.0	21.1	22.0		25	22
Wt of DRIED SOIL + CONTAINER - W2,g	21.7	17.2	17.6	17.8		23	20
Wt of DRY CONTAINER, W3,g	8.5	8.4	8.7	8.6		12	12
WT OF MOISTURE, W1 - W2,g	6.5	3.8	3.5	4.2		2	2
Wt of DRIED SOIL, W2 - W3,g	13.2	8.8	8.9	9.2		11	8
MOISTURE CONTENT, (W1-W2)x100/(W2-W3)	**49.2**	**43.4**	**39.3**	**45.6.**		**18.2**	**25.0**
No of BLOWS	**45**	**35**	**25**	**15**		21.6	

TableA.2: Compaction Test Result for Natural Soil

Vol of water added	**300CC**		**360CC**		**420CC**		**480CC**		**540CC**	
Wt, mould + wet soil, g [W2]	4844		4938		4960		4918		4900	
Wt, mould g [W1]	3058		3058		3058		3058		3058	
Wt, wet soil, g [W2 - W1]	1786		1880		1902		1860		1842	
Wet Density, KG/M3 [Dw]	**1.790**		**1.880**		**1.900**		**1.860**		**1.840**	
MC can No	**26**	**29**	**35**	**25**	**28**	**34**	**27**	**10**	**28**	**34**
Wt, wet soil + can, g [w2]	56	65	57	57	44	39	69	61	54	54
Wt, dry soil + can, g [w3]	52	60	52	52	40	36	62	54	48	48
Wt, can, g [w1]	12	12	12	12	13	15	15	13	15	14
Wt of moisture, g [w2 - w3]	4	5	5	5	4	3	7	7	6	6
Wt of dry soil,g [w3 - w1]	40	48	40	40	27	21	47	41	33	34
Moisture Cont,% [[100x[w2-w3]/[w3-w1]]]	*10.0*	*10.4*	*12.5*	*12.5*	*14.8*	*143*	*14.9*	*17.1*	*18.2*	*17.6*
Moisture content (average), **MC**	**10.2**		**12.5**		**14.6**		**16.0**		**17.9**	
Dry Density, Dd [Dwx100]/[100+MC]	**1.624**		**1.671**		**1.659**		**1.604**		**1.560**	

Table A.3: Analysis of Variance for Liquid Limit Test Result for Soil-Lime – Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	289	48.16667	2.5666667	0% LIME	6	261	43.5	50.3
2% BITUMEN	6	283	47.16667	16.566667	2% LIME	6	254	42.33333	19.86667
4% BITUMEN	6	263	43.83333	8.5666667	4% LIME	6	240	40	22.8
6% BITUMEN	6	246	41	7.6	6% LIME	6	276	46	17.2
8% BITUMEN	6	235	39.16667	5.7666667	8% LIME	6	246	41	12
10% BITUMEN	6	222	37	9.2	10% LIME	6	261	43.5	19.9

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	593.8889	5	118.7778	25.500954	4.57E-09	2.602987
LIME	134.8889	5	26.97778	5.7919847	0.001103	2.602987
Error	116.4444	25	4.657778			
Total	845.2222	35				

Table A.4: Analysis of Variance for Plastic Limit Test Result for Soil-Lime – Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	164	27.33333	17.86667	0% LIME	6	152	25.33333	3.466667
2% BITUMEN	6	164	27.33333	12.26667	2% LIME	6	159	26.5	2.7
4% BITUMEN	6	172	28.66667	10.66667	4% LIME	6	161	26.83333	3.366667
6% BITUMEN	6	173	28.83333	8.566667	6% LIME	6	176	29.33333	4.266667
8% BITUMEN	6	179	29.83333	4.966667	8% LIME	6	195	32.5	0.3
10% BITUMEN	6	180	30	4.4	10% LIME	6	189	31.5	1.9

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	40.33333	5	8.066667	5.084034	0.002375	2.602987
LIME	254	5	50.8	32.01681	4.25E-10	2.602987
Error	39.66667	25	1.586667			
Total	334	35				

Table A.5: Analysis of Variance for PlasticityIndex Test Result for Soil-Lime – Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	125	20.83333	19.36667	0% LIME	6	109	18.16667	77.36667
2% BITUMEN	6	119	19.83333	40.56667	2% LIME	6	95	15.83333	35.76667
4% BITUMEN	6	91	15.16667	25.36667	4% LIME	6	79	13.16667	41.76667
6% BITUMEN	6	73	12.16667	13.36667	6% LIME	6	100	16.66667	37.46667
8% BITUMEN	6	56	9.333333	6.666667	8% LIME	6	51	8.5	13.1
10% BITUMEN	6	42	7	4	10% LIME	6	72	12	16

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	937.2222	5	187.4444	27.54735	2.06E-09	2.602987
LIME	376.5556	5	75.31111	11.06793	1.06E-05	2.602987
Error	170.1111	25	6.804444			
Total	1483.889	35				

Table A.6: Analysis of Variance for Maximum Dry Density Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	9.73	1.621667	0.001097	0% LIME	6	9.47	1.578333	0.002777
2% BITUMEN	6	9.41	1.568333	0.000537	2% LIME	6	9.37	1.561667	0.002657
4% BITUMEN	6	9.27	1.545	0.00059	4% LIME	6	9.26	1.543333	0.001827
6% BITUMEN	6	9.1	1.516667	0.000227	6% LIME	6	9.15	1.525	0.00107
8% BITUMEN	6	9.05	1.508333	0.000457	8% LIME	6	9.18	1.53	0.00164
10% BITUMEN	6	9.15	1.525	0.00027	10% LIME	6	9.28	1.546667	0.001547

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	0.053592	5	0.010718	67.12944	1.12498E-13	2.602987
LIME	0.011892	5	0.002378	14.89562	8.20029E-07	2.602987
Error	0.003992	25	0.00016			
Total	0.069475	35				

Table A.7: Analysis of Variance for Optimum Moisture Content Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	83.3	13.88333	0.249667	0% LIME	6	76.4	12.73333	0.110667
2% BITUMEN	6	82.1	13.68333	0.241667	2% LIME	6	79	13.16667	0.066667
4% BITUMEN	6	80.9	13.48333	0.233667	4% LIME	6	79.9	13.31667	0.069667
6% BITUMEN	6	79.7	13.28333	0.233667	6% LIME	6	81.5	13.58333	0.097667
8% BITUMEN	6	78.8	13.13333	0.334667	8% LIME	6	85.4	14.23333	0.042667
10% BITUMEN	6	80	13.33333	0.358667	10% LIME	6	82.6	13.76667	0.114667

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	2.3	5	0.46	54.7619	1.156E-12	2.602987
LIME	8.05	5	1.61	191.6667	4.301E-19	2.602987
Error	0.21	25	0.0084			
Total	10.56	35				

Table A.8: Analysis of Variance for Unconfined Compressive Strength (7 days curing) Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	3167	527.8333	69148.57	0% LIME	6	1610	268.3333	961.0667
2% BITUMEN	6	3669	611.5	85737.1	2% LIME	6	1876	312.6667	663.8667
4% BITUMEN	6	3764	627.3333	91589.87	4% LIME	6	3222	537	8322.8
6% BITUMEN	6	4298	716.3333	125953.5	6% LIME	6	5142	857	8641.6
8% BITUMEN	6	3694	615.6667	65702.67	8% LIME	6	5195	865.8333	15591.77
10% BITUMEN	6	3213	535.5	35482.3	10% LIME	6	4760	793.3333	23513.07

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	142991.8	5	28598.36	4.914516	0.0028721	2.602987
LIME	2222591	5	444518.2	76.3887	2.515E-14	2.602987
Error	145479	25	5819.161			
Total	2511062	35				

Table A.9: Analysis of Variance for Unconfined Compressive Strength (14 days curing) Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	2880	480	46358	0% LIME	6	1940	323.3333	1422.267
2% BITUMEN	6	3332	555.3333	77753.47	2% LIME	6	2382	397	6832
4% BITUMEN	6	4076	679.3333	67551.87	4% LIME	6	3033	505.5	16234.7
6% BITUMEN	6	4352	725.3333	95617.07	6% LIME	6	4126	687.6667	6501.467
8% BITUMEN	6	3927	654.5	50543.1	8% LIME	6	4967	827.8333	19342.97
10% BITUMEN	6	3581	596.8333	46865.37	10% LIME	6	5700	950	13476

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	239923.9	5	47984.78	15.16143	6.988E-07	2.602987
LIME	1844321	5	368864.2	116.5476	1.722E-16	2.602987
Error	79123.11	25	3164.924			
Total	2163368	35				

Table A.10: Analysis of Variance for Unconfined Compressive Strength (28 days curing) Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	3663	610.5	89331.5	0% LIME	6	2096	349.3333	3442.667
2% BITUMEN	6	4268	711.3333	99841.07	2% LIME	6	2782	463.6667	7587.067
4% BITUMEN	6	4575	762.5	98864.3	4% LIME	6	3995	665.8333	10314.97
6% BITUMEN	6	5956	992.6667	271227.5	6% LIME	6	4812	802	11535.2
8% BITUMEN	6	4603	767.1667	106301.8	8% LIME	6	6192	1032	29991.6
10% BITUMEN	6	4244	707.3333	74207.07	10% LIME	6	7432	1238.667	90685.07

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	490030.9	5	98006.18	8.821378	6.2859E-05	2.602987
LIME	3421114	5	684222.8	61.58578	3.0308E-13	2.602987
Error	277751.9	25	11110.08			
Total	4188897	35				

Table A.11: Analysis of Variance for Unconfined Compressive Strength (7 days curing, 7 days soaking) Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	1065	177.5	12286.3	0% LIME	6	598	99.66667	1887.067
2% BITUMEN	6	1539	256.5	16023.5	2% LIME	6	1088	181.3333	3709.867
4% BITUMEN	6	1612	268.6667	11125.07	4% LIME	6	1402	233.6667	4165.067
6% BITUMEN	6	2049	341.5	25259.1	6% LIME	6	1822	303.6667	2234.267
8% BITUMEN	6	1699	283.1667	8824.967	8% LIME	6	2054	342.3333	3257.867
10% BITUMEN	6	1355	225.8333	5069.367	10% LIME	6	2355	392.5	11025.5

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	91392.81	5	18278.56	11.42257	8.1688E-06	2.602987
LIME	352936.1	5	70587.23	44.11111	1.30775E-11	2.602987
Error	40005.36	25	1600.214			
Total	484334.3	35				

Table A.12: Analysis of Variance for California Bearing Ratio (soaking) Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	79	13.16667	84.96667	0% LIME	6	34	5.666667	7.866667
2% BITUMEN	6	95	15.83333	88.56667	2% LIME	6	60	10	15.6
4% BITUMEN	6	117	19.5	78.3	4% LIME	6	100	16.66667	31.46667
6% BITUMEN	6	139	23.16667	135.3667	6% LIME	6	130	21.66667	7.066667
8% BITUMEN	6	138	23	225.2	8% LIME	6	157	26.16667	18.96667
10% BITUMEN	6	120	20	128.8	10% LIME	6	207	34.5	79.1

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	471.5556	5	94.31111	7.171342	0.0002771	2.602987
LIME	3377.222	5	675.4444	51.36026	2.387E-12	2.602987
Error	328.7778	25	13.15111			
Total	4177.556	35				

Table A.13: Analysis of Variance for Resistance to Loss in Strength Test Result for Soil-Lime –Bitumen mixes

SUMMARY	*Count*	*Sum*	*Average*	*Variance*	*SUMMARY*	*Count*	*Sum*	*Average*	*Variance*
0% BITUMEN	6	204.736	34.12266	114.4762	0% LIME	6	179.734	29.95566	119.077
2% BITUMEN	6	280.2269	46.70448	309.467	2% LIME	6	270.4932	45.08221	111.1782
4% BITUMEN	6	235.1479	39.19132	51.75961	4% LIME	6	282.0215	47.00358	173.7899
6% BITUMEN	6	281.0021	46.83369	79.03199	6% LIME	6	266.1697	44.36162	47.48295
8% BITUMEN	6	261.1234	43.52057	8.839185	8% LIME	6	249.1116	41.5186	12.69018
10% BITUMEN	6	230.5488	38.4248	8.76612	10% LIME	6	245.2551	40.87584	40.75091

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
BITUMEN	772.9366	5	154.5873	2.205984	0.0855798	2.602987
LIME	1109.791	5	221.9583	3.167378	0.0237792	2.602987
Error	1751.909	25	70.07635			
Total	3634.637	35				

Table A.14: Two – way analysis of variance result for soil-lime-bitumen mixtures.

Property	Source of Variation	Degree of Freedom	F_{CAL}	P – value	F_{CRIT}	Remarks
Liquid Limit	Bitumen	5	25.5	4.6E-09	2.6	Significant effect
	Lime	5	5.79	1.1E-03	2.6	Significant effect
Plastic Limit	Bitumen	5	5.08	2.4E-03	2.6	Significant effect
	Lime	5	32.01	4.3E-10	2.6	Significant effect
Plasticity Index	Bitumen	5	27.55	2.1E-09	2.6	Significant effect
	Lime	5	11.07	1.1E-05	2.6	Significant effect
MDD	Bitumen	5	67.1	1.1E-13	2.6	Significant effect
	Lime	5	14.9	8.2E-07	2.6	Significant effect
OMC	Bitumen	5	54.8	1.2E-12	2.6	Significant effect
	Lime	5	191.7	4.3E-19	2.6	Significant effect
7 Days UCS	Bitumen	5	4.9	2.8E-03	2.6	Significant effect
	Lime	5	76.4	2.5E-14	2.6	Significant effect
14 Days UCS	Bitumen	5	15.2	7.0E-07	2.6	Significant effect
	Lime	5	116.6	1.7E-16	2.6	Significant effect
28 Days UCS	Bitumen	5	8.8	6.3E-05	2.6	Significant effect
	Lime	5	61.6	3.0E-13	2.6	Significant effect
7 + 7 Days UCS	Bitumen	5	11.4	8.2E-06	2.6	Significant effect
	Lime	5	44.1	1.3E-11	2.6	Significant effect
CBR (Soaked)	Bitumen	5	7.2	2.8E-04	2.6	Significant effect
	Lime	5	51.4	2.4E-12	2.6	Significant effect
DURABILITY	Bitumen	5	2.2	8.6E-02	2.6	No Significant effect
	Lime	5	3.2	2.4E-02	2.6	Significant effect