

**LABORATORY AND FIELD EXPERIMENTATION ON THE USE OF CEMENT
AND CALCIUM CARBIDE RESIDUE FOR STABILIZATION OF LATERITIC
SOIL**

BY

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ABSTRACT

A fine lateritic soil collected at Lapai-Gwari along Talba farms, Niger State, Nigeria, was collected using the method of disturbed sampling. The soil was characterized and treated with cement and calcium carbide residue (CCR). The untreated lateritic soil and lateritic soil treated with 4 and 4% cement which was admixed with 0 and 2% CCR each. Compaction tests and California Bearing Ration (CBR) tests were carried out in the laboratory to evaluate the laboratory MDD and OMC as well as the laboratory CBR. The soil was divided into category A (soil treated with 4% cement and 2% CCR), category B (soil treated with 4% cement alone) and category C (soil treated with 0% cement and 0% CCR). These three categories of soil mixtures were constituted with their predetermined moisture contents and placed on a road at three different sections (A, B and C). A light weight roller was used to compact the soil mixes of two lifts at 150mm thick each. The roller was allowed enough passes until the laboratory predetermined MDD was achieved. In-situ densities were evaluated using sand replacement method while in-situ CBR was evaluated using dynamic cone penetration (DCP) tests after 1 day, 7, 14, 28, 60 and 90 days. Result revealed that laboratory MDD and In-situ densities were observed to be in agreement. However, the laboratory CBR and field in-situ CBR showed wide differences. The in-situ CBR values were observed to increase with increase in curing days which confirms the action of the chemicals used in the stabilization. Based on the results obtained, the use of 4% cement plus 2% CCR is recommended for the treatment of Lapai-Gwari laterite for use as base material. The use of CCR as stabilizing agent will ensure economy in road construction, while providing an effective way of disposing CCR which has negative impact on the environment.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Treatment of soils to make them suitable for use as road building material is an age-long practice, normally carried out with the aid of stabilizing agents. Stabilizing agents are additives used to initiate reactions that help improve the strength properties of soils. Common traditional stabilizing agents include cement, lime and bitumen (Ingles and Metcalf, 1992). In tropical region of the world laterite is the common soil used for road construction. Ola (1983) stated that laterite is one of the most common borrow material that is stabilized and used in the construction of different layers of flexible pavement.

Laterite is a common construction material available in almost all the countries of the humid tropics of the world. It is a soil group commonly found in the leached soils of the humid tropics and is formed under weathering systems that cause the process of laterization (Gidigas, 1976). Lateritic soils are composed almost entirely of iron and aluminium oxides. According to Thagesen (1996), they are usually reddish in colour and are the least soluble product of rock weathering in tropical climates. They are formed in regions of high temperature and abundant rainfall, where the soils are highly leached as common in the tropics (Okafo and Okonkwo, 2009). Nwankwoala and Amadi (2013) stated that lateritic soils are very important in the construction industries in Nigeria, as construction material and foundation support for engineering structures.

Almost all laterites are rusty-red coloration, because of high iron oxide content they develop by intensive and prolonged weathering of the underlying parent rock. Tropical weathering (laterization) is a prolonged process of chemical weathering which produces a wide variety in thickness, grade, chemistry and ore mineralogy of the resulting soils (Tardy, 1997). They

are 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 (Ola, 1983). They are formed in hot, wet tropical regions with an annual rainfall between 750 to 3000mm, usually in area with a significant dry season on a variety of different types of rocks with high iron content (Bell, 1993). Ford (1999) stated that laterite is found mainly but not exclusively, as a residual weathering product partially or totally decomposed basalts and other basic to intermediate igneous rocks

Nigeria is one of the countries blessed with vast deposits of laterite, which is residual in nature and one of the cheapest materials for road construction (Umar and Elinwa, 2005). However, not all of these deposits are suitable for use as base and sub-base materials in their natural state. Treatment with additives is normally required before such class of laterites can attain the desired properties (Umar and Elinwa, 2005).

Soil stabilization is the method of improving soil properties by adding suitable stabilizers to it. It is very important to achieve the required strength for the soil especially in road construction project where an unsuitable base can lead to sudden failure of road (Garber and Hoel, 2000). Soil stabilization is defined as the alteration of any property of a soil to improve its engineering performance. It can be done manually, mechanically or chemically depending on the type of material to be stabilized (Murthy, 2012). Garber and Hoel (2000) described soil stabilization as the treatment of natural soil to improve its engineering properties. In general, soil stabilization is the process of creating or improving certain desired properties in a soil material so as to render it useful for a specific purpose.

Lime and cement have been meaningfully used for soil stabilization and improvement (Riells and Brooks, 1999). Improvement in soil properties treated with cement has been

attributed to hydration reaction of the cement, while changes in properties of soil associated with the use of lime can be attributed to the cation exchange, pozzolanic and carbonation reactions of lime (Riells and Brooks, 1999).

A number of researches have been made on utilization of locally-available waste materials to stabilize soil for construction purposes such as bagasse ash, rice husk ash, locust bean podash, and sugarcane straw ash mixed to stabilize laterite (Osinubi and Stephen, 2005). Researchers have shown that utilization of wastes has resulted to considerable savings in construction costs as well as improvement in engineering properties of the soil (Umar and Elinwa, 2005).

Some deposits of lateritic soils are in many cases, classified as fair to poor clayey soil that exhibit unstable properties. This has led to the development of soil stabilization techniques which could alter the physical and engineering properties of such soils thereby improving their properties (Onyelowo and Okafor, 2012). The use of calcium carbide residue in addition to cement and other pozzolanic materials, have been studied by researchers (Horpibulsuket *al.*, 2013; Isah and Sharmila, 2015; Akinwumiet *al.*, 2019; Manasseh and Edeh, 2014; Bandyopadhyay *et al.*, 2016). All these studies were carried out at laboratory stage to evaluate the potential use of calcium carbide residue as stabilizing material. This study was intended to investigate the laboratory and field performance of lateritic soil treated with cement and Calcium Carbide Residue (CCR).

1.2 Problem Statement

Road building materials in developing nations have been a major challenge to highway engineers in the construction industries. This problem is compounded, where locally available materials may not be suitable for use in road construction. To overcome such challenges, emphasis is normally placed on importation of suitable materials from other

locations, a factor responsible for the high cost of road construction. To reduce cost, one viable alternative is stabilization of the locally available material within the vicinity, for use in the construction. One of such materials that are readily available in Nigeria is laterite (Manassehand Edeh, 2014). CCR is a by-product from the production of acetylene gas (C_2H_2), used in oxy-acetylene welding. It has similar chemical and mineralogical composition to hydrated lime (Osinubi, 2009). CCR is a waste which normally goes to the dump sites creating nuisance to the environment. CCR has been identified in recent years as an alternative and sustainable stabilization additive. According to Horpibulsuk (2012) soil stabilization using CCR yielded comparatively higher strength than that of hydrated lime. Little or no work has been done to investigate the field performance of soil treated with cement and CCR.

1.3 Aim and Objectives of Study

1.3.1 Aim

The aim of this research is to study stabilization of deficient lateritic soil using cement and Calcium Carbide residue.

1.3.2 Objectives

In order to achieve the aim, the following objectives were considered;

- i. Determination of index properties of the lateritic soil,
- ii. Determination of compaction characteristics of the lateritic soil and the lateritic soil treated with varied composition of cement and CCR.
- iii. Determination of California Bearing Ratio (CBR) of the natural lateritic soil and the lateritic soil treated with varied composition of cement and CCR.
- iv. Determination of field performance of the treated soil.

1.4 Justification

Although, there is abundance of laterite soil in Nigeria and its potentials as construction material, most available laterite soil, in their natural state are not suitable for use in road construction, hence they need to be improved in order to exhibit good engineering properties. This study tends to investigate the suitability of using both cement and calcium carbide residue in improving the engineering properties of deficient lateritic soil.

1.5 Scope of the Research

The scope of this research involved sample collection, determination of index properties of the lateritic soil, compaction characteristic, CBR of the lateritic soil and lateritic soil treated with varied composition of cement and Calcium carbide residue. Also place and compacted the mixture and predetermined field densities and CBR. The field CBR and densities of the compacted surface was determined after 1, 7, 28, 60 and 90 days of placement.

2.0

LITERATURE REVIEW

2.1 Laterite

The historical identification of lateritic soil as an earth material, with distinctive properties, dates back to 1807 when Buchanan first encountered a material in India, which he called laterite. He further defined it as soft enough to be readily cut into blocks by an iron instrument during wet condition, but which upon exposure to air quickly becomes as hard as brick, and is reasonably resistant to the action of air and water Vallerga and Rananandana (1969).

The name laterite was coined by an English surgeon Francis Buchanan in 1807 in India from a Latin word "later" meaning brick in the 19th century. He coined the term laterite when he wrote "What I have called indurated clay is one of the most valuable materials for building. It is diffused in immense masses, without any appearance of stratification and is placed over the granite that forms the basis of Malayala. It is full of cavities and pores, and contains a very large quantity of iron in the form of yellow and red colours. In the mass, while excluded from the air, it is so soft, that any iron instrument readily cuts it, and is dug up in square masses with a pick-axe, and immediately cut into the shape wanted with a trowel, or large knife. It very soon becomes as hard as brick, and resists the air and water much better than any brick I have ever seen. The most proper English name would be laterite, from lateritis, the appellation that may be given to it in science". Since then lots of researches and write ups have been carried out on laterite and a lot of terms referring to many soil types have been produced. There is a tendency to apply the term to any red soil and rock in the tropics (Charman, 1988).

Laterite is a common construction material available in almost all the countries of the humid tropics of the world. It is a soil group commonly found in the leached soils of the humid tropics and is formed under weathering systems that cause the process of laterization

(Gidigas, 1976). Its formation is favoured by the factors encouraging laterization processes such as high intensity rainfall, high vegetation cover, permeable soil profile and alternating ground water movement. 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. These soils may contain substantial amount of silica in the form of clay silicate minerals (Mustapha *et al.*, 2014).

Lateritic soils are soils that are composed almost entirely of iron and aluminium oxides; they are usually reddish in colour and are the least soluble product of rock weathering in tropical climates (Osinubi, 2009). They are formed in regions of high temperature and abundant rainfall, where the soils are highly leached as common in the tropics (Okafor and Okwonkwo, 2009). Lateritic soils are very important in the construction industries and activities as construction material and foundation support for engineering structures (Onyelowe and Okafor, 2012).

Lateritic soil can be defined as the products of tropical weathering with red, reddish brown and dark brown colour, with or without nodules or concretions and generally but not exclusively found below hardened ferruginous crust or hard pan, (Ola, 1983). Laterite is defined as a highly weathered tropical soil rich in secondary oxide of any or a combination of iron, aluminium and manganese (Onyelowe and Okafor, 2012).

Laterite is described as a product of in-situ weathering in igneous, sedimentary, and metamorphic rocks commonly found under unsaturated conditions (Osinubi, 2009). Lateritic soil is one of the most important and common materials used in earth work engineering construction in the tropics and subtropics where it is in abundance.

2.1.1 Formation of laterite

Lateritic soils are composed almost entirely of iron and aluminium oxides. They are usually reddish in colour and are soluble product of rock weathering in tropical climates (Osinubi 2009). They are formed in regions of high temperature and rainfall, where the soils are highly leached as common in the tropics Okeet *al.* (2009) and Nwankwoalaet *al.* (2014). Lateritic soils are very important in the construction industries and activities as construction material and foundation support for engineering structures Nwankwoala and Amadi (2013) and Nwankwoalaet *al.* (2014).

Laterites occur mostly in tropical and sub-tropical regions with hot, humid climatic conditions. It has been suggested that a minimum annual temperature of around 25°C is needed for their formation, and in seasonal situations there should be a coincidence of the warm and wet periods. If there is high rainfall during the cold seasons, laterite does not develop freely. The minimum rain fall required for the formation of laterites is generally at least 750mm. The higher rain fall above this value, the greater the leaching effect, which removes the silica, reducing the Silica/Sesquioxide ratio and increase the degree of laterization (Bell, 1993).

Lateritic soil in its natural state generally have low bearing capacity and low strength due to high content of clay. When lateritic soil contains a large amount of clay materials its stability and strength and stability cannot be guaranteed under load in presence of moisture. Lateritic soil consists of high plastic clay; the plasticity of the soil may result to cracks and damage on pavement, roadways, building foundations or any civil engineering construction projects (Alhassan, 2008)

Laterite soils are formed in hot, wet tropical regions with an annual rainfall between 750mm and 3000mm, usually in area with a significant dry season on a variety of different types of rocks with high iron content (Bell, 1993).

Laterite soils are formed in situ from the intense weathering of parent material, whether primary or sedimentary, in the tropical and subtropical climate environment. This weathering process primarily involves the progressive chemical alteration of primary minerals, the release of iron and aluminium sesquioxides, increasing loss of silica and the increasing dominance of new clay materials (such as smectites, allophanes, halloysite, and as weathering progresses, kaolinite) formed from dissolved materials (Charman, 1988). The genesis of laterite as the weathering process which involves leaching of silica, formation of colloidal sesquioxides, and precipitation of the oxides with increasing crystallinity and dehydration as the soil is weathered (Osinubi, 2009). The three major processes of weathering are the physical, chemical and biological processes. The physical weathering is pre-dominant in the dry climates while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature (Charman, 1988).

Their formation consists of leaching out silica and bases and accumulation of oxide of iron, aluminium or both which is known as laterization (Amadi and Okeiyi, 2017). In this process, sesquioxides of iron and aluminium are absorbed into the surface of the clayey constituents through the interaction of the positively and negatively charged particles.

As the disintegration of underlying rocks occurs, the primary elements are broken down by the processes of physical and chemical weathering to simple ionic form. The silica and bases in the weathered material such as sodium, potassium, calcium magnesium etc are washed out by the percolated rain water and oxides and hydroxides of sesquioxides are accumulated, enriching the soil and giving the soil its typical red colour (Charman, 1988). The topography and drainage of an area also influences the rate of weathering because to some extent, it determines the amount of water available for laterization to occur and the rate at which it moves through the weathering zone (Osinubi and Stephen, 2005). The structure of lateritic soil varies with the type of parent rock from which it was formed, the location and also on

the weathering processes that lead to its formation. The structural development depends on the deposition of iron oxides at different stages of the weathering process. Field and laboratory studies have shown that residual soils consist of different zones of weathering with differing morphological, physical, and geotechnical characteristics; and vary for different locations due to the heterogeneous nature and highly variable degree of weathering (Osinubi and Stephen, 2005). Research has shown that laterite soils possess very favourable geotechnical properties, and this is evident in the widespread use of the materials in the construction industry (Sherwood, 1993). The geotechnical properties of laterite are very much affected by compaction. Before using this soil for any construction purpose, the soil has to undergo some stabilization process so as to avoid failure or collapse of the construction work. Unlike other soil types, the properties of laterite vary from region to region. The physical properties of residual soils, commonly known as the index properties, vary from region to region due to the heterogeneous nature and highly variable degree of weathering controlled by regional climatic and topographic conditions, and the nature of bedrock, (Little and Nair, 2009). It also varies with the depth of the soil and can be determined by simple laboratory tests. Studies on effect of weathering on physical properties of laterite soil was carried out by (Little and Nair 2009).

Laterite soil chemistry and mineralogy is shown by studies to greatly influence the geotechnical properties, and in certain circumstances, significantly affects the economic potential in the construction industry (Sherwood, 1993). The soil profile of laterite is defined as that in which laterite horizon exists or is capable of developing under favourable conditions (Sherwood, 1993). The alteration of rock by the processes of chemical weathering takes place progressively through a series of events and stages which result in a profile of weathering (Cortellazzo and Cola, 1999).

2.1.2 Geotechnical properties of lateritic soils

The recognition of soil behaviour in solving engineering and environmental problems such as swelling soil especially expansive lateritic soils that can cause revealing damage to road construction and other engineering application is the sole aim of geotechnical engineering (Abubakar, 2006; Oke and Amadi, 2008). One of the major causes of road accident is bad road which is usually caused by wrong application of constructional materials especially lateritic soil as sub-base and base material by construction companies (Oke *et al.*, 2009 and Nwankwoala *et al.*, 2014). The characteristics and durability of any constructional material is a function of its efficiency in response to the load applied on it (Oke *et al.*, 2009 and Nwankwoala and Amadi, 2013). The mineralogical composition of the lateritic soil has an influence on the geotechnical parameters such as specific gravity, shear strength, swelling potential, Atterberg limits and bearing capacity (Nwankwoala and Amadi, 2013).

2.2 Soil Stabilization

Stabilization of lateritic soil can be described as the treatment of natural soil to improve its engineering properties. In general, soil stabilization is the process of creating or improving certain desired properties in laterite soil materials so as to render it stable for construction purposes (Garber and Hoel, 2000). Soil stabilization involves the use of stabilizing agents in weak soils to improve its geotechnical properties such as compressibility, strength, permeability durability (Garber and Hoel, 2000).

Soil stabilization is a method of improving soil properties by blending and mixing other materials. The process also includes improving the shear strength parameters of soil and thus increasing the bearing capacity of soil. Stabilization consists of various methods employed for modifying the properties of a soil to improve its engineering performance Habiba (2017).

Lateritic soil stabilization is widely known as an effective alternative for improving engineering properties of soil (MacLaren and White, 2003). Stabilization can be derived from mechanical or chemical means. Mechanical stabilization or compaction is the densification of soil by application of mechanical energy, while chemical stabilization involves mixing or injecting soil with chemically active compounds such as cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen (MacLaren and White, 2003). Among these, the most widely used chemical additives are cement, lime and fly ash, (Horpibulsuk et al., 2013). These additives are most frequently associated with improving the strength and reducing the swelling properties of laterite soils and can be used with a variety of soils (Horpibulsuk et al., 2013).

Garber and Hoel (2000) describe soil stabilization as the treatment of natural soil to improve its engineering properties. In general, soil stabilization is the process of creating or improving certain desired properties in a soil material so as to render it stable useful for a specific purpose. Since the inception of this process of stabilization, most soil materials, which have been thought not useful, have found application in many areas of both civil and building construction.

Most of stabilization has to be undertaken in soft soils such as silty, clayey peat or organic soils in order to achieve desirable engineering properties. Fine-grained granular materials are the easiest to stabilize due to their large surface area in relation to their particle diameter (Sherwood, 1993). Soil stabilization aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water to be affected than other soils due to alteration of plasticity properties of clays (Sherwood, 1993). The simplest stabilization processes are compaction and drainage for instance, if water drains out of wet soil it becomes stronger. The other process is by improving gradation of

particle size and further improvement can be achieved by adding binders to the weak soils (Rogers *et al.*, 1993).

2.2.1 Methods of stabilization

Soil stabilization is based on physical, chemical, or biological methods for improving properties of natural soil using a specific engineering approach. Generally, the processes of soil stabilization involves analysis of the properties of a given soil, finding a specific economical method for modifying the soil, determining feasibility, and using the method and materials during construction, costs of materials, construction and maintenance Hilf (1991).

Four methods of stabilization offering practical and economical solutions can be categorized as: granular stabilization, chemical stabilization, thermal stabilization, and electrokinetic stabilization. Granular stabilization is a combined method of physical and chemical stabilization to modify the granular bearing skeleton with cementing materials Hilf (1991).

Stabilization method can be achieved in two ways:

- i. In situ stabilization
- ii. Ex - situ stabilization

The decision to adopt any type of technology depends on which soil properties have to be modified. The main properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability Sherwood (1993).

2.2.2 In-situ stabilization

The method involves on site soil improvement by applying stabilizing agent without removing the bulk soil. This technology offer benefit of improving soils for deep foundations, shallow foundations and contaminated sites. Planning of the design mix involves the selection and assessment of engineering properties of stabilized soil and improved ground. The purpose is to determine the dimensions of improved ground on the basis of appropriate

stability and settlement analyses to satisfy the functional requirements of the supported structure (Keller, 2011). The technology can be accomplished by injection into soils a cementitious material such cement and lime in dry or wet forms. The choice to either use dry or wet deep mixing methods depend among other things; the in-situ soil conditions, in situ moisture contents, effectiveness of binders to be used, and the nature of construction to be founded. Depending on the depth of treatment, the in situ stabilization may be regarded as either deep mixing method or mass stabilization (Porbaha et al., 2005).

2.2.3 Ex-situ stabilization

The technology involves dislodging of the soils and or sediments from the original position and moving it to other place for the purpose of amendment. These can be encountered in dredging of river channel and Ports. The main objectives of dredging can be either for amending the contaminated sediments to reduce toxicity and mobility or to maintain or deepen navigation channels for the safe passage of ships and boats (EuroSoilStab, 2002). Offsite treatment of the sediment can be done in Confined Disposal Facilities (CDF) and then be used or disposed at designated site. Method of removal, means of transportation, availability of treatment location, disposal site or demand for reuse is key factors to consider when planning for ex-situ stabilization (Hebib and Farrell, (1999). Some stabilization techniques are:

- i. Mechanical Stabilization
- ii. Chemical Stabilization

2.2.4 Mechanical Stabilization

The Objective of mechanical stabilization is to admixture different available soils and when compacted, will give the desired stability. In some areas the natural soil at an existing location may have unstable properties in nature or poor California bearing ratio (CBR)

values. This indicates the presence of clay, silt or fine sand. Suitable soil may be selected and this is to be mixed with the available soils to improve the soil properties at a lesser cost in manpower and materials to achieve best results or increasing the strength or CBR value of the admixtures Guo (2014).

Under this category, soil stabilization can be achieved through physical process by altering the physical nature of native soil particles by either induced vibration or compaction or by incorporating other physical properties such as barriers and nailing.

2.2.5 Chemical stabilization

This is the oldest and most common method by which chemical compounds are added to the soil to improve its volume stability, strength and stress-strain behaviour, permeability, and durability. Chemical reactions, including hydration, ion exchange, pozzolanic reaction, flocculation, precipitation, polymerization, oxidation, and carbonation, occur between the soil and additives (Hans and Sibel, 1991).

Under this category, soil stabilization depends mainly on chemical reactions between stabilizers, cementitious material and soil minerals, pozzolanic materials to achieve the desired result.

Through soil stabilization, unbound materials can be stabilized with cementitious materials such as cement, lime, fly ash, bitumen or combination of these (Ingles and Metcalf, 1992). For a successful stabilization, a laboratory tests followed by field tests may be required in order to determine the engineering and environmental properties. Laboratory tests although may produce higher strength than corresponding material from the field, but will help to assess the effectiveness of stabilized materials in the field. Results from the laboratory tests, will enhance the knowledge on the choice of binders and amounts (EuroSoilStab, 2002).

For the past several years researchers have recognized the use of locally available materials which are cost effective available from industrial and agricultural wastes to improve the engineering properties of laterite soils with the aim to reduce stabilization costs, related to conventional stabilizing agents such as cement and lime as well as the emission of CO₂ related to cement manufacturing process. Waste materials, such as fly ash, rice husk ash, sawdust ash, sugarcane straw ash, locust bean pod ash, bagasse ash and coconut shell ash etc has been widely applied in practice in addition to cement and lime, (Alhassan, 2008). Ca(OH)₂ rich material like calcium carbide residue (CCR) together with pozzolanic materials such as Fly ash, rice husk ash, biomass ash etc have been widely used to completely replace cement in stabilization of laterite soils (Horpibulsuket *et al.*, 2013).

Nnochiri and Aderinlewo (2016) investigated the geotechnical properties of lateritic soil stabilized with banana leaves ash. Engineering tests such as compaction, California bearing ratio and unconfined compressive strength tests were carried out on the lateritic soil at their natural states and at when the banana leaves ashes were added to the soil at varying proportions of 0, 2, 4, 6, 8 and 10% by weight of soil. The result of the strength tests showed that the banana leaves ash enhanced the strength of the lateritic soil. The unsoaked CBR value of the soil at its natural state was 10.42 % and it got to optimum value of 28.10% by addition of 4% banana leaves ash by weight of soil. The un-confined compressive strength improved from 209.18 kN/m² at natural state to 233.77 kN/m² at 4% banana leaves ashes. They concluded that the banana leaves ash satisfactorily act as cheap stabilizing agents for subgrade purposes.

Ayeni (2016) evaluated the potential of limestone ash for the improvement of some geotechnical properties of Laterite soils for construction purposes. The geotechnical properties of the soil were determined both in the natural state and after stabilization with 2, 4,

6, 8 and 10% of limestone ash waste at a normal curing of 6 hours. Accelerated curing at 40, 60 and 80°C for 24 hours was carried out for compressive strength tests for 4 and 6% respectively due to similarities in the values of both the CBR and compressive strength. The dry density and plasticity index decreased while liquid limit, plastic limit, compressive strength and CBR increased with increasing percentages of limestone ash. Maximum strength was achieved at 6% proportion of limestone ash for CBR and compressive strength respectively. The results of this research indicate that limestone ash is comparatively suitable for the chemical stabilization of laterite soils.

Nnochiri and Ogundipe (2016) assessed the geotechnical properties of lateritic soil stabilized with Ground-nut Husk Ash. Engineering property tests such as California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS) and compaction tests were performed on both the natural soil sample and the stabilized lateritic soil, which was stabilized by adding Ground -nut Husk Ash, GHA, in percentages of 2, 4, 6, 8 and 10 by weight of the soil. The results showed that the addition of GHA enhanced the strength of the soil sample. The Maximum Dry Density (MDD) reduced from 1960 to 1760 kg/m³ at 10% GHA by weight of soil. The Optimum Moisture Content (OMC) increased from 12.70 to 14.95%, also at 10% GHA by weight of soil. The unsoaked CBR values increased from 24.42% to 72.88% finally, the UCS values increased from 510.25 to 1186.46 kN/m², for both CBR and UCS, the values were at 10% GHA by weight of soil. They concluded that GHA performs satisfactorily as a cheap stabilizing agent for stabilizing lateritic soils especially for subgrade and sub base purposes in road construction.

2.3 Stabilizing Agents

These are primary binders or secondary binders materials that when in contact with water or in the presence of pozzolanic minerals reacts with water to form cementitious composite materials (MacLaren and White, 2003). The commonly used binders are:

- i. cement
- ii. lime
- iii. fly ash
- iv. blast furnace slag

2.3.1 Cement

Cement is the oldest binding agent since the invention of soil stabilization technology in 1960's. It may be considered as primary stabilizing agent or hydraulic binder because it can be used alone to bring about the stabilizing action required (Sherwood, 1993).

Afolayan (2017) evaluated the effect of Cement and Lime on selected engineering properties of lateritic soils. In order to study this effect, fresh laterite was obtained and tested for its index and geotechnical properties. Afterwards, the soil sample was altered with additives proportions which include 0, 2.5, 5, 7.5, and 10% of both cement and lime replacement by dry weight. On examination laterite, it was discovered that the laterite can be classified as good. After carrying out the engineering properties on the sample, it was observed that the maximum dry density reduced with increase in the lime content and for cement content at 2.5 and 10% but increases at 5-7.5%. Also, the optimum moisture content increased for increasing lime content and fluctuates for increasing cement content. However, an increase in the CBR was noticed for increase in cement but a low effect for lime addition. He concluded that lime and cement modifies the chemical property of the tested soil with cement the most suitable.

Dumessa and Verma (2018) assessed the effects of cement on the behaviour of compacted lateritic soil to examine the benefits of cement stabilized lateritic over selective borrow material both in the quality and saving cost. The basic index and engineering properties of lateritic soil were determined following the ASTM procedure and classified as A-7-5 and

inorganic silt according to the AASHTO classification system and Unified Soil Classification System, respectively. The soil samples were subsequently stabilized with 3, 5, 7 and 9% cement. Results showed that stabilization of the lateritic soils with cement increased the maximum dry density, California bearing ratio (CBR), and Un-confined Compressive Strength while there was a reduction in Optimum Moisture Content as cement up to 7%. The effect of curing time indicated marginal on the compaction characteristics and CBR, whereas it revealed substantial effects in unconfined compressive strength tests results. There-fore, they concluded that Lateritic soils stabilized with 5% cement proved better quality than the selective borrow material taken from the existing nearby areas and its economic benefits over selective borrow material by 26.95%. Thus, the stabilization of lateritic with cement is effective in the constructions of stable and durable sub-grade and subbase course layer. Cement reaction is not dependent on soil minerals, and the key role is its reaction with water that may be available in any soil (EuroSoilStab, 2002). This can be the reason why cement is used to stabilize a wide range of soils. Numerous types of cement are available in the market; these are ordinary Portland cement, blast furnace cement, sulphate resistant cement and high alumina cement. Usually the choice of cement depends on type of soil to be treated and desired final strength (EuroSoilStab, 2002).

Hydration process is a process under which cement reaction takes place. The process starts when cement is mixed with water and other components for a desired application resulting into hardening phenomena. The hardening (setting) of cement will enclose soil as glue, but it will not change the structure of soil (EuroSoilStab, 2002). The hydration reaction is slow proceeding from the surface of the cement grains and the centre of the grains may remain unhydrated (Sherwood, 1993). Cement hydration Ca(OH)_2 is a complex process with a complex series of unknown chemical reactions (MacLaren and White, 2003).

2.3.2 Lime

Lime provides an economical way of soil stabilization. Lime modification describes an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reaction (Sherwood, 1993).

In contrast to cement, delay in compaction for lime-stabilized soils may have some advantages. Lime stabilized soil require mellowing period to allow lime to diffuse through the soil thus producing maximum effects on plasticity. After this period, lime stabilized soil may be remixed and given its final compaction resulting into remarkable strength (Sherwood, 1993). Different researchers have reported the effective use of lime in soil stabilization. Lime according to (Little and Nair (2009) can be used to stabilize fine grained soils with benefits of reduction in the plasticity index and swell potential, increase in soil workability and strength properties. For effective stabilization of soil with lime, (Sherwood, 1993) recommended the use of lime in soils with plasticity index value of 10 and above. Negiet *al.* (2013) reported that lime is an excellent stabilizing agent, especially in the treatment of highly active soils which undergo frequent swelling and shrinkage. Garber and Hoel (2010) recommended the treatment of soil with lime to reduce the plasticity index value to 10 % or less than 12 % before the use of cement in soil stabilization.

Amadi and Okeiyi (2017) undertook a laboratory study to evaluate and compare the stabilization effectiveness of different percentages (0, 2.5, 5, 7.5 and 10%) of quick and hydrated lime when applied separately to locally available lateritic soil. Performance evaluation experiments included: Atterberg limits, compaction, unconfined compression tests, California bearing ratio (CBR), swelling potential using CBR instrument and hydraulic conductivity. The soil mixtures used for unconfined compressive strength (UCS), CBR, swelling potential and hydraulic conductivity tests were compacted at optimum moisture content using the British standard light compactive effort and cured for 28 days. It was found that treatment with lime on plasticity characteristics resulted in a reduction of

plasticity index (PI) of soil mixtures while the quicklime caused the soil to have lower plasticity. The addition of either quick or hydrated lime resulted in a decrease in the maximum dry unit weight and a slight increase in the optimum moisture content while hydrated lime treated specimens exhibited higher dry unit weight than that achieved with quick lime addition. There is generally an increase in strength with lime content regardless of the type and higher UCS especially at higher dosages (7.5 and 10%) was produced when soil sample was treated with quicklime. Results of CBR test for the stabilized soil show that the addition of either the quicklime or hydrated lime significantly improved the bearing strength (CBR) of the soil while quicklime-stabilized soil have superior load bearing capacity. Finally, the two types of lime are effective in reducing swelling potential and quicklime treated specimens reached slightly lower swelling values than the hydrated lime while no appreciable distinction in hydraulic conductivity values of specimens treated with the two types of lime was observed. They concluded that quicklime is adjudged to have exhibited somewhat superior engineering properties and therefore creates a more effective stabilization alternative for the soil.

Todingra *et al.* (2017) conducted Laboratory and field testing programs to evaluate lime-cement treated laterite soil for application in the evaluation procedures of pavement structures. The laboratory program consisted of performing unconfined compression test on lime-cement treated laterite soil. The field testing program included conducting in-situ tests of light weight deflectometer (LWD), and field CBR. Based on the performance of the treated laterite soil observed in the study they suggested that the treated laterite soil with lime and cement content of 12 and 5%, respectively, can be employed as a pavement foundation layer to bear the vehicle loads. Quicklime when mixed with wet soils, immediately takes up to 32% of its own weight of water from the surrounding soil to form hydrated lime; the generated heat accompanied by this reaction will further cause loss of

water due to evaporation which in turn results into increased plastic limit of soil (Sherwood, 1993).

Lime stabilizations technology is mostly widely used in geotechnical and environmental applications. Some of applications include encapsulation of contaminants, rendering of backfill such as wet cohesive soil, highway capping, slope stabilization and foundation improvement such as in use of lime pile or lime-stabilized soil columns (Ingles and Metcalf, 1992).

2.3.3 Fly-ash

Fly ash is a by-product of coal fired electric power generation facilities; it has little cementitious properties compared to lime and cement. Most of the fly ashes belong to secondary binders; these binders cannot produce the desired effect on their own.

However, in the presence of a small amount of activator, it can react chemically to form cementitious compound that contributes to improved strength of soft soil. Fly ashes are readily available, cheaper and environmental friendly. There are two main classes of flyashes; class C and class F (White, 2005). Class C fly ashes are produced from burning sub bituminous coal; it has high cementing properties because of high content of free CaO. Class C from lignite has the highest CaO (above 30%) resulting in self-cementing characteristics (FM 5-410). Class F fly ashes are produced by burning anthracite and bituminous coal; it has low self-cementing properties due to limited amount of free CaO available for flocculation of clay minerals and thus require addition of activators such as lime or cement.

Cokca (2001) used high-calcium and low-calcium class C fly ashes for stabilization of an expansive soil and evaluation of the expansive soil-lime, expansive soil-cement, and expansive soil-fly ash systems. Lime, cement and fly ash were added to the expansive soil at

different percentages. The specimens were subjected to chemical composition, grain size distribution, consistency limits, and free swell tests. Also, the Specimens with fly ash were cured and after that they were subjected to oedometer free swell tests. It can be concluded that the expansive soil can be successfully stabilized by fly ashes. Furthermore, plasticity index, activity, and swelling potential of the samples decreased with increasing percentage of stabilizer and curing time.

Negiet *al.* (2013) evaluated the effectiveness of clayey soil blended with sand and fly ash for soil stabilization by studying the subgrade characteristics. The purpose of this work is to find a solution for proper disposal of fly ash and also provides good subgrade material for pavement construction. The results showed that substantial improvement in compaction and California bearing ratio of composite containing clay, sand and fly ash. The swelling of the clay also reduced after stabilization. The maximum dry density of clay-sand-fly ash mix decreased with the addition of fly ash and optimum moisture content increased. Thus the stabilized soil can be used for construction of flexible pavements in low traffic areas.

Bose (2012) used fly ash to stabilize highly plastic clay. The geo-engineering properties such as, atterberg limits, grain size distribution, linear shrinkage, free swell index, welling pressure, compaction characteristics, unconfined compressive strength and CBR value of virgin clay and stabilize with fly ash were evaluated. Expansive soil was stabilized with various proportion of fly ash. The results showed that plasticity index of clay-fly ash mixes decreased with increase in fly ash content. Thus addition of fly ash increases its workability by colloidal reaction and changing its grain size. The free swell index value and swelling pressure of expansive clay mixed with fly ash decreased with increase in fly ash content. Furthermore, addition of fly ash reduced the optimum moisture content but the dry density increased and unconfined compressive strength of clay-fly ash mixes is found to be

maximum. So, it is concluded that the fly ash has a good potential for improving the engineering properties of expansive soil.

Horpibulsuk(2012)presented the characteristics of Municipal Solid Waste (MSW) incineration ash and evaluates this ash in road pavement layers through the mixture of ash with a clay soil. Chemical, physical, and mechanical tests and the mechanistic-empirical design for a pavement structure were carried out on the pure soil and also in the soil mixture with the addition of different ash content. The results showed that fly ash reduced the expansion of the material, showing an increase in CBR and resilient modulus value. Furthermore, content and type of ash was important in final results and it showed the efficacy of MSW fly ash for its use in base road pavement layers.

Mir and Sridharan (2013) studied adding, high calcium and low calcium fly ashes in different proportions to a highly expansive black cotton soil. The objective of the study was to study the effect of fly ashes on the physical, compaction, and swelling potential of black cotton soils that were reached from laboratory tests and utilization of waste material without disruptive effect on the environment. The results showed that the liquid limits, compaction characteristics and swelling potential of expansive soil–fly ash mixtures are significantly modified and improved. With the addition of fly ash to black cotton soil the maximum dry unit weight of the mixtures decreases with increase in optimum moisture content and it can be attributed to the improvement in gradation of the fly ash. Furthermore, compressibility characteristics of the expansive soil are improved with the addition of fly ash.

2.3.4 Calcium carbide residue

Calcium Carbide Residue (CCR) is a by-product of the acetylene production process that contains mainly calcium hydroxide $\text{Ca}(\text{OH})_2$. Compared to hydrated lime, Calcium carbide

residue (CCR) has similar chemical and mineralogical compositions (Horpibulsuket *al.*,2013). Calcium Carbide Residue (CCR) has been identified in recent years as an alternative and sustainable stabilization additive (Du *etal.* 2009; Horpibulsuk *etal.* 2013).

Consoliet *al.* (2001) reported the possibility of using CCR and fly ash to stabilize a nonplastic, silty sand. The study of soil stabilization with a mixture of CCR and pozzolanic materials is an engineering, economic, and environmental challenge for geotechnical engineers and researchers.

Chai and Roongreung (2003) investigated that the ratio of calcium carbide residue to rice husk ash of 50:50 by weight obtains the highest compressive strength of soil. The compressive strength of soil could be as high as 15.6 MPa at curing age of 28 days and increased to 19.1 MPa at 180 days.

Duet *al.* (2009) investigated strength and California bearing ratio properties of natural soils treated by calcium carbide residue which is used as a bankment filling material in China Highway Engineering practice. From the tests, it is found that calcium carbide residue treated soils have better performance than that of lime treated soils.

Horpibulsuk (2012) studied soil stabilization by calcium carbide residue and fly ash and he revealed that the input of CCR reduces specific gravity and soil plasticity; thus, the maximum dry unit weight and water sensitivity. Reduction in plasticity index and increase in CBR value was reported by Gawu and Gidigasu (2013) when calcium carbide residue was used in the stabilization of laterite.

Vichanet *al.* (2013) reported 50 % improvement in UCS values when Bangkok clay soil was stabilized with 15 % calcium carbide residue and Biomass ash in a ratio of (CCR: BA) 60:40.

Isah and Sharmila (2015) reported improvement in plasticity index value and increase in CBR value when a CI and CH soil were stabilized with a combination of 4 % calcium carbide residue plus 4 % coconut shell ash, and 6 % calcium carbide residue and 4 % coconut shell ash.

Manasseh and Edeh (2014) attributed differences in the properties of soil treated with hydrated lime and calcium carbide residue to loss of reactivity of calcium carbide residue due to carbonation reaction which takes place when calcium carbide slurry is exposed to air during disposal and the open drying of the slurry to remove moisture from the waste before usage in soil stabilization.

Although there have been several recent studies on the mechanical properties of CCR stabilized soils, these were predominantly undertaken in small-scale laboratory settings only. Limited field trials have been attempted to demonstrate the mechanical properties of CCR stabilized soft soils. In this study, laboratory and field tests were conducted to investigate the mechanical performances of the CCR and cement stabilized lateritic soil.

2.3.5 Blast furnace slag

These are the by-product in pig iron production. The chemical compositions are similar to that of cement. It is however, not cementitious compound by itself, but it possesses latent hydraulic properties which upon addition of lime or alkaline material the hydraulic properties can develop (Sherwood, 1993).

2.3.6 Pozzolanas

Pozzolanas are siliceous and aluminous materials, which in itself possess little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementitious properties (ASTM C 595). Clay minerals such as kaolinite,

montmorillonite, mica and illite are pozzolanic in nature. Artificial pozzolanas such as ashes are products obtained by heat treatment of natural materials containing pozzolanas such as clays, shales and certain silicious rocks. Plants when burnt, silica taken from soils as nutrients remains behind in the ashes contributing to pozzolanic element. Rice husk ash and rice straw and bagasse are rich in silica and make an excellent pozzolana (Sherwood, 1993).

2.4 Factors Affecting the Strength of Stabilized Soil

Presence of organic matters, sulphates, sulphides and carbon dioxide in the stabilized soils may contribute to undesirable strength of stabilized materials (Little and Nair, 2009).

2.4.1 Organic matter

In many cases, the top layers of most soil constitute large amount of organic matters. However, in well drained soils organic matter may extend to a depth of 1.5 m (Sherwood, 1993). Soil organic matters react with hydration product, example calcium hydroxide ($\text{Ca}(\text{OH})_2$) resulting into low pH value. The resulting low pH value may retard the hydration process and affect the hardening of stabilized soils making it difficult or impossible to compact.

2.4.2 Sulphates

The use of calcium-based stabilizer in sulphate-rich soils causes the stabilized sulphate rich soil in the presence of excess moisture to react and form calcium sulphotoaluminate (ettringite) and or thamausite, the product which occupy a greater volume than the combined volume of reactants. However, excess water to one initially present during the time of mixing may be required to dissolve sulphate in order to allow the reaction to proceed (Little and Nair, 2009).

2.4.3 Sulphides

In many of waste materials and industrial by-product, sulphides in form of iron pyrites (FeS_2) may be present. Oxidation of FeS_2 will produce sulphuric acid, which in the presence of calcium carbonate, may react to form gypsum (hydrated calcium sulphate).

The hydrated sulphate so formed, and in the presence of excess water may attack the stabilized material in a similar way as sulphate (Sherwood, 1993). Even so, gypsum can also be found in natural soil (Little and Nair, 2009).

2.4.4 Compaction

In practice, the effect of addition of binder to the density of soil is of significant importance. Stabilized mixture has lower maximum dry density than that of unstabilized soil for a given degree of compaction. The optimum moisture content increases with increasing binders (Sherwood, 1993). In cement stabilized soils, hydration process takes place immediately after cement comes into contact with water. This process involves hardening of soil mix which means that it is necessary to compact the soil mix as soon as possible. Any delay in compaction may result in hardening of stabilized soil mass and therefore extra compaction effort may be required to bring the same effect. That may lead to serious bond breakage and hence loss of strength. Stabilized clay soils are more likely to be affected than other soil (Sherwood, 1993).

2.4.5 Moisture content

In stabilized soils, enough moisture content is essential not only for hydration process to proceed but also for efficient compaction. Fully hydrated cement takes up about 20% of its own weight of water from the surrounding (Sherwood, 1993); on other hand, Quicklime (CaO) takes up about 32% of its own weight of water from the surrounding (Roger and Glendinning, 1993). Insufficient moisture content will cause binders to compete with soils in order to gain these amounts of moisture. For soils with great soil-water affinity such

as clay, peat and organic soils, the hydration process may be retarded due to insufficient moisture content, which will ultimately affect the final strength.

2.4.6 Temperature

Pozzolanic reaction is sensitive to changes in temperature. In the field, temperature varies continuously throughout the day. Pozzolanic reactions between binders and soil particles will slow down at low temperature and result into lower strength of the stabilized mass. In cold regions, it may be advisable to stabilize the soil during the warm season (Sherwood, 1993).

2.4.7 Freeze-thaw and dry-wet effect

Stabilized soils cannot withstand freeze-thaw cycles. Therefore, in the field, it may be necessary to protect the stabilized soils against frost damage (Maher *et al.*, 2003).

Shrinkage forces in stabilized soil will depend on the chemical reactions of the binder. Cement stabilized soil are susceptible to frequent dry-wet cycles due to climate changes in temperature which may give rise to stresses within a stabilized soil and, therefore, should be protected from such effects (Maher *et al.*, 2003).

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials

The materials used for this study include;

- i. Laterite
- ii. Cement
- iii. Calcium Carbide Residue (CCR)
- iv. Water

3.1.1 Laterite

Lateritic soil sample used for this work was obtained from a borrow pit located at Lapai-Gwarivillage of Niger State, using tipper Lorries and tipped close to the targeted location.

3.1.2 Cement

Cement used was purchased from building materials market in Minna. The cement used in this research was ordinary Portland cement (OPC) which conformed to the specification and standard before usage and it was Dangote brand of cement.

3.1.3 Calcium carbide residue

Calcium Carbide Residue (CCR) used in the study was obtained from gas welding shop located at KeterenGwari mechanic village, Minna, Niger State. It was dried in an open air and ground into fine particles using pestle and mortar and pass through 75 micro sieve.

3.1.4 Water

The water for mixing and curing was obtained from tap, clean and free from impurities.

3.2 Method

The disturbed sampling method was used in this work and the samples were collected from tipped deficient lateritic soils.

3.3 Laboratory Tests

Preliminary laboratory analysis on natural laterite was carried out to ascertain its suitability for engineering purposes, particularly the index properties, such as natural moisture content, particles size analysis, compaction test, Atterberg limit test, specific gravity, California bearing ratio (CBR) in accordance with BS 1377 (1990) and BS 1924 (1990).

A substantial amount of lateritic silty soil was collected from a borrow pit at Lapai-Gwari using Tipper Lorries. These soils were deposited close to the target road as shown in plate I. Large amount of CCR was then collected from local welders, dried, pulverized and sieved through British Standard sieve 0.075mm before use.



Plate 1: Lateritic soil dump



Plate II: Calcium carbide residue

Index properties tests were carried out on the untreated and treated lateritic soil at Civil Engineering Laboratory, Federal University of Technology, Minna, Nigeria.

Compaction tests were carried out on the natural soil and lateritic soil mixed with varied composition of 4% cement and 4% cement plus 2% calcium carbide residue (CCR).

These materials were manually mixed to allow for uniformity in the samples to be tested and samples were taken from each of the stockpiled materials and carried to Civil Engineering laboratory for tests in order to obtain 0% additive, 4% cement and 4% cement plus 2% CCR by weight of the soil. This was carried out to determine the mixture that will give the highest MDD and the subsequent OMC.

3.3.1 Natural moisture content test

The moisture content of a soil is defined as the ratio of mass of the water in the voids to the mass of the solids present in the sample. The test was carried out in accordance with BS1377 (1990) and BS 1924 (1990), which covers the determination of the moisture contents of the natural and treated sample.

Some empty, cleaned and dried moisture cans were identified and weighed before usage. A small amount of soil sample was placed in the can and weighed; the can containing wet soil was placed in an oven for 24 hours. It was then removed from the oven and allowed to cool. The can containing the dry sample was then weighed and recorded for moisture content determination. Therefore, moisture content of the soil was determined from the following equation;

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

Where

W = Natural Moisture content (%)

M₁ = Mass of empty container

M₂ = Mass of container and wet soil

M₃ = Mass of container and dry soil

3.3.2 Particlesize distribution analysis

This test was carried out in accordance to BS 1377 (1990) which covers the determination of particle size distribution. The sample of soil was air dried for 24 hours before carried out the test. 300g of soil was measured in the petri dish and soaked for 24 hours. Sieve size 75 μ m was used to wash the soil sample until water passed through was clean. The wet soil sample retained on the sieve was carefully placed on the pan and carried to the oven for 24 hours to dry. After drying the sample, the electronic weighing balance was then set and initialized. Each of the empty sieves in the set of sieves was weighed and corresponding values were recorded respectively. Afterwards the sieves sizes were arranged in descending order (5mm to 0.075mm) to form the set of the sieves. The oven dried sample was removed from the oven and allowed to cool for about 10 minutes. The oven dried sample was poured into the set of sieves through the largest size to the smallest size and placed on the mechanical shaker for mechanically shaken to ensure adequate passage. Then, each of the sieve plus the retained sample were gently removed and weighed on the electronic balance and the corresponding readings were recorded respectively. The soil retained in each sieve was then calculated and expressed as a percentage of the total washed sample.

3.3.3 Liquid limit test

Cone Penetrometer equipment was used for Liquid Limit test and conducted in accordance to BS 1377 (1990). 200g of air dried sample was weighed on the weighing balance and passed through 0.425mm sieve size. The Weighed sample was placed on the glass plate and mixed with water thoroughly with the aid of palette knives or spatulas in order to obtain adequate paste consistency. The soil paste was compacted into the penetrometer metal cup with a spatula to avoid void spaces and to level the soil to the top edge of the penetrometer metal cup. The cone penetrometer was adjusted to 0.0mm reading. The penetrometer metal cup filled with compacted sample was placed beneath the cone. The

knob was adjusted until the tip of the cone was slightly in contact with the top of the levelled soil in the metal cup. Hence, the knob was pressed for a few seconds, which helps in releasing the cone to allow penetration into the soil and the reading was recorded. Some portion of the soil was taken for moisture content determination.

3.3.4 Plastic limit test

The plastic limit test was carried out in accordance to BS 1377 (1990). A small sample from the sample used for liquid limit test was rolled into a thread like structure on the glass plate with the aid of hand. The rolling process continues until a thread-like soil of 3mm diameter was achieved which crumble at this stage. When crumbling stage was obtained, the crumb soil were placed into the empty cans and weighed immediately for moisture content determination.

3.3.5 Plasticity index

The numerical difference between the liquid limit and plastic limit of a soil sample is known as plasticity index. This method was used to calculate the plasticity index of the varying samples, through the equation given below and it was expressed as percentage.

$$PI=LL-PL \quad (3.2)$$

Where

LL= Liquid Limit (%)

PL= Plastic Limit (%)

3.3.6 Compaction test

This test was conducted in accordance to BS 1377 (1990) and the British Standard Light (Standard Proctor Test ASTM D4318) compaction energy effort was used in this research. The empty compaction mould attached with the base plate was weighed on the electronic

weighing balance and all empty moisture content cans were weighed and recorded respectively. Also the volume of the mould was calculated and recorded. A 300g of air dried soil sample was weighed on the electronic weighing balance. The weighed sample was poured onto the large tray and was mallet was use to break the soil lumps into the required fine particles sizes. A required quantity of water was added to the soil sample in the metal tray and mixed thoroughly with the aid of both hands. After mixing, the wet soil was poured into the mould and compacted in five layers and each layer was rammed 25 blows using 4.5 kg rammer dropped above the soil. Then after the fifth layer, collar of the mould was removed and the compacted soil was levelled off gently to the brim of the mould by using spatula and palette knife. The compacted sample with the mould and base plate was weighed on electronic balance and recorded. After weighing, the base plate was removed and some quantity of the soil sample was taken from the top and bottom of the mould, poured into two weighed empty moisture content cans for moisture content determination.

The remaining soil in the mould was removed, broken into the smaller sizes with the help of mallet and mixed with the remainder of the original sample in the metal tray. Suitable quantity of water was gradually added to the soil and mixed thoroughly using both hands. This process above was repeated five more times until the strength of the soil fails and the weight of the compacted soil was evaluated. The moisture content for each of the process was obtained and the bulk density as well as the dry density was calculated.

Also the values obtained during the compaction process were used to plot graph of dry density against moisture content in order to obtain the compaction curve. From the compaction curve, the maximum dry density (MDD) and the optimum moisture content (OMC) were obtained. The above procedure was repeated for 4% cement and 4% cement plus 2% CCR respectively. The bulk density and dry density was determined from the equation below;

$$P = \left(\frac{M_2 - M_1}{V} \right) \quad (3.3)$$

M_1 = Mass of empty mould and base plate (g)

M_2 = Mass of mould and base plate + wet soil (g)

V = Volume of mould (cm³)

Also dry density,

$$P_d = \frac{P}{1 + W/100} \quad (3.4)$$

Where W = Moisture content of the soil and

P = bulk density

3.3.7 California bearing ratio (CBR) test

The CBR test was conducted in accordance to BS 1377 (1990). A 6000g of air dried soil was weighed on the electronic weighing balance. The sample was poured on the metal tray and the soil lumps was crushed to fine particles with the aid of mallet. Suitable quantity of water was reasonably added to the soil in the metal tray. Then, the soil sample was mixed thoroughly and divided into five portions. The first portion was poured into the CBR mould and was rammed 60 blows uniformly by using the 4.5 kg rammer then the second portion was added and 60 blows of rammer was applied, this procedure continues up to the fifth layer. The collar of the CBR mould was removed and the soil sample was levelled to the brim of the CBR mould. The CBR mould together with the compacted soil sample was placed on the base of the CBR test machine and a single load placed on top of the mould. The plunger was made to rest on the soil sample and the bolts at the bottom and top was well tightened. The lower and the upper dial gauge that indicates the depth of penetration and force of action of the plunger was set in contact with the single load placed on the mould and the dial gauge was set to zero mark respectively, then the CBR test machine was switched on and operated electrically. The penetration of the plunger was observed between the range

of 0.25mm intervals and the corresponding readings were taken from the upper dial gauge for the top section of the soil sample. After the final reading was taken, the motor of the CBR test machine was automatically stopped before reversing the direction of the motor in order to turn the base of the CBR machine upside down and considered the bottom surface of the soil in the CBR mould.

The values of force on plunger obtained for the top and bottom readings at 2.5 mm and 5.0 mm penetration was calculated and converted to CBR values and the highest CBR value was taken as the CBR value for that soil. The above procedures were repeated for treated soil sample with varied composition of 4% cement and 4% cement plus 2% calcium carbide residue (CCR) respectively.

The treated and untreated specimens in the cylindrical mould were sealed in an air tight nylon bag to prevent loss of moisture content and buried inside sharp sand under normal room temperature for 24 hours and 7 days curing after which they were test at the end of their curing period.



Plate III: California Bearing Ratio Machine

3.3.8 Specific gravity test

The specific gravity of a soil is defined as the ratio of its mass in air of a given volume of dry soil solids to the mass of an equal volume of distilled water. The test was carried out in accordance to BS 1377 (1990) that covers the determination of specific gravity of the soil samples.

Some empty cleaned and dried bottles were identified, weighed and recorded. A small quantity of oven dried soil sample was poured in each bottle, weighed, and the Corresponding values were recorded. Then, some clean water was added to soil sample in the bottle, filled to the maximum capacity, weighed and recorded. The mouth of the bottle was tightly closed and shaken for about 10 minutes in order to allow the soil sample to dissolve. The dissolved soil sample with water was later removed and the bottle left empty and cleaned. Finally, the empty bottle was filled with cleaned water, flush with the top hole, weighed and recorded for specific gravity determination.

The above procedure was repeated for the second time and the average was taken as the specific gravity. Therefore, the specific gravity was calculated from the following equation

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \times 100 \quad (3.5)$$

Where

G_s = Specific gravity

M_1 = Mass of empty dry bottle

M_2 = Mass of bottle + dry soil sample

M_3 = Mass of bottle + soil + water

M_4 = Mass of bottle + water

3.4 Field Tests

untreated section. The materials were batched by weight which section A was filled with 12000kg of lateritic soil, 480kg of cement and 280kg of CCR, section B contained 12000kg of lateritic soil and 480kg of cement, while section C was filled with only 12000kg of lateritic soil. These sections were demarcated with wooden plank and the soil was manually dry mixed with cement and CCR after which predetermined amount of water was added as shown in plate IV.

Then the constituted materials were placed in appropriate positions in 150mm lift and compacted with a light roller. The roller was allowed enough number of passes in order to achieve the targeted degree of compaction (95%) as shown in plate V.



Plate IV: Dry mix of soil, cement and CCR.



Plate V: Compaction with light roller

3.4.2 Construction techniques

The in-situ field test was carried out on an access road between central workshop and civil Engineering laboratory at Federal University of Technology, Minna. The existing road subgrade was cleared from organic matters and the in-situ soil also known as ground soil was levelled and compacted prior to the filling of soils for embankment construction. The working areas were demarcated and pegged with iron pegs and then the sections were

separated with wooden planks to avoid materials moving from one section to the other during, placing, mixing and compaction. The homogeneously mixed materials for each section were placed and spread manually in two lifts of 150mm each above the ground level. Then compaction process commences immediately for the three sections with a light vibratory roller, which rolled and compacted the material in lifts for several times and finally achieved 300mm compacted thickness which are shown in plate VI and VII. Finally, in-situ field density tests using sand replacement method and in-situ CBR tests using dynamic cone penetration (DCP) tests were carried out on the finished surfaces, the test continues until three trials gave close results and it was conducted after 1, 7, 14, 28, 60 and 90 days of curing and the readings were recorded for the three sections.



Plate VI: Compaction with light roller **Plate VII:** Compacted finished surface.

3.4.3 Field density test

The degree of compaction in the field was evaluated by sand cone test specified by BS 1377(1990). In-situ density test in this study was conducted on the compacted finished surfaces by sand replacement method and samples for in-situ density test were taken from a point in each of the three sections of the compacted lifts in order to determine their maximum in-situ density as well as to compare the effectiveness of the results. The principle of this in-

situ density test was excavation of cylindrical shaped of field materials and replaced with dry sand and in-situ density tests were carried out at the three sections after 1, 7, 14, 28, 60 and 90 days of curing which are shown in plate VIII.



Plate VIII: In-situ density test on compacted finished surface.

3.4.4 Field CBR test

The field CBR was determined using dynamic cone penetration (DCP) method was conducted on the compacted finished surfaces of section A, section B and section C simultaneously in order to determine the bearing capacity and compaction characteristic of the pavement. The compacted finished layer was approximately 300mm thickness; therefore, calculation of dynamic cone penetration index (DCPI) was based only on the data obtained from 0 to 300mm in depth and dynamic cone penetration (DCP) tests were conducted at the three sections after 1, 7, 14, 28, 60 and 90 days of curing.

The relationship between DCP and CBR was determined using the following equation derived by Garber and Hoel (2000).

$$\text{Log (CBR)} = 2.46 - 1.2 \text{ Log (PI)} \quad (3.6)$$

Where PI = Penetration index (mm/blow).

Three dynamic cone penetration tests are carried out on each testing day while two in-situ density tests are carried out on each testing day. This is to obtain averages for each of the tests which are shown in plate IX.

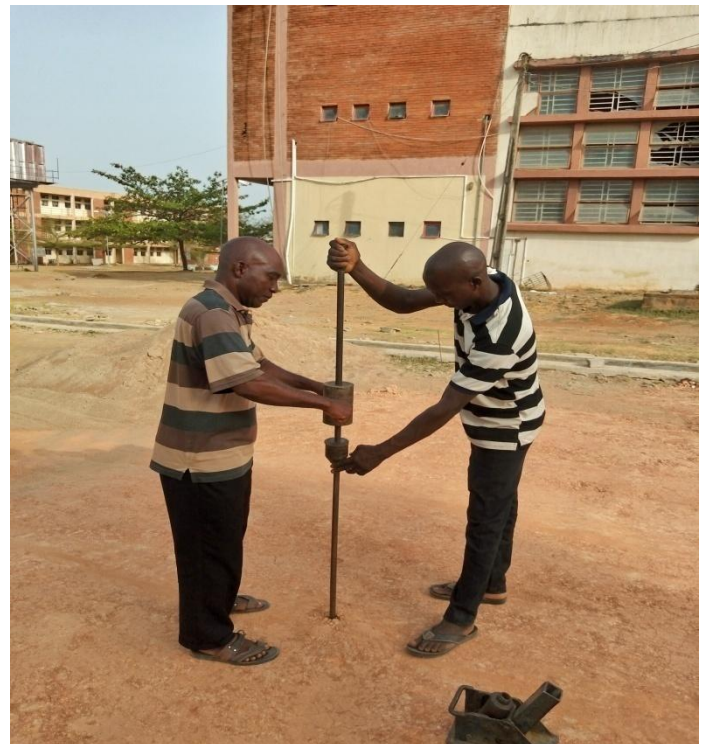
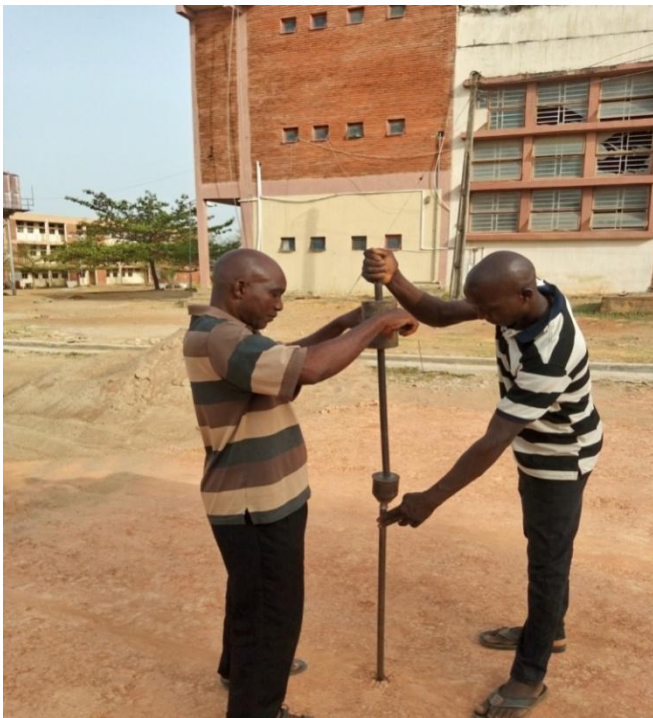


Plate IX: DCP test on compacted finished surface

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Preamble

Laboratory tests were carried out in order to determine the index and strength properties of the natural lateritic soil and lateritic soil treated admixed with cement and CCR at varied composition of 0% additive, 4% cement and 4% cement plus 2% CCR respectively. These tests were performed in the Civil Engineering Laboratory building, Federal University of Technology, Minna. The results obtained are shown in Tables and these includes Atterberg Limit test, Compaction test, California Bearing Ratio (CBR) test, Sieve Analysis and Specific Gravity tests as attached. The results of the index properties test and gradation plot of the natural soil were represented on Table 4.1 and appendix F.

Table 4.1: Summary Result of Index Properties of Natural Soil

Property	Quantity
Moisture Content (%)	17.94
Maximum Dry Density (g/cm ³)	1.82
Percentage passing BS 0.075mm (%)	74.77
Liquid limit (%)	49.49
Plastic limit (%)	31.22
Plasticity Index (%)	18.27
California Bearing Ratio (CBR) %Unsoaked	34.77
Specific Gravity	2.43
AASHTO Classification	A-7-6
Colour	Reddish-brown
Unified soil classification system (USCS)	CL

4.2 Index Properties of Lateritic Soil Samples

Table 4.1 above shows the results of index properties of the lateritic soil used in this research work. The lateritic soil was found to be an A-7-6 and clay of low plasticity soil by the AASHTO and USCS soil classification system respectively and specific gravity was 2.43.

The geotechnical properties of the natural lateritic soil clearly showed that it is only suitable for use as fill material for road construction and cannot be used as sub-base and base course materials specified by Nigerian General Specification (1997) and therefore, required stabilization to improve its engineering properties.

4.3 Laboratory and Field Density

4.3.1 Laboratory density

Table 4.2 shows the results of maximum dry density and optimum moisture content which were determined from laboratory compaction test performed in accordance with BS 1377 (1990) by modified compaction energy level. Compaction test was carried out on the natural soil sample and the stabilized lateritic soil which was treated by admixed with 0% additive, 4% cement and 4% cement plus 2% CCR and gradation plot of MDD and OMC are presented in appendix C, D and E respectively.

Table 4.2: Summary of MDD & OMC

Additives	MDD	OMC
0% additives	1.822	17.94
4% cement	1.815	19.47
4% cement + 2% CCR	1.756	18.36

The results showed that the MDD decreased with increase in additives from 1.822g/cm³ at 0% additive to 1.815g/cm³ at 4% cement additive and finally, to 1.756g/cm³ at 4% cement plus 2% calcium carbide residue (CCR). This is probable due to agglomeration of clay particles due to the reaction of cement and the lateritic clay.

Addition of cement and calcium carbide residue (CCR) increased the values of optimum moisture content (OMC) from 17.94% at 0% additives to 19.47% at 4% cement additive and finally, to 18.36% at 4% cement plus 2% CCR. The increase in OMC observed was due to more water required by cement during the reaction. This trend observed was similar to the findings of (Mannasseh and Edeh, 2014).

4.3.2 Field density result

The field densities were determined using the method of sand replacement as shown on plate Viii. The results of the test are shown on table 4.3.

Table 4.3: Summary of field density test

Section of the road	Day	FD	FMC
A 4% Cement+ 2% CCR	1	1.596	8.56
	7	1.712	3.51
	14	1.781	5.77
	28	1.786	10.42
	60	1.645	13.30
	90	1.615	15.54
B 4% Cement	1	1.545	7.56
	7	1.719	4.57
	14	1.784	4.72
	28	1.788	9.72
	60	1.650	11.50
	90	1.550	14.45
C 0% Additive	1	1.516	4.96
	7	1.728	3.6
	14	1.737	2.82
	28	1.766	8.66
	60	1.590	11.89
	90	1.427	12.88

The test was conducted on the finished road surface after 95% compaction of the three sections was achieved.

In section A, lateritic soil stabilized with a mixture of 4% cement plus 2% CCR, the field density was increased from 1.596 g/cm³ at 1 day curing to 1.615g/cm³ at 90 days curing, section B, lateritic soil stabilized with only 4% cement content, the field density was increased from 1.545 g/cm³ at 1 day curing to 1.739 g/cm³ at 7 days curing and decreased to 1.550g/cm³ at 90 days curing, while the field density of section C, untreated lateritic soil, was decreased from 1.516 g/cm³ at 1 day curing to 1.427g/cm³ at 90 days curing.

Generally, the field moisture content in section A, stabilized lateritic soil with a mixture of 4% cement plus 2% CCR was reduced from 8.56% at 1 day curing to 5.77% at 14 days curing and increased to 15.54% at 90 days curing, section B, stabilized lateritic soil with only 4% cement content, the field moisture content was increased from 7.56 at 1 day curing to 14.45% at 90 days curing and section C, untreated lateritic soil, the field moisture content was reduced from 4.96 at 1 day curing to 2.82% at 14 days curing and increased to 12.88% at 90 days curing. The trend showed increase in dry densities with increase in curing days and general reduction in moisture content with increase in curing days. This trend observed was similar to those obtained by (Mannasseh and Edeh, 2014).

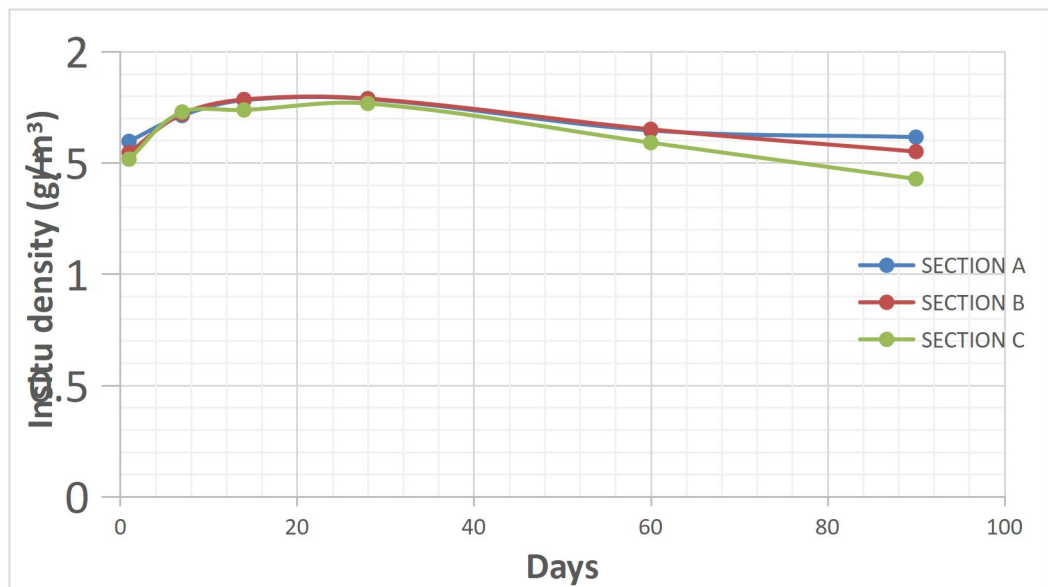


Figure 4.1: Variation of the in-situ densities with curing times for the three sections

4.4 Laboratory and Field CBR Result

4.4.1 Laboratory CBR result

The laboratory CBR test was carried out in accordance with BS 1377 (1990). It was conducted on both the natural soil and the stabilized lateritic soil which was stabilized by 0% additive, 4% cement and 4% cement plus 2% calcium carbide residue (CCR) and cured for 7 days by unsoaked method. This test was performed in the civil engineering laboratory Federal University of Technology, Minna and the results are shown on Table 4.4. The values were averages obtained from the top and bottom of each CBR specimen.

Table 4.4: Summary of Laboratory CBR test

Composition	CBR (%) 2.5m	CBR (%) 5.0mm	Average CBR (%) at 2.5 + 5.0mm
0% Additive	36.92	32.62	34.77
4% Cement	140.61		140.61
4% Cement + 2% CCR	238.79		238.79

Results showed that the unsoaked (CBR) values of natural soil increased from 34.77% at 0% additive to 140.61% when the lateritic soil was stabilized with only 4% cement content by weight of the soil and showed a maximum CBR value of 238.79% when the lateritic soil was stabilized with admixed of 4% cement and 2% calcium carbide residue (CCR) by weight of the soil. This represents 687% increase in CBR value on addition of 4% cement plus 2% CCR.

According to Nigeria General Specification (1997) the CBR value of natural lateritic soil can be used as fill material, a treated lateritic soil with 4% cement and admixed of 4% cement plus 2% CCR can be used as sub-base and base courses material respectively.

4.4.2 Field CBR result

The in-situ CBR test was conducted on the compacted finished surfaces of the road by the dynamic cone penetration method and the CBR value was determined from the following equation developed by (Garber and Hoel 2000).

$$\text{Log (CBR)} = 2.46 - 1.2 \text{ Log (PI)} \quad (4.1)$$

Table 4.5: Average number of blows for 300mm thick compacted with various curing time

Section of the road	Curing Time (d)						
	1 day	7 days	14 days	28 days	60 days	90 days	
Section A	3	4	5	7	7	7	
	4	4	5	7	8	10	
	20	23	30	30	32	36	
Converted CBR	17.4	20.5	27.9	30.6	33.1	38.2	
Section B	3	4	5	6	7	7	
	3	4	5	5	7	9	
	18	21	29	30	30	35	
Converted CBR	15.3	18.8	27.0	28.6	30.6	36.5	
Section C	2	4	3	3	4	3	
	3	3	4	4	7	5	
	16	18	20	23	25	24	
Converted CBR	12.9	15.7	17.4	19.9	24.2	21.4	

Table 4.6: summary of field CBR (%) values at varied number of curing time

Section of the road	Curing Time (d)						
	1 day	7 days	14 days	28 days	60 days	90 days	
Section A	17.4	20.5	27.9	30.6	33.1	38.2	
Section B	15.3	18.8	27.0	28.6	30.6	36.5	
Section C	12.9	15.7	17.4	19.9	24.2	21.4	

Table 4.5 and 4.6 displays the results of penetration index and CBR values at varied number of curing days for all testing points in section A, section B and section C respectively.

The average values of blow were used to obtain the penetration index used in the above equation to determine the CBR values. The CBR value in section B increases marginally from 15.3% at 1 day of curing to 30.6% at 60 days of curing, after which the values became larger to 36.5% at 90 days of curing.

This trend is similar to section C, whose CBR was 12.9% at 1 day of curing and increases marginally to 24.2% at 60 days of curing. The values reduced to 21.4% at 90 days of curing.

Section A, a stabilized soil admixed with 4% cement plus 2% CCR indicates marginal increase in CBR value from 17.4% at 1 day of curing to 33.1% at 60 days of curing and relatively further increased to 38.2% at 90 days of curing. The higher in-situ CBR compared to category A and B is due to the reaction of cement with the soil to bond the soil particles together.

The occurrence of calcium carbide residue (CCR) in this medium tends to increase the CBR values above other categories. However, the laboratory CBR values were found to be far higher than the in-situ CBR values.

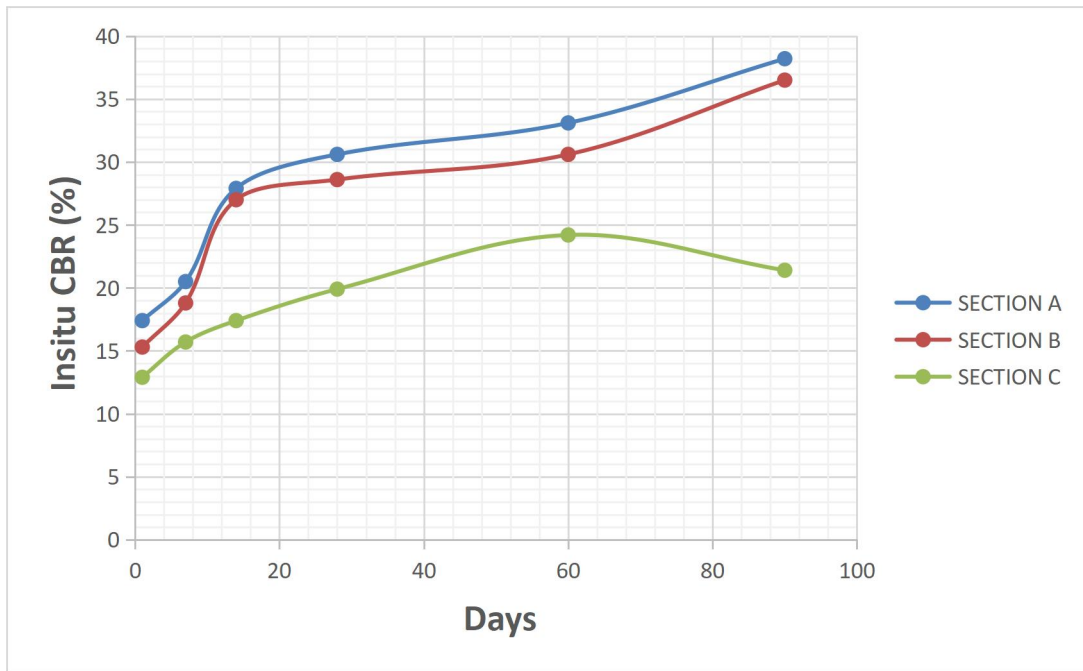


Figure 4.2: Variation of in-situ CBR with curing times for the three sections

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Based on the results of this research, the following conclusions can be drawn;

A fine lateritic soil used in this study was classified as clay of low plasticity (CL) according to unified soil classification system and A-7-6 according to AASHTO soil classification test.

The laboratory compaction densities and field densities tend to agree to large extent.

The laboratory CBR values and In-situ CBR values differ to a large extent. However, the in-situ CBR values of the untreated lateritic soil increased marginally when treated with 4% cement plus 2% CCR.

The field performance of the stabilized soil admixed with 4% cement plus 2% calcium carbide residue indicate higher in-situ CBR value of 38.2% at 90 days of curing.

5.2 Recommendations

Based on this research work:

1. It is recommended that CCR can be use to stabilized poor soil in order to improve its geotechnical properties for potential use as highway construction material.
2. Also due to scarcity of good materials and high cost of road construction materials, CCR can easily be obtain at no cost and use as stabilizing agent to poor soil.
3. Lapai-Gwari lateritic soil treated with varied composition of 4% cement plus 2% calcium carbide residue whose in-situ CBR value was 38.2% at 90 days of curing is recommended for use as subbase material in pavement work.

5.3 Contribution to Knowledge

The study investigated the possibility of treating A-7-6 lateritic soil with cement and calcium carbide residue (CCR), and established that addition of 4 and 2%andcalcium carbide residue (CCR) respectively increased the California bearing ratio (CBR) of the soil from 34.77% to 238.79%, representing 687% increase in strength of the soil.

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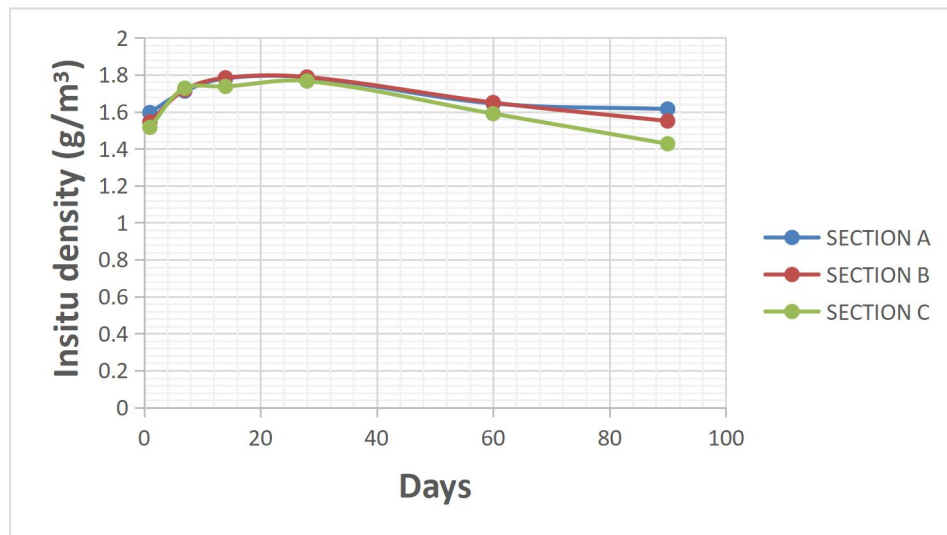
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APPENDICES

APPENDIX A: In- situ density rest result

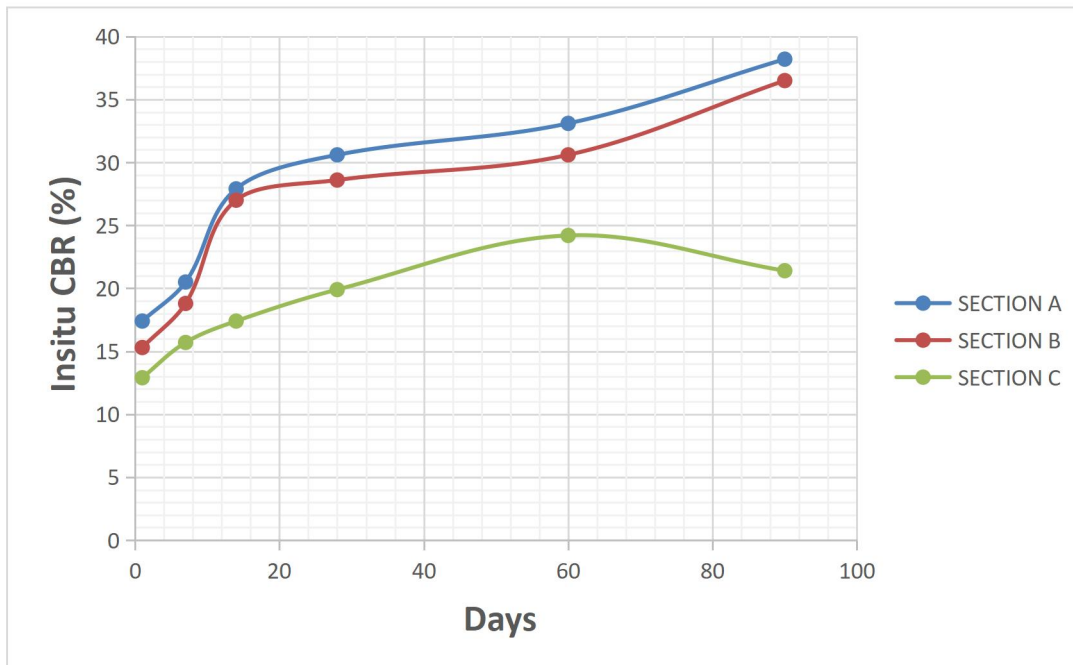
In- Situ Density Test (Sand Replacement Method)			
Days	Section A	Section B	Section C
1	1.596	1.545	1.516
7	1.712	1.719	1.728
14	1.781	1.784	1.737
28	1.786	1.788	1.766
60	1.645	1.65	1.59
90	1.615	1.55	1.43



APPENDIX B: In- situ CBR test result

In- Situ CBR Test (Dynamic Cone Penetrometer Method)

Days	Section A	Section B	Section C
1	17.4	15.3	12.9
7	20.5	18.8	15.7
14	27.9	27	17.4
28	30.6	28.6	19.9
60	33.1	30.6	24.20
90	38.2	36.5	21.40



APPENDIX C: Compaction test result

COMPACTION TEST

Project :										
Test Location : BP FUT GIDAN KWANO										
Sample no. : natural soil			Volume of Mold : 944			cm³		Date : 2/1/2020		
Sample Description : pinkish clay soil										

Weight of Mold (g)	3685		3685		3685		3685		3685	
Weight of Mold+Wet Soil (g)	5426		5470		5645		5726		5695	
Weight of Wet Soil (g)	1,741.00		1,785.00		1,960.00		2,041.00		2,010.00	
Wet Density (g/cm ³)	1.84		1.89		2.08		2.16		2.13	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	38.2	38.7	38.2	39.1	38.1	38.0	38.4	38.0	35.1	38.7
Weight of Can + Wet Soil (g)	88.9	103.9	109.3	124.3	94.7	91.2	88.3	87.0	109.4	124.2
Weight of Can + Dry Soil (g)	85.3	99.2	102.0	115.0	87.1	84.0	80.3	79.2	95.0	107.7
Weight of Water (g)	3.60	4.70	7.30	9.30	7.60	7.20	8.00	7.80	14.40	16.50
Weight of Dry Soil (g)	47.10	60.50	63.80	75.90	49.00	46.00	41.90	41.20	59.90	69.00
Moisture Content (g)	7.64	7.77	11.44	12.25	15.51	15.65	19.09	18.93	24.04	23.91
Ave. Moisture Content (g)	7.71		11.85		15.58		19.01		23.98	
Dry Density (g/cm ³)	1.7123		1.6906		1.7964		1.8167		1.7175	



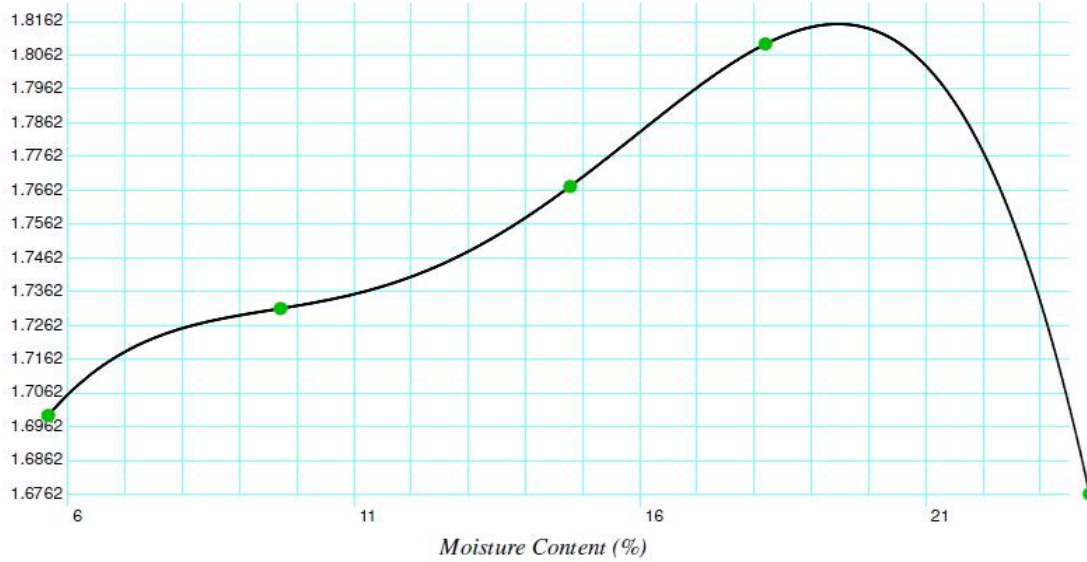
APPENDIX D: Compaction test result

COMPACTION TEST

Project : Master Project	
Test Location : GK FUT Minna	
Sample no. : 4% cement Addition	Volume of Mold : 944 cm ³ Date :
Sample Description : Pinkish clay soil	

Weight of Mold (g)	3685		3685		3685		3685		3685	
Weight of Mold+Wet Soil (g)	5380		5478		5600		5704		5645	
Weight of Wet Soil (g)	1,695.00		1,793.00		1,915.00		2,019.00		1,960.00	
Wet Density (g/cm ³)	1.80		1.90		2.03		2.14		2.08	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	38.6	38.1	39.1	38.2	38.8	37.9	38.1	37.8	39.1	38.3
Weight of Can + Wet Soil (g)	80.7	71.0	75.1	94.9	81.6	95.0	89.3	95.1	98.8	99.0
Weight of Can + Dry Soil (g)	78.5	69.2	71.9	89.9	76.2	87.5	81.4	86.3	87.4	87.2
Weight of Water (g)	2.20	1.80	3.20	5.00	5.40	7.50	7.90	8.80	11.40	11.80
Weight of Dry Soil (g)	39.90	31.10	32.80	51.70	37.40	49.60	43.30	48.50	48.30	48.90
Moisture Content (g)	5.51	5.79	9.76	9.67	14.44	15.12	18.24	18.14	23.60	24.13
Ave. Moisture Content (g)	5.65		9.71		14.78		18.19		23.87	
Dry Density (g/cm ³)	1.6995		1.7312		1.7674		1.8095		1.6762	

Dry Density (g/cm³)



MDD : 1.8155 g/cm³

OMC : 19.47 %

Test executed by: _____

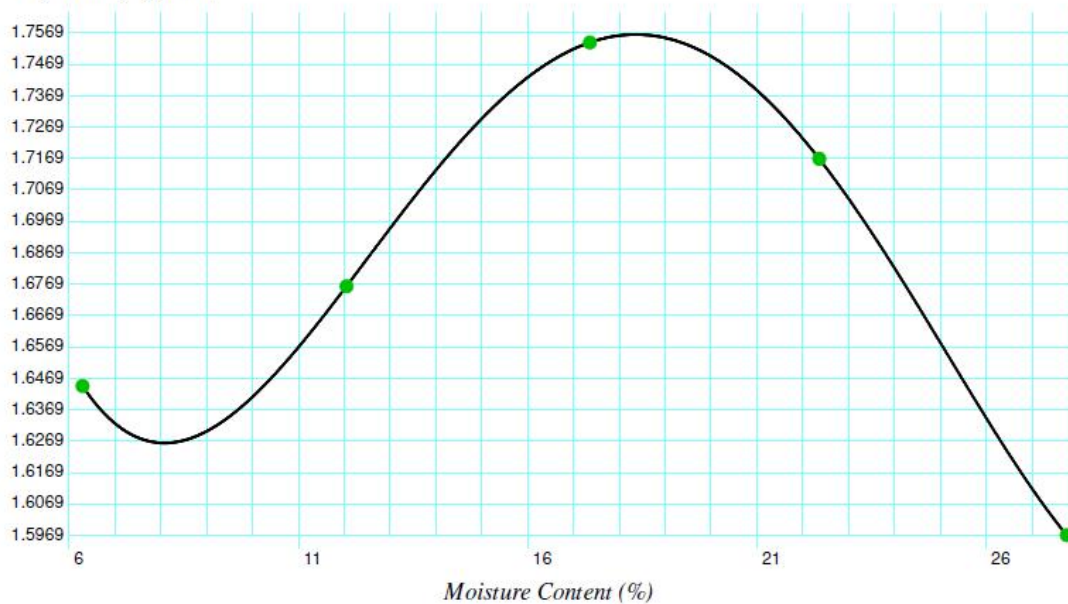
APPENDIX E: Compaction test result

COMPACTION TEST

Project : Master project	
Test Location : GK FUT Minna	
Sample no. : 4%cm/2%carbide Add	Volume of Mold : 944 cm ³ Date :
Sample Description : Pinkish Brown clay soil	

Weight of Mold (g)	3685		3685		3685		3685		3685	
Weight of Mold+Wet Soil (g)	5335		5458		5628		5668		5611	
Weight of Wet Soil (g)	1,650.00		1,773.00		1,943.00		1,983.00		1,926.00	
Wet Density (g/cm ³)	1.75		1.88		2.06		2.10		2.04	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	22.5	24.4	24.2	23.0	24.5	24.3	23.3	24.8	22.8	24.1
Weight of Can + Wet Soil (g)	81.5	62.8	63.9	73.6	68.2	68.5	74.6	75.3	72.7	75.9
Weight of Can + Dry Soil (g)	77.9	60.6	59.6	68.2	61.7	62.0	65.4	65.9	61.9	64.6
Weight of Water (g)	3.60	2.20	4.30	5.40	6.50	6.50	9.20	9.40	10.80	11.30
Weight of Dry Soil (g)	55.40	36.20	35.40	45.20	37.20	37.70	42.10	41.10	39.10	40.50
Moisture Content (g)	6.50	6.08	12.15	11.95	17.47	17.24	21.85	22.87	27.62	27.90
Ave. Moisture Content (g)	6.29		12.05		17.36		22.36		27.76	
Dry Density (g/cm ³)	1.6445		1.6762		1.7538		1.7167		1.5969	

Dry Density (g/cm³)



MDD : 1.7564 g/cm³

OMC : 18.36 %

Test executed by: _____

APPENDIX F: Sieve size analysis result

5.000	0	0.00	100.00
3.350	1	0.33	99.67
2.000	2.6	0.87	98.80
1.180	6.5	2.17	96.63
0.850	4.9	1.63	95.00
0.600	8.3	2.77	92.23
0.425	11.2	3.73	88.50
0.300	8.1	2.70	85.80
0.150	28.1	9.37	76.43
0.075	5	1.67	74.77

