

**STABILISATION OF TROPICAL BLACK CLAY USING CALCIUM CARBIDE
RESIDUE AND COCONUT SHELL ASH AS ADMIXTURE**

BY

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AUGUST, 2021

ABSTRACT

A Tropical Black Clay (TBC), obtained by method of disturbed sampling, at a depth of between 0.5m to 1.5m, at Gwako village, Abuja, Nigeria, was treated with 0, 2, 4, 6, 8, and 10% each of Calcium Carbide Residue (CCR) and Coconut Shell Ash (CSA). Effect of the additives was investigated with respect to compaction characteristics (Maximum Dry Density-MDD and Optimum Moisture Content-OMC) and Unconfined Compressive Strength (UCS) cured at 1, 7, 14, 28, 60 and 90 days. Results of index properties of the soil indicated that it is classified under CH and A-7-5, according to Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) respectively. There was a general decrease in MDD of the treated soil with increase in dosage of the additives from 1.784 g/cm³ at 0% CSA and 0% CCR to a minimum value of 1.666 g/cm³ at 8% CSA and 4% CCR, while the OMC increased with increase in dosage of the additives from 18.0% at 0% CSA and 0% CCR to a maximum value of 23.7% at 4% CSA and 8% CCR. UCS of the treated soil showed general increase with increase in dosage of the both additives and curing period from 19.196 kN/m² at 0% CSA and 0% CCR to a maximum value of 1194.972 kN/m² at 0% CSA and 10% CCR after 90 days curing. It was therefore, concluded that, although the treated soil did not meet the 1710 kN/m² UCS value require for base materials, the requirement of 687-1373 kN/m² for sub-base materials was achieved within the composition considered and therefore could be used as sub-base materials for lightly trafficked roads.

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ABBREVIATIONS, GLOSSARIES AND SYMBOLS

AASHTO	American Association for State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BA	Bagasse Ash
BS	British Standard
BLA	Banana Leaf Ash
BSL	British Standard Light
CBR	California Bearing Ratio
CCR	Calcium Carbide Residue
CH	Clay of High Plasticity
CL	clayey soil
CSA	Coconut Shell Ash
CSP	Coconut Shell Powder
FA	Fly ash
FAO	Food and Agriculture Organization
FUTA	Federal University of Technology, Akure
FUTMINNA	Federal University of Technology, Minna
G _s	specific gravity
LL	liquid Limit
M	mass
m	metre
MDD	Maximum Dry Density
M _s	Mass of compacted soil
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement

TBC	Tropical Black Clay
TRRL	Transport and Road Research Laboratory
PI	plasticity Index
PL	plastic Limit
RHA	Rice Husk Ash
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
V_s	Volume of mould
ρ_b	Bulk density
ρ_d	Dry density
w	Natural moisture content

CHAPTER ONE

1.0

INTRODUCTION

1.1 Background of the Study

Soil is the fundamental and most economical of construction materials. It bears the loads of structures and pavements transmitted through foundations and sub-bases, respectively. Researchers have shown that the suitability of a soil as a construction material is a function of its geotechnical properties (Bowels, 1984; Venkatramaiah, 2012). The use of the existing soil at a construction site for engineering purpose may be hindered by poor engineering properties including poor bearing capacity, higher compressibility, and alternate shrink and swell behavior. Soils whose use is limited as a result of their engineering properties are referred to as deficient soils (Alhassan and Mustapha, 2015).

Deficient soils are regarded as soils which do not meet some or all the criteria required for their satisfactory performance as geotechnical structures. These could either be for base courses for road, embankment for dam, subsoil base, clay liners for containment of leachates and backfill for retaining walls (Alhassan and Mustapha, 2015). In the tropical region, these soils could be lateritic soils, tropical black clay, collapsible soils or any other tropical soils.

Expansive soils are fine grained soil or decomposed rocks that show huge volume change when exposed to fluctuations of moisture content (Hesham, 2013). Typical example of an expansive soil is the “Tropical black clay” found in North-East Nigeria and other parts of the world such as India and USA. They are so named because of their colour and suitability for growing cotton. Tropical black clay have colours ranging from light grey to dark grey and black (Oriola and Moses, 2011).

Tropical black clay is confined to semi-arid regions of tropical and temperate climatic zones and is abundant where the annual evaporation exceeds the precipitation (Chen, 1988; Warren and Kirby; 2004). They are vastly available in North Eastern Nigeria lying within the Chad basin and the upper Benue trough (Adesunloye, 1987).

Tropical black clay occurs in continuous stretches as superficial deposits and is typical of flat terrains with poor drainage. The absence of quartz in the clay mineralogy enhances formation of fine-grained soil material, which is impermeable and waterlogged (Balogun, 1991). The mineralogy of this soil is dominated by the presence of this montmorillonite, which is highly responsible for the attendant shrink-swell behavior of the soil depending on the amount of available moisture in the soil and is the main cause of the many problems associated with these soils. Shrinkage during the dry season often lead to surface cracks that could open up to 50mm or more and several millimeters deep (Osinubi *et al.*, 2011).

Tropical black clay presents various challenges to engineers all over the world due to their characteristic of severe loss of strength and swelling with respect to changes in moisture regime. As a result, structures, roads and highways constructed on them are subject to severe deformations and frequent repairs, leading to high cost of maintenance. Various efforts have been made to stabilize tropical black clay with cement, lime, admixtures and waste products to make them meet requirements for construction works (Osinubi *et al.*, 2009; Ola, 1983; Balogun 1991).

Soil stabilisation is the alteration of one or more properties, by mechanical or chemical means, to create an improved material, possessing the desired engineering properties (Onyeloweand Okafor, 2012). Earlier, soil improvement has been in the qualitative sense only, but more recently, it has also become associated with quantitative values like compressive strength, shearing strength, load bearing quality, adsorption, softening and

reduction in strength and durability, which are related to soil performance (Amu *et al.*, 2005). Soils may be stabilised to increase strength and durability or to prevent erosion and dust generation (Onyelowe and Okafor, 2012). Soil stabilisation deals with the physical and chemical methods to make the stabilised soil serve its purpose as pavement component material (Koteswara, 2011). Osinubi and Katte (1991) referred to soil stabilisation as the alteration or control of any soil property. It covers not only increase or decrease of any soil property, but also the variation of any property with changes in environmental condition, namely moisture or pressure. Stabilisation process could be mechanical or chemical, biological, electrokinetic and thermal.

Mechanical method includes replacement with non-expansive fill, compaction, soil reinforcement, addition of aggregates and mechanical remediation. Moreover, the most widespread mechanical stabilisation methods are rewetting, removal and replacement (Nelson and Miller, 1992). Chemical stabilisation enhances geotechnical properties of soils, by addition of some different materials such as fly ash, quick lime, Portland cement, bitumen, calcium chloride, chemical or bio-remediation, marble dust and scrap tire rubber (Arash *et al.*, 2012). Some of these additives are industrial and agricultural wastes.

The ability to blend naturally occurring soil with some industrial and agricultural wastes to give better engineering properties in both strength and water proofing is very essential. Cost effective and locally available stabilisers such as Calcium Carbide Residue (CCR) and Coconut Shell Ash (CSA) are used for the stabilisation in this project.

1.2 Statement of the Research Problem

Tropical black clay belongs to the smectite group, which includes montmorillonite that are highly expansive and are the most troublesome clay minerals (Nelson and Miller, 1992). They are susceptible to seasonal volumetric changes, exhibit severe cracking when dry,

swell and yield low bearing strengths when wet, ultimately resulting to heavy depressions and settlement (Seehra, 2008). These problems cause extensive damage to light structures and pavement resting on them (Jones and Holtz, 1993).

The effort made by researchers (Mohammedbhai and Baguant, 1990; Osinubi, 2000; Cokca, 2001; Medjo and Riskowski, 2004; Akinmade, 2008; Ochebo, 2008) to obtain cheaper additives which can be used to substitute the conventional soil improving additives (cement, lime and bitumen) led to the consideration of agricultural and industrial waste resources such as rice husk ash (RHA), bagasse ash (BA), locust bean waste ash (LBWA), Fly ash and Iron ore tailing. Thus, the use of cheap admixtures to replace or supplement cement or lime stabilised soils especially wastes from agricultural and industrial products will ultimately reduce the cost of construction works where expansive soils are found (Chesner *et al.*, 2002; Phanikumar and Sharma 2004; Cokca 2001).

1.3 Aim and Objectives of the Research

The aim of this research is to investigate the effect of Calcium Carbide Residue (CCR) and Coconut Shell Ash (CSA) on Tropical Black Clay (TBC).

To achieve this aim, the objectives are to:-

- i. determine the chemical composition of Calcium Carbide Residue and Coconut Shell Ash.
- ii. determine the geotechnical properties of tropical black clay.
- iii. determine the effect of Calcium Carbide Residue and Coconut Shell Ash on the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) of tropical black clay.
- iv. determine the effect of Calcium Carbide Residue and Coconut Shell Ash on the Unconfined Compressive Strength (UCS) of tropical black clay.

1.4 Scope of the Research

The scope of this research focused on the laboratory tests using tropical black clay. The following tests were conducted:

- i. Chemical composition of Calcium Carbide Residue (CCR) and Coconut Shell Ash (CSA).
- ii. Index properties test on the natural soil.
- iii. Compaction test on both the natural and stabilised soil with 0% CSA + (0, 2, 4, 6, 8 and 10)% CCR, 2% CSA + (0, 2, 4, 6, 8 and 10)% CCR, 4% CSA + (0, 2, 4, 6, 8 and 10)% CCR, 6% CSA + (0, 2, 4, 6, 8 and 10)% CCR, 8% CSA + (0, 2, 4, 6, 8 and 10)% CCR and 10% CSA + (0, 2, 4, 6, 8 and 10)% CCR.
- iv. Unconfined Compressive Strength (UCS) test on both the natural and stabilized soil cured at 1, 7, 14, 28, 60 and 90 days.

1.5 Justification of the Research

The rising cost of conventional soil stabilising additives such as cement and lime has motivated the search for cheaper and locally available materials for improving properties of deficient soils to meet geotechnical engineering requirements. Thus, the use of industrial and agricultural waste, such as calcium carbide residue and coconut shell ash which will considerably reduce the cost of construction and as well reducing the environmental hazards the disposal of these waste causes.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Weathering and Soil Formation

Weathering is a pedogenic process by which soils are formed as a result of disintegration or decomposition of rocks (Gidigasuan and Gawu, 2013). This process can be chemical, mechanical, physical and biological. Chemical weathering causes changes in rocks at and near the earth's surface by interaction between the rocks and chemically active components of the earth's atmosphere, principally water, carbon dioxide and oxygen (Gidigasuan and Gawu, 2013).

Within a given area on the earth's surface and at a given geologic time, the particular effects of weathering is assessed in terms of at least five variables: (1) climate, (2) biological activity, (3) topography, as it effects underground movement of water, or contributes to erosional removal of weathered material (4) parent material (rocks) and (5) time (Gidigasuan and Gawu, 2013).

A great variety of physical, chemical, and biological processes act to break down rock masses. Physical processes reduce particle size, increase surface area, and increase bulk volume. Chemical and biological processes may cause complete changes in physical, chemical and mineralogical composition of the end products. Robinson (1949) recognized two main stages in chemical weathering. The first stage involves destruction of mineral phases, and the second stage formation of secondary products. These two stages involve various processes which result in two main types of materials of morphogenic interest example: (i) weathering residues and secondary materials that occur insitu, and (ii) materials which are transported before deposition. The first process of physical or mechanical weathering is also designated as disintegration, while the second stage is decomposition (Gidigasuan, 2012). Disintegration results in decreased size of rocks and

minerals without appreciably affecting their composition. In the case of decomposition, however, definite chemical and physico-chemical changes take place, soluble materials are released, and new minerals are synthesized or are left as resistant end products (Gidigasuan and Gawu, 2013).

2.1.1 Physical weathering of rocks

Rocks exposed to environmental conditions are subjected to differential expansion and contraction resulting in disintegration. The rock pieces retain the chemical and petrological composition of the original parent rock. Physical weathering processes cause insitu breakdown without chemical change. Five processes are important (Mitchell and Soga, 2005):

- i. Unloading
- ii. Thermal expansion and contraction
- iii. Crystal growth, including frost action
- iv. Colloid plucking
- v. Organic activity

Physical weathering processes are generally the forerunners of chemical weathering and their main contributions are to loosen rock masses, reduce particle sizes, and increase the available surface area for chemical attack (Alonso and Pineda, 2006). Physical process of rock weathering usually results to formation of granular materials (soils) whose properties are greatly influenced by gradation and density.

2.1.2 Chemical weathering of rocks

Chemical weathering involves decomposition of rocks resulting in production of secondary minerals, thus the chemistry and petrological characteristics of the parent rock is greatly altered (Bobrowsky and Marker, 2018).

The process of chemical weathering is started by ingress and percolation into the soil, of slightly acidic rainwater, contaminated with organic impurities. The rainwater, at tropical climatic conditions tends to dissolve soluble minerals such as silica, alkalis, alkali-earth metals etc. from the soil and bedrock, leaving behind concentrates of iron and aluminum oxides (Bobrowsky and Marker, 2018). Continual drainage and leaching of these soluble minerals, with time deplete them. The processes of chemical weathering are listed below (Blyth and De Freitas, 1976; Mitchell and Soga, 2005).

- i. Hydrolysis
- ii. Hydration
- iii. Carbonation and related activity processes
- iv. Oxidation
- v. Solution

Chemical process of weathering usually results to formation of clay minerals (soils), whose properties are greatly impacted on the soils.

2.1.3 Products of weathering

The rock forming minerals such as quartz, feldspar, mica, amphibole, olivine, pyroxene, are the main constituents of crystalline rocks which are formed at great depth at phases which are only stable at very high temperatures and pressures (particularly true for mafic and ultramafic rocks than felsic rocks) (Wilson, 2004). When these rocks are exposed to the surface (environment) the equilibrium of the minerals is disturbed and the more the new conditions differ from the original, the more the minerals are inclined to adjusting themselves to the new environment and are most likely to decompose under suitable conditions (Weinert, 1980). The products of weathering, several of which will generally coexist at one time, include (Mitchell and Soga, 2005):

- i. Unaltered minerals that are either highly resistant or freshly exposed
- ii. Newly formed, more stable minerals having the same structure as the original mineral
- iii. Newly formed minerals having a form similar to the original, but a changed internal structure
- iv. Products of disrupted minerals, either at or transported from the site. Such minerals might include:
 - a. Clay minerals
 - b. Colloidal gels of Al_2O_3 and SiO_2
 - c. Zeolites
 - d. Cations and anions in solution
 - e. Mineral precipitates

The resulting materials are either granular or fine in nature. Some of these materials are stable under civil engineering structures, while some can be quite problematic.

2.2 Problematic Soils

Problematic soils are structurally unstable soils which pose engineering problems in the design and construction of civil engineering structures (Dixit, 2016). They are considered problematic because, firstly these soils cannot be classified by grain size and plasticity parameters alone due to non-reproducibility and inconsistency of test results using standard sample preparation methods and testing procedures. Secondly, they are characterized by unusual in-service behavior, including such phenomena as heaving, swell, collapse, dispersion, erosion, excessive creep, subsidence and high compressibility which could be attributed to factors such as composition, nature of pore fluids, fabric and mineralogy (Gidigas, 2009; Bell and Culshaw, 2001).

There are many problematic soils. Some of which are:

- i. Basic rocks subjected to rapid physico-chemical weathering in tropical environment
- ii. Pedogenic materials (lateritic rocks, calcretes, ferricretes and silcretes)
- iii. Expansive/shrinkable soils (weathered shales, fissured stiff clays, mudstone and tropical black clay)
- iv. Some lateritic and saprolitic soils
- v. Dispersive and erodible soils
- vi. Volcanic soils

Some of the peculiar characteristics associated with these problematic soils from literature are:

- i. Unstable mineralogical composition (commonly, montmorillonite, allophone and halloysite)
- ii. High insitu moisture contents
- iii. Unstable soil structure and fabric
- iv. High water absorption capacity
- v. Low insitu density
- vi. Variable degree of desiccation with depth
- vii. Variable density-moisture relations with pre-test sample preparation methods
- viii. Liquefaction prone
- ix. Erodible/dispersive
- x. Collapse phenomena
- xi. Shrinkage and expansivity
- xii. Difficulty to stabilize in terms of controlling volume, strength, water absorption capacity and durability
- xiii. Post construction problems

2.2.1 Expansive soils

Expansive soil is a clayey soil in nature which expands when it comes in contact with water and shrinks when the water evaporates (Jaharoun, 2015). This soil is generally found in arid and semi-arid regions of the world. A lot of damages occur to structures founded on this type of soil. The damages normally appear as cracks on buildings, canal beds and linings, pavements, lifting of water supply pipeline and sewerage lines (Sabat, 2012). Expansive soils owe their expansive character mainly to the constituent clay mineral. Soils containing a considerable amount of montmorillonite minerals will exhibit high swelling and shrinkage characteristics (Murthy, 2001). It has an octahedral sheet sandwiched between two silica sheets. When this mineral is exposed to moisture, water is absorbed between interlayering lattice structures and exert an upward pressure. This upward pressure, known as swelling pressure, causes most of the damages associated with these soils. Most of the structural damage by expansive soils results from the differential rather than the total movement of the foundation soil as a result of swelling (Holtz and Hart, 1978). An example of expansive soils is the black cotton soil, which is dark–grey to black in colour (Jaharoun, 2015). The name originated from India where locations of these soils are favourable for growing cotton. It is also the Nigerian type of expansive soils (Osinubi, 1999). Other tropical clays found in Nigeria have also been known to exhibit expansive properties. These types of clays are tropical black clays.

2.2.2 Origin of tropical black clay

Tropical black clay otherwise known as Canada’s swelling clays, India’s black cotton soil and Kenya black cotton soils can be found all over the world (Chen, 1988). Warren and Kirby (2004) refer to tropical black clay as swelling soils, heaving soils and volume change soils.

Saleh and Utpal (1993) described expansive clay formations as being favoured by the geology, climatic condition and the environment of extreme disintegration, strong hydration and restrained leaching. Expansive soils swell and shrink considerably with changes in moisture content. Due to the problems associated with these soils, road and building construction on this type of soil, demand special knowledge which is in the hands of relatively small group of experts (Ibrahim, 1977).

Black cotton soil is tropical black clay having a colour characteristic of dark grey to black, with high clay content and usually expansive in nature and contains over 50% of montmorillonite, which is the principal clay mineral (Rajakumar and Meenambal, 2015). These soils dominated semi-arid regions of tropical-temperate zones and mostly found anywhere in the world, especially where the annual evaporation exceeds the precipitation (Rajakumar and Meenambal,2015). Tropical black clays possess shrink-swell characteristics and cause damages (settlement) to structural pavement resting on them. However, this happens as a result of variation in moisture content that is not evenly distributed (Osinubi *et al.*, 2010; Akinmade, 2008). Generally, deposits of black cotton soil in the field show a general pattern of cracks during the dry season of the year. Cracks measuring about 70mm wide and over 1m deep have been observed and may extend up to 3m or more in case of high deposits (Osinubi, 2000;Nelson and Miller, 1992).

Tropical black clays are usually very hard when dry, but losses their strength when wet. They exhibit very low bearing capacity, low permeability and high volume change due to the presence of montmorillonite and illite clay minerals in them (Alhassan and Mustapha, 2017). However, in their natural state, these properties make them unsuitable for construction of embankment, highway, building or any other load bearing structure (Das, 1998). This type of soil group are found in the North Eastern part of Nigeria and exhibits large volume changes with respect to the variation of seasonal moisture content (Eberemu

et al., 2012). Some of the major roads that link Nigeria with the neighbouring republics of Cameroon and Chad pass through these problematic soils. These roads carry Nigeria's exports of fuel and other produce which approached some 250,000 tonnes per annum from the main Nigerian ports through Kano and Maiduguri to these neighbouring countries (Ibrahim, 1977). The problem of road construction on these soils is further complicated by the fact that about 75% of the commercial vehicles travelling out of Nigeria are overloaded. Also, most of the vehicles transporting fuel to the neighbouring countries often have axle loads in excess of 14 tonnes and some axle loads are heavier than 20 tonnes (Ola, 1983).

2.2.3 Mineralogical and chemical composition of tropical black clay

Ola (1983) showed that the Nigerian black cotton soils contains about 70% montmorillonite and 30% kaolinite and the predominance of this montmorillonite is due to the weathering of the basic constituents. The swell and shrink characteristics of the soil are largely due to montmorillonite mineral, while kaolinite is likely responsible for high strength value because of its inability to swell with absorbed water.

The montmorillonite clay structure consists of layer sheet formed and stacked one above the other and the bonding between successive layers is by van der Waals forces and by cation that may be present to balance charge deficiencies in the structure. These bonds are however, weak and easily separated by cleavage or adsorption of water and other liquids. There is an extensive isomorphous substitution for aluminium and silicon with its lattice which gives the clay a net negative charge resulting in the water absorbing tendencies and an attraction for hydroxyl ions and water molecules to the clay surface (Nelson and Miller, 1992). On the other hand, kaolinite is a relatively inactive clay mineral. Its structure is made of repeating layers of elemental silica-gibbsite sheets in 1:1 lattice (Das, 2007). To improve the performance of those soils, they are often stabilised.

2.3 Soil Stabilisation

Soil stabilization is the permanent physical and chemical alteration of soils to enhance their physical properties (Mekonnen *et al.*, 2020). In its broadest sense, it includes compaction, preconsolidation, drainage, and many such processes. However, the term stabilization is generally restricted to the process which alters the soil material itself for improvement of its properties (Mekonnen *et al.*, 2020). It is the collective term for any physical, chemical, or biological method, or combination of such methods employed to improve certain properties of natural soil to make it serve for intended engineering purposes (Clough and Duncan, 1991). The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Ingles and Metcalf, 1972; Sherwood, 1993; EuroSoilStab, 2002).

Soils may be stabilised to increase their strength and durability or to prevent erosion and dust generation (Onyelowe and Okafor, 2012). Soil stabilisation deals with the physical, physico-chemical and chemical methods to make the stabilised soil to serve its purpose as pavement component material (Koteswara, 2011). According to Johnson *et al.* (1988) stabilisation infers improvement in both strength and durability. Osinubi and Katte (1991) referred to soil stabilisation as the alteration or control of any soil property. It covers not only the increase or decrease of any soil property, but also the variation of any property with changes in environmental condition, namely moisture or pressure. Primarily, the objectives of stabilisation are to improve soil strength, to decrease permeability and water absorption and to improve bearing capacity and durability under cyclical conditions such as varying moisture content (Akinmade, 2008). Soil stabilisation has been used in the building of roads, aircraft runways, earth dams and embankment in erosion control (Kawamura and Diamond, 1975). Soil stabilisation could be mechanically, chemically or biologically done.

2.3.1 Mechanical stabilisation

A soil structure is said to be mechanically stable when it can resist lateral displacement under load (Akinmade, 2008). A stable soil can be obtained through controlled grading of the coarse aggregate, fine aggregate, silt and clay correctly proportioned and fully compacted (Akinmade, 2008). The process of controlled grading is referred to as mechanical stabilisation (Gilliot, 1987). Therefore, mechanical stabilisation involves soil densification through the application of mechanical energy. Mechanical energy brings about the reduction of air void in the soil, with little or no reduction in water content. It also lowers the permeability of the soil and an increase in dry density (Markwick, 1944).

2.3.2 Chemical stabilisation

Soil stabilisation using chemical admixtures is the oldest and most widespread method of ground improvement (Jaharoun, 2015). Chemical stabilisation is mixing of soil with one of or a combination of admixtures of powder, slurry, or liquid form for the general objectives of improving or controlling its volume stability, strength and stress-strain behavior, permeability, and durability (Winterkorn and Pamukçu, 1990). Traditional stabilisers such as Portland cement, lime and bitumen are used to stabilize soil. However, there are varieties of non-traditional additives available from the commercial sector, such as polymer emulsions, acids, lignin derivatives, enzymes, tree resin emulsions, and silicates. Inorganic salts such as sodium chloride and calcium chloride have long been used in stabilisation (Jaharoun, 2015).

Their main function is to reduce plasticity and facilitate densification (Gilliot, 1987; Yamanouchi, 1975; Habercom, 1978; Balogun, 1991; Osinubi and Medubi, 1997).

2.3.2.1 Cement stabilisation

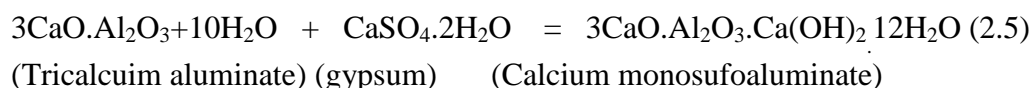
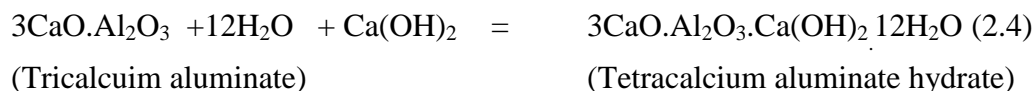
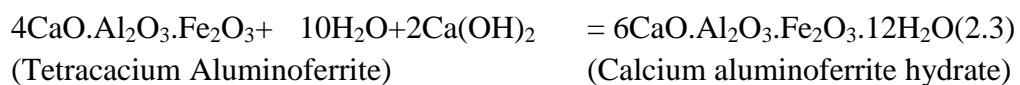
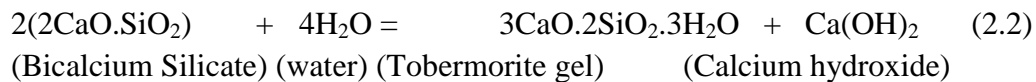
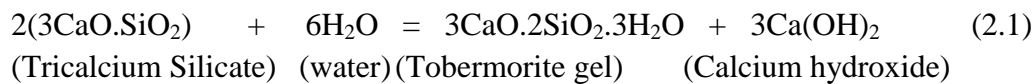
Stabilisation of soils with cement usually involves mixing of predetermined quantities of the additive with pulverized soil and the mixture is termed soil-cement (Akinmade, 2008).

Soil-cement mixtures have been used for road sub-base or base course. Stabilisation of soil with Ordinary Portland Cement (OPC) produces hardened materials which are capable of bearing loads for engineering purposes. TRRL (1977) recommended a minimum of 15 % for soil fraction passing 0.425 mm sieve and plasticity index greater or equal to 10. Generally, gravels require about 10 % by dry weight of cement, sand requires about 7–10 %, silt about 12 to 15 % and clays, 12 to 20 % by dry weight of cement (Gilliot, 1987). When soils are sandy, mixing them with cement can be carried out at or slightly about optimum moisture content but with clayey soils mixing is most readily achieved with moisture slightly below optimum (Akinmade, 2008).

Portland cement is a finely divided material that results from inter grinding clinker and gypsum (Prusinski and Bhattacharja, 1999). A Portland cement particle is a heterogeneous substance, containing minute tricalcium silicate (C_3S), dicalcium silicate (C_2S), tricalcium aluminate (C_3A), and a solid solution described as tetracalciumalmino – ferrite (C_4A) (Lea, 1956). These four main constituents are major strength producing compounds. According to Bergado *et al.* (1996) there are two major chemical reactions which are induced by the addition of cement to clay and govern the soil cement stabilisation process: the primary hydration reaction of the cement and water, and the secondary pozzolanic reactions between the limes released by the cement and the clay minerals.

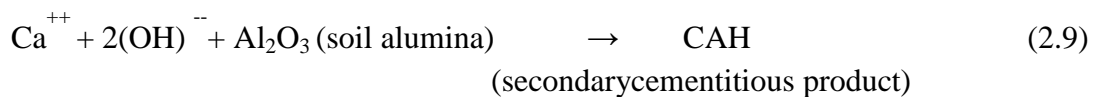
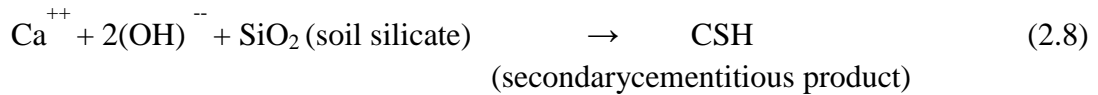
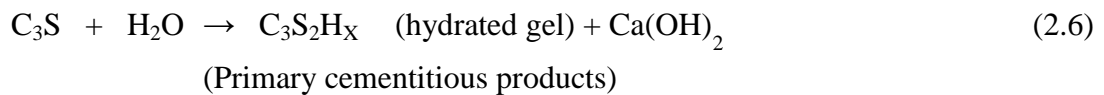
The primary hydration products are hydrated calcium silicates (C_2SH_x , $C_3S_2H_x$), hydrated calcium aluminates (C_3AH_x , C_4AH_x) and hydrated lime $Ca(OH)_2$. The first two of the hydration products listed above are main cementitious products formed and the hydrated lime is deposited as a separate crystalline solid phase. Describing these crystals as being more reactive than ordinary lime, the calcium in the crystals and in the pore solution is available to the clay for continued stabilisation of the soil particles (Herzog and Mitchell, 1965). These cement particles bind the adjacent cement grains together during hardening

and form a hardened skeleton matrix, which encloses unaltered soil particles. The soil silica and alumina, which are inherently acidic, will then be dissolved by the strong bases of cement compounds from the clay minerals and amorphous materials on the surface of the clay particle, which is similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina will then gradually react with calcium ions liberated from the hydrolysis of cement to form a new insoluble compound (secondary cementing product), which hardens when cured and thereby stabilises the soil. When Portland cement is mixed with water, hydration is initiated and calcium concentration in the soils builds up quite rapidly. The solution becomes saturated with calcium hydroxide within 12 min of water – cement contact (Gartner *et al.*, 1985). The transformation of portland cement with addition of water into cement paste can be chemically illustrated as follows:



Calcium ions (Ca^{+2}) are released in solution, they are available for stabilising the clayey soil. A drop in calcium content after 12 hours is attributed to the setting of cement, where a substantial amount of calcium and water are consumed to form C-S-H and $\text{Ca}(\text{OH})_2$. Further explained that the reactions which take place in soil – cement stabilisation can be represented in the equation below. The reactions given are for tricalcium silicate (C_3S)

only, because they are the most important constituents of portland cement (Bergado *et al.*, 1996):



Ola (1983) reported that stabilisation of tropical black clay with cement alone is not suitable. The non-suitability of cement as good stabiliser is based on the economics/cost of application, because the presence of montmorillonite in the soils retards hydration and hence high percentages of cement would be required for adequate stabilisation. Furthermore, the effect of cement on the Atterberg limits is minimal and workability is not improved. Table 2.1 shows the Approximate Oxides Composition Limits of Ordinary Portland Cement.

Table 2.1: Approximate oxides composition limits of ordinary portland cement

Oxide	Percentage content
CaO	60 - 67.0
SiO ₂	17.0 - 25.0
Al ₂ O ₃	3.0 - 8.0
Fe ₂ O ₃	0.5 - 6.0
MgO	0.1 - 4.0
Alkalies (K ₂ O, Na ₂ O)	0.4 - 1.3
SO ₃	1.3 - 3.0
Insoluble residue	0.3 - 5.0
Loss on ignition	1.0 - 1.5

Source: Neville (1996)

2.3.2.2 *Lime stabilisation*

Lime stabilisation may refer to pozzolanic reaction in which pozzolana reacts with lime in presence of water to produce cementitious compounds (Sherwood, 1993, EuroSoilStab, 2002). The effect can be brought by either quicklime, CaO or hydrated lime, Ca(OH)₂.

Slurry lime also can be used in dry soils conditions where water may be required to achieve effective compaction (Hicks, 2002). There are two phases of stabilisation in a lime-soil system. The first, one is an immediate reaction of cation exchange; the second is flocculation agglomeration. The second one occurs to some extent with all fine-grained soils; due to textural changes caused by these reactions, the soils are improved. These improvements are reflected in improved workability, immediate strength improvement and reduce swell susceptibility (Ramadas *et al.*, 2011).

When lime is mixed with clayey material in the presence of water, several chemical reactions take place. They include cation exchange, flocculation-agglomeration, pozzolanic reaction, and carbonation (Mallela *et al.*, 2004). Cation exchange and flocculation-agglomeration are the primary reactions, which take place immediately after mixing. During these reactions, the monovalent cations that are generally associated with clay minerals are replaced by the divalent calcium ions. These reactions contribute to immediate changes in plasticity index, workability, and strength gain.

Eades and Grim (1960) found that there exists a chemical reaction between lime and pure clay minerals (kaolinite, illite and montmorillonite) with accompanying increase in bearing value. The quantity of lime needed to effectively treat a soil to develop increased strength varies with the type of clay mineral present (Ramlakhan *et al.*, 2013). Akawwi and Al-Kharabsheh (2002) reported that the swelling and shrinkage potential of soils are affected by mineralogical constituents and surrounding environment. The optimum amount of lime

for maximum strength gain, according to Eades and Grim (1960) is 4 - 6 % for kaolinite, about 8 % for illite and montmorillonite.

2.3.3 Biological stabilisation

It is achieved through afforestation or planting, and its main purpose is erosion control. Root traits such as architectural, morphological, physiological and biotic play an important role in both the physical and chemical development of soils enabling structural stability of the soil. This method is suitable for terrain exposed to water and wind influences, which are not meant for building. However, initially, planting has to be supported by other types of soil stabilisation from the moment seeds or seedlings are planted till the moment the plants become strong. Otherwise, along with the surface layer, seeds or young plants would be carried away by water flow or wind.

2.3.3.1 Microbial stabilisation

Microbial stabilisation is an emerging biological method for soil stabilisation, and microbes constitute the largest biomass on earth (Karol, 2003). Generally, the microorganisms which biodegrade complex compounds in soil and water are bacteria and fungi. Some engineering properties are improved by natural microbial biological processes.

The process of microbial stabilisation cements the sediments consisting of calcium, magnesium, iron, manganese, and aluminum through a network of chemical reactions, and the engineering properties of soil are altered by the byproducts of these reactions (DeJong *et al.*, 2006). The permeability, stiffness, compressibility, shear strength, and volumetric behavior are improved in such bio-mediated systems. The permeability is reduced by 10^{-3} , and the shear strength and stiffness are increased by 10^2 , according to results reported for bio-mediated improvement processes (DeJong *et al.*, 2010).

The improvement of soil properties through microbial stabilisation has been achieved in many areas such as liquefaction mitigation, settlement reduction, and slope stabilisation. In

addition, some environmentally-friendly factors such as energy storage and carbon sequestration are also provided by bio-mediated systems (DeJong *et al.*, 2010). However, some drawbacks exist for microbial stabilisation: the process is usually slower and more complex compared to chemical grouting; the time rate and various environmental factors such as temperature, pH, and concentrations of donors must be better-controlled during the deployment of microbial stabilisation approaches (Ivanov and Chu, 2008).

2.3.4 Agricultural waste stabilisation

Several studies have shown the importance of agricultural waste products as an alternative to conventional material for soil improvement (Olonode, 2010; Oluremi *et al.*, 2012; Ako and Yusuf, 2016, Ishola *et al.*, 2019). The residues generated from the growing and processing of raw agricultural materials such as fruits, vegetables, meat, poultry, dairy products, and crops are regarded as agricultural wastes. The increases in global population lead to a high rate of agricultural products which in turn increases the waste produce yearly (Agamuthu, 2009). Interestingly, wastes from agricultural product like locust bean, Rice husk, Palm oil, Banana Tree, Sugarcane, Bamboo Tree and palm kernel have widely been used to improve the geotechnical properties of tropical soils (lateritic and black cotton soils) and other soils (Bello *et al.*, 2015; Sadeeq *et al.*, 2015). Report shows that about 998 million tones of agricultural waste are generated every year (Agamuth, 2009). This prompted researchers to focus on how these materials can be use as a material for soil improvement.

2.3.4.1 Rice husk ash (RHA)

Rice husk is an agricultural waste obtained from milling of rice/rice mill factories (Yoobanpot and Jamsawang, 2014). Rice mill factories create a waste product called raw husk which is used as heating fuel in rice mill processing and produces the Rice Husk Ash (RHA) as a byproduct from the burnt raw husk. Annually about 10 tons of rice husk is

produced globally (Bello *et al.*, 2014). Oyetola and Abdullahi (2006) reported that in Nigeria, about 2.0 million tons of rice is produced annually. In Niger State, about 96,600 tons of rice grains are produced. This production shows that high quantity of rice is being consumed, which in turn increase the quantity of rice husk generated annually. Generally, rice husk ash has been reported to be a pozzolanic material with about 67-70 % (silica), 4.9 % (aluminum) and 0.95% (iron oxides) (Oyetola and Abdullahi, 2006).

Choobbasti *et al.* (2010) conducted a laboratory test to determine the influence of adding RHA on the amount and quickness of soil and lime reaction. They reported that the combination of soil with 4% lime gives the maximum amount of cohesion and angle of internal friction which is similar to the situation of using 3 and 5% RHA. However, addition of RHA to a combination of soil with 4% lime caused an increase in dilation parameter but shows a decreasing rate in the combination of soil with 6% lime. Addition of lime and RHA to the soil caused a decrease in the deformability and gives more brittle materials. Moreover, this action causes an increase in shear strength. The maximum shear strength of soil with lime combination occurs at 5% RHA for 28 days of soil samples.

Chao-Lung *et al.* (2011) carried out a laboratory test to investigate effect of marble dust on the strength and durability of an expansive soil, stabilised with rice husk ash (RHA). Their study showed that the optimum percentage of RHA was 10% based on unconfined compressive strength (UCS) tests. However, compaction, UCS, soaked CBR, swelling pressure, and durability tests were conducted on the samples after 7 days of curing. The result of UCS and soaked CBR of the RHA stabilised expansive soil showed an increase of up to 20%.

Yoobanpot and Jamsawang (2014) investigated compressive strength characteristics of soft soil, improved using RHA as a partial replacement for cement. The results show that 30% of RHA gave strength of 424, 722, 915 and 1,126 kPa at 3, 7, 14 and 28 days curing

respectively. They concluded that the increase in strength of the stabilised soil was relative to the formation of major reaction products such as calcium silicate hydrate.

2.3.4.2 Bagasse ash (BA)

Bagasse is the fibrous residue obtained from sugar cane after the extraction of sugar juice at sugar cane mills (Osinubi and Stephen, 2005). It is also regarded as the residue obtained from the incineration of bagasse in sugar producing factories. The ash has been reported to be a good pozzolanic material (Osinubi and Alhassan, 2008; Osinubi and Mustapha, 2008) with possession of silica and alumina contents of up to 77.29 and 10.95% respectively, for the ash obtained from bagasse burnt at 700°C.

Osinubi *et al.* (2009) investigated the geotechnical properties of improved deficient lateritic soil with up to 12% bagasse ash by dry weight of the soil. The soil samples were subjected to particle size analysis, compaction, Unconfined Compressive Strength (UCS), California Bearing Ratio (CBR) and durability tests. The energy level employed for the compactions test was British Standard Light (BSL). The results showed changes in moisture-density relationships, resulting in lower Maximum Dry Densities (MDD), higher Optimum Moisture Contents (OMC) and reduction in fine fractions with higher bagasse ash content. UCS and CBR values were given as 836 kN/m² and 16%, respectively at 2% bagasse ash treatment of the lateritic soil. They concluded that since these values were below 1700 kN/m² and 80% for UCS and CBR, respectively, recommended for adequate cement stabilisation. It implies that bagasse ash cannot be used as a ‘standalone’ stabiliser but should be employed in admixture stabilisation.

Salahudeen and Ochepe (2015) carried out laboratory tests on lateritic soil to evaluate the effect of Bagasse Ash (BA) on the California bearing ratio of lateritic soil. The tests were performed on natural and bagasse ash treated soil. The treated specimens were prepared by

mixing the soil with bagasse ash in steps of 0, 2, 4, 6 and 8% by dry weight of soil. Results of their investigation showed that the specific gravity of the soil samples decreased from 2.61 for the natural soil to 2.48 at 8% bagasse ash content. However, the liquid and plastic limits increased from 36.32 to 38.0 and 21.30%, to 21.54% respectively at 2% bagasse ash content. The Maximum Dry Density (MDD) of the soil increased from 1.48 Mg/m³ for the natural soil to a peak value of 1.49 Mg/m³ at 8% bagasse ash content with the corresponding increase in OMC from 18.5% for the natural soil to 19.0% at 4% bagasse ash content. Unsoaked California Bearing Ratio (CBR) of the lateritic soil treated with 4% BA content and above met the minimum CBR value of 30% specified by (BS, 1990) for material suitable for use as sub-base when determined at MDD and OMC. However, the highest CBR value of 62% recorded at 8% BA content failed to meet the 80% CBR value for base course recommended by Federal Ministry of Works, Nigerian (1997). General Specifications for Road and Bridge Works.

Wubshet and Tadesse (2014) investigated stabilisation of expansive soil using bagasse ash and lime. The natural soil, which was classified under A-7-5 on AASHTO soil classification, was stabilised using 3% lime, 15% bagasse ash and 15% bagasse ash in combination with 3% lime by dry weight of the soil. Effect of the additives on the soil was investigated with respect to plasticity, compaction and CBR. The results showed an increase in OMC and CBR, decrease in MDD and plasticity of the soil for all additives. They concluded that combining two locally available materials (bagasse ash and lime) can effectively improve the properties of expansive soil.

2.3.4.3 Banana leaf ash (BLA)

Banana is produced in 135 countries and among the highest producer in the year 2017 were Indian and China, which approximated to about 38% of the total world population (FAO, 2017). This showed the wide application and high consumption level of banana in the

countries. Banana leaf is large, flexible, and waterproof and often used as ecologically friendly disposable food containers or as "plates" in South Asia and several Southeast Asian countries. Nigeria is one of the largest banana and Plantain growing countries in Africa (Olutaiwo and Olushola, 2017). Nigeria produces 6.32 million tonnes of banana annually (NHRIFAO, 2011). It is also the largest plantain producing country in West Africa, making the crop one of the important staples in the country (Olutaiwo and Olushola, 2017). Banana leaves constitute large percentage by mass of the plant. This leaves litter the environment.

Nnochiri and Aderinlewo (2016) investigated the geotechnical properties of lateritic soil stabilised with Banana Leaf Ash (BLA). They carried out preliminary soil tests (natural moisture content, specific gravity, and Atterberg limit) on the natural soil sample, collected from the Federal University of Technology, Akure (FUTA), Nigeria. Tests such as compaction, California bearing ratio, and unconfined compressive strength tests were performed on the soil at natural and stabilised states with up to 10% banana leaf ash content by weight of the dry soil. The result showed that banana leaf ash enhanced the strength of the lateritic soil. The unsoaked CBR and unconfined compressive strength values improved at 4% BLA. They therefore concluded that banana leaf ash satisfactorily acted as cheap stabilising agents for subgrade purposes.

2.3.4.4 Coconut shell ash (CSA)

Cocosnucifera trees, otherwise known as coconut palm trees, grow abundantly along the coast line of countries within 15° of the equator (Amu *et al.*, 2011). They prosper in sandy, saline soil and in tropical climates (Nnochiri and Ogundipe, 2017). A healthy coconut tree will produce approximately 120 watermelon-sized husks per year, each with a coconut imbedded inside (Nnochiri and Ogundipe, 2017). There are three constituents of the CocosNucifera that can be used for fuel: the husk, the coconut shell, and the coconut oil

that is in the white coconut “meat” or copra, as it is usually called (Amu *et al.*, 2011). Thus, the coconut tree is a very abundant, renewable source of energy. When coconuts are harvested, the husks are removed, thereby leaving the shell and the copra. These husks are considered as waste materials and are usually dumped. When consumers buy the coconut, they buy it with the shell and when it is to be consumed it is broken and the shell removed. Large quantities of the shells can be obtained in places where coconut meat is used in food processing. The husk and the shell are both regarded as waste materials.

These materials are then burnt into ashes in furnace at a very high temperature to produce the coconut shell and husk ash (Amu *et al.*, 2011).

Many researchers have stated that the coconut shell can be reused as material replacement in construction industry where this may lead to reduction in construction material cost and save the dumping spaces (Gunasekaran *et al.*, 2013). In Nigeria, coconut shell can be a source of energy (biofuel) for boilers, and the rest of coconut shell is used as gravel for plantation roads maintenance (Onyelowe, 2016b). Furthermore, some other researchers showed that the coconut shell can be turned to powder form by subjecting it to incineration process and it can be used for active carbon production, mosquito repellent coil and as filler in plastic.

According to Madakson *et al.* (2012), coconut shell ash consists of major chemical composition such as silicon dioxide (SiO_2), aluminum oxide (Al_2O_3), magnesium oxide and iron (III) oxides. Silicon dioxide (SiO_2), aluminum oxide (Al_2O_3) and iron (III) oxides are known to be substances that can react with the products from cement hydration process and provide additional strength (Madakson *et al.*, 2012). Moreover, coconut shells possess higher resistance against crushing, abrasion and impact, compared to normal crushed aggregate (Kukarni and Gaikwad, 2013). Research indicates that the comparative strength characteristic between coconut shell and palm kernel shell concrete is in the favor of

concrete with coconut shell, as greater compressive strength was recorded compared with that containing palm kernel shell at the same mix proportion.

Kambli and Mathapati (2014) stated that coconut shell can be used as lightweight aggregate, which can help to reduce material cost in construction. Table 2.1 shows the chemical composition of coconut shell ash.

Rashmi *et al.* (2016) studied the effect of coconut shell ash on the properties of expansive soil. Three different percentages (5, 10, and 15%) of the coconut shell ash was used.

The result of the study showed that the plasticity index reduced with addition of various percentages of CSA, the maximum dry density increased to 10% CSA, while the optimum moisture content reduced. It was therefore concluded that coconut shell ash can effectively use to stabilise expansive soil. Tables 2.2 show the chemical composition of coconut shell ash.

Table 2.2: Chemical composition of coconut shell ash

Oxide	Percentage composition (%)		
	Utser and Taku (2012)	Isah(2014)	Onyelowe(2016a)
SiO ₂	37.97	44.05	30.20
Al ₂ O ₃	24.12	14.60	3.52
Fe ₂ O ₃	15.48	12.40	3.61
CaO	4.98	4.57	4.99
K ₂ O	0.83	0.52	0.98
Na ₂ O	0.95	0.45	1.97
MgO	1.89	14.20	21.36
SO ₃	0.71	-	-
MnO	0.81	0.22	0.96
ZnO	-	0.30	-
P ₂ O ₅	0.32	-	2.47
TiO ₂	-	-	1.03
LOI	11.94	8.69	19.67

Segun and Oluyemisi (2017) assessed the effect of coconut shell ash on lime-stabilised lateritic soil. Preliminary tests were carried out on the natural soil sample for identification and classification purposes, also tests such as CBR, UCS and compaction were carried out on the natural soil sample. Lime was mixed with the soil in the proportion of 2, 4, 6, 8 and 10%. The authors reported that 6% inclusion of lime was the optimum for stabilisation of the soil as the lowest plasticity index was obtained at that proportion. Addition of CSA to the lime stabilised soil showed increased in CBR values from 53.6% at 0% CSA to 66.4% at 6% CSA. They also observed that OMC increased from 21.44 to 26.10% at 0 and 10% CSA respectively. However, MDD reduced from 1342 to 1255 kg/m³ at 0 and 10% respectively. They concluded that CSA is a potential and effective complement for lime stabilisation of lateritic soil.

Athira *et al.* (2017) studied the effectiveness of coconut shell powder and lime in stabilising the expansive soil. Tests were performed for both the natural and stabilised soil with different dosages of the coconut shell powder (0, 3, 6, 9 and 12%) and lime (3, 6 and 9%) after curing periods of 0, 7 and 28 days. The results showed that MDD of the soil treated with coconut shell powder and lime increased. Also the optimum compressive strength was obtained at 9% CSP and 6% lime. They concluded that the compressive strength of the expansive soil increased by 228% after 28 days of curing.

2.3.5 Stabilisation using industrial waste

Industrial wastes are usually by products of various industrial processes and are formed during or as result of the process involved. They are usually dumped in the open or stored in ponds for disposal in the vicinity of the plants. There are several industrial wastes that are produced all over the world which can be used in soil stabilisation. Some of the industrial wastes that have been used in soil stabilisation include fly ash, iron ore tailing, phosphogypsum, cement kiln dust, steel slag, silica fume, lime kiln dust, waste water

sludge ash, calcium carbide residue, glass waste, limestone waste ash, cement bypass dust, copper slag, granulated blast furnace slag to name a few.

2.3.5.1 Fly ash

Fly ash is one of the most plentiful and industrial by-products. It is generated in vast quantities as a by-product of burning coal at electric power plants (Senol *et al.*, 2006). Fly ash generated by coal combustion based power plants typically fall within the ASTM fly ash classes C and F (Reyes and Pando, 2007). Yudhbir and Honjo (1991) stated that the pozzolanic fly ashes can be advantageously made used to improve the geotechnical properties of black cotton soils.

Zala (2013) studied the stabilisation of expansive clay using fly ash material. He conducted some tests to deal with the influence of fly ash to mitigate the expansive clay. In the research, highly expansive soil was mixed with different percentages of fly ash (15, 20, and 30%). According to the test results, the unconfined compression stress of the clay rose from 114 to 123 kN/m² after added 20% fly ash, the liquid limit and plastic limit were reduces from 74.4 to 72.5% and 38.4 to 32.93% respectively with increased in fly ash of up to 30%. The results also depict that MDD at 14% optimum moisture content, was increased from 1.68 to 1.71 g/cm³, after adding fly ash to 30%.

Gyanen *et al.* (2013) studied effect of fly ash on expansive soil, by treating the soil with fly ash at 5% increment to 25%. At 20% the liquid limit was observed to reduced by 55%. Plasticity index reduced by 86% at 20% of fly ash. At 15% of fly ash, 75% decrease was recorded in the swelling index of the soil and the specific gravity of the soil also improved.

2.3.5.2 Iron ore tailing

Iron ore tailing is an industrial by-product obtained from mining industry. Their effective disposal of is a great challenge to both civil and mining engineers. In recent years there has been increase in trend to utilize mine tailings for geotechnical applications, provided they are treated with some admixtures (Ramesh *et al.*, 2012).

The Itakpe iron ore deposit in Nigeria, which has a total estimated reserve of about 182.5 million metric tonnes consists mainly of quartzite with magnetite and hematite (Placid *et al.*, 2018). The deposits establish to supply iron ore concentrates to Ajaokuta and the Delta steel plants (Ajaka, 2009). Mine tailings can be effectively utilized for civil engineering constructions which will minimize the disposal problems and reduce the environmental hazards (Pebble Project, 2005).

Surendra *et al.* (2007) stated that addition of gold mine tailings and ordinary portland cement at different proportions to black cotton and red soil for manufacturing of cement-tailings bricks and soil-tailings bricks resulted in increase in the compressive strength and reveals that soil-tailings are more economical than cement-tailings bricks.

Ramesh *et al.* (2013) reported that the liquid limit, plastic limit and plasticity index of black cotton soil and mine tailings mixtures decrease stating that reduction in plasticity index is indication of improvement of properties of soil with the addition of lime and mine tailings. The unconfined compressive strength of red earth increased with increase in mine tailings. However, the strength increase peaked at 10% mine tailings content with and without curing, which was considered as optimum. The unconfined compressive strength of red earth, treated with optimum percentage of mine tailings increases further with addition of various percentage of lime with and without curing due to pozzolanic reaction. The increase in strength with addition of 3% lime was taken as the optimum percentage. From the study,

it was concluded that mine tailings can be effectively used for lime stabilisation of soils. The authors stated that the use of mine tailings for geotechnical application will minimize the disposal problem and reduce the environmental hazards.

2.3.5.3 *Calcium carbide residue (CCR)*

Calcium Carbide Residue is a by-product from acetylene gas production (Neeraj and Ahirwar, 2014). This gas is used around the world for welding, lighting, metal cutting and to ripen fruit (Neeraj and Ahirwar, 2014). Calcium carbide residue is obtained from a reaction between calcium carbide and water to form acetylene gas and calcium hydroxide in a slurry form, which mainly consists of calcium hydroxide Ca(OH)_2 , along with silicon dioxide SiO_2 , CaCO_3 and other metal oxides (Gurugubelli *et al.*, 2017). For high content of natural pozzolanic materials in clayey soil, calcium hydroxide $[\text{Ca(OH)}_2]$ is a rich material that can be used to produce high strength material. For environmental and economic impact such rich waste materials can be utilized collectively with natural pozzolanic material to form a cementitious material. Calcium carbide residue production is described in the following equation:



From the equation above, if 64g of calcium carbide (CaC_2) is used, it will provides 26g of acetylene gas (C_2H_2) and 74g of CCR in terms of Ca(OH)_2 (Kumrawat and Ahirwar, 2014). Tables 2.3 show the chemical composition of calcium carbide residue.

Table 2.3: Chemical composition of calcium carbide residue

Oxide	Percentage composition (%)		
	Manasseh and Ejelikwu (2013)	Joel and Edeh (2016)	Du <i>et al.</i> (2016)
SiO ₂	2.69	1.54	2.840
Al ₂ O ₃	1.78	0.50	2.160
Fe ₂ O ₃	0.17	0.03	0.150
CaO	61.41	67.08	68.990
K ₂ O	0.10	0.05	-
Na ₂ O	0.18	0.02	0.027
MgO	0.80	1.26	0.120
SO ₃	-	-	0.760
MnO	-	0.05	-
P ₂ O ₅	-	-	0.003
TiO ₂	-	0.32	0.031
SrO	-	-	0.031
LOI	32.51	26.85	24.850

Isah and Sharmila (2015) investigated the stabilisation of CL and CH soils using calcium carbide residue (CCR) and coconut shell ash (CSA). Preliminary test were performed on the natural soils. Strength tests such as compaction and unconfined compressive strength (UCS) tests were also performed on both the natural and stabilised soils. CCR were fixed at 4 and 6% for CL and CH respectively, and then CSA was varied 4, 9, 14 and 19%. The result showed that the MDD decrease with respect to that of the natural soils as both soils were treated with CCR and CSA, while OMC increased. The UCS test with combination of (4% CCR + 4% CSA) for CL soil and (6% CCR +4% CSA) for CH soil at 7 days curing period improved strength by 11.38 and 6.03 times higher than that of the natural soil. They concluded that the CCR and CSA can be employed for expansive soil stabilization subject to further researches.

Gurugubelli *et al.* (2017) carried out a laboratory study on the strength improvement of expansive soil treated with calcium carbide residue and fly ash. In order to obtain the optimum content of the CCR and CCR:FA blends. Atterberg limits, compaction,

unconfined compressive strength and California Bearing Ratio (CBR) tests were conducted. The UCS samples were tested under different curing periods (7, 14, 21 and 28 days) and CBR tests were carried out after a curing period of 3 days. From the Atterberg limits result it was observed that with addition of CCR, the plasticity index of the black cotton soil reduced. By increasing the fly ash content to the optimum CCR %, there was increased in strength up to certain percentage after which it decreased. They concluded that using 6% CCR or 8% CCR + 10% FA blends will improve black cotton soil for pavement construction.

Neeraj and Ahirwar (2014) studied performance of black cotton soil, treated with CCR and stone dust. Soil characterization was carried out according to the American Standard for Testing and Materials (ASTM), which consist of index properties, compaction, Unconfined Compressive Strength and California Bearing Ratio (CBR). Various percentages of CCR and stone dust were mixed with the soil samples. It was observed that the input of CCR reduced the MDD, while OMC increased. The stone dust also increases the MDD of the blended CCR-clay mixture, which indicates a packing effect and improvement of unconfined compressive strength and CBR. They concluded that combination of equal amount of stone dust and CCR is more effective, (10%-10%) than the addition of stone dust and CCR individually, to black cotton soil in controlling swelling behavior. They therefore stated that this application will go a long way in reducing the total cost of stabilisation.

Oyediran and Fadamoro (2015) studied the strength characteristics of two genetically different shales treated with both Rice Husk Ash (RHA) and Coconut Husk Ash (CHA). Geotechnical properties test were performed on the natural shales. 2 to 20 % by weight of both RHA and CHA were separately added to Okitipupa (SW) and Enugu (SE) shales with the subsequent determination of Plasticity Index (PI), Maximum Dry Density (MDD), Optimum Moisture Content (OMC), Unconfined Compressive Strength (UCS) and

California Bearing Ratio (CBR). The results showed that the RHA and CHA were found to possess pozzolanic properties such that their addition to shale in modest amounts (not more than 10 % by weight) has beneficial effect on the strength characteristics. Addition of RHA produced shales with reduced PI, higher UCS, increased MDD and more pronounced reduction in OMC when compared with the CHA stabilised shales. However in general, addition of 10% RHA and 6–10% CHA brought about optimal effect on the geotechnical properties of shales and as such can be regarded as the optimum content. They concluded that the materials can thus serve as suitable alternatives to modify and stabilise problematic shale and hence help reduce construction costs, environment hazards and ultimately bring about shales with improved geotechnical properties.

2.4 Pozzolanas

Pozzolans are defined as siliceous and aluminous materials which themselves possess little or no cementitious value but will, in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (Robert, 1993). More recent development has shown the potentials of utilizing agricultural wastes for producing cementitious compounds in conjunction with lime. Many plants, during their growth pick up silica from the ground, into the structure of their leaves, stalks and other parts (Akinmade, 2008). Therefore, when such agricultural products are burnt, organic materials which are the largest proportion are broken down and disappear as carbon dioxide, and water vapour (Akinmade, 2008). The resulting ash consists mainly of inorganic residue, mostly silica. When silica is in its reactive form, it can combine with lime to form insoluble cementitious compounds (Smith, 1992).

2.4.1 Chemical principles of pozzolanic reactions

Pozzolanic reactions take place when significant quantities of reactive CaO, Al₂O₃ and SiO₂ are mixed in presence of water (Seco *et al.*, 2012). Usually CaO is added as lime or cement, meanwhile Al₂O₃ and SiO₂ can be present in the material to develop cementation gels to be added as cement or, for example, with a pozzolan. In this process the hydration of the CaO liberates OH⁻ ions, which causes an increase pH values to approximate 12.4 (Seco *et al.*, 2012). Under these conditions pozzolanic reactions occur: the Si and Al combine with the available Ca, resulting in cementitious compounds called Calcium Silicate Hydrates (CSH) and Calcium Aluminate Hydrates (CAH) (Dermatas and Meng, 2003; Nalbantoglu, 2004; Guney *et al.*, 2007; Yong and Ouhadi, 2007; Chen and Lin, 2009). A simplified qualitative representation of these reactions is summarized below:



These compounds are responsible for improving the mechanical properties of the mix, due to increasing development of pozzolanic reactions over time; some authors stated that this may take place over years (Wild *et al.*, 1998).

Where these oxides are not available in sufficient quantities in the materials to be cemented, they must be incorporated with the binder (Seco *et al.*, 2012). In these cases, it is particularly advantageous to use stabilisers, like OPC, those rich in SiO₂ and Al₂O₃ as well as in CaO (Wild *et al.*, 1998; Degirmenci *et al.*, 2007) or for example the use of lime and pozzolan mixes. When these oxides are present in a material that needs to be cemented, it is not necessary to add them as a binder. This situation is most usual in stabilisation of clay soils where oxides are present in matrix. They are naturally rich in Si and Al oxides, which

become soluble at high pH conditions and then become available for development of the pozzolanic reactions (Seco *et al.*, 2012). The improved mechanical capacities achieved in each case depend on the amount, reactivity and concentration of the oxides, the size and shape of the particles and also on the curing conditions (Misra *et al.*, 2005; Yarbasi *et al.*, 2007; Goktepe *et al.*, 2008). One adverse effect related to pozzolanic reactions is caused by presence of sulphate (SO_4^{2-}) (Seco *et al.*, 2012). It may often appear both in industrial wastes and in natural soils, provoking the formation of a highly hydrated and expansive mineral, called ettringite ($\text{Ca}_6\text{Al}_2(\text{SO}_4)_3(\text{OH})_{12}\cdot 26\text{H}_2\text{O}$) (Seco *et al.*, 2012). The mechanics of the formation of ettringite are not yet well established (Mohamed, 2000), although the conditions for its formation are well known and (Ouhadi and Yong, 2003) are:

- i. High pH,
- ii. Presence of soluble Al,
- iii. Presence of soluble Ca,
- iv. Presence of soluble sulfate and
- v. Availability of water.

2.4.2 Properties of pozzolanas

The chemical composition of pozzolanas can be roughly summarized as follows:

Silica + Alumina + Iron Oxide – Not less than 70 percent

Other Oxides and Alkalis – Not more than 15 percent

Loss on ignition – Not more than 15 percent

The ASTM C618 – 78 specifications for pozzolanas is given in Table 2.4

Table 2.4: Properties of pozzolanas

Property	Class N	Class F	Class C
Chemical Properties			
SiO ₂ + Al ₂ O ₃ + Fe ₂ O ₃ (%)	70	70	70
SO ₃ (Max %)	4	5	5
MgO (Max %)	5	5	5
Loss on ignition	10	2	6
Physical Properties			
Moisture content (%)	3	3	3
Fineness (%) on sieve No. 200 (mm)	85	85	85
Pozzolanic Activity			
Index with OPC at 28 days (%)	75	75	75
Pozzolanic Activity Index with lime at 7 days	5.5	5.5	5.5

2.4.3 Fineness of pozzolanas

An essential physical property of a cementing material that affects its affinity for water is its fineness. The activity of pozzolanas is increased by fine grinding. ASTM C618-78 specification requires that the percentage passing sieve No. 200 \geq 85% (Lea, 1956, Smith, 1992).

2.4.4 Use of pozzolana in soil improvement

In order to minimize the high cost of soil improvement when conventional additives are used, geotechnical Engineers (Cokca, 2001; Medjo and Riskowiski, 2004) have focused on pozzolanas for use as substitute or partial replacement for the standard stabilizers. A large amount of these materials are obtained from agricultural wastes. When plant residues are burnt, organic materials which are the largest constituents are broken down and disappear as carbon dioxide and water vapour. The ash which remains contains mostly inorganic residue, notable silica in amorphous form, which can react with oxides in the soil, thus aiding improvement of the soil properties.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials

The materials used for this study are Tropical Black Clay (TBC), Coconut Shell Ash (CSA) and Calcium Carbide Residue (CCR).

3.1.1 Tropical black clay (TBC)

The tropical black clay used for this project was collected by method of disturbed sampling at depth of between 0.5 to 1.5m at Gwako village latitude of 8°58'22.8" N and a longitude of 7°7'8.4" E along Gwagwalada express way area of Abuja, Nigeria. The sample was preserved in polythene bags and transferred to the Geotechnical laboratory of Federal University of Technology, Minna.

3.1.2 Coconut shell ash (CSA)

The coconut shell was obtained from local coconut dealers in Minna, Niger State. It was sun dried to remove moisture. The ash was obtained through burning and the resulting ash was sieved through BS No 200 (75µm) before usage.

3.1.3 Calcium carbide residue (CCR)

The calcium carbide residue was obtained from panel beaters in Minna. It was dried, grind and sieved through sieve No. 200 (75µm) before usage.

3.2 Methodology

3.2.1 Determination of natural moisture content

The natural moisture content of the soil sample was carried out in accordance with BS 1377 (1990) part 2. Two empty cans were used. The cans were clean, label and weigh as (M_1). Portion of the wet soil sample were placed in the empty cans and weigh as (M_2). The cans

were then placed in the oven at temperature of between 105⁰C to 110⁰C for 24 hours to dry. The can plus dry sample was removed and weigh as (M₃). The average moisture content was calculated in percentage, using equation (3.1).

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

Where: w = natural moisture content in percentage.

M₁ = Mass of empty can

M₂ = Mass of can + wet soil

M₃ = Mass of can + dry soil

3.2.2 Determination of particle size distribution

This was carried out in accordance with BS 1377 (1990) part 2. 300g of the soil sample was soaked for 24 hours and washed through BS No. 200 sieve. The particles retained after washing was oven dried. The sample was sieved through BS sieves with sizes 5.00mm, 3.35mm, 2.00mm, 1.18mm, 850µm, 600µm, 425µm, 300µm, 150µm, 75µm and base pan. The percentage mass retained was determined for each sieve size and used in the computations for plotting the particle size distribution curve.

$$\text{Percentage Retained} = \frac{\text{mass of the soils sample on each sieve}}{\text{total mass of soils sample}} \times 100 \quad (3.2)$$

3.2.3 Specific gravity (Gs)

Specific gravity of both soil, CCR and CSA was carried out in accordance with BS 1377 (1990) part 2. A clean dry pycnometer of 250ml was weighed to the nearest 0.01g as (M₁). Air dried soil sample of about 20g was placed in the pycnometer and weighed to the nearest 0.01g as (M₂). The pycnometer was then half filled with water and shaken to allow the trapped air to be released and further filled to the mark of the pycnometer. The mass of sample and water filled to the mark was weighed to the nearest 0.01g as (M₃). The

pycnometer was emptied and washed thoroughly before refilling it with water only and weighed to the nearest 0.01g as (M₄). Specific gravity was then calculated using the equation (3.3).

$$G_s = \frac{M_2 - M_1}{(M_4 - M_1) - (M_3 - M_2)} \quad (3.3)$$

Where: G_s = Specific Gravity.

M₁ = Weight of bottle

M₂ = Weight of bottle + dry soil

M₃ = Weight of bottle + soil + water

M₄ = Weight of bottle + water.

3.2.4 Atterberg limits

These tests were performed using the Casagrande apparatus on the soil sample passing through a sieve No.40 (425µm) to determine the liquid limit, plastic limit and plasticity index for the natural soil sample in accordance with BS 1377 (1990) Part 2.

3.2.4.1 Liquid limit (LL)

This was carried out in accordance with BS 1377 (1990) part 2. A soil sample of about 200g was taken from material passing through sieve 425µm and placed on a flat glass and mixed thoroughly with water using palette knives until it became a thick homogeneous paste. The test was carried out for groove closures at different numbers of blows. After each of the groove closures, moisture content samples were taken. The moisture content of the soil sample at the closed point was determined. The liquid limit was determined from the graph of the moisture content plotted against the number of blows.

3.2.4.2 Plastic limit (PL)

This was carried out in accordance with BS 1377 (1990) part 2. About 20g of soil sample passing sieve 425 μ m was placed on a glass plate and then mixed thoroughly with water. The moist soil sample was rolled between the fingers and palm at a sufficient pressure to form a uniform thread of 3mm by forward and backward movement of the hand. The rolling and re-balling was continued until the thread crumbled under the pressure of rolling and the sample could no longer be rolled into a thread. The crumbled sample was taken for moisture content determination. Two moisture content samples were being taken and their average in percent was computed as the plastic limit of the soil.

3.2.4.3 Plasticity index (PI)

The Plasticity Index (PI) was obtained by determining the difference between Liquid Limit (LL) and the Plastic Limit (PL) of the soil.

$$PI = LL - PL \quad (3.4)$$

3.2.5 Compaction characteristics

The test was carried out in accordance with BS 1377 (1990) part 4 and BS 1924 (1990), using standard proctor energy level for both the natural and stabilized soil at different percentages of the admixtures.

3kg of air dried soil sample was weighed and mixed thoroughly using varying quantity of water for every compaction. The soil samples were compacted in three (3) layers, with each layer receiving twenty five (25) blows of 2.5kg rammer dropped from a height of 300mm. The blows were distributed uniformly over the surface of the soil. The collar was then removed and the compacted soil was leveled off at the top of the mould with a straight edge. The mould and the compacted soil were weighed. The immediate top and bottom of the compacted soil samples were taken for determination of moisture content. The whole soil sample was then removed from the mould and placed on the large mixing tray. Water

was added to the sample and mixed properly. The compaction was repeated using the same number of blows and layers as described above. This was done until a reduction in weight of the mould and the soil sample was noticed. The values of dry density and moisture content were calculated and the graph of dry density was plotted against the moisture content, where the point of Optimum Moisture Content (OMC) and the Maximum Dry Density (MDD) were determined. The bulk density was calculated using the equation (3.5).

$$\rho_b = \frac{M_s}{V_s} \quad (3.5)$$

Where: ρ_b = Bulk density (g/cm^3)

M_s = Mass of compacted soil (g)

V_s = Volume of mould (cm^3)

The dry density was determined using equation (3.6)

$$\rho_d = \frac{100\rho_b}{(100+W)} \quad (3.6)$$

Where: ρ_d = Dry density (g/cm^3)

ρ_b = Bulk density (g/cm^3)

w = Moisture content (%).

3.2.6 Unconfined compressive strength (UCS)

Unconfined Compressive Strength (UCS) test was conducted in accordance with BS 1377 (1990) Part 7 and BS 1924 (1990) by using the standard proctor energy level results for the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD).

The samples were moulded at the respective MDD and OMC and cured in moist sand for 1, 7, 14, 28, 60 and 90 days. At the end of the curing period, the specimens were then tested for unconfined compressive strength.

$$\text{Compressivestrength} = \frac{\text{Failureload}}{\text{Surfaceareaofspecimen}} \quad (3.7)$$

CHAPTER FOUR

4.0

RESULTS AND DISCUSSION

4.1 Chemical Composition of Coconut Shell Ash and Calcium Carbide Residue

Table 4.1 shows the chemical composition of Coconut Shell Ash (CSA) and Calcium Carbide Residue (CCR) used respectively. The cementing characteristics of CSA are dependent on its oxide composition. From the results it was observed that the combined percentages of Silicon dioxide (SiO_2), Aluminum oxide (Al_2O_3), and Iron oxide (Fe_2O_3) in the CSA is more than 70%, which is an indication of good pozzolana and it falls under class N pozzolanas as specified by ASTM C618-78 classification of Pozzolanas. Also observed that the chemical composition of CCR is almost the same to that of Ordinary Portland cement. For instance, CaO which is the major constituent of Ordinary Portland cement is also a major constituent in CCR. However, the amount of calcium oxide (CaO) in CSA was found to be very low compared to that in the CCR. The high content of CaO in the CCR also aided those in CSA in providing the required improvement in the stabilisation process. This trend of oxide composition is similar to that reported by (Utser and Taku, 2012; Manasseh and Ejelikwu, 2013).

Table 4.1: Detectable chemical composition of coconut shell ash and calcium carbide residue

Oxide	Percentage composition (%)	
	CSA	CCR
SiO_2	31.73	2.45
Al_2O_3	26.17	1.83
Fe_2O_3	16.78	0.12
CaO	4.85	65.04
K_2O	0.78	0.08
Na_2O	2.75	0.13
MgO	3.29	1.81
SO_3	0.64	0.57
MnO	0.79	-
P_2O_5	1.18	0.71
TiO_2	1.42	-
LOI	9.62	26.67

4.2 Properties of the Natural Tropical Black Clay

The results (Table 4.2) of preliminary tests conducted on the natural soil, for identification and classification purposes showed that the fraction passing BS No. 200 sieve (75 μ m) is 63.17% which is an indication that the soil is fine grained. The natural moisture content of the soil is 13.65%. The index properties are presented in Table 4.2, while Figure 4.1 shows the particle size distribution curve. From the results, the soil is classified as CH and A-7-5(15) according to Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO), respectively. The soil is greyish black in colour, and has specific gravity of 2.63, Maximum Dry Density (MDD) of 1.784 g/cm³, Optimum Moisture Content (OMC) of 18.0% and Unconfined Compressive Strength (UCS) of 19.196 kN/m².

The above results indicates that the geotechnical properties of the natural soil fall below the standard recommended for most civil engineering construction works, especially for base course materials in highway construction (Osinubi and Medubi, 1997), and therefore need stabilisation.

Table 4.2: Geotechnical properties of the natural tropical black clay

Property	Values
Fraction passing BS No 200 sieve (%)	63.17
Natural moisture content (%)	13.65
Specific gravity	2.63
Liquid limit (%)	58
Plastic limit (%)	33.88
Plasticity index (%)	24.14
Group index	15
USCS	CH
AASHTO classification	A-7-5
MDD (g/cm ³)	1.784
OMC (%)	18.0
UCS (kN/m ²)	19.196
Colour	Greyish black

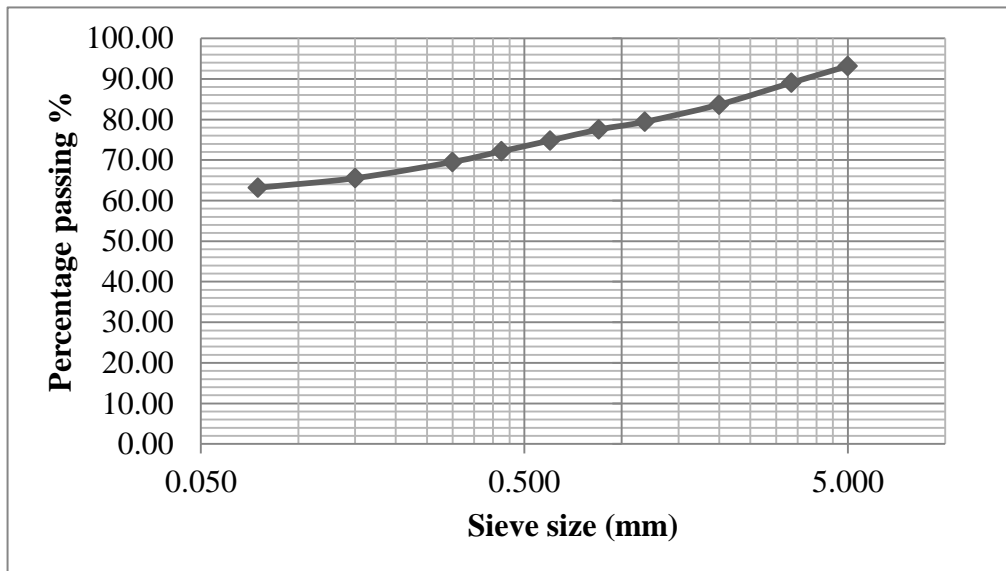


Figure 4.1: Particle size distribution curve for the natural soil

4.3 Variation of Compaction Characteristics with Higher Dosage of the Additives

4.3.1 Maximum dry density

Variation of Maximum Dry Density (MDD) with dosage of CSA at various percentages of CCR is presented in Figure 4.2 (see Table B1 in Appendix B). From the Figure, it can be observed that at constant dosage of CCR, MDD of the stabilised soil generally decreased with increase in percentage of CSA from 1.784 g/cm³ at 0% CSA and 0% CCR to a minimum value of 1.666 g/cm³ at 8% CSA and 4% CCR. This represents 6.6% decrease in MDD of the soil. The decreased could be attributed to the lower specific gravities of both CSA and CCR as compared to that of the soil. With the soil having specific gravity of 2.63, which is typical for clay soil, as compared to lower specific gravities of CSA (2.26) and CCR (2.35); with increase in percentage composition of CSA and CCR in the soil, the specific gravity will tend to reduce. This trend of MDD variation is similar to that reported by Isah and Sharmila (2015), when they treated CL and CH soils with CCR and CSA. Furthermore, the reduction in MDD of the treated soil may also be due to flocculated and agglomerated clay particles occupying larger spaces leading to corresponding decrease in dry density (Akinmade, 2008; Oyediran and Fadamoro, 2015).

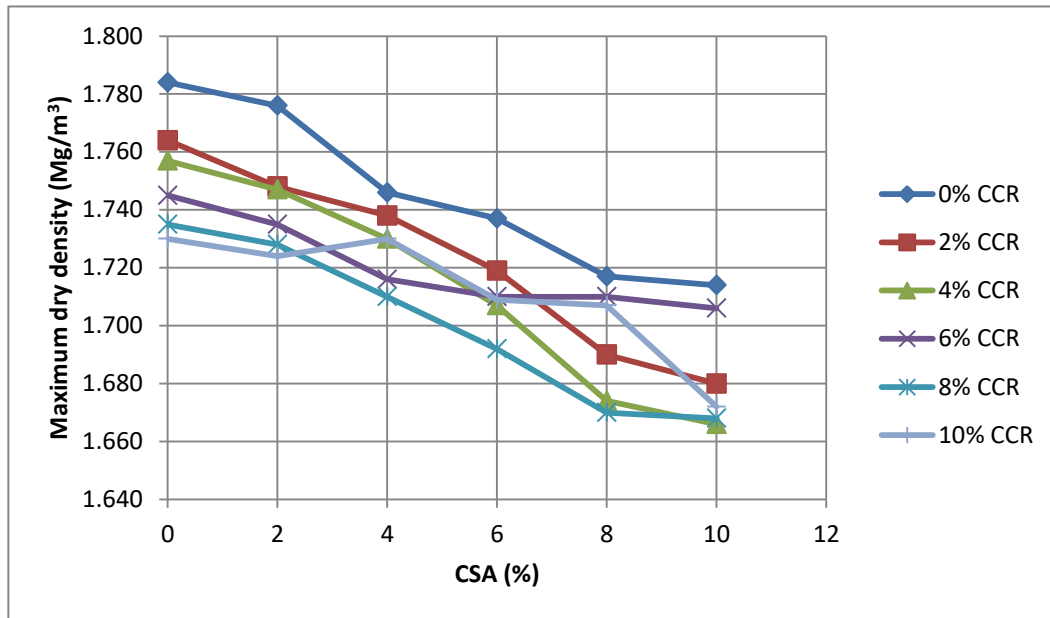


Figure 4.2: Variation of MDD with increase in dosage of CSA and CCR

4.3.2 Optimum moisture content

Variation of Optimum Moisture Content (OMC) with dosage of CSA, at various percentages of CCR is presented in Figure 4.3 (see Table B2 in Appendix B). It was observed that at constant dosage of CCR, OMC of the stabilised soil generally increased with increase in percentage of CSA, from 18.0% at 0% CSA and 0% CCR to a maximum value of 23.7% at 4% CSA and 8% CCR. This represents 31.7% increase in OMC of the natural soil. The increased values of OMC could be attributed to the reaction, between the CCR and CSA in the soil. While CCR helps in flocculating the soil (typical when lime is used to improve soil) CSA react with the CCR, silica and alumina in the soil. Furthermore inclusion of the stabilisers helps in reducing the free silt content and clay fractions thereby forming materials that are coarser. This reaction requires water to proceed, hence increase in the OMC. This observed trend of OMC variation is also similar to that reported by Isah and Sharmila (2015).

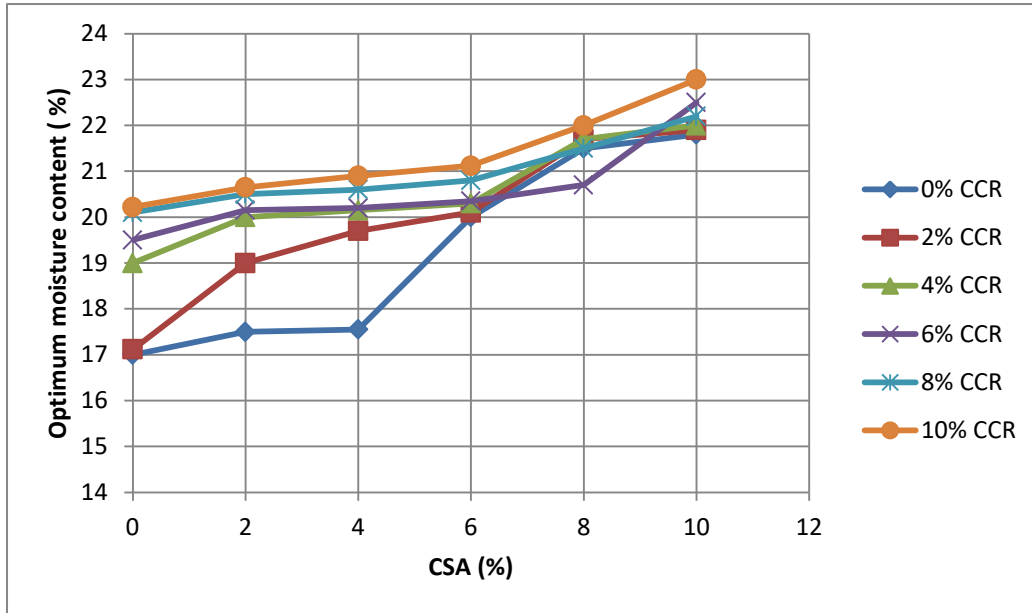


Figure 4.3: Variation of OMC with increase in dosage of CSA and CCR

4.4 Variation of Unconfined Compressive Strength with Higher Dosage of the Additives

Variations of Unconfined Compressive Strength (UCS) of the tropical black clay with varying dosages of both CSA and CCR are shown in Figures 4.4, 4.5, 4.6, 4.7, 4.8 and 4.9 (see Table C1, C2, C3, C4, C5 and C6 in Appendix C) when cured for 1, 7, 14, 28, 60 and 90 days, respectively.

From Figure 4.4, the results of UCS of the stabilised soil, after 1 day of curing increased from 19.196 kN/m² at 0% CSA and 0% CCR to a maximum value of 381.37 kN/m² at 6% CSA and 10% CCR, which represents 1886.7% increase in strength of the natural black clay. Subsequent increase in the dosages of CSA shows a decrease to 181.936 kN/m² at 10% CSA and 10% CCR contents. The decrease in UCS values could be as a result of the excess CSA introduced in to the soil and therefore forming weak bonds between the soil and the cementitious compounds formed (Segun and Oluyemisi, 2017).

A general pattern was observed from the Figures which show the UCS values after 7, 14, 28, 60 and 90 days curing increased from 9.636 kN/m² at 0% CSA and 0% CCR to a

maximum value of 1194.972 kN/m² at 0% CSA and 10% CCR contents. This represents 12301.1% increase in the UCS. The maximum strength was achieved after 90 days of curing periods at 0% CSA and 10% CCR contents. It was observed that as the percentage composition of both CSA and CCR increased, unconfined compressive strength of the treated soil increased. Also, observation and comparison of the figures revealed that as the curing period increases, unconfined compressive strength of the treated soil increased. This noticeable increase in unconfined compressive strength of the treated soil, with increase in curing time is attributed to the pozzolanic reaction between silica and alumina, brought about by the CSA in the composite material (Osinubi and Madubi, 1997). Since pozzolanic reaction is relatively a time based reaction, the effect is more pronounced beyond 14 days curing period, which is vividly seen between 28 (Figure 4.7), 60 (Figure 4.8) and 90 days (Figure 4.9) of curing.

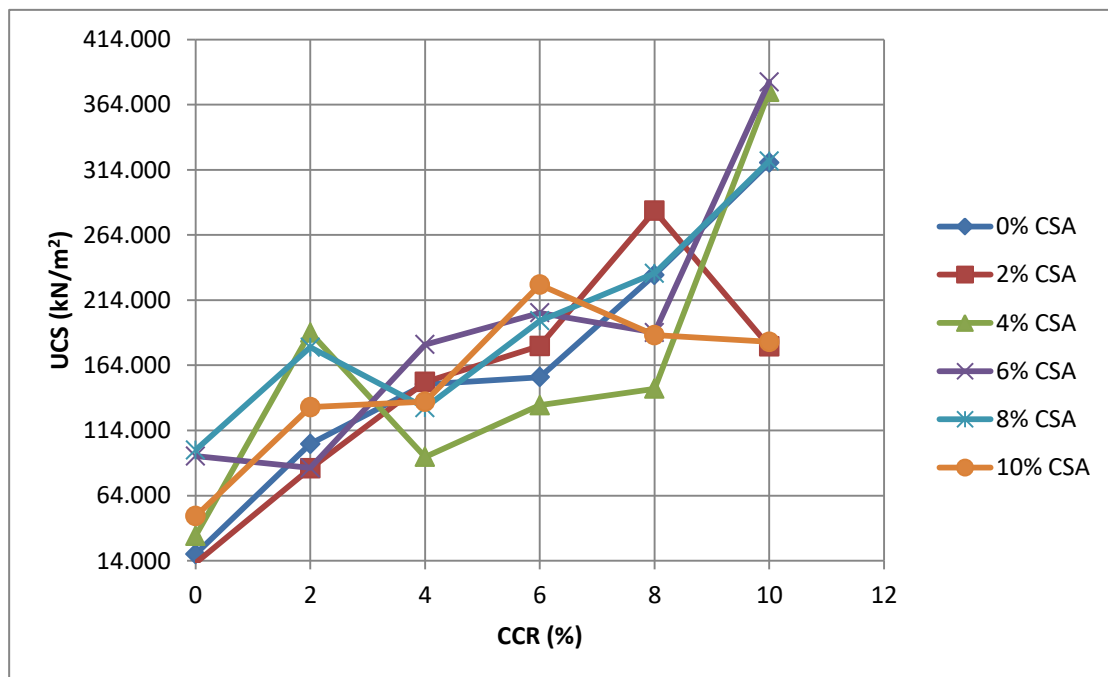


Figure 4.4: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 1 day curing period

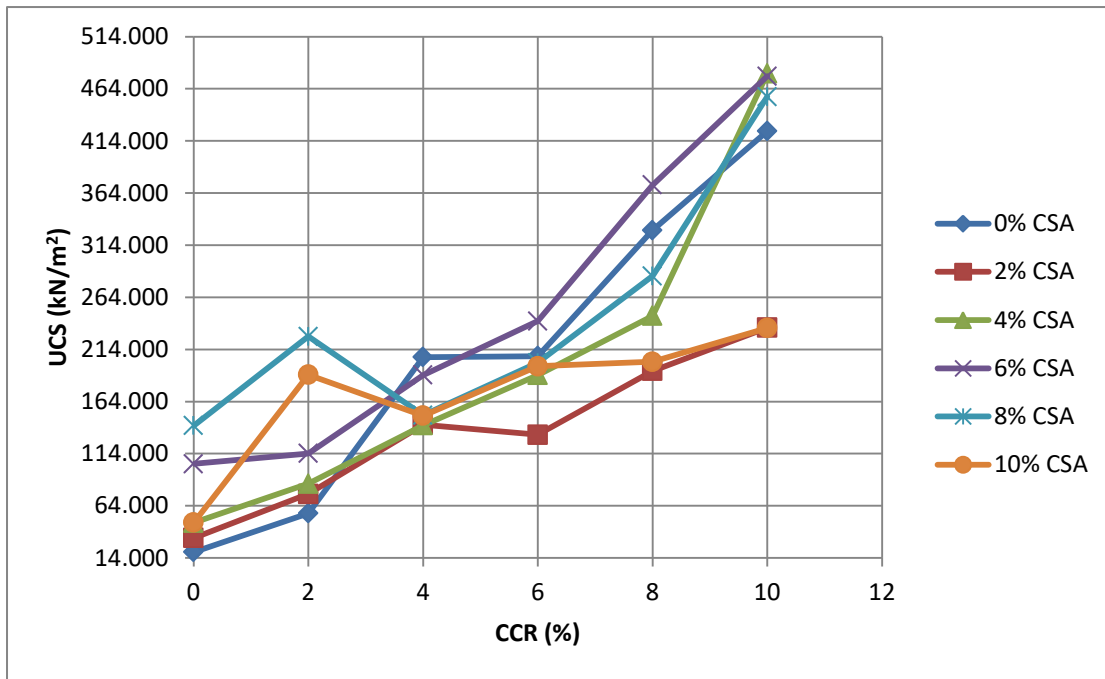


Figure 4.5: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 7 days curing period

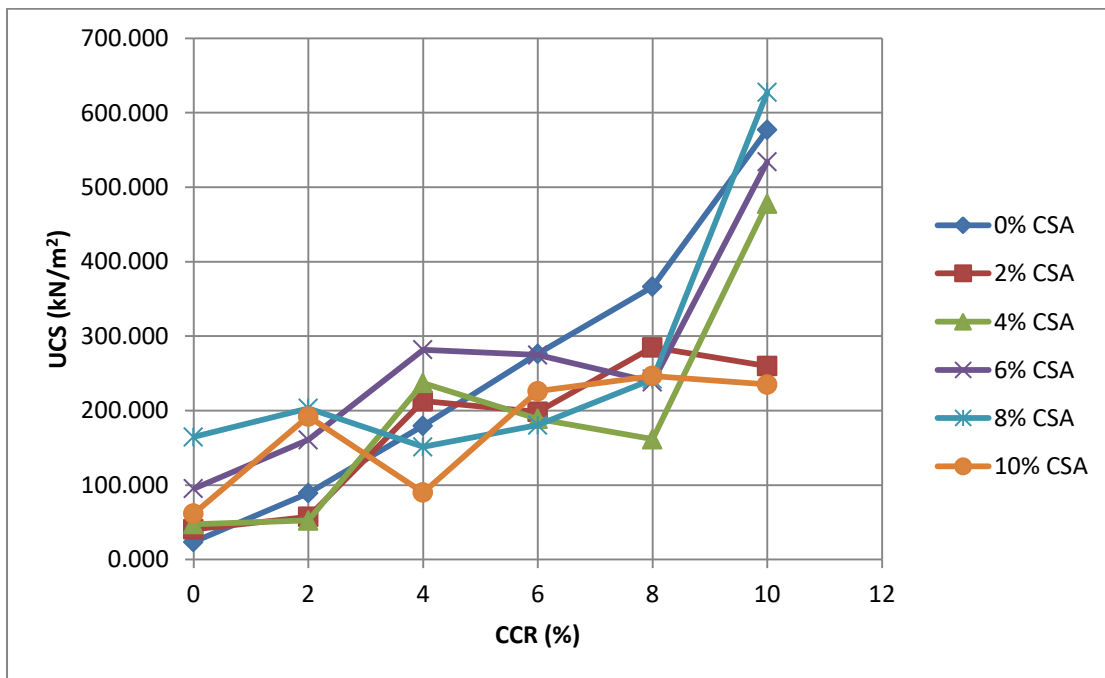


Figure 4.6: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 14 day curing period

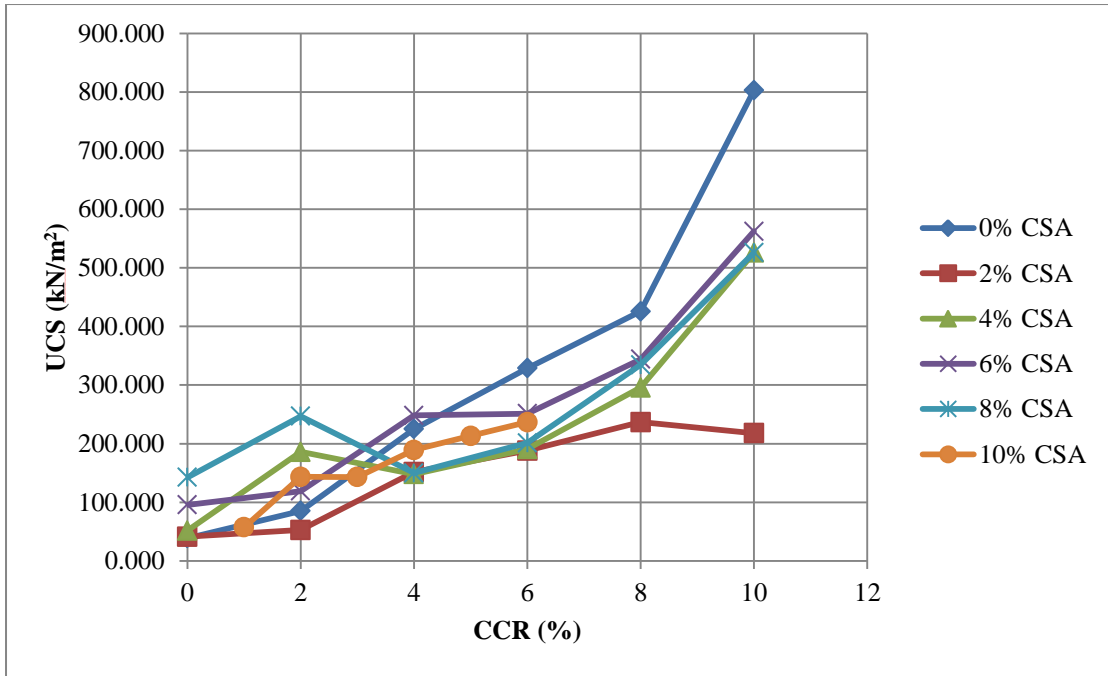


Figure 4.7: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 28 day curing period

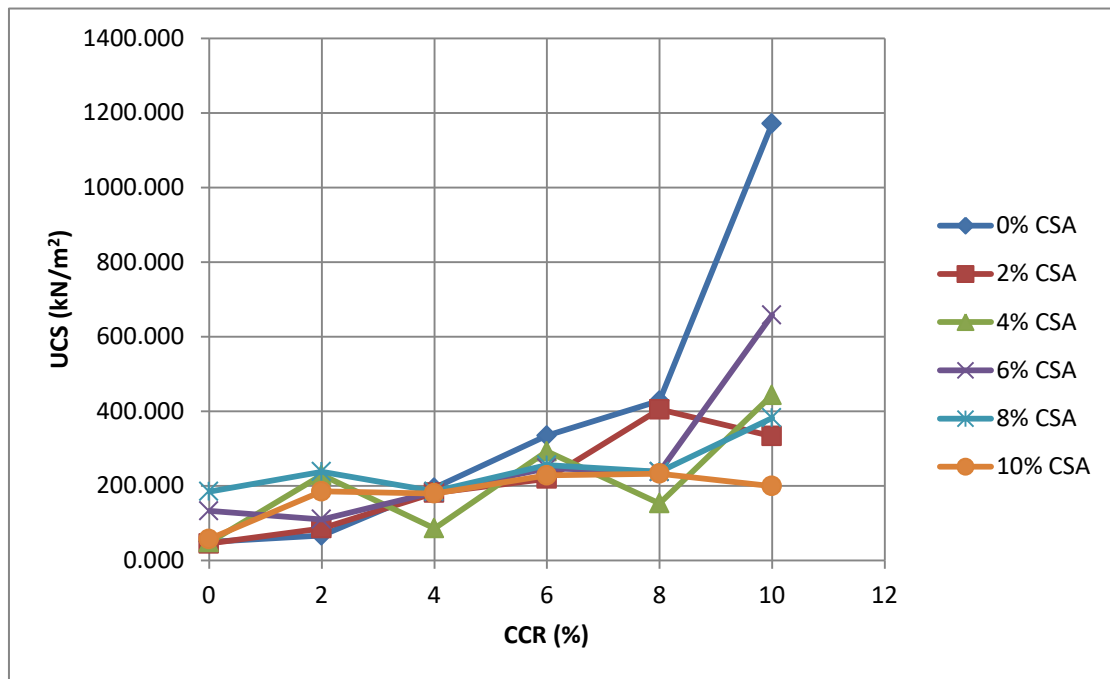


Figure 4.8: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 60 day curing period

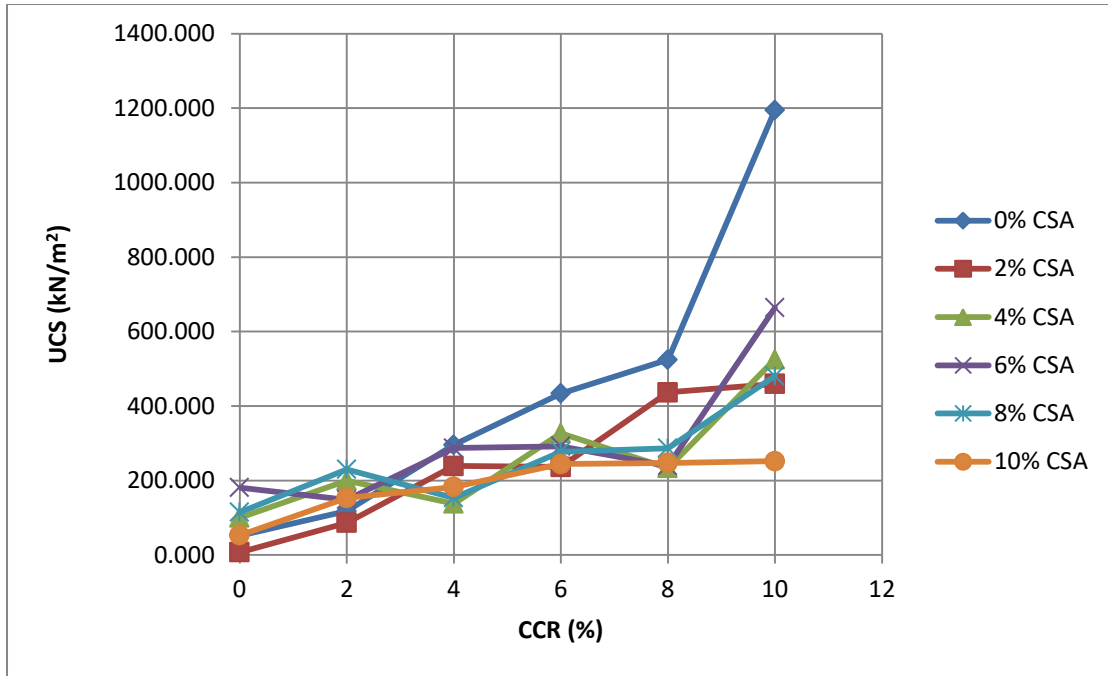


Figure 4.9: Variation of unconfined compressive strength with varying dosages of both CSA and CCR after 90 day curing period

Although the 1710 kN/m² specified by TRRL (1977) for base materials was not achieved within the composition considered, the requirement of 687-1373 kN/m² for sub-base as specified by Ingles and Metcalf (1972) was met. Therefore CCR and CSA can be use for treatment of tropical black clay for use as sub-base material.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the study, the following conclusion is drawn:

The combined percentages of Silicon dioxide (SiO_2), Aluminum oxide (Al_2O_3), and Iron oxide (Fe_2O_3) in the CSA is more than 70%, which is an indication of good pozzolana and it falls under class N specified by ASTM C618-78 classification of Pozzolanas.

The Tropical Black Clay (TBC) studied was classified under CH and A-7-5, according to Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Official (AASHTO) respectively. Soils under these groups are generally classified as material of poor engineering properties that are not suitable for road construction unless the properties are improved.

There was a general decrease in maximum dry density of the treated soil with increase in dosage of the additives from 1.784 g/cm^3 at 0% CSA and 0% CCR to a minimum value of 1.666 g/cm^3 at 8% CSA and 4% CCR.

Optimum moisture content of the treated soil increased with increase in dosage of the additives from 18.0% at 0% CSA and 0% CCR to a maximum value of 23.7% at 4% CSA and 8% CCR.

Unconfined compressive strength of the treated soil showed general increase with both increase in dosage of the additives and curing period from 19.196 kN/m^2 at 0% CSA and 0% CCR to a maximum value of 1194.972 kN/m^2 at 0% CSA and 10% CCR after 90 days curing.

Although the 1710 kN/m^2 specified by TRRL (1977) for base materials was not achieved within the composition considered, the requirement of $687\text{-}1373 \text{ kN/m}^2$ for sub-base as specified by Ingles and Metcalf (1972) was met. Therefore CCR and CSA can be use for treatment of tropical black clay for use as sub-base material.

This suggest that using CSA and CCR at large scale level, in geotechnical engineering practice will help in the provision of stable and durable structures, reduce cost of soil improvement and also reduces environmental nuisance the unused waste causes.

5.2 Recommendations

From the study, the following recommendations are drawn:

1. The 1710 kN/m^2 specified by TRRL (1977) for base materials was not achieved within the composition considered, the requirement of $687\text{-}1373 \text{ kN/m}^2$ for sub-base as specified by Ingles and Metcalf (1972) was met. Therefore CCR and CSA can be use for treatment of tropical black clay for use as sub-base material for lightly trafficked roads.
2. The maximum percentage of CSA and CCR should not exceed 15%. And it is obvious higher CSA decreased the strength and hence may not be needed.

5.3 Contribution to Knowledge

The study showed that Tropical Black Clay (TBC) can be stabilised with 2 – 10% Calcium Carbide Residue (CCR) and Coconut Shell Ash (CSA), with reasonable strength improvement for use as sub-base in lightly trafficked roads.

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APPENDICES

Appendix A: Results of index properties of test samples

Table A1: Sieve analysis result for natural tropical black clay

Sieve size (mm)	Mass of sieve (g)	Mass of sieve+Soil (g)	Mass retained (g)	Percentage Retained	Percentage Passing
5.000	474.70	495.20	20.50	6.83	93.17
3.350	468.00	480.30	12.30	4.10	89.07
2.000	416.80	433.00	16.20	5.40	83.67
1.180	384.90	397.60	12.70	4.23	79.43
0.850	351.70	357.40	5.70	1.90	77.53
0.600	467.80	476.00	8.20	2.73	74.80
0.425	435.10	443.00	7.90	2.63	72.17
0.300	313.00	320.90	7.90	2.63	69.53
0.150	420.40	432.40	12.00	4.00	65.53
0.075	398.50	405.60	7.10	2.37	63.17

Table A2: Natural moisture content of tropical black clay

Can Number	1	2
Weight of can (g)	23.4	26.1
Weight of can+wet soil (g)	57.6	58.5
Weight of can+dry soil (g)	53.5	54.6
weight of dry soil (g)	30.1	28.5
weighy of water (g)	4.1	3.9
Moisture content (%)	13.62	13.68
Average MC (%)	13.65	

Table A3: Specific gravity of natural tropical black clay

Pycnometer Number	1	2	3
Mass of pycnometer(g)	69.0	69.0	69.0
Mass of pycnometer+dry soil(g)	111.1	116.2	110.1
Mass of pycnometer+drysoil+water(g)	194.4	197.3	194.2
Mass of pycnometer+water(g)	168.4	168.4	168.4
Specific gravity (Gs)	2.61	2.58	2.69
Average Gs	2.63		

Table A4: Specific gravity of calcium carbide residue

Pycnometer Number	1	2	3
Mass of pycnometer(g)	69.0	69.0	69.0
Mass of pycnometer+dry soil(g)	92.3	93.6	87.5
Mass of pycnometer+drysoil+water(g)	188.5	188.9	185.7
Mass of pycnometer+water(g)	175.0	175.0	175.0
Specific gravity (Gs)	2.38	2.30	2.37
Average Gs		2.35	

Table A5: Specific gravity of coconut shell ash

Pycnometer Number	1	2	3
Mass of pycnometer(g)	69.0	69.0	69.0
Mass of pycnometer+dry soil(g)	85.2	83.9	87.0
Mass of pycnometer+drysoil+water(g)	178.1	176.4	177.9
Mass of pycnometer+water(g)	168.4	168.4	168.4
Specific gravity (Gs)	2.49	2.16	2.12
Average Gs		2.26	

Table A6: Atterberg limits of natural tropical black clay

Can Number	LIQUID LIMIT					PLASTIC LIMIT	
	1	2	3	4	5	1	2
Penetration	6.20	9.50	13.50	16.00	23.90		
Weight of Can (g)	9.90	24.30	38.40	38.30	23.20	22.6	24.8
Weight of can+wet soil (g)	24.70	39.70	58.80	64.90	56.30	30.8	32.4
Weight of can+Dry soil (g)	21.00	35.50	52.30	55.30	43.30	28.7	30.5
Weight of Moisture (g)	3.70	4.20	6.50	9.60	13.00	2.1	1.9
Weight of Dry Soil (g)	11.10	11.20	13.90	17.00	20.10	6.1	5.7
Moisture Content (%)	33.33	37.50	46.76	56.47	64.68	34.43	33.33
Liquid Limit (%)	58.00		Average Plastic Limit			33.88	

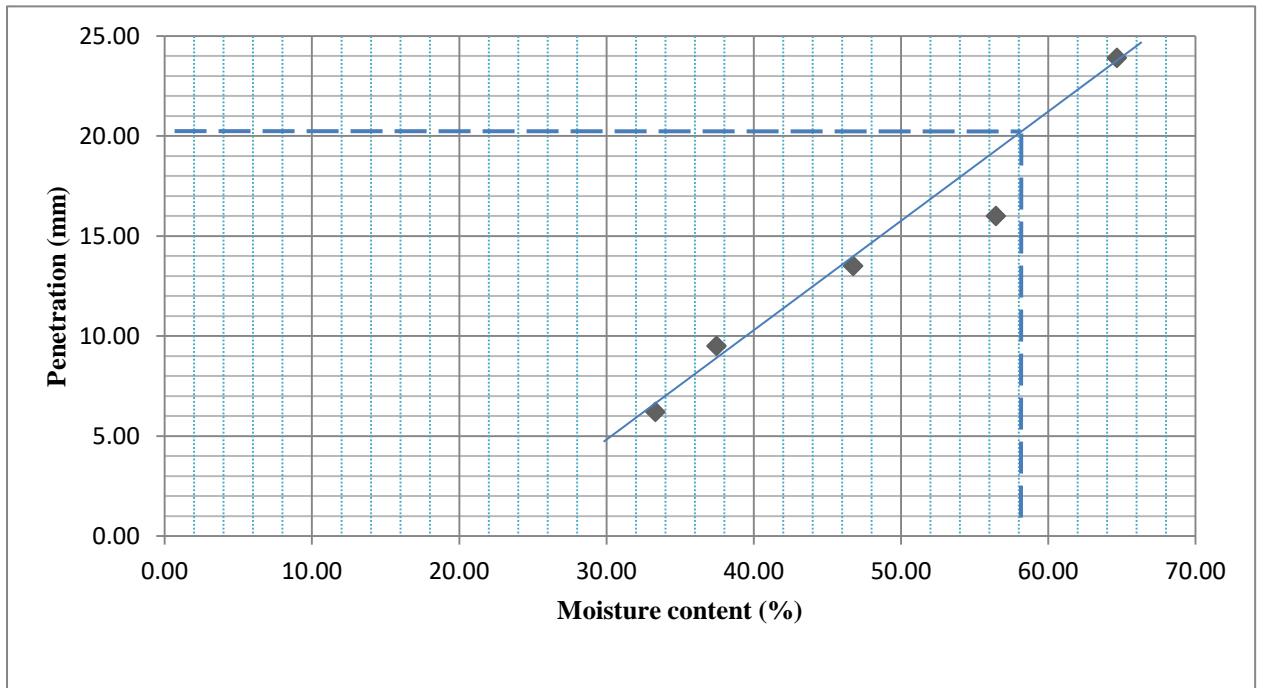


Figure A: Cone penetration test plot

Appendix B: Results of compaction characteristics of samples

Table B1: Results of maximum dry density (g/cm^3) with various dosages of CSA and CCR

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	1.784	1.736	1.719	1.7	1.73	1.707
2	1.746	1.69	1.748	1.695	1.71	1.709
4	1.776	1.68	1.674	1.716	1.668	1.73
6	1.717	1.705	1.757	1.71	1.672	1.724
8	1.738	1.764	1.666	1.71	1.71	1.728
10	1.714	1.738	1.73	1.706	1.67	1.672

Table B2: Results of optimum moisture content (%) with various dosages of CSA and CCR

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	18.00	19.70	20.51	21.50	20.00	19.00
2	21.50	21.50	20.00	22.50	20.50	23.00
4	20.00	21.60	22.00	19.00	23.70	17.00
6	17.50	20.00	16.00	20.00	20.80	16.50
8	17.00	17.00	21.70	18.70	19.50	17.60
10	21.80	19.00	20.00	20.00	22.20	22.00

Appendix C: Results of strength test of samples

Table C1: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 1 day curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	19.196	103.613	149.485	154.788	233.251	319.391
2	12.377	84.889	151.119	178.726	282.581	178.483
4	33.147	189.153	93.683	133.301	146	373.827
6	94.321	85.234	179.938	204.163	189.153	381.37
8	98.904	177.756	131.335	198.075	234.527	320.693
10	48.246	131.871	136.026	225.759	187.111	181.936

Table C2: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 7 days curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	9.636	56.823	206.665	207.507	328.338	423.298
2	3.103	75.457	141.865	132.229	193.359	234.846
4	47.799	189.409	85.234	246.231	141.291	478.627
6	104.315	114.258	371.836	238.037	241.171	476.074
8	140.908	226.371	150.914	201.291	284.496	456.419
10	47.863	189.664	150.506	197.807	202.242	234.527

Table C3: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 14 days curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	38.596	89.12	179.696	276.366	366.36	576.918
2	0.629	57.282	212.798	198.343	284.879	259.735
4	47.48	52.368	237.08	188.898	161.648	477.989
6	95.087	160.78	281.637	274.643	238.356	533.918
8	164.616	203.066	151.323	180.423	242.147	627.238
10	61.641	192.051	90.09	226.065	246.563	235.165

Table C4: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 28 days curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	23.931	85.349	225.146	328.785	425.595	802.817
2	1.258	52.649	151.731	188.132	236.761	217.82
4	51.736	185.918	47.927	190.94	95.725	525.788
6	95.598	119.019	344.152	248.222	251.305	562.1
8	142.631	246.895	149.893	201.291	334.592	525.788
10	57.512	143.397	143.014	189.409	213.372	236.761

Table C5: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 60 days curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	9.675	66.382	194.406	334.146	427.893	1171.17
2	4.997	85.693	181.151	218.994	404.663	332.805
4	48.182	228.822	85.808	292.397	152.548	443.343
6	132.586	109.791	183.091	247.227	237.399	657.863
8	184.425	237.718	184.922	255.357	237.718	381.881
10	57.206	184.922	179.453	227.903	232.338	198.879

Table C6: Results of unconfined compressive strength (kN/m²) with various dosages of CSA and CCR after 90 days curing period

Coconut Shell Ash (%)	Calcium Carbide Residue (%)					
	0	2	4	6	8	10
0	52.158	117.583	295.562	433.808	524.384	1194.972
2	7.446	86.268	238.994	236.442	436.814	459.316
4	100.11	199.951	137.877	327.202	233.276	525.086
6	180.908	148.374	287.176	291.963	239.314	664.028
8	114.717	229.741	153.569	277.234	286.793	480.542
10	52.649	152.956	182.606	244.1	247.227	251.981