# SUSTAINABLE USE OF CALCIUM CARBIDE RESIDUE ADMIXED WITH ZEOLITE FOR STABILIZATION OF CLAYEY SOIL

BY

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FEBRUARY, 2022

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A 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 (GEOTECHNICAL ENGINEERING)

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#### ABSTRACT

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 sustainable use of calcium carbide residue (CCR) admixed with zeolite to a stabilize clayey soil. Tests to determine the index properties, microstructural properties and unconfined compressive strength (UCS) of the natural soil and its stabilization with varying percentages of CCR and Zeolite were carried out. The outcome showed that the clay mixed with zeolite alone resulted in to gradual increase in UCS after 1-day curing from 54kN/m<sup>2</sup> at 0% zeolite to 94kN/m<sup>2</sup> at 6% zeolite. This represents 74% increase in strength. With addition of 5% CCR after 1 day curing, the UCS increased from 378kN/m<sup>2</sup> at 0% zeolite to maximum of 579kN/m<sup>2</sup> at 2% zeolite after which the values reduced to 371kN/m<sup>2</sup> at 6% zeolite. This also represents 53% increase in strength which must have been generated from the reaction between zeolite and CCR and clay soil. On variation of the CCR and Zeolite percentages, the UCS increase was maximum at 146, 873 and 1460 kN/m<sup>2</sup> for 0, 5 and 10% CCR respectively with the optimal zeolite occurring at 4% zeolite. This shows that CCR and Zeolite are viable additives for stabilization of clay soils.

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#### **CHAPTER ONE**

### 1.0 INTRODUCTION

#### **1.1** Background to the Study

The increasing population of the world, especially developing nations has led to increasing demand for roadways, railways, housing facilities and other infrastructures (Balarabe and Mary, 2018). Soil with higher stability is required to bear the weight of these structures; generally speaking, the stability of any construction related structure indirectly or directly depends on the soil stability. Expansive soils swell or increase in volume in humid seasons on absorption of water, and shrink or reduce in volume because of evaporation of water in dry seasons.

As a result of the alternate swelling and shrinkage of expansive soils, structures such as foundations, pavements and residential buildings etc. constructed on it experienced severe damage. The objective of expansive soil stabilization may be to stabilize volume change characteristics, modify plasticity and improve workability, or modify plasticity and volume change characteristics while substantially improving strength. Generally, engineers aim is to achieve the last one because the subgrade layer must not only be volumetrically stable, but must also support traffic or building loads. Targeting strength gain is also one of the reasons why a particular method of soil improvement is adopted. Unlike manufactured products, some soils have to be improved to meet the engineering specifications for the intended use, this is because they come with some deficiencies due to the geologic processes that the soil have undergone during the formation (Jayanthi and Singh, 2016).

The issue then becomes what method to be used to achieve stabilization and how to validate stabilization in clay soils. Chemical stabilization as an extensively used soil improvement technique for expansive soil has the advantage of achieving adequate strength over a short period of time. For the past several years, researchers have recognized the use of locally available materials which are cost effective, available from industrial and agricultural wastes to improve the properties of expansive soils with the aim to reduce stabilization costs. The most effective chemical for soil stabilization was identified to be cement. However, the cost of its production and emission of carbon dioxide related to its production made it expensive for use as stabilizer for soils. Calcium carbide residue being an industrial waste which creates nuisance in the environment, utilizing them in stabilization will pave a way of turning waste materials to something useful and economical (Balarabe and Mary, 2018). Ca(OH)<sub>2</sub> rich waste material like calcium carbide residue (CCR) together with pozzolanic materials such as Fly ash, rice husk ash, biomass ash etc. have been widely used to completely replace cement in stabilization of expansive soils (Kampala et al., 2013). Calcium carbide residue (CCR) is a by-product of the acetylene production process that contains mainly calcium hydroxide Ca(OH)<sub>2</sub>. Zeolites have been used in many applications because they can function as a molecular sieve owing to their ion-exchange ability, adsorption and absorption properties, crystal structure and silica content, as well as their lightweight, porous structures.

This study focused on the sustainable use of Calcium carbide residue admixed with Zeolite for stabilization of clay soil using Unconfined compressive strength (UCS) as evaluation criteria.

## **1.2** Statement of the Research Problem

Unlike manufactured products, some soils have to be improved to meet the engineering specifications for the intended use, this is because they come with some deficiencies due to geologic processes the soil have undergone during the formation (Jayanthi and Singh, 2016). Civil engineers are often faced with the problem of construction on or with soils, which in many cases do not possess sufficient strength and high bearing capacity to support the load imposed on them, either during the construction or the service life of the structures. As a result of the poor performances of these soils, it is important for engineers to device various methods of improving the properties of the soil, depending majorly on the soil type, its characteristics and degree of improvement desired.

## **1.3** Aim and Objectives of the Study

The aim of this research is to determine the sustainable use of calcium carbide residue admixed with zeolite for the stabilization of clayey soil.

The objectives are to;

- 1. Determine the index properties of the clay soil.
- 2. Determine the microstructural composition characteristics of the natural and stabilized clay.
- 3. Determine the unconfined compressive strength of the natural and stabilized clay at 1, 7, 14, 28, 60 and 90 days of curing.

## **1.4** Justification for the Study

The conventional method of using cement and lime for stabilization is more expensive, thus increasing the cost of civil engineering construction. This study will go a long way to minimize environmental pollution caused by waste calcium carbide residue (CCR) and to improve deficient clay soils with cheap and environmentally friendly waste materials (CCR and Zeolite). Consequently, the cost of civil engineering construction will reduce and the use of cement minimized.

### **1.5** Scope of the Study

This research work was limited to laboratory investigations on clay soil samples collected behind the library complex of Niger State Polytechnic, Zungeru of Niger State, Nigeria. The Calcium Carbide Residue used for this study was collected from local welders at Keteren-Gwari mechanic site, Minna, Nigeria while the zeolite was purchased from a commercial market in Kaduna in Kaduna State of Nigeria. The Index properties, Compaction and Unconfined Compressive Strength tests were conducted in Civil Engineering Laboratory, Federal University of Technology, Minna. The unconfined compressive strength of the natural and treated clay was tested after 1, 7, 14, 28, 60 and 90 days of curing. The microstructural characteristics of the material and additives were carried out in the Ithemba laboratory and physics laboratory, university of cape town.

### **CHAPTER TWO**

## 2.0 LITERATURE REVIEW

# 2.1 Clay

Clay is a finely-grained natural rock or soil material that combines one or more clay minerals with possible traces of quartz (SiO<sub>2</sub>), metal oxides (Al<sub>2</sub>O<sub>3</sub>, MgO) and organic matter. Geologic clay deposits are mostly composed of phyllosilicate minerals containing variable amounts of water trapped in the mineral structure. Clays are plastic due to particle size and geometry as well as water content, and become hard, brittle and non-plastic upon drying or firing. (https://www.turfonline.co.uk)



Figure 2.1: Clay soil baking hard during hot dry weather (Source: Nazile, 2018)

Clay minerals are also 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. 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 (Figure 2.2a). Illite has layers made from two silica plates and one alumina plate (Figure 2.2b). 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 (Figure 2.2c). This event causes the swelling of such clay.



S:Silica layer, A:Alumina layer, K:Potassium ioans

Figure 2.2: Display of structure of common clay minerals (Source: Nazile, 2018)

## 2.2 Clay Properties

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 (Figure 2.3). 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 behavior of the soil. Studies generally accept clays as fully saturated in geotechnical engineering. Therefore, the behavior 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. The interaction of these forces controls the engineering behavior 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. 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. (Nazile, 2018).

## 2.3 Structure of Clay and Physio-Chemical Properties

Around the clay that is faced with a liquid, there are distance-varying pushpull 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 (Figure 2.3).



Figure 2.3: Clay particle and surface charge display (Source: Nazile, 2018)

Forces between the particles are important for clay, because the behavior of clay depends on the geological history and structure. This difference in the orientation of fine-grained soils affects the engineering behavior 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 behavior of soils-bearing structures, as well as the strength between the structure, texture and performance of soils. Ingles examined soil fabric during consolidation. Due to an increase in the degree of particle orientation, the total volume of the voids was reduced.



Flocculated type Disperced type

Figure 2.4: Clay particle orientations (Source: Nazile, 2018)

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. 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. Figure 2.5 shows further compression with progress of soil sedimentation. Later, other researchers also proposed fabric models.



Figure 2.5: Casagrande's fabric model (Source: Nazile, 2018)

Researchers provided an elementary particle arrangement, a single clay, silt or sand, which is shown in Figure 2.6(a) and (b), the group effect of clay plates is shown in Figure 2.6(c) and the interaction between silt and sand is shown in Figure 2.6(d). Particle assemblages contain one or more elementary particle arrangements or small particle clusters. Pore spaces are defined with spacing between elementary particle arrangements and particle assemblages. 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).



**Figure 2.6:** Arrangement of clay particles. (a) Elementary particle arrangements of clay; (b) Elementary particle arrangements of sand and silt; (c) Clay assemblages; (d) Clay coated silt and sand arrangement; (e) Not fully defined arrangement (Source: Nazile, 2018)

## 2.4 Role of Clay in Geotechnical Engineering

Studies on soil behavior that do not consider the physio-chemical and microstructural properties of clay soils may be missing important information regarding the soil's physical and mechanical properties. This is because most physical and mechanical behaviors can be explained by the soil's physio-chemical and microstructural properties. In general, clay is an unwanted material because it creates significant engineering problems. Unlike other minerals of the same size, clay forms mud when mixed with water. Clay has plasticity and can be shaped into dough, and when cooked it turns into a

solid with great strength increments. Clay generally shows a volume increase when wet, and when it is dried, its volume decreases, which creates many cracks (Nazile, 2018).

#### 2.5 Physical and Mechanical Behavior of Clay

In geotechnical engineering, it is important to identify a clay type, as the type directly affects the important properties of clay, such as Atterberg limit, hydraulic conductivity, swelling-shrinkage, settlement (compression) and shear resistance. Atterberg limit, known as consistency limits, define the relationship between ground particles and water and the state of the soil relative to varying water contents. With increasing moisture content, clay changes from solid state, to semisolid state, to plastic state and to liquid state, which is given in Figure 2.7. In Figure 2.7, the clay-water mixture shows a total volume reduction, which is equivalent to the volume of water lost around the liquid and plastic limits, as the clay transitions from liquid to dry, and if the decrease in water content continues, no reduction in volume is observed. This limit value is called the shrinkage limit. Therefore, the shrinkage limit is the moisture content at which the soil volume will not reduce further if the moisture content is reduced. The plastic limit is the moisture content at which the soil changes from a semisolid to a plastic (flexible) state. The liquid limit is the moisture content at which the soil changes from a plastic to a viscous fluid state. In geotechnical engineering, the liquid and plastic limits are commonly used. These limits are used to classify a fine-grained soil, according to the Unified Soil Classification system, AASHTO system or TS1500 (Turkey).



Figure 2.7: Water content-volume relationships of soils (Source: Nazile, 2018)

#### 2.5.1 Hydraulic conductivity properties of clay

Water is a problem in geotechnical engineering, such as water in voids in the ground mass, flowing in pores, or in the pressure or stress that water creates in the pores. Clay plays an important role in the emergence of water problems, especially on fine soils, and these problems include permeability, shear resistance, setting and swelling problems. In addition, capillarity, freezing and infiltration can be additional issues. Structures built on clay and slope stability are particularly problematic when affected by water. Dams and dikes also cause the destruction of structures without leakage and piping. Therefore, it is necessary to estimate the quantity of underground seepage under various hydraulic conditions to investigate problems that involve pumping water for underground construction and for stability analyses of earthen dams and earth-retaining structures that are subject to seepage forces.

The hydraulic conductivity coefficient commonly used in geotechnical engineering is also used for permeability. Hydraulic conductivity is a property that expresses how water flows in the soil. Soils are permeable due to the existence of interconnected voids, through which water can flow from the points of high energy to the points of low energy. Fluid viscosity, pore-size distribution, grain-size distribution, void ratio, roughness of particles and the degree of soil saturation affect the hydraulic conductivity of soils. Clay soil has electrical ions, so the hydraulic conductivity of clays affects the ionic concentration and thickness of water layers held to the clay particles. Table 2.1 provides the typical values for soils. The hydraulic conductivity value of soils determines the constant head test (for coarse soils) and the falling head test (for fine-grained soils).

**Table 2.1:** Hydraulic conductivity of soils

| Soil Type | Clean gravel | Coarse sand | Fine sand  | Silty clay   | Clay      |
|-----------|--------------|-------------|------------|--------------|-----------|
| K(cm/s)   | 100-1.0      | 1.0-0.01    | 0.01-0.001 | 0.001-0.0001 | <0.000001 |

(Source: Das, 1998)

## 2.5.2 Swelling-shrinkage behaviour of clay

The effect of swelling-shrinkage on fine-grained soils is often seen as a problem in geotechnical engineering applications. Shrinkage behavior in clay soils is effective in reducing the strength in a slope and a foundation's bearing capacity. Shrinkage is usually visible from evaporation in dry climates, reduction of groundwater and sudden arid periods. Swelling can be seen due to rising water. These volume changes are harmful to heavy construction and road coverings. Swelling occurs when the inflation pressure is greater than the pressure from the covering or structure. The material damage from the swelling-shrinkage of soils is more likely to occur in the United States due to greater water pressure, floods, typhoons and earthquakes. Jones and Jefferson (2012) estimated that shrinking and swelling soils cause approximately \$2.3 billion in

damage annually to small buildings and road surfaces in the United States. This amount of damage is twice the amount of damage incurred from floods, earthquakes and hurricanes.

Swelling pressure depends on the type of clay mineral, soil structure and fabric, cation exchange capacity, pH, cementation and organic matter. Any cohesive soil can involve clay minerals, but montmorillonite or bentonite clay minerals are more active regarding swelling-shrinkage. Swelling is calculated by swelling experiments with chemical and mineralogical analysis, soil indices and some empirical formulas from soil classifications. The shrinkage limit is determined from a laboratory test or approximate calculation recommended by Casagrande. Properties of clay improve with chemical additives such as cement, lime, lime-fly ash, cement-fly ash, calcium chloride and so on.

Structures transfer loads to the subsoil through their foundations. The imposed stress from the structure compresses the subsoil. This compression of soil mass leads to a decrease in the volume of the mass, which results in the settlement of the structure, and this should be kept within tolerable limits. Therefore, settlement (compression) should be estimated before construction. The settlement is defined as the compression of a soil layer due to the construction of foundations or other loads. The compression is seen in deformation, relocation of soil particles and expulsion of water or air from void spaces. In general, the soil settlement under load falls into three categories: immediate or elastic settlement, which is caused by the elastic deformation of dry soil or moist and saturated soils without change in the moisture content; primary consolidation settlement, which is the result of a volume change in saturated cohesive soils because of the expulsion of water occupying void spaces; and secondary consolidation settlement is the volume change under a constant effective stress due to the plastic adjustment of soil fabrics.

#### 2.6 Soil Stabilization

Soil stabilization is a method of improving soil properties by blending and mixing it with other materials. Soil stabilization can also be seen as a process of improving the shear strength parameters of soil and thus increasing the bearing capacity of soil. It is required when the soil available for construction is not suitable to carry structural load. Soil stabilization is used to reduce permeability and compressibility of the soil mass in earth structures and to increase its shear strength. Soil stabilization involves the use of stabilizing agents (binder materials) in weak soils to improve its geotechnical properties such as compressibility, strength, permeability and durability. One may achieve stabilization 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 soil voids. Soil stabilizing additives are used to improve the properties of less-desirable rood soils. When used these stabilizing agents can improve and maintain soil moisture content, increase soil particle cohesion and serve as cementing and water proofing agents (Habiba, 2017). A difficult problem in civil engineering works exists when the sub-grade is found to be clay soil. Soils having high clay content have the tendency to swell when their moisture content is allowed to increase. Many researches have been done on the subject of soil stabilization using various additives, the most common methods of soil stabilization of clay soils in pavement work are cement and lime stabilization. The high strengths obtained from cement and lime stabilization may not always be required, however, and there is justification for seeking cheaper additives

which may be used to alter the soil properties. Chemical stabilization is commonly used to improve engineering properties of problematic soils (Nima *et al.*, 2018). Lime or calcium carbonate is oldest traditional chemical stabilizer used for soil stabilization.

#### 2.6.1 Soil stabilization methods

Soils can be stabilized by the addition of cement or lime. Such stabilization processes improve the various engineering properties of the stabilized soil and generate an improved construction material. Increase in soil strength, durability stiffness, and reduction in soil plasticity and swelling/shrinkage potential are the benefit of soil stabilization (Das *et al.*, 2015). The concept of stabilization is 5000 years old.

Stabilized earth roads were used in ancient Egypt and Mesopotamia and that the Greeks and Romans used stabilization tests were performed in the United States in 1904 (Firoozi *et al.*, 2014). Cement was introduced as a stabilizer to construct a street in Sarasota, Florida, in 1915 (American Concrete Institute) 1997, and lime was first involved in short stretches of highway with the expansion of roads to cater for the growth of vehicle traffic in 1924 (Khan *et al.*, 2014). Traditional stabilizers typically depend on pozzolanic reactions and cation exchange to modify and stabilize soil. Pozzolanic reactions occur when siliceous and aluminous materials react chemically with calcium hydroxide at regular temperatures to form cementitious compounds. On the other hand, a cation exchange occurs when the soil is able to exchange free cations available in the exchange locations (Little, 2009).

Generally, soil stabilization is a method of improving soil properties by blending and mixing other materials. Improvements include increasing the dry unit weight, bearing

capabilities, volume changes, the performance of in situ subsoils, sands, and other waste materials in order to strengthen road surfaces and other geotechnical applications.

#### 2.6.1.1 Stabilization using cement

One of the common methods of chemical stabilization is to mix soil with cement to form a product named as soil-cement. Soil-cement can be defined as a mixture of soil and measured amounts of Portland cement and water and compacted to the desired density. Soil-cement has been used as a base material as an adoption of improved measure in many projects, such as slope protection of dams and embankments, pavement of highways, building pads, terminals for rail and truck, composting facilities, cheap base for streets, parking lots, channels and reservoir linings, mass soil-cement placement for dikes, foundation stabilization etc. The soil-cement technique has been practiced almost for 100 years. It serves to amend the mechanical and the engineering properties of the soil. The new performance depends on the ability of the additives to react with the mixing soil. There are four main properties of soil; strength, permeability, volume stability, and durability that can be enhanced with additives (Anggraini et al., 2014). The choice of a specific additive depends on the type of soil, service that is required to serve and the surrounding environment. When water is mixed with cement, hydration occurs, meaning cementing compounds of calcium–silicate–hydrate (C–S–H) and calcium-aluminate-hydrate (C-A-H) are formed and excess calcium hydroxide (CaOH) is released, approximately 31% by weight. Formation of C-S-H and C-A-H occurs when crystals begin forming a few hours after the water and cement are mixed; crystals will continue to form as long as unreacted cement particles and free water remain within the mixture (Khan, 2015). Five standard types of Portland cement (Types I through V) are available as specified by ASTM C150. The process of cementation and the results of soil-cement and lime stabilization are similar, they used in quantities too

small to provide high-strength cementing action. They reduce the plasticity of clay soils. Calcium chloride or sodium chloride are added to the soil to retain moisture and also control dust, to hold fine material for better compaction, and to reduce frost heave by lowering the freezing point of water in the soil. Kezdi (1979) reports that cement treatment slightly increases the maximum dry density of sand and highly plastic clays but it decreases the maximum dry density of silt. In contrast, other studies shows that cement increases the optimum water content but decreases the maximum dry density of sandy soils. Cement increases plastic limit and reduces liquid limit, which mainly reduces plasticity index (Kezdi, 1979). The other significant effects of soil-cement stabilization is reduction in shrinkage and swell potential, increase in strength, elastic modulus, and resistance against the effect of moisture, freeze, and thaw. Cement treated soils show a brittle behavior compare to non-treated. Cement can be applied to stabilize any type of soil, except soils with organic content greater than 2% or having pH lower than 5.3 (ACI 230.1R-90, 1990). The use of cement in granular soils has proven to be economical and effective because smaller amounts of cement are required. In addition, soils that have a Plasticity Index value higher than 30 are difficult to mix with cement. To avoid this issue, lime can be added prior to mixing in cement; this initial step will keep soils more workable (Firoozi et al., 2014).

Swell pressure decrease as the stabilizer content increased in cement treated samples. Cementitious links develop between the calcium silicate and calcium aluminate found in Portland cement with the soil particles (Khan *et al.*, 2015). Unlike lime, hydration in cement occurs at a faster pace which allows for an immediate strength gain. Therefore, there is no need of a mellowing period when stabilizing with cement; compaction of soil–cement samples is typically conducted within 2 hour of initial mixing. The strength gain achieved during compaction may be below the ultimate strength of a soil–cement sample. However, the cement stabilized soil will continue to gain strength over the course of several days (Chittoori, 2008).

There are many factors contributing to the length of curing time required for strength gain in soil–cement samples. These include ambient air temperature, relative humidity, type of cement used, and concentration of cement used. The relative strength is sensitive to the previously mentioned factors, while the relative compaction is not. Faster wind speed, lower relative humidity and longer delay in compaction commonly result in a poor strength.

#### 2.6.1.2 Stabilization using lime

Limestone is broken down at elevated temperatures to form lime (Chittoori, 2008). As a result, three forms of lime are produced: quicklime (calcium oxide–CaO), hydrated lime (calcium hydroxide–Ca[OH]<sub>2</sub>), and hydrated lime slurry; all of which can be used to treat soils. Quicklime is manufactured by chemically transforming calcium carbonate (limestone– CaCO<sub>3</sub>) into calcium oxide. Furthermore, hydrated lime is created when quicklime chemically reacts with water. When hydrated lime is mixed with clay particles, it permanently forms strong cementitious bonds (Louafi *et al.*, 2015). Lime has been known to reduce the swelling potential, plasticity index and maximum dry density of the soil, and increases its optimum water content, shrinkage limit and strength. It improves the workability and compact ability of subgrade soils.

If quicklime is used, it immediately hydrates (chemically combines with water) and releases heat. Soils are dried, because water present in the soil participates in this reaction, and because the heat generated can evaporate additional moisture. The hydrated lime produced by these initial reactions will subsequently react with clay particles. These subsequent reactions will slowly produce additional drying because they reduce the soil's moisture holding capacity. If hydrated lime or hydrated lime slurry is used instead of quicklime, drying occurs only through the chemical changes in the soil that reduce its capacity to hold water and increase its stability. After initial mixing, the calcium ions ( $Ca^2+$ ) from 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. At this stage the Plasticity Index of the soil decreases dramatically, as does its tendency to swell and shrink. The process, which is called "flocculation and agglomeration," generally occurs in a matter of hours.

When adequate quantities of lime and water are added, the pH of the soil quickly increases to above 10.5, which enables the clay particles to break down. Silica and alumina are released and react with calcium from the lime to form calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH). Cocoanut shell ash and calcium-aluminate-hydrates are cementitious products similar to those formed in Portland cement. 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. The matrix formed is permanent, durable, and significantly impermeable, producing a structural layer that is both strong and flexible. However, alteration of particle structure occurs slowly, depending upon the type of clay present, a mellowing period from one to four days is allowed to obtain a homogeneous, friable mixture. These reactions maybe summarized as:

$$Ca^{2+}+OH^-+$$
 Soluble Clay Silica  $\rightarrow$  Calcium Silicate Hydrate (CSH) (2.1)

 $Ca^{2+}+OH^{-}+$  Soluble Clay Alumina  $\rightarrow$  Calcium Alumina Hydrate (CAH) (2.2)

Lime stabilization enhances engineering properties of soils, such as improved strength, higher resistance to fracture, fatigue, and permanent deformation, enhanced resilient properties, reduction in swelling; and resistance to the harmful effects of moisture. The most considerable improvements in these properties are observed in moderately to highly plastic clays (Solanki *et al.*, 2010).

Over the time, the properties of treated soil affect the strength gain. Soil pH, organic content, the quantity of exchangeable sodium, clay mineralogy, natural drainage, weathering conditions, extractable iron, carbonates and silica-alumina ratio are some of the properties which influence the gain in strength. The stabilization of acidic soil using lime, resulted in lower compressive strength than that of alkaline soil. Broderick and Daniel (1990). reported that the lime and cement stabilized soils are less vulnerable to attack by organic chemicals in comparison to untreated soils. Haraguchi et al. (1994) investigated the variation of the engineering properties of freshly cement-stabilized decomposed granite soil cured in water and in 0.2 N acid solution, and indicated that the California bearing ratio obtained from the specimens cured in the 0.2 N acid solution was lower than that cured in water (Haraguchi et al., 1994). The strong alkaline conditions were able to release silica and alumina from the clay mineral and eventually react with lime to form new cementation products. The success of the lime treatment process is highly dependent on the available lime content, curing time, soil type, soil pH and clay minerals (Kassim, 2004). Limited research has been conducted to determine whether pH variations will affect properties of lime-stabilized soils. Additional studies are therefore necessary to explain the erosion mechanism of lime-stabilized soils due to pH variations (Abdullah and Abdullah, 2013). However, experience shows that lime will react with many medium-, moderately fine and fine grained soils to produce

decreased plasticity, increased workability, reduced swell, and increased strength. Soils classified according to the USCS as CH, CL, MH, ML, OH, OL, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GWGC, GP-GC, ML-CL, and GM-GC should be considered as potentially capable of being stabilized with lime. Lime should be considered with all soils having a PI greater than 10 and more than 25 percent of the soil passing the No. 200 sieve [FM 5-472/NAVFAC MO 330/AFJMAN 32-1221(I)].

## 2.6.1.3 Stabilization using fly ash

Fly ash is one of four coal combustion products (CCPs) that are produced as a byproduct of burning coal, two major groups, Class C and Class F fly ash are produced. Burning lignite and subbituminous coal produces Class C fly ash. However, burning anthracite, other known as bituminous coal, produces Class F fly ash, (Bose, 2012). Although there can be multiple variations of the chemical additive, fly ash particles generally consist of hollow spheres of silicon, aluminum, and iron oxides and unoxidized carbon all of which make both classes of fly ash pozzolans-siliceous or siliceous and aluminous materials, Pandian et al. (2013). It is typically viewed as nonplastic fine silt (ML) when using the Unified Soil Classification System (USCS). Class F fly ash is not used as often because it requires an activator, either lime or cement, to form pozzolanic stabilized mixtures (PSMs) since it is not a self-cementing material, Firoozi et al. (2015). Zulkifley et al. (2014) assessed the effect of specification fly ash on the engineering properties of tropical soils from Hawaii. They observed that the fly ash reduced the liquid limit and plasticity index, and enhanced the California bearing ratio (CBR) and unconfined compressive strength. Radhakrishnan et al. (2014) performed unconfined compressive strength (UCS) and one-dimensional free swell tests on soil-lime mixtures altered with Class F fly ash. Observations showed that lime and

fly ash are a good combination for stabilizing silty and sandy soils. It drastically increased the stiffness of the final product. Rupnow *et al.* (2015) used Class C fly ash with lime to develop guidelines for estimating the coefficient of structural layer for the base layer of flexible pavement. The required base layer thickness decreased with the addition of both additives. Fly ash, when mixed with lime, can be used effectively to stabilize most coarse- and medium-grained soils; however, the PI should not be greater than 25. Soils classified by the USCS as SW, SP, SP-SC, SW-SC, SW-SM, GW, GP, GP-GC, GW-GC, GP-GM, GW-GM, GC-GM, and SC-SM can be stabilized with fly ash.

#### 2.6.1.4 Soil stabilization using fibers

The use of hair-sized polypropylene fibers in soil stabilization requests has been common in soil stabilization projects for its low cost compared with other stabilization agents. These materials have a high resistance towards chemical and biological degradation and do not cause leaching in the soil. Mohammed (2013) studied the improvement in the properties of swelling clay using hair fibers. He found that there was no considerable or reasonable change in the Atterberg limits due to introduction of hay fiber. The maximum dry density (MDD) decreased with the addition of hair. The optimum moisture content (OMC) decreased with increasing hay content up-to 1.0% then increased. The shrinkage limit decreased with increasing hay content up-to 1.0% then increased. The unconfined compression strength decreased with increasing of hay content to soil ratio. The direct shear strength increased notably with hay addition. The tensile strength of the air dried mixture increased with hay addition and the swelling decreased with hay addition.

Mehdi (2017) studied unconfined compressive strength (UCS) of reinforced clays with the waste of carpet fibers. He found that carpet waste fibers mixed with clay soils, prepared at the same dry unit weight, can considerably increase the unconfined compression strength (UCS), decrease the post peak strength loss, and alter the failure behavior from brittle to ductile. The results also indicated that the relative benefit of fibers to enhance the unconfined compression strength of the clay soils is largely dependent on initial dry unit weight and water content of the soil. Cristelo et al. (2015) studied the performance of fiber reinforced clayey soil. They found that the excessively compressible clayey soil reinforced with randomly mixed fibers, resulted in higher ultimate bearing capacity, and lesser settlement at the ultimate load. Yilmaz (2015) in studying the compaction characteristics of reinforced soil, found that the addition of fiber, affected compaction behavior of samples. The increment in fiber content and length of fiber caused increment in optimum moisture content (OMC) and slight decrement of maximum dry density (MDD). Anggraini et al. (2014) found that with the inclusion of fibers into the soil, the unconfined compressive strength and the corresponding strain at failure increase up to an optimum fiber content and length, and decrease thereafter. In addition; inclusion of fibers in soil increases the strain at failure, and therefore makes the reinforced soil matrix more ductile. The settlement under a particular load in unreinforced soil is much more than the reinforced soil. Minimum settlement being observed for the soil reinforced with polypropylene fibers.

Anggraini *et al.* (2014) studied the stiffness and strength response of coir fiberreinforced tropical soil. They found that the stress–strain behavior of soil improved by incorporating coir fibers into the soil. The maximum increase in strength was observed when the fiber length is between 15 and 25 mm. The stiffness of soil increases and also the immediate settlement of soil reduced considerably due to the inclusion of fibers in the soil. Shukla *et al.* (2010) studied analytical model of granular soils reinforced with fibers, under high confining pressures. They found that the apparent cohesion and shear strength increase is proportional to the fiber content and aspect ratio. The increase in shear strength of the granular soil stabilized with fibers was mostly from the apparent cohesion, and the contribution from the increase in normal confining stress was quite limited. Estabragh *et al.* (2013) studied the mechanical behavior of a fiber-clay composite with natural fiber. They found that the inclusion of the fiber reinforcement within the soil causes a decrease in pre-consolidation stress and increases compressibility and swelling indices. In addition, the strength and angle of internal friction increase considerably for the total and effective stresses.

## 2.6.2 Mechanisms of stabilization

Mechanisms of stabilization that utilize cement, lime, or fly ash were summarized as follows (Little, 1999):

- 1. Cation exchange: sodium, magnesium, and other cations are replaced by the calcium cations from the available calcium hydroxide.
- Flocculation and agglomeration: flocculation of the clay particles increases the effective grain size and reduces plasticity, thus increasing the strength of the matrix.
- 3. Pozzolanic reaction: the high pH environment created by the available calcium hydroxide solubilizes silicates and aluminates at the clay surface, which in turn react with calcium ions to form cementitious products that are composed primarily of calcium silicate hydrates or calcium aluminate hydrates, or both.
Li, (2014) provided a detailed description of the hydration process of cement concrete: If a proper amount of moisture is available to ensure complete hydration of the cement, then each cement particle is covered with water and thus forms a gel-like film. These coated particles of cement coat the aggregate or soil particles as well. At this stage, the cement has not begun to set. The reaction between the water and the cement forms small single crystals. With the development of hydration, the single crystals begin to grow into one another and form a crystalline network. The adjoining crystals are attracted to one another by van der Waals forces instead of a chemical bond. Figure 2.8 illustrates this process.



Figure 2.8: Advantages and disadvantages of calcium-based additives (Source: Li, 2014)

Soil stabilization using calcium-based additives involve advantages and disadvantages. Some of the advantages and disadvantages of calcium-based additives are discussed here.

#### 2.6.3 Advantages of calcium-based additives

The following list presents some of the engineering advantages of calcium-based soil stabilization (Solanki, 2010);

- a) Functions as a working platform (expedites construction).
- **b**) Reduces the need for dusting.
- c) Waterproofs the soil.
- d) Upgrades marginal aggregates or soils.
- e) Improves strength.
- f) Improves durability.
- g) Controls volume changes of soils.
- h) Improves soil workability.
- i) Dries wet soils.
- j) Reduces pavement thickness requirements.
- **k**) Conserves aggregate.
- I) Reduces construction and haul costs.
- m) Conserves energy.
- n) Provides a temporary or permanent wearing surface.

#### 2.6.4 Disadvantages of calcium-based additives

In spite of the significant advantages mentioned above, many serious disadvantages are inherent from calcium-based stabilizers.

a) Negative environmental impact: Global warming is a serious threat which our planet is facing. Carbon dioxide (CO<sub>2</sub>) is one of the main factors for this warming. Cement manufacturing is a process which emits  $CO_2$  in large amount

(Gao *et al.*, 2015). Cement industry alone produces about 10% of total  $CO_2$  fossil fuels, electricity, and transportation.

b) Sulfate attack and carbonation: Two deleterious chemical reactions occur in the soils treated with calcium-based materials. One of the sulfate salts existing in the soil and two lime carbonation. In the soils containing sulfates, any calcium-based additives result in heaving and disintegration leading to a loss in strength.

Soil composition, groundwater and mixed water can be the source of sulfates (Alsh. Chemical reaction between calcium and aluminum present in the soil mineralogy in the company of soluble sulfate and water produce ettringite and/or thaumasite. Ettringite is a hydrous calcium aluminum sulfate mineral with formula:  $Ca_6Al_2(SO_4)3(OH)_{12}.26H_2^O$ . It is a colorless to yellow mineral crystallizing in the trigonal system. The prismatic crystals are typically colorless, turning white on partial dehydration. Calcium sulfate sources, such as gypsum, are intentionally added to Portland cement to regulate early hydration reactions to prevent flash setting, improve strength development, and reduce drying shrinkage. Sulfate and aluminate are also present in supplementary cementitious materials and admixtures. Gypsum and other sulfate compounds react with calcium aluminate in the cement to form ettringite within the first few hours after mixing with water. Most of the sulfate in the cement is normally consumed to form ettringite at early ages. The formation of ettringite in the fresh, plastic concrete is the mechanism that controls stiffening. At this stage ettringite is uniformly and discretely dispersed throughout the cement paste at a submicroscopic level (less than a micrometer in cross-section). Thaumasite is a silicate mineral with chemical formula Ca<sub>3</sub>Si(OH)6(CO<sub>3</sub>) (SO<sub>4</sub>).12H<sub>2</sub>O. It occurs as colorless to white prismatic hexagonal crystals, typically

as acicular radiating groups. It also occurs as fibrous masses. Its Mohs hardness is 3.5 and it has a specific gravity of 1.88–1.9.

#### c) Effect of organic materials:

The high microbial biomass present in the organic soil activates rates of decomposition in organic soils treated with lime resulting into reduction in pH value. In addition, clay minerals are in lesser quantity in the organic soil. Therefore, the organic materials slow down the pozzolanic reaction necessary to attain soil strength. The organic soil is characterized by high water with holding capacity that may lead to a reduction in available water for the hydration process. Furthermore, organic materials coat the additive particles thus holding up the hydration process.

There is a great tendency for the reaction between calcium ions and black acid generated from lime to produce insoluble calcium humid acid. The decomposition of organic materials hinders the polymerization of silicate. Thus, the cementitious formation is blocked by organic materials. Though, it is key to know that not all of the organic materials block cementitious reaction. Some have no effects such as chloronaphthalene and others retard hydration reaction but do not affect soil strength gain such as ethylene glycol, benzoic acid, cellulose (Firoozi *et al.*, 2015).

Cement-treated soil to face the similar situation as mentioned for soils treated with lime. Shahram, *et al.* (2016) reported that soil containing organic acids and having pH less than 9 inhibit the cementing reaction. For the purpose to reduce the negative effect of organic materials, bentonite is added to the organic soil. This treatment provides two benefits, one, bentonite has high water retention ability, which is favorable for lime hydration. Two, bentonite is the source of silica for pozzolanic reaction and can become a filler. Kaolinite and zeolite may also be used for lime treated soil having humid acid. This gives enough amount of silica that is required for pozzolanic process.

## 2.7 Problems in sulfate soils stabilized with lime/cement

Though lime stabilization improves the volume and strength characteristics of the expansive soils, there are some limitations to lime stabilization. These limitations include the presence of organic carbon and soluble sulfates. It has been reported that the presence of organic carbon in excess of one percent can interfere with the pozzolanic reactions, leading to low strength gains. Compared to organic carbon, the presence of sulfates is of higher concern because lime treatment in these types of soils leads to excessive heaving and pavement failures. It has been reported that when soils contain sulfate minerals such as gypsum (CaSO $_4$ ·2H $_2$ O) and sodium sulfate (Na $_2$ SO $_4$ ) in their natural formation and are treated with calcium-based stabilizers, adverse reactions occur, causing severe heave and pavement distress. These adverse reactions are due to formation of expansive minerals, ettringite, (Ca<sub>6</sub>. [AL(OH)<sub>6</sub>]<sub>2</sub>. (SO<sub>4</sub>)<sub>3</sub>.26H<sub>2</sub>O) and thaumasite (Ca<sub>6</sub>. [AL(OH)<sub>6</sub>]<sub>2</sub>. (SO<sub>4</sub>)<sub>3</sub>.26H<sub>2</sub>O). There are several sources of sulfates in soils, produced from primary or secondary origin. Primary sources can be defined as the direct sources of sulfates in their natural form, as sulfate-bearing minerals such as gypsum while, the secondary sources are those that are not a direct source of sulfate but give out sulfates as a by-product of oxidation or other forms of chemical interactions.

### 2.8 Stabilization Using Calcium Carbide Residue (CCR) and Other Admixers

In addition to cement, lime  $(Ca(OH)_2$ -rich material) has been widely used to stabilize clayey soils. The dissociation of  $Ca(OH)_2$  leads to an increase in the pH values of the pore water. Strong bases dissolve the silica and alumina from the clay particles (a natural pozzolanic material) in a manner similar to the reaction between a weak acid and a strong base. The hydrous silica and alumina then gradually react with the calcium ions (pozzolanic reaction), which hardens with time.

Several researches have been conducted to evaluate the potential of calcium carbide residue for stabilization of deficient soils. The study by Du et al. (2016) was a good presentation of the potential of calcium carbide residue for use as material for subgrade stabilization of weak clay soil. Field California bearing ratio, plate load test, Benkelman beam deflection test and dynamic cone penetration (DCP) tests were used as evaluation criteria to evaluate the potential of calcium carbide residue for stabilization of weak subgrade clay soil. Results showed that treatment of soft subgrade with calcium carbide residue increased the value of California bearing ratio and resilient modulus and resulted in to a low construction cost. Binary blending of calcium carbide residue and palm oil fuel ash were used to stabilize fine grained soils. Atterberg limits and unconfined compression strength were used to evaluate the effect of calcium carbide residue and palm oil fuel ash on the fine grained soils. The work of Vichan et al. (2013) investigated the amount of calcium carbide residue and biomass ash (BA) required to cause Pozzolanic reaction. The author used SEM and XRD to elucidate the Pozzolanic reaction going on in the clay after addition of calcium carbide residue and BA. High increase in unconfined compression strength was recorded due to Pozzolanic reaction between the binders and the clay.

Clay of intermediate plasticity (CI) and clay of high plasticity (CH) were both treated with calcium carbide residue and Cocoanut shell ash (CSA) to improve their strength and stability (Balarabe & Mary, 2018). 4% calcium carbide residue and 6% calcium carbide residue were mixed with the clay of intermediate plasticity and clay of high plasticity respectively, which were in turn admixed with 4, 9, 14 and 19% cocoanut

shell ash each. The maximum dry density was observed to decrease with increase in cocoanut shell ash while the optimum moisture content decreased in the same order. The unconfined compression strength values of the clay of intermediate plasticity clay recorded 11.38 times the value of the untreated clay soil while the clay of high plasticity clay recorded 6.03 times the value of the untreated clay soil. The study by Akinwumi et al. (2019) focused on the potential use of calcium carbide residue to stabilize tropical sands for pavement structures. The researcher used Atterberg limits, calcium carbide residue and California bearing ratio as evaluation criteria to determine the effect of calcium carbide residue on tropical sandy soil. The unconfined compression strength values increased from 220kN/m<sup>2</sup> for untreated clay soil to 420kN/m<sup>2</sup> on addition of 16% calcium carbide residue while the California bearing ratio increased from 54% for untreated clay to 66% on addition of 16% calcium carbide residue. The study on the effect of cement and calcium carbide residue on the engineering properties of residual lateritic soil was carried out by Edeh and Joel (2014). 2 - 10% cement and 2 - 10%calcium carbide residue were each used to stabilize Ikpayongo laterite to determine the effect of cement and calcium carbide residue on the geotechnical properties of the lateritic soil. The author concluded that the calcium carbide residue and California bearing ratio values increased from 534kN/m<sup>2</sup> and 28% for untreated soil respectively to 3157kN/m<sup>2</sup> and 180% respectively for clay stabilized with combination of 10% cement and 10% calcium carbide residue. The use of two industrial wastes (ground granulated blast furnace slag and Calcium carbide residue), to stabilize soil was investigated. Addition of these two additives improved the permeability, unconfined compression strength and California bearing ratio of the soil thereby reducing the thickness of pavements.

To improve economic and environmental impacts, some waste Ca(OH)<sub>2</sub>-rich materials can be utilized together with waste pozzolanic materials, such as fly ash, biomass ash and rice husk ash to develop a cementitious material. Calcium carbide residue (CCR) is a by-product of the acetylene production process that contains mainly calcium hydroxide, Ca(OH)<sub>2</sub>. Between 1995 and 1998, the demand for calcium carbide for the production of acetylene gas in Thailand was 74,000 tone. This demand is continuously increasing each year. Due to its highly basic pH, calcium carbide residue has been little utilized and was typically gone to a disposal area in the form of slurry. After being sundried for a few days, the slurry form changes to a dry form.

Jaturapitakkul and Roongreung (2003), have introduced a cementitious material that is a mixture of calcium carbide residue and rice husk ash. The cementing property was identified as a pozzolanic reaction between the two materials, and no Portland cement was included in the mixture. Horpibulsuk *et al.* (2012) and Kampala and Horpibulsuk (2013) explained the possible mechanism controlling the engineering properties of Calcium carbide residue stabilized clay based on macro- and micro-scale observations. The optimum water content (OWC) of the stabilized clay exhibits the highest strength because it engenders the densest packing and highest cementitious products. Strength improvement for a particular curing time is classified into three zones: active, inert and deterioration (Figure. 2.8). The data were obtained from an unconfined compression test under unsoaked condition on Calcium carbide residue stabilized samples at optimum water content. All the input Ca(OH)<sub>2</sub> is consumed by the natural pozzolanic material in the soil to produce a pozzolanic reaction. This active zone can be determined from the calcium carbide residue fixed in point, which is obtained simply from the

index test. Calcium carbide residue fixation is defined as the calcium carbide residue content at which the plasticity index of the calcium carbide residue - clay mixture changes insignificantly with the calcium carbide residue input. Strength development in the inert zone tends to slow down as shown in Figure 2.8.



Figure 2.9: Improvement zones (Source: Horpibulsuk *et al.*, 2012)

Thus, the incremental gradient becomes nearly zero and does not make any further significant improvement. A decrease in strength, which appears when the calcium carbide residue content is in the deterioration zone, is caused by unsoundness due to free lime. This free lime [Ca(OH)<sub>2</sub>] is clearly observed by the thermal gravity analysis (TGA) (Horpibulsuk *et al.*, 2012). Even with the high unsoaked strength in the active zone (Figure 2.9), Kampala *et al.* (2013) found that the wet-dry cycled strength of stabilized clay was considered insufficient according to recommendations by the ACI (1990) and the U.S. Army Corps of Engineers (2004). The input of Fly Ash (as a calcium carbide residue replacement) may improve the strength of calcium carbide residue stabilized clay when the calcium carbide residue content is in excess of the active zone (that is in inert and deterioration zones) where natural pozzolanic material in the soil is not in sufficient quantities to react with the Ca(OH)<sub>2</sub>. However, the optimal

input of Fly Ash and the mechanism controlling strength development in these two zones are not clearly understood.

The tensile strength, fracture toughness, modulus and energy of cementitious materials are dependent on the hydration degree (Hoover, *et.al.*, 2015).

## 2.9 Stabilization Using Zeolite as Additives

Zeolites are three-dimensional, microporous, crystalline solids with well-defined structures that contain aluminium, silicon and oxygen in their regular framework; cations and water are located in the pores. The silicon and aluminium atoms are tetrahedrally coordinated with each other through shared oxygen atoms. Compositionally, zeolites are similar to clay minerals. More specifically, both are alumino-silicates. Similar to other pozzolanic materials, zeolite substitution can improve the strength of cement by pozzolanic reaction with Ca(OH)2, prevent undesirable expansion due to alkali aggregate reaction, reduce the porosity of the blended cement paste, and improve the interfacial microstructure properties between the blended cement paste due to this permeability get reduced by using zeolite and also maximum dry density and optimum moisture content get increased (Yadev and Umanshankar, 2018).

Zeolites have been known to possess valuable physiochemical properties such as adsorption, cation exchange, molecular sieving and catalysis (Golbald *et al.*, 2017). Millions of tons of zeolite tuff exist in Turkey, especially clinoptilolite tuffs, which are widely available.

Banana fibre and zeolite can enhance the stability and properties of a bitumen mix. A comparison of bitumen mix with and without using banana fibre and zeolite were made. The zeolite releases the water after heating which helps in lowering the mixing and

compacting temperature which helps in saving of fuel consumption which proves to be economical (Neetin and Avanni, 2019).

A very limited number of studies on the stabilization of problematic soils using zeolite have been conducted (Hossain and Isah, 2016). This research is an attempt to stabilize problematic soil samples (clay soil) using calcium carbide residue admixed with zeolite.

## **CHAPTER THREE**

## 3.0 MATERIALS AND METHODS

### 3.1 Materials

The materials used in carrying out this research work were clay soil, calcium carbide residue, and zeolite. Potable water was used in preparing the samples.

# 3.1.1 Clay soil

The disturbed sample of the clay soil used in this research work was collected behind the Library complex of Niger State Polytechnic, Zungeru of Niger State, Nigeria (Plate I). This area is situated within (latitude 9<sup>o</sup> 44<sup>1</sup> 46.987<sup>11</sup> N and longitude 6<sup>o</sup>8<sup>1</sup>6.099<sup>11</sup>E). The top soil in the area was removed to a depth of 1m before the soil samples were collected by disturbed sampling method. The clay soil was air-dried and pulverized as specified in BS 1377 (1992).



Plate I: Clay Soil obtained from Zungeru Polytechnic, Niger state

## 3.1.2 Calcium carbide residue

The calcium carbide residue used in this study was collected from local welders at Keteren-Gwari mechanic site, Minna, Nigeria. The sludge was then air-dried, crushed and sieved through British Standard sieve with 0.075mm aperture before use, as shown in plate II.



Plate II: Calcium Carbide Residue

## 3.1.3 Zeolite

The zeolite used in this study was purchased from a commercial market in Kaduna in Kaduna State of Nigeria. The zeolite was observed to be powdered smooth as shown in plate III, and was used as supplied.



Plate III: Zeolite

## 3.1.4 Water

The water used during the process of this research was obtained from the bore hole close to civil engineering laboratory, Federal University of Technology Minna, Niger state.

#### **3.2 Methods**

The methods used in this study involves characterizing the untreated clay soil by determining its index properties. The method of B.S 1377 (1992) was used. Unconfined compressive strength tests were conducted on the clay samples mixed with 0, 5 and 10% calcium carbide residue, which was admixed with 0, 2, 4 and 6% zeolite so as to determine the effect of zeolite on calcium carbide residue modified clay soil. The stabilized samples were cured and tested for 1, 7, 14, 28, 60 and 90 days to determine the effect of curing period on the unconfined compressive strength value of a calcium carbide residue modified clay soil. Meanwhile, for all the tests, appropriate amount of water was mixed with the clay, mixed with appropriate chemicals, before conducting tests. The microstructural test carried out include X-ray diffraction test (XRD) and scanning electron microscopy (S.E.M). They show the mineral composition and elemental composition of the clay respectively.

## **3.2.1 Natural moisture content**

The apparatus used includes: Moisture Cans, Electronics and manual weighing balance, Drying oven, Soil sample, Spatula. The procedure adopted involved weighing three empty cans to the nearest 0.01g (M<sub>1</sub>). About 30g of fresh soil sample was placed in each of the cans and weighed again to the nearest 0.01g (M<sub>2</sub>). The cans were placed in an oven at 100°C for 24 hours to dry. The cans containing the dried soil were weighed (M<sub>3</sub>). The natural moisture content was then determined from:

$$Mc = \frac{M_2 - M_3}{M_3 - M_1}$$
(3.1)

Where;

M<sub>1</sub>= Mass of empty container
M<sub>2</sub> = Mass of container with wet soil
M<sub>3</sub> = Mass of container with dry soil
Results are presented in Appendix A (Table A1).

#### **3.2.2 Particle size distribution (sieve analysis) test**

This test was carried out in accordance with BS 1377 (1990) using the method of wash sieving. 300g of the clay soil was weighed and placed in a dry clean plate. The dried soil sample was then covered with enough water and allowed to stay for 24 hours. This is to allow water to permeate the clay clods to soak in water. The soil was then washed through 75µm sieve aperture sieve until the water passing through it, became clear (almost the same colour with water). The soil retained on the sieve was transferred to a plate and was oven dried. Sieves were arranged according to their aperture sizes from 5 mm on top through 3.35 mm, 2.00 mm, 1.18 mm, 0.6 mm, 0.425 mm, 0.30 mm, 0.212 mm, 0.15 mm, and 0.075 mm to the pan on the bottom. The oven dried soil on the plate was weighed and poured into the sieve arranged. It was vibrated for about ten minutes. Meanwhile, the sieves were weighed empty before the sieving commenced. Each of the sieves were again weighed after sieving and the change in masses recorded as mass retained on each of the sieves. From the mass retained, cumulative mass retained, percent mass retained, and percent mass passing were all obtained. The coefficient of

curvature (Cc) along with uniformity coefficient (Cu) was determined by plotting the grain size curve for the aggregates.

Hence,

$$Cu = \frac{B_{60}}{B_{10}}$$
(3.2)

$$C_{\rm C} = \frac{(B_{30})^2}{(B_{10} \times B_{60})}$$
(3.3)

Results are presented in Appendix D (Table D1).

## **3.2.3** Atterberg limit test

## 3.2.3.1 Liquid limit (cone penetrometer method)

200g of a portion of soil sieved through BS sieve with 0.425 mm aperture was thoroughly mixed with distilled water to form a uniform paste. A portion of the paste was filled into a standard cup of Cone Penetrometer device. The cup filled with the paste was placed on a flat base of the penetrometer device and directly under a standard cone. A knob on the device was then pressed and the cone dropped into the cup filled with clay paste. The penetrometer reading was noted and recorded. A small portion of the soil paste was immediately collected from the cup for moisture content determination. The remaining portion of the clay paste was removed and placed on a flat surface where small water was added to the paste and the mixture repeated thoroughly. The whole process was repeated five times with addition of water at each stage. The penetration was finally plotted against moisture content. Moisture content at 20 penetrations is the Liquid Limit of the clay.

Results are presented in appendix B (Table B1).

## 3.2.3.2 Plastic limit test

20g of the pulverized clay soil passing sieve having aperture size  $425\mu$ m was taken from the sample. It was thoroughly mixed on the glass plate with sufficient distilled water to make it plastic enough to be shaped into a small ball. The ball was then rolled between the hand and the glass plate to form a thread of 3 mm. The process was continued until the thread showed sign of crumbling. Some of the crumbling materials were taken for moisture content determination. The whole process was repeated to obtain two values which were averaged to give the plastic limit.

## 3.2.3.3 Plasticity index

Plasticity index (PI) is the numerical difference between the liquid limit and the plastic limit of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties.

It is calculated as follows:

| PI = LL - PL | (3    | .4 | ) |
|--------------|-------|----|---|
|              | · · · |    | - |

Where

LL = liquid limit, and

PL = Plastic limit.

#### 3.2.4 Specific gravity

The determination of specific gravity was carried out according to BS 1377 (1990) test (B) for fine–grained soils. It is the ratio between the unit masses of soil particles and

water. Determination of the volume of a mass of dry clay was obtained by placing the clay in a glass density bottle filled completely with the desired distilled water. The density bottle and the stopper were weighed to the nearest 0.001g (m<sub>1</sub>). The air-dried clay was transferred into the density bottle; the content and the cover were weighed as m<sub>2</sub>. Water was then added just enough to cover the soil; the solution was gently stirred to remove any air bubble. The bottle was then filled up and covered. The covered bottle was then wiped dry and the whole weighed to the nearest 0.001g (as m<sub>3</sub>). The bottle was subsequently emptied and filled completely with water, wiped dry and weighed to the nearest 0.001g (m<sub>4</sub>). The specific gravity was calculated using the equation below:

$$G_{s} = \frac{m_{2} - m_{1}}{(m_{4} - m_{1}) - (m_{3} - m_{2})}$$
(3.5)

Where,

 $G_s$  = Specific gravity

 $m_1$  = Weight of density bottle (g)

 $m_2$  = Weight of density bottle plus dry soil (g)

 $m_3$  = Weight of bottle, soil and water (g)

 $m_4$  = weight of bottle and water only (g)

Results are presented in appendix C (Table C1).

#### 3.2.5 Microstructural component test

#### 3.2.5.1 X- ray diffraction test

The powdered sample was placed and clipped in to the sample holder. A Bruker AXS D8 X-ray diffractometer system coupled with Cu-K $\alpha$  radiation of 40 kV and a current of 40 mA. The  $\lambda$  for K $\alpha$  was 0.1541 nm, scanning rate of 1.5°/min, while a step width of 0.05° was used over the 2 $\theta$  range and the diffractograms were recorded in the 2 $\theta$  range

of  $20^{\circ}$  -  $90^{\circ}$  on the powder clay to identify and determine the mineralogical phase characteristics and the estimation of the average crystallite size of minerals in the clay soil. Other operating and instrumental conditions are shown in Appendix G (Table G1).

The Scherer equation shown below was used to determine the crystal size from half height peak width

$$d = \frac{k\lambda}{\beta\cos\theta}$$
(3.6)

Where d is crystallite size in nanometre, K=0.94,  $\lambda$  is the wavelength of the X-ray which is 0.1541 nm,  $\theta$  is the half-diffraction angle,  $\beta$  is the full width at half-maximum in radian.

#### 3.2.5.2 Scanning electron microscopy (SEM)

The sample was exposed to a highly focused beam of electrons from an electron gun. This beam of electron was focused to a small spot on the sample surface, using the SEM objective lens. variables like the accelerating voltage used, size of aperture employed, and the distance between the sample and the electron gun (working distance) where optimised to achieve the best quality images. The Scanning electron microscopy relied on the detection of high energy electrons emitted from the surface of the sample. The SEM images were transmitted to a connected monitor, this gave the advantage of being able to magnify the images to any required extent as shown in plate III.



Plate III: SEM test set up

## **3.2.6** Unconfined compressive strength test

The apparatus used include loading frame (consisting of two metal rings), proving ring of 2KN capacity and 1N accuracy, dial gauge (0.01mm accuracy), specimen trimming tools, frictionless end plates, sample extractors, split samplers, vernier calipers, stop watch, oven, weighing balances sensitive to 0.01g, split mould, 38mm diameter, 76mm long, and water content cans.

The specimen was prepared from a disturbed soil sample. The materials were wrapped in a thin rubber membrane and thoroughly worked with the finger to assure complete remoulding. Care was taken to avoid entrapped air, so as to obtain a uniform density. The compaction of the disturbed material was done using a mould of circular cross section with diameter 36mm. the specimen was later trimmed and removed from the mould.

The soil specimen was placed at the desired water content and density in the large mould. The sampling tube was pushed into the large mould and removed when fully

filled with the specimen. The split mould was coated lightly with a thin layer of grease and weighed, after which the soil was extracted out of the sampling tube into the split mould using the sample extractor and a knife. After trimming both ends of the specimen, the mould and the specimen was weighed and the specimen was then removed by splitting the mould into two parts, while the upper plate was adjusted to make contact with the specimen, the dial gauge was adjusted to zero and proving ring gauge was seated. Compression loads were then applied to produce axial strain, dial gauge readings were recorded, the compression load reading was taken at an interval of 0.5mm of the deformation dial reading. The test was continued until failure surfaces were fully developed. Finally, the sample from the failure zone of the specimen was taken and its water content determined.

From the data recorded, the unconfined compressive strength was calculated as shown below.

$$e = \frac{\Delta L}{Lo} \tag{3.7}$$

Where e = Axial strain

 $\Delta L$  = change within specimen length as read from the strain dial indicator.

Lo = the initial length of the specimen.

$$A = \frac{Ao}{(1-e)} \tag{3.8}$$

Where A = corrected arear of the specimen and

Ao = initial average cross sectional area of the specimen.

$$\sigma = \frac{P}{A}$$

# Where $\sigma$ = unconfined compressive strength and

P = the compressive force. (axial load)

A = average cross sectional area.



Plate V: Sample preparation

Results are presented in appendix E (Tables E1 - E9).

## **CHAPTER FOUR**

# 4.0 **RESULTS AND DISCUSSION**

# 4.1 Index Properties of clay sample

The Laboratory tests results of index properties of the clay are shown in Tables 4.1.

| Description                        | Result           |
|------------------------------------|------------------|
| Clay passing through Sieve 5.00mm  | 100%             |
| Clay passing through Sieve 2.00mm  | 96.30%<br>86.30% |
| Clay passing through Sieve 0.425mm |                  |
| Clay passing through Sieve 0.075mm |                  |
| Specific gravity                   | 67.83%           |
| LL (%)                             | 2.76             |
| PL (%)                             | 42.28            |
| PI (%)                             | 23.09            |
| AASHTO Classification              | 19 19            |
| USCS Classification                | A-7-5            |
|                                    | CI CI            |
|                                    | CL               |

 Table 4.1: Index properties of clay sample

This clay is classified as clay of low plasticity (CL) based on Unified Soil Classification System.

## 4.2 Mineralogical composition of clay

## 4.2.1 X-ray diffraction result of clay

The mineralogical composition of the natural clay is shown in figure 4.1.



Figure 4.1: X-ray diffraction test result on natural clay sample

From Figure 4.1 the major minerals contained in the clay soil, are Quartz, Albite and Kaolinite. This is in agreement with (Das, 1998). Kaolinite is a secondary clay mineral, which must have resulted into the high liquid limit recorded in the clay soil. On stabilizing the clay, there was an increase in the number of minerals which may be as a result of reactions between the clay particles and the stabilising agent in the presence of the additive as shown in Figure 4.2



Figure 4.2: X-ray diffraction test result on stabilized clay

#### 4.2.2 Result of scanning electron microscopy (SEM) at two stages

The result of the scanning electron microscopy shows the character of the soil microstructure. With the aid of a microscope, the advancement in microstructural development from the first stage (clay + 10% calcium carbide residue) to the second stage (clay + 10% calcium carbide residue + 2% zeolite) can be clearly seen. At the initial stage, there was agglomeration of particles with pore spaces and evidence of whitish substances suspected to be calcium silicate hydrate(C-S-H), on addition of 2% zeolite, the particulate structure become denser with disappearance of major pore spaces, higher calcium silicate hydrate must have been formed to fill the pore spaces, which resulted to the higher strength at that level. This is in conformity with (Nazile, 2021).



**Figure 4.3:** Clay + 10% Calcium Carbide Residue.



Figure 4.4: Clay + 10% Calcium Carbide Residue + 2% Zeolite

# 4.3 Unconfined Compressive Strength Result

Appendix E (Table E1) shows the unconfined compressive strength result of the modified and unmodified clay sample. The modified clay sample was cured for 1day, 7days, 14days, 28days, 60days and 90days.

## 4.3.1 Effect of Zeolite on Unconfined Compressive Strength Result

## of Calcium Carbide Residue stabilized clay

The effect of zeolite on Calcium Carbide Residue stabilized clay soil is shown in figures 4 and 5, for 1-day and 7- days curing. The clay mixed with zeolite alone resulted in to gradual increase in unconfined compression strength after 1-day curing from 54kN/m2 at 0% zeolite to 94kN/m2 at 6% zeolite. This represents 74% increase in strength. With addition of 5% Calcium Carbide Residue after 1-day curing, the unconfined compressive strength result increased from 378kN/m2 at 0% zeolite to maximum of 579kN/m2 at 2% zeolite after which the values reduced to 371kN/m2 at 6% zeolite. This also represents 53% increase in strength which must have been generated from the reaction between zeolite and Calcium Carbide Residue and clay soil. This is in conformity with Edeh and Joel, 2014.



UCS VERSUS ZEOLITE AT ONE DAY CURING

Figure 4.5: Variation of Unconfined Compressive Strength Result with zeolite for oneday curing



**→**0% CCR **→**5% CCR **→**10%

Figure 4.6: Variation of Unconfined Compressive Strength Result with zeolite for Seven days curing

## 4.3.2 Effect of curing period on unconfined compressive strength result

For sample containing 0% Calcium Carbide Residue, the unconfined compressive strength result value tends to increase from 146 kN/m<sup>2</sup> at the end of 7 days curing duration to 218 kN/m<sup>2</sup> at the end of 90 days curing duration. This represents 49.3% increase in unconfined compressive strength.

For sample containing 5% Calcium Carbide Residue, the unconfined compressive strength result value tends to increase from 606 kN/m<sup>2</sup> at the end of 7 days curing duration to 1019 kN/m<sup>2</sup> at the end of 90 days curing duration. This represents 68.2% increase in unconfined compressive strength as shown in Figure 4.7

For sample containing 10% Calcium Carbide Residue, the unconfined compressive strength result value tends to increase from 1043 kN/m<sup>2</sup> at the end of 7 days curing duration to 2085 kN/m<sup>2</sup> at the end of 90 days curing duration. This represents 99.9 % increase in unconfined compressive strength as shown in appendix E (Table E4).



UCS versus zeolite at 5% CCR

**Figure 4.7:** Variation of unconfined compressive strength result with curing period for 5% calcium carbide residue

#### **CHAPTER FIVE**

#### 5.0 CONCLUSION AND RECOMMENDATIONS

## 5.1 Conclusion

The following conclusions were made from the outcome of this work:

From the index properties result obtained, it was observed that the clay is classified as A-7-5 according to American Association of state highway and transportation official (AASHTO) and CL according to unified soil classification system (USCS).

From the microstructural analysis result, the X-ray diffraction result of the natural clay shows major minerals to include Quartz, Albite and Kaolinite, on addition of calcium carbide residue and zeolite to the clay, there was formation of new minerals including anorthite, muscovite and calcite. The scanning electron microscopy result of clay mixed with calcium carbide residue only shows agglomeration of particles with pore spaces and evidence of whitish substances suspected to be calcium silicate hydrate (C-S-H). addition of zeolite to the mixture, gave a more compact structure with discementitous calcium silicate hydrate (C-S-H) filling the pore spaces

It was observed that at a constant curing duration and constant amount of Calcium Carbide Residue, the compressive strength tends to increase with increase in percentage of zeolite to an optimum zeolite percentage, followed by a decrease in strength.

The unconfined compressive strength of the natural clay increased from a maximum of 54 kN/m<sup>2</sup> at 0% zeolite to 94 kN/m<sup>2</sup> at 6% zeolite. On addition of 5% CCR, it increased from 378 kN/m<sup>2</sup> at 0% zeolite to maximum of 579 kN/m<sup>2</sup> at 2%. The increase in

strength confirms the pozolanic reaction between the pozzolant (zeolite) and the calcium carbide residue.

# 5.2 **Recommendations**

The following recommendations from the study carried out are given:

- i. The use of calcium carbide residue admixed with zeolite is viable for clay stabilization as it leads to increase in compressive strength.
- ii. To achieve a high compressive strength, a calculated high percentage of calcium carbide residue and an optimum zeolite percentage should be applied.
- iii. More studies should be carried out on the use of calcium carbide residue admixed with zeolite for stabilization of clay.

# 5.3 Contribution to Knowledge

This research result shows a strength increase from 378  $kN/m^2$  to 579 $kN/m^2$  at 5% CCR, with an optimum zeolite of 2%. This represent 53.17% increase in strength as shown in Appendix E, and is priceless for developers interested in the research area.

#### REFERENCES

- Abdullah, N. & Abdullah, R. (2013). Effect of humic acid on microstructure of limetreated organic clay. *International Journal of Engineering Research and Technology*, 11(2): 1827–1833, ISSN: 2278-0181.
- Akinwumi, I. I., Oluseyi, O. A., Michael, C., Adebanji, S. & Ayodele, O. (2019). Application and strength development of subgrade materials stabilized with calcium carbide waste in flexible pavement construction. *Adeleke University Journal of Engineering and Technology*, 2, 55-65, doi:10.2495/UTI80261.
- Anagnostopoulos, C. A., Dimitrios, T. & Kiriakos. B. (2013). Shear strength behaviour of polypropylene fibre reinforced cohesive soils. *Geomechenical and Geoengineering*, 9, 241–251, doi:10.1080/17486025.2013.804213.
- Anggraini, V., Huat, B., Asadi, A. & Nahazanan, H. (2014). Effect of coir fibre and lime on geotechnical properties of marine clay soil. 7th International Congress on Environmental Geotechnics, 14, 1430-1437, ISBN: 9781922107237
- ACI Committee 318. (1990). Building code requirements for structural concrete: (ACI 318-95); and commentary (ACI 318R-95). *Farmington Hills, MI: American concrete institute*.
- Balarabe, I. B. W. & Mary, S. R. M. (2018). Soil stabilization using calcium carbide residue and coconut shell ash. *Journal of Basic and Applied Engineering Research*, 2, 1039-1044, <u>http://www.krishisanskriti.org/jbaer.html</u>.
- Bose, B. (2012). Geo engineering properties of expansive soil stabilized with fly ash. *Eectronic Journal of Geotechnical Engineering*, 17, 1339–1353.
- Broderick, G. P. & Daniel D. E. (1990). Stabilizing compacted clay against chemical attack. *Electronic Journal of Geotechnical Engineering*, 116, 1549–1567.
- BS 1377, (1992). British Standard Methods of test for soils for civil engineering purposes. British Standards Institution. London
- Chittoori, B. C. S. (2009). Clay mineralogy effects on long-term performance of chemically treated expansive clays. Doctoral dissertation, The University of Texas at Arlington. ProQuest publishing.
- Cristelo, N., Victor, M., Anthonio, T., Tiago, M. & Nuno, A. (2015). Influence of discrete fibre reinforcement on the uniaxial compression response and seismic wave velocity of a cement-stabilized sandy-clay. *Geotextand Geomembrane*, 43,1–13, doi:10.1016/j.geotexmem.2014.11.007

- Das, M. B. (1998). Principles of Geotechnical Engineering, (4th edition) PWS Publishing Division of International Thomas Publishing.
- Du, J. Y., Ning-jun, J., Suksan, H. & Arul, A. (2016). Field evaluation of subgrade soil stabilized with calcium carbide residue. *Soils and Foundations*, 56, 301-314, https://doi.org/10.1016/j.sandf.2016.02.012.
- Edeh, E. J. & Joel, M. (2014). Stabilization of ikpayongo latrite with cement and calcium carbide waste. *Global Journal of Pure and Applied Sciences*, 20, 49-55. doi:10.4314/gjpas.v20i1.8
- Estabragh, A. R., Ali, T. B. & Javadi, A. A. (2013). A study on the mechanical behavior of a fiber-clay composite with natural fiber. *International Journal of Geotechnical Engineering* 31, 501–510, doi:10.1007/s10706-012-9602-6.
- Firoozi, A. A., Raihan, M. & Mohd, R.T. (2014). Analysis of the load bearing capacity of two and three-layered soil. *Electronic Journal of Geotechnical Engineering*, 19, 4683–4692.
- Firoozi, A. A., Mohd, R. T. & Tanveer A. K. (2015). The influence of freeze-thaw cycles on unconfined compressive strength of clay soils treated with lime. *Jurnal Teknologi*, 76, 107–113, doi:10.11113/jt.v76.4127.
- Firoozi, A. A., Mohd, R. T., Ali, A. F. & Tanveer A. K. (2015). Effect of ultrasonic treatment on clay microfabric evaluation by atomic force microscopy. *Journal* of Measurement, 66, 244–252, doi:10.1016/j.measurement.2015.02.033.
- Gao, T., Lei, S., Ming, S. & Fengnan, C. (2015). Analysis on differences of carbon dioxide emission from cement production and their major determinants. *Journal* of Cleaner Production, 103, 160–170, doi:10.1016/j.jclepro.2014.11.026.
- Golbald, S., Koshnoud, P. & Abu-Zahra, N. (2017). Synthesis of 4A zeolite and characterization of calcium ion and silver ion exchange forms. *Journal of Minerals and Material Characterization and Engineeiring*. 5,237-251. https://doi.org/10.4236/jimmce. 2017.55020
- Habiba, A. (2017). Review on different types of soil stabilization. *International Journal* of *Transportation Engineering*, 2, 19-24, doi:10.11648/j.ijtet.20170302.12.
- Haraguchi, M., Miyadera, K., Uemura, K., Sumizawa, T., Furukawa, t., Yamada, k., Akiyama, S. & Yamada, Y. (1994). Angiogenic activity of enzymes. Nature 368:198, doi: 10.1038/368198a0.
- Hoover, C. G. & Ulm, F. J. (2015). Experimental chemo-mechanics of early-age fracture properties of cement paste. *Cemement and Concrete Reserch*, 75, 42–52, doi:10.1016/j.cemconres.2015.04.004.

- Horpibulsuk, S., Phetchuay, C. & Chinkulkijniwat, A. (2012). Soil Stabilization by Calcium Carbide Residue and Fly Ash. *Journal of Materials in Civil Engineering*. 24(2): 184–193, doi:10.1061/(ASCE)MT.1943-5533.0000370.
- Horpibulsuk, S., Phetchuay, C., Chinkulkijniwat, A. & Cholaphatsorn, A. (2013). Strength development in Silty Clay Stabilized with Calcium Carbide Residue and Fly Ash. *Journal of Soils and Foundations*. 53, 477–486, doi:10.1016/j.sand/j.sandf. 2013.06.001.
- Hussain, M. & Isah, S. (2016). Influence of zeolite and cement additions on mechanical behavior of sandy soil. *Journal of Rock Mechanics and Geotechnical Engineering*. 15, 54-60, doi:10.1016/j.jrmge.2016.01.008.
- Ismail, A., Baghini, M. S., Karim, M. R., Shokori, F., Al-Mansob, R. A. & Firoozi, A. A. (2014). Laboratory investigation on the strength characteristics of cementtreated base. *Transportation Technology Publications*, 507, 353–360, doi:10.4028/www.scientific.net/AMM.507.353.
- Jaturapitakkul, C. & Roongreung, B. (2003). Cementing Material from Calcium Carbide Residue-Rice Hush Ash. *Journal of Materials in Civil Engineering*, 15(5): 470– 475.
- Jayanthi, P. & Singh, D. N. (2016). Utilization of sustainable materials for soil stabilization. Advances in Civil Engineering Materials, 5, doi:10.1520/ACEM20150013.
- Jones, L. D. & Jefferson, I. (2012). Expansive soils. *ICE Manual of Geotechnical Engineering*, 1, 413 441, ISBN:978072775707
- Kampala A., Horpibulsuk S., Chinkullijniwat A. & Shen S. (2013). Engineering properties of recycled calcium carbide. *Construction and Building Materials*. 46, 203–210, doi:10.1016/j.conbuildmat.2013.04.037.
- Kassim, K. A. & Chern, K. K. (2004). Lime stabilized Malaysian cohesive soils. *Malaysian Journal of Civil Engineering*, 16, 13–23, doi:10.11113/mjce.v16.15658
- Kezdi, A. (1979). Stabilized earth roads development in geotechnical engineering. *Elsevier Science*, ISBN:9780444596406.
- Khan, T.A., Mohd, R. A., Firoozi A. A. & Firoozi, A. A. (2015). Strength tests of enzyme-treated illite and black soil mixtures. *Proceedings of the Institution of Civil Engineers- Engineering Sustainability*, 169, 214–222, doi:10.1680/jensu.15.00023.
- Khemissa, M. & Mahamedi A. (2014). Cement and lime mixture stabilization of an expansive over consolidated clay. *Applied Clay Science*, 95, 104–110, doi:10.1016/j.clay. 2014.03.017.

- Little, D. N. (1999). Evaluation of structural properties of lime stabilized soils and aggregates. *Summary of Findings Prepared for the National Lime* Association, 1, 1–89.
- Little, D. N. & Nair S. (2009). Recommended practice for stabilization of subgrade soils and base materials. *National Cooperative Highway Research Program*, 144, 11-24.
- Louafi, B., Billal, H. & Ramdane, B. (2015). Improvement of geotechnical characteristics of clay soil using lime. *Advanced Materials Research*, 1105, 315–319,doi:10.4028/www.scientific.net/amr.11053.315.
- Mehdi, M. (2017). Developing a practical technique for predicting the shear strength of fibre reinforced clay. *Geosynthetic International*, 25, 1-39, doi:10.1680/jgein.17.00033
- Mohammed, A. (2013). Improvement of swelling properties using hay fibres. *Construction and Building Materials*, 38, 242-247, doi:10.1016/j.conbuildmat.2012.08.031.
- Nima, L., Farshid, V., Ehsan, G. & Ahmed S. (2018). Sustainable use of calcium carbide residue in the stabilization of clay. *Journal of Materials in Civil Engineering*. 30 (6): 1943-1561, doi:10.1061/(ASCE)MT.1943-5533.0002313.
- Neetin, K. & Avanni, C. (2019). Effect of using banana fibres and zeolite as an additive on the engineering properties of bituminous concrete. *International Journal of Civil Engineering and Technology*. 10(4): 367-372, ISSN, 0976-6308.
- Nazile, U. (2018). The importance of clay in geotechnical engineering. *Book Citation Index*, 1, 84 - 96, doi:10.5772/intechopen.75817.
- Nazile, U. (2021). The significance of scanning electron microscopy (SEM) analysis on the microstructure analysis of improved clay: an overview. *Open Geosciences*. 13, 197-218, doi:10.1515/geo-2020-0145.
- Pandian, N. S. (2013). Fly ash characterization with reference to geotechnical applications. *Journal of the Indian Institute of Science*. 84, 189–216.
- Radhakrishnan, G., Kumar, M. A. & Raju, G. P. (2014). Swelling properties of expansive soils treated with chemicals and fly ash. *American Journal of Engineering Research* 3(4):245–250, ISSN:2320-0936.
- Rupnow, T. D., Mahdi, M., & Wu, Z. (2015). Class C fly ash stabilization of recycled asphalt pavement and soil. *Journal of the Transportation Research Board*, 1–19, doi:10.1177/0361198120961385.
- Sharma, V., Hemant, K. V. & Bhanu, M. M. (2015). Enhancing compressive strength of soil using natural fibers. *Construction and Building Materials*, 93, 943–949, doi:10.1016/j.conbuildmat.2015.05.065.

- Shahram, P, Bujang, H., Afshin, A. & Mohammad, H (2016). Model study of alkaliactivated waste binder for soil stabilization. *International Journal of Geosynthetics and Ground Engineering*. 2 (4):1-12, https://doi.org/ 10.1007/s40891-016-0075-1.
- Shukla, S. K., Sivakugan, N. & Singh A. K. (2010). Analytical model for fiberreinforced granular soils under high confining stresses. *Journal of Materials in Civil Engineering*, 22, 935–942, doi:10.1061/(ASCE)MT.1943-5533.0000081.
- Solanki, P. (2010). Resilient modulus of clay subgrades stabilized with lime, class C fly ash, and cement kiln dust for pavement design. *Journal of Transportation Research Board* 2186, 101 110, doi:10.3141/2186-11.
- U.S. Army Corps of Engineers, (2004). Soil Stabilization for Pavements. TM5-822 14/AFJMAN 32/1019 http://www.wbdg.org/ccb/DOD/UFC/ufc\_3\_25011.
- Vichan, S., Rachan, R. & Horpibulsuk S. (2013). Strength and Microstructure Development in Bangkok Clay Stabilized with Calcium Carbide Residue and Biomass Ash. *Science Asia.* 39, 186–193, doi:10.2306scienceasia1513-1874.2013.39.186.
- Yadev, R. K. & Umanshankar, K. (2018). Effect of zeolite and cement addition on geotechnical properties of granular soil. *International Research Journal of Engineering and Technology*. 5(5): 851-856, ISSN, 2395-0036.
- Yilmaz, Y., (2015). Compaction and strength characteristics of fly ash and fiber amended clayey soil. *Engineering Geology*, 188, 168–177, doi:10.1016/j.enggeo. 2015.01.018.
- Zulkifley, M. T. M., Tham, F., John, K. R., Roslan, H., Ahmed, F. A., Paramanthan, S. & Muhammad, A. (2014). A review of the stabilization of tropical lowland peats. *Buletin of Engineering Geology and Environment*. 73, 733–746, doi:10.1007/s10064-013-0549-5.
# **APPENDICES**

# Appendix A: Result for natural moisture content

 Table A1: Natural moisture content result.

| Can Number          |                | 1     | 2     | 3     |
|---------------------|----------------|-------|-------|-------|
|                     |                |       |       |       |
| Can Weight          |                | 37.65 | 37.41 | 36.25 |
| Weight of Can + Wet | Soil           | 95.33 | 89.64 | 84.86 |
| Weight of Can + Wet | Can + Drv Soil | 86.31 | 81.98 | 77.76 |
| Soil                |                |       |       |       |
| Weight of Can + Wet |                |       |       |       |
| Soil                | Moisture       | 9.02  | 7.66  | 7.1   |
| Weight of Can + Wet |                |       |       |       |
| Soil                | Dry Soil       | 48.66 | 44.57 | 41.51 |
| Moisture Content    |                | 18.54 | 17.19 | 17.1  |

Appendix B: Result for Atterberg limit test.

**Table B1:** Liquid limit and plastic limit tests for the natural soil

### FUT Minna

| LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD) |                   |                   |  |  |  |  |
|---|-------------------|-------------------|--|--|--|--|
| Project : Kaka work                                   |                   |                   |  |  |  |  |
| Test Location :                                       |                   |                   |  |  |  |  |
| Sample no. :  | Depth of sample : | Date : 13/10/2020 |  |  |  |  |
| Sample Description :                                  |                   |                   |  |  |  |  |

|                          |       | LIQUID LIMIT |       |             |              | PLASTIC LIMIT |       |
|--------------------------|-------|--------------|-------|-------------|--------------|---------------|-------|
| Can Number               | 1     | 2            | 3     | 4           | 5            | 1             | 2     |
| Penetration              | 6.0   | 10.5         | 14.5  | 19.5        | 22.5         |               |       |
| Can Weight               | 38.19 | 35.95        | 35.20 | 36.91       | 36.89        | 35.33         | 35.73 |
| Weight of Can + Wet Soil | 52.54 | 54.27        | 51.25 | 51.10       | 54.19        | 42.13         | 43.26 |
| Weight of Can + Dry Soil | 49.49 | 49.53        | 46.56 | 46.46       | 48.14        | 40.74         | 41.71 |
| Weight of Moisture       | 3.05  | 4.74         | 4.69  | 4.64        | 6.05         | 1.39          | 1.55  |
| Weight of Dry Soil       | 11.30 | 13.58        | 11.36 | 9.55        | 11.25        | 5.41          | 5.98  |
| Moisture Content         | 26.99 | 34.90        | 41.29 | 48.59       | 53.78        | 25.69         | 25.92 |
| Liquid Limit             | 49.7  | 6%           |       | Average Pla | stic Limit : | 25.8          | 1 %   |



Test executed by:

# Appendix C: Result for specific gravity test

| Trial  | 1      | 2      | 3      |
|--|--------|--------|--------|
| Mass of empty Cylinder (m1)g                       | 66.32  | 63.19  | 68.22  |
| Mass of cylinder + dry soil (m <sub>2</sub> )g     | 93.98  | 97.77  | 98.67  |
| Mass of cylinder + soil + water (m <sub>3</sub> )g | 183.23 | 184.93 | 186.54 |
| Mass of cylinder + water (m <sub>4</sub> )g        | 165.67 | 162.86 | 167.06 |
| $m_2 - m_1(g)$                                     | 27.66  | 34.58  | 30.45  |
| $m_4 - m_1(g)$                                     | 99.35  | 99.67  | 98.84  |
| $m_3 - m_2(g)$                                     | 89.25  | 87.16  | 87.78  |
| Gs   | 2.763  | 2.764  | 2.753  |
| Average <b>G</b> s                                 |        | 2.76   |        |

 Table C1: Specific gravity for natural soil

Appendix D: Result for grain size analysis and hydrometer test

 Table D1:
 grain size analysis and hydrometer test result.

| Project |             |         |        |        |              |            |         |     |         |        |      |
|---------|-------------|---------|--------|--------|--------------|------------|---------|-----|---------|--------|------|
|         | t: Kaka w   | ork     |        |        |              |            |         |     |         |        |      |
| fest Lo | ocation :   | Zunger  | ru cla | у      |              |            |         |     |         |        |      |
| Sample  | e no. :     |         |        |        | Initial Sam  | ple mass : | 300     | g   | Date :  | 17/10/ | 2020 |
| Sample  | e Descript  | ion :   |        |        |              |            |         |     |         |        |      |
|         | Sieve si    | ize (mr | n)     | Mass 1 | retained (g) | % Re       | etained | %   | Passing |        |      |
|         |             | 5.000   |        |        | 0            |            | 0.00    |     | 100.00  | )      |      |
|         |             | 3.350   |        |        | 1.3          |            | 0.43    |     | 99.57   |        |      |
|         |             | 2.000   |        |        | 9.8          |            | 3.27    |     | 96.30   | )      |      |
|         |             | 1.180   |        |        | 11.1         |            | 3.70    |     | 92.60   | )      |      |
|         |             | 0.850   |        |        | 6.1          |            | 2.03    |     | 90.57   | '      |      |
|         |             | 0.600   |        |        | 7.7          |            | 2.57    |     | 88.00   | )      |      |
|         |             | 0.425   |        |        | 5.1          |            | 1.70    |     | 86.30   | )      |      |
|         |             | 0.300   |        |        | 11.2         |            | 3.73    |     | 82.57   | '      |      |
|         |             | 0.150   |        |        | 26.3         |            | 8.77    |     | 73.80   | )      |      |
|         |             | 0.075   |        |        | 17.9         |            | 5.97    |     | 67.83   | ;      |      |
| ercent  | tage Passir | ng      |        |        |              |            |         |     |         |        |      |
| 100     |             |         |        |        |              |            |         |     | -       |        |      |
| 90      |             |         |        |        |              |            |         | • • | -       |        |      |
| 80      |             |         |        |        |              |            |         |     |         |        |      |
| 70      |             |         |        |        |              |            |         |     |         |        |      |
| 60      |             |         |        |        |              |            |         |     |         |        |      |
| 50      |             |         |        |        |              |            |         |     |         |        |      |
| 30      |             |         |        |        |              |            |         |     |         |        |      |
| 40      |             |         |        |        |              |            |         |     |         |        |      |
| 30      |             |         |        |        |              |            |         |     |         |        |      |
| 20      |             |         |        |        |              |            |         |     |         |        |      |
| 10      |             |         |        |        |              |            |         |     |         |        |      |
|         |             |         |        |        |              |            |         |     |         |        |      |
| 0       |             |         |        |        |              |            |         |     |         |        |      |

| GRAIN | SIZE | ANALS | SIS |
|-------|------|-------|-----|
| GNAIN | SILL | ADALI |     |

Appendix E: Result for unconfined compressive strength test.

| UCS        | 1DAY | 7DAYS | 14DAYS | 28DAYS | 60DAYS | 90DAYS |
|------------|------|-------|--------|--------|--------|--------|
| 0CCR/0Z    | 54   | 54    | 54     | 54     | 54     | 54     |
| 0CCR/2%Z   | 57   | 82    | 107    | 102    | 121    | 179    |
| 0CCR/4%Z   | 87   | 102   | 126    | 146    | 175    | 179    |
| 0CCR/6%Z   | 94   | 146   | 146    | 150    | 184    | 218    |
| 5%CCR/0Z   | 378  | 398   | 437    | 509    | 606    | 994    |
| 5%CCR/2%Z  | 579  | 728   | 1043   | 897    | 897    | 1261   |
| 5%CCR/4%Z  | 397  | 873   | 1043   | 912    | 728    | 1067   |
| 5%CCR/6%Z  | 371  | 606   | 703    | 485    | 631    | 1019   |
| 10%CCR/0Z  | 416  | 340   | 825    | 728    | 1698   | 2619   |
| 10%CCR/2%Z | 686  | 1455  | 1576   | 1382   | 1940   | 2813   |
| 10%CCR/4%Z | 554  | 1455  | 1552   | 1392   | 1867   | 2667   |
| 10%CCR/6Z  | 321  | 1043  | 1310   | 1164   | 1285   | 2085   |

**Table E1:** Unconfined compressive strength test result

**Table E2:**Unconfined compressive strength versus zeolite at 0% calcium carbideresidue.

| ZEOLITE % | 1DAY | 7DAYS | 14DAYS | 28DAYS | 60DAYS | 90DAYS |
|-----------|------|-------|--------|--------|--------|--------|
| 0         | 54   | 54    | 54     | 54     | 54     | 54     |
| 2         | 57   | 82    | 107    | 102    | 121    | 179    |
| 4         | 87   | 102   | 126    | 146    | 175    | 179    |
| 6         | 94   | 146   | 146    | 150    | 184    | 218    |



**Table E3:**Unconfined compressive strength versus zeolite at 10% calcium carbideresidue.

| ZEOLITE % | 1DAY | 7DAYS | 14DAYS | 28DAYS | 60DAYS | 90DAYS |
|-----------|------|-------|--------|--------|--------|--------|
| 0         | 416  | 340   | 825    | 728    | 1698   | 2619   |
| 2         | 686  | 1455  | 1576   | 1382   | 1940   | 2813   |
| 4         | 554  | 1455  | 1552   | 1392   | 1867   | 2667   |
| 6         | 321  | 1043  | 1310   | 1164   | 1285   | 2085   |



| ZEOLITE (%) | UCS (KN/M <sup>3</sup> ) |     |            |
|-------------|--------------------------|-----|------------|
| 0           | 54                       | 378 | 416        |
| 2           | 57                       | 579 | 686        |
| 4           | 87                       | 397 | 554        |
| 6           | 94                       | 371 | <u>321</u> |

**Table E4:**Unconfined compressive strength variation for one-day curing period.



| ZEOLITE (%) | UCS (KN/M²) |     |      |
|-------------|-------------|-----|------|
|             |             |     |      |
| 0           | 54          | 398 | 340  |
| 2           | 82          | 728 | 1455 |
|             |             |     |      |
| 4           | 102         | 873 | 1455 |
| 6           | 146         | 606 | 1043 |

**Table E5:**Unconfined compressive strength variation for seven-days curing period.



**→**0% CCR **→**5% CCR **→**10%

| ZEOLITE<br>(%) | UC  | S(KN/M <sup>2</sup> ) |      |
|----------------|-----|-----------------------|------|
| 0              | 54  | 437                   | 825  |
| 2              | 107 | 1043                  | 1576 |
| 4              | 126 | 1043                  | 1552 |
| 6              | 146 | 703                   | 1310 |

**Table E6:**Unconfined compressive strength variation for fourteen-days curingperiod.



**→**-0% CCR **→**-5% CCR **→**-10%

| ZEOLITE<br>(%) | UCS<br>(KN/M <sup>2</sup> ) |            |             |
|----------------|-----------------------------|------------|-------------|
| 0              | 54                          | 509        | 728         |
| 2              | 102                         | 897        | 1382        |
| 4              | 146                         | 912        | 1392        |
| 6              | 150                         | <u>485</u> | <u>1164</u> |

**Table E7:**Unconfined compressive strength variation for twenty-one days curingperiod.

UCS versus zeolite at twenty one days curing period



**→**0% CCR **→**5% CCR **→**10%

| ZEOLITE<br>(%) | UC<br>(KN | S<br>N/M <sup>2</sup> ) |      |  |
|----------------|-----------|-------------------------|------|--|
| 0              | 54        | 606                     | 1698 |  |
| 2              | 121       | 897                     | 1940 |  |
| 4              | 175       | 728                     | 1867 |  |
| 6              | 184       | 631                     | 1285 |  |

**Table E8:**Unconfined compressive strength variation for sixty days curing period.



**→**0% CCR **→**5% CCR **→**10%

| ZEOLITE<br>(%) | UCS<br>(KN/M <sup>2</sup> ) |             |      |
|----------------|-----------------------------|-------------|------|
| 0              | 54                          | 994         | 2619 |
| 2              | 179                         | 1261        | 2813 |
| 4              | 179                         | 1067        | 2667 |
| 6              | 218                         | <u>1019</u> | 2085 |

**Table E9:**Unconfined compressive strength variation for ninety days curing period.

UCS versus zeolite at sixty days curing period



**→**-0% CCR **→**-5% CCR **→**-10%

# Appendix F: Mineralogical test result.

| Condition  |  |
|------------|--|
| 0.5 s      |  |
| Cu-Ka      |  |
| 1.542 Å    |  |
| 40 Kv      |  |
| 40 Ma      |  |
| 20°<20<90° |  |
| 0.2°       |  |
| 60s/step   |  |
|            |  |

**TABLE F1:** Bruker D8 Advance X-ray diffraction operating parameters.