

**BITUMEN STABILIZATION OF LATERITE PRETREATED WITH HEAT AS A
FLEXIBLE PAVEMENT MATERIAL**

BY

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**A DISSERTATION SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES,
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**DEPARTMENT OF CIVIL ENGINEERING,
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ZARIA, NIGERIA**

MARCH, 2017

DECLARATION

I hereby sincerely declare that this dissertation entitled “**BITUMEN STABILIZATION OF LATERITE PRETREATED WITH HEAT AS A FLEXIBLE PAVEMENT MATERIAL**” has been prepared by me and that it is the record of my research work. It has not been submitted to any previous publications for the award of any certificate, degree or higher degree. All quotations are indicated and the sources of the information appropriately acknowledged by means of references.

Abubakar, Nma Madiwu

Signature

Date

CERTIFICATION

This research dissertation entitled “**BITUMEN STABILIZATION OF LATERITE PRETREATED WITH HEAT AS A FLEXIBLE PAVEMENT MATERIAL**” by **NMA MADIWU ABUBAKAR**, meets the regulations governing the award of the degree of **Masters of Science in Civil Engineering** of Civil Engineering Department of the 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 Almighty Allah who gave me the ability to complete this work. This research dissertation is also dedicated to my family, relatives, friends and well-wishers.

ACKNOWLEDGEMENT

All praises are due to Almighty Allah who made it possible for me to accomplish this work.

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ABSTRACT

The search for an alternative to the conventional way of treating soil for use in road work, informed laterite soil sample classified as A-7-5(11) on AASHTO soil classification system, collected from a borrow pit at Shika, along Zaria-Funtua road in Kaduna State of Nigeria was pretreated with heat and stabilized with bitumen. Sample of the laterite soil subjected to heat treatment at temperatures of 37°C (room temperature), 200°C 400°C and 600°C for one hour each and stabilized with 0, 3, 6 and 9% cutback bitumen, were subjected to Atterberg's limits test, particle size analysis, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) tests. Optimum CBR value of 28.52 % corresponding to 44.8 % increase in strength above the value obtained for natural laterite, was obtained when laterite was subjected to heat at a temperature of 400°C for a period of one hour and stabilized with 6 % bitumen. Maximum 7 day UCS value of 417.23 kN/m² corresponding to 56.6 % increase in strength above value obtained for the natural laterite was obtained when the laterite was subjected to heat at a temperature of 200°C at a period of one hour and stabilized with 3 % bitumen. The highest resistance to loss in strength after 7 days soaking of 24.58% for West Africa Standard Compaction (WASC) energy was obtained when laterite was subjected to heat at a temperature of 200°C for one hour with 3 % bitumen stabilization. This falls below recommended 80% resistance to loss in strength, though, after 4 days soaking. Based on the results obtained, A-7-5(11) laterite soil, pretreated with heat at a temperature of 200°C for a duration of one hour, and stabilized with 3 % bitumen content can be used as subgrade/fill material. Its usage as sub-base or base material will require some further treatment. The results of tests from the study will serve as a guide in the selection of pretreatment temperature for effective and economic stabilization of laterite as flexible pavement materials.

CHAPTER ONE

INTRODUCTION

1.1 Background of the Study

A flexible pavement is a layered system consisting essentially of superimposed layers of selected and processed materials that are placed on the basement soil or subgrade. The main structural function of a pavement is to support the wheel loads applied to the carriageway and distribute them to the underlying subgrade. Wheel load applied to the pavement is largely transmitted to the subgrade by the lateral distribution with increasing depth of different layers of the pavements that deflects momentarily under load but rebounds to its original level on removal of load. The different layers that make up a typical flexible pavement consist of the surfacing material, the base course, the sub-base and the subgrade material. The thickness of each layer increases from the surfacing material to the sub-base course. In practice the layers are made up of different materials. It ranges from natural materials to stabilized materials and unbound/bituminous granular layer materials (O'Flaherty, 2002).

The different impact of climate change on the different sector including road construction has been reported by different researchers. Carrera *et al.* (2009) reported that higher intensity rainfall and storm can lead to base and subgrade destabilization, erosion of the road platforms, raveling and blocking of the drainage system due to accumulation of debris. Mills *et al.* (2007) in specific studies undertaken for pavements in southern Canada analysed the effects of temperature change on pavement infrastructure and results showed that rutting and longitudinal and alligator cracking would be exacerbated by high temperature. Kinsella and McGuire (2005) in study of the impact of increased rainfall on the State highways in the West of the Island in New Zealand submitted that among the significant effects observed are consequent increase of erosion and risk

of landslide due to expected increase of rainfall. Dore *et al.* (2005) show that longitudinal cracking outside the wheel tracks and transversal are only due to climate condition. And according to Guthrie *et al.*(2002) shrinkage cracking is imminent with the use of cementitious stabilization and is expected to be of particular concern in the context of climate change in that cracks would allow water to reach sensitive underlying subgrades while Saleh (2006) observed with a foam-bitumen- treated soil that the stiffness of a wet specimen can be 50% lower than that of a dry specimen and is a very important factor as the need in the light of future climate change is for material that will sustain even high level moisture. There are also shortages of conventional materials used in road construction in many developing countries in addition to their cost. The high cost also extends to crushed stones used in the construction of stone base and this is not able to keep up with the needs of their burgeoning economies. As a consequence, greater attention is now being paid to the use of locally-available substitute materials such as stabilized soils and ‘waste’ materials to meet road construction needs (Colwill and Raul, 1998; O’Flaherty, 2002; Saleh, 2006).

In practice there are a number of Stabilization techniques ranging from mechanical or granular stabilization, cement stabilization, lime and lime-pozzolan stabilization and bituminous stabilization, as identified by (Colwill and Asis, 1998; O’Flaherty, 2002), there is the need according to (Saleh, 2006; Carrera and Dawson, 2010) for alternative methods of stabilization such as the use of heat and cutback bitumen to cushion the effects of temperature.

1.2 Statement of the Problem

The challenge posed by increased rainfall in the world today will no doubt have great consequences on infrastructural development in the future. For road pavements to perform their functions effectively in the future, the effect of increased rainfall on the materials used in their

construction must be considered. Based on researchers' observations there is need for a new method of enhancing the suitability of lateritic materials for use in road work.

In light of the above, this study attempted to evaluate the potential of lateritic soil treated with regulated heat of Room temperature (37°C), 200°C, 400°C and 600°C and cutback bitumen as an alternative pavement material. This treatment (stabilization technique) was intended to reduce the influence of water on pavement making materials. This will be achieved by waterproofing action of cutback bitumen and use of heat to make the clay fraction inactive.

1.3 Justification of the Study

The study would enhance the suitability of laterite materials and avoid the need to import same, which consequently can lead to more cost. The study would also provide pavement material that would withstand high intensity rainfall due to climate change.

The economic justification of using bitumen as a stabilizer can be attributed to confirmation by (Oshinowo *et al*,1982; Sheik,2003; FMSMD, 2006; FMSMD, 2015; Adebayo and Jimoh, 2015) of a huge deposit of untapped bitumen of high grade in Nigeria and reason advanced by (Osinubi and Oyelakin ,2012) that the cost of blending cement with the soil is usually very expensive and savings due to reduced maintenance cost throughout the life of the pavement.

1.4 Aim and Objectives

1.4.1 Aim

The main aim of this study was to investigate the suitability of lateritic soil treated with heat and admixed with cutback bitumen, for use as subgrade materials.

1.4.2 Objectives

1. To determine the effect of heat on the Atterberg limit of the laterite.
2. To determine the compaction characteristics of laterite treated with heat and cut-back bitumen.
3. To determine the strength indices (UCS, CBR) of laterite treated with heat and cut-back bitumen.
4. To determine the durability of laterite treated with heat and cutback bitumen.

1.5 Scope of Work

The study was limited to determining the effect of heat and cutback bitumen on the index properties of lateritic soil; laboratory determination of compaction characteristics (Maximum Dry Density-MDD and Optimum Moisture Content-OMC) of both the natural soil and soil treated with heat and cutback bitumen; laboratory determination of UCS and CBR of both the natural laterite and the laterite treated with heat and cut-back bitumen; and laboratory determination of the durability of both the natural and stabilized soil.

CHAPTER TWO

LITERATURE REVIEW

2.1 Laterite and its Usage in Flexible Pavement Work

The most common residual soil profile is the lateritic weathering profile (Alhassan *et al*, 2012). Gidigasu and Kuma (1987) defined these soil profiles as those in which lateritic horizons exists or are capable of developing under favorable conditions. According to Gidigasu (1976) Laterite is defined as a soil group, which is commonly found in the leached soils of humid tropics and is formed under weathering systems that cause the process of laterization. Laterites are the most common tropically pedogenic surface deposits in Australasia, Africa and South America Gidigasu (1976). Osinubi (2004) asserted that the geotechnical characteristics and field performance of lateritic soils are influenced considerably by their pedogenesis, degree of weathering, morphological characteristics, chemical and mineral compositions as well as prevailing environmental conditions. Alhassan and Mustapha (2012) categorized lateritic weathering profile, derived from granite basement into three major horizons below the humus stained top soil: the first horizon is the sesquioxide rich lateritic horizon (sometimes gravelly and/or hardened in-situ as crust); the second horizon is the mottled zone with evidence of enrichment of sesquioxide sand; the third horizon which overlies the parent rock is referred to as the pallid or leached zone and contains rocks suffering from chemical and mineralogical changes, but retaining their physical appearance.

According to Maignien (1966) 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 (Maignien, 1966).

Wild (2012) describes laterite as hard materials, rich in iron oxides. This hardness is retained even when the material is immersed in water. The iron occurs mainly as goethite, hematite and amorphous iron oxides. The material is usually coloured reddish brown with a moderately high density (2.5 to 3.6 g/cc) and usually contains secondary aluminum. The silica content is generally low, but some quartz and sometimes Kaolinite is present. Laterite often occurs on remnants and old land surfaces. Regrettably the word laterite has been used to describe a wide range of materials as noted by Wild (2012).

Furthermore, Fadamiro and Ogunsemi (2010) defined laterite as a porous soil ranging from soft earthy material to hard rock, which ranges in colour from white to dark red depending on the amount of iron in the soil. They explained that it is found below the earth surface and chemically made of silicate and alumina, which is formed by weathering of rocks, hence, giving rise to many variations of laterite in composition and properties.

According to Ola (1983) laterite is defined as the products of tropical weathering with red, reddish brown, and dark brown colour, with or without nodules or concreting and generally (but not exclusively) found below hardened ferruginous crust or hard pan.

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 (Maignien, 1966).

Laterite formation process can be categorized into three major stages namely; decomposition, leaching and dehydration/desiccation. The first stage is decomposition which is characterized by physical and chemical break down of primary minerals and the release of constituent elements. The second stage involves the leaching, under appropriate drainage conditions of combined silica and bases and the relative accumulation or enrichment from outside sources (absolute accumulation) of oxides and hydroxides of sesquioxides (mainly Al_2O_3 , and Fe_2O_3 , the most resistant components to leaching). The third stage (dehydration or desiccation) involves partial or complete dehydration (Sometimes involving hardening) of the sesquioxide rich materials and secondary minerals (Joel & Edeh, 2015).

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 (Maignien, 1966).

According to Amadi (2011), Laterite and Lateritic soils form a group comprising a wide variety of red, brown and yellow fine-grained residual soil of light texture as well as nodular gravel and cement soils. They may vary from a loose material to a massive rock. They are characterized by the presence of iron and aluminium oxides or hydroxides, particularly those of

iron, which gives the colour to the soil. For engineering purposes, the term “laterite” is confined to the coarse grained vermicular concrete materials, including massive laterite. The term “laterite soil” refers to material with lower concentration of oxides (Amadi, 2011). Laterite is a common construction material available in almost all the countries of the humid tropics of the world (Alhassan *et al*, 2014)

Laterite soils can be used as base course for roads, in some cases, without any improvement, but there are other cases where laterization process is not complete and soil groups such as laterite, lateritic and laterized soils are formed (Mustapha *et al.*, 2014). These soils may contain substantial amount of silica in the form of clay silicate minerals and could affect their strength and the stability. According to Ola (1983) the lower the silicon-sesquioxide ratio ($\text{SiO}_2/\text{Al}_2\text{O}_3+\text{Fe}_2\text{O}_3$) of the materials, the more advanced the laterization process is likely to be. Generally, ratios of silicon-sesquioxide that are less than 1.33, are indicative of laterites, those between 1.33 and 2.00 are lateritic soils, while those greater than 2.00 are non-laterites (Alhassan, 2006). However, according to (Pinard *et al*, 2014) Lateritic material is a generic term to refer to either laterite or lateritic soil.

Lateritic soils are mostly recommended for use as sub-base and base course materials in highway construction. They work well in pavement construction, particularly when their special characteristics are carefully recognized. As a result of its structural strength, it can be used as a very suitable subgrade in highway construction. The harden type of laterite can be used as a good base course material in highway construction (Amadi, 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. One of the main uses of laterites for construction purposes is the

production of Compressed Earth Blocks (CEB) (lemouagna *et al*, 2011). Laterite crusts were originally widely used for the construction of monuments and dwellings (Maignien, 1966).

2.2. Bituminous Materials

Bituminous materials are used widely all over the world in highway construction. These hydrocarbons are found in natural deposits or are obtained as a product of the distillation of crude petroleum. The bituminous materials used in highway construction are either asphalts or tars.

All bituminous materials consist primarily of bitumen and have strong adhesive properties with colors ranging from dark brown to black. They vary in consistency from liquid to solid; thus they are divided into liquids, semisolids, and solids. The solid form is usually hard and brittle at normal temperatures but will flow when subjected to long, continuous loading. The liquid form is obtained from the semisolid or solid forms by heating, dissolving in solvents, or breaking the material into minute particles and dispersing them in water with an emulsifier to form an asphalt emulsion.

The liquid asphalt obtained by fluxing are referred to as asphalt cutbacks. The asphalt cutbacks are slow-curing asphalts, medium-curing cutback asphalts, and rapid-curing cutback asphalts. They are used mainly in cold-laid plant mixes, road mixes (mixed-in-place), and as surface treatments (Garber and Hoel, 2009).

2.2.1 Slow-Curing (SC) Asphalts

Slow-curing (SC) asphalts can be obtained directly as slow-curing straight run asphalts through the distillation of crude petroleum or as slow-curing cutback asphalts by “cutting back

” asphalt cement with a heavy distillate, such as diesel oil. They have lower viscosities than asphalt cement and are very slow to harden. Slow-curing asphalts usually are designated as SC-70, SC-250, SC-800, or SC-3000, where the numbers relate to the approximate kinematic viscosity in centistokes at 60°C (140°F), (Garber and Hoel, 2009).

2.2.2 Medium-Curing (MC) Asphalts

Medium-curing (MC) asphalts are produced by fluxing, or cutting back, the residual asphalt (usually 120 to 150 penetration) with light fuel oil or kerosene. The term medium refers to the medium volatility of the kerosene-type diluter used. Medium curing cutback asphalts harden faster than slow-curing liquid asphalts, although consistencies of the different grades are similar to those of the slow-curing asphalts. However, the MC-30 is a unique grade in this series as it is very fluid and has no counterpart in the SC and RC series. The fluidity of medium-curing asphalt depends on the amount of solvent in the material. MC-3000, for example, may have only 20% of the solvent by volume, whereas MC-70 may have up to 45%. These medium-curing asphalts can be used for the construction of pavement bases, surfaces, and surface treatments (Garber and Hoel, 2009).

2.2.3 Rapid-Curing Cutbacks Asphalts

Rapid-curing (RC) cutback asphalts are produced by blending asphalt cement with a petroleum distillate that will evaporate easily, thereby facilitating a quick change from the liquid form at the time of application to the consistency of the original asphalt cement. Gasoline or naphtha generally is used as the solvent for this series of asphalts. The grade of rapid-curing asphalt required dictates the amount of solvent to be added to the residual asphalt cement. For example, RC-3000 requires about 15% of distillate,

whereas RC-70 requires about 40%. These grades of asphalt can be used for jobs similar to those for which the MC series is used (Garber and Hoel, 2009).

2.2.4 Sources of Asphalts in Nigeria and the World

Asphalt is obtained from seeps or pools of natural deposits in different parts of the world or as a product of the distillation of crude petroleum.

In Nigeria the only local source of bitumen at present is at the Kaduna refinery which is also based on external sources, because it is a by-product of imported heavy crude from Venezuela, whereas Nigeria can boast of a huge deposit of tar sand in Ilubirin, Agbabu and Loda, Ondo State, which is still largely untapped (Adebayo and Jimoh, 2015).

Nigeria has the second largest deposit of bitumen in the world, spanning approximately 120km. The bitumen typically occurs both on the surface and sub-surface. The estimated probable reserves of bitumen in Ondo State (Southwest region of Nigeria) is 16 billion barrels, while that of tar sands and heavy oil is estimated at 42 billion barrels, almost as twice the amount of existing reserves of crude petroleum (FMSMD,2015).

The Nigerian tar sand deposit is characterized to yield bitumen of 25% and designated as good to medium grade for reasonable economic investment since the yield is more than 5-10%. (Oshinowo *et al*,1982; Sheik,2003; FMSMD, 2006; Adebayo and Jimoh, 2015). The Nigerian tar sand deposit is equally appropriate in term of pavement surfacing work in Nigeria (Adebayo and Jimoh, 2015).

It is important to note that bitumen extracted from the Nigeria tar sand has been used to fabricate cold patch mats used in pot hole sealing, and conclusion was drawn by (Adebayo and Jimoh, 2015) that for economic consideration, the locally produced bituminous cold patch mats produced at a cost of N 1,671.90 per square meter has about 300% cost advantage over south

African BRP mats and may even imply a much higher potential of generating more employment if manufactured in large and commercial quantity (Adebayo and Jimoh, 2015)

Natural deposits of asphalt occur as either native asphalt or rock asphalt. The largest deposit of native asphalt is known to have existed in Iraq several thousand years ago.

Native asphalts also have been found in Trinidad, Bermuda, and the La Brea asphalt pits in Los Angeles, California (Garber and Hoel, 2009). Native asphalt (after being softened with petroleum fluxes) was at one time used extensively as binders in highway construction. The properties of native asphalt vary from one deposit to another, particularly with respect to the amount of insoluble material the asphalt contains. The Trinidad deposit, for example, contains about 40% insoluble organic and inorganic materials, whereas the Bermuda material contains about 6% of such material.

Rock asphalt is a natural deposit of sandstone or limestone rocks filled with asphalt. Deposits have been found in California, Texas, Oklahoma, and Alabama. The amount of asphalt varies from one deposit to another and can be as low as 4.5 percent and as high as 18 percent. Rock asphalt can be used to surface roads after the mined or quarried material has been suitably processed. This process includes adding suitable mineral aggregates, asphalt binder, and oil, which facilitates the flowing of the material. Rock asphalt is not used widely because of its high transportation costs.

Another source of the asphalt materials is from the distillation of petroleum and are in the form of different types of asphalts, which include asphalt cements, slow-curing liquid asphalts, medium-curing liquid asphalts, rapid-curing liquid asphalts, and asphalt emulsions.

The quantity of asphalt obtained from crude petroleum is dependent on the American

Petroleum Institute (API) gravity of the petroleum. In general, large quantities of asphalt are obtained from crude petroleum with low API gravity (Garber and Hoel, 2009).

2.2.5 Properties of Bitumen

The properties of asphalt materials pertinent to pavement construction can be classified into four main categories viz; Consistency, Ageing and temperature sustainability, Rate of curing and Resistance to water action (Garber and Hoel, 2009).

2.2.6 Consistency

The consistency properties of an asphalt material according to (Garber and Hoel, 2009) usually are considered under two conditions: (1) variation of consistency with temperature and (2) consistency at a specified temperature.

2.2.6.1 Variation of Consistency with Temperature

The consistency of any asphalt material changes as the temperature varies. The change in consistency of different asphalt materials may differ considerably even for the same amount of temperature change. For example, if a sample of blown semisolid asphalt and a sample of semisolid regular paving-grade asphalt with the same consistency at a given temperature are heated to a high enough temperature, the consistencies of the two materials will be different at the high temperatures with the regular paving-grade asphalt being much softer than the blown asphalt. Further increase in temperature eventually will result in the liquefaction of the paving asphalt at a temperature much lower than that at which the blown asphalt liquefies. If these two asphalts then are cooled down gradually to about the freezing temperature of water, the blown asphalt will be much softer than the paving-grade asphalt. Thus, the consistency of the blown asphalt is affected less by temperature changes than the consistency of regular paving-grade asphalt. This property of asphalt materials is known as

temperature susceptibility. The temperature susceptibility of a given asphalt depends on the crude oil from which the asphalt is obtained, although variation in temperature susceptibility of paving-grade asphalts from different crudes is not as high as that between regular paving-grade asphalt and blown asphalt (Garber and Hoel, 2009).

2.2.6.2 Consistency at a Specified Temperature

As stated earlier, the consistency of an asphalt material will vary from solid to liquid depending on the temperature of the material. It is therefore essential that when the consistency of an asphalt material is given; the associated temperature also should be given (Garber and Hoel, 2009).

2.2.7 Aging and Temperature Sustainability

When asphaltic materials are exposed to environmental elements, natural deterioration gradually takes place, and the materials eventually lose their plasticity and become brittle. This change is caused primarily by chemical and physical reactions that take place in the material. This natural deterioration of the asphalt material is known as weathering. For paving asphalt to act successfully as a binder, the weathering must be minimized as much as possible. The ability of an asphalt material to resist weathering is described as the durability of the material. Some of the factors that influence weathering are oxidation, volatilization, temperature, and exposed surface area (Garber and Hoel, 2009).

2.2.8 Rate of Curing

Curing is defined as the process through which an asphalt material increases its consistency as it loses solvent by evaporation. The rate of curing of any cutback asphalt material depends on the distillate used in the cutting-back process. This is an important characteristic of cutback

materials, since the rate of curing indicates the time that should elapse before a cutback will attain a consistency that is thick enough for the binder to perform satisfactorily. The rate of curing is affected by both inherent and external factors.

The important inherent factors are; volatility of the solvent, quantity of the solvent in the cutback and consistency of the base material. The more volatile the solvent is, the faster it can evaporate from the asphalt material, and therefore, the higher the curing rate of the material. This is why gasoline and naphtha are used for rapid-curing cutbacks, whereas light fuel oil and kerosene are used for medium-curing cutbacks.

For any given type of solvent, the smaller the quantity used, the less time is required for it to evaporate, and therefore, the faster the asphalt material will cure. Also, the higher the penetration of the base asphalt, the longer it takes for the asphalt cutback to cure.

The important external factors that affect curing rate are temperature, ratio of surface area to volume and wind velocity across exposed surface. These three external forces are related directly to the rate of curing in that the higher these factors are, the higher the rate of curing. Unfortunately, these factors cannot be controlled or predicted in the field, which makes it extremely difficult to predict the expected curing time. The curing rates of different asphalt materials usually are compared with the assumption that the external factors are held constant (Garber and Hoel, 2009).

2.2.9 Resistance to Water Action

When asphalt materials are used in pavement construction, it is important that the asphalt continues to adhere to the aggregates even with the presence of water. If this bond between the asphalt and the aggregates is lost, the asphalt will strip from the

aggregates, resulting in the deterioration of the pavement. The asphalt therefore must sustain its ability to adhere to the aggregates even in the presence of water. In hot-mix, hot-laid asphalt concrete, where the aggregates are thoroughly dried before mixing, stripping does not normally occur and so no preventive action is usually taken. However, when water is added to a hot-mix, cold-laid asphalt concrete, commercial antistrip additives usually are added to improve the asphalt's ability to adhere to the aggregates (Garber and Hoel, 2009).

2.3 Soil Stabilization

Soil stabilization is any treatment (including, technically, compaction) applied to a soil to improve its strength and reduce its vulnerability to water; if the treated soil is able to withstand the stresses imposed on it by traffic under all weather conditions without deformation, then it is generally regarded as stable. This definition applies irrespective of whether the treatment is applied to a soil in situ or after the soil has been removed and placed in a pavement or embankment (O'Flaherty, 2002).

In practice, the main methods by which soils are stabilized for road purposes according to (O'Flaherty, 2002) are: (1) mechanical or granular stabilization; (2) cement stabilization; (3) lime and lime-pozzolan stabilization; and (4) bituminous stabilization.

A guide to selecting appropriate method of stabilization as specified by O'Flaherty (2002) is presented in Figure 2.1. The parameters used in determining appropriate method of stabilization are particle size and plasticity index. Materials that are usually suitable for bitumen stabilization are those with less than 25% passing 75 μ m sieve and with plasticity index (PI) less or equal to 10. Suitability for bitumen stabilization of materials with more than 25% passing 75 μ m and 10<PI<20 is doubtful.

For cement and cementitious blend, more than 25% the material must pass 75µm with $PI \leq 10$ and less than 25% material passing 75µm with $PI > 10$ requirements are usually suitable, while suitability of those materials with more than 25% passing 75µm and $10 < PI < 20$ is doubtful.

For lime stabilization, the usual suitable requirements for the materials are more than 25% passing 75µm with $PI \geq 20$ and less than 25% passing 75µm with $PI > 10$. For those materials with more than 25% passing 75µm with $PI \leq 10$ and less than 25% passing 75µm with $PI \leq 10$, the suitability are doubtful.

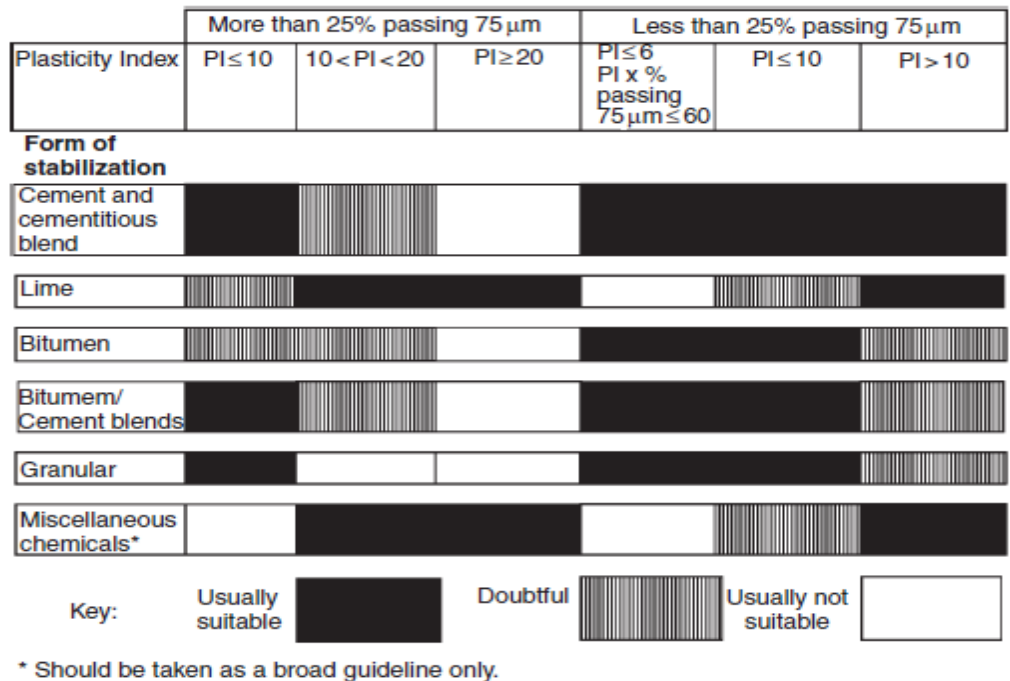


Fig. 2.1 Guide to selecting a method of soil stabilization
(After O’Flaherty ,2002)

2.3.1 Use of Bitumen in Soil Stabilization

Bituminous stabilization of soils is well documented (Ola, 1975; Ola, 1978; Harris *et al*, 1983; Osinubi and Bajeh, 1994, Muhammad, 2014; Yathish143, 2014).

Ola (1978) carried out a study on geotechnical properties and behavior of some stabilized Nigerian lateritic soils and concluded that bituminous stabilization should be used only for the sub-base course materials of highly traffic roads of the area studied. He recommended stabilization of A-1, A-2 and A-3 with 2 to 3% bitumen.

Harris *et al.* (1983) conducted bituminous stabilization of fines during construction of Baiomoro-Gashua road in Nigeria, and concluded that the require stabilities could be achieved with 5.5% RC2 and 5% water at 30 days curing.

Osinubi and Bajeh (1994) carried out bituminous stabilization of laterite using up to 8% of the stabilizer, and concluded that optimum stabilizer content of about 5% by weight of the dry soil is adequate for stabilizing the A-7-6 lateritic, using bitumen emulsion and cut-back bitumen.

However, the use of bitumen emulsion for the stabilization of laterite soils is not well documented. Elsewhere emulsions have been very successfully used on roads carrying up to 1500 vehicles per day (Osinubi and Bajeh, 1994). According to O'Flaherty (2002), the mechanisms involved in the stabilization of a soil with a bituminous material (usually hot bitumen, cutback bitumen, or anionic or cationic bitumen emulsion) are very different from those involved with cement or lime. With coarse-grained non-plastic soils, the main function of the bituminous material is to add cohesive strength. Thus, the stabilization emphasis with granular soils such as gravels and sands, and sandy soils, is upon the thorough admixing of an optimum amount of binder so that particles are thinly coated with binder and held together without loss of particle interlock. This mechanism was ascertained by Ola (1978), who found that for the sand-

bitumen mixtures used, the optimum bitumen content was about 2%, while triaxial test results indicated that with 4% bitumen content, the effective cohesion of the sand increased from 13.8 to 55.2 kN/m² with decrease in the angle of internal friction from 31 to 18°C.

In the case of a soil that already has cohesion the bituminous material is admixed in order to waterproof the soil and maintain its existing strength. Here the emphasis is upon impeding the entry of water by adding sufficient bituminous material to (a) wrap soil particles or agglomerates of particles in thin bituminous films or membranes, and (b) plug the soil-void 'channels' (O'Flaherty, 2002).

According to Austroads (2006), bitumen is applicable to granular materials with low cohesion and low plasticity and has stabilizing effect of decreasing permeability, improves cohesion, strength and decrease moisture sensitivity by coating fines. According to FM 5-410, the recommended bituminous materials for use with soils having different gradations are:

For Open-Graded Aggregate

Rapid- and medium-curing liquid asphalts with grades RC-250, RC-800 and MC-3000

Medium-setting asphalt emulsion with grades MS-2 and CMS-2

For Well-Graded Aggregate with little or No Material Passing the No 200 Sieve

Rapid- and medium-curing liquid asphalts with grades RC-250, RC-800, MC-250 and MC-800

Slow -curing liquid asphalts with grades SC-250, and SC-800

Medium-setting and slow-setting asphalt emulsion with grades MS-2, CMS-2, SS-1 and CSS-1

For Aggregate with a considerable percentage of fine Aggregate and Material Passing the No 200 Sieve

Medium -curing liquid asphalts with grades MC-250, and MC-800

Slow –curing liquid asphalts with grades SC-250, and SC-800

Slow- setting asphalt emulsion with grades SS-1, SS-1h, CSS-1 and CSS-1h

Medium- setting asphalt emulsions with grades MS-2 and CMS-2

The type of bitumen to be used depends on the type of soil to be stabilized, the method of construction, and the weather conditions. The specific type and grade of bitumen depends on the characteristics of the aggregate, the type of construction equipment, and the climatic conditions.

According to Muhammad (2014), the Highway Research Board of USA, indicates four types of soil-bitumen, viz; Soil-bitumen (proper), Sand-bitumen, Water-proofed Clay Concrete and Oiled Earth. According to (Field Manual 5-410,1992) the recommended soil gradations for bituminous stabilized subgrade materials and base or subbase course materials are shown in the Tables 2.1 and 2.2, respectively. Mechanical stabilization may be required to bring soil to proper gradation.

Table 2.1: Recommended gradations for bituminous- stabilized sub-grade materials

Sieve Size	Percentage Passing (%)
3-inch (75 mm)	100
No. 4 (4.75 mm)	50 – 100
No. 30 (600 µm)	38 – 100
No. 200 (75 µm)	2 – 30

Source: (Field Manual 5-410, Military Soils Engineering ,1992)

Table 2.2: Recommended gradations for bituminous- stabilized subbase and base courses materials

Sieve Size	1½-in(37.5 mm)	1-in (25.0 mm)	¾-in (19.0 mm)	½ -in (12.5 mm)
	Max	Max	Max	Max
1½-in(37.5 mm)	100	--	--	--
1-in (25.0 mm)	84 ± 9	100	--	--
¾-in (19.0 mm)	76 ± 9	83 ± 9	100	--
½ -in (12.5 mm)	66 ± 9	73 ± 9	82 ± 9	100
3/8in (9.5 mm)	59 ± 9	64 ± 9	72 ± 9	83 ± 9
No.4 (4.75 mm)	45 ± 9	48 ± 9	54 ± 9	62 ± 9
No.8 (2.36 mm)	35 ± 9	37 ± 9	41 ± 9	47 ± 9
No.16 (1.18 mm)	27 ± 9	28 ± 9	32 ± 9	36 ± 9
No. 30 (600 µm)	20 ± 9	21 ± 9	24 ± 9	28 ± 9
No. 50 (300 µm)	14 ± 7	16 ± 7	17 ± 7	20 ± 7
No.100 (150 µm)	9 ± 5	11 ± 5	12 ± 5	14 ± 5
No.200 (75 µm)	5 ± 2	5 ± 2	5 ± 2	5 ± 2

Source: (Field Manual 5-410, Military Soils Engineering ,1992)

2.3.2 Use of Bitumen in Soil Stabilization and Its Effects on Properties of Laterite

According to Ola (1978), the maximum dry density for A-2-4 and A-7-6 lateritic soils– bitumen (Shelmac S.125) show a consistent decrease in maximum dry densities with increase in bitumen contents (0%- 8%). For an A-3 sand bitumen mixture, increase in the maximum dry density up to bitumen content of 3% and subsequently a decrease was observed, and attributed to the grain size distribution and specific gravities of the soil and stabilizer. While the optimum moisture content decreased with bitumen content. Decrease in moisture content is partly due to

fluid nature of the cut back bitumen, already occupying the space normally filled by water and since there is no water – bitumen reaction, like the hydration of cement, less water is required. Ola (1978) equally reported that the CBR value of an A-7-6 soil stabilized with bitumen, increased from 28 to 83 %.

Osinubi and Bajeh (1994) reported improvement in soil properties when bitumen was used in the stabilization of soil. Optimum strength was obtained with the use of 5% bitumen. Yathish, 143 (2014) reported optimum strength when 4% bitumen was used in soil stabilization of a sandy–clay soil. Harris *et al* (1983) found for bituminous stabilization of fine during construction of Baiomoro-Gashua road in Nigeria that the require stabilities could be achieved with 5.5% RC2 and 5% water at 30 days curing.

2.3.3 Use of Heat in Soil Stabilisation

Different researchers have reported impressive results in the use of heat in the stabilization of soil, Terashi and Juran (2000) reported application of heat within a temperature range of 300°C to 1,000°C, and observed that the artificial heating is naturally more effective.

According to (Arora, 2007; Muhammad ,2014; Ocholi and Joel, 2014), when soil is heated, its strength increases, because of decrease in water content of the soil, and decrease in electric repulsion between the clay particles. According to him, on heating to above 100°C, the absorbed water is driven off and the strength of the soil is further increased, while on heating to temperature of 400°C to 600°C, some irreversible changes occurs which makes the soil non-plastic and non-expansive.

The effect of heat on the properties of soil was also reported by Ola (1978), indicating that the angle of internal friction increased from 22 to 26° with some slight increase in cohesion when

iron was introduced in the form of Fe^{3+} . On heating this specimen to 250°C for 24 hours and completely remoulding the material before testing, the friction angle increased to 28° with a slight decrease in cohesion. Ola (1978) observed that the procedures presented are not necessarily similar to those obtained during the formation of lateritic soils they nevertheless infer that both heat and cementing agents are essential for the formation of lateritic soils.

The use of heat in soil stabilization was equally reported by Ocholi and Joel (2014) based on their investigation on the effect of variation in temperature on the compressive strength and water absorption of lateritic interlocking bricks stabilized with 6% and 8 % cement, subjected to temperatures of 100°C , 200°C , 300°C , 400°C and 500°C , over a period of one hour. Results of test showed that compressive strength of bricks stabilized with 6 % and 8 % cement content increased from 2.24 N/mm^2 and 2.72 N/mm^2 at room temperature (35°C) to 4.03 N/mm^2 and 4.16 N/mm^2 , respectively when subjected to a temperature of 500°C . However, when bricks were immersed in water for 24 hours, after heating, 28-day compressive strength of bricks stabilized with 6 % and 8 % cement decreased from 1.09 N/mm^2 and 2.10 N/mm^2 at room temperature (35°C) to 0.76 N/mm^2 and 1.36 N/mm^2 respectively when subjected to a temperature of 500°C . Water absorption values of bricks stabilized with 6 % and 8 % cement increased from 1.91 % and 2.00 % at room temperature (35°C) to 21.72 % and 17.44 % respectively when bricks were subjected to a temperature of 500° . Although the trust of this investigation which was on the effect of heat on laterite stabilized with cement is different from this research which is on the effect of treatment with heat and bitumen on the properties of soil, the use of heat at different temperatures in stabilization demonstrated is relevant to the study.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials

The materials used for this study are as discussed below

3.1.1 Lateritic Soil

The laterite soil sample was collected from a borrow pit at Shika (Longitude $7^{\circ} 31' 7''$ E latitude $11^{\circ} 13' 34''$ N) along Zaria - Funtua road, Kaduna State, Nigeria. Disturbed samples were taken from a depth of between 1.5m to 2.0 m after removal of the top soil.

3.1.2 Cutback bitumen

Cutback bitumen used for the work was obtained from Mararaban Jos, along Zaria - Kaduna road, Kaduna State.

3.2 Methods

Laboratory tests performed on the samples were in accordance with BS1377 (1990) for the natural laterite and BS1924 (1990) for laterite treated with heat and admixed with cutback bitumen.

3.2.1 Cutback bitumen Tests

The following tests were used to characterize the cutback bitumen:

3.2.1.1 Distillation test

The distillation test of cut back bitumen measures the amount and character of volatile constituents it contains. The test determined the percentages, by volume, of distillate fractions at specified temperatures. The test was carried out in accordance with AASHTO Designation T78 (2007).

3.2.1.2 Viscosity test

Viscosity is the property of a fluid that retards its ability to flow. Viscosity test was carried out in accordance with AASHTO T72-83(2007) using Saybolt Furol viscometer. The viscosity of the cutback bitumen was determined at temperature of 25⁰C.

3.2.1.3 Flash point test

This test was carried out in accordance with AASHTO T48 (2007). The flash point test was carried out by heating a sample of the cutback bitumen at a uniform rate whilst periodically passing a small flame across the material. It indicates temperature at which the vapours first burn with a brief flash in the flash point of the binder.

3.2.2 Heat Treatment

The natural soil sample used was air – dried and crushed until it passed BS sieve No. 4 (4.76 mm) aperture. An approximately 60 kg of the air–dried sample each was weighed into four different bags and treated with regulated heat at temperatures of 0⁰C, 200⁰C, 400⁰C, and 600⁰C for the duration of one hour each. The heating was done using melting furnace of over a 1000⁰C capacity thermocouple, which usually starts reading from room temperature (37⁰C). In this regards the room temperature was taken as the control (0⁰C) temperature. The heating was carriedout at Electromechanical Central workshop, Sheda Science and Technology Complex Abuja.

3.2.3 Grain size analysis (Wet Sieving/Hydrometer Analysis)

The particle size analysis was carried out using wet sieving and sedimentation methods, in accordance with BS 1377 (1990), part 2, section 9. Lateritic soil sample of 300 g was taken for

each of the heat treated soil samples and soaked for 24 hours and washed through BS sieve No. 200. The materials retained were collected and oven-dried for 24 hours while materials that passed through the BS sieve No. 200 was used for sedimentation test. The sieve analysis was conducted on the oven – dried samples, by sieving through a set of BS sieve between 5 mm to 75 μm diameter. The percentages passing through sieves were then calculated and plotted on a semi-log graph; percentage passing against sieve sizes

The sedimentation test was however conducted on the solution kept in 1000 ml cylinder using Sodium Hexa- metaphosphate and sodium hydroxide as dispersing agents. The hydrometer analysis was done in accordance with section 9.5 of the BS 1377 (1990), part 2. The percentages of fines were calculated and combined with the percentages of soil passing obtained in the wet sieve analysis for all the samples heat treated at different temperatures and the full particle size distribution for the various samples obtained.

3.2.4 Atterberg Limit Test

Details of the atterberg limit test are as discussed below

3.2.4.1 Liquid Limit (LL)

Cone Penetration Method was adopted for this test. The test was carried out in accordance with BS 1377 (1990), part 2, section 4. About 300 g of the sample of the laterite treated at different temperatures pulverized and passed through BS No. 40 sieve (425 μm aperture) was used for the test. The samples were mixed thoroughly with distilled water for about 10 minutes and kept in polythene bags for 24 hours to mature. After portions being used for plastic limit were taken from the prepared and matured samples, the remaining samples were placed on the glass plates and mixed for about 10 minutes with spatulas to obtain uniform distribution of water throughout the samples. Soil pastes were then placed in cup. The cone and dial gauge were then

adjusted and cone penetrations measured. The moisture contents of the soils from each penetration reading for all samples were taken and determined in accordance with procedure outlined in BS 1377 (1990), Part 3, Section 3.2. The test was repeated 5 times each for all the laterite treated at different temperatures. Each cone penetration (mm) is plotted as ordinate, against the corresponding moisture content (%) as abscissa, both to linear scales and best straight line fitting these points are drawn. From the graphs, the moisture contents corresponding to a cone penetration of 20mm were read off to the nearest 0.1% and results were reported to the nearest whole number as the liquid limits. The results obtained from this test are used in classification of the soil sample.

3.2.4.2 Plastic Limit (PL)

About 20 g of the portion of the prepared and matured sample set aside from the liquid limit test was used in the test being carried out in accordance with BS 1377 (1990), part 2, section 5.3. The 20 g sample was spread in the glass mixing plate so that it could partially dry and was intermittently mixed to avoid drying out. The soil was first kneaded and then shaped into a ball. The ball was then moulded between the fingers and rolled between the palms of the hands so that the warmth of the hands slowly dries it. When slight cracks began to appear on the surface, the ball was divided into two portions each of about 10 g and further divided into four equal parts. Each part was rolled between two fingers of one on the surface of the glass plate, into about 6 mm diameter by use of a steady pressure. The rolling was continued until it had been rolled to about 3 mm diameter when the threads crumbled. The crumbled threads were then kept in the moisture containers, weighed and oven-dried for about 24 hours, after which, the dry samples were weighed again. The moisture content was determined accordingly. Two replicate moisture content samples were taken as previously done above. The average of the two moisture

contents gives the plastic limit (PL). The values of the plastic limit for the laterite treated with heat at various temperatures were recorded and tabulated. The result of this test was used in classification of the soil sample.

3.2.4.3 Plasticity Index (PI)

The plasticity index is the percentage moisture range over which the soil is in plastic state. It was Calculated as plasticity index, $PI = LL-PL$. The values of the plasticity index for the laterite treated with heat at various temperatures were recorded and tabulated. The values obtained from this test were used in the determination of soil activity.

3.2.5 Specific Gravity

The determination of specific gravity was carried out according to BS 1377 (1990) test (B) for fine-grained soils. The density bottle and the stopper were weighed to the nearest 0.001 g (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 air bubble. The bottle was then completely filled up and covered. The covered bottle was then wiped dry and the whole weighed to the nearest 0.001 g as (M_3). The bottle was subsequently emptied and filled completely with water, wiped dry and weighed to the nearest 0.001 g (M_4).

The specific gravity is calculated using eq. (3.1):

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1)(M_3 - M_1)} \quad (3.1)$$

where:

G.s = Specific gravity

M_1 = mass of empty density bottle (g)

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

M_3 = mass of density bottle +soil+ water (g)

M_4 = mass of density bottle filled with water (g)

To obtain a more accurate result three density bottles were used and their average taken.

3.2.6 Compaction Test

The Compaction test was carried out using West African Standard (WAS) compactive effort, as it is the conventional energy level commonly used in the region and recommended by the Nigerian General Specification (1997). 3000 g of the soil/bitumen were mixed thoroughly with 8 % of water (and the water is added at 8 % for each of the compactions). The sample was then compacted into the 1000 cm³ BS mould in five layers of approximately equal mass with each layer receiving 10 blows of 4.5 kg rammer. Two representative samples (one from the top and the other from the bottom) were then taken from the compacted soil for the determination of moisture content. The sample was then removed from the mould, crushed and 8 % of water was added, and the same procedure was repeated until minimum of five set of samples were taken for moisture content determination. For those tests that required soil-bitumen mixture at each of the temperature treatments, predetermined mass of the heat treated soil sample was thoroughly hand mixed with the required amounts of cutback bitumen using 0%, 3%, 6% and 9% cutback bitumen by dry weight of laterite for about five minutes. The required amount of water was then added and again mixed by hand for another five minutes and allowed for between 10 to 15 minutes before compaction. Percentage of bitumen content was calculated as % of weight of the heat treated laterite sample at various temperatures.

The wet and dry densities were calculated with the below formulae.

$$\rho_b = \frac{W_2 - W_1}{V} \quad (3.2)$$

$$\rho_d = \frac{(\rho_b \times 100)}{(100 + Mc)} \quad (3.3)$$

Where

- ρ_b = Bulk density (Mg/m³)
 ρ_d = Dry density (Mg/m³)
 Mc = Moisture Content (%)
 W_1 = weight of mould (g)
 W_2 = weight of wet soil + mould (g)
 $w_2 - w_1$ = weight of wet soil (g)
 V = Volume of Mould = 1000 cm³

The MDD and OMC obtained from this test were used in preparing the specimens for strength tests. The results are as presented in Table 3.1

Table 3.1 : Compaction Results (WASC)

Heat Treatment (°C)	Bitumen Content (%)							
	0%	3%	6%	9%	0%	3%	6%	9%
	MDD(Mg/m ³)				OMC (%)			
0°C	1.951	1.928	1.9279	1.9849	17.28	15.7	15.68	12.71
200°C	1.9215	1.8611	1.8571	1.9817	17.06	16.01	15.03	12.99
400°C	1.980	1.9445	1.9422	1.9740	17.5	13.79	13.62	11.62
600°C	1.9533	1.9057	1.89	1.9537	19.57	16.25	16.4	12.51

3.2.7 California Bearing Ratio (CBR) Determination

The California bearing ratio (CBR) test was conducted in accordance with BS 1377 (1990), part 4, section 7 and BS 1924 (1990) for the natural lateritic soil and soil- bitumen stabilized soils respectively. The test was used in the determination of the California Bearing Ratio (CBR) of the treated specimens, which were obtained by measuring the relationship between the force and penetration, when a cylindrical plunger of cross-sectional area 1935mm^2 was made to penetrate at a rate 1.3 mm/min until the specimen failed. Both the natural soil and soil-bitumen specimen were compacted at a WASC energy level. At any value of penetration, the ratio of force to a standard force gave the California Bearing Ratio (CBR).

The CBR values are calculated as follows:

$$CBR \text{ at } 2.5\text{mm} = \left(\frac{\text{Load}}{13.2}\right) \times 100 \quad (3.4)$$

$$CBR \text{ at } 5.0\text{mm} = \left(\frac{\text{Load}}{19.6}\right) \times 100 \quad (3.5)$$

The greater of the two values and as their means where the value are within 10 % of each other was used as CBR value.

Natural soil samples of 5000 g each and soil- bitumen mixed at their respective optimum moisture contents in the CBR mould at WAS compaction energy level was used for the test. The samples were compacted in five layers with 25 blows from the 4.5 kg rammer.

The base plates were removed after compaction and the compacted specimens were enclosed in polyethene bags and cured under controlled condition for 6 days and were immersed in water for 24 hours before testing according to Nigerian General Specifications (1997).

The results are presented in Table 3.2

Table 3.2: Effect of heat and bitumen on CBR value of Laterite

Heat Treatment	Bitumen Content (%)			
	0%	3%	6%	9%
	CBR (%)			
0°C	19.69	23.86	23.08	22.47
200°C	19.38	26.24	22.31	21.26
400°C	18.34	20.11	28.52	22.23
600°C	11.88	22.46	22.32	20.84

3.2.8 Unconfined Compressive Strength (UCS) Test

The Unconfined Compressive Strength (UCS) test was carried out in accordance with BS 1924 (1990). The compaction was carried out using WAS compactive effort at the determined maximum dry density (obtained during the moisture- density test) of the pretreated laterite at room temperature (37°C) designated as control (0°C), 200°C, 400°C, and 600°C and admixed with bitumen contents levels of 0%, 3%, 6% and 9% of weight of the heat treated laterite samples. 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 wax-cured for 7 days, second for 14 days and the third for 28 days. At the elapsed day of wax curing, the specimens were de-waxed and 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 occurred. The UCS of the sample was determined at the point on the stress–strain curve at which failure occurred.

The UCS was calculated from the following equation.

$$\delta = \frac{[R \times C_r \times (100 - E\%) \times 1000 \text{ kN/m}^2]}{(100 \times A_o)} \quad (3.6)$$

Where:

E % = Strain in percent

v = Amount of compression at any stage (mm)

R = Load ring reading at strain E (kN)

C_r = Mean calibration of load ring

L_o = Initial length of specimen (mm)

A_o = Initial cross sectional area (mm²)

□ = Compressive stress at strain E (kN/m)

The results are as presented in Table 3.3

Table 3.3: Unconfined Compressive Strength (UCS) Results

Heat Treatment/ Bitumen Content	7 day cured	14 day cured	28 day cured	7 day Cured and 7 day Soaked	Durability (%)	Loss in Strength (%)
0°C,0%	266.43	389.48	457.90	45.05	11.57	88.43
0°C,3%	336.45	402.61	512.26	66.79	16.59	83.41
0°C,6%	283.29	304.81	412.64	48.68	15.97	84.03
0°C,9%	219.20	291.45	351.77	39.29	13.48	86.52
200°C,0%	360.17	452.08	442.48	67.72	14.98	85.02
200°C,3%	417.23	491.02	511.16	120.69	24.58	75.42
200°C,6%	377.12	471.29	506.48	106.46	22.59	77.41
200°C,9%	274.37	320.11	401.74	52.71	16.47	79.01
400°C,0%	293.96	392.95	485.31	36.55	9.30	90.7
400°C,3%	360.44	475.13	570.29	88.06	18.53	81.47
400°C,6%	311.01	405.23	486.92	72.01	17.77	82.23
400°C,9%	305.13	356.21	456.34	58.26	16.36	83.64
600°C,0%	327.20	373.75	459.19	47.29	12.65	87.35
600°C,3%	331.24	379.40	462.54	91.47	24.11	75.89
600°C,6%	274.38	318.35	399.66	74.33	23.35	76.65
600°C,9%	272.66	300.41	374.25	65.97	21.96	78.04

3.2.9 Durability Test

The durability assessment of the heated soil- cutback bitumen specimens were carried out by water immersion tests for the measurement of resistance to a loss in strength.

The resistance to loss in strength was determined as ratio of the unconfined compressive strength (UCS) of specimens' wax-cured for 7 days, dewaxed top and bottom, and later immersed in water for another 7 days to the UCS of specimens' wax-cured for 14 days.

The results are presented in Table 3.3

CHAPTER FOUR

RESULTS ANALYSIS AND DISCUSSIONS

4.1 Properties of the Soil and Bitumen

The properties of the soil and bitumen used are discussed below

4.1.1 Index properties of the Natural soil

Index properties of the natural soil as determined from the different tests are shown in Table 4.1. The results showed that the lateritic soil have 72.60 % soil passing sieve No. 200, liquid limit of 51.4 %, plastic limit of 38.02 % and plasticity index of 13.41 %. It has CBR value of 19.69 % for West African Standard Compaction (WASC) energy level. Based on the above soil sample properties, the soil can be classified as an A-7-5(11) and MH soil using AASHTO system (AASHTO, 1986) and Unified Soil Classification System (ASTM, 1992) respectively. The soil is suitable for use as a fill/subgrade material based on the Federal Ministry of Works and Housing (1997 requirements of percentage passing BS sieve 200 of ≥ 35 % , Liquid Limit (LL) ≥ 50 % and CBR of 3 – 10 % at relative compaction ≤ 100 % of Modified AASHTO or WAS. The soil sample is reddish brown in colour and has 18.34 % clay fraction as determined by the hydrometre analysis and shown in Table 4.3.

Table 4.1: Index properties of the natural lateritic soil

Properties	Quantity
Percentage Passing BS Sieve No 200 (%)	72.60
Liquid Limit, (%)	51.43
Plastic Limit (%)	38.02
Plasticity Index (%)	13.41
AASHTO classification	A-7-5 (11)
USCS Classification	MH
Specific gravity	2.72
Maximum Dry Density (Mg/m ³)	1.951
Optimum Moisture Content (%)	17.28
Unconfined Compressive Strength KN/m ²	266.43
California Bearing Ratio, % (after 24hrs soaking)	19.69
Colour	Reddish Brown

4.1.2 Properties of the Bitumen Sample

The results of the tests carried out on the cutback bitumen used is shown in Table 4.2. The results showed specific gravity of the bitumen as 0.92, Saybolt Furol viscosity at 25⁰C as 77 secs, distillate, percentage by volume of total distillate to 225⁰C as 16%, distillate, percentage by volume of total distillate to 260⁰C as 54%, distillate, percentage by volume of total distillate to 315⁰C as 79% and residue from distillation to 360⁰C volume, percent of sample by difference was 61%. Based on the above bitumen properties and comparison with requirements in standard specifications for Transportation Materials and Methods of sampling (AASHTO, 2007), the cutback bitumen can be categorised as MC-30 grade cutback bitumen.

Table 4.2: Properties of cutback bitumen

Properties	Quantity	AASHTO Standards
Specific Gravity at 25 ⁰ C	0.92	-
Furol Viscosity at 25 ⁰ C in (s)	77	75 - 150
Flash Point (Cleveland open cup)	47	Min. 38
Distillation:		
Distillate, percentage by volume of total distillate to 225 ⁰ C	16	Max. 25
Distillate, percentage by volume of total distillate to 260 ⁰ C	54	40 - 70
Distillate, percentage by volume of total distillate to 315 ⁰ C	79	75 - 93
Residue from distillation to 360 ⁰ C volume percent of sample by difference	61	Min. 50
Grade	MC – 30	

4.2 Effects of Heat Treatment on Index Properties of the Soil Samples

The effect of heat treatment on index properties of the soil samples are discussed below

4.2.1 Grain Size Distribution

The result of grain size distribution of laterite, subjected to heat treatment at different temperature is as shown in Table 4.3 while the grain size distribution curves for heat treatment at control temperature is shown in figure 4.1. The results of variation of heat treatment at control and treatment with 200⁰C , 400⁰C and 600⁰C samples are as shown in figures 4.2, 4.3 and 4.4 in that order. The percentage passing sieve size 0.002 mm (clay fraction) increased by 9.32 % with increase in heat treatment temperature from control temperature (37⁰C) to 200⁰C heat treatment. This can be attributed to loss in adsorbed water and reduction of plasticity by clay fraction due to heat as advanced by (Arora,2007; and Muhammad 2014) and reason advanced by Ola (1978) that removal of cementing agents in the laterite caused significant increases in the percentages of fines passing the various sieve sizes. This observation is also consistent with the submission by Ola (1978) that clay helps to bond finer particles together to form larger aggregate.

A decrease of 24.48% and 45.64% in percentages of fines that passed through sieve size 0.002 mm (clay fraction) were observed at 400⁰C and 600⁰C heat treatment temperatures

respectively. This can be attributed to conversion of clay clods into aggregates as advanced by (Arora,2007; and Muhammad 2014) and reason advanced by Ocholi and Joel (2014) that first irreversible reaction; dehydroxylation take place at these temperatures resulting in part of the actual clay structures (the hydroxyl group) been driven off as steam

Table 4.3: Grain size Analysis Result of laterite treated with different heat

Sieve size (mm)	0°C	200°C	400°C	600°C
	Percentage Passing (%)			
5	99.9	99.87	100	100
3.35	99.63	98.83	99.4	99.2
2	91.13	87.23	86.67	89.9
1.18	87.1	82.33	81.2	85.6
0.85	85.1	80.03	78.73	83.63
0.6	83.33	77.8	76.47	81.8
0.425	81.57	75.83	74.4	80.03
0.3	79.77	73.67	72.27	78.27
0.15	75.53	68.5	67.53	73.37
0.075	72.6	65.1	64.27	69.93
0.042	49.658	50.228	46.669	51.844
0.031	45.185	44.962	41.64	45.197
0.022	40.711	40.912	35.002	38.329
0.016	37.356	36.861	29.772	33.233
0.012	34.224	34.431	25.346	27.473
0.009	30.198	31.393	21.524	22.82
0.006	26.171	27.95	16.898	18.167
0.005	23.04	24.709	13.478	14.179
0.004	19.461	21.266	9.656	10.635
0.003	18.79	20.658	9.253	10.192
0.002	18.342	20.051	8.851	9.97
0.001	17.895	19.443	8.449	9.748

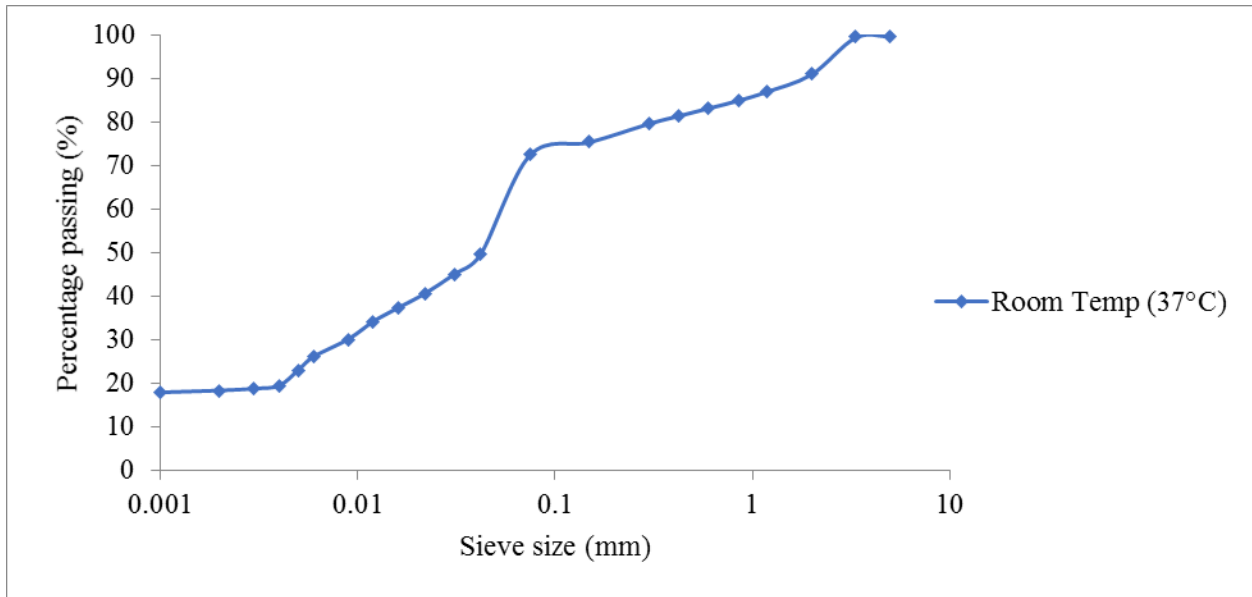


Figure 4.1: Particle size distribution curve of control laterite sample

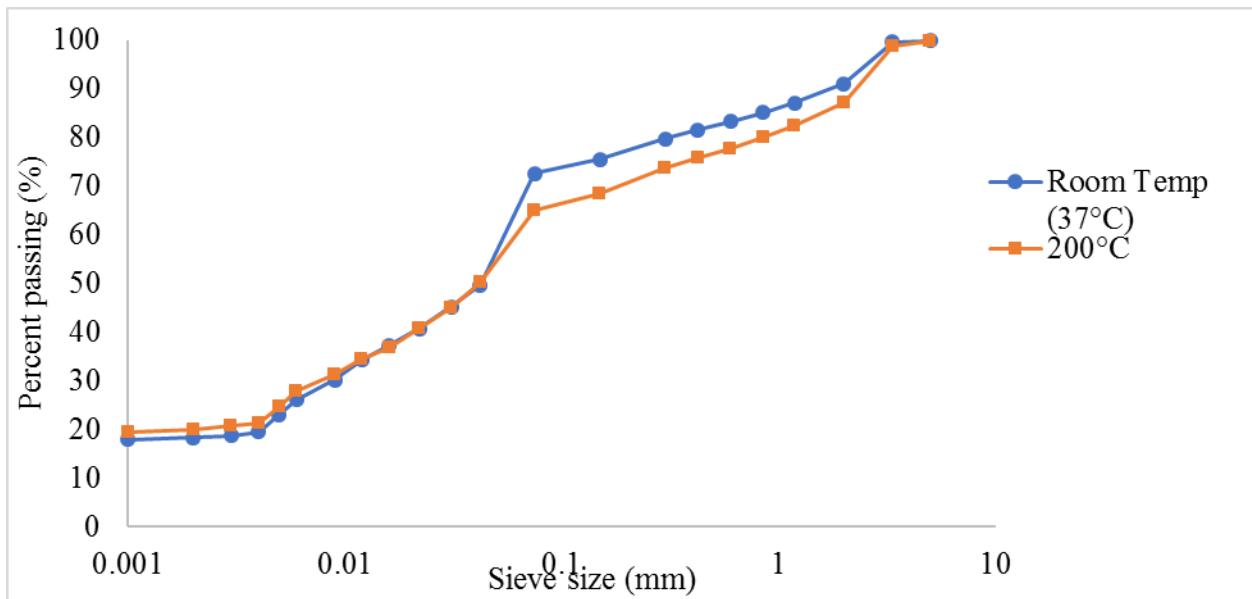


Figure 4.2: Variation of heat treatment at control and 200°C

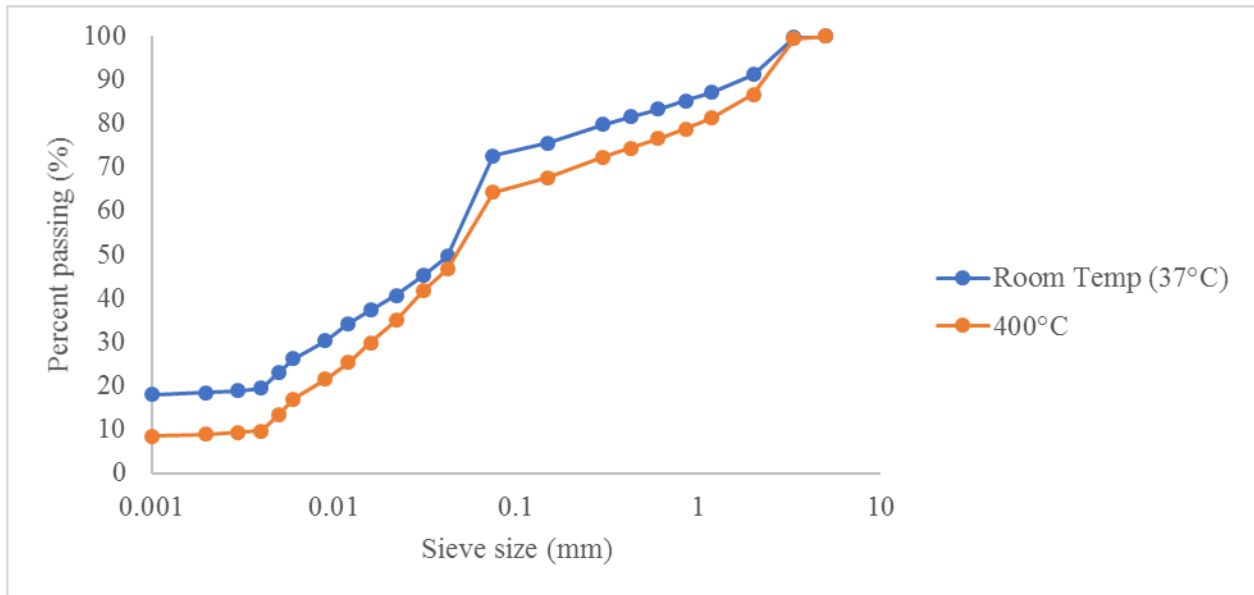


Figure 4.3: Variation of heat treatment at control and 400°C

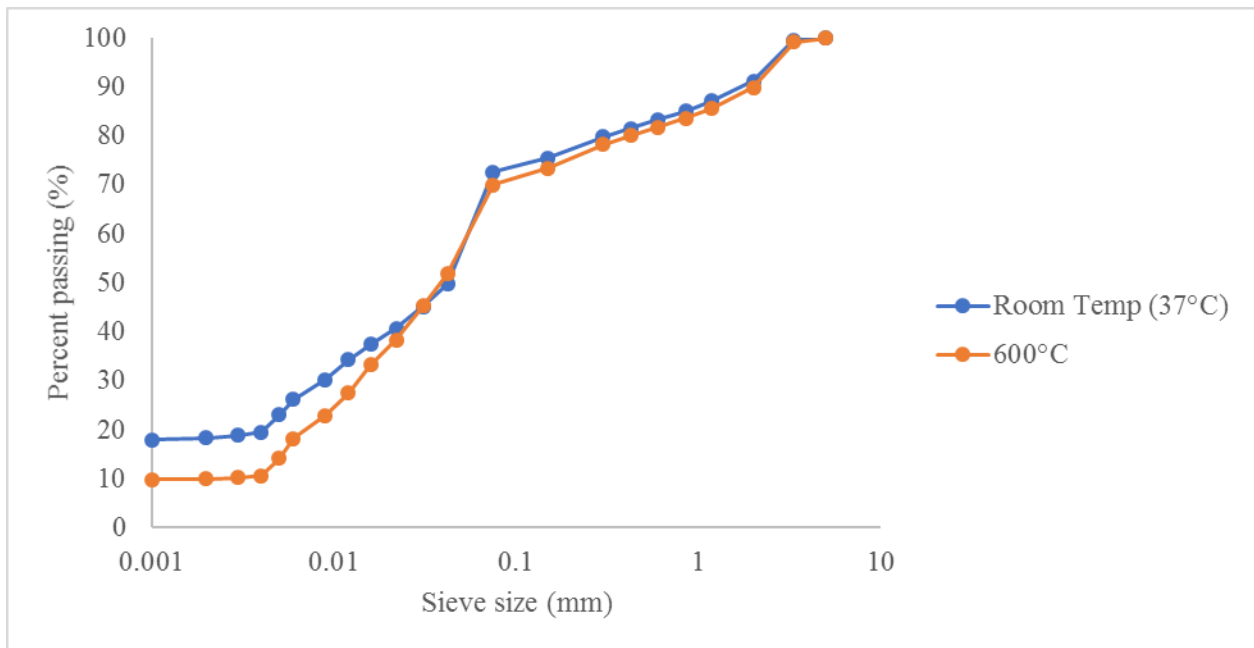


Figure 4.4: Variation of heat treatment at control and 600°C

4.2.2 Activity

Variation of activity (ratio of plasticity index to percentage of clay fraction) of the laterite soil with heat treatment at different temperature is presented in Table 4.4, and indicated that activity of the natural soil sample is 0.73. Based on Arora (2007) a value less than 0.75 is an indication that the soil is predominated by kaolinite type of clay mineral and is inactive. Activity value of 0.42 was obtained when the laterite was treated with heat at temperature of 200°C. This implies that activity of the soil sample can be reduce with heat at temperature of 200°C and changes in the volume of material during swelling or shrinkage that can lead to distress in pavement structure can be reduced as it depends on the activity of the soil. The reduction in value can be linked to percentage of clay fraction in the sample due to heat. Activity values of 1.05 and 0.97 at 400°C and 600°C heat treatment respectively can be link to percentages, nature and characteristics of the clay mineral and higher degree of dessication of hydrated parent laterite that might have occurred during the process of laterization- which reduces the influence of clay content on plasticity index that invariably determines activity of the soil. It can equally be attributed to irreversible reaction which according to (Arora,2007; Muhammad,2014;) occurs when soil is heated to temperatures of 400°C to 600°C.

Table 4.4: Variation of Laterite Sample Activity with Heat Treatment at Different Temperatures

Heat Treatment	0°C	200°C	400°C	600°C
Plasticity Index (PI) %	13.41	8.33	9.32	9.63
Clay fraction (%)	18.342	20.051	8.851	9.97
Activity (A)	0.73	0.42	1.05	0.97

4.2.3 Atterberg Limits

The effect of varying heat treatment on the Atterberg's limits of the samples is shown in Table 4.5. The result shows that liquid limit decreased by 6.05%, 7.76% and 4.69% when treated with heat at 200°C, 400°C and 600°C respectively, while Plastic limit increased with increase in heat treatment temperature. Decrease in values observed can be attributed to the clay content in the soils after heat treatment; loss in plasticity and cohesion by the soils after heating. This is caused by increase in effective grain sizes due to conversion of the clay clods into silt and sand size particles due to heat, which reduces the effect of surface forces on soil particles and decreased the liquid limit, while the plastic limit increased. The decrease in liquid limit observed implies reduction in compressibility and increase in shear strength of the treated soil samples when compare with the requirement for subgrade materials shown in Table 4.6

The plasticity index value of laterite decreased when treated with heat at 200°C from 13.41% to 8.33% which corresponds to 37.88 %. The observed trend can be attributed to percentages of clay minerals found in the soil sample and lost of water from pores of the treated soil that influence the plasticity of the soil sample.

The plasticity index values of 9.32 % and 9.63 % at temperature of 400°C and 600°C respectively may be attributed to relative amount of active clay fraction in the soil sample, limitations of heat on laterization, which the parent soil was subjected to and irreversible changes that may have occurred when the soil was treated with heat at higher temperature. The above results relative to the requirements by the Federal Ministry of Works & Housing (1997) for different layers of flexible pavement shown in Table 4.6 implies an improvement in PI of the treated soil samples as the result showed decrease in PI by 30.50% and 28.19% for 400°C and 600°C respectively. This also means reduction in compressibility and increase in shear strength of the treated soil samples.

Table 4.5 Atterberg's Limits test result of laterite sample with varying heat treatment

Heat Treatment	0°C	200°C	400°C	600°C
Liquid limit (LL) %	51.43	48.30	47.44	49.02
Plastic limit (PL) %	38.02	39.97	38.12	39.39
Plasticity Index (PI) %	13.41	8.33	9.32	9.63

The hypothesis that, the higher the degree of laterization, the less the influence of clay content on plasticity index. This was explained by Gidigas (1976) in terms of clay mineral coatings by sesquioxides which tend to suppress the surface activity of the clay minerals.

TABLE 4.6: Federal Ministry of Works and Housing Specification Requirement For Different Layers of Flexible Pavement.

Clauses	Specification (1997)	Remark
	<u>Material Suitable for Sub-grade/Fill</u>	
6201,	% passing sieve 200 \geq 35%	
6122	Liquid limit (LL) \geq 50%	
	Plasticity Index (PI) \geq 30%	Specification is silent on CBR.
	Relative Compaction \geq 100% of BS	But 3-10% is common.
	<u>Material Suitable for Sub-base</u>	
6201	% passing sieve 200 \geq 35%	
	Liquid limit (LL) \geq 35%	Type I
	Plasticity Index (PI) \geq 12%	Sub-base
	CBR (24 Hrs Soaking) \geq 30%	
	Relative Compaction \geq 100% of WAS	
	<u>Material Suitable for Base-course</u>	
6201	% passing sieve 200 \geq 35%	
	Liquid limit (LL) \geq 35%	
	Plasticity Index (PI) \geq 12%	
	CBR (Unsoaked) \geq 80%	
	Relative Compaction \geq 100% of Mod. AASHTO or WAS	

Source: (FMW & H, 1997)

4.2.4 Specific Gravity

The result of specific gravity with varying heat treatment at different temperature is shown in Table 4.7. The result show gradual decrease from 2.72 at heat treatment temperature of 0°C to 2.61 when treated with heat at temperature of 600°C. This result can be attributed to removal of adsorbed water in the pores of soil samples and conversion of clay clods to light aggregates as a result of heat treatment. This will implies reduction in workability of the treated materials

Table 4.7: Specific Gravity test result of laterite sample with varying heat treatment

Heat Treatment	0°C	200°C	400°C	600°C
Specific Gravity	2.72	2.69	2.66	2.61

4.3 Effects of Heat and Bitumen on Compaction Characteristics

4.3.1 Maximum Dry Density (MDD)

The effect of heat and bitumen content on the maximum dry density of laterite is presented in Figure 4.5. From the figure, MDD was observed to generally decreased with bitumen content up to 6% after which, increase in MDD was observed. This decrease can be attributed to the relatively lower specific gravity value of (0.9) of the bitumen, as against the values for the soils. Increase in MDD beyond 6% bitumen content for all the treatments can be attributed to the formation of soil-bitumen pseudo-granules, which have higher density than the soil or bitumen.

The slight reduction in MDD by 1.51% observed between 0°C and 200°C heat treatment temperature at 0% bitumen content, can be attributed to the commencement of process of transition from clay particles to granular ones, which is characterized by higher void ratio. Increase in MDD observed with 0% bitumen content at 400°C and 600°C heat treatment temperatures by 1.49% and 0.1% respectively can be attributed to lost of actual clay structure as advanced by Ocholi and Joel (2014) which makes the lateritic soil behave like coarse-grained

soils. The trend in MDD with bitumen content is in agreement with the observed trend by Ola (1978) and Osinubi and Badeh (1994). These densities serve as control for the densities that must be achieved at site with these soil/ soil-bitumen samples.

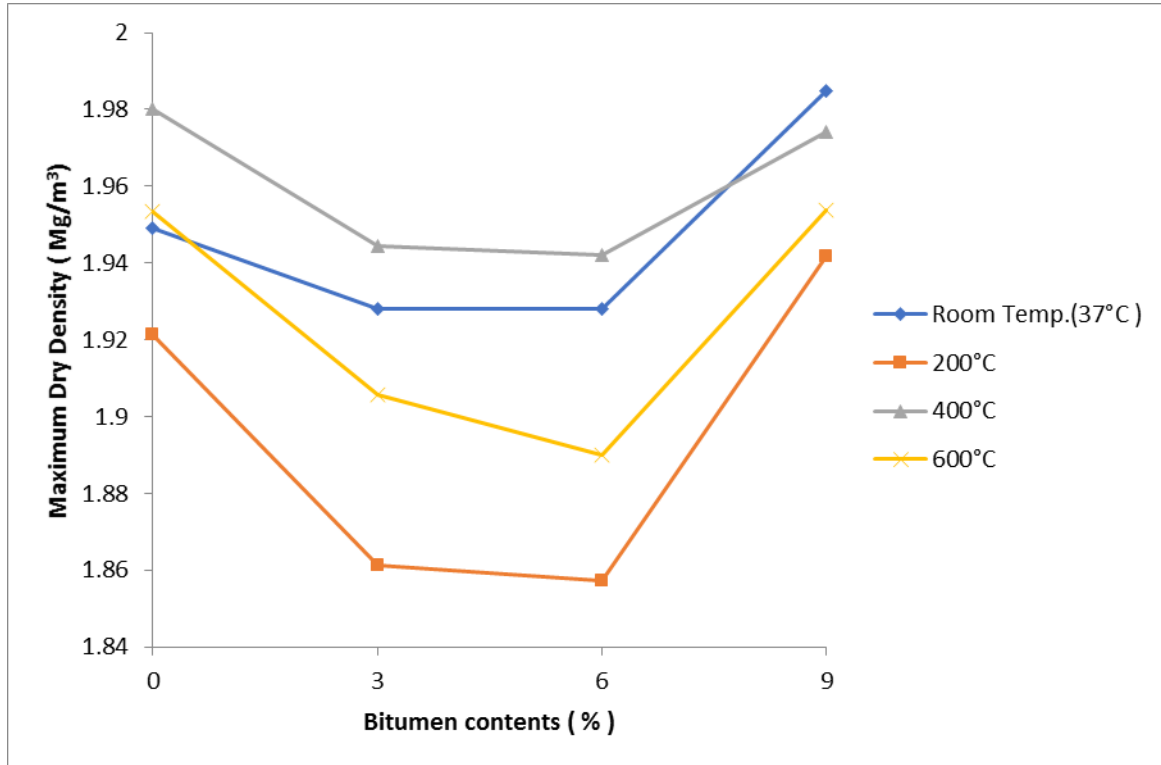


Figure 4.5: Variation of MDD with various bitumen content

4.3.2 Optimum Moisture Content (OMC)

The result of variation of OMC with bitumen content is shown in Figure 4.6. The result indicates that OMC values decrease with bitumen content. This can be attributed to the bitumen, being partly fluid occupying the space normally filled by water, and the fact that there is no water-bitumen reaction as such less water is required. It can also be attributed to coating property of the bitumen, which makes access of water by the coated soil particles difficult. This trend of OMC is similar to those reported by Ola (1978) and Osinubi and Badeh (1994). These

OMC values would serve as guide for the required moisture contents to achieving the desirable densities with these soil/ soil-bitumen samples.

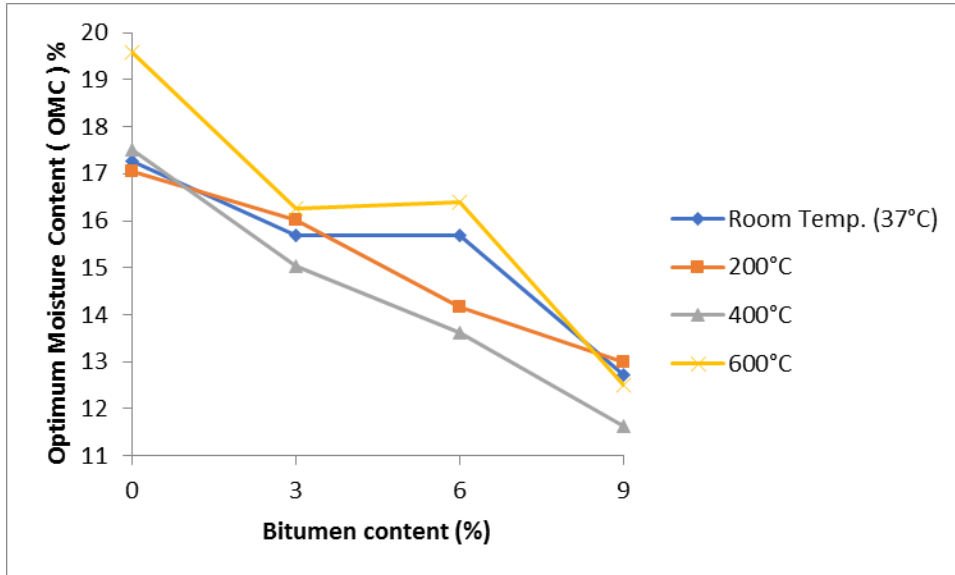


Figure 4.6: Variation of OMC with various bitumen content.

4.4 Effect of Heat and Bitumen Treatment on Strength Characteristics

4.4.1 California Bearing Ratio (CBR)

The variation of CBR with bitumen content at respective heat treatment temperature is shown in Figure 4.7. Observation of the trend for heat treatment at temperatures of 0°C, 200°C and 600°C show an increase in CBR value with bitumen content up to 3%. This is as a result of the increase in binding, impacted by the binder (bitumen) between the particles. Above 3% bitumen content, decrease in CBR value was observed up to 9% bitumen content. This may be as a result of the reduction in inter particle binding property above 3% bitumen content, the bitumen instead of acting as a binder, it acts as a lubricant separating the particles which results in the reduction in CBR values. This trend with bitumen content is similar to those reported by Ola (1978) and Osinubi and Badeh (1994).

The trend for heat treatment at temperature of 400°C which shows an increase in CBR value with increase in bitumen content up to 6% as against 3% for others can be linked to its activity value of 1.05 due to percentages, nature and characteristic of clay mineral in fines of the lateritic soil treated at temperatures of 400°C. This implies that bitumen content of up to 6% can be used for the studied soil depending on the activity value of soil achieved due to effect of heat. The trend observed equally indicates a reduction in CBR value with increase in temperature of heat treatment. The observed trend may be attributed to the reduction in fines content of the soil as the temperature increased from 0°C to 600°C, as shown on the particle size distribution results presented on Table 3.2. This is as a result of the breakdown of the clay mineral structures of the soil by the heat, thereby making the soil to behave more as silt-sand, with less cohesion. At high temperature, the silt-sand behavior of the soil, possessing less cohesion may be responsible for the reduction in CBR values. This shows that unless the soil samples would be admixed with adhesive materials, use of only heat will not guarantee improvement in CBR value due to lost in cohesion.

The maximum values of CBR obtained for specimens waxed cured for 6 days and soaked for 24 hours as specified by the Nigerian General Specification (1997) showed that both the unheated laterite and heat treated laterite have maximum CBR values at 3% bitumen content except the lateritic soil treated at temperature of 400°C. Maximum CBR value of 23.86% at 3% bitumen content was obtained for the untreated laterite. At heating temperature of 200°C maximum CBR value of 26.24% at 3% bitumen content was obtained. While maximum CBR value of 28.52%, was obtained when the laterite was treated with 6 % bitumen after it was subjected to a heat treatment temperature of 400°C.

The difference in the results of maximum value of CBRs' obtained in relation to the 46% maximum value of CBR obtained in Osinubi and Badeh (1994) and 28% obtained by Ola (1978) can be attributed to difference in index properties of various lateritic soil, type of compactive effort, and mode of curing before testing, adopted by various researchers and effect of heat treatment on the soil samples studied.

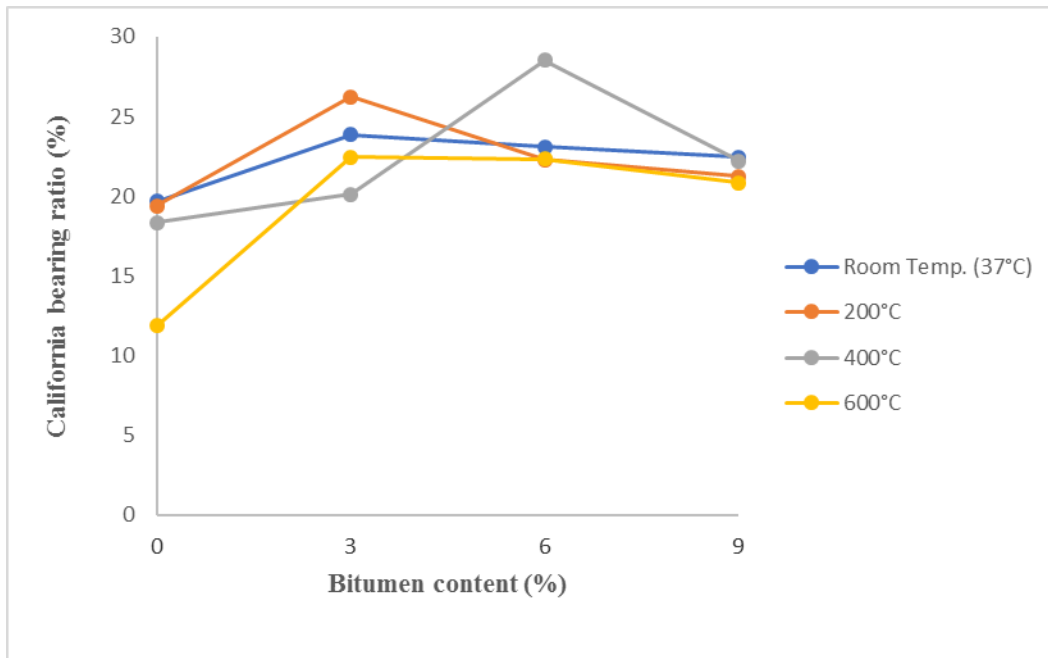


Figure 4.7: Variation of CBR with various bitumen content

Since the values specified by the Nigerian General Specification (1997) for CBR at the energy level of WASC for subgrade, sub-base and base course materials are 10%, 30% and 80% respectively, these materials can only be used as subgrade/fill material, its usage as subbase will require some further treatment such as the use of higher grade of cutback bitumen and foam bitumen.

4.4.2 Unconfined Compressive Strength (UCS)

The results of variations of 7, 14 and 28 day UCS values at different heat treatment temperature with bitumen content are as shown in Figures 4.8, 4.9 and 4.10 respectively.

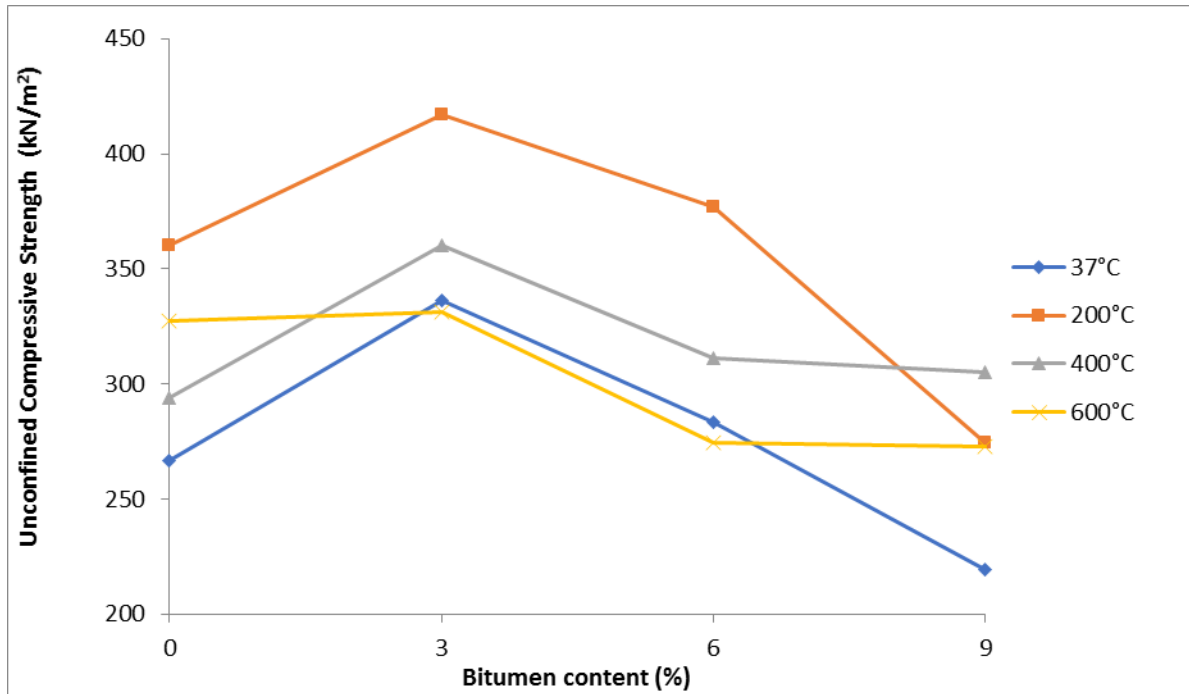


Figure 4.8: Variation of 7 day compressive strength with various bitumen content

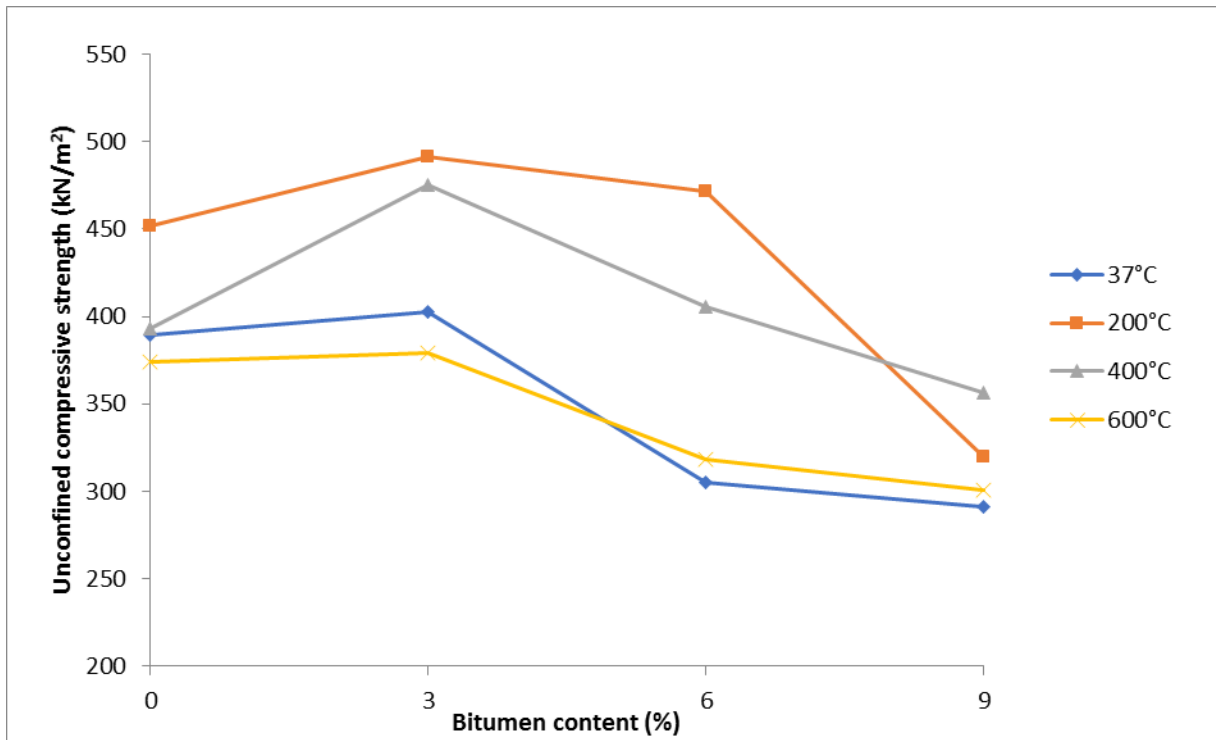


Figure 4.9: Variation of 14 day compressive strength with various bitumen content

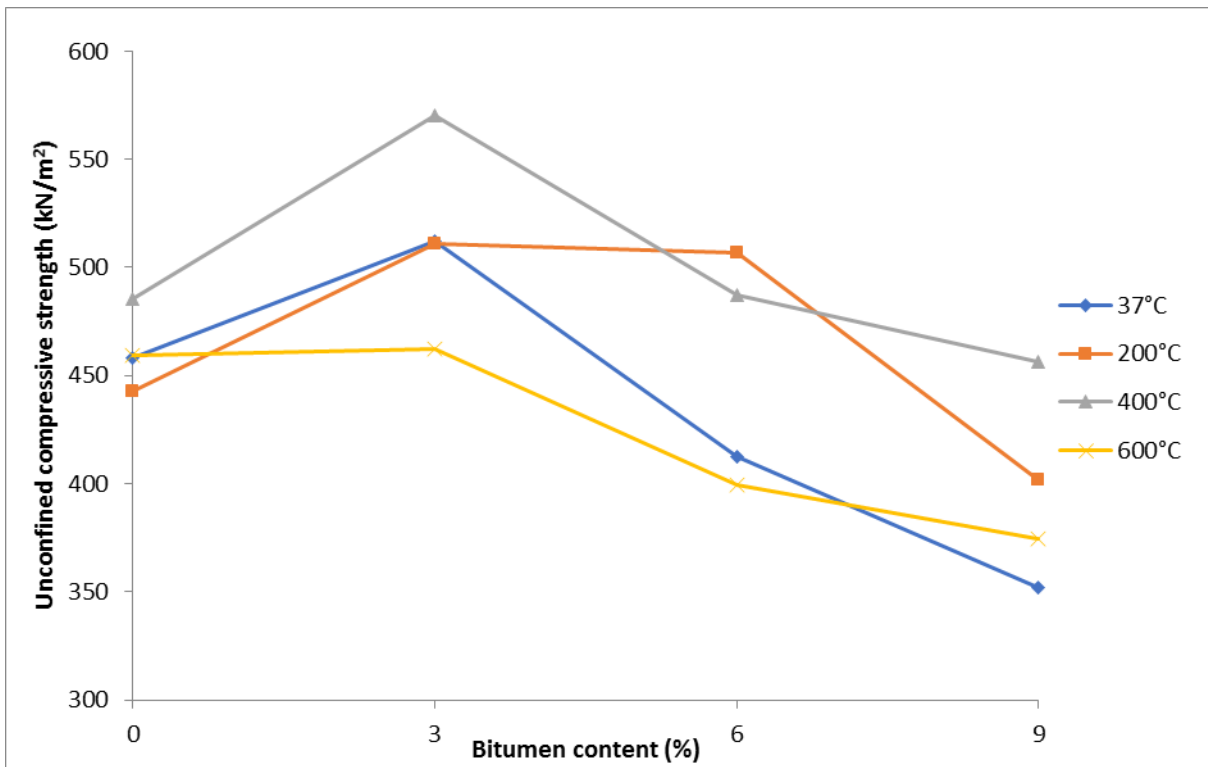


Figure 4.10: Variation of 28 day compressive strength with bitumen content

Variation of UCS values with bitumen content at different heat treatment temperature cured for 7 days, 14days and 28 days show similar curves and generally indicated increase in compressive strength with bitumen content up to 3%. This is as a result of the increase in binding, impacted by the binder (bitumen) between the particles. Above 3% bitumen content, decrease in UCS value was observed up to 9% bitumen content. This may be as a result of reduction in inter particle binding property above the 3% bitumen content, the bitumen instead of acting as a binder, it acts as a lubricant separating the particles which resulted in the reduction in UCS values. Increase in UCS values with bitumen content up to 3% before a decreased with increase in bitumen content, is in line with the observation of Ola (1978) , Osinubi and Bajeh (1994) and Yathish143 (2014).

The 7 days UCS value of laterite subjected to different heat treatment temperature without treatment with bitumen were 266.43 kN/m², 360.17 kN/m², 293.96 kN/m², and 327.20 kN/m² at treatment temperature of 37°C, 200°C, 400°C and 600°C respectively. It shows that the compressive strength increased as temperature of treatment increased.

The reason for increase in strength after 200°C heat treatment can be as advanced by (Arora, 2007; Muhammad, 2014; and Ocholi and Joel, 2014) to dehydration of clay fraction as water in the pores of the clay fraction evaporated after heating, while the clay fraction loses its plasticity, resulting in voids that affect the strength of laterite. Also, as the soil is heated to this temperature range, electric repulsion between clay particles is decreased and the strength of the soil is increased. Increase in strength observed at 400°C and 600°C heat treatments can be attributed to reasons also advanced by (Arora, 2007; Muhammad, 2014; and Ocholi and Joel, 2014) to conversion of clay clods to aggregates which increased strength of the soils.

7 day UCS values of laterite at different heat treatment temperature, and 3% bitumen content were 336.45, 417.23, 360.44 and 331.01 kN/m² at treatment temperature of 0°C, 200°C, 400°C and 600°C respectively.

4.4.3 Durability

Variation of resistance to loss in strength at different heat treatment temperature with bitumen content is shown in Figure 4.11. Observation of the trend shows general increase in value of resistance to loss in strength as the bitumen content increased up to 3% after which it decreased gradually up to 9% bitumen content. This shows that at 3% bitumen content, there was optimum resistance to ingress of water by the specimen. After the 3% bitumen content the resistance decreased as the bitumen content increased up to 9%. It was equally observed from the graph that at higher bitumen contents, all the soils show about the same water proofing capacity. These observations can be attributed to waterproofing effects of bitumen, which prevents the penetration of water into the specimen. The values of resistance to loss in strength after 7 days soaking at the 3% bitumen content for the heat treatment at temperatures of 0°C, 200°C, 400°C and 600°C, are 16.59%, 24.58%, 18.53% and 24.11% respectively. These results show that for all the treatments, resistance to losses in strength at optimum bitumen contents (3%) are far below the recommended 80% resistance to loss in strength after 4 days soaking but specimen of heat treatment at temperature of 200°C, exhibits better water retarding ability than other specimens. This can be linked to the reduction in activity of the lateritic soil with heat treatment at temperature of 200°C used and binding property of cutback bitumen. The low value of durability obtained can as well be attributed to low viscosity nature of the cutback bitumen (MC - 30) used. The trend of resistance to loss in strength is similar to those reported by Ola (1978), Osinubi and Badeh (1994) and Muhammad (2014) although their compactions were conducted

using standard proctor energy level and durability determined after 7day soaking as percentage of oven cured specimen.

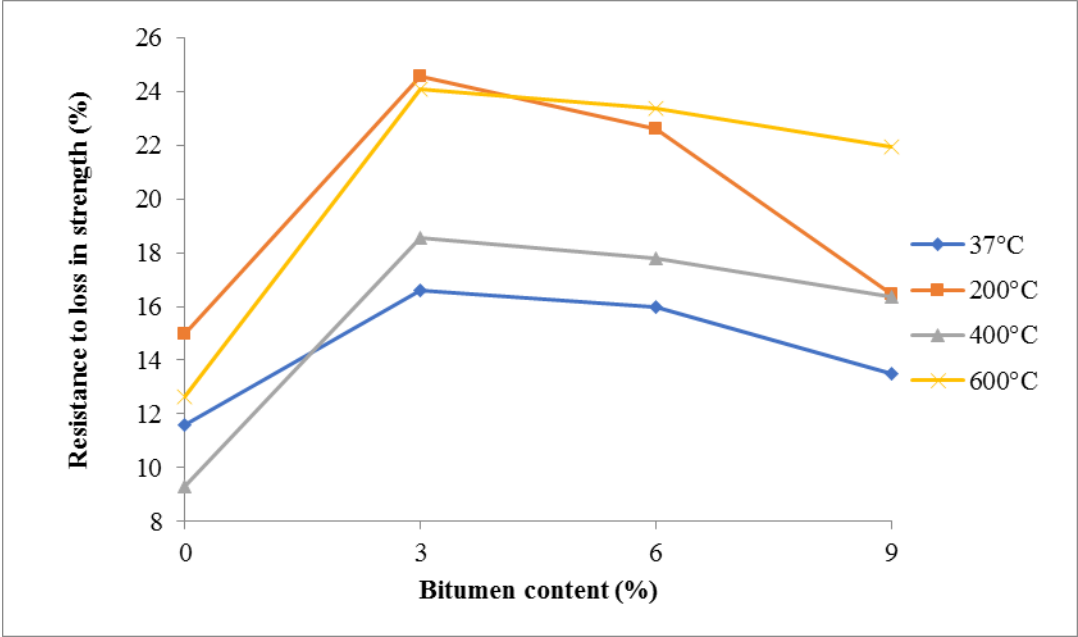


Figure 4.11: Variation of Resistance to loss in strength with bitumen content

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATION

5.1 Summary of Research Findings

1. The lateritic soil studied can be classified as an A-7-5 (11) and MH soil using AASHTO and Unified classification system respectively.
2. The percentage passing sieve size 0.002 mm (clay fraction) increased by 9.32 %, 24.48% and 45.64% when treated with heat treatment temperature at 200°C, 400°C and 600°C respectively.
3. The use of heat at 200°C, 400°C and 600°C showed decrease in liquid limits by 6.05%, 7.76% and 4.69% respectively, while Plastic limit increased with increase in heat treatment temperatures..The plasticity index value of laterite samples decreased from 13.41% to 8.33% which corresponds to showed decrease in PI by 37.88 %, 30.50% and 28.19% when treated with heat at 200°C, 400°C and 600°C respectively .
4. The pretreatment with heat at temperature of 200°C showed reduction in activity of the soil sample from 0.73 to 0.45 which corresponds to 42.46% reduction. This implies improvement in soil samples, because reduction in activity can result into reduction in changes in the volume of material during swelling or shrinkage that can lead to distress in pavement structure.
5. The moisture–density relationship of laterite subjected to heat treatment at different temperature and cutback bitumen mixtures at the West African Standard Compaction

(WASC) energy, showed that Maximum Dry Density (MDD) generally decreased with bitumen content between 0 % and 6% bitumen content and increased between 6 and 9% bitumen content, at all heat treatment temperatures. Optimum moisture content (OMC) decreased with bitumen content at all heat treatment temperatures.

6. The maximum CBR value of 28.52 % at West African Standard Compaction (WASC) energy, of laterite specimens, cured for 6 days and soaked for 24 hours, as specified by the Nigerian General Specification (Roads & Bridges) (1997), was obtained when laterite was treated with 6 % bitumen and subjected to heat treatment temperature of 400°C for one hour. This correspond to 44.8% increase in strength
7. Since the values specified for CBR at the energy level of WASC for subgrade, sub-base and base course materials by the Nigerian code are 10, 30 and 80% respectively. The lateritic soil material can only be used as sub grade/fill material. Its usage as sub-base will require some further treatment.
8. Maximum 7 day UCS value of 417.23 kN/m² corresponding to 56.6 % increase in strength above values obtained with the natural laterite was obtained when laterite was subjected to heat at a temperature of 200°C for one hour and treated with 3 % bitumen.
9. Highest resistance to loss in strength of 24.58% for WAS compaction energy obtained when laterite was subjected to heat at a temperature of 200°C for one hour and treated with 3 % bitumen fell below allowable 80% resistance to loss in strength recommended

by Ola (1975) though the immersion in water were above the recommended 4 days soaking period.

5.2 Conclusion

The following conclusions can be drawn from the study.

1. The use of heat improved the properties of the soil studied, because at 200°C, 400°C and 600°C decrease in liquid limits by 6.05%, 7.76% and 4.69% respectively were observed, while Plastic limit increased with increase in heat treatment temperatures..The plasticity index value of laterite samples decreased by 37.88 %, 30.50% and 28.19% when treated with heat at 200°C, 400°C and 600°C respectively . The decrease in liquid limits and plasticity index imply reduction in compressibility and increase in shear strength of the treated soil samples.
2. The moisture–density relationship of laterite subjected to heat treatment at different temperature and cutback bitumen mixtures at the West African Standard Compaction (WASC) energy, showed that Maximum Dry Density (MDD) generally decrease with bitumen content between 0 % and 6% bitumen content and increased between 6 and 9% bitumen content, at all heat treatment temperatures. Optimum moisture content (OMC) decreased with bitumen content, and heat treatment temperature.
3. Maximum 7 day UCS value of 417.23 kN/m² corresponding to 56.6 % increase in strength above values obtained with the natural laterite was obtained when laterite was subjected to heat at a temperature of 200°C for one hour and treated with 3 % bitumen.
4. Highest resistance to loss in strength of 24.58% for WAS compaction energy obtained when laterite was subjected to heat at a temperature of 200° for one hour and treated with

3 % bitumen fall below allowable 80% resistance to loss in strength recommended by Ola (1978) though the immersion in water were above the recommended 4 days soaking period.

5.3 Recommendations

Based on findings from the study, heat at a temperature of 200°C for one hour with 3 % bitumen content is suggested as treatment for the MH laterite studied before use in flexible pavement as sub grade/fill material and before use as a material under rigid pavement or under Airfield etc. Its usage as sub-base will require some further treatment.

Trial of higher grades in the series of cut-back bitumen for the stabilization be also considered in future research

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PLATES

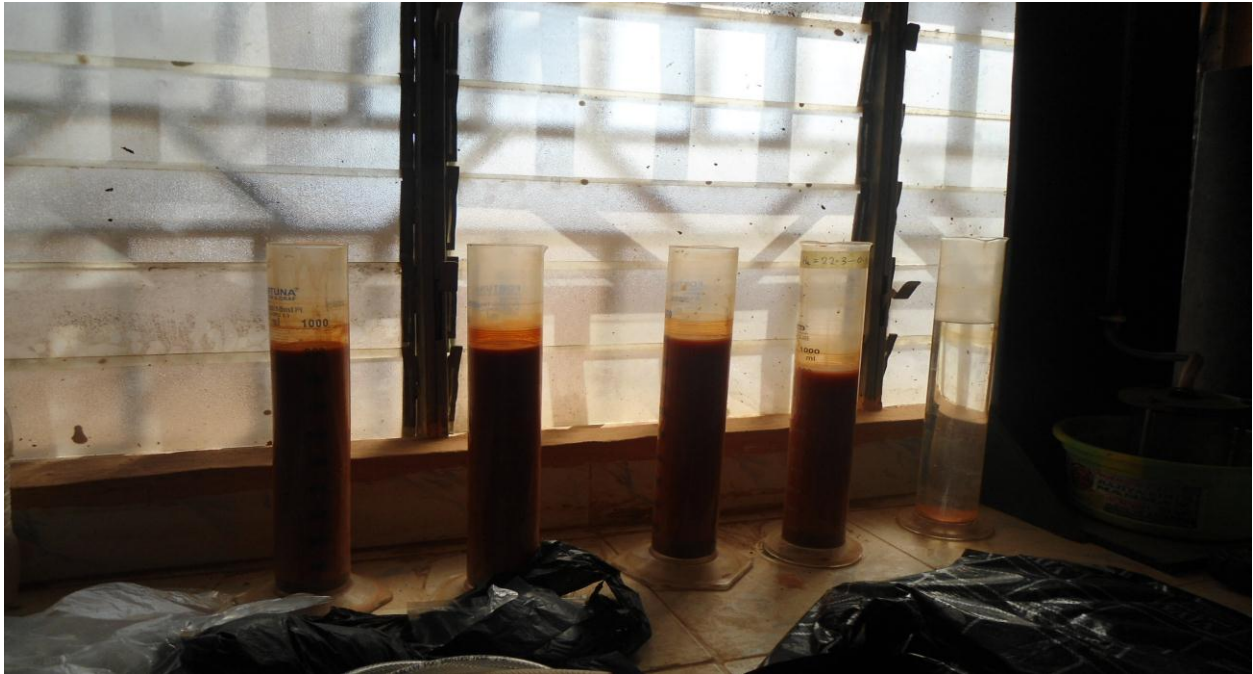


Plate I: Hydrometer Analysis of Untreated and Treated Soils



Plate II: Some UCS specimen under soaked condition



Plate III: UCS Specimen under curing



Plate IV: UCS specimen under test



Plate V: Failure pattern of the UCS Specimen



Plate V1: CBR specimen preparation



Plate VII: CBR Specimen strength measurement

