STABILISATION OF LATERITIC SOIL WITH CALCIUM CARBIDE WASTE FOR PAVEMENT APPLICATION

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MEng/SIPET/2018/8318

THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF THE DEGREE OF MASTER OF ENGINEERING IN CIVIL ENGINEERING (HIGHWAY AND TRANSPORTATION ENGINEERING)

APRIL, 2023

ABSTRACT

Soil Stabilisation is the process of improving the shear strength parameters of soil and thus increasing its bearing capacity. The poor soil along Ajaokuta-Lokoja Road was stabilised using Calcium carbide through the process of chemical stabilisation, but various tests was conducted on the soil which include physical and chemical property test, Specific gravity test, Compaction test, Unified compressive test (UCS) and California bearing ratio test (CBR). It was discovered that the soil belongs to the A-7-6 (13) according to AASHTO and CL group according to the Unified Soil Classification System, the XRD results shows that Quartz has the highest composition of 50.00%, montmorillonite 8.28%, anothoclase of 8.32% with anorthite having 6.12%, the particle size distribution gave a uniformly distributed graph, the specific gravity of 2.66 with MDD=1.99g/cm^3, OMC=21.00%, the UCS of the BSH increases with increase in curing duration, with the peak value for the 0 day, 7 days and 28 days curing ranging from 440, 810 and 1040 with the increasing concentration of the calcium carbide in the laterite, the CBR increased with an increase in the increase in CCW with the unsoaked CBR giving higher value compared to soak CBR for all the percentage of replacement of calcium carbide using the three energy levels which indicate that the shear parameters of the poor soil was improve making it suitable for use for pavement application, it is therefore recommended that Stabilisation with additives like Calcium carbide waste is recommended to improve the bearing capacity of the lateritic soil. Cement and CCW should be used to stabilize an unsuitable A-7-6 lateritic soil., for long strata of such lateritic clay soil, total complete removal and replacement is recommended.

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ABBREVIATIONS, GLOSSARIES AND SYMBOLS

C.C.W.	Calcium Carbide Waste
N.N.B.S =	Nigeria National Bureau of Statistics, Transport Statistic; NationalBureau of Statistics
C. B. R.	California Bearing Ratio
U.I.	University of Ibadan
D. C.	Direct Current
U. C. S.	Unconfined Comprehensive Strength
D. S.T.	Direct Shear Test
P.L.	Plastic Limit
L.L	Liquid Limit
B.S. H.	British Standard Heavy Compaction
B. S. L.	British Standard Light Compaction
W. A. S.	West African standard Compaction
M. D. D.	Maximum Dry Density

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

The growing in the population of the world, especially in developing nations (like Nigerian) has led to increasing demand for roadways, railways, housing facilities and other social infrastructures which are constructed basically on the soil. Soil with higher stability is essential to bear the weight of these structures. Generally speaking, the stability of any structure constructed on indirectly or directly depends on the soil stability. Road pavement is the mark path for transportation, which plays a great role in the industrialisation and civilisation of today and is vital to the organisation, comfort, convenience, economic growth and well-being of any society (World Bank, 2011). In many countries of the world roads or highways are the most utilized transportation infrastructures (Nigeria National Bureau of Statistics, Transport Statistic, N.N.B.S, 2017). Poor road infrastructure can lead to loss of lives and properties, high vehicle operating costs, road traffic congestion, a general rise in prices of goods and services, and consequently, impact negatively on the economy of a country (Minten and Kyle, 1999).

Provision of safe, efficient, affordable and reliable transport system for citizens of any nation is one of the major challenges facing mankind in the 21st century, the challenge of providing such transport facilities is not particularly acute to rural dwellers, it also affects urban areas in many less developed countries, where expansion of the urban population due to the high population growth rate and massive rural-urban drift have compounded the quest for provision of such transport system for citizens, Van-Rooy and Oyelami (2016). In recent years, the potential of laterite to be recognised, valued and desirable for use as construction material is being rediscovered, when referring to earth, soil, laterite or mud in pavement construction., they are materials excavated from the sub-soil layers of the ground kneaded together for highway construction.

Materials used for road construction pavement construction include bitumen, granite, cement and laterite. Laterite is an iron-rich, sub-aerial, weathering product, commonly believed to evolve as a result of intense, in situ substrate alteration under tropical or sub-tropical climatic conditions (Widdowson, 2009). It comprises an important subset of a wider range of ferruginous and related aluminous weathering products (Widdowson, 2009). Lateritic soils according to Bello and Adegoke (2010) can be categorized into laterite, lateritic, and non-lateritic soils, but stressed that such definition may not be convenient from an engineering point of view, especially where there is lack of adequate laboratory facilities, because mode of differentiation is based on silica (SiO_2) to sesquioxide (Fe_2O_3, Al_2O_3) ratios, less than 1.33 are indicative of laterites, those between 1.33 and 2.00 show laterite and those greater than 2.00 are of non- lateritic types (Bell, 1993). Soil materials that are used for earthworks foundation of road pavements construction play an important role in ensuring stability, durability and usability of such roads, (Akinwumi, *et al.*, 2019). Suitable natural soils are, however, getting fast depleted; making some road pavement designers and constructors favour the Stabilisation of in situ soils with poor engineering properties so as to make such poor soils good for construction, over their replacement with suitable materials in order to reduce road construction costs. A soil can be said to be unsuitable for use as earthworks material if it: is difficult to work with (that is, it has a high plasticity), has low strength, has a tendency to retain moisture and high natural moisture content (Onyelowe, 2011).

Soil Stabilisation is the process of improving the shear strength parameters of soil and thus increasing its bearing capacity Ekeng, Bejor and Ibiang (2016). It is required when the soil available for construction is not suitable to carry the structural load. Soil Stabilisation is also defined as the modification or preservation of one or more soil properties to improve the engineering characteristics and performance, Afrin (2017). One may achieve Stabilisation by mechanically mixing the natural soil and stabilizing material together so as to achieve a homogeneous mixture or by adding stabilizing material to an undisturbed soil deposit and obtaining interaction by letting it permeate through the soil voids (Afrin, 2017). Chemical Stabilisation is achieved by mixing the soils with additives such as calcium chloride, Portland cement, lime, and carbide waste. This study focuses on chemical Stabilisation using calcium chloride and carbide waste ash as additives. Common chemical stabilizing agents are additives used to initiate reactions that help improve the bearing capacity of soils (Onyelowe**,** 2012). The most common available stabilizing agents according to; Ingles and Metcalf (1972) include cement, lime and bitumen. The word “lime” is a term that has different meaning to different professionals. For a highway engineer the term refers to either quicklime or hydrated lime and not limestone or agricultural lime. Lime according to Jawad, *et al.*, (2014) is probably the most routinely used traditional stabilizing agent in the treatment of soil for use in pavement construction.

Though the benefits to be derived from the use of lime in soil Stabilisation are enormous, these cannot be achieved in Nigeria, due to the high cost of lime and nonavailability of hydrated lime (Joel and Edeh, 2016). The cost of hydrated lime is approximately twice the cost of cement.

However large quantity of calcium carbide residue is generated in large quantities in different urban and semi urban centres, (Jaturapitakkul, 2013). Calcium carbide residue is a lime by-product obtained from acetylene gas (C_2H_2) production process. Calcium Carbide Waste (CCW) is a by-product of the acetylene production process that contains mainly calcium hydroxide $Ca(OH)_2$. Compared to hydrated lime, CCW has similar chemical and mineralogical compositions, (Joel and Edeh, 2016). The $Ca(OH)_2$ contents are approximately 96.5% and 76.7% for hydrated lime and CCW respectively, CaO contents are 90.13% and 70.78% for the hydrated lime and the CCW, respectively. The high $Ca(OH)_2$ and CaO contents of CCW indicates that it can react with pozzolanic material such as CSA and produce a cementitious materials, (Sharan, *et al.,* 2018). The production of CCW is best described by the following equation:

$$CaS_2 + 2H_2O \rightarrow C_2H_2 + Ca(OH)_2 \qquad (1.1)$$

Source: Sharan, Ullas and Yathiraju (2018).

1.2 Statement of the Research Problem

The characteristic of clay soils is that they swell in volume when they get wet and shrink in volume as they dry, thus making them undesirable for most Civil Engineering construction work. This expansive nature of clay soil sometimes exists to a great depth and stretches along road pavements. This impairs the quality and durability of road pavements constructed on them. Unfortunately in Nigerian most contractors do not consider the need to modify or treat those soils before being used as sub-grade materials and this results in about 90% of Nigerian roads failure, (Onyelowe *et al.,* 2019; Onyelowe, 2012). With the foregoing, Civil Engineers in Nigeria have a role to play to continue to be relevant in this field and if they hope to change the dashing hope and expectations of Nigerians in the aspect of providing affordable, efficient, reliable and durable road pavement (Onyelowe, 2011).

1.3 Aim and Objectives of the Study

1.3.1 Aim of the study

The aim of this study is to investigate stabilisation of lateritic soil with calcium carbide waste for pavement application.

1.3.2 Objectives of the Study

The objectives of the study are to:

i. determine the engineering properties of stabilized lateritic soil.
ii. examine the chemical composition of lateritic soil.
iii. determine the effect of Calcium carbide waste on the compaction characteristics of the lateritic soil
iv. examine the effect of calcium carbide waste on the CBR of lateritic soil

1.4 Justification of the Study

This study focused on the cause of the frequent road pavement failure. According to many literatures; material types, soil type and combination could be responsible for how long a road pavement will last. It is therefore important to determine impact of Calcium carbide waste as a stabilizing agent to be used in the Stabilisation of lateritic soil used during of Ajaokuta-Lokoja road pavement in an attempt to improve the properties of the failing road pavement for a better performance. Various soil modification or Stabilisation guidelines are used to modify or improve the quality of soil. It is necessary for designers to take into consideration the local economic factors as well as environmental conditions and object available at such locations for design to modify such soil. This study therefore, attempts to examine the Stabilisation of Ajaokuta-Lokoja laterite with calcium carbide residue waste for use as a pavement material

1.5 Scope of the Study

The study will focused on the Stabilisation of laterite using calcium carbide waste to improve its engineering properties so as to make it suitable for pavement construction. Soil sample were collected along Ajaokuta-Lokoja road and tested to determine; the index properties of the natural soil, the chemical composition of the soil and calcium carbide waste, the compaction characteristics of the soil with calcium carbide waste mixture, unconfined compressive strength and the CBR of the soil with Calcium carbide mixtures.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Transportation System

Transportation is a requirement for every nation, regardless of its industrial capacity, population size or technological development. Transportation can be defined as the movement of goods, persons and animals from one point to another by various means. Moving goods and people from one place to another is critical to maintain strong economic and political ties between regions in the same state. With a land area of 910,768 sq. km, population estimate of 150 million people and GDP-growth rate of 6% per annum, the centrality of effective public transportation in Nigeria is readily seen. Nigeria„s transportation infrastructure is in a dismal state and falls short of the countries it would like to be compared with. It is insufficient to meet the transformation agenda of the current administration, as a tool for achieving rapid economic growth and development (Walker, 1959). Globally Nigeria ranks low in the quality of its infrastructure which impacts the ease of doing business. Low investments in transportation have resulted in the current infrastructural deficit. Key challenges include inadequate investment and poor management of transport infrastructure which have created a huge infrastructural deficit. The Nigerian transport systems, right from inception, were poorly designed and are unable to scale up to meet gr eater demand, a design flaw which causes traffic congestion on roads, overstressed railways, faltering airfields, and mass-transport blind spots (Igwe *et al.,* 2013).

Transportation system is divided into five basic modes which include Road transportation, Railway transportation, water transportation, air transportation and pipeline transportation.

2.2 Modes of transportation

They basic modes of transportation include, these are Road Transportation, rail transportation, water transportation, air transportation and Pipeline transport**.**

i) Road transport

Road transport exists in all parts of the world; this involves the use of motor vehicles (cars, lorries, buses, bicycles, and trucks). There are various types of roads according to size and functions, some roads are tarred while others are not. The best of these roads are the modern roads which link major towns. Road transport, when compared with other modes of transportation, is more flexible. It is relatively cheaper and faster. Road transport has a high capacity for carrying goods over short distances. Maintenance is one of the major disadvantages of this mode of transport.

ii) Railway transport

Railways were developed during the period of the industrial revolution in the 19th century; these were partly for political reasons and for economic reasons. In many countries, they were built especially to penetrate isolated regions and help promote political unity. The major advantage of railway transport includes the provision of reliable services. It has the ability to convey heavy and bulky goods; it is also very cheap, safe, and also comfortable for passengers over a long distance.

iii) Water transport

Water transport is very important because it is the cheapest way of transporting bulky goods over a long distance. In the world, there are two major types of water transport namely: Inland water transport and ocean water transport.

a) Inland water transport: This is the system of transport through all navigable rivers, lakes, and man-made canals. Many large rivers in different parts of the world are used by ships and barges for transportation; the main rivers where inland water transport are important are the Rhine and Dambe in Europe, Zaire in Africa, the Nile in Africa, the Mississippi in the USA.

b) Ocean Waterways: However, Ocean waterways carry a lot of the world"s trade, the majority of the bulky goods, materials, and passengers pass through ocean waterways from one country to another at the cheapest cost.

iv) Air transport

Air transport is the newest means of transport; it was introduced in 1903 but developed into full means of transporting people and goods in the 1930s. The greatest air transportation started after the Second World War (WWII). This mode of transportation can be used for both domestic and international flights.

v) Pipeline transport

This system of transportation involves the use of hollow pipes in the transportation of water, crude oil, (petroleum) and gas. This mode of transportation is safer than using tankers or trailers in the transportation of these liquids.

For the purpose of this study, road transportation is considered.

2.3 Road Classification in Nigeria

Transportation through roads are the most prone to accident when compared to other modes of transport. Road Transportation involve the movements of goods and commuters from one place to another with the use of the road or land, means of transportation like cars, motorcycles, bicycles, lorries, buses and trucks all utilize the road as their mode of transport.

Generally, roads can be put into 3 types. They are:

i. Trunk A road
ii. Trunk B Road and
iii. Trunk C road.

Trunk A Road: Trunk Road "A" forms the skeleton of the national road grid. It cuts across regional boundaries in the country. This class of road is constructed, maintained and financed by the Federal Government through the Federal Ministry of Works and Housing. These are roads constructed by the Federal Government. An example is the Lagos-Ibadan road in Nigeria.

Trunk B Road: Trunk Road "B" is the second category of main roads in Nigeria. It links the major cities within States. These roads are financed by the State governments. An example is the Lasu-Igando road in Lagos.

Trunk C Road: They are minor roads maintained by the various local governments. Generally, road transportation is built on Pavement, which bears the applied load from the different means of road transportation.

2.4 Road Pavement

Pavement, in civil engineering is a durable surfacing of a road, airstrip, or similar area that primarily transmits loads to the sub-base and underlying soil. The wearing surface of a rigid pavement is usually constructed of Portland cement concrete such that it acts like a beam over any irregularities in the underlying supporting material. The wearing surface of flexible pavements, on the other hand, is usually constructed of bituminous materials such that they remain in contact with the underlying material even when minor irregularities occur (Nicholas *et al.*, 1999). Generally, flexible pavements are constructed of a bituminous surface under-laid with a layer of granular material and a layer of fine materials. However, rigid pavements consist of Portland cement concrete and may or may not have a base course between the subgrade and the concrete surface. Road pavements are divided into three main categories, these are

i. Rigid pavement
ii. Composite pavement and
iii. Flexible pavement

2.4.1 Rigid pavement

A rigid pavement structure is composed of a hydraulic cement concrete surface course and underlying base and sub base courses (if used). Another term commonly used is Portland cement concrete (PCC) pavement, although with today‟s pozzolanic additives, cements may no longer be technically classified as "Portland". The surface course is the stiffest layer and provides the majority of strength. The base or sub base layers are orders of magnitude less stiff than the PCC surface but still make important contributions to pavement drainage and frost protection and provide a working platform for construction equipment. Rigid pavements are substantially „stiffer" than flexible pavements due to the high modulus of elasticity of the PCC material, resulting in very low deflections under loading. The rigid pavements can be analyzed by the plate theory. Rigid pavements can have reinforcing steel, which is generally used to handle thermal stresses to reduce or eliminate joints and maintain tight crack widths, as shown in Figure 2.1.

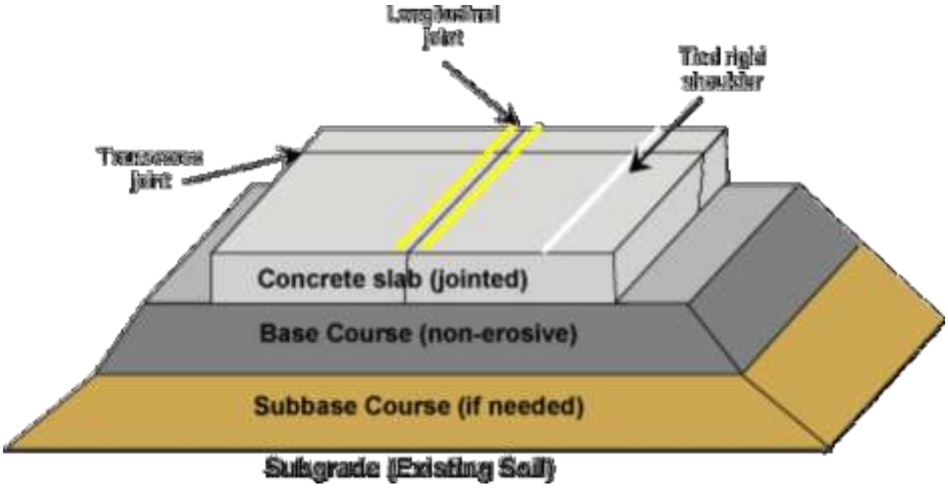

Figure 2.1: Rigid Pavement Cross Section

Source: (Chava *et al.*, 2016)

2.4.2 Composite pavement

A composite pavement is composed of both hot mix asphalt (HMA) and hydraulic cement concrete. Typically, composite pavements are asphalt overlays on top of concrete pavements. The HMA overlay may have been placed as the final stage of initial construction, or as part of a rehabilitation or safety treatment. Composite pavement behavior under traffic loading is essentially the same as rigid pavement.

For the focus of this study, flexible pavement will be considered.

2.4.3 Flexible pavement

Flexible pavement structure is typically composed of several layers of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. Flexible pavements can be analyzed as a multi-layer system under loading. A typical flexible pavement structure consists of the surface course and underlying base and sub base courses. Each of these layers contributes to structural support and drainage. When hot mix asphalt (HMA) is used as the surface course, it is the stiffest and may contribute the most to pavement strength, which is depending on the thickness. The underlying layers are less stiff, but are still important to pavement strength as well as drainage and frost protection. When a seal coat is used as the surface course, the base generally is the layer that contributes most to the structural stiffness. A typical structural design results in a series of layers that gradually decrease in material quality with depth.

2.4.3.1 Structural components of a flexible pavement

Flexible pavements consist of a subgrade (prepared roadbed), the sub-base, the base and the wearing surface. This latter, when made of Hot Mix Asphalt becomes stiffer and contribute more to the pavement strength. The performance of the pavement depends on the satisfactory performance of each component, as shown in Figure 2.2.

Figure 2.2: Schematic of a Flexible Pavement

(Source: Chava *et.. al.,* 2016)

i) Subgrade

The subgrade is the natural material located along the horizontal alignment of the pavement (It serves as the foundation of the pavement structure). Depending on the type of pavement being constructed, it is necessary to treat the subgrade material to achieve the required the strength properties.

ii) Sub-base course:

Located immediately above the subgrade, the sub-base component consists of material of a superior quality to that which generally is used for subgrade construction. When the quality of the subgrade material meets the requirements of the sub-base material, the sub-base component may be omitted (Nicholas *et al.,* 1999). When the sub-base material does not correspond to the requirements, a process of treating soils to improve their engineering properties known as Stabilisation can be used. In fact, the available material should be treated with other materials to achieve the necessary properties.

iii) Base course:

The base course is placed above the sub-base (above the subgrade if the sub-base course is not used). It consists of granular materials such as sand, crushed stone, crushed or uncrushed gravel and crushed or uncrushed slag. Usually, the base course materials include stricter requirements than those for sub-base course. In some cases, to increase the stiffness characteristics of heavy-duty pavements, the base course can be treated with asphalt or Portland cement.

iv) Surface course:

The surface course is the upper layer of the pavement section located immediately above the base course. The surface course in flexible pavements consists generally of a mixture of mineral aggregates and asphaltic materials. It must be able to withstand a wide variety of factors that can accelerate the deterioration process of the pavement.

2.4.3.2 Flexible pavement materials

According to Agbede *et al*., (2015) the following materials are basically used for road flexible road pavement construction;

i. Bitumen,
ii. Granite and
iii. Soil/Laterite.

For the purpose of this study, the soil is considered

Soil: Soil is an accumulation or deposit of earth material, derived naturally from the disintegration of rocks or decay of vegetation that can be excavated readily with power equipment in the field or disintegrated by gentle mechanical means in the laboratory. The supporting soil beneath pavement and its special under courses is called sub grade. For a soil to be used as a pavement it must possess some desirable properties, the desirable properties of sub grade soil as a highway material are:

i. Stability
ii. Incompressibility
iii. Permanency of strength
iv. Minimum changes in volume and stability under adverse conditions of weather and ground water
v. Good drainage, and
vi. Ease of compaction

2.5 Properties of Soil

All soils contain mineral particles, organic matter, water and air. The combinations of these determine the soil‟s properties – its texture, structure, porosity, chemistry and colour.

2.5.1 Physical properties of soil

The physical properties of soil to be discussed include texture, soil structure and porosity.

i. Texture

Soil is made up of different-sized particles. Soil texture refers to the size of the particles that make up the soil and depends on the proportion of sand, silt and clay-sized particles and organic matter in the soil. Sandy soils feel gritty when rubbed between your fingers. Silts feel smooth like flour. Most clay are sticky and mouldable. If you‟ve ever used pottery clay, you‟ll know the feeling.

ii. Soil structure

Soil structure describes the way the sand, silt and clay particles are clumped together. Organic matter (decaying plants and animals) and soil organisms like earthworms and bacteria influence soil structure. Clays, organic matter and materials excreted by soil organisms bind the soil particles together to form aggregates. Soil structure is important for plant growth, regulating the movement of air and water, influencing root development and affecting nutrient availability. Good quality soils are friable (crumbly) and have fine aggregates so the soil breaks up easily if you squeeze it. Poor soil structure has coarse, very firm clods or no structure at all.

iii. Soil porosity

Soil porosity refers to the pores within the soil particles; porosity influences the movement of air and water. Healthy soils have many pores between and within the aggregates, poor quality soils have few visible pores, cracks or holes. The way in which a soil is managed can affect its porosity. For example, look at areas around your school where students regularly walk. If the grass is worn away and the soil is exposed, it often looks different because it has been compacted and has had its structure and porosity altered. These are also areas where puddles form because the water is not able to drain away.

iv. Soil colour

Soil colours range from black to red to white; sometimes it can even be blue, brown or red. Soil colour mostly comes from organic matter and iron. Topsoil is often dark because of organic matter. An even, single colour indicates the soil is well drained. In contrast, rusty spots and grey patches (sometimes even a light blue in colour) indicate poor drainage.

2.5.2 Geotechnical properties of soils

Different geotechnical property of soils has different influence on the civil engineering structures. These various properties dependent upon each other, the properties are discussed as follow:

i. Specific gravity

Specific gravity is the ratio of the mass of soil solids to the mass of an equal volume of water. It is an important index property of soils that is closely linked with mineralogy or chemical composition (Oyediran and Durojaiye, 2011) and also reflects the history of weathering (Tuncer and Lohnes, 1977). It is relatively important as far as the qualitative behaviour of the soil is concerned (Raj, 2012) and useful in soil mineral classification, for example iron minerals have a larger value of specific gravity than silica"s (Bowles, 2012). It gives an idea about suitability of the soil as a construction material; higher value of specific gravity gives more strength for roads and foundations. It is also used in calculation of void ratio, porosity, degree of saturation and other soil parameters (Prakash and Jain, 2002)

Based on the study, Roy and Dass (2014) found that increase in specific gravity can increase the shear strength parameters (cohesion and angle of shearing resistance). Roy, (2016) observed that increase in specific gravity also increases the California bearing ratio i.e. strength of the subgrade materials used in road construction.

ii. Density index

The degree of compaction of fine grained soils is measured in relation to maximum dry density for a certain compactive effort, like 90% of light compaction density or proctor density. But in the case of coarse grained soils, a different sort of index is used for compaction. Depending upon the shape, size, and gradation of soil grains, coarse grained soils can remain in two extreme states of compaction, namely in the loosest and densest states. Any intermediate state of compaction can be compared to these two extreme states using an index called relative density or density index.

Density index is expressed in percent and is defined as the ratio of the difference between the void ratio of a Cohesionless soil in the loosest state and any given void ratio to the difference between its void ratios in the loosest and the densest states (IS:2720, 1983). It is a measure of the degree of compactness, and the stability of a stratum (Raj, 2012). In real sense, it expresses the ratio of actual decrease in volume of voids in a sandy soil to the maximum possible decrease in volume of voids i.e. how far the sand under investigation can capable to the further densification beyond its natural state. Its determination is helpful in compaction of coarse grained soils and in evaluating safe bearing capacity of sandy soils.

iii. Consistency limits

The consistency of a fine-grained soil is largely influenced by the water content of the soil. A gradual decrease in water content of a fine-grained soil slurry causes the soil to pass from the liquid state to a plastic state, from the plastic state to a semi-solid state, and finally to the solid state. The water contents at these changes of state are different for different soils. The water contents that correspond to these changes of state are called the Atterberg limits. The water contents corresponding to transition from one state to the next are known as the liquid limit, the plastic limit and the shrinkage limit (Kaniraj, 1988). The liquid limit of a soil is the water content, expressed as percentage of the weight of the oven dried soil, at the boundary between the liquid and plastic states of consistency of the soil (IS: 2720, 1970). The soil has negligibly small shear strength (Kaniraj, 1988). The plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil, at the boundary between the plastic and semi-solid states of consistency of the soil (IS: 2720, 1970).

The plastic limit for different soils has a narrow range of numerical values. Sand has no plastic stage, but very fine sand exhibits slight plasticity. The plastic limit is an important soil property. Earth roads are easily usable at this water content. Excavation work and agricultural cultivation can be carried out with the least effort with soils at the plastic limit. Soil is said to be in the plastic range when it possesses water content in between liquid limit and plastic limit. The range of the plastic state is given by the difference between liquid limit and plastic limit and is defined as the plasticity index. The plasticity index is used in soil classification and in various correlations with other soil properties as a basic soil characteristic (Raj, 2012). Based on the plasticity index, the soils were classified by Atterberg, shows the correlations between the plasticity index, soil type, degree of plasticity and degree of cohesiveness. Laskar and Pal (2012) found that plasticity depends on grain size of soil, with the increase of sand content plasticity index of soil decreases, which might be due to decrease of inter molecular attraction force. Due to decrease of attraction force, liquid limit of the soil decreases and accordingly plasticity index decreases. But as the clay content increases inter molecular attraction force increases and liquid limit increases.

The shrinkage limit is the maximum water content expressed as a percentage of oven-dried weight at which any further reduction in water content will not cause a decrease in volume of the soil mass, the soil mass being prepared initially from remoulded soil

(IS: 2720, 1972). The finer the particles of the soil, the greater are the amount of shrinkage. Soils that contain montmorillonite clay mineral shrink more. Such soils shrink heterogeneously during summer, as a result of which cracks develop on the surface. Further, these soils imbibe more and more water during the monsoon and swell. Soils that shrink and swell are categorized as expansive soils. Indian black cotton soils belong to this group (Raj, 2012). According to Prakash and Jain (2002), the value of shrinkage limit is used for understanding the swelling and shrinkage properties of cohesive soils. It is used for calculating the shrinkage factors which helps in the design problems of the structures made of the soils or/and resting on soil. It gives an idea about the suitability of the soil as a construction material in foundations, roads, embankments and dams. It helps in knowing the state of given soil.

According to Ersoy *et al*. (2013) consistency is an important property and is a useful measure for the processing of very fine clayey soils. Plasticity and cohesion reflect the soil consistency and workability of the soils. However, these properties of the soils play an essential role in many engineering projects, such as the construction of the clay core in an earth fill dam, the construction of a layer of low permeability covering a deposit of polluted material, the design of foundations, retaining walls and slab bridges, and determining the stability of the soil on a slope. Agbede *et al*., (2015) conducted the study at University of Ibadan (UI), Nigeria. The building under study was two storey with basement complex and housed offices, classrooms, a laboratory, library and a computer room. This building is located in a flat, low terrain with an upper layer of loose lateritic clayey soils while the underlying soil is sandy soil mixed with silty clay material. The cracks were observed due to expansive soil supporting the foundation of the building. The soil foundation contains high amount of clay with high plasticity index.

iv. Particle size analysis

The percentage of different sizes of soil particles coarser than 75 µ is determined by sieve analysis whereas less than 75 µ are determined by hydrometer analysis. Based on the particle size analysis, particle size distribution curves are plotted. The particle size distribution curve (gradation curve) represents the distribution of particles of different sizes in the soil mass (Mallo and Umbugadu, 2012). It gives an idea regarding the gradation of the soil i.e. it is possible to identify whether a soil is well graded or poorly graded. In mechanical soil Stabilisation, the main principle is to mix a few selected soils in such a proportion that a desired grain size distribution is obtained for the design mix. Hence for proportioning the selected soils, the grain size distribution of each soil is required to be known. Apparao and Rao (1955) explained that the grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air fields, and so on. Raj, (2012) stated that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity, and frost susceptibility. Very relevant and useful information may be obtained from grain size curve such as (i) the total percentage of larger or finer particles than a given size and (ii) the uniformity or the range in grain-size distribution.

Bowles (2012) found that particle-size is one of the suitability criteria of soils for roads, airfield, levee, dam, and other embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used. The susceptibility to frost action in soil, an extremely important consideration in colder climates, can be predicted from the particle-size analysis. Very fine soil particles are easily carried in suspension by percolating soil water, and under drainage systems are rapidly filled with sediments unless they are properly surrounded by a filter made of appropriately graded granular materials. The proper gradation of this filter material can be predicted from the particle-size analysis. Particle-size of the filter materials must be larger than the soil being protected so that the filter pores could permit passage of water but collect the smaller soil particles from suspension.

Research by Dafalla (2013), showed the sand shape whether rounded, sub-rounded, or angular will affect the shearing strength of soil. Angular grains provide more interlock and increased shear resistance. The gradation and size of the sand affect the shear resistance. Well-graded materials provide more grain to grain area contact than poorly graded materials. Porosity and spaces available for clay within the sand is an important while considering the mixtures of clays and sands.

v. Compaction

Soil compaction is one of the ground improvement techniques. It is a process in which by expending compactive energy on soil, the soil grains are more closely rearranged. Compaction increases the shear strength of soil and reduces its compressibility and permeability (Oke and Amadi, 2008; Murthy, 2002) explained that when an earth dam is properly compacted, the shear strength of the material is increased and dam becomes more stable. Since the soil becomes dense, its permeability gets decreased. The decrease in the permeability of the dam decreases the seepage loss of the water stored.

The settlement of the dam also decreases due to the increase in the density of the materials.

According to Prakash and Jain (2002), compaction of soils increases the density, shear strength, bearing capacity but reduces their void ratio, porosity, permeability and settlements. The results are useful in the stability of field problems like earthen dams, embankments, roads and airfields. The moisture content at which the soils are compacted in the field is controlled by the value of optimum moisture content determined by the laboratory proctor compaction test. The compaction energy applied in the field is also controlled by the maximum dry density determined in the laboratory.

Durgunoglu *et al*., (2003), used heavy dynamic compaction method for the compaction of foundation subsoil of Carrefoursa Hypermarket and Trade Center in Bursa, Turkey. In order to increase the bearing capacity of the foundations sub soils as well as to control the total and differential settlements underneath the foundations.

vi. Consolidation

When a soil layer is subjected to compressive stress due to construction activities, it undergoes compression. The compression is caused by rearrangement of particles, seepage of water, crushing of particles, and elastic distortions. Settlement of a structure is analyzed for three reasons: appearance of structure, utility of the structure, and damage to the structure. The aesthetic view of a structure can be spoiled due to the presence of cracks or tilt of the structure caused by settlement. Settlement caused to a structure can damage some of the utilities like cranes, drains, pumps, electrical lines etc. Further settlement can cause a structure to fail structurally and collapse. Settlement is the combination of time-independent (immediate compression) and time-dependent compression, called consolidation. (Raj, 2012).

According to Prakash and Jain (2002), the main aim of a consolidation test is to obtain soil data which are used in predicting the rate and amount of settlement of structure founded on clay primarily due to volume change of the clay. The information obtained for foundations resting on clay are: (i) total settlement of foundation under any given load, (ii) time required for total settlement due to primary consolidation, (iii) settlement for any given time and load and (iv) time required for any percentage of total settlement or consolidation,

Abeele (1985) explained that lowering of water table or dewatering is probably the best known cause of massive settlement. When submerged, soil particles are subjected to buoyancy. Upon dewatering, the buoyancy is removed and the apparent increase in pressure results in consolidation, even though there is no increase in external load. Vibrations can also have a densification effect on soils and lead to subsequent settlement. The effects can be severe when the vibration frequency matches the soil's natural frequency. Soils often fail and settle disastrously as a result of earthquakes. Devastating landslides are often one of the results of such occurrences. Of the three phases of soil, only the solid phase controls the resistance to compression and shear. Water, present in a moist soil is highly incompressible but as a liquid, is not capable of resisting shear loads. Air, present in unsaturated soils, will not support compression or shear loads.

Naik *et al*. (2011) carried out settlement study for an Institutional Building located in South Goa, India, which developed cracks when the construction had reached till the plinth beam level. It was found that some foundations were located above the natural ground at a depth of 2 m in unconsolidated filled up ground of an abandoned laterite stone quarry, where SPT (Standard penetration Test) was found to be less than 12, which resulted for differential settlement. This differential settlement was observed towards the front left corner of the Building which was lying on the filled up ground. The differential settlement led to cracks in the plinth beam and Foundation Concrete.

vii. Permeability

The amount, distribution, and movement of water in soil have an important role on the properties and behaviour of soil. The engineer should know the principles of fluid flow, as groundwater conditions are frequently encountered on construction projects. Water pressure is always measured relative to atmospheric pressure, and water table is the level at which the pressure is atmospheric. Soil mass is divided into two zones with respect to the water table: (i) below the water table (a saturated zone with 100% degree of saturation) and (ii) just above the water table (called the capillary zone with degree of saturation ≤ 100%) (Raj, 2012). Data from field permeability tests are needed in the design of various civil engineering works, such as cut-off wall design of earth dams, to ascertain the pumping capacity for dewatering excavations and to obtain aquifer constants (Raj, 2012). The permeability of soils has a decisive effect on the stability of foundations, seepage loss through embankments of reservoirs, drainage of subgrades, excavation of open cuts in water bearing sand, and rate of flow of water into wells (Murthy, 2002).

Prakash and Jain (2002) explained that water flowing through soil exerts considerable seepage forces, which have direct effect on the safety of hydraulic structures. The rate of settlement of compressible clay layer under load depends on its permeability. The quantity of stored water escaping through and beneath an earthen dam depends on the permeability of the embankment and the foundation respectively. The rate of drainage of water through wells and excavated foundation pits depends on the coefficient of permeability of the soils. Shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure is controlled by its permeability.

viii. Shear strength

The shear resistance of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. The shear strength parameters of soils are defined as cohesion and the internal angle friction. The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain, and direction of the strain. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, fissures and cementation (Poulos, 1989). The shear strength parameters of a granular soil are directly correlated to the maximum particle size, the coefficient of uniformity, the density, the applied normal stress, and the gravel and fines content of the sample. It can be said that the shear strength parameters are a result of the frictional forces of the particles, as they slide and interlock during shearing (Yagiz, 2001).

Different researchers (Prakash and Jain, 2002; Raj, 2012) explained that the capability of a soil to support a loading from a structure, or to support its overburden, or to sustain a slope in equilibrium is governed by its shear strength. The shear strength of a soil is of prime importance for foundation design, earth and rock fill dam design, highway and airfield design, stability of slopes and cuts, and lateral earth pressure problems. It is highly complex because of various factors involved in it such as the heterogeneous nature of the soil, the water table location, the drainage facility, the type and nature of construction, the stress history, time, chemical action, or environmental conditions.

Research carried out by Prakash and Jain (2002), showed that, confining pressures play the significant role in changing the behaviour of soils in deep foundations. Similarly in high rise earth dams, the confining pressures are of very high magnitude. Triaxial test is the only test to simulate these confining pressures. For short term stability of foundations, dams and slopes, shear strength parameters for unconsolidated undrained or consolidated undrained conditions are used; while for long term stability shear parameters corresponding to consolidated drained conditions give more reliable results.

Akayuli, *et al.,* (2013) found that the friction angle is high for a sandy soil than its cohesion and vice versa for clayey soil. Shanyoug *et al.,* (2009) in their study concluded that there is a general increase in cohesion with clay content. As more clay is introduced into the sandy materials, the clay particles fill the void spaces in between the sand particles and begin to induce the sand with interlocking behaviour. Hence, clayey sand soils are expected to exhibit low cohesion whereas the cohesion increases with high clay content.

Dafalla (2013) observed that the mineralogy can have a major role in the shearing strength capacity of clays. The cementation between particles can either be due to a chemical bond or physicochemical bond. Swelling and shrinkage in expansive soils are of two extreme opposite effects on the shearing strength. The shear strength is generally low for fully expanded clay while dry shrinking clay is capable of developing higher cohesion and angle of internal friction. The study indicated that choosing the appropriate mix or using appropriate quantity of clay, can help to achieve required shear strength. Very moist clay-sand mixture showed steep drop in both cohesion and angle of internal friction when the clay content is high.

According to Murthy (2002) and El-Maksoud (2006), cohesion is mainly due to the intermolecular bond between the adsorbed water surrounding each grain, especially in fine-grained soils. According to Mollahasani *et al*. (2011), the soils with high plasticity like clayey soils have higher cohesion and lower internal angle of shearing resistance. Conversely, as the soil grain size increases like sands, the soil cohesion decreases.

Soil that is deficit of the above listed properties are termed as poor soil, which must be treated before it can be used as a pavement subgrade material.

2.6. Expansive Soil

Expansive soils are highly problematic as they have a tendency to increase in volume on absorption of water and to shrink on drying (Lee and Ian, 2015). On absorption of water the density of these soils decrease and they become slushy. They become hard on drying due to increase in density (Phanikumar *et al.,* 2009). The volumetric change in these soils is attributed to seasonal variations in the ground water profile resulting in changes in the moisture content (Rees and Thomas, 1993).

Expansive or swelling soil is a highly plastic soil that normally contains montmorillonite and other active clay minerals. Expansive soil is a commonly identified problem which has made scientists concerned about the design, protection, and operating of highway and structural systems. Expansive soils can be found in arid/semi-arid areas, where even moderate expansive soils can cause major damages to the structure or in humid environments where just expansive soils with high plasticity index (PL) can lead the structure to be damaged. The behaviours of an expansive soil can be affected by many factors, among which the principal ones are the availability of moisture, and the amount and type of the clay-size particles in the soil. It is worth mentioning that when the water changes in expansive soil, the volume would be changed as well. These volume changes can lead to either swelling or shrinkage and that is why expansive soils are also known as swell/shrink soils (Ardani, 1992; Day, 2000; Zemenu *et al.,* 2009; Ito and Azam, 2010; Liu, *et al.,* 2015). Although expansion can be the result of the chemically induced changes, most of the times soils which have swelling and shrinkage behaviour contain expansive clay minerals. It can be resulted in the fact that, the more the clay exists in the soil, the higher the soil swells and the more water the soil can absorb. In addition, the more water they absorb, the more their volume increases. In other words, when the water is attracted by these types of soils, they would get increased in volume and hence they would swell. In contrast, the shrinkage happens when the soil gets dry. Researchers have reported the safe expansion percentage equal or smaller than 10% for most of the expansive clays (Jones and Jefferson, 2012).

Desiccated cracks of expansive soil deposits in dry season is commonly seen , cracks measuring 70 mm wide and over 1 m deep have been observed and may extend up to 3m or more in case of high deposits (Jones, and Jefferson, 2012). Heaving ground due to swelling of expansive soil in rainy season or due to any cause of moisture alteration in this soil mass could also be observed. Hence undesired engineering properties of expansive soil:-weakness of the foundation soil from volumetric instability, swelling pressures development due to swelling ground is responsible for many damages of engineering structures.

2.7 Stabilisation of Soils

Soil Stabilisation is the process of altering some soil properties by different methods, mechanical or chemical in order to produce an improved soil material which has all the desired engineering properties, also Soil Stabilisation is referred to making major improvements to the engineering properties of soils by amending the natural soil characteristics with an additive. These additives may include other soils or materials such as Portland cement, lime, fly ash, asphalt cement, polymers, and fibres (The History of Air Force Civil Engineer, 2012). Soils are generally stabilized to increase their strength and durability or to prevent erosion and dust formation in soils. The main aim is the creation of a soil material or system that will hold under the design use conditions and for the designed life of the engineering project. The properties of soil vary a great deal at different places or in certain cases even at one place; the success of soil Stabilisation depends on soil testing. Various methods are employed to stabilize soil and the method should be verified in the laboratory with the soil material before applying it on the field. Traditionally, additives such as bitumen, cement, and lime have achieved widespread use. Bitumen is typically used as a soil surface treatment to limit dust and loss of fines. Cement is used to provide strength to soil. Lime is often used in clay soils to control plasticity.

2.7.1 Purpose of soil stabilisation

There are three purposes for soil Stabilisation (The History of Air Force Civil Engineer, 2012). The first purpose is strength improvement, to enhance its load-bearing capacity. The second purpose is for dust control by binding soil particles together, to eliminate or alleviate dust, generated by the operation of equipment and aircraft during dry weather or in arid climates. The third purpose is soil waterproofing, which is done to preserve the natural or constructed strength of a soil by preventing the entry of surface water.

2.7.2 Types of lime stabilisation

They are basically two broad categories of soil Stabilisation namely mechanical Stabilisation and chemical Stabilisation.

i. Mechanical stabilisation

Mechanical Stabilisation is the process of improving the properties of the soil by changing its gradation. This process includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration techniques and sometime blasting. The stability of the soil in this method relies on the inherent properties of the soil material. Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. Mechanical Stabilisation is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

ii. Chemical stabilisation

Chemical Stabilisation: Chemical Stabilisation of soil comprises of changing the physico-synthetic around and within clay particles where by the earth obliges less water to fulfil the static imbalance. Calcium chloride being hygroscopic and deliquescent is used as a water retentive additive in mechanically stabilized soil bases and surfacing. The vapour pressure gets lowered, surface tension increases and rate of evaporation decreases. The freezing point of pure water gets lowered and it results in prevention or reduction of frost heave. The depressing the electric double layer, the salt reduces the water pick up and thus the loss of strength of fine grained soils. Calcium chloride acts as a soil flocculent and facilitates compaction. Frequent application of calcium chloride may be necessary to make up for the loss of chemical by leaching action. For the salt to be effective, the relative humidity of the atmosphere should be above 30%. Sodium chloride is the other chemical that can be used for this purpose with a stabilizing action similar to that of calcium chloride. Sodium silicate is yet another chemical used for this purpose in combination with other chemicals such as calcium chloride, polymers, chrome lignin, alkyl chlorosilanes, siliconites, amines and quaternary ammonium salts, sodium hexametaphosphate, phosphoric acid combined with a wetting agent, (Rogers and Glendinning 1993).

2.8 Type of Stabilizers

The following are the various types of traditional stabilizers readily used for soil Stabilisation

2.8.1 Lime stabilisation

When Stabilisation of soil is done by mixing soil with lime in proper proportion, the process is known as soil-lime Stabilisation. Lime is an excellent choice for short term modification of soil properties. Lime can modify almost all fine grained soils but the greater improvement occurs in clay soils of moderate to high plasticity (National Lime Association, 2004). Modification occurs because calcium cations supplied by hydrated lime replace the cations normally present on the surface of the clay mineral, promoted by the high pH environment of the lime-water system. Thus, the clay surface mineralogy is altered, producing the following benefits; Plasticity reduction, Reduction in moisture- holding capacity (drying), Swell reduction, Improved stability and Ability to construct a solid working platform. Lime in the form of quicklime (calcium oxide-CaO), hydrated lime (calcium hydroxide- $Ca(OH)_2$), or lime slurry can be used to treat soils. Quicklime is manufactured by chemically transforming calcium carbonate (limestone-$CaCO_3$) into calcium oxide. Hydrated lime is created when quicklime chemically reacts with water. When hydrated lime reacts with clay particles permanently transforms clay into a strong cementitious matrix. (Lockett and Moore, 1981).

2.8.1.1 Lime modification

Modification refers to soil improvement that occurs in the short term, during or shortly after mixing, (Joel and Edeh, 2013.) There are two other important types of lime treatment used in construction operation. Firstly, because quicklime chemically combines with water, it can be used very effectively to dry wet soils. Heat from this reaction help to dry wet soils. Secondly lime treatment can significantly improve soil workability and short term strength to enable projects to be completed more easily (National Lime Association, 2004).

2.8.1.2 Chemistry of lime treatment of soil

When lime and water are added to a clay soil, chemical reactions begin to occur almost immediately in the form of;

i. **Drying**: if quicklime is used, it immediately hydrates (i.e chemically combines with water) and release heat (exothermic reaction). Soils are dried, because water present in the soil participates in this reaction and also because the heat generated can evaporate additional moisture. The hydrated lime produced by these initial reactions will subsequently react with the clay particle

$$CaO_{(s)} \quad + \quad H_2O_{(l)} \quad \rightarrow \quad Ca(OH)_{2(aq)} \quad + \quad Heat \qquad (2.1)$$

The reaction with the clay particles will slowly produce additional drying which reduce the soils moisture holding capacity. If hydrated lime or hydrated lime slurry is used instead of quicklime, drying occurs only through the chemical change in the soil that reduces its capacity to hold water and thus increase it stability (National Lime Association, 2004).

ii. **Modification**: Lime modification described an increase in strength brought by cation exchange capacity rather than cementing effect brought by pozzolanic reactions (Sherwood, 1993). This process is carried out by initially mixing the lime (hydrated lime) with the clay soil, the calcium ion (Ca^{2+}) from the hydrated lime migrate to the surface of the clay particles and displace water and other ions. The soil becomes friable and granular, making it easier to work and compact (National Lime Association, 2004). At this stage the plasticity index (PI) of the soil decreases dramatically as does its tendency to swell and shrink, this process which is called „"flocculation and agglomeration"" generally occurs in a matter of hours. It makes the clay soil dry and less susceptible to water content changes (Rogers and Glendinning., 1993; National Lime Association, 2004).

iii. **Stabilisation**: when adequate quantities of lime and water are added, the soil pH quickly increases to above 10.5, which enables the clay particles to break down; the increase in the pH causes an increase in the exchange capacity (National Lime Association, 2004; Makusa, 2012). Like cement, lime when it reacts with wet clay minerals result into increased pH which favours solubility of siliceous and aluminous compounds, which react with calcium to form calcium silica and calcium alumina hydrates a cementitious product similar to those of cement paste (Makusa, 2012); (National Lime Association, 2004; and Baser, 2009). They form the matrix that contributes to the strength of lime-stabilized soil layers. As this matrix forms, the soil is transformed from a sandy, granular material to a hard relatively impermeable layer with significant load bearing capacity. This process begins for hours and can continue for years in a properly designed system. The matrix formed is permanent, durable and significantly impermeable producing a structural layer that is both strong and flexible (National Lime Association, 2004).

2.8.1.3 Lime stabilisation applications and advantages

It is an age-old practice to use lime in one form or the other to improve the engineering behaviour of clayey soils. Because of the proven success of lime Stabilisation in the field of highways and air-field pavements, this technique is now being extended for deep in-situ treatment of clayey soils to improve their strength and reduce compressibility. Lime Stabilisation is covered extensively in the literature (Rogers and Glendinning, 1993; Quaint *et al.*, 2000; Little *et al.*, 1987; Mitchell, 1986; Armani and Moonfish, 1972; Stocker, 1972; Thompson, 1969). Lime will primarily react with medium, moderately fine, and fine-grained soils to produce decreased elasticity, increased workability, reduced swell, and increased strength.

Such improved soil properties are the result of three basic chemical reactions (Fang, 1991).

Lime has been found to react successfully with medium, moderately fine and fine grained soils causing a decrease in plasticity and swell potential of expansive soils, and an increase in their workability and strength properties (Robert, 2009). The effect of lime on soil can be categorized into two groups; immediate and long-term Stabilisation. Increased workability of soil is the result of immediate improvement which is the main contributor in early construction stages. Increased strength and durability is considered long-term Stabilisation that takes place during and after curing.

Suitability

Lime works best for clayey soils, especially those with moderate to high plasticity index (PI>15). Little *et al.,* (2010) suggested that soils classified by Unified Soil Classification System as CH, CL, MH, SC, SM, GC, SW-SC, SP-SC, SM-SC, GP-GC, and GM-GC can be stabilized by lime treatment. Aggregates with plastic fines, caliche and other marginal bases that contain appreciable amount of material passing #40 sieve are also capable of being stabilized with lime (Little, 1987). Therefore, strengths of soil stabilized with lime must be verified through strength tests such as California Bearing Ratio (CBR), unconfined compressive strength, or resilient modulus. Lime contents between to 10 percent are typically capable of producing significant strength gains (Little *et al.,* 2010).

2.8.2 Calcium carbide

Calcium carbide, also known as calcium acetylide, is a chemical compound with the chemical formula of CaC_2. It is mainly used industrially in the production of acetylene and calcium cyanamide. The pure material is colorless , however pieces of technical-grade calcium carbide are grey or brown and consist of about 80–85% of CaC_2 (the rest is CaO (calcium oxide), Ca_3P_2 (calcium phosphide), CaS (calcium sulfide), Ca_3N_2 (calcium nitride), SiC (silicon carbide), and so on.

In the presence of trace moisture, technical-grade calcium carbide emits an unpleasant odour reminiscent of garlic.

2.8.2.1 Production of calcium carbide

Calcium carbide is produced industrially in an electric arc furnace from a mixture of lime and coke at approximately 2,200 °C (3,990 °F). This method has not changed since its invention in 1892:

$$CaO + 3C \rightarrow CaC_2 + CO \qquad (2.2)$$

The high temperature required for this reaction is not practically achievable by traditional combustion, so the reaction is performed in an electric arc furnace with graphite electrodes. The carbide product produced generally contains around 80% calcium carbide by weight. The carbide is crushed to produce small lumps that can range from a few mm up to 50 mm. The impurities are concentrated in the finer fractions. The CaC_2 content of the product is estimated by measuring the amount of aacetylene produced on hydrolysis. Impurities present in the carbide include phosphide, which produces phosphine when hydrolyzed.

2.8.2.2 Importance calcium carbide

The following are the importance of calcium carbide

i. Production of acetylene

The reaction of calcium carbide with water, producing acetylene and calcium hydroxide, Sharan, *et al.*, (2018).

$$CaC_{2(s)} + 2H_2O_{(aq)} \rightarrow C_2H_{2(g)} + Ca(OH)_{2(aq)} \qquad (2.3)$$

This reaction was the basis of the industrial manufacture of acetylene, and is the major industrial use of calcium carbide. Today acetylene is mainly manufactured by the partial combustion of methane or appears as a side product in the ethylene stream from cracking of hydrocarbons. Approximately 400,000 tones are produced this way annually. In China, acetylene derived from calcium carbide remains a raw material for the chemical industry, in particular for the production of polyvinyl chloride. Locally produced acetylene is more economical than using imported oil (Dun, 2006).

Production of calcium cyanamide

Calcium carbide reacts with nitrogen at high temperature to form calcium cyanamide: (Greenwood and Earnshaw, 1997).

$$CaC_2 + N_2 \rightarrow CaCN_2 + C \qquad (2.4)$$

This is ccommonly known as nitrolime, calcium cyanamide is used as fertilizer. It is hydrolyzed to cyanamide, H_2NCN (Greenwood and Earnshaw, 1997).

ii. Steelmaking

Calcium carbide is used:

a. In the desulfurization of iron (pig iron, cast iron and steel)
b. As a fuel in steelmaking to extend the scrap ratio to liquid iron, depending on economics.
c. As a powerful deoxidizer at ladle treatment facilities.

iii. Carbide lamps

Calcium carbide is used in carbide lamps. Water dripping on carbide produces acetylene gas, which burns and produces light. While these lamps gave steadier and brighter light than candles, they were dangerous in coal mines, where flammable methane gas made them a serious hazard. The presence of flammable gases in coal mines led to miner safety lamps such as the Davy lamp, in which wire gauze reduces the risk of methane ignition. Carbide lamps were still used extensively in slate, copper, and tin mines where methane is not a serious hazard. Most miners' lamps have now been replaced by electric lamps. Carbide lamps are still used for mining in some less wealthy countries.

iv. Other uses

Calcium carbide is sometimes used as source of acetylene gas, which is a ripening agent similar to ethylene. (Abeles and Gahagan, 1968). However, this is illegal in some countries as, in the production of acetylene from calcium carbide, contamination often leads to trace production of phosphine and arsine. These impurities can be removed by passing the acetylene gas through acidified copper sulfate solution, but, in developing countries, this precaution is often neglected. Calcium carbide, together with calcium phosphide, is used in floating, self-igniting naval signal flares.

2.8.2.3 *Impact of carbide waste on clay soil*

The cost of construction is high partly due to the high cost of cement. Any effort at introducing cheaper alternatives or partial replacement of cement will reduce cost. Wastes that have cementitious properties can become a useful source of cheap materials for soil improvement, thereby reducing the cost of construction projects on sites that have unsuitable soils, an alternative is found in Calcium Carbide Waste (CCW) which is a by-product of the flammable acetylene gas production. It is called lime hydrate or carbide lime sludge. Calcium carbide is a chemical compound containing calcium and carbide, with a chemical formula of CaC_2. Pure calcium carbide is colourless, but most of the material is produced industrially. The reaction of calcium carbide with water produces this all important gas that is used in oxygen – acetylene welding, among other uses. The chemical reaction equation is;

$$\underset{\text{Calcium carbide}}{CaC_{2(s)}} + \underset{\text{Water}}{2H_2O_{(l)}} \rightarrow \underset{\text{Hydrated Lime}}{Ca(OH)_{2(aq)}} + \underset{\text{Acetylene}}{C_2H_{2(g)}} \qquad (2.5)$$

The CCW from the production of acetylene gas are normally disposed of by land filling which may create further problem, like, the leaching of harmful compound and Alkali to ground water. CCW is produced from welding shops dotting the urban areas of the country. This by-product of the acetylene gas production is an example of waste that constitutes an environmental nuisance. If converted to a construction material, it will both reduce cost of construction and environmental pollution. Calcium which possesses silica and alumina which makes it a binder makes it efficient to bind loosed and expansive clay. Researches have shown that CCW generally reduced the soil‟s specific gravity, plasticity index and maximum dry unit weight. A direct proportionality was also found between the CCW content and each of the liquid and plastic limits, optimum moisture content, CBR and UCS. Thus, the soil became more workable and its strength properties were improved by Stabilisation with CCR. Consequently, the subgrade characteristics of the soil for use as earthwork materials for road construction was improved. Based on strength properties of the stabilized soil, an optimal application of 4% CCR was found suitable for the Stabilisation of the sand with similar properties as those studied. The use of CCR for stabilizing sand for road construction is recommended as a cheap and sustainable approach for developing countries. Akinwumi *et al.* (2019)

2.8.3 Portland cement

When Stabilisation of soil is done by mixing with cement it is known as soil-cement Stabilisation. Soil-cement is a mixture of pulverized soil and measured amount of cement and water, compacted to the desired density and cured (Liu and Evett, 1998). The role of cement is to improve the engineering properties of available soil such as strength, compressibility, permeability, swelling potential, frost susceptibility and sensitivity to changes in moisture content. Soil cement materials range from semi flexible to semi rigid depending on the type of soil and amount of cement used.

Cement consists of numerous minerals and is manufactured by combining cement clinker with gypsum. Cement mixed with water forms calcium silicate hydrate and calcium hydroxide ($Ca(OH)_2$). Calcium silicate hydrate forms on the surfaces of the cement particles and because it has a strongly cementing effect, it binds the soil together and increases its strength. Since the hydraulic reaction takes place considerably faster than the pozzolanic reaction, cement stabilized soil normally attains higher strength than lime stabilized soil, particularly in the first 26 days.

Suitability

Cement Stabilisation is perfectly suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space of the coarse aggregate particles. Little *et al.,* (1987), suggested that, plasticity index (PI) should be less than 30% for sandy materials, and less than 20% for fine-grained soils with more than 50 percent by weight passing 75µm. The liquid limit (LL) should be less than 40% in order to ensure proper mixing. However, the water-cement ratio is primary factor governing behaviour of cement stabilized soil. The water-cement ratio is defined as the ratio of moisture content of the soil to the cement content, with both the moisture content and cement content expressed in terms of dry weight of soil.

2.8.4 Fly ash

Fly ash is a by-product of coal combustion in power plants. Fly ash contains silica, alumina, and calcium oxides, iron oxide and alkalis in its composition, and is considered as a pozzolanic material Cockrell and Leonard (1970). The most common elemental compositions of fly ash include amorphous oxide (mainly SiO_2, Al_2O_3), and metal oxides i.e. TiO_2, Fe_2O_3, MnO, MgO, CaO, Na_2O, K_2O, P_2O_5, SO_3 and organic carbons. There are two types of fly ash; type "C" and type "F". This classification is based on the chemical composition. Fly ash type "C" contains 10% to 16% amount of free lime (Cockrell and Leonard 1970). This type of fly ash produces pozzolanic and cementitious reactions. Cockrell and Leonard (1970), publicized that, colour is one of the important physical properties of fly ash in terms of estimating the lime content qualitatively. Lighter colour of fly ash indicates the presence of high calcium oxide and darker colours of fly ash represent high organic content. Fly ash can be used to improve the engineering properties of soil. However it must be well-known that fly ash properties are highly variable and depend on chemical composition of coal and combustion technology.

2.8.5 Stabilisation using salt ($NaCl$, $MgCl_2$, $CaCl_2$)

Hassnen, (2013) reported increase in unconfined compressive strength of soil treated with 8% Nacl up to 700kN/m^2 , also results showed that maximum dry density of soil was increased from 1.85-1.92gcm^{-3} with increase of 8% Nacl in soil sample. Soil samples were prepared from commercial clay, River Aire soil, sand, and gravel. The study further showed that addition of salt resulted in increase in resilient modulus. This is potentially useful for long-term highway pavement subgrade applications.

Tamadher *et al.*, (2007), conducted laboratory test to investigate the effect of adding different chloride compounds i.e. ($NaCl$, $MgCl_2$, $CaCl_2$) on the engineering properties of silty clay soil. Various amounts of salts (2, 4, and 8% by weight) were added to the soil to study the effect of salts on the compaction characteristics, consistency limits and compressive strength. Test results showed a maximum dry density increased from 17.5KN/m^3 to 19.0KN/m^3 and decreased the optimum moisture content from 15% to 13%. The liquid limit, plastic limit and plasticity index decreased with the increase in salt content. The unconfined compressive strength increased as the salt content increased.

2.8.6 Stabilisation using polymers

Polymers consist of hydrocarbon chains, and these chains become entwined within the soil particles thus producing a stabilizing effect. In effect, the polymers act as a binder to glue the soil particles together reducing dust, and even stabilizing the entire soil matrix (Orts *et al.*, 2007). Tingle and Santoni (2003) performed unconfined compressive strength testing on lean clay and fat clay treated with various natural and synthetic polymers. For the lean clay, the greatest increase in strength compared to untreated samples was obtained from treatment with lignosulfonate. Treatment with synthetic polymer also showed an increase in strength for the lean clay, although not as great of an increase as encountered with lignosulfonates treatment. For the fat clay, treatment with synthetic polymer also showed increases in strength. Lignosulfonates treatment of the fat clay was not included in the testing program.

Jeb and Rose (2007) also demonstrated that lignosulfonates could be an effective stabilizer. The lignosulfonates was used to treat a soil-aggregate mixture, and then California Bearing Ratio (CBR) tests were performed on compacted samples. Unsoaked specimens showed the greatest increases in CBR value after curing for a week. Soaked specimens still showed an increase in strength after curing for a week, but the strength increase was markedly less than that seen with unsoaked specimens. This phenomenon seems to be linked to the hydrophilic nature of the lignosulfonates, as it will tend to dissolve in water. Testing performed by Kim, Gopalakrishnan and Ceylan (2012) using lignins mixed with Iowa class 10 soil (CL) results indicate that the biofuel products have excellent resistance to moisture degradation.

2.8.7 Stabilisation using molasses

Molasses is the most valuable by-product from the sugar industry. The molasses referred to in this research is blackstrap molasses, which is the product of raw sugar from sugar cane. Blackstrap molasses is the final byproduct of the third boiling cycle in the sugar making process. This type of molasses has a very dark colour and is extremely viscous and contains approximately 20% sucrose, 20% reducing sugar, 10% ash, 20% organic non-sugar, and 20% water (Lewis, 1993). When high additive contents are used (5% plus) gravel loss reduction realized (Phil, 2014). Testing performed by Shirsavkar (2010) verified that molasses can be an effective soil stabilizer. Soil modified with molasses by adding 5%, 5.5% 6.0%, 6.5%, 7.0% and 7.5% to gravel-clay sample, test results show that, value of California Bearing Ratio (CBR) found to increase by 5.12%, 22.67%, 24.68%, 34.00% 23.12% and 22.02%.

Also by adding 6.5% of molasses in soil sample, the value of liquid limit and plastic limit increased while plasticity index of modified soil get reduced. M"Ndegwa, (2011) suggested that Stabilisation of expansive clay soil with molasses increased the California Bearing Ratio (CBR) values and load bearing ability of the soil. Therefore molasses can be used as stabilizing agent for expansive clay soil. Also, molasses mixed with expansive clay soil reduced its swelling tendencies. Therefore, it is clear that laboratory works by other researchers have not highlighted the impact and improvement on permeability, cohesion and internal angle of friction of soil following the addition of molasses during field Stabilisation.

2.8.7.1 Environmental impact of molasses in soil Stabilisation

Food grade molasses do not contain chemicals that might cause site contamination; therefore, it can be used for soil Stabilisation (O"Neill, 2011). While, chemical products from industrial materials and waste products currently used as soil stabilizer contain compound that might be harmful for human being especially when it comes into contact with water (Metzler and Jarvis, 1985). Portland cement is chemical soil stabilizer which is corrosive. When contact with wet or dry material can cause serious, potentially irreversible tissue damage from chemical burns, particularly to the eyes. Eyes contact by larger amounts of wet or dry cement may cause blindness (Canada building material, 2010). Natural products are likely to biodegrade in the environment and therefore toxic effects are expected to be minimal. Organic petroleum products which include used oils, solvents, cutback solvents, asphalt emulsions, dust oils, and tars have higher environmental impacts. Several studies have shown that waste oils may contain known toxic and carcinogenic compounds (Metzler and Jarvis, 1985). Organic petroleum-based products have also been found to be toxic to avian mallard eggs. When the eggs were exposed to a concentration of 0.5 mL/egg, 60% death was observed by 18 days of development (Hoffman and Eastin, 1981).

Application of all types of chemical soil stabilizers should not be ruled out or permitted under all conditions. Instead, guidelines should be drafted to indicate where specific chemical soil stabilizer should be applied. Application of chemical soil stabilizer should be avoided near sensitive environments, near water bodies and fractured rock, in areas with a shallow groundwater table, and other areas where water could quickly reach the saturated zone. Site-specific characteristics should be considered when approving the use of chemical soil stabilizer. Finally, information on environmental impacts and effectiveness of chemical soil stabilizer proposed for use in soil Stabilisation should be carefully assessed before approving it. The advantages (example is improved air quality) and disadvantages (cause contamination to soils) associated with chemical soil stabilizer should be considered in risk management analysis.

2.8.7.2 Performance of soil stabilized with molasses and bio-enzymes The performance of the pavement is dependent on the type and properties of thesub-grade soil (Greeshma and Lamanto, 2015). Soil properties can be modified by using eco- friendly and liquid additives such as Bio-Enzymes or Molasses (Greeshma and Lamanto, 2015). They act on the soil to reduce the voids between soil particles and minimize absorbed water when soil is compacted at maximum dry density and optimum moisture content (Greeshma and Lamanto, 2015), they conducted laboratory test to investigate behaviour of Organic Clay stabilized with Bio-Enzymes on engineering properties of soil and results showed that value of Liquid Limit (LL) of soil increased by 28%, while decreasing Shrinkage limit (SL) by 30%. Unconfined Compressive Strength (UCS) of treated soil increased 12 times that of untreated soil. Also study performed by Ravi *et al.,* (2015) on effect of molasses on strength of soil showed that, Unconfined Compressive Strength of soil increased by 94% when 6% molasses content added to Intermediate Compressible Clay (CI) also California Bearing Ratio (CBR) of Intermediate Compressible Clay (CI) increased by 6.37%. This means that, molasses played a role in improvement of soil cohesion which ultimately lead to increase Unconfined Compressive strength and resistance to penetration during

CBR test.

2.8.8 Rice husk ash stabilisation

Disposal of solid waste on the land fill can be minimized if the waste is having desirable properties such that they can be utilized for various geotechnical application viz. land reclamation, construction of embankment etc. There are several methods used for improving geotechnical properties of problematic soils that includes densification (such as shallow compaction, dynamic deep compaction, pre-loading), drainage, inclusions (such as geosynthetics and stone columns), and Stabilisations, Horpibulsuk *et al.*, (2012).

Chemical Stabilisation of the problematic soils is especially significant in concerning with the treatment of soft fine-grained, expansive soils, and collapsible loess deposits. Soil Stabilisation is the process which is used to improve the engineering properties of the soil and thus making it more stable. Soil Stabilisation is required when the soil available for construction is not suitable for the intended purpose. It includes compaction, preconsolidation, drainage and many other such processes. Rice husk ash (RHA) is a pozzolanic material that could be potentially used in soil Stabilisation, though it is moderately produced and readily available. When rice husk is burnt under controlled temperature, ash is produced and about 17%-25% of rice husk‟s weight remains ash. Rice husk ash and rice straw and bagasse are rich in silica and make an excellent pozzolana. Pozzolanas are siliceous and aluminous materials, which in itself possess little or no cementations value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form compounds possessing cementations properties. The

Rice Husk Ash would appear to be an inert material with the silica in the crystalline form suggested by the structure of the particles, it is very unlikely that it would react with lime to form calcium silicates. It is also unlikely that it would be as reactive as fly ash, which is more finely divided. So Rice Husk Ash would give great results when it used as a stabilizing material.

2.8.9 Bituminous stabilisation

Bituminous soil Stabilisation refers to a process by which a controlled amount of bituminous material is thoroughly mixed with an existing soil or aggregate material to form a stable base or wearing surface. Bitumen increases the cohesion and load-bearing capacity of the soil and renders it resistant to the action of water. Bitumen Stabilisation accomplished by using asphalt cement, asphalt cutback or asphalt emulsions. The type of bitumen to be used depends on the type of soil to be stabilized, method of construction and weather conditions. In frost areas, the use of tar as binder must be avoided because of its high temperature maximum susceptibility.

Asphalts and tars are bituminous materials which are used for Stabilisation of soil, generally for pavement construction. Bituminous materials when added to a soil, it imparts both cohesion and reduced water absorption.

2.8.10 Thermal stabilisation

Thermal change causes a marked improvement in the properties of the soil. Thermal Stabilisation is done either by heating the soil or by cooling it.

Heating: As the soil is heated, its water content decreases. Electric repulsion between clay particles is decreased and the strength of the soil is increased.

Freezing: cooling causes a small loss of strength of clayey soils due to an increase in inter-particles repulsion. However, if the temperature is reduced to the freezing point, the pore water freezes and the soil is stabilized.

2.8.11 Electrical stabilisation

Electrical Stabilisation of clayey soils is done by a process known as electro-osmosis. As a direct current (DC) is passed through a clayey soil, pore water migrates to the negative electrode (cathode). It occurs because of attraction of positive ions (Cations) that are present in water towards cathode. The strength of the soil is considerably increased due to removal of water. Electro-osmosis is an expensive method, and is mainly used for drainage of cohesive soils. Incidentally, the properties of the soil are also improved.

2.8.12 Stabilisation by geo-textile and fabrics

Geotextiles are porous fabrics made of synthetic materials such as polyethylene, polyester, nylons and polyvinyl chloride. Woven, non-woven and grid form varieties of geotextiles are available. Geotextiles have a high strength. When properly embedded in soil, it contributes to its stability. It is used in the construction of unpaved roads over soft soils. Reinforcing the soil for Stabilisation by metallic strips into it and providing an anchor or tie back to restrain a facing skin element. Past research has shown that the strength and load-bearing capacity of subgrades and base course materials can be improved through the inclusion of non-biodegradable reinforcing materials, such as fibers, geotextiles, geogrids, and geocomposites. Use of these materials can improve the performance and durability of future highways and may reduce the cost of construction. At present, most of the research on these materials is based on tests conducted in the laboratory that are only partially complete. Further laboratory tests and evaluations will be necessary to develop design specifications based on material properties, and these specifications will need to be verified using large-scale field tests.

2.8.13 Recycled and waste products

Improved chemical and mechanical Stabilisation techniques are needed for such waste materials as crushed old asphalt pavement, copper and zinc slag, paper mill sludge, and rubber tire chips. The need to recycle many potentially hazardous materials, it will be necessary to develop a realistic, economical and effective means of assessing the risk of pollution posed by these materials through leachates and emissions. In some cases, risk evaluation is hampered by restrictive environmental constraints, and this issue needs to be addressed as well.

2.9 Soil Test

Soil inspection or say geotechnical inspection is very important in understanding the physical properties of soil and the rocks beneath. This is required to ascertain the type of foundation required for the proposed construction. Various tests are done to explore the sub surface and surface characteristics of soil

2.9.1 Atterberg limits

The purpose of this test is to determine the consistency and plasticity of fine-grained soils with varying degrees of moisture. The test includes the determination of the Liquid Limits, Plastic Limits and the Shrinkage Limit Index, conducted in accordance with Test 1(A) BS 1377 (1990) Part 2.

i. **Liquid limit:**

It is the water content of the soil between the liquid state and plastic state of the soil. It can be defined as the minimum water content at which the soil, though in liquid state, shows small shearing strength against flowing. It is measured by the Cassagrande‟s apparatus and is denoted by LL.

ii. **Plastic limit:**

This limit lies between the plastic and semi-solid state of the soil. It is determined by rolling out a thread of the soil on a flat surface which is non-porous. It is the minimum water content at which the soil just begins to crumble while rolling into a thread of approximately 3mm diameter. Plastic limit is denoted by PL.

iii. **Shrinkage limit:**

This limit is achieved when further loss of water from the soil does not reduce the volume of the soil. It can be more accurately defined as the lowest water content at which the soil can still be completely saturated. It is denoted by SL.

2.9.2 Particle size distribution

Soil at any place is composed of particles of a variety of sizes and shapes, sizes ranging from a few microns to a few centimeters are present sometimes in the same soil sample. The distribution of particles of different sizes determines many physical properties of the soil such as its strength, permeability, density. Particle size distribution is found out by two methods, first is sieve analysis which is done for coarse grained soils only and the other method is sedimentation analysis used for fine grained soil sample. Both are followed by plotting the results on a semi-log graph. The percentage finer N as the ordinate and the particle diameter i.e. sieve size as the abscissa on a logarithmic scale. The curve generated from the result gives us an idea of the type and gradation of the soil. If the curve is higher up or is more towards the left, it means that the soil has more representation from the finer particles; if it is towards the right, we can deduce that the soil has more of the coarse grained particles.

The soil may be of two types- well graded or poorly graded (uniformly graded). Well graded soils have particles from all the size ranges in a good amount. On the other hand, it is said to be poorly or uniformly graded if it has particles of some sizes in excess and deficiency of particles of other sizes. Sometimes the curve has a flat portion also which means there is an absence of particles of intermediate size, these soils are also known as gap graded or skip graded. For analysis of the particle distribution, we sometimes use D_{10}, D_{30}, and D_{60} etc. terms which represents a size in mm such that 10%, 30% and 60% of particles respectively are finer than that size. The size of D_{10} also called the effective size or diameter is a very useful data. There is a term called uniformity coefficient C_u which comes from the ratio of D_{60} and D_{10}, it gives a measure of the range of the particle size of the soil sample.

2.9.3 Specific gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words we can define it as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than equal volume of water. Different types of soil have different specific gravities, general range for specific gravity of soils, as shown in Table 2.1:

Table 2.1: Standard specific Gravity of soil

Substance	Specific Gravity
Sand	2.63-2.67
Silt	2.65-2.7
Clay and Silty clay	2.67-2.9
Organic soil	<2.0

(Source: Osama *et al.*, 2008)

2.9.4 Shear strength

Shearing stresses are induced in a loaded soil and when these stresses reach their limiting value, deformation starts in the soil which leads to failure of the soil mass. The shear strength of a soil is its resistance to the deformation caused by the shear stresses acting on the loaded soil. The shear strength of a soil is one of the most important characteristics. There are several experiments which are used to determine shear strength such as DST or UCS etc. The shear resistance offered is made up of three parts:

i. The structural resistance to the soil displacement caused due to the soil particles getting interlocked,

ii. The frictional resistance at the contact point of various particles, and

iii. Cohesion or adhesion between the surfaces of the particles.

In case of Cohesionless soils, the shear strength is entirely dependent upon the frictional resistance, while in others it comes from the internal friction as well as the cohesion.

Methods for measuring shear strength:

a) Direct shear test (DST)

This is the most common test used to determine the shear strength of the soil. In this experiment the soil is put inside a shear box closed from all sides and force is applied from one side until the soil fails. The shear stress is calculated by dividing this force with the area of the soil mass. This test can be performed in three conditions; undrained, drained and consolidated undrained depending upon the setup of the experiment.

b) Unconfined compression test (UCS test)

This test is a specific case of triaxial test where the horizontal forces acting are zero. There is no confining pressure in this test and the soil sample tested is subjected to vertical loading only. The specimen used is cylindrical and is loaded till it fails due to shear.

2.9.5 X-ray Diffraction Test

X-ray diffraction is used to determine the composition of minerals in clay soil with the aid of the X-ray to gather a signal. Once the composition is understood inferences can then be made in relation to the swell potential of clay minerals. To start, the characteristics of crystals need to be revisited specifically the structure of the crystal. The atoms in a crystal are arranged in a definite orderly manner to form a three-dimensional network termed lattice. Positions within the lattice where atoms or atomic groups are located are termed lattice points (Moore and Reynolds, 1997). Only 14 different arrangements of lattice points in space are possible these are the Bravais space lattices. This information, along with the distances that separate parallel planes is important for the identification and classification of different minerals (Moore and Reynolds, 1997).

2.10 Previous Research on Soil Stabilisation

Akinwumi *et al.* (2019) investigated the use of calcium carbide residue as a stabilizer for tropical sand used as pavement material, Wastes that have cementitious properties can become a useful source of cheap materials for soil improvement, thereby reducing the cost of construction projects on sites that have unsuitable soils. This research work investigated the effects of the application of calcium carbide residue (CCR) to a tropical soil on its geotechnical properties in order to assess the suitability of the stabilized soil for use as a road pavement material. Tests to determine the grain size distribution, specific gravity, liquid and plastic limits, compaction, California bearing ratio (CBR) and unconfined compressive strength (UCS) of the natural soil and its Stabilisation with varying percentages of CCR were carried out.

The outcome showed that increasing application of CCR generally reduced the soil‟s specific gravity, plasticity index and maximum dry unit weight. A direct proportionality was also found between the CCR content and each of the liquid and plastic limits, optimum moisture content, CBR and UCS. Thus, the soil became more workable and its strength properties were improved by Stabilisation with CCR. Consequently, the subgrade characteristics of the soil for use as earthwork materials for road construction were improved. Based on strength properties of the stabilized soil, an optimal application of 4% CCR was found suitable for the Stabilisation of the sand with similar properties as those studied. The use of CCR for stabilizing sand for road construction is recommended as a cheap and sustainable approach for developing countries.

Sharana *et al.,* (2018) carried out a research on Soil Stabilisation by utilizing phosphogypsum and calcium carbide residue, in their study, The impact of balancing out specialist like Phosphogypsum and Calcium carbide build-up has been contemplated for quality, change in shifting rates (2, 4, 6 and 8%). The unconfined pressure tests (UCS) of the dirt with a various level of added substances were resolved independently. Soil adjustment is the way toward enhancing the properties of soil and making it more stable. The minerals present in stabilizers will impart strength by reacting with the soil. This is Economic and effective use of locally available materials to improve the geotechnical properties by ground improvement techniques. Calcium carbide residue (CCR) is obtained as by-products from manufacturing acetylene gas.

This CCR contains high amount of calcium content which acts as binding material. Basic properties of soil like Atterberg limits, swelling potential, compaction characteristics and strength characteristics were determined. The dirt example is treated with various rates of CCR (2%, 4%, 6% and 8%). Most extreme dry thickness diminishes with increment in CCR substance and ideal dampness content increases. The swelling nature of dirt abatement with increment in CCR content. The soil samples were treated with CCR and determined the strength characteristics by UCS and CBR. By the treatment MDD, OMC and swelling nature were determined with various percentages of CCR with soil.

Amadi and Okeiyi (2017) carried out a research on the Use of quick and hydrated lime in Stabilisation of lateritic soil: comparative analysis of laboratory data, in their study, A laboratory study was undertaken to evaluate and compare the Stabilisation effectiveness of different percentages (0, 2.5, 5, 7.5, 10%) of quick and hydrated lime when applied separately to locally available lateritic soil, a major soil group in the tropical and sub-tropical regions. Performance evaluation experiments included: Atterberg limits, compaction, unconfined compression tests, California bearing ratio (CBR), swelling potential using CBR instrument and hydraulic conductivity. The soil mixtures used for unconfined compressive strength (UCS), CBR, swelling potential and hydraulic conductivity tests were compacted at optimum moisture content using the British standard light compactive effort and cured for 28 days. It was found that the quicklime caused the soil to have lower plasticity while hydrated lime yielded higher dry unit weight. Also, higher UCS especially at higher dosages (7.5 and 10%) was produced when soil sample was treated with quicklime. Similarly, the CBR values for quicklime sample clearly indicate that quicklime-stabilized soil have superior load bearing capacity. Finally, quicklime treated specimens reached slightly lower swelling values than the hydrated lime while no appreciable distinction in hydraulic conductivity values of specimens treated with the two types of lime was observed. From the foregoing results, quicklime is adjudged to have exhibited somewhat superior engineering properties and therefore creates a more effective Stabilisation alternative for the soil.

Joel and Edeh, (2016), studied the use of Calcium Carbide Residue-Hydrated Lime Blend as Stabilizing Agent, in their study, they tested the suitability of the blend of calcium carbide residue and hydrated lime to serve as a stabilizing agent, in order to overcome the challenge of availability and high cost of hydrated lime in Nigeria was investigated. Calcium carbide residue was replaced with hydrated lime in incremental order of 10 %, from 0 % to 100 %. The blend of calcium carbide residue partially replaced with hydrated lime was used in incremental order of 2 % from 0 % to 8 % to treat laterite soil obtained from Ikpayongo. The sample of laterite soil treated with different percentages of the blend of calcium carbide residue and hydrated lime were subjected to Atterberg"s limits test, compaction test, unconfined compression strength test, durability test and California bearing ratio test. Based on result of test calcium carbide residue partially replaced with 70 % hydrated lime is recommended for use as a stabilizing agent. The use of calcium carbide residue as a stabilizing agent will provide an effective way of disposing calcium carbide residue which has negative impact on the environment, in addition to ensuring economy in the use of lime in soil Stabilisation.

Balarabe and Sharmila (2015) carried out a study on Soil Stabilisation Using Calcium Carbide Residue and Coconut Shell Ash. Ion their study, it was observed that Calcium rich and silica rich waste materials are abundantly available in many countries. These wastes ended in a waste dump there by polluting environment and endangering the lives of the people living within the vicinity. Calcium carbide residue CCR and coconut shell ash CSA are such wastes produce as result of industrial and agricultural activities. Utilizing these wastes for Stabilisation purposes may result in providing a product with adequate strength for construction purposes. In this research, CCR and CSA were employed in stabilizing CI and CH soils, CCR was fixed at 4% and 6% in CI and CH respectively using index properties tests and then CSA was varied (4, 9, 14, and 19%). Standard proctor test results showed general decrease in MDD values and increase in OMC values which may be obvious as the specific gravity of the additives is less than that of the soil. Also UCC test results indicated a tremendous improvement in the strength of both the soils with the improvement of up to 11.38 and 6.03 times the strength of the virgin soils at 7 days curing period with combination of S1+4%CCR+4%CSA and S2+6%CCR+4%CSA respectively. Hence CCR and CSA can be employed for expansive soil Stabilisation subject to further researches.

Joel and Edeh (2014) researched on the Stabilisation of Ikpayongo Laterite with Cement and Calcium Carbide Waste, in their study Laterite obtained from Ikpayongo was stabilized with 2-10% cement and 2-10% Calcium Carbide waste, for use as pavement material. Atterberg"s limits test, California bearing ratio (CBR) and unconfined compressive strength (UCS) tests were conducted on the natural laterite and the treated soil specimens. The plasticity index of the natural laterite reduced from 14% to a minimum value of 5% when treated with a mixture of 10% cement plus 10% calcium carbide waste, strength indices of the laterite was greatly improved as the 7 day UCS and CBR values of Ikpayongo laterite increased from 534 kN/m^2 and 28% respectively to 3157 kN/m^2 and 180 % respectively, when treated with a combination of 10% cement plus 10% calcium carbide waste. Based on results obtained from the study, the use of a mixture of 8% cement plus 10% calcium carbide waste, 10% cement plus 10 % calcium carbide waste are recommended for the treatment of Ikpayongo laterite for use as base material. The use of calcium carbide waste in the Stabilisation of soil will ensure economy in road construction, while providing an effective way of disposing calcium carbide waste.

Joel and Edeh (2013) researched on the Soil Modification and Stabilisation Potential of Calcium Carbide Waste. Laterite was treated with calcium carbide waste and lime as the control, to ascertain its modification and Stabilisation potential, in incremental order of 2% up to 10 %. Atterberg limits test, compaction test, California bearing ratio (CBR) and unconfined compressive strength (UCS) test was performed on laterite treated with both additives. The use of $\chi 2$ test to compare results of tests, showed that there is no significant difference between the modification potential of both additive as reflected in $\chi 2$ values of 1.293, 0.995 and 0.650 obtained from the comparison of liquid limit, plastic limit and plasticity index test results. However difference was observed with CBR and 7 day UCS test results as x^2 values of 13.75 and 11.64 respectively were higher than the standard value of 9.49 obtained from statistical Table at 4 degree of freedom and 5 % level of significance. Based on result of tests, calcium carbide waste is recommended for use in soil modification and Stabilisation, as usage will provide an effective way of disposing calcium carbide waste.

Agbede and Joel (2011) carried out a research on the effect of carbide waste on the properties of Makurdi shale and burnt bricks made from the admixtures, in their study they investigate the effects of a by-product of oxy-acetylene gas welding, carbide waste (CW) on the engineering properties of Makurdi shale blended with 0 to 8 % of CW for use in brick production. Atterberg limits and specific gravity of shale blended with CW were determined. Compressive strength and water absorption tests were conducted on triplicate cube specimens of each admixture which were fired to 800 oC. In addition, X-ray fluorescence tests were performed on the shale and CW. Results showed that Makurdi shale comprise mainly of silica and alumina while CW was essentially calcium oxide. The results suggested that 4 % CW mix was suitable to stabilize the shale. The study also showed that heat treatment of Makurdi shale and Makurdi shale: CW admixtures yielded bricks of good quality.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Preamble

This chapter relate the methods and Materials used in order to achieve the aim and objectives of this research

3.2 Description of Study Area

Ajaokuta is a Local Government Area in Kogi State, Nigeria and a town within the left bank of the Niger River. The headquarters of the LGA are in the town of Egayin in the south of the area at 6°40′11″N 8°48′19″E. The aerial view is shown in Plate I

Plate I: Aerial view of Ajaokuta-Lokoja

3.3 Materials

This research was conducted in stages to achieve the objectives of the study. The principal materials used in this research are:

i. Lateritic Soil,

ii. Calcium Carbide Waste.

iii. Water

3.3.1 Laterite

The laterite sample that was used for this study was collected along Ajaokuta Lokoja express way. The disturbed sample was taken and transported to Federal University Technology (FUT) Minna laboratory of the Department of Civil Engineering for analysis.

3.3.2 Calcium carbide waste (CCW)

The Calcium Carbide Waste (CCW) used for this study was obtained from Ajaokuta Mechanic village by African Camp Geregu Junction Ajaokuta Kogi State and transported to Federal University Technology (FUT) Minna Laboratory for analysis.

3.3.3 Water for mixing

The water used for this study was clean portable water in accordance with BS EN 1008:2002 and was obtained within the premises of the permanent site of the Federal University of Technology, Minna, Niger State.

3.4 Methods

This research work commenced with the sourcing, collection and conveying of the soil sample to the laboratory for classification tests in order to determine the appearance in terms of colour, dry strength and textural systems of the soil sample. The stage for the method is as shown in Figure 3.1.

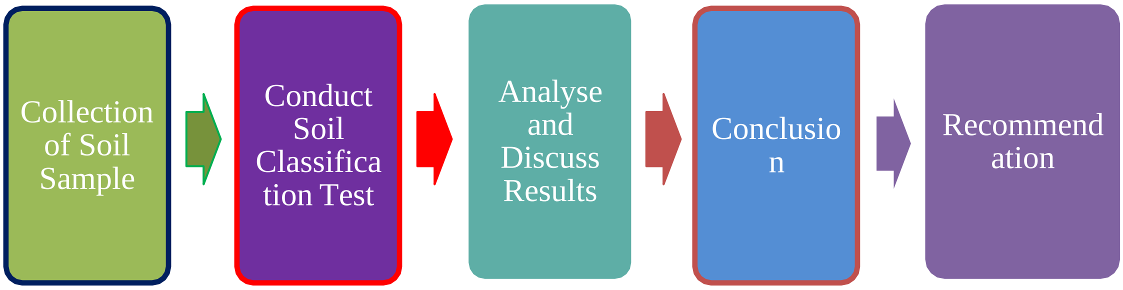

Figure 3.1: Stages of the Research

The laboratory tests performed on the natural soil so as to determine its engineering properties were in accordance with BS 1377 (1990) and BS 1924 (1990). The following tests were conducted:

i. Chemical and mineralogical constituents Tests
ii. Moisture content determination Test
iii. Soil classification
iv. Particle size distribution (Wet Sieve Analysis) Test
v. Atterberg limit test
vi. Specific gravity test
vii. Compaction test
viii. Unconfined compressive strength (UCS)
ix. California Bearing Ratio (CBR)

3.4.1 Chemical and Mineralogical Test of the Sample

This test involves X-Ray diffraction test and Scanning electron microscopy (SEM) Test

3.4.1.1 X- Ray Diffraction Test

This test is to identify and determine the mineralogical phase characteristics and the estimation of the average crystallite size of minerals in the tropical clay soil. This was done by placing and clipping the powdered sample to the sample holder and a Bruker

AXS D8 X-ray diffractometer system coupled with C_u-K_α radiation of 40 kV and a current of 40 mA applied. The λ for K_α was 0.1541 nm with scanning rate of 1.5°/min. A step width of 0.05° was used over the 2θ range and the diffractograms recorded in the 2θ range of (20-90) ° on the powder tropical clay sample. The XRD machine used is showed in Figure. 3.2

Plate II : XRD Machine

3.4.1.2 Scanning electron microscopy (SEM) Test

This test uses a focused beam of high energy electrons to generate a variety of signals at the surface of the solid sample to obtain the surface topography and elemental composition of the sample.

Exactly 0.05 mg of the powdered sample was sprinkled on a sample holder covered with carbon adhesive tape and sputter coated with Au-Pd using Quorum T15OT for 5 minutes prior to analysis. The sputter coated sample was then characterized using Zeiss Auriga SEM in order to analyze the morphology and microstructure of the sample.

3.4.2 Natural moisture content

Disturbed clay soil samples was collected in bags and its moisture content determined by oven drying method, the test was conducted in accordance with Part 2 of BS 1377 (1990). The moisture content of the soil is calculated as a percentage of the ratio of the mass of water to mass of dry soil as expressed in Equation (3.1)

$$\text{Moisture content (m) \%} = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \qquad 3.1$$

Where M_1 = mass of empty specimen container (g)

M_2 = mass of specimen container + moist soil (g)

M_3 = mass of specimen container + dried soil (g)

3.4.3 Soil classification

Unified system and its derivatives classify soil by type rather than by engineering suitability for specific uses, although they can nevertheless be used to infer suitability.

By contrast, the system defined by the American Association of State Highway and Transportation Officials (AASHTO, 2012) does not classify soils by type (for example sands, clays) but simply divides them into seven major groups, essentially classifying soils according to their suitability as subgrades, each material can be given a „group index" value defined as:

$$\text{group index} = (F - 35)\left[0.2 + 0.005(\text{LL} - 40)\right] + (F - 15)(\text{PI} - 10) \tag{3.2}$$

where F is the percentage passing the 75 μm (0.075 mm) sieve, expressed as a whole number. This percentage is based only on the material passing the 75 mm sieve.

LL is the liquid limit and

PI is the plasticity index.

This is usually shown in brackets after the soil class. When applying the formula, the following rules are used:

the group index is reported to the nearest whole number and, if it is nega- tive, it is reported as zero; when calculating the group index of subgroups A-2-6 and A-2-7, only the plasticity index portion of the formula should be used; and because of the criteria that define subgroups A-1-a, A-1-b, A-2-4, A-2-5 and group A3, their group index will always be zero, so the group index is usually omitted from these classes. Subgroup A-7-6 materials have high plasticity indices in relation to the liquid limits and are subject to extremely high volume change.

Originally the group index was used directly to obtain pavement thickness designs, using the „group index method" but this approach has long since been superseded and the group index is used only as a guide.

Table: AASTHO Soil Classification System

GENERAL CLASSIFICATION	GRANULAR MATERIALS (35% or less passing 0.075mm sieve)							SILT-CLAY MATERIALS (> 35% passing 0.075mm sieve)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, % passing:											
2mm	50 max	–	–	–	–	–	–	–	–	–	–
0.425mm	30 max	50 max	51 min	–	–	–	–	–	–	–	–
0.075mm	10 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Fraction passing 0.425mm:											
Liquid limit	–		Non-plastic	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max			10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min*
Usual types of significant constituents	Stone fragments, gravel, sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as a subgrade	Excellent to good							Fair to poor			

*Plasticity index of A-7-5 subgroup is equal to or less than liquid limit – 30.
*Plasticity index of A-7-6 subgroup is greater than liquid limit – 30.

(Source; AASHTO 1986).

3.4.4 Particle size analysis

The purpose of Sieve Analysis was to determine the percentage of various grain sizes. Grain size distribution is used to determine the textural classification of the soil which will then be used to evaluate the engineering characteristics of the lateritic. The soil samples will be washed through BS No. 200 sieve or 0.075mm and the material retained will be oven dried for 24 hours and sieved by agitating the dried samples through a range of sieves from sieve No. 20 mm to 0.075 mm aperture sieve while the material passing will be turned into a sedimentation cylinder for hydrometer analysis. The particle size analysis test was carried out in accordance with BS 1377; 1990 Part 2.

3.4.5 Atterberg limits tests

The test includes Liquid limit test, Plastic limit (PL) test and Plasticity index (PI)

3.4.5.1 Liquid limit test

A Cassagrande liquid limit device was used prior to the commencement of the test, the sample was oven dried and sieved and that passing through the BS sieve No. 36 (0.425mm) would be used for this test. A representative sample is mixed with small amount of water at the start of the test and a part of the moist soil sample was placed in the brass cup using a knife edge. The groove opening was made using the grooving tool. This will be repeated for groove closure at 10, 20, 30 and 40 blows. After each groove was made , samples of it will be taken from the point of groove closure and put in moisture can to determine moisture content (%) after oven drying for not less than 24 hours. The liquid limits (LL) of the sample is the moisture content (%) corresponding to 25 blows as will be obtained from the graph of moisture content (%) against the No. of blows. B.S 1377 (1990) describes the procedure for the determination of liquid limit test of the soil used for this research work.

3.4.5.2 Plastic limit (PL) test

The plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the Liquid Limit to determine the plasticity index which when plotted against the liquid limit provides a means to classify cohesive soils. About 40g of the soil was air-dried, placed on a smooth mixing disc and then mixed with water to make it plastic enough to shape into a small ball. The sample was then divided into two samples and rolled between the pan and the glass plate.

The two samples were in turn divided into two other samples each and rolled between the palm of the hand and the glass plate using enough pressure to reduce the diameter to about 3 mm thread. The procedure was repeated until the thread crumbles or shears both longitudinally and transversely when rolled to about 3 mm diameter. The pieces of the crumbled threads were gathered in a can and taken for moisture content determination. The procedure was repeated three times with a fresh sample of plastic soil each time.

The plasticity index (PI) of the soil is the difference between the liquid limits of the natural/various mixes of the soil and their corresponding plastic limits. It is the range of water content where the soil is plastic. Many engineering properties have been found to empirically correlate with the PI and it is also a useful engineering classification of fine-grained soil.

3.4.5.3 **Plasticity index (PI)**

Plasticity Index (PI) this is calculated from the difference between liquid limit (LL) and plastic limit (PL).

Mathematically, PI = LL – PL (3.3)

3.4.6 Specific gravity

Specific Gravity (GS) of any material is defined as the ratio of the weight of given volume of that material to the weight of an equal volume of water, the test was carried out in accordance with Part 2 of BS 1377 (1990).

The specific gravity bottle was weighed when empty as W_1 and when it is filled with water completely (W_4). It was then emptied of water and dry. A representative soil sample of about 50g was placed in the bottle and weighed as $W_{2.}$ Water was then added into the bottle containing the soil sample and it is thoroughly stirred using the stirring rod. The bottle was then filled to the brim with water and left undisturbed for 24 hours and weighed as W_3. The specific gravity was then calculated using equations (3.4) and (3.5) below

$$Specific\ Gravity(G.S) = \frac{weight\ of\ sample}{weight\ of water\ displace} \tag{3.5}$$

$$G.S = \frac{(W_2 - W_1)}{(W_4 - W_1) - (W_3 - W_2)} \tag{3.6}$$

3.4.7 Unconfined compressive strength

The Unconfined Compression Test (UCS) consist of trimming the sample of some diameter to adequate $^{L}/_{d}$ ratio, placing it in a compression device, measuring load and corresponding displacement selected time or displaced intervals. It is used to evaluate the shear strength of the soil.

BSH : 10 blows by 5layers

BSL : 3 blows by 3layers

WAS: 6 Blows 3 layers

$$\text{Undrained Shear Strength} = \frac{unconfined\ compressive\ strength}{2} \tag{3.7}$$

$$\text{Su} = \frac{qu}{2} \tag{3.8}$$

3.4.7.1. Procedure for conducting UCS test

The Unconfined Compression Test (UCS) was conducted using the following procedures:

i. The sample was mixed at a desired water content and density in the large mould.
ii. The sampling tube was filled with the soil.
iii. The sample was saturated in the sampling tube by soaking.
iv. The split mould was coated lightly with a thin layer of grease. The mould was weighed.
v. The sample was extruded out of the sampling tube into the split mould, using the sample extractor and the knife.
vi. The two ends of the specimen was Trimmed in the split mould. Weigh the mould with the specimen.
vii. The specimen from the split mould was removed by splitting the mould into two parts.
viii. The length and diameter of the specimen was measured using vernier calipers.
ix. The bottom of the specimen was placed on the bottom plate of the compression machine. And the upper plate was adjusted to make contact with the specimen.
x. The dial gauge was adjusted and the proving ring gauge to zero.
xi. The compression load was applied to cause an axial strain at the rate of ½ to 2% per minute.
xii. The dial gauge reading was recorded, and the proving ring reading every thirty seconds up to a strain of 6%.
xiii. The tested Continue until failure surfaces have clearly developed or until an axial strain of 20% is reached.
xiv. The angle between the failure surface and the horizontal was measured.
xv. The sample from the failure zone of the specimen was taken for the water content determination.

3.4.8 California bearing ratio (CBR)

This is an empirical test which gives an indication of the shear strength characteristic of the soil. The CBR test is currently used in pavement design for both roads and airfields. The laboratory CBR test measures the shearing resistance of a soil under control and density conditions. The compaction energies were achieved as follows

BSH: 62 blows by 5 layers (Modified Proctor)

BSL : 25 blows by 3 layers (Standard Proctor)

WAS: 25 blows 5 layers (West African Standard) .

3.4.8.1. Procedure for conducting CBR test

The California bearing ratio (CBR) was conducted using the following procedures:

i. ¾ in (19 mm) sieve was to sieve the soil specimen. All material that passes through the sieve can be used for the test. But some of the material was retained in the sieve.

ii. After sieving, 3 samples of specimens each containing 6.8 kg (15 lb) were produced

iii. Specimen 1,2,3 was compacted with about 62 blows for BSH and 25 blows for BSL and WAS & 56 blows respectively. This provided variations in the percentage of maximum dry density.

iv. Sufficient amounts of water were mixed with specimens to maintain optimum water content.

v. The mold was attached to the base plate with the extension collar. Then the weight shall be measured. Then a spacer disk was placed into the mould with filter paper on top of the spacer disk.

vi. The mould was filled with soil in 3 layers. For example: for specimen 1, we have to provide 20 blows per layer with the rammer for the compaction.

vii. The water content of the material was measured before and after the compaction procedure.

viii. Then the extension collar was removed and the top of the was trimmed with a straightedge to smoothen the surface.

ix. The other two specimens were compacted following the same procedures mentioned above.

x. The disk spacer and base plate were removed, and then the weight of the Mold plus compacted soil was measured.

xi. Then invert the mould and soil and attach the base plate to the mould with a coarse filter paper.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Preamble

This chapter presents and discusses, with deductions, all the results obtained from the laboratory tests carried out in order to determine the physical and mineralogical properties, compaction and consolidation characteristics of the lateritic soil from selected borrow pits in Lokoja Ajaokuta.

4.2 Chemical and Mineralogy Constituents of lateritic Soil

This section discuss the X- Ray diffraction test (XRD) of the lateritic soil and X-Ray Florescence (XRF) of the Lateritic Clay Soil

4.2.1 X- Ray diffraction test (XRD) of the lateritic soil

Table 4.1 contains the mineralogical composition of the lateritic soil. The XRD results are graphically represented in Figures 4.1, 4.2 and 4.3 respectively.

The Energy Dispersive Spectroscopy (EDS) test results revealed that the lateritic soil contains 24.9% of Carbon, 39.8% of Oxygen, 0.76% of Magnesium, 9.8% of Aluminum, 17.4% of Silicon, 0.59% of Potassium, 0.54% of Calcium, 0.36% of Titanium and 5.6% of Iron. Figure 4.1 contains the Scanning Electronic Microscopy (SEM) of the lateritic soil compacted at the Standard Proctor compaction energy and further revealed occasional presence of air voids in the flecks of clay particles of the soil sample, which are. This is akin and in agreement to the claims of Abdullah, *et al.,* (2018), Zhang *et al*., (2013) Jaiswal and Lai (2016).

Table 4.1, Figure 4.2 and Figure 4.3 further indicates to contain substantial composition of secondary minerals, prominent amongst these minerals is montmorillonite; this mineral is responsible for increased activity of the clay soil upon water intake.

Table 4.1: Mineralogical Composition of lateritic Soil

Description	Quantity
Quartz (%)	50.00
Ankerite (%)	6.25
Calcium Silicide (%)	6.25
Montmorillonite (%)	8.28
Anorthite (%)	6.12
Sodium Aluminum Silicate Hydrate	6.25
Anothoclase	8.32
Orthoclase	10.07

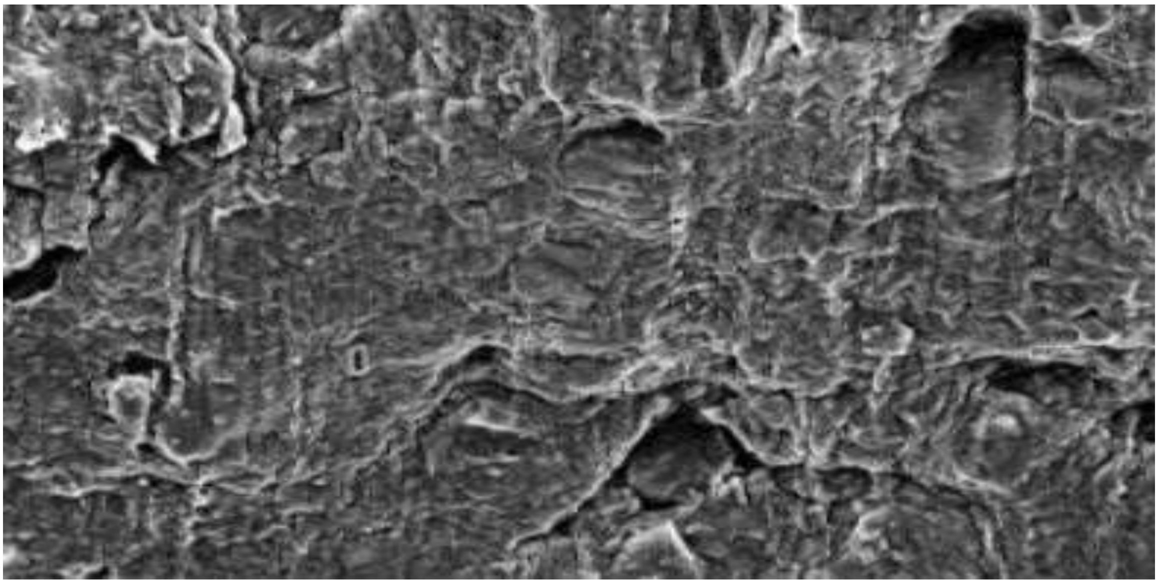

Figure 4.1: Scanning electron microscopy (SEM) of lateritic soil

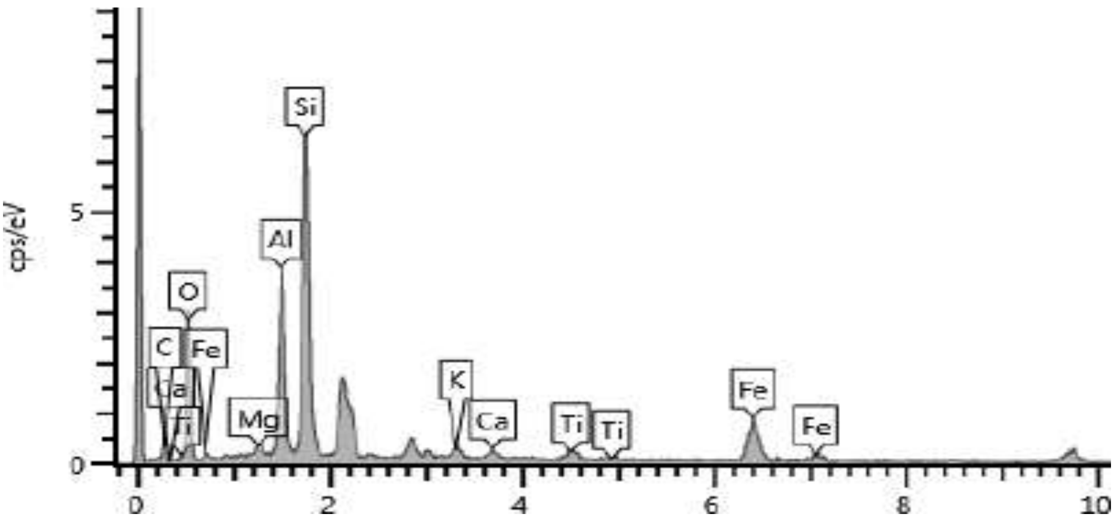

Figure 4.2: EDS of lateritic soil

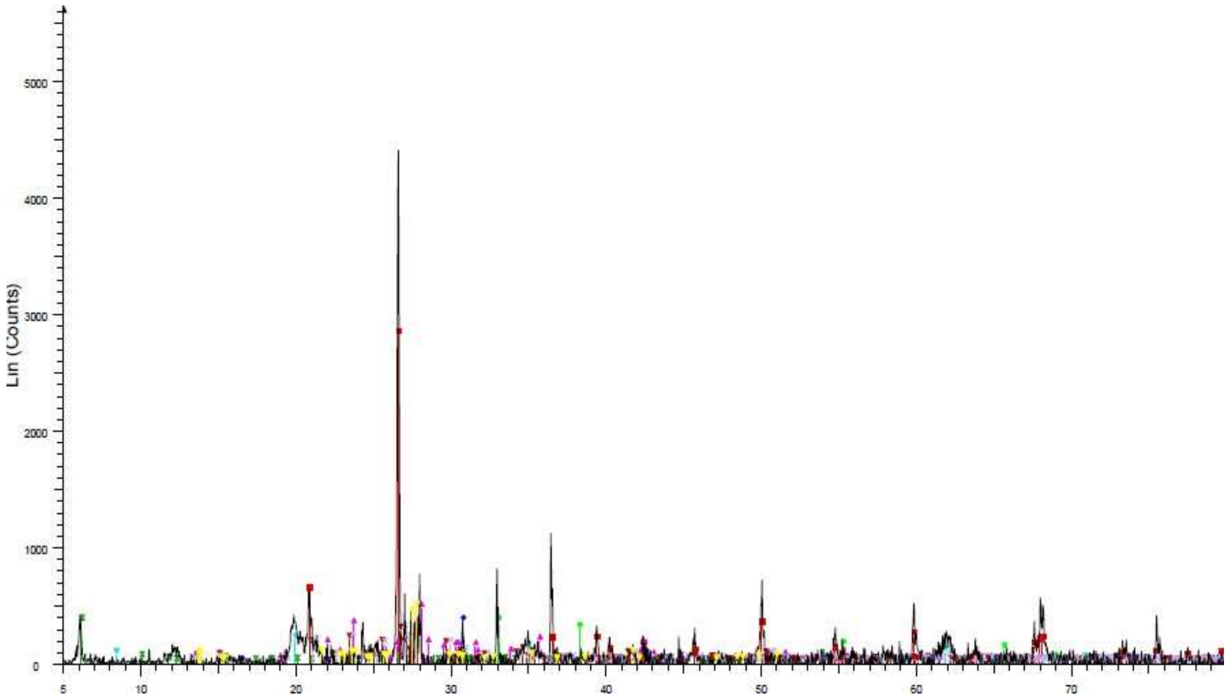

Figure 4.3: X-RD of lateritic soil

It is additionally manifested in the results that the soil contains less of Anorthite and more of montmorillonite minerals. The presence of montmorillonite is due to the weathering of the basic constituents and exhibits a swelling tendency in the presence of water; this is the main cause of the many problems such as pavement failure and excessive settlement associated with the soil (Ikeagwuani, 2016).

Zubair et *al.,* (2017) asserts that the montmorillonite mineral in lateritic soil is made up of bonded sheet-like units with the basic structure of each unit comprising of gibbsite sheet or octahedral sheets sandwiched between two silica sheets held by weak Van Der Waal forces. The montmorillonite structure is very unstable, hence can induce the replacement of the silica in the tetrahedral ions with Aluminum ions and, this ultimately, will result in water absorbing tendencies and low bearing capacity of the soil.

4.2.2 X-Ray florescence (XRF) of the lateritic soil

The chemical makeup of this lateritic soil obtained from the X-Ray Fluorescence (XRF) test result is as shown in Table 4.2. The high composition of Silicon Dioxide in the soil is responsible for the supply of pozzolanic reactivity; this makes the soil weak and not fit for use as core material in most civil and or geotechnical engineering works. The chemical make is shown in Table 4.2.

Table 4.2: X-Ray Fluorescence (XRF) of Lateritic Soil

Oxides	Composition
CuO	0.003
NiO	0
Fe_2O_3	10.348
MnO	0.166
Cr_2O_3	0.031
TiO_2	1.615
CaO	1.963
Al_2O_3	4.523
MgO	0.558
ZnO	0.006
SiO_2	23.302
Total	42.515
Balance	57.485

It is further discovered that the soil contains Titanium Oxide. Research has it that this Oxide (pigment) is responsible for the darkish or darkish grey colour of the soil. The lateritic soil was also found to contain other active minerals including Magnesium, Aluminium and Iron Oxides, which are responsible for its expansive and shrinkage behaviours.

4.3 Physical Properties of Test Samples

The results of the various classification tests conducted to determine the physical properties as well as the index properties of the lateritic soil used are as shown and presented in Table 4.3

The index property tests conducted on the lateritic soil revealed that the soil has low moisture content. The natural moisture content is an indicator of the amount of water present in a soil. Balarabe and Sharmila (2015) observed that lateritic soils often have a natural moisture content of 50% or more, hence a moisture content of 35.06% indicates that the clay soil used in this research is wet.

The particle size distribution curve is shown in Figure 4.4 with liquid limit and plasticity index flanges for silt-clay materials indicates that the soil belongs to the CL group in the Unified Soil Classification System (ASTM, 1992) or A-7-6 (13) soil group of the AASHTO soil classification system (AASHTO, 1986). From Tables C1 to C5, Figures F1 to F5 in the appendices, it was found that the soil has a specific gravity of 2.66 with MDD=1.99g/cm^3, OMC=21.00% for the Modified Standard Proctor energy levels and MDD=1.51g/cm^3, OMC=29.1% for Reduced Standard Proctor energy level as contained in Tables B1 to B5, Figures E1 to E5 respectively in the Appendices.

The results of the various classification tests conducted to determine the physical properties as well as the index properties of the lateritic clay soil used are as shown and presented in Table 4.3 and Table 4.4 respectively.

Table 4.3: Index Properties of the Soil Sample

Property	Quantity
Percentage passing BS No 200 sieve	82.20
Natural Moisture Content, %	20.00
Liquid Limit, %	46.00
Plastic Limit, %	25.32
Plasticity Index, %	20.68
Specific Gravity	2.66
	A-7-6 (13)
AASHTO Classification	CH
USCS	Varies with compaction energy
Maximum Dry Density, g/cm^3	Varies with compaction energy
Optimum Moisture Content, %	Grey
Colour	Montmorillonite
Dominant clay mineral	

4.3.1 Particle size distribution

The grain size distribution presented in Table 4.3 and Figure 4.4

Table 4.4: Particle Size distribution of Lokoja-Ajaokuta Lateritic Soil.

Sieve Designation	Percentage by Weight Mass. Retained	% Retained	**% Passing**
5.00	0.10	0.03	99.97
3.35	0.30	0.10	99.87
2.36	0.70	0.23	99.63
2.00	0.10	0.03	99.60
1.180	2.40	0.80	98.80
0.850	3.10	1.03	97.77
0.600	5.30	1.77	96.00
0.425	5.90	1.97	94.03
0.300	4.10	1.37	92.67
0.150	18.30	6.10	86.57
0.075	13.10	4.37	82.20

The index property tests conducted on the lateritic soil revealed that the soil has low moisture content. The natural moisture content is an indicator of the amount of water present in a soil. Arora (2011) observed that clay soils often have a natural moisture content of 50% or more, hence a moisture content of 35.06% indicates that the clay soil used in this research is wet.

The particle size distribution curve is shown in Figure 4.3 with liquid limit and plasticity index flanges for silt-clay materials in Figure 4.4 (AASHTO, 1986) indicates that the soil belongs to the CH group in the Unified Soil Classification System (ASTM, 1992) or A-7-6 (13) soil group of the AASHTO soil classification system (AASHTO, 1986).

The soil is greyish (from wet to dry states) with a natural moisture content of 35.06%, liquid limit of 64.29%, and plastic limit of 28.37% According to Whitlow (1995), liquid limit less than 35% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity and between 70% to 90% very high plasticity and greater than 90% extremely high plasticity according to Table 4.2. This shows that the soil sample has high plasticity and natural water content which influences compressibility and falls below the standard recommendation for use in most civil and geotechnical construction works especially highway construction (Abdulkarim *et al.,* 2021)

4.3.2 Atterberge limit

The variation of liquid limit (L.L) plastic limit (P.L) and plastic index (P.I) values are shown in Figure 4.4.

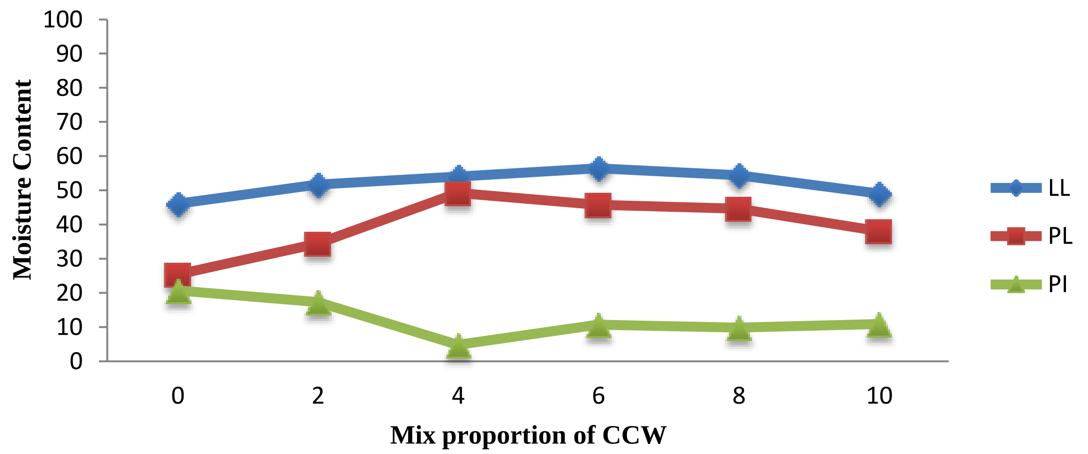

Figure.4.4: Atterberg"s limit results of mixed samples

According to the BS specification for road works, material for sub-base are required to have maximum LL of 45 %, P.L of 15 % and L.S of 18% while materials for base are required to have maximum LL of 25 %, PL of 10 % and LS of 6 %. From the plot it could be deduced that the liquid limit of the soil improve with an increase in the percentage addition of Calcium carbide residue. This further explains that the plasticity index and liquid limit are above the maximum of 12% and 30% values respectively recommended for sub-base/base soils by Federal Ministry of Works and Housing (FMWH) specification, 1997) due to the tendency of excessive accumulation of water resulting in high compressibility and loss of shear strength (Arora, 2011).

4.4 Compaction Characteristics

The three compaction energy levels used to study the compaction characteristics of the lateritic soil are; British Standard Heavy Compaction (BSH), British Standard Light Compaction (BSL) and West African standard Compaction (WAS). The compaction energy levels used are shown in Table 4.1, Figures 4.3 and 4.4 respectively.

Table 4.5: Variables of the Different compactive Efforts used in Nigeria in Joules

Compactive Effort	Weight of Rammer (kg)	Number of Blows per layer	Number of layers	Drop Height of Rammer (m)	Compaction Energy (kN-m/m^3)
BSL	2.5	25	3	0.30	**595.95**
WAS	4.5	25	5	0.45	**993.26**
BSH	4.5	62	5	0.45	**2681.80**

Source: (Osinubi and Nwaiwu, 2006)

Lateritic soil stabilized with Calcium carbide residue were compacted using British Standard Heavy Compaction (BSH), British Standard Light Compaction (BSL) and West African standard Compaction (WAS) to obtain the moisture - density relationships of the various mixes. The test results show a trend of increase in the maximum dry density (MDD) as their respective Optimum Moisture Content increases as shown in Figure 4.5a and Figure 4.5b

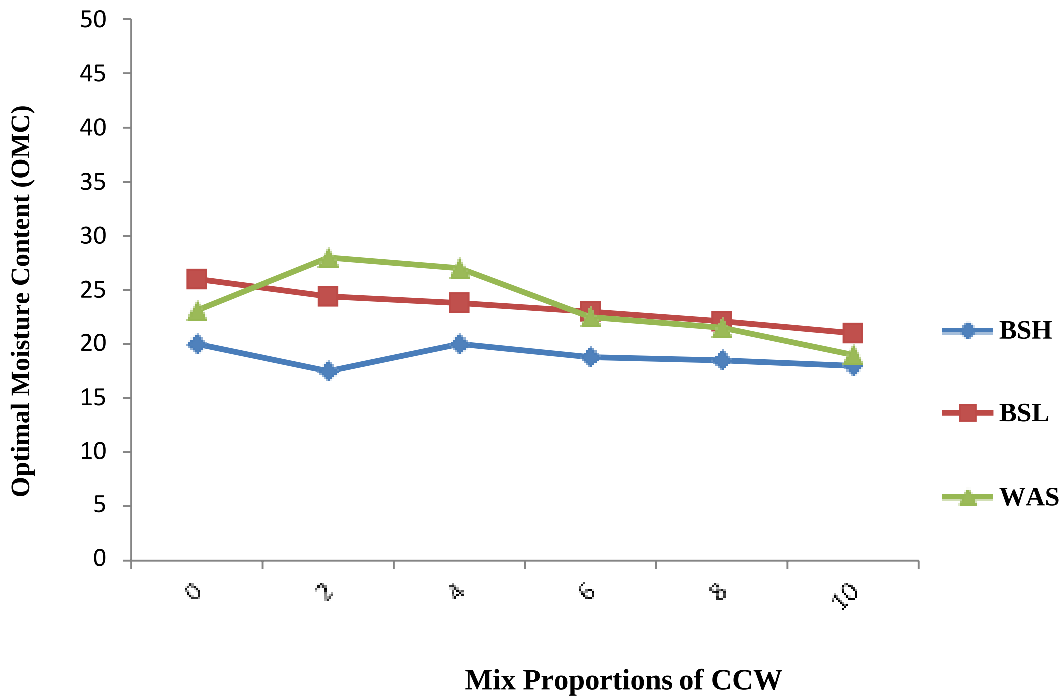

Figure 4.5a: Variation of OMC with increase in CCW

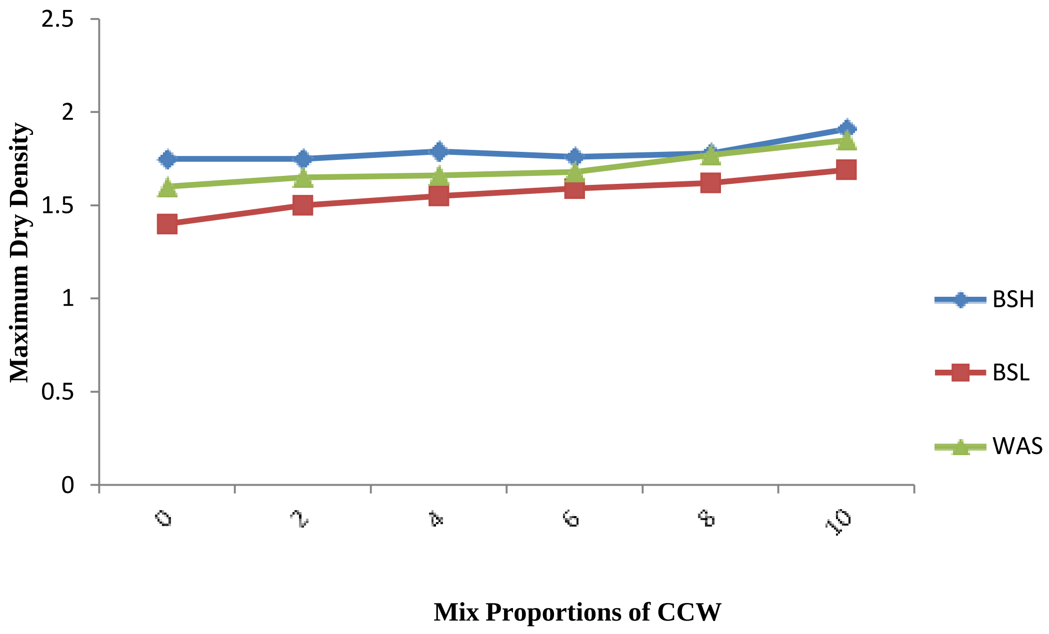

Figure 4.5b: Variation of MDD with increase in CCW

From Figure 4.5a, it is generally observed that the OMC decrease with increase in CCW content. This is attributed to the reaction of the CCW with the soil, which is more of flocculation reaction, which is transforming the fine soil to more of granular nature. This is in agreement with the findings of National Lime Association, (2004) and Horpibulsuk *et al.*, (2012). Depending on the energy level, it is observed that the higher the energy, the lower the OMC, this means that the OMC decrease in the order of BSL-WAS-BSH

From Figure 4.5b, it is observed that the MDD increased with increase in CCW content. This is also attributed to the reaction of the CCW with the soil, whereby fine soil is transformed to more of granular nature. This is in agreement with the findings of Sharana, Ullas and Yathiraju (2018) and Balarabe and Sharmila (2015), who are both of the opinion that a granular soil has more MDD. Depending on the energy level, it is observed that the higher the energy, the higher the MDD, this means that the OMC Increase in the order of BSL-WAS-BSH

4.5 Unconfined Compressive Strength Test

Three compaction energy levels used to study the unconfined compressive strength test of the lateritic soil are; British Standard Heavy Compaction (BSH), British Standard Light Compaction (BSL) and West African standard Compaction (WAS). The summary of the variation of the UCS with respect to the different energy level is shown Table 4.6, Figures 4.6a and 4.6b and Figure 4.6c respectively.

Table 4.6: Summary of UCS result for BSH, BSL and WAS

CCW (%)	Days of Curing	BSH	WAS	BSL
0%	0	120	110	60
	7	160	160	80
	28	195	505	125
2%	0	290	250	120
	7	220	335	110
	28	390	430	145
4%	0	280	200	90
	7	215	420	140
	28	290	470	170
6%	0	255	200	105
	7	440	410	200
	28	490	490	225
8%	0	410	290	210
	7	720	595	295
	28	830	840	330
10%	0	440	340	170
	7	810	595	340
	28	1020	1040	445

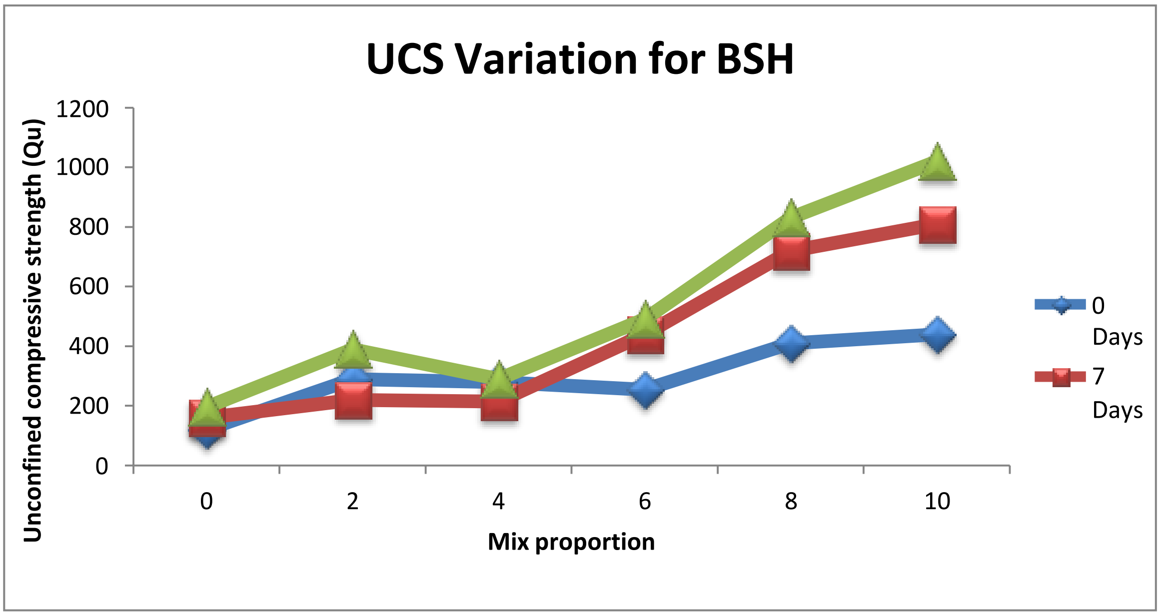

Figure 4.6a. Variation of UCS for 0, 7 and 28 days of various mixes for BSH

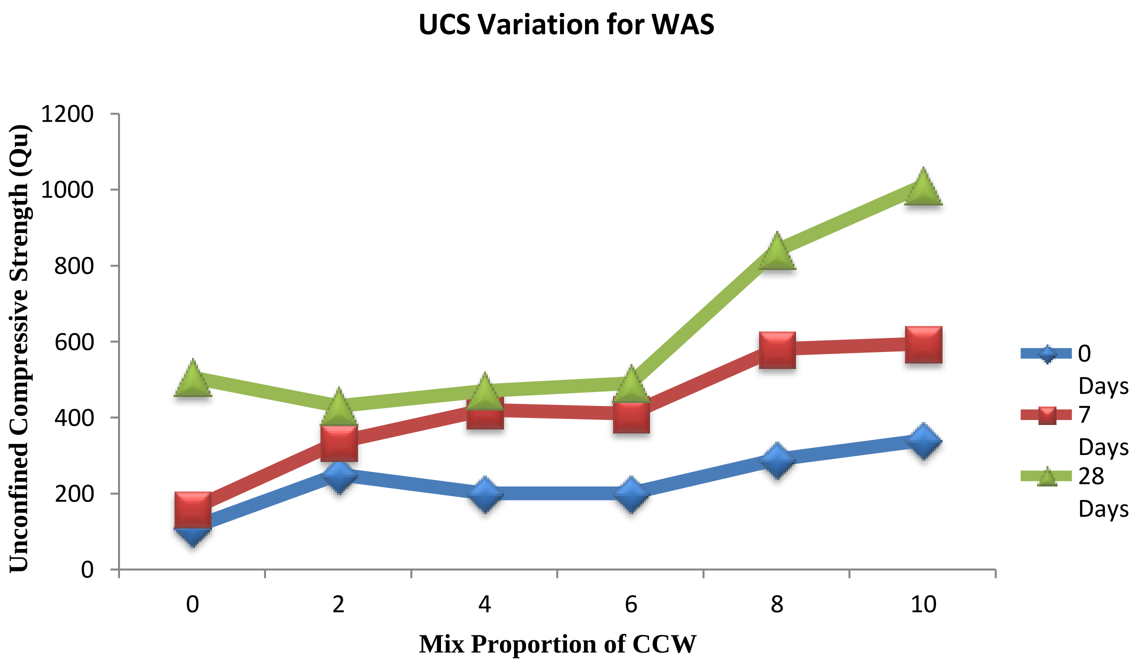

Figure 4.6b: Variation of UCS for 0, 7 and 28 days of various mixes for WAS

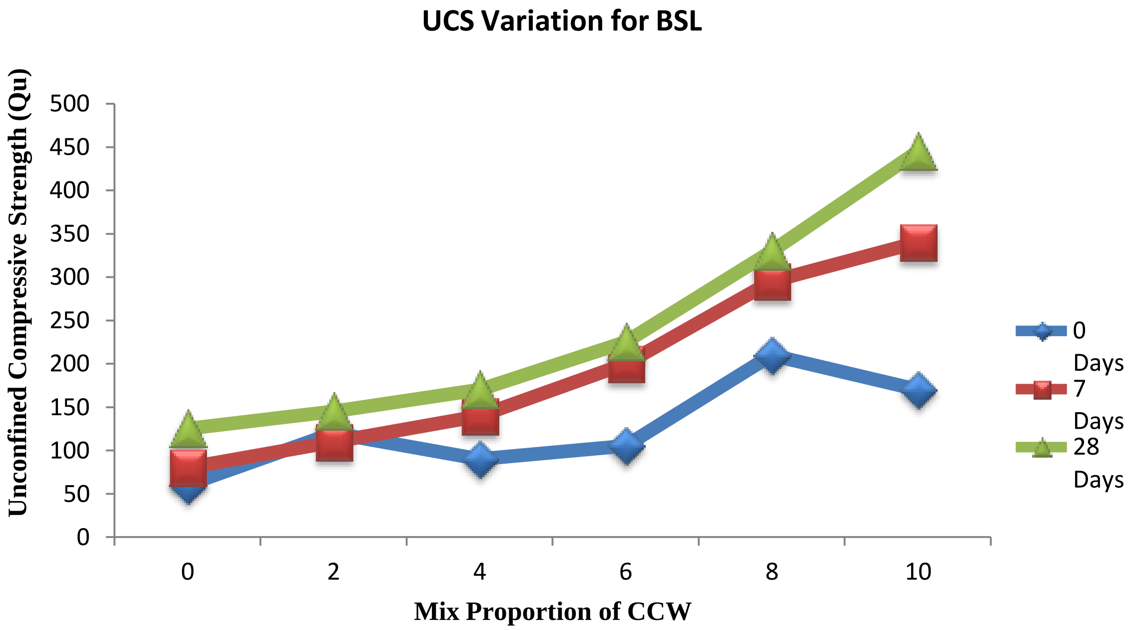

Figure 4.6c: Variation of UCS for 0, 7 and 28 days of various mixes for BSL

From Figure 4.6a, 4.6b and 4.6c above, a common array is noticed in which strength improves as the CCW replacement increases. It was also noticed that the days of curing had a significant impact on the unconfined compressive strength i.e.. The UCS of the BSH increases with increase in curing duration, with the peak value for the 0 day, 7 days and 28 days curing ranging from 440, 810 and 1040 with the increasing concentration of the calcium carbide in the laterite, also the UCS of WAS increases with increase in curing duration, with the peak value for the 0 day, 7 days and 28 days curing ranging from 340, 595 and 1010 with the increasing concentration of the calcium carbide in the laterite. Finally the UCS of BSL increases with increase in curing duration, with the peak value for the 0 day, 7 days and 28 days curing ranging from 170, 340 and 445 with the increasing concentration of the calcium carbide in the laterite.

4.6 California Bearing Ratio (CBR)

Three compaction energy levels used for the California bearing ratio test of the lateritic soil are; British Standard Heavy Compaction (BSH), British Standard Light Compaction (BSL) and West African standard Compaction (WAS). The summarized CBR values is shown on Table 4.7, Figure 4.8a shows a plot for unsoaked CBR values for BSH, BSL and WAS respectively, while 4.8b shows a plot for soaked CBR values for BSH, BSL and WAS respectively.

Table 4.7: CBR values of Soaked and Unsoaked

CCW Composition %	**California Bearing Ratio (CBR)**					
	BSH		BSL		WAS	
	Unsoaked	**Soaked**	**Unsoaked**	**Soaked**	**Unsoaked**	**Soaked**
0	9.96	6.30	1.71	3.33	7.72	3.88
2	16.59	5.88	11.45	3.74	13.22	4.20
4	24.95	28.8	13.57	15.55	20.45	22.70
6	39.30	32.66	14.78	19.50	30.45	23.50
8	43.99	26.12	16.62	23.50	26.12	20.50
10	47.23	38.92	32.02	15.39	38.92	15.39

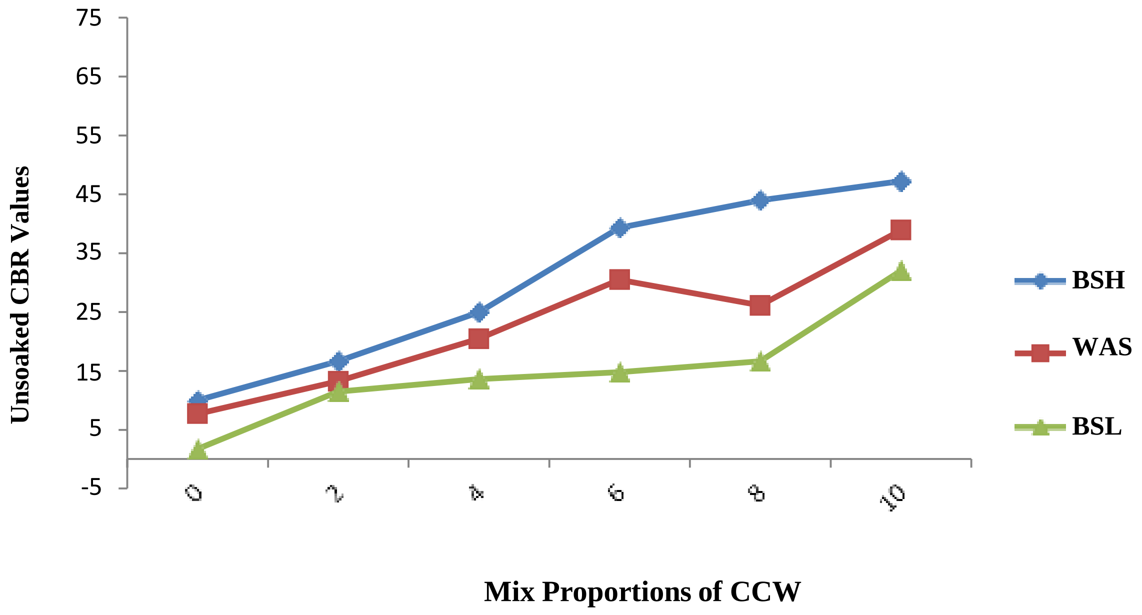

Figure 4.7a: Plot of Unsoaked CBR

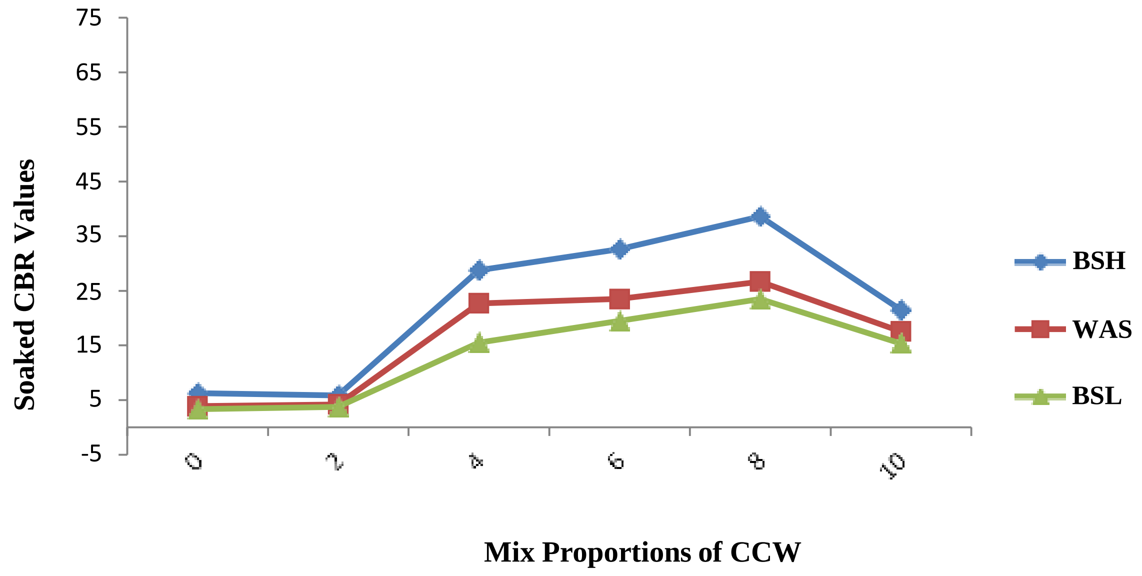

Figure 4.7b: Plot of Soaked CBR

From figure 4.8a and 4.8b Unsoaked CBR samples gives higher value compared to soak CBR for all the percentage of replacement of calcium carbide using the three energy level, that is, the CBR increased with an increase in the increase in CCW with 8% having the highest value. This is could be that the unsoaked soil specimen was partially saturated and having high suction pressure leads to high pressure between the lateritic particles results high CBR value. It can be concluded that the CBR value of a given soil is controlled by the densification (the CBR value of the soil is dependent on the relative dry unit weight (Horpibulsuk, *et al.*., 2012). The factors affecting CBR value are soil texture, moisture, and density. The testing procedure employed will depend on the type of material being tested. Granular soils were not greatly affected by swelling during the soaking period, and, therefore, the surcharge weights are not significant during this part of test (Yoder and Witczak 1975). In contrast, claylike soils, which are greatly affected by swelling pressures, will yield CBR values depending upon the weight of the surcharges used during the soaking period.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the investigation on the lateritic soil from Ajaokuta-Lokoja, the following conclusions were drawn;

The natural soil used in this research is greyish clay (from wet to dry states) with a natural moisture content of 20.00%, liquid limit of 46.00%, plastic limit of 25.32%, plasticity index of 20.68% and specific gravity of 2.66. It belongs to the CH group according to the Unified Soil Classification System (ASTM, 1992) or A-7-6 (13) soil group of the American Association of State Highway and Transportation Official"s classification system (AASHTO). The mineralogical screening of the soil sample showed that it contains less of kaolinite and more of montmorillonite minerals which is responsible for the partial swelling behaviour of the lateritic soil in the presence of water.

Optimum Moisture Contents (OMC) and Maximum Dry Densities (MDD) of the soil are established. A reduction in the OMC from 20.0 to 18.05 is observed at BSH, while an increase in OMC is observed from 26.00 to 27.00 using BSL compation and reduction is also observed from 23.10 to 19.00 in the WAS energy level respectively. An increased in the MDD was observed from 1.75 to 1.91 .000 g/cm^3 at the BSH , a decrease is observed from 1.60 to 1.51 on BSL while thw MDD of WAS remain relatively unchanged at 1.68.

Unsoaked CBR samples gives higher value compared to soak CBR for all the percentage of replacement of calcium carbide using the three energy level. This is could be that the unsoaked soil specimen was partially saturated and having high suction pressure leads to high pressure between the lateritic particles results high CBR value.5.2

5.2 Recommendations

1. Geotechnical properties of this soil should be investigated before undertaking any engineering construction on it.
2. Stabilisation with additives like Calcium carbide waste is recommended to improve the bearing capacity of the lateritic soil.
3. Cement and CCW should be used to stabilize an unsuitable A-7-6 lateritic soil.
4. For long strata of such lateritic clay soil, total comaplete removal and repalcement is recommended, such that the subabse may be laid on the hard bed or employ the use of high density for compaction before the placement of any engineering structures.

5.3 Contribution to Knowledge

From the study it was proven that the compaction characteristics, unconfined compressive strength and California bearing ratio of the lateritic soil improved with variation in CCW addition, thereby improving the bearing and share parameters of the stabilised soil, also the outcome of this research is a validation of literatures presented in this study as regarding stabilisation of soil using calcium carbide.

REFERENCES

AASHTO. (1986). Guide Specifications for Design and Construction of Segmental Concrete Bridges, First Edition, GSCB-1. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. (2012). Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 31st Edition, HM-31. American Association of State and Highway Transportation Officials, Washington, DC.

Abdulkarim, I. I., Sa‟eed Y. U. & Suleiman A. Y. (2021). Performance evaluation of the effect of metakaolin on strength properties of non-lateritic soil. *International Journal of Engineering and Technology Research.* 3 (1); 1-25. ISSN-2329-7309 (Print)

Abdullah, H. H., Shahin, M. A. & Sarker, P. (2018). Use of Fly-Ash Geo-polymer Incorporating Ground Granulated Slag for Stabilisation of Kaolin Clay Cured at Ambient Temperature, *Geotechnical and Geological Engineering. (37)2.* Doi: 10.1007/s10706-018-0644-2

Abeele, W. V. (1985). Consolidation and Shear Failure Leading to Subsidence and Settlement Part I, November 1985, Los Alamos National Laboratory, Los Alamos., New Mexico 87545.

Abeles, F. B. & Gahagan, H. E. (1968). Abscission: The Role of Ethylene, Ethylene Analogues, Carbon Dioxide, and Oxygen. Plant Physiol. 43 (8): 1255–1258. doi:10.1104/pp.43.8.1255. PMC 1087003. PMID 16656908.

Afrin, H. (2017). A Review on Different Types Soil Stabilisation Techniques. *International Journal of Transportation Engineering and Technology*. 3(2), 19-24. doi: 10.11648/j.ijtet.20170302.12

Agbede, I. O, & Joel, M (2011). Effect of carbide waste on the properties of Makurdi shale and burnt bricks made from the admixtures. *American Journal of Scientific and Industrial Research.* 2(4):670-673, Doi: 10.5251/ajsir.2011.2.4.670.673

Agbede, O. A., Jatau, N. D., Oluokun, G. O. and Akinniyi, B. D., (2015), Geotechnical investigation into the causes of cracks in building: A case study of Dr. Egbogha Building, University of Ibadan, Nigeria., International Journal of Engineering and Science Invention, 4 (11), 18-22.

Akayuli, C., Ofosu, B., Nyako, S. O. & Opuni, K. O., (2013), The influence of observed clay content on shear strength and compressibility of residual sandy soils., *Int J Eng Res Appl.*, 3 (4), Jul-Aug, 2538-2542.

Akinwumi, I. I., Ajayi, M. C., Agarana, M. C., Ogbiye, S. A., Ojuri , O. & David, A. O. (2019) Investigation of calcium carbide residue as stabilizer for tropical sand used as pavement material. In: 24th *International Conference on Urban Transport and the Environment*, 19(21), 285-294.

Amadi, A. A. & Okeiyi, A. (2017). Use of quick and hydrated lime in Stabilisation of lateritic soil: comparative analysis of laboratory data. *International journal of Geo-Engineering* , 8:3. DOI 10.1186/s40703-017-0041-3.

Apparao, K. V. S. & Rao, V. C. S. (1995). Soil Testing Laboratory Manual and Question Bank, Universal Science Press, New Delhi.

Ardani, A., (1992). Expansive Soil Treatment Methods in Colorado". Colorado Department of Transportation.

Armani, A. & Moonfish, G.A., (1972), "Lime Stabilisation of Organic Soils", *Highway Research Record*, (381), 37-45.

Arora, K. R. (2011).Soil Mechanics and Foundation Engineering (Geotechnical Engineering), *Standard Publishers Distributors, Delhi.*

ASTM. (1992). Specification for Fly Ash and Raw Calcined Natural Pozzolana for use as a Mineral Admixture in Portland Cement Concrete. *American Society forTesting and Materials*, Philadelphia.

Balarabe, W. I. & Sharmila S. R. (2015). Soil Stabilisation Using Calcium Carbide Residue and Coconut Shell Ash. *Journal of Basic and Applied Engineering Research*, 2(11); 1039-1044. Print ISSN: 2350-0077; Online ISSN: 2350-0255.

Baser, O., (2009), Stabilisation of Expansive Soils Using Waste Marble Dust. Behzad, K. and Huat, B. B. K. (2008): Peat Soil Stabilisation using Ordinary Portland Cement, Polypropylene Fibers, and Air Curing Technique, *Electronic Journal of Geotechnical Engineering*. 13(1), 1-19.

Bell, F. G., (1993). Engineering Geology. Blackwell Scientific: London, Uk.

Bello, A. A & Adegoke, C. W. (2010). "Evaluation of Geotechnical Properties of Ilesha East South West Nigeria"s lateritic Soil." *Pacific Journal of Science and Technology*. 11(2) 617-624.

Bowles, J. E., (2012). Engineering Properties of Soils and their Measurements, 4th edition, McGraw Hill Education (India) Private Limited, New Delhi, 2012.

BS (British Standard), 1377-2. (1990). Methods of test for Soils for Civil Engineering purposes. ISBN 0 580 17867 6.

Canada, Building Material, (2010). Portland cement material safety data sheet. Published report. Canada.

Chava, A. Sreenatha S. R., Vasudeva M. N. & Madan S, M. (2016). Comparative study on analysis and design of flexible and rigid pavements. *International journal of innovative technology and research*. (IJITR). 4(5); 3797-3802.

Cockrell, C. F. & Leonard, J. W. (1970). Characterization and Utilization Studies of Limestone Modified Fly Ash," *Coal Research Bureau*,. 2(60), 14-22.

Dafalla, M. A., (2013). Effects of clay and moisture content on direct shear tests for clay-sand mixtures, *Advance Material Science and Engineering*, Volume 2013, http://dx.doi.org/10.1155/2013/562726, 1-8.

Day, R. W., 2000. Geotechnical Engineer's Portable Handbook, McGraw-Hill Professional, New York.

Dun, Y. A. (2006). Troubles in the PVC industry. Hong Kong Trade Development Council. Archived from the original.

Durgunoglu, H. T., Varaksin, S., Briet, S. & Karadayilar, T. (2003). A case study on soil improvement with heavy dynamic compaction. *European Conference on Soil Mechanics and Geotechnical Engineering*. 13(1), 651-656.

Ekeng,E. E., Bejor, I. E. and Ibiang, I. E.(2016). The effect of geotechnical properties on civil engineering structures in cross river state, south-south region of Nigeria. *International Journal of Scientific and Engineering Research*,7(8). ISSN 2229-55

El-Maksoud, M. A .F. (2006). Laboratory determining of soil strength parameters in calcareous soils and their effect on chiselling draft prediction, *Proc. Energy Efficiency and Agricultural Engineering, Int. Conf.*, Rousse, Bulgaria.

Ersoy, H., Karsli, M.B., Cellek, S., Kul, B., Baykan, I. and Parsons, R. L., (2013). Estimation of the soil strength parameters in Tertiary volcanic regolith (NE Turkey) using analytical hierarchy process., *J. Earth Syst. Sci*., 122 (6), 1545–1555.

Fang, H. Y., (1991), *Foundation Engineering Handbook*, 2 ed., Van Nostrand Reinhold Publishing Company, New York.

Federal Ministry of Works & Housing (FMWH), (1997). General Specifications for Roads and Bridges. Volume II. Federal Highway Department, Lagos, Nigeria..

Greenwood, N. & Earnshaw, A. (1997). Chemistry of the Elements (2nd ed.). Butterworth-Heinemann. p. 298. ISBN 978-0-08-037941-8.

Greeshma, N. E., & Lamanto T. S. (2015). Enzyme Stabilisation of Organic Clay. Published Proceedings, *5th Young Indian Geotechnical Engineers Conference* March 14-15, 2015 Vadodara,-India.

Hassnen, M. J. (2013). Stabilisation of Soft Soils Using Salts of Chloride. *Journal of Babylon University/Engineering Sciences/* (5)21

Hoffman D. J. & Eastin W. C. (1981). Effects of Industrial Effluents, Heavy Metals, and Organic Solvents on Mallard Embryo Development. Toxicology Letters. 9(1), 35-40.

Horpibulsuk, S. Suddeepong, A. Chamket, P & Chinkulkijniwat, A. (2012). Compaction behaviour of fine-grained soils, lateritic soils and crushed rocks., *The Japanese Geotechnical Society.*

Igwe, C. N., Oyelola, O. T., Ajiboshin I. O., & Raheem S (2013). A Review: Nigeria‟s Transportation System and the Place of Entrepreneurs. *Journal of Sustainable Development Studies*, 2(3), 168-180

Ikeagwuani, C. C. (2016). Compressibility characteristics of black cotton soil admixed with sawdust ash and lime. *Nigerian Journal of Technology* (NIJOTECH). 35 (4); 718 – 725.

Ingles, O. G. & Metcalf, J. B. (1972), Soil Stabilisation, Butterworth property, Ltd., Australia.

IS: 2720 – Part 5, (1970). Determination of liquid and plastic limits, BIS, New Delhi.

IS: 2720 – Part 6, (1972). Determination of shrinkage factors, BIS, New Delhi.

IS:2720 – Part 14, (1983) Determination of density index (relative density) of cohesionless soils, BIS, New Delhi.

Ito, M., & Azam, S. (2010). Determination of Swelling and Shrinkage Properties of Undisturbed Expansive Soils. *Geotechnical and Geological Engineering*, 28, (4), 413-422.

Jaiswal, M. & Lai, B., (2016).Impact of Rice Husk Ash on Soil Stability (Including Micro Level Investigation), *Indian Journal of Science and Technology*, 9(30) 1-7.

Jaturapitakkul, C. (2013). Use of calcium carbide residue and bagasse ash mixtures as a new cementitious material in concrete,

Jawad, I. T., Taha, M. R., Majeed, Z. H. and Khan, T. A. (2014). Soil stabilisation using lime: Advantages, disadvantages and proposing a potential alternative, *Research Journal of Applied Sciences, Engineering and Technology*, 8 (4): 510-520.

Jeb S.T., & Rose L. S, (2007). Stabilisation of clay soils with Non-traditional additives. Published paper.

Joel, M. & Edeh J. E. (2014) Researched on the Stabilisation of Ikpayongo Laterite with Cement and Calcium Carbide Waste, *Global Journal of Pure and Applied Sciences* 20(1), 49-55

Joel, M. & Edeh J. E. (2016). Calcium Carbide Residue-Hydrated Lime Blend as Stabilizing Agent, *International Journal of Science and Technology* 5 (12), 18-27

Joel, M, & Edeh, J. E. (2013). Soil Modification and Stabilisation Potential of Calcium Carbide Waste. *Advanced Materials Research* (824), 29-36

Jones, L. D. & Jefferson, I. (2012). *Expansive Soils.* ICE Publishing,. 413-441.

Kaniraj, S. R. (1988). Design Aids in Soil Mechanics and Foundation Engineering, McGraw Hill Education (India) Private Limited, New Delhi, 1988.

Kim, S., Gopalakrishnan K. & Ceylan H. (2012). Moisture Susceptibility of Subgrade Soils Stabilized by Lignin-Based Renewable Energy Coproduct." *Journal of Transportation Engineering*. 138(11), 1283–1290.

Laskar, A. & Pal, S. K., (2012), Geotechnical characteristics of two different soils and their mixture and relationships between parameters. *Electronic Journal of Geotechnical Engineering*, (17), 2821-2832.

Lee, D. J, & Ian, J. (2015). Expansive Soils. Institution of Civil Engineers Manuals series

Lewis, R. J. (1993). Hawley‟s Condensed Chemical Dictionary, Twelfth edition, New York.

Little, D. N., Males, E. H., Prusinski, J. R., & Stewart, B. (2010): Cementitious Stabilisation;

Little, D. N., Thompson, M.R., Terrell, R.L., Epps, J.A. & Barenberg, E. J. (1987), Soil Stabilisation for Roadways and Airfields, Final Report, Submitted to the Air Force Engineering and Services Center, Tyndall Air Force Base, FL.

Liu, C. & Evett, B.J. (1998). Soils and Foundations, 4th edition, Prentice-Hall, Inc, United States of America.

Liu, S., Lu, Y., Weng, L., & Bai, F. (2015). "Field Study of Treatment for Expansive Soil/rock Channel Slope with Soilbags". *Geotextiles and Geomembranes*, 43(4), 283-292.

Lockett, L. W., & Moore, R. K. (1981). Lime-Soil Mixture Design Considerations for Soils of Southeastern United States. Transportation Research Record , 839, 20-25.

M‟Ndegwa, J. K. (2011).The Effect of Cane Molasses on Strength of Expansive Clay Soil. *Emerging Trends in Engineering and Applied Sciences*. 2 (6), 1034-1041

Makusa, G. P, (2012). Soil Stabilisation Methods and Materials, In Engineering Practise, Mining and Geotechnical Engineering, Lulea University of Technology, Lulea, Sweden.

Mallo, S. J. & Umbugadu, A. A., (2012). Geotechnical study of the properties of soils: a case study of Nasarawa – Eggon town and Environs, Northern Nigeria., *Canadian Journal of Earth Sciences.,* 7 (1), 40 – 47.

Metzler, S. C., & Jarvis C. (1985). Effects of Waste Oil Contamination. *Journal of Environmental Progres*s. 4(1), 61-65.

Minten, B. & Kyle, S. (1999). Effect of distance and road quality on food collection, marketing margins, and traders' wages: Evidence from the former Zaire. *Journal of Development Economics*, 60(2), 467–495. DOI: 10.1016/s0304-3878(99)00049-8.

Mitchell, J. K., (1986). Practical Problems from Surprising Soil Behavior. *Journal of Geotechnical Engineering Division*, ASCE, 112, (3); pp. 259-289.

Mollahasani, A., Alavi, A. H., Gandomi, A. H. & Rashed, A. (2011). Nonlinear neural-based modelling of soil cohesion intercept. KSCE J CIV ENG., 15(5), 831-840.

Moore, D. M. & Reynolds, R. C., Jr. (1997). X-Ray Diffraction and the Identification and 630 Analysis of Clay Minerals. . Oxford University Press, Oxford.

Murthy, V. N. S., (2002). Principles of Soil Mechanics and Foundation Engineering, UBS Publishers‟ Distributors Ltd., New Delhi.

Naik, S., Naik, N. P., Kandolkar, S.S. & Mandrekar, R.. L. (2011). Settlement structure, A case study. *Indian Geotechnical Conference, Kochi*, 1031-1034.

National Lime Association (2004): Lime-treated Soil Construction Manual: Lime Stabilisation and Lime Modification. National Lime Association, Bulletin 326, USA.

Nicholas, J. Garber R. & Lester A. H. (1999). *Traffic and highway engineering,* 4th Edition

Nigeria National Bureau of Statistics, Transport Statistic; National Bureau of Statistics. Online. www.nigerianstat.gov.ng/pdfuploads/Transport.pdf. Accessed on: 26 Apr. 2017.

O‟Neill, J. M. (2011). Molasses used in cleanup of polluted sites. Published report. North-Jersey. Cited on 01/11/2012.

Oke, S.A. & Amadi, A.N., (2008). An assessment of the geotechnical properties of the sub-soil of parts of Federal University of Technology, Minna, Gidan Kwano Campus, for foundation design and construction., *Journal Science Education Technology*, 1 (2), 87 – 102.

Onyelowe, K. C., Duc B. V., Clifford I., Orji F. & Ugwuanyi. H. (2019). Rheology of mechanical properties of soft soil and Stabilisation protocols in the developing countries-Nigeria. KeAi Global Imaptacts. 2(1); 8-14.

Onyelowe, K. C. (2011). Geosynthetics and Geotechnical Properties of Soil in a Developing World; A Lesson for Nigeria, *Electronic Journal of Geotechnical Engineering*, USA, 16(1), 1481-1487.

Onyelowe, K. C. (2012). Soil Stabilisation techniques and procedures in the developing countries-Nigeria. *Global Journal. of Engineering. and technology.* 5(1), 65-69

Orts W., Roa-Espinosa A., Sojka, R., Glenn G., Imam S., Erlacher K., and Pedersen J. (2007).Use of Synthetic Polymers and Biopolymers for Soil Stabilisation in Agricultural, Construction, and Military Applications. *Journal of Material in Civil Engineering*. 2(19); 58–66.

Osama, K. N., Ahmed A. A. & Rasheed A.J. (2008). Determination of saline soils specific gravity. *Jordan Journal of Civil Engineering.* 2(1); 1-19.

Osinubi, K. J & Nwaiwu, M. O. C., 2006. Design of compacted lateritic soil liners and covers. *Journal of Geotechnical and Geo-environmental Engineering*. 132, (12): 203-213.

Oyediran, I. A. & Durojaiye, H. F. (2011). Variability in the Geotechnical properties of some residual clay soils from south-western Nigeria. *International Journal of Scientific & Engineering Research*. 2(9); 235-240. ISSN 2229-5518 IJSER. http://www.ijser.org

Phanikumar, B. R. Amshumalini, C. & Karthika, R (2009). Effect of lime on engineering behaviour of expansive clays. IGC 2009, Guntur, India

Phil P. G. (2014). Sustainability Issues Surrounding unpaved roads. Published document, South Africa.

Poulos, S. J, (1989). Liquefaction Related Phenomena; In: Advanced Dam Engineering for Design Construction and Rehabilitation Van Nostrand Reinhold (ed.) Jansen R. B., pp. 292–320.

Prakash, S. & Jain P. K., (2002). Engineering Soil Testing, Nem Chand and Bros, Roorkee.

Quaint, B. S., Seksinsky, E. J. & Li, J., (2000). Incorporating Subgrade Lime Stabilisation into Pavement Design, Transportation Research Record, No. 1721, TRB, *National Research Council*, Washington, D.C., 3-8.

Raj, P. P., (2012). Soil Mechanics and Foundation Engineering, Dorling Kindersley (India) Pvt. Ltd., New Delhi.

Ravi, E., Sharma A., Manikandan A.T, Kaarthick G., & Jameel A. A. (2015). Effect of molasses on strength of soil. *International Journal of Advanced Research Trends in Engineering and Technology* (IJARTET). 2,(2); 19-28.

Rees, S. W. & Thomas, H. R. (1993). Simulating seasonal ground movement in unsaturated clay. *Journal of Geotechnical Engineering, 119*, (7), 1127-1143.

Robert M, B. (2009), Soil Stabilisation with Fly ash and Rice Husk Ash. *International Journal of Research and Reviews in Applied Sciences*, 1 (3) 209-217.

Rogers, C. D. F. & Glendinning, S. (1993). Modification of clay soils using lime. In C. a. Rogers (Ed.), *Proceeding of the Seminar held at Loughborough University on Lime Stabilisation* 1, 99-114).

Roy, S. & Dass, G., (2014). Statistical models for the prediction of shear strength parameters at Sirsa, India. I*nternational Journal of Civil and Structural Engineering*., 4(4), 483-498.

Roy, S., (2016). Assessment of soaked California Bearing Ratio value using geotechnical properties of soils, *Resources and Environment*, 6(4), 80-87.

Shanyoug, W., Chan, D. & Lam, K.C., (2009). Experimental study of the fines content on dynamic compaction grouting in completely decomposed granite of Hong Kong., Constr Build Mater., 23, 1249 -1264.

Sharana, K., Ullas J, &, Yathiraju H. M. (2018). Soil stabilisation by utilising phosphogypsum and calcium carbide residue. *International Research Journal of Engineering and Technology* (IRJET) e-ISSN: 2395-0056. 5(6) 1688-1692 p-ISSN: 2395-0072.

Sharana, K., Ullas J. & Yathiraju H. M. (2018). Soil Stabilisation by utilizing phosphogypsum and calcium carbide residue. *International Research Journal of Engineering and Technology*. 5(6). 1688-1692

Sherwood, P. (1993). Soil Stabilisation with cement and lime. State of the Art Review. London: *Transport Research Laboratory*, HMSO.

Shirsavkar, S. S. (2010). Innovation in Road Construction. Published paper. Government Polytechnic, Beed–India.10p. http://www.ejge.com/2010/Ppr10.112/Ppr10.112a2r.pdf. Cited on 11/11/2012

Stocker, P. T. (1972). Diffusion and Diffuse Cementation in Lime and Cement Stabilized Clayey Soils, Special Report 8, *Australian Road Research Board*, Victoria, Australia.

Tamadher, T. A., Anuar B. K., Zamri B. C. (2007). Stabilisation of Silty Clay Soil using Chloride Compounds. *Engineering Science and Technology*. 2(1):102-110

The History of Air Force Civil Engineers, (2012) 1907-2012. Department of Defense (DOD), Department of the Air Force, ISBN: 9780160925344

Thompson, M. R., (1969), Engineering Properties of Lime-Soil Mixtures, *Journal of Materials*, ASTM, 4 (4), 968-969.

Tingle, J. S., & Santoni, R. L. (2003). Stabilisation of Clay Soils with Non-traditional Additives. *Transportation Research Board*. 2(1819):72–84.

Tuncer, E. R. & Lohnes, R. A., (1977). An engineering classification for basalt-derived lateritic soils. Eng. Geol., 4, 319– 339.

Van-Rooy, J. L. & Oyelami C. A. (2016). A review of the use of lateritic soils in the construction/development of sustainable housing in Africa: *A geological perspective- Journal of African Earth Sciences*. 119 (1) 226-237. https://doi.org/10.1016/j.jafrearsci.2016.03.018.

Walker, G. J. (1959), Traffic and Transport in Nigeria; The Example of an Under Developed Tropical Territory. Her Majestey's Stationary Office, London. 1959.

Widdowson, M. (2009). Evolution of Laterite in Goa. IN MASCARENHAS, A. A. K., GLENN (Ed.) *Natural Resources of Goa: A geological perspective.* . India: Geological Society of Goa.

World Bank: Washington DC, (2011). The International Bank for Reconstruction and Development, The World Bank, Railway Reform: Toolkit for Improving Rail Sector Performance, The International Bank for Reconstruction and Development, Transportation in the New Millennium, Louisiana, USA.

Yagiz, S., (2001). Brief note on the influence of shape and percentage of gravel on the shear strength of sand and gravel mixture. Bull. Eng. Geol. Environ., 60(4), 321-323.

Yoder E. J. & Witczak, M. W. (1975). Principle of Pavement Design., Materials Characterization, John Wiley and Sons, Canada, pp.253

Zemenu, G., Martine, A., & Roger, C. (2009). Analysis of the Behaviour of a Natural Expansive Soil Under Cyclic Drying and Wetting. *Bulletin of Engineering Geology And The Environment.*

Zhang, M., Guo, H., El-Korchi, T., Zhang, G. & Tao, M. (2013). Experimental feasibility Study of Geopolymer as the Next Generation Soil Stabilizer, Construction and Building Materials. 47, 1468-1478.

Zubair S., Lawalenna S., Tri H., & Johannes, P. (2017). Mechanical Characteristic of Ferro Laterite Soil with Cement Stabilisation as a Subgrade Material. *International Journal of Civil Engineering and Technology*. 8(3); 609–616.

APPENDICES