STRENGTH ASSESSMENT OF CALCIUM CARBIDE RESIDUE STABILIZED CLAY ADMIXED WITH ZEOLITE

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ABSTRACT

Clay soil collected at Niger State Polytechnic, Zungeru, was characterized and classified as A-7-6 (GI- 12) CH soil according to American Association of State Highway and Transportation Officials (AASHTO) soil classification system. The clay was treated with 5% Calcium Carbide Residue (CCR) and was admixed with 2% zeolite. Unconfined compressive strength (UCS) test using 40 samples each was used as evaluation criteria to measure the effect of zeolite on CCR stabilized clay soil. Results showed significant increase in UCS with increase in zeolite and CCR. In the absence of zeolite, the mean UCS was 337.15 kN/m^2 all at 28 days curing strength, at the addition of zeolite into the CCR stabilized soil a sharp increase in the mean UCS was noticed it increased to 704.23 kN/m^2 at 28 days curing. Standard deviation was calculated for both CCR stabilized soil samples and CCR admixed zeolite soil samples as 12.5 and 12.27 which shows a deviation towards accuracy for the zeolite stabilized soil samples. A confidence limit analysis was also carried out to establish a performance range of soil strength using the mean and standard deviation calculation. 95% Confidence limit for the mean value of strength of soil stabilized with 5% CCR and 0% zeolite was 381.02kN/m2 upper bound and 373.28 kN/m^2 lower bound while the 99% confidence limit was 382.25 kN/m^2 upper bound and 372.05kN/m2 lower bound. When the confidence limit for the 2% zeolite admixed CCR stabilized soil was calculated, for 95% confidence limit 708.03 kN/m^2 upper bound and 700.42 kN/m^2 lower bound and for 99% confidence limit 709.23 kN/m^2 upper bound and 699.22 kN/m^2 lower bound. This values show a certainty of obtaining similar results for the range of the strength UCS values tested of any given soil samples of the same nature as described in the index properties of the clay studied. Confidence limits tests for the standard deviation of the set of samples was computed and as presented in the result shows the allowable range for both 95% and 99% values for both of the sample mixes.

TABLE OF CONTENTS

CHAPTER ONE

CHAPTER TWO

CHAPTER THREE

CHAPTER FOUR

CHAPTER FIVE

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

An engineer refers to soil as any uncemented aggregation of inert particles formed by the disintegrating of rocks that retains air spaces in particles that are saturated by water or air (Craig, 1998). Weathering processes that generate (decomposition of ferro-alumino silicate minerals, leaching of the linked silicia and base, and permanent deposition of sequioxides within the profile) produce clay, a type of leached soil found in the humid tropics (Osinubi, 2003).

There are a lot of clay gravels and pisoliths in tropical nations like Nigeria, are suitable for gravel roads (Osinubi and Bajeh, 1994). A clay's strength and stability under load, especially in the presence of moisture, cannot be guaranteed if it contains a considerable proportion of clay elements. Many tropical areas, including Nigeria, have numerous clay variations, and it is typically more cost-effective to improve existing soil rather than seek out alternative soil (Mustapha, 2006). Meanwhile, soil enhancement could be accomplished by alteration, stabilization, or a combination of the two. Soil stabilization, on the other hand, is the process of improving the strength of soils. Regardless of their classification, they have the durability to the point where they are totally suitable for building.

Some studies believe that clay soils are good road construction materials, however, (Umar and Elinwa, 2020) stated that not all deposits are suitable as road construction materials in their natural state as such there is need to treat such clay soil to attain desired properties. This research work focuses on the strength capacity of lateritic soil after treatment. Due to the high clay concentration in the soil, it has a limited bearing capacity in its natural state, which does not ensure its performance, especially in the presence of moisture and continuous load (Alhassan *et al.* 2008)

1.2 Statement of the Research Problem

Clay has been a widely used construction material in nearly all countries in the world's wet tropics since its discovery near the end of the nineteenth century. For all engineering structures constructed on clayey soils, deformations occur when these soils are exposed to any additional load or due to dissolution of clay when these soils are wet, soaked or leached with water (Kawther *et al*. 2018). In order to attain the desired strength and stability, it will be essential to stabilize the available clay.

Although calcium carbide residue has been shown as a good additive for stabilizing poor soils, (Prinya *et al*. 2020) it allows for a strength gain on the soil property, limiting its applicability to low-traffic roadways. Zeolites show pozzolanic activity when combined with diverse geological materials due to their surface area and porosity in their arrangement, which boosts the cementing property of stabilized soils (Dunn *et al*. 1980). As a result, the viability of zeolite use on the strength property of CCR stabilized soil was investigated in this study. A statistical methodology was employed to compare the fluctuation in effect of various additives as they affect the soil compressive strength.

1.3 Aim and Objectives of the Study

The purpose of this study was to evaluate the effect of zeolite on the strength property of calcium carbide residue stabilized soil. To achieve this, the following objectives were considered.

1. Determination of the untreated soil's index qualities.
2. Determination of the untreated soil's compaction characteristics at a standard proctor compaction energy level.
3. Evaluation of the effect of admixture content on the Unconfined Compressive Strength of CCR-treated soil and CCR-and-zeolite-treated soil.

4. Determination of the influence of calcium carbide residue and zeolite admixture on strength using statistical method.

1.4 Scope of the Study

This work was limited to the clay soil collected at Niger State Polytechnic, Zungeru, Niger State, Nigeria. The optimum moisture content (OMC) and maximum dry densities (MDD) of the mix ratio were determined using standard compactive effort. The soil was split in to two, one of which is stabilized with 5% calcium carbide residue alone while the other was stabilized with a blend of 5% calcium carbide residue and 2% zeolite. Forty (40) specimen each of the two groups of the mixture were casted and cured for 28 days.

For the mixture of clay sample and 5% CCR and stabilized soil admixed with 2% zeolite, the curing time for the unconfined compressive strength (UCS) test was limited to 28 days. However, the following tests were conducted out on natural clay soil in accordance with BS 1377 (1990). The tests include:

i. Sieve Analysis

ii. Atterberg Limits test

iii. Compaction

iv. Unconfined Compression

The study majorly featured a statistical method which compared the effect of the strength property on the soil stabilizer mixture admixed at a specific ratio with a new material to ascertain its efficiency.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Clay Soil

Clay minerals are called secondary silicates, because they are formed from the weathering of primary rock-forming minerals. Clay minerals occur in small particle sizes (<0.002 mm) and are very fine grained and flake shaped; they are separated from sand, gravel and silt due to the negative electrical load on the crystal edges and positive electrical load on the face. Clay minerals consist of two basic structures. First, silica oxygen is formed through the bonding of silicon ions to the oxygen atoms on all four sides (tetrahedron). Second, an octagon forms with aluminum and magnesium ions coordinated on eight-sides with oxygen and hydroxyl ions (octahedron). All clay minerals are formed from octahedral and tetrahedral sheets with certain types of cations, which are in various forms and connected to each other in a certain system. Changes in the structures of the octahedral and tetrahedral sheets result in the formation of different clay minerals (Holtz and Kovacs, 1981). More common clay mineral groups include kaolinite, illite and smectite (montmorillonite). Kaolinite consists of silica and alumina plates, and these plates are connected very strongly, because kaolin clay is very stable. Illite has layers made from two silica plates and one alumina plate. However, illite contains potassium ions between each layer; this characteristic makes the structure of the clay stronger than smectite. Smectite has layers made from two silica plates and one alumina plate. Because there is a very weak bond between the layers, large quantities of water can easily enter the structure, this event causes the swelling of such clay (Al-Khafaji & Andersland, 1992).

2.2 Properties of Clays

Certain features of the clay affect the structure of the soil, which determines its properties such as strength, hydraulic conduction, settlement and swelling. These features include isomorph substitution and surface anion and cation exchange capacity. This event is called isomorphic substitution if the octahedral or tetrahedral sites are replaced by a different atom normally found elsewhere. The specific surface area is the property of solids, which is defined as the total surface area of a material per unit of mass. With the separation of hydroxyl ions from the clay surface, which results in crystal deficiency at the crystal head, anions subsequently attach to the surface and organic molecule content causes an electrical load imbalance. This imbalance results in clay's extreme affinity to water and cations in the environment. Water is a dipolar molecule, namely, it has one positive and one negative charge. The surface of the clay crystal is electrostatically held to the water molecule. In addition, water is held to the clay crystal by hydrogen bonding. Also, negatively charged clay surfaces attract cations in the water. The cation/anion changes in the clay minerals are different between clay minerals. Therefore, it is expected that the clay that attracts more water molecules to the surface will have more plasticity, more swelling/shrinkage and more volume change, depending on the load on it. Thus, water influences clay minerals. For example, the water content changes consistency limits and this affects the ground plasticity. Ultimately the change in clay plasticity directly affects the mechanical behaviour of the soil. Studies generally accept clays as fully saturated in geotechnical engineering. Therefore, the behaviour of clays is affected by the individual clay particle arrangements and pore water content. The surfaces of clays are negatively charged, and so they tend to adsorb the positively charged cations in pore water. In this way, the cations on the surface of a clay particle that are entering the water spread into the liquid. This spreading is called the double layer. Briefly, the cations are distributed around the negatively charged surface of the clay particles, with the greatest density near the surface and decreased density with increasing distance from the surface. The cations form a positively charged layer and the double layer is created with a negatively charged surface of the clay particles. The double layer affects the arrangements of the clay particles, and hence, the physical and mechanical properties of the soil are also affected (Dunn *et al*. 1980). The interaction of these forces controls the engineering behaviour of soils to a great extent. At the same time, this interaction leads to the formation of different compositions and settlements in the soil planes, which are defined as structures in clay soils (Holtz, 1981). Environmental temperature, precipitation, groundwater level and pH and salinity all play roles in clay properties, as well as in the conversion of rock into clay.

Clay derived from the same rock can be different under different environmental conditions around the clay that is faced with a liquid, there are distance-varying push-pull curves. If there is a force lifting the two clay minerals, the particles clump together. This is called flocculation. If the net force is thrust, the particles are separated from each other; this is called dispersion. Particle orientations of soils vary between flocculated and dispersed. Forces between the particles are important for clay, because the behaviour of clay depends on the geological history and structure. This difference in the orientation of fine-grained soils affects the engineering behaviour of the soil. The geological process in the formation of soils in nature constitutes the arrangement of soils. For this reason, geotechnical engineering studies are interested in the physical and mechanical behaviour of soils-bearing structures, as well as the strength between the structure, texture and performance of soils. There are many studies on the soil-orientation effects on the soil properties such as strength, hydraulic conductivity and the swelling-shrinkage relative to each particle (Ingles, 1968; Norsyahariati *et al.* 2016). Ingles, (1968) examined soil fabric during consolidation. Due to an increase in the degree of particle orientation, the total volume of the voids was reduced.

Flocculation increases depending on the electrolyte concentration, ion valence, temperature, decreasing dielectric constant, hydrated ion diameter, pH value and surface absorbed ions. Soil engineering properties depend on the size, shape, a high amount of surface area and a negative surface charge of clay particles. In 1925, Terzaghi proposed the concept of clay arrangement. He said that clay minerals stick to each other at the points of contact, with forces sufficiently strong to construct a honeycomb structure. In 1932, it was shown by Casagrande that this honeycomb shape is a special structure in clay-containing soils and this structure can vary depending on many characteristics of the environment (Holtz, 1981). Compression progresses with soil sedimentation later, other researchers also proposed fabric models (Lambe, 1953; Tan, 1957; Push, 1973; Yong, 1973). Collins and McGown, (1974) defined the elementary particle arrangement, particle assemblages and pore spaces in the fabric model. Researchers provided an elementary particle arrangement, a single clay, silt or sand, and the interaction between silt and sand particle assemblages containing one or more elementary particle arrangements or small particle clusters.

Pore spaces are defined with spacing between elementary particle arrangements and particle assemblages. Bennet and Hulbert, (1986) suggested that the fabric of soils is mostly determined by the physical arrangement of particles, which is gained at the time of sediment deposition by the physical–chemical conditions of the depositional environment. The fabrics of soils describe clusters, clusters are formed by other clusters and space between the clusters and structure of the soils describes the fabric, mineral content and decontamination forces. Also, the fabrics of soils can sometimes be visualized under a microscope. The structure of soils can be examined more extensively by an X-ray diffractometer (XRD) and a scanning electron microscope (SEM).

2.3 Stabilization

Various scholars described stabilization in their own terminology but with the same meaning: (Gillout 1968, Gidigasu 1976, AFMAN 1994, Toro 1997) defined stabilization as the mixing of soils and other additions in such a way that, when it is applied, it is stable so would undergo material change in its qualities under given conditions and to a specified extent, and would remain in its stable compacted state without suffering any change under the action of weather and traffic. Strength, permeability, volume stability, and durability are the essential attributes in engineering practice that need to be improved. (Toro, 1997). Stabilization can be applied to a wide range of engineering services. These include the following.

a. Foundation: Ensure volume stability, limit permeability, or increase strength to reduce settlement or heave under buildings (Toro, 1997)
b. Excavation works: By reinforcing the surrounding soils or altering their permeability, support can be provided in pits, trenches, tunnels, and other structures. Accelerated drainage and grouting are the most common techniques used.
c. Pavement construction: It offers highways, airports, and railways with stronger and longer-lasting pavements. Mechanical or chemical stabilization are the main ways employed.
d. Slope Stability: On cut slopes, embankments, and natural slopes, this is to prevent slides. Drainage, surface sealing, and other similar methods of application were described by Anderson and Richard, (1989), to have been a long-standing tradition in Hong Kong. China, a soil cement plaster, was commonly used to protect slopes from infiltration and erosion. One part cement, three parts hydrated lime, and twenty parts soil and water are mixed to the desired consistency and then applied to the slope's surfaces.

e. Water Retention: Lime treatment to minimize erosion or reduce permeability of the soil are popular ways in use to protect structures from water erosion, especially for dams, tanks, canals, and other structures.

f. Environmental conservation: Increase the resistance of soil to natural weathering from wind or water to avoid erosion, battle dusting of road surfaces, and so on. The use of stabilized clay liners to prevent pollution of ground water from waste deposited on the ground surface has evolved into a new branch of environmental geotechnics. The stabilization is mostly by cement, lime, and other materials (Kerry *et al*. 1995)

g. Reduced Thickness: AFMAN, (1994) is used to reduce the thickness of a pavement below what it would have been if it hadn't been stabilized because CBR curves are widely used in pavement design, stabilizing the soil to increase its strength and stability can help to lower the pavement's thickness.

Various stabilizing processes or technologies have been tried throughout the previous four to five decades. Portland cement, hydrated lime, gypsum, alkalis, sodium chloride, calcium chloride, silicate resins, ammonium compounds, and agricultural and industrial waste products are examples of mechanical and chemical additives. Portland cement, lime, bitumen, and agricultural waste are the most extensively used chemical additives in most developing countries due to their low cost and/or availability.

2.3.1 Mechanical stabilization

Mechanical stabilization is the first and most fundamental of all stabilization procedures, according to (Gidigasu, 1976). Some research such as (Gidigasu, 1976, Ola, 1983, Singh, 1991, and AFMAN 1994). Have all characterized mechanical stabilization in their own terms as the process of mixing adequately proportioned soil aggregates with some binder soils, such as clay, and compacting the combination to a stable layer. Proportioning and compaction are the fundamental principles of mechanical stability. Singh, (1991) due to the lack of cohesiveness between the aggregates, aggregate soil mixtures with no or a small amount of fines can only be stable in limited conditions. The permeability of these materials will be high with the addition of some binder, mechanical stabilization of these materials will have a high permeability. Even under unconfined conditions, mechanical stabilization of these components with the addition of additional binder soils like clay would improve the mixture's strength and stability. Furthermore, where a soil deposit consists primarily of fine materials, the compacted aggregate grains will not be in contact with one another, reducing the soil's stability. This can also be rectified by adding a sufficient quantity of coarse aggregate to the soil manually to improve its strength and stability. The notion of mechanical stabilization was sparked by comparable situations found during the construction of the Maiduguri - Gambaru road, where the major superficial soil deposit was black cotton soil. Ibrahim, (1983) discovered that a good mix of 5 to 6% hydrated lime (by total weight of the clay-sand mix) and around 40% - 60% sand by dry weight of the clay gave the needed stability. To be mechanically stable, a soil mixture's particle size distribution must fit the Fuller and Thompson grading curve developed in 1907 grading curve, as described by Singh, (1991), with the following empirical relationship:

$$\%\text{ Passing any sieve} = 100\sqrt{\overline{\frac{h\quad h}{_{h\ h}}}} \tag{2.1}$$

If this empirical relationship is satisfied, particle sizes will entirely fill the spaces of larger sizes. Singh, (1991) identified certain parameters that influence the mechanically stabilized layer's stability. These are some of them:

a. Proper proportions of mix i.e. gradation: The amount of fines present in the mixture should be sufficient to fill up the spaces in the aggregates to get maximal strength and stability from mechanically stabilized soils.

b. Strength of the aggregates itself: The crushing strength of the aggregates will regulate the strength of the mix if the right percentage is reached. The stability of the mixtures will be harmed by aggregates with low crushing strength.

c. Properties of the soil to be mixed: The strength of a mechanically stabilized soil mixture may be affected by the soil properties. If, for example, a soil with a high plasticity index is added to the mix, the compacted mix's stability will deteriorate under soaking conditions. As a result, checking the qualities of the soil to be utilized in a mechanically stabilized mixture is critical.

d. Presence of harmful ingredients: Harmful elements such as sulphates and micas might have a bad impact on the compacted mix. Calcium chloride, for example, could be useful (Singh, 1991).

e. Amount of compaction: Compaction is the most significant component in all soil stabilizing techniques. Researchers such as (Gidigasu 1976; Ola 1983; and Osinubi 1998) have found that increasing compaction energy to the West African Standard (WAS) compaction energy increases the strength of stabilized soils.

2.3.2 Bituminous stabilization

Yoder and Witczal, (1975) classified bitumen as a water proofing agent, and one of the key features it imparts on soil when used for stability is water proofing. Bitumen stabilizes soils through two ways in general (AFMAN 1994; Ola, 1975; Gidigasu 1976). The first is the cementation or adhesive process, which strengthens coarse-grained soils but may decrease the strength of fine grained soils. A water proofing phonemenon is one of the basic mechanisms in asphalt stabilization of fine-grained soils. The bitumen coating on soil particles or agglomerates limits or slows water penetration, possibly resulting in a reduction in soil strength. Both the water proofing and adhesion mechanisms are present in non-cohesive materials such as sands and gravels. The increase in strength due to adhesion will last until the optimum bitumen concentration is reached, at which point the strength will decrease due to a reduction in grain to grain contact and hence lower strength mobilization. Laboratory tests on the potentials of various types of bitumen in clay soil stabilization have been undertaken by (Ola 1975; Osinubi and Bajeh 1994; Osinubi 2001a). They all documented an increase in strength of A-7-6 clay soils to an optimum bitumen level of 4 percent Ola, (1975), 5 percent Osinubi and Bajeh, (1994), and 6 percent Osinubi, (2001b), beyond which the strength declined in all cases. The experiments also revealed that as bitumen percentage increases, maximum dry densities and optimal moisture content drop, but do not grow at increasing bitumen content.

2.3.3 Lime stabilization

Lime stabilization is largely limited to mild to moderate locations since lime stabilized soils are susceptible to cracking during freezing and thawing (Gidigasu, 1976). The impact of lime on soil can be divided into three main reactions:

a. Changes in the water film surrounding clay minerals;
b. Soil particle coagulation or flocculation; and
c. Lime reaction with soil components to produce new chemicals.

According to Ola, (1975), there is no direct hydration to form cementous compounds in lime stabilization; rather, the lime-clay reaction has physical and chemical components. In the physical reaction of cation absorption, calcium replaces any other ion present as a base-exchange ion. Following then, flocculation into coarse particles occurs, resulting in an increase in strength almost immediately. This is consistent with Osinubi's (1999) description of the hydration of soil cement.

When lime is introduced to the soil, the partial dissociation of calcium hydroxide causes an instantaneous increase in the pH of the molding water. At soil surfaces, the calcium ions react with reactive silicia or alumina (or both) to form insoluble calcium silicates or aluminates (or both), which solidify on cure to stabilize the soil. Ola, (1975) reported a 70 percent to 300 percent increase in strength. The stabilized soil, however, was only acceptable as a sub basis. (Singh, 1991). They also noticed a drop in maximum dry density for soil-lime mixes as the lime component increased up to higher percentages, when it remained constant. However, when the lime level rises, the ideal moisture content rises as well, eventually becoming constant at higher lime contents.

2.3.4 Cement stabilization

For the widest range of clay soils, this has been found to be the most effective of all the stabilizers. The process of cement stability, according to (Ola, 1975; Gidigasu, 1976; Singh, 1991; AFMAN, 1994); focuses around the hydration products of cement, which are independent of the soil. Calcium silicate hydrates, calcium aluminate hydrates, and hydrated calcium silicate. The first two products constitute the major cementitious components, whereas the lime is the deposited as a separate crystalline solid phase. These increase in strength is due to the development of cementation linkages between these hydration products and soil particles. The released lime could react with any pozzolanic elements in the soil, such as clay, to generate a secondary cementitious substance that helps with inter-particle bonding. Stabilization with cement occurs in cohesive soils due to a decrease in plasticity and the creation of a matrix enclosing clay in cement (Singh, 1991)

Substantial increases in strength have been recorded for cement stabilized clay soils by (Ola, 1975), Adeyemi and Abolurin, (1994), Osinubi, (2000) Osinubi, (2001a), when unconfined compressive strength of the stabilized clays. Variation of maximum dry densities with increase in cement content depends on whether the soil is fine grained lateritic soils cause agglomeration and flocculation of the particles into coarser particles and hence reduce the maximum dry densities at small cement content from small percentage of cement to optimum cement content where it will begin to increase again. Cement, on the other hand, fills the voids between the soil particles in coarse-grained soils, increasing the maximum dry density from the start, even if the cement content is low. Maximum dry densities rise with cement content until it reaches optimum levels, at which point it may fall or remain constant. The optimum moisture increase slightly with increase cement content and begins to drop again at optimum content.

2.3.5 Pozzolana

Pozzolana is a siliceous or amorphous siliceous and aluminous material that, in finely divided form and in the presence of moisture, chemically reacts with lime (liberated by hydrating Portland cement) at ordinary temperatures to form compounds with cementation properties, according to the American Society of Testing and Materials (ASTM C618-93), (1978). According to Neville, (2000), the original reason for using these materials are usually economics; they existed as natural deposits requiring non or little processing, and some time because they are by products or wastes from industrial processes. However, finely divided pozzolanas are necessary because silicia can only mix with calcium hydroxide $Ca(OH)_2$ in the presence of water to generate stable calcium silicate with cementitious characteristics.

Egyptians, Romans, and Greeks built using only gypsum and time over 2000 years ago, but Kuwaitis and Indians built with plain earth, mud bricks, burnt clay, and lime. However, with the introduction of ordinary Portland cement (OPC) in 1824, the focus moved to using OPC as an effective cementing ingredient for construction. However, these older pozzolanas-volcanic ash, turfs, pumicides, opaline shales and chert, fly ash, basalt stone are still in use in Kenya, India, and Guyana (Smith, 1992)

Recent research has also shown that using agricultural waste in conjunction with lime to make cementitious compounds is feasible. Many plants absorb silicia from the ground into the structure of their leaves, stalks, and other parts as they grow. When residue of these plants are burnt, organic material, which is the largest proportion, is broken down and disappears as carbon dioxide and water vapors etc. The ash remaining contains mostly in organic residue notably silicia (Smith, 1992). Pozzolana can be divided into two groups; natural and artificial pozzolanas.

2.3.5.1 Natural pozzolanas

These are mostly materials of volcanic origin but could include certain diatomaceous earth. These include (Ogbonyomi, 1998)

a. Volcanic tuff: These materials originate from the deposition of volcanic dust and ash. These materials have been thrown out by volcanoes in recent times or long ago which contain finely divided materials, high in siliceous or aluminous content.
b. Diatomaceous earth: These are composed of the siliceous skeletons of diatoms deposited from either fresh or seawater. In some cases, the deposits are mixed with sand or clay.

2.3.5.2 Artificial pozzolanas

The main artificial pozzolanas are burnt slays and shales, pulverized fuel ash Neville, (2000) some other recent discoveries are calcined basalt stone, Calcium Carbide Residue (CCR) and bagasse ash. The last two obtained from industrial wastes.

a. Burnt clay and shales: These types of pozzolanas are produced by burning suitable clays or shales at a temperature of between 600 to 900°C depending upon the nature of the clay and the condition of burning. These materials turn pozzolanic because of the constituents of the clay, which are mainly hydrated alumino silicates with three broad groups of kaolinites, illites and montmorillonites.
b. Pulverized fuel ash: This is a by product of burning of coal in boilers and approximates to burnt clay, which is high in alumina and iron oxide in composition. The particles are roughly spherical in shape and in finely divided form has specific surface of 2000 to 5000cm^2/g (Neville, 2000)
c. Calcined basalt stone: It has been reported by Smith, (1992) that basalt treated to a temperature range of 500 to 800°C exhibits pozzolanic activity. The optimum temperature will differ from one stone source to another.
d. Zeolite: Zeolite is a common mineral formed by the weathering and modification of various rocks. This substance is safe for the environment and has a high contaminant absorption rate. Furthermore, because of its ability to regulate pH, it rarely creates acidic or alkaline conditions. As a result, zeolite addition can improve soil strength while also being environmentally friendly. Misari *et al.* (1998), is suggested for both surface and deep soil improvement. Because soaking has such a large impact on metrics, it is recommended that future studies look at the impact of soaking on UCS and soil strength factors.

2.4 Admixture Stabilization

There are times when a single chemical additive, such as cement, lime, or bitumen, cannot restore a soil's lacking qualities, or when a desired strength cannot be reached with a little amount of a single additive. In certain cases, two or more chemical additives may be necessary to restore the missing qualities or required strength while using a reasonable amount of an additive. Cement, for example, is the cheapest and most widely accessible soil stabilization additive in Nigeria, however it cannot be used economically on particularly soft clays such as black cotton clays. However, adding lime to this sort of clay can aid to increase its workability, resulting in a significant reduction in the amount of cement required for effective stability of that clay.

With the addition of any pozzolanic ingredient to the cement, a long rise in strength can be accomplished with a small amount of cement. This is caused by an interaction between the pozzolanas and the lime released during the cement hydration reaction. Because the only criteria available have been cement and lime, some researchers have decided to use a new evaluation criterion for soil stabilization utilizing a combination of an additive and a pozzolana. (Osinubi, 1999) developed an evaluation criterion for cement stabilized residual black cotton soil when time is used as an attendant high durability due to enhanced pozzolanic reaction of the soil-lime-cement bearing ration of 55 percent are recommended as evaluation criteria in comparison to the UCS of 1720kN/m^2 and (CBR) if 80% recommended by BS 1924 (1990) and Nigerian General Specification for Road and Bridge Works (1997).

Toro, (1997) conducted laboratory tests on the stability of various chemical additives in admixtures and found a significant rise in strength (CBR) and (UCS) after a fair number of days of curing, usually 7, 14, and 28 days. However, as the maximum dry density and optimum moisture content of these mixes increase, the maximum dry density and optimum moisture content of these mixtures increase in admixture content is not consistent like those of single additives. Rather it could increase and decrease at no constant additive contents due to complex reactions between the additives and the soil to be stabilized.

Using natural or manufactured zeolites as pozzolanic materials to produce cement mixes has been the subject of much investigation in recent years. Liguori *et al.* (2013) due to their large specific surface area and porosity in their structure, zeolites have been found to have pozzolanic activity when coupled with various geological materials. When calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH) are mixed with minerals like lime and calcium carbide, they form calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH), which harden (Colella, 1999). Zeolites are crystalline aluminosilicates made up of AlSi (O_4) tetrahedrons with open three-dimensional structures. They are categorized based on the amount of SiO_2 and Al_2O_3 in their composition. Nagrockiene and Girskas, (2016) point out that this sort of addition has a regular structure, consistent pores, and thermal stability when compared to other additives. Due to its pores and channels, it is also a good water absorbent (Vejmelková *et al.* 2015). The ion-exchange property of zeolite, where hollow structural gaps create an environment for accepting Na+, K+, Mg2+, Ba2+, and Ca2+ catalysts (Terzic *et, al.* 2018), is another distinguishing feature.

Investigating the pozzolanic activity of zeolites as one of the fundamental conditions to be employed as a stabilizer is one of the concerns that has come up frequently in past studies. Perraki *et al.* (2003), Poon *et al.* (1999), and Samimi *et al.* (2017) are few of the works that can be highlighted in this regard. According to studies, zeolites have a higher adsorption capacity than clays. Perraki *et al.* (2003) found that mixing zeolite with cement leads in the consumption of calcium hydroxide and the creation of hydration products (such as cement). Natural zeolites are rich in reactive SiO_2 and Al_2O_3, which are frequently employed in the cement industry, according to Poon *et al*. (1999). Zeolite, like other pozzolanic minerals, plays a vital role in concrete strength (Samimi *et al*. 2017). Perraki *et al*. (2003) looked at the effects of zeolite and pumice pozzolana on concrete compressive strength, exchange characteristics, and chlorine penetration resistance. In concrete, they used pumice (10%) and zeolite (15%) as alternatives for portland cement. When compared to its pumice counterpart Samimi *et al*. (2017) the results showed that the investigated zeolite had a stronger pozzolanic reaction. Guidobaldi *et al*. (2017) studied the reactivity of pyroclastic soils rich in zeolite in response to lime addition. They discovered that adding lime to soil can improve its strength and flexibility, which is a result of soil fragility. Improved pyroclastic soils after a short period of lime application demonstrate strong reactivity across the microstructure as a result of pozzolanic reactions. Guidobaldi *et al*. (2017). The mechanical characteristics of natural zeolite-containing compounds have remained largely unexplored. The mechanical characteristics and durability of zeolite-containing concretes were investigated by Markiv *et al*. (2016). They discovered that zeolite-containing concretes had lower 90-day compressive strength than zeolite-free concretes. The findings also show that utilizing a lubricator and sparger in zeolite can be very effective in inducing concrete to develop resistance to water penetration, creep, and freezing Markiv *et al.* (2016). The impact of chemical properties of natural zeolites and zeolite calcination on the effectiveness of cement mixtures were explored by Seraj *et al*. (2016). They did this by measuring the specific surface area of natural and calcinated zeolite samples. They next investigated the cement including natural and calcitic materials using the rheological slurry test, viscosity, and stress yield of slurry. The findings showed that calcination alters the structure of natural zeolites and reduces their specific surface area, but that the mixture's viscosity and yield stress improve over time. Seraj *et al*. (2016). Mola-Abasi and Shooshpasha, (2016) looked into the effects of zeolite and cement on the mechanical behaviour of wind-blown sands and found a correlation between the cement-zeolite ratio and mixture porosity and uniaxial compressive strength. In wind-blown sand soil Mola-Abasi and Shooshpasha, (2016) replacing zeolite for cement resulted in improved uniaxial compressive strength. Pozzolana in concrete is thought to cause decreased strength during shorter curing durations and increased strength over longer periods of time, as well as optimal substitution ratios, according to studies with significant pozzolana substitution in concrete, desirable ratios are always achieved (Vagelis, 2000, Targana *et al.* 2004). This characteristic is also seen in most zeolites as a pozzolana (Poon *et al*. 1999). According to studies, using natural zeolites in concrete improves the durability and performance and mechanical properties (Mohseni *et al.* 2017). Because of their high absorption capacity, zeolites improve the durability of concrete (Tran *et al*. 2019). Natural zeolite in concrete not only has no detrimental effects, according to Ramezanianpour *et al.* (2015) and Vejmelková *et al.* (2015), but it also greatly improves the compressive strength of concrete. Ding *et al*. (1999); Sabet *et al*. (2013); Mohseni *et al*. (2017); Chan and Ji, (1999).

According to research, zeolites can impact soil hydraulic conductivity characteristics, with coarse-grained zeolites causing increased hydraulic conductivity (Hong *et al*. 2011; Du *et al*. 2015). Zeolites are generally considered to be one of the most environmentally friendly additives. According to Kaya and Durukan, (2004) and Jin *et al*. (2010) zeolite additions boost chemical compatibility in calcium bentonite soils. Zeolites have a high metal absorption capacity and are commonly employed as an absorbent in gravel and bentonite soils to remove ammonium and heavy metals. As a result, they are extremely useful for increasing the blockage of pollutants through absorption. Mola-Abasi *et al.* (2016) tested cement and zeolitic cement samples for environmental and microstructural factors. In comparison to cement compounds, they found that zeolite-containing compounds have a higher contamination absorptivity. Adding zeolite to the sand-cement mixture also produces structural changes, as the zeolite fills the volume of hollow pores, causing the pozzolanic reaction to occur. The result, according to microstructural studies, is increased resistance during the 28-day curing period (Mola-Abasi *et al*. 2016). Cheshomi *et al.* (2017) investigated the effect of lime and cement on the swelling characteristics and Atterberg limits of sulphate clay soil in a series of tests. They discovered that the swelling pressure, liquid limit, and plasticity limit of soil increase as a result of the development of ettringite in response to the addition of lime. The addition of portland cement to sulfate clay soil, on the other hand, resulted in a decrease in all of the above metrics due to the production of cement gels. Cheshomi *et al*. (2017) found that adding varying percentages of lime and cement to the stated soil reduced the soil's plasticity limit but had a modest influence on swelling pressure. Mahedi *et al*. (2018) also investigated the efficacy of two types of cement (Type I/II and V) and steel slag in stabilizing expansive clay. The findings revealed that soil Atterberg limits containing liquid limit (LL) and plastic limit (PL) declined initially up to a particular level of cement concentration (8–10%), then began to climb somewhat as the cement content increased. Increases in stabilizer concentration and curing time increased the uniaxial compression strength of soils. Finally, slag increased the performance of cement and proved to be a better option for treating expansive soils with high sulfate levels.

2. 5 Inferential Statistics

2.5.1 Random variables

An event that is neither certain nor possible is referred to as random, signifying that it may or may not occur under essentially identical conditions. That means the outcome of the experiment is not known in advance before it has taken place. A random variable or a variate is a variable whose value is uncertain, unpredictable, or nondeterministic, such as the strength of a soil or any other material strength, and whose distribution is known. A random variable can take on a value whose size is determined by a specific occurrence or outcome. This outcome is noted by an observation or measurement of an experiment in which test are made and records maintained (Kottegoda and Rosso, 1997). Each outcome of a random variable defined with respect to sample space, corresponds to a numerical value of the random variable. A random variable can be formally viewed as a function defined on the sample space of an experiment such that there is a numerical value of the random variable corresponding to each possible outcome; that is, there is a probability associated with each occurrence in sample space. An uppercase letter denotes a random variable and the corresponding lowercase letters represents the value that it assumes. For example, one may refer to strength determined in an experiment as X; then $x = 729\text{N/mm}^2$ say is a particular value that the random variable X may take.

2.5.2 Discrete random variables

A random variable x is called discrete if the range of x is countable, that is if there exist a finite or denumerable set of real numbers x_1, x_2 …….such that x takes on values only within that set. The random variable can assume only a specific discrete integer values. These values x_1, x_2...are called possible values of the discrete random variable x. In order to characterize the discrete random variable completely, it suffices to know the probability, $P = (X = x_i)$

2.5.3 Continuous random variables

A random variable is continuous if the variable could assume any value on the positive real axis. A continuous variable can take any value within two limits, determined by physical or theoretical means. Such a value can in theory be specified using an unlimited number of decimal places but is limited in practice by the accuracy of the measuring device used, such as a flow gage or weighing device (Kottegoda and Rosso, 1997)

2.5.4 Mean or expected value of random variables

The sample arithmetic mean of a set of data is simply the average of the observed data. By analogy, the centroid is the center mass of a solid body. Corresponding to this geometrical definition is the statistical expected value E[X] of a random variable X. It is often called the population mean and is denoted by μx. The expected value gives a broad view of the random variable's behaviour without revealing all of the intricacies of its probability mass function, pmf (if discrete), or probability density function, pdf (if it is continuous). The distributions of two random variables with the same expected value might be significantly different. The pmf gives the weighted average of a random variable X as

$$\mu x = E[X] = \Sigma_{all} X_i PX(X_i) \text{ For discrete random variables} \tag{2.2}$$

If X is a continuous random variable its pdf gives the weighted average of a random variable X as

$$\mu x = E[X] = \int_{-\infty}^{+\infty} \quad (\,) \quad \text{for continuous random variables} \tag{2.3}$$

The mean is the most useful single number in engineering application for theoretical, comparative and other purposes. It is an important measure of the central tendency of the random variable and is well-suited to represent the phenomenon studied (Kottegoda and Rosso, 1997)

2.6 Confidence Limits for the Mean

Using the table of the distribution, we can define 95 percent, 99 percent, and other confidence intervals. In this way, we can estimate the population mean within defined confidence bounds

(Kottegoda and Rosso, 1977). This is to ensure that the mean of the strengths determined through the experiments is accurate. The 95 percent and 99 percent confidence limits are the most regularly utilized.

For small sample theory, N < 30, the confidence limits are calculated from the expression below;

$$95\% \text{ confidence limits are: } \mu \pm 0.975 \frac{\quad}{\sqrt{\quad -1}} \tag{2.4}$$

$$99\% \text{ confidence limits are: } \mu \pm 0.995 \frac{\quad}{\sqrt{\quad -1}} \tag{2.5}$$

0.975 and 0.995 are percentage values for t distribution with v degrees of freedom.

For large sample theory, N > 30, the confidence limits are calculated from the expression below;

$$95\% \text{ confidence limits are } \mu \pm 1.96 \frac{\quad}{\sqrt{\quad}} \tag{2.6}$$

$$99\% \text{ confidence limits are } \mu \pm 2.58 \frac{\quad}{\sqrt{\quad}} \tag{2.7}$$

Where μ is the mean, S is the standard deviation, v is the degree of freedom and N is the number of samples tested. The determined mean is the true one if the value lies between the lower and upper values of confidence limits calculated.

2.6.1 Variance of random variables

The variance of a random variable is a non-negative quantity that indicates how far the random variable's values are expected to be scattered. The bigger the variance, the more the random variable's individual values deviate from the mean. The smaller the variance, the closer the random variable's individual values are to the mean. The variance gives a picture of how tightly the anticipated value of the distribution is concentrated. It is a measure of a distribution's "spread" around its mean. V[X] represents variation. (Kottegoda and Rosso, 1997)

Var [X or σ^2 and can be written as

$$\text{Var}[X] = \sigma^2 = E[(X - E[X])^2] = E[X^2 - 2XE[X] + (E[X])^2] \tag{2.8}$$

$$= E[X^2] - 2E[X]E[X] + (E[X])^2 - (E[X])^2$$

This result shows that the variance is equal to the difference between the mean of the squares and the square of the mean.

2.6.2 Standard deviation of the random variables

The standard deviation σx is defined as the positive square root of the variance. It is measured in the same units as the variable and is therefore more practically meaningful as a measure of dispersion than the variance, because it can be compared directly with the mean. It can be written as $= \sqrt{\ }^{2}$ (Kottegoda and Rosso, 1997)

2.6.3 Confidence limits for standard deviation

For the x2 distribution, we can define 95 percent, 99 percent, or other confidence limits and intervals. In this way, we may estimate the population standard deviation in terms of a sample standard deviation S within specified confidence bounds. This is to ensure that the standard deviation of the strength calculated by the experiments is accurate. The 95 percent and 99 percent confidence limits are regularly utilized.

For small sample theory, N < 30, the confidence limits are calculated from the expressions below;

95% confidence limits are given by $\frac{\sqrt{\ }}{\ _{0.975}}$ and $\frac{\sqrt{\ }}{\ _{0.025}}$

The values 0.975 and 0.025 represent respectively the 2.5 and 97.5 percentile values.

For degree of freedom, V = N – k (2.9)

99% confidence limits are given by

$$\frac{\overline{\;}\,\sqrt{}}{}_{0.995} \text{ and } \frac{\overline{\;}\,\sqrt{}}{}_{0.005} \qquad (2.10)$$

Where v is the degree of freedom, N is the number of samples and k is the number of parameters determined in the test and used in the calculation. Here only the mean and the standard deviation are the parameters calculated.

For large values of v (*v ≥ 30)* we can use the fact that ($\sqrt{2}\;^{2}$ - $\sqrt{2\overline{\;\;} - 1^{2}}$) is $\overline{\text{very norma}}$lly distributed with mean zero and standard deviation one, so that normal distribution tables can be used if *v ≥ 30* (Kottegoda and Rosso, 1997). Then if Z_p is the z score percentile of the standardized normal distribution, we can write, to a high degree of approximation,

$$\sqrt{\overline{\;\;\;}}\,_{2^{2}} - \overline{\;\;\;\;\;}_{-1}\,\sqrt{2} = Z_p \text{ or } \sqrt{\overline{\;\;\;}}\,_{2^{2}} = Z_p + \sqrt{2}\,\overline{\;\;\;\;\;}_{-1}$$

$$^{2} = \frac{1}{2}\,\Big(\frac{\;}{+\sqrt{2}\,-1}\Big)^{2} \qquad (2.11)$$

95% confidence limits are given by $\frac{\overline{\;}\,\sqrt{}}{}_{0.975}$ and $\frac{\overline{\;}\,\sqrt{}}{}_{0.025}$

99% confidence limits are given by $\frac{\overline{\;}\,\sqrt{}}{}_{0.995}$ and $\frac{\overline{\;}\,\sqrt{}}{}_{0.005}$

The determined standard deviation is the true one if the values lies between the lower and upper values of the confidence limits calculated.

2.6.4 Coefficient of variation of random variables

The coefficient of variation of random variables, Vx provides a relative measure of dispersion that is dimensionless. It is usually given as a percentage. It can be written as

$$Vx = \text{—} \qquad (2.12)$$

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Soil and Location of Study Area

The clay soil sample was taken from Niger State Polytechnic, Zungeru, in Niger State, Nigeria. The clay was collected using the method of disturbed sampling. The soil was collected at a depth of 2.5m to 3.0m. Visual inspection revealed the layer of pisolithic gravels at a depth of between 0.8m to about 1.5m below which the clay is fine in texture. It can be inferred that the clay used for this study is a fine lateritic soil.

3.2 Chemical Additives

3.2.1 Calcium carbide

The calcium carbide residue used in this study was obtained from a local panel beater workshop along kpakungu axis as a byproduct of acetylene gas. The process involves the reaction of calcium carbide with water which produces a reaction of calcium carbide (CaC_2) with water which produces a slurry that is basically hydrated lime (Ca (OH) ^{2}). A sample was collected and dried for about 12-24 hours then grinded into a powder form and sieved through sieve size 200µmm B.S

sieve. Nima *et. al.* (2018) In a work investigated the use of CCR to improve the engineering properties of green bentonite (with predominantly montmorillonite) unconfined compressive strength (UCS). The CCR doses were varied with curing times and the highest strength improvement was recorded at 9 – 12% of CCR which was at 28 days curing. Also after a micro level assessment was performed using field emission scanning electron microscopy (FESEM), X-ray fluorescence (XRF), particle size analysis test and Nitrogen-based Brunauer-Emmett-teller (N_2-BET) surface area analysis test.

3.2.2 Zeolite

This was gotten from a commercial market in Zaria. Zeolites can be described as materials made up of micro-porous aluminosilicate crystals which are used as ion exchangers in the detergent industry, in radioactive waste storage, in the treatment of liquid waste, as separators in purification, drying, and environmental treatment, in the catalytic cracking of petroleum and in refining petrochemical, coal and fine chemical industries. Breck (1974); Chiang and Chao (2001); Xu *et.al.* (2007) The properties which enables them to perform all these functions in several industrial applications are their uniform pore size and shape, the mobility of their cations to act as catalysts and their hydrophilic and/or hydrophobic nature to some solutes4 . Primarily, zeolites are built from [SiO $[AlO^4]^{5-}$ tetrahedral which are infinitely extended in a three dimensional network that is linked together by a shared oxygen atom. This $[SiO_4]^{4-}$ and $[AlO_4]^{5-}$ tetrahedral can be represented by TO_4 which is often referred to as the primary building block (PBU), where **T** represents either the silicon (Si Aluminium (Al^{3+}) ions while the **O_4** represents each of the oxygen atom that is shared by the two **T** atoms. Ivan, (2012)

3.3 Methods

Index test and strength test were carried out on natural clay soil to ascertain the suitability or otherwise of the natural soil for use in pavement structure according to BS 1377 (1990). The second category of test were carried out on the soil admixed with calcium carbide residue and zeolite at a mix ratio established by otherwise deterministic means and from literature found to give a consistent percentage hence 10% was selected as CCR mix ratios according to BS 1924 (1990) to evaluate the suitability of these admixture as clay soil stabilizer. The percentage of calcium carbide used was 10% which was admixed with 2% (also from deterministic and literature suggestions) zeolite by weight of the dry soil. The mixtures were compacted to predetermined maximum dry densities and optimum moisture contents for unconfined compressive strength test.

3.4 Soil Classification Test

3.4.1 Determination of natural moisture content

The procedure was in accordance with BS 1377 (1990). Some quantity of soil was kept in an air tight polythene leather as soon as the soil was collected from the borrow pit and was brought to the laboratory. Two containers were cleaned, and weighed. About 30g of the soil was crumbled and placed in each container. The container and content were weighed to the nearest 0.01g and placed in an oven at a temperature of 105°C to 110°C for a period of 24 hours. After drying, the container was removed from the oven and allowed to cool. The container and the content were again weighed. The natural moisture content is given by;

$$W = \frac{-}{-} \times 100\% \tag{3.1}$$

Where

Wc = weight of container (g)

Ww = Weight of container + wet soil (g)

Wd = Weight of container + dry soil (g)

W = Moisture content (%)

3.4.2 Sieve analysis

The test was carried out using the wash sieving method in line with BS 1377 (1990). Weighing 200g of moist soil and placing it in a clean, dried container with enough water to cover it. The mixture was allowed to sit overnight to allow water to percolate through the fine soil clods. The soil was then washed through two sieves (2mm aperture size up and 75µmm BS aperture size down), with the water flowing through the 75m BS sieve collected in a container and utilized for hydrometer testing. Both the soil retained on the 2mm BS sieve size and the soil retained on the 75µmm BS sieve size were dried in an oven and dry sieved.

3.4.2.1 Dry sieve analysis

From 2mm on top to 1.18mm, 0.6mm, 0.425mm, 0.30mm, 0.212mm, 0.075mm at the bottom, sieves were positioned according to their aperture sizes. Both the soil held on 2mm sieve size and the soil retained on 75mm sieve size were weighed and poured into the top sieve after drying and sieved for five to ten minutes through the arrangement of the sieves. In the meantime, the sieves were weighed empty before being used. . After sieving, each sieve was weighed again, and the difference in masses was recorded as mass retained on each sieve. The cumulative mass retained, percentage mass retained, and percentage mass passing were all calculated from the mass retained.

3.4.2.2 Hydrometer analysis

All of the soil that had been washed through a 75µmm BS sieve was collected and poured into a 100-millilitre graduated cylinder, which was then filled to the 1000-millilitre mark. The cylinder's cork was inserted, and the solution was thoroughly agitated for about 5 minutes before being placed on top of a flat surface. A hydrometer was immediately inserted, and readings were taken after 15 seconds, 30 seconds, 1 minute, and 2 minutes, 4 minutes, 10minutes, 20minutes, 30minutes, 1 hr, 2hr, 4hr, 6hr, 12hr and 24hr using a thermometer, the temperature of the mixture was continuously measured during the process. The particle size distribution of silt and clay size particles was determined in this manner.

3.4.3 Atterberg limits

The Atterberg Limit Test were conducted in accordance with BS 1377(1990)

3.4.3.1 Liquid limit

To make a homogenous paste, 200g of a part of the soil sieved through a BS sieve with a 425µmm aperture was thoroughly combined with distilled water. In the Casagrande equipment cup, a bit of the paste was inserted. Using a grooving tool, the paste in the cup was separated along the cup diameter through the center. The crank was rotated at around two revolutions per second, and the number of blows required to seal the groove in the sample was counted. For moisture content determination, around 10g of soil was extracted from the closed groove. A small amount of the soil mixture was added to the cup and mixed in with the soil in the cup, and the process was repeated until two consecutive runs resulted in the same number of blows for closure. The moisture content was plotted against the number of blows. Through the points, a straight line was drawn. As the liquid limit, the moisture content in percentage corresponding to the 25blows ordinate was recorded.

3.4.3.2 Plastic limit

The sample contained 20g of soil that passed through a 425µmm BS sieve. On a glass plate, it was thoroughly mixed with enough distilled water to make it pliable enough to be molded into a little ball. After that, the ball was rolled between the hand and the glass plate to make a 3mm thread. The procedure was repeated until the thread began to fray. The moisture content of some of the collapsing components was determined. The operation was performed three times to produce three values, which were then averaged to determine the plastic limit.

3.4.3.3 Plasticity index

Plasticity index is the difference between the liquid limit and the plastic limit

$$PI = LL - PL \tag{3.2}$$

Where PI = Plasticity Index

AL = Liquid Limit

PL = Plastic Limit

3.5 Soil Strength

The mass of the natural wet soil required to fill both the UCS mould has been given by BS 1377 (1990) as a function of maximum dry density and optimum moisture content of the compacted natural soil.

3.5.1 Unconfined compressive strength

The mould used for this test was a cubic split mould 7cm ×7cm×7cm and volume of this mould was used to obtain the mass of the material (soil, cement, bagasse ash) from the predetermined maximum dry density that was used for the compacted mix. For each of the mix ratios thoroughly to uniform color. Predetermined amount of water (from optimum moisture content) was then added and mix thoroughly until the mixture formed a uniform paste. The mixture was compacted into the mould in layers (5 layers for BSH compaction energy) in the same manner as is done for the CBR test. In any case, the last layer was compacted until the soil levelled off with the mould. After compaction, the soil was removed from the mould and kept off with the mould. After compaction, the soil was removed from the mould and kept in an airtight polythene to avoid moisture escaping from the specimen. The samples were then allowed to cure for 28 days while the last batch was for 7 days and there dewaxed top and bottom and soaked in water for another 7 days.

After curing, the samples were crushed using universal testing machine at the rate of 1723 kN/m^2 per minute until the failure load for each of the specimen was recorded. The compressive stress for each sample was calculated as

Compressive stress = $\frac{\qquad}{\qquad}$ (kN/m^2) (3.3)

3.6 Confidence Limit Calculation for Mean and Standard Deviation

Confidence limit for mean of strength failure stress of 40 samples at 5% CCR and 2 % zeolite admixture combination with clay soil.

1. **Mean Confidence Limit**

Table 3.1: Confidence Limit Result Sheet for Soil Sample Strength

	95% CONFIDENCE LIMIT		**99% CONFIDENCE LIMIT**	
	5% CCR 0% ZEO	**5%CCR 2%ZEO**	**5%CCR 0%ZEO**	**5%CCR 2% ZEO**
Average	377.15	704.23	377.15	704.23
Standard Deviation	12.50	12.27	12.50	12.27
Sample Size	40	40	40	40
Confidence Coff	1.96	1.96	2.58	2.58
Margin of Error	3.87	3.80	5.10	5.00
Upper Bound	381.02	708.03	382.25	709.23
Lower Bound	373.28	700.42	372.05	699.22
Max	398	728	398	728
Min	354	684	354	684
Range	44	44	44	44
Confidence interval	4.00	3.92	4.00	3.92

2. **Standard Deviation Confidence Limit**

Confidence Limit for the standard deviation of strength failure of 40 samples at 5% CCR with clay soil

For 95% confidence level where clay soil is stabilized using only 5% CCR c = 0.95

Mean = 377.15

Standard Deviation = 12.5

Ჯdf ≈ $\frac{(\ -1)^{2}}{{}_{2}}$ where df = n-1 = 40 – 1 = 39

Tracing using 0.95 and df 039 we obtain the Ӽmin and Ӽmax values from the chart in the appendix

$$P(\ 24.43 \le \text{Ӽ}_{2}\text{df} \le 59.3) = 0.95 = 95\%$$

3.7

$$24.43 \le \frac{(\ -1)\ 2}{2} \le 59.3$$

$$24.43 \le \frac{39(12.5)2}{2} \le 59.3$$

$$\overline{\ 1\ } \ge \overline{39(12.5)^2} \ge \overline{\ 1\ }$$

$24.43^2 59.3$

$$\frac{39(12.5)^2}{24.43} \ge \sigma_2 \ge \frac{39(12.5)^2}{59.3}$$

$$\frac{39(12.5)^2}{59.3} \le \sigma^2 \le \frac{39(12.5)^2}{24.43}$$

$$\sqrt{\frac{39(12.5)^2}{59.3}} \le \sigma \le \sqrt{\frac{39(12.5)^2}{24.43}}$$

$$10.14 \le \sigma \le 15.84$$

For 99% confidence level where clay soil is stabilized using only 5% CCR c = 0.99

Mean = 377.15

Standard Deviation = 12.5

$\text{Ӽ}^2\text{df} \approx \frac{(\ -1)^{2}}{2}$ where df = n-1 = 40 – 1 = 39

Tracing using 0.95 and df 039 we obtain the Ҳmin and Ҳmax values from the chart in the appendix

$$P(22.16 \le Ҳ_{2}df \le 66.77) = 0.99 = 99\% \qquad (3.4)$$

$$22.16 \le \frac{(\ -1)\ 2}{2} \le 66.77$$

$$22.16 \le \frac{39(12.5)2}{2} \le 66.77$$

$$\frac{1}{22.16} \ge \frac{39(12.5)^2}{} \ge \frac{1}{66.77}$$

$$\frac{39(12.5)^2}{22.16} \ge \sigma 2 \ge \frac{39(12.5)^2}{66.77}$$

$$\frac{39(12.5)^2}{66.77} \le \sigma 2 \le \frac{39(12.5)^2}{22.16}$$

$$\sqrt{\frac{39(12.5)^2}{66.77}} \le \sigma \le \sqrt{\frac{39(12.5)^2}{22.16}}$$

$$9.55 \le \sigma \le 16.58$$

Confidence Limit for the standard deviation of strength failure of 40 samples at 5% CCR and 2% zeolite admixture combination with clay soil

For 95% confidence level where clay soil is stabilized using 5% CCR admixed with 2 % zeolite c = 0.95

Mean = 704.23

Standard Deviation = 12.27

$$\text{Ӽ}df \approx \frac{(\;-1)^{\;2}}{2} \quad \text{where } df = n-1 = 40 - 1 = 39$$

Tracing using 0.95 and df 039 we obtain the Ӽmin and Ӽmax values from the chart in the appendix

$$P(\,24.43 \le \text{Ӽ}\,2df \le 59.3) = 0.95 = 95\%$$

$$24.43 \le \frac{(\;-1)\;2}{2} \le 59.3$$

$$24.43 \le \frac{39(12.27)2}{2} \le 59.3$$

$$\frac{1}{24.43^{2}59.3} \ge 39(12.27)^{2} \ge 1$$

$$\frac{39(12.27)^{2}}{24.43} \ge \sigma 2 \ge \frac{39(12.27)^{2}}{59.3}$$

$$\frac{39(12.27)^{2}}{59.3} \le \sigma 2 \le \frac{39(12.27)^{2}}{24.43}$$

$$\sqrt{\frac{39(12.27)^{2}}{59.3}} \le \sigma \le \sqrt{\frac{39(12.27)^{2}}{24.43}}$$

$$9.95 \le \sigma \le 15.50$$

For 99% confidence level where clay soil is stabilized using 5% CCR admixed with 2% zeolite c = 0.99

Mean = 704.23

Standard Deviation = 12.27

$\chi^{2} df \approx \frac{(\ -1)\ ^{2}}{{}^{2}}$ where df = n-1 = 40 – 1 = 39

Tracing using 0.95 and df 039 we obtain the Ӽmin and Ӽmax values from the chart in the appendix

$P(\ 22.16 \le \chi_{2} df \le 66.77) = 0.99 = 99\%$

$$22.16 \le \frac{(\ -1)\ 2}{{}^{2}} \le 66.77$$

$$22.16 \le \frac{39(12.27)2}{{}^{2}} \le 66.77$$

$$\frac{}{1} \ge \frac{}{39(12.27)^{2}} \ge \frac{}{1}$$

22.16²66.77

$$\frac{39(12.27)^{2}}{22.16} \ge \sigma 2 \ge \frac{39(12.27)^{2}}{66.77}$$

$$\frac{39(12.27)^{2}}{66.77} \le \sigma 2 \le \frac{39(12.27)^{2}}{22.16}$$

$$\sqrt{\frac{39(12.27)^{2}}{66.77}} \le \sigma \le \sqrt{\frac{39(12.27)^{2}}{22.16}}$$

$9.38 \le \sigma \le 16.28$

CHAPTER FOUR

4.0 DISCUSSION OF RESULTS

4.1 Physical Properties Tests

The natural soil identification test results, as well as other attributes, are shown in a Table 4.1. The natural soil particle size distribution is also depicted in the illustration. The soil was classed as A-7-6 subgroup and group index (GI=12) by the American Association of State Highway and Transportation Officers (AASHTO) Soil classification system, as well as CL, clay of low plasticity according to unified soil classification system (USCS). According to the AASHTO soil classification system and the Nigerian General Specification for Road and Bridge Works (1997), this soil in its natural state is rated poor for sub base and base material in pavement structures and would require stabilization to improve its strength and durability.

Table 4.1: Specific Gravity Results

	SPECIFIC GRAVITY		
Bottle Number	Nat Soil	Nat Soil	Nat Soil
Mass of Bottle	69	69	69
Mass of bottle + Wet Soil	187.1	187.4	187.2
Mass of bottle + Dry Soil	99.4	99.8	99.6
Mass of bottle + water	168.4	168.4	168.4
Soecific Gravity, Gs	2.6	2.61	2.59
Average Specific Gravity		2.6	

The specific gravity test result of the soil which is 2.66 is shown in Table 4.1. The Atterberg Limit Test Result for the 87% passing soil from sieve 75µmm is shown in Table 4.2 which shows liquid limit to be 57.46% and plastic limit as 26.57%. Also Figure 4.1 is the graph of the liquid Limit and plastic limit curve; this shows the dispersive behaviour of the clay particle in the presence of water which is indicative of the swelling cause by absorption of moisture beyond 18% OMC of the natural soil and agrees with the report of (Nima *et al.* 2018) The plasticity Index is calculated to give 30.89%.

Table 4.2: Liquid Limit and Plastic Limit

Project: Tersoo Thesis							
Test Location:							
Sample NO: Clay soil	Initial Sample mass : 300g					Date: 29/03/2020	
Sample Description:							
		LIQUID LIMIT				PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	7.5	11.6	15.2	19.4	24.8		
Can Weight	28	25.4	23.4	22.5	26.1	20.4	22.6
Weight of can + wet soil	44.9	46.9	45.5	48.6	57.3	26.1	26.9
Weight of can + dry soil	40.4	40.3	38.1	39.1	45.1	24.9	26
Weight of Moisture	4.5	6.6	7.4	9.5	12.2	1.2	0.9
Weight of Dry Soil	12.4	14.9	14.7	16.6	19	4.5	3.4
Moisture Content	36.29	44.3	50.23	57.23	64.21	26.67	26.47
Liquid Limit	57.46%		Average Plastic Limit			26.57	

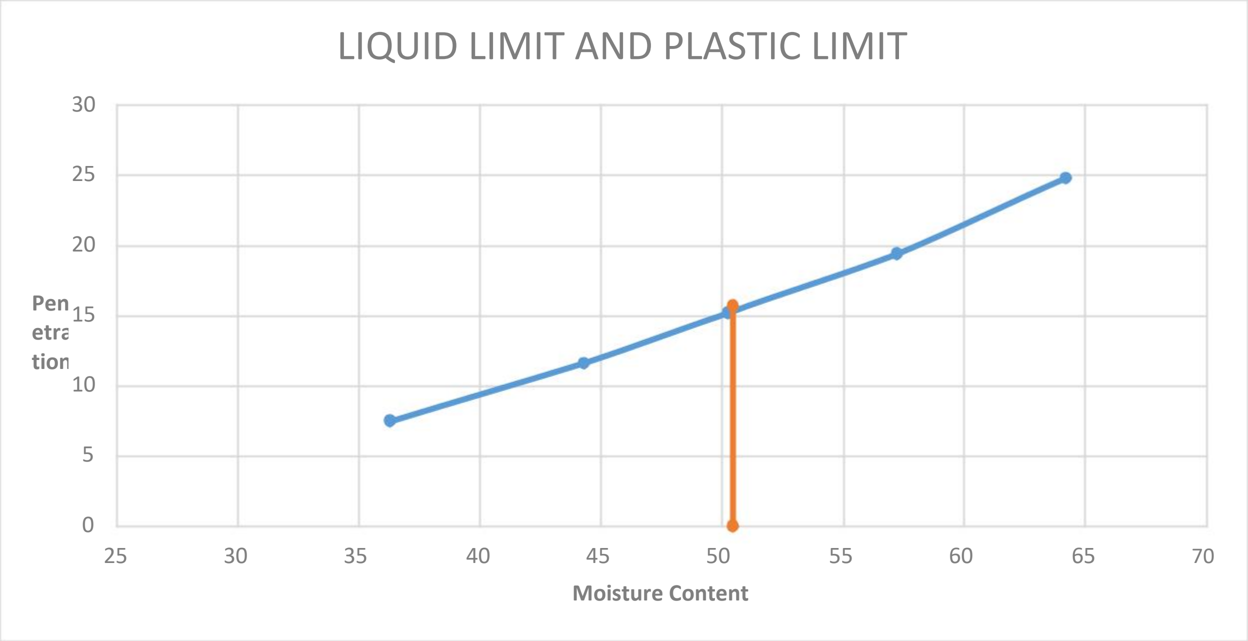


Figure 4.1: Liquid Limit and Plastic Limit

4.2 Oxide Composition of Calcium Carbide Residue and Zeolite

The oxide composition of calcium carbide residue and zeolite used in this study was determined using X-ray Diffraction method with the result displayed in Figure 4.2. The total percentage composition of iron oxide (Fe_2O_3 = 3.96%) Silicon oxide (SiO_2 = 57.95%) and Aluminium oxide (Al_2O_3 = 8.23%) was found to be 70.14%.

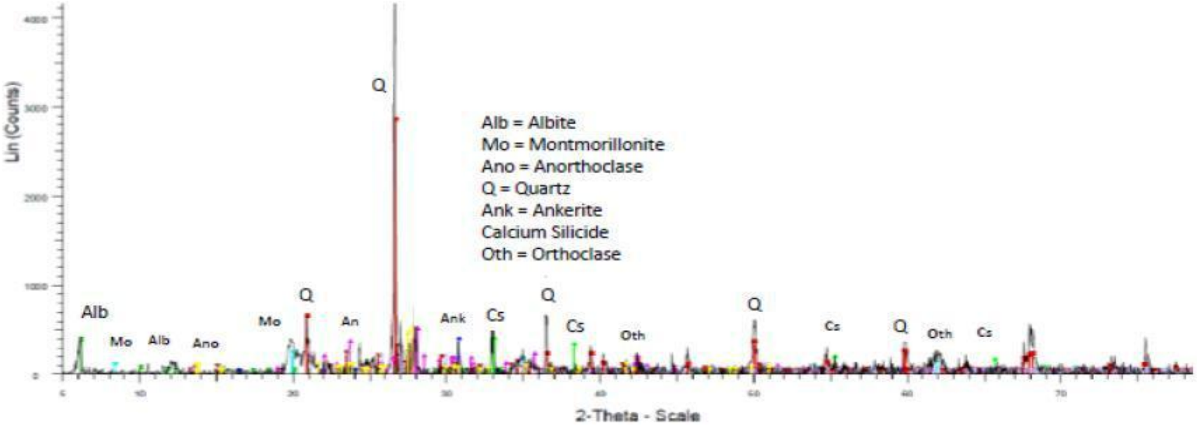


Figure 4.2: X-Ray Diffraction Test (XRD)

This value is within the required value of 70% specified for pozzolanas (ASTMC 618 -78, 1978). The value is lower than the value obtained by (Ogbonyomi, 1998). The difference might have resulted from the method of preparation of CCR and zeolite species and the age of the CCR with exposure to continuous reactions at exposure to water which hydrolyzes at each wetting into Calcium Hydroxide (CaOH) A poor mean strength will mean completely unreactive calcium carbide residue. (Osinubi, 2001a)

Table 4.3: Physiochemical Properties of Zeolite and Calcium Carbide Residue

Characteristics	Zeolite	CCR	Clay
SiO_2	61.5	6.49	68.9
Al_2O_3	8.00	2.55	15
TiO_2	0.12	3.25	0.18
Fe_2O_3	0.91	70.78	23.5
MgO	0.70	0.70	1.13
CaO	2.39	0.66	1.33
Na_2O	1.06	ND	ND
K_2O	1.14	7.94	3.6
pH_{pzc}	3.20	1.35	1.7
CEC (meq kg^{-1})	910.0	240.7	26.7
Surface Area ($m^2 g^{-1}$)	30	29	13

Table 4.3 shows a summary of the Chemical properties of the clay soil, zeolite and calcium carbide residue. The complimentary role of the constituent chemicals in the soil and additive shows how stabilization is achieved using these two materials.

Another relation is shown in Figure 4.3 and Figure 4.4 is the SEM of soil stabilized with 5% CCR and soil re-stabilized with 2% zeolite. The ettringe formation of the new flocs displays the improved cohesiveness between the soil particles. This lends credence to the particle structure of the clay as it hydrates with CCR to form bonded particles and the amorphous crystalline nature of the zeolite particle with further creates a hardening effect by its absorptive potential of the reacted soil and CCR particles, this agrees with (Mustapha, 2006)

4.3 Stabilization

4.3.1 Compaction characteristics

Many of the engineering properties of a soil are dependent on the moisture and dry density at which the soil is compacted (Toro, 1997). Therefore it is necessary to achieve the desired relative density of 95% or more on field relative to that obtained in the laboratory. From the result of the compaction test at 5% Calcium Carbide Residue (CCR) stabilized clay soil, the optimum moisture content (OMC) was found to be 18% and the maximum dry density (MDD) also found to be 1.684g/cm^3, this result is shown in Table 4.4 and graph displaying OMC and MDD Figure 4.3. This result was used to carry out the Unconfined Compressive Strength Test reported in Table 4.5.

Table 4.4: Average Moisture Content vs Dry Density Table

Project: Tersoo Thesis										
Test Location:										
Sample no:			Volume of mould: 944cm^3						Date: 29/03/2020	
Sample Description:										
Clay soil										
Weight of mold (g)	4480		4480		4480		4480		4480	
Weight of mold + wet soil (g)	6230		6360		6390		6410		6370	
Weight of wet soil (g)	1750		1880		1910		1930		1890	
Wet Density (g/cm^3)	1.85		1.99		2.02		2.04		2	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of can (g)	38.3	34.8	38.7	38.3	38.1	35.2	38.2	38.6	38.4	38.9
Weight of can + wet soil (g)	116.5	87.7	97.7	87.6	111.5	97.6	80.7	102.2	90.9	101.9
Weight of can + dry soil (g)	109	82.2	88.7	79.8	97.8	86.2	72	88.9	79	87.8
Weight of water (g)	7.5	5.5	9	7.8	13.7	11.4	8.7	13.3	11.9	14.1
Weight of dry soil (g)	70.7	47.4	50	41.5	59.7	51	33.8	50.3	40.6	48.9
Moisture content (g)	10.61	11.6	18	18.8	22.95	22.35	25.74	26.44	29.31	28.83
Ave. Moisture content (g)	11.11		18.4		22.65		26.09		29.07	
Dry Density (g/cm^3)	1.6685		1.6821		1.6497		1.6214		1.5512	

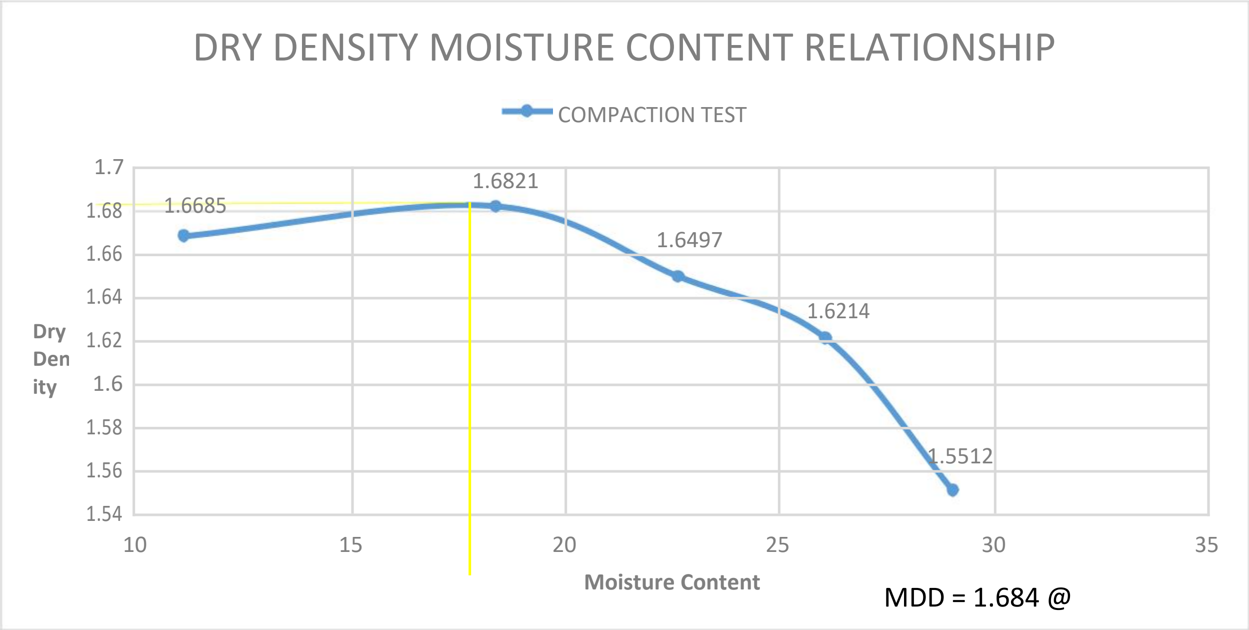


Figure 4.4: Average Moisture Content vs Dry Density Graph for Clay Soil and 5% CCR

4.3.2 Strength characteristics

4.3.2.1 Unconfined compressive strength

Unconfined compressive strength (UCS) has being the most common and adaptable method of evaluating the strength of stabilized soil (Mustapha, 2006). This research is not focused on strength development as it has been established in literature the strength development of the use of calcium carbide and another admixture like zeolite. However it seeks to verify the extent to which the effectiveness of the recommended strength capacity of the stabilized soil within a specified sample mass. The statistical test carried out on the result helps us to understand the authenticity of the calculated mean of the failure stresses and shows the range of performance of the soil within the condition specified by the objectives. It is the primary test that is recommended for determining the amount of additive that should be used in soil stabilization (Singh, 1991). The results of the UCS test of soil stabilized with Calcium Carbide residue and zeolite at fixed percentages of 5% and 2% is shown. Appendix for BSL compaction energy level, because of the nature of the soil particles requiring little effort to cause a rearrangement of the soil structure in the presence of moisture, cured for 28 days as it aims to measure strength gain to the mean time duration of strength gain of pozzolana reaction with soils (Osinubi, 1998).

Table 4.5: Unconfined Compressive Strength Failure Stresses of two batched Samples

5% CCR- 0% ZEOLITE			5% CCR-2% ZEOLITE		
Failure Stress kN/m^2	Mean Stress	Std. Dev.	Failure Stress kN/m^2	Mean Stress	Std. Dev.
398			715		
382			700		
388			712		
368			687		
374			728		
384			702		
354			684		

390			695		
376			688		
381			708		
385			694		
397			690		
367			698		
377			721		
384			715		
358			700		
354			716		
388			721		
374			694		
371			689		
385			692		
392			712		
390			695		
364			690		
358			698		
374			725		
381			720		
390			712		
385			714		
380			722		
362			704		
358			695		
369			705		
360			714		
384			701		
370			721		
378			708		
398			698		
390			688		
368	377.15	12.50	698	704.23	12.27

From Table 4.5 a marginal consistency is observed among the samples tested and a mean score of about 337.15 kN/m^2 for CCR stabilized soil with 5% zeolite and 704.23 kN/m^2 for CCR stabilized soil admixed with 2% zeolite. Their standard deviations are 12.5 and 12.27 respectively. This shows that the data points generated from the experiment represent an almost absolute scenario in material behaviour during testing.

4.3.2.2. Comparison of UCS results with recommended standards

Overseas Road Note 31 (1993) recommends the range of 3000 -6000 kN/m^2 for specimen cured for 7 days, as a road base for heavily trafficked roads (CBI) and the range of 1500 -3000 kN/m^2 as a road base for lightly trafficked roads (CB2) Singh, (1991) also recommend UCS of 1720 kN/m^2 for specimen cured for 7 days for lightly trafficked road bases (CB2) from 2750– 3450 kN/m^2 for heavily trafficked road bases (CB1). Nigerian specification (1997) also recommended 1720 kN/m^2 with specimen cured for 7 days for lightly trafficked road bases which is in an agreeable range with both Road Note 31 (1993) and Singh (1991)

For 1720 kN/m^2 recommended for lightly trafficked (CB2) road bases by Road Note 31 (1993), Singh, (1991) and Nigerian general specification (1997) after 7 days curing period, the peak replacement level selected for the BSH energy level was adequate.

4.3.2.3 The effect of the mean and standard deviation on the UCS results

The mean for 5% CCR and 0% zeolite is 337.15, with a standard deviation of 12.5 as presented in table C1 Appendix. This is lower than the higher mean for 5% CCR and 2 percent zeolite, which is 704.23 kN/m^2, with a lower standard deviation of 12.27 and represents 86.7% increment from the original strength mean value. This means that the second samples with zeolite admixture have much less individual variation in their strength values. It can be deduced that as the zeolite percentage increases, the likelihood of getting a more uniform result with the strength of a larger number of samples increases as well. This lends credence to the requirement for a higher zeolite proportion.

4.3.2.4 The effect of confidence limits for mean and standard deviation of UCS results

The calculations that yielded the upper and lower bound values for the mean confidence values are shown in the Table 4.6. It also depicts the level of uncertainty surrounding the effect of the strength UCS value for any specific soil sample subjected to the same conditions. The sample size was set at 40 to guarantee that the confidence interval remained small and that the estimated results were more exact. The 95 percent and 99 percent confidence limits represent the likelihood of a selected sample behaviour performing within the lower and upper bounds in 95 and 99 percent of the time, respectively.

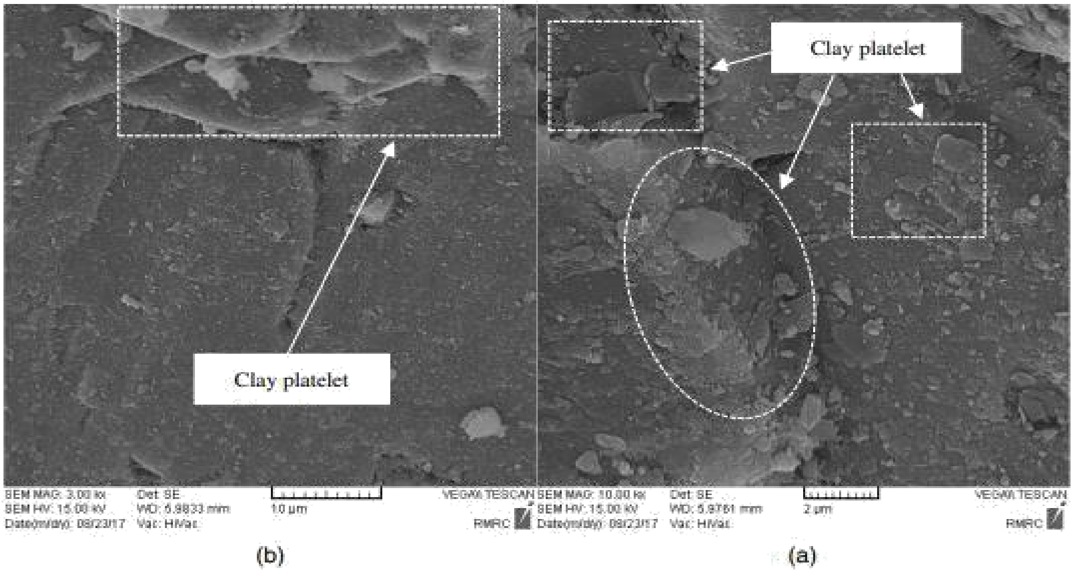
Table 4.6: Confidence Limit (99% and 95%) for Mean Strength and Standard Deviation

	95% CONFIDENCE LIMIT		**99% CONFIDENCE LIMIT**	
	5% CCR 0% ZEO	**5%CCR 2%ZEO**	**5%CCR 0%ZEO**	**5%CCR 2% ZEO**
Average	377.15	704.23	377.15	704.23
Standard Deviation	12.50	12.27	12.50	12.27
Sample Size	40	40	40	40
Confidence Coff	1.96	1.96	2.58	2.58
Margin of Error	3.87	3.80	5.10	5.00
Upper Bound	381.02	708.03	382.25	709.23
Lower Bound	373.28	700.42	372.05	699.22
Max	398	728	398	728
Min	354	684	354	684
	95% CONFIDENCE LIMIT STANDARD DEVIATION		**99% CONFIDENCE LIMIT STANDARD DEVIATION**	
	5% CCR 0% ZEO	**5%CCR 2%ZEO**	**5%CCR 0%ZEO**	**5%CCR 2% ZEO**
UPPER BOUND	15.84	15.5	16.58	16.28
LOWER BOUND	10.14	9.95	9.55	9.38

The 95 percent confidence limit and 99 percent confidence limit for failure stress represent the maximum range within which any given soil sample with the same features as examined will fall for failure stress if subjected to the same circumstance. This test's inferential significance is that it provides an almost exact prediction of the soil behaviour for any given admixture ratio. As it stands, we know that A-7-6 soil stabilized with 5% CCR and 2% zeolite will produce a shear stress value of not more than 12.27 from the mean of 704.23 kN/m^2 to be significant. This suggests that based on the results of this experiment, there is a 95 percent to 99 percent chance of obtaining a statistically meaningful soil strength UCS value between 691.96 kN/m^2 and 716.5 kN/m^2 as an experimental result for this particular admixture ratio and by statistical analysis from range (9.95 – 15.5). There is also a 99% chance of obtaining statistically correct result for soil strength UCS value that is between (9.38 – 16.28) from the mean value.

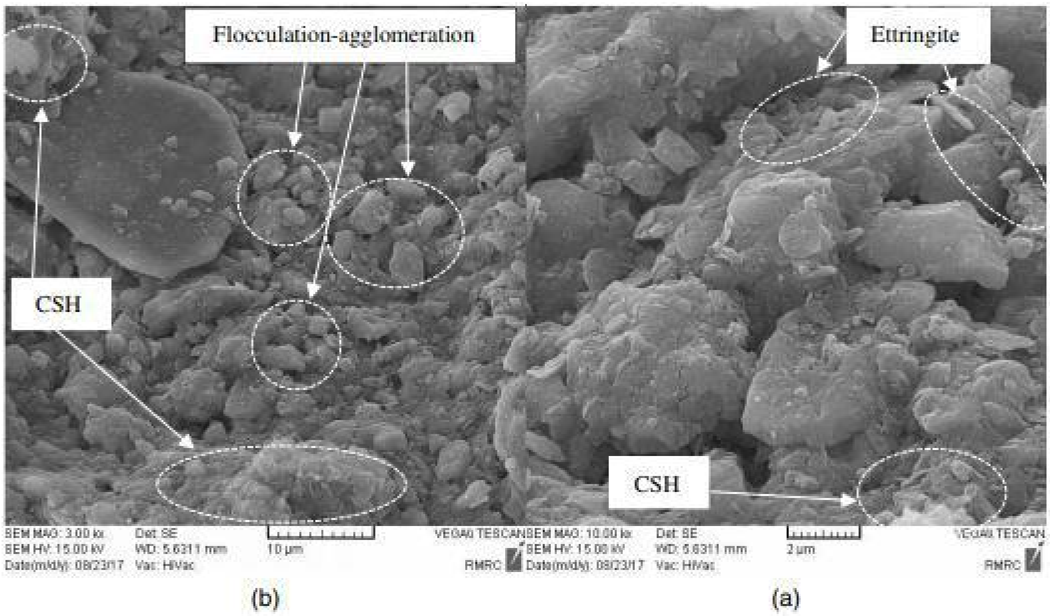
4.3.3 Microstructural analysis of material and stabilizing agents

In this investigation, SEM imaging was employed to dig deeper into the reasons underlying the observed results. For this objective, SEM was utilized to examine the microstructure of the soil and the causes of changes in strength parameters, as well as to supplement the findings. Figure 4.5 a and b shows a SEM of nonstabilized clayey sand soil at magnifications of 10,000× Figure 4.5 (a) Appendix and 3,000× Figure 4.6 (b).



(b) (a)

Figure 4.5: SEM images of nonstabilized clayey soil at magnifications of (a) 10,000×; and (b) 3,000×



(b) (a)

Figure 4.6: SEM images of soil stabilized with 5% CCR and 2% zeolite during curing time of 28 days at magnifications of (a) 10,000×; and (b) 3,000×

The soil structure is visible in the figures because stratified and clay scales may be observed. Soil does not work consistently in this type of structure and does not have a high strength. During the 28-day curing period, SEM pictures of calcium carbide stabilized soil with 2% zeolite are shown in Figure 4.6 (a) and (b) Appendix. As can be seen, the sample's layered structure has been destroyed as a result of zeolite stabilization during the curing process, and it has become an integrated structure. This could be owing to the occurrence of chemical events like ion exchange as a result of the addition of zeolite. These reactions result in a reduction in double-layer thickness and the integration of the soil structure. When zeolite is added to the clay and CCR matrix combination, the porosity of the stabilized soil is reduced, which improves the surface characteristics of micro particles in the final combination. Previous research has also revealed that adding zeolite to the soil alters the soil structure, causing empty pores to fill and pozzolanic reactions to occur, in 28 days of treatment Mola Abasi *et al.* (2016), there was a gain in strength. Furthermore, the created CSH product increased considerably in response to zeolite stabilization, resulting in the development of ettringite crystals and a significant increase in the binding between clay particles, in correspondence to the XRD data.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The purpose of the research was to interpret the strength behaviour of zeolite on calcium carbide stabilized clay soil. The soil taken from Niger State Polytechnic Zungeru was classed as A-7-6 (GI= 12) subgroup according to AASHTO soil classification and clay of low plasticity CL, with a plasticity index of 30.2% which revealed the dispersed nature of the soil particles and confirmed the need for stabilization by chemical method to improve the strength property of the soil.

Standard proctor compaction energy British Standard Light (BSL) helped to achieve a reported Optimum moisture content (OMC) of 18% with a maximum dry density (MDD) of 1.684g/cm^3 which was then used to carry out the unconfined compressive strength (UCS) test for the stabilized soil with zeolite addition.

A tabular presentation is made of the results from the UCS test carried out for Calcium carbide stabilized clay at 5% and Stabilized clay admixed with zeolite 2% for sample size of (40) to ensure that variability between samples was established. In general, 5% calcium carbide concentration soil treated with 2% zeolite generated a minor increase in MDD from the range of 1.443 g/cm^3 – 1.684 g/cm^3 while the OMC demonstrated an increase in value for all samples from 12% - 18% at the same admixture combination. For the samples examined, the average UCS value increased significantly from 377.15 kN/m^2 to around 704.23 kN/m^2 after 28 days of curing.

As it stands, we know that A-7-6 soil stabilized with 5% CCR and 2% zeolite will produce a shear stress value of not more than 12.27 from the mean of 704.23 kN/m^2 to be significant. Also soil stabilized with 5% CCR and 0% zeolite will produce a shear stress value not more than 12.5 from the mean of 377.15 kN/m^2 to be statistically significant.

5.2 Recommendations

1. 10% by dry weight addition of CCR to unsuitable soil causes an acceptable UCS value for road bases Manasseh (2012), an attempt to balance the use of CCR with admixture of zeolite should be tried with a balanced mix proportion of 5% CCR and 5% zeolite instead of 5% CCR and 2% zeolite as reported by this study.

2. The admixture ratio used (which was 5% CCR and 2% zeolite at a 18% OMC and 1.684 g/cm^3 maximum dry density gave the optimum strength UCs value of 704.23 kN/m^2) in this research did not meet the criteria for use as low traffic road base material which is 1720 kN/m^2 Road note 31 (1993) due to the low cation exchange potential of the calcium carbide residue (CCR) however the statistical method applied to ascertain the confidence limit of this admixture has shown that at this rate of 5% CCR and 2% zeolite in stabilizing clay; in whatever variation will not yield any value close to the minimum requirement for standard low vehicle road bearing road. It is therefore advised to try this method for already established values and establish a range of admixture for any given suitable stabilization mix ratio used.

3. Other Statistical Analysis like Reliability can be used to further explain how the efficiency of the fixed mixture of recommended soil will perform under sustained loading.

5.3 Contribution to Knowledge

The study established that: The clay stabilized with 5% Calcium carbide residue (CCR) admixed with 2% zeolite increased the unconfined compressive strength (UCS) to a value of 3589 kN/m^2

which was adequate for base material of heavily trafficked roads. CCR which is a waste material can be beneficially used in road construction.

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APPENDICES

APPENDIX A: INDEX PROPERTIES OF SOIL

Table A1: Particle Size Distribution

Project: Tersoo Thesis

Test Location:

Sample NO: Clay soil		Initial Sample mass : 300g		Date: 29/03/2020

Sample Description:

Sieve size(mm)	Mass Retained(g)	% Retained	%Passing
5	0.4	0.13	99.87
3.35	1.3	0.43	99.87
2	2.6	0.87	98.57
1.18	4.9	1.63	96.93
0.85	3.4	1.13	95.8
0.6	6.4	2.13	93.67
0.425	9.1	3.03	90.63
0.3	7.6	2.53	88.1
0.15	23.1	7.7	80.4
0.075	4.1	1.37	79.03

Hydrometer Analysis

Table A2: Hydrometer Analysis

Input Parameters		
Velocity of wateer at 25. C temperature	0.00000922	gs/cm2
Specific gravity of soil	2.6	
Weight of dry soil	50	g
Zero correction	7	G
Miniscus correction	1	

Time (MN)	Ra	T	tc = -4.85 + 0.25T	Rc = Ra-Zc+ Tc	% Finer = (Rcxa)/Ws	Recorrected for Minuscus	L = 16.3 - 0.164Ra	k	D(mm)	Actual % finer wrt to total fines in soil mass
									0.075	79.03
1	50	27	1.9	44.9	90.70	51	7.936	0.013148	0.0370	71.68
2	48	25	1.4	42.4	85.65	49	8.264	0.013148	0.0267	67.69
4	47	27	1.9	41.9	84.64	48	8.428	0.013148	0.0191	66.89
8	46	27	1.9	40.9	82.62	47	8.592	0.013148	0.0136	65.29
15	46	27	1.9	40.9	82.62	47	8.592	0.013148	0.0100	65.29
30	44	27	1.9	38.9	78.58	45	8.92	0.013148	0.0072	62.10
60	42	27	1.9	36.9	74.54	43	9.248	0.013148	0.0052	58.91
120	40	27	1.9	34.9	70.50	41	9.576	0.013148	0.0037	55.71
240	38	27	1.9	32.9	66.46	39	9.904	0.013148	0.0027	52.52
480	34	26	1.65	28.65	57.87	35	10.56	0.013148	0.0020	45.74
1440	32	25	1.4	26.4	53.33	33	10.888	0.013148	0.0011	42.15

Ra	Actual Hydrometer reading
T	Temperature
Tc	Temperature Correction
Rc	Corrected Hydrometer reading
Zc	Zero Correction
L	Effective Depth
	where, $K = \sqrt{\frac{30\eta}{-1}}$
D	Diameter of particles in mm

$$D = \sqrt{\frac{30\eta}{(\quad -1)}}\ \sqrt{\frac{\ }{\ }}$$

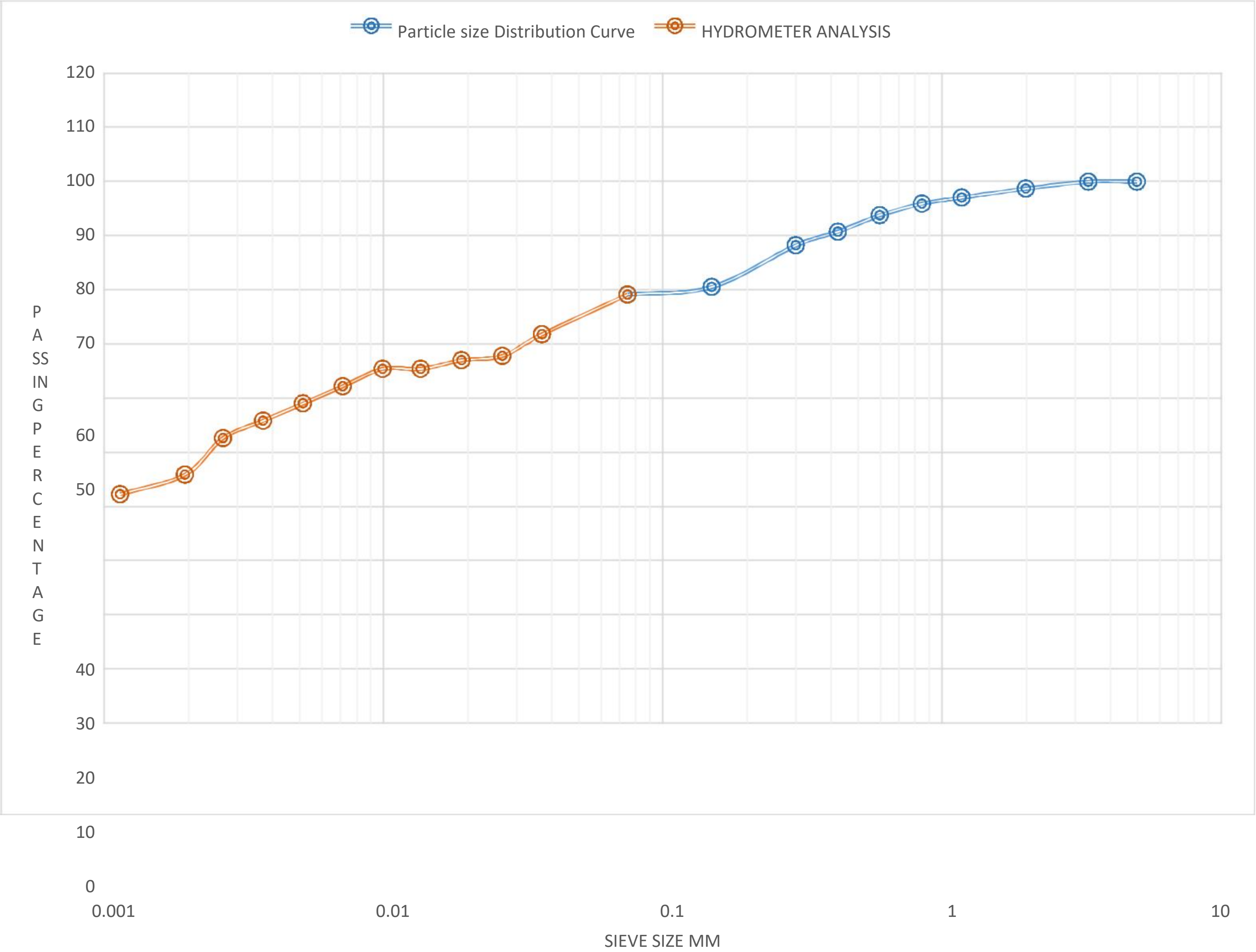

Figure A1: Grain size Analysis and Hydrometer Graph