

**STABILIZATION OF CLAYEY SOIL USING CEMENT AND VOLCANIC ASH FOR
SUSTAINABLE ROAD CONSTRUCTION**

BY

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MEng/SEET/2017/7502**

**DEPARTMENT OF CIVIL ENGINEERING
FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA**

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ABSTRACT

A residual clay soil was collected at Bako village along Gwagwalada-Garki road in Federal Capital Territory of Nigeria using the method of disturbed sampling. The clay was characterized and microstructural tests including X-ray Diffraction (XRD) test, Scanning Electron Microscopy (SEM) test and Electron Dispersive Spectroscopy (EDS) tests were also conducted on the clay soil. The clay was then compacted at Standard Proctor energy level to obtain the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC). Unconfined Compressive Strength (UCS) tests were then conducted on the natural clay and clay mixed with 0, 2, 4 and 6% cement which are in turn admixed with 0, 3, 6, 9, 12 and 15% Volcanic Ash (VA) each. The clay also consists majorly of montmorillonite, ankerite, calcium silicide, anorthite, anothoclase and orthoclase minerals. The SEM test revealed flaky nature of the clay with pore spaces. The natural clay specimen compacted at standard proctor energy level mixed with varied composition of VA showed 2.5 times increase in UCS compared to the value of the natural clay. For clay made of 6% cement with varied percentage VA, the UCS increased by 80% compared with UCS of specimen containing 6% cement-0% VA. The optimal VA value for maximum UCS values lies between 6% and 9% VA.

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ABBREVIATIONS, GLOSSARIES AND SYMBOLS

BS	British Standard
VA	Volcanic Ash
CL	clayey soil
FUTMINNA	Federal University of Technology, Minna
Gs	specific gravity
LL	liquid Limit
M	mass
m	meter
MH	Silt of high plasticity
ML	Silt of low plasticity
NMC	natural moisture content
NP	Non-Plastic
OPC	ordinary Portland cement
PI	plasticity Index
PL	plastic Limit
SC	clayey sand
UCS	unconfined comprehensive strength
USCS	unified soil classification system
XRD	x-ray diffraction
XRF	x-ray fluorescence
SEM	scanning electron microscopy
EDS	electron dispersive spectroscopy

CHAPTER ONE

INTRODUCTION

1.0

1.1 Background of the Study

Clay soils constitutes major problems in geotechnical engineering compared to other forms of soils. Their particles are normally surrounded by negative charges resulting from the ions around the particles. Soft clay soil has the characteristics of high deformation on application of small loads while stiff unsaturated clay soils have the tendency of large swelling with ingress of water. Therefore, the application of clay soils as foundation soil for structures either suffers excessive deformation on application of load or swells excessively with ingress of water. In most occasions, road pavement structures have been founded on clay soils or clay soil have been used as pavement material due to non-availability of quality pavement materials. In any of these cases, the clay soil will require stabilization to improve its strength and durability.

A lot of stabilization methods ranging from mechanical, chemical and biological forms have been used to stabilize clay soils. Mechanical stabilization entails mixture of two or more soils of different gradation in order to achieve optimal mixture which when compacted will result in to most dense state. Some of the mechanical stabilizations includes the work of Ibrahim (1983) who stabilized black cotton soil with sharp sand found as deposit below the superficial black cotton soil deposit. The mixture was used for the construction of Maiduguri-Gamburu road in north-eastern Nigeria. Another mechanical stabilization is the stabilization of clay soils using non-plastic silt (Alhaji and Sadiku, 2015). The Unconfined Compressive Strength (UCS) was observed to increase from 272

and 770 kN/m² for British Standard Light (BSL) and British Standard Heavy (BSH) compaction energy levels at 0% replacement to 295 and 795 kN/m² for BSL and BSH compaction energy level respectively at 10% replacement after which the values reduced to 22 and 60 kN/m² for BSL and BSH compaction energy levels respectively at 70% replacement. Stabilization of clay soil using cold reclaimed asphalt pavement (RAP) is also a form of mechanical stabilization (Alhaji *et al.*, 2014). The result showed 7.2% increase in UCS and 9.2% increase in California Bearing Ratio (CBR). Mechanically stabilized clay soil using quarry dust was treated with cement kiln dust (Amadi and Osu, 2016). It was observed that curing time affects the strength of the specimen tremendously. The UCS increased from 1.25 to 5.25 times higher than the value of specimen tested immediately after preparation. Some other mechanical forms of stabilization are studies by (Ola, 1981; Louafi and Bahar 2012; Diouf *et al.*, 1990) on the mechanical stabilization involving clay soils mixed with sandy soil.

Chemical stabilization entails mixture of chemical substance like cement, lime, or mixture of both, to a deficient soil in order to improve its strength and durability. Stabilization of soils using cement or mixture of cement and pozzolana has become a common phenomenon and has yield positive result in many instances. Portland cement alone have been used to stabilize soil for road base (Ismail *et al.*, 2014). Tremendous improvement in strength and durability was recorded with the addition of Portland cement. Some other works include the stabilization of lateritic clay using cement and bagasse ash as pozzolana (Osinubi and Alhaji, 2009) variation of compressive strength of cement-treated marine clay with water content (Tsuchida and Tang, 2015), effect of sodium silicate and promoter on

the strength of cement stabilized clay (Ma *et al.*, 2015), cohesive soil stabilized with a mixture of cement and rice husk ash (Prasad *et al.*, 2017), improvement of weak residual soil using cement and rice husk ash (Basha *et al.*, 2005). All these studies recorded higher strength compared to the use of cement alone.

The common biological stabilization in use is the application of enzymes for improvement of soils. The work by (Khan and Taha, 2015; Sahoo and Sridevi, 2018; Muguda and Nagaraj, 2019; Rao and Hanuma, 2019; Somireddy, 2019; Renjith *et al.*, 2017) are indication of the potential of enzymes as soil stabilizers. Volcanic ash are products from volcanic eruptions and are deposited around the vents of the eruption epicenter. A lot of studies on the engineering use of volcanic ash have been carried out in Indonesia and some other countries in Africa. However, little or no study have been done on the volcanic ash found around the recent volcanic eruptions in Jos Plateau area of Nigeria. This research is intended to evaluate the effect of volcanic ash in Jos Plateau on the engineering properties of cement stabilized clay soil.

1.2 Statement of Research Problem

Clay exists in many regions of the world and has constituted one of the most troublesome soils for civil engineering construction. Most times, they exist on a large span of a region such that road construction must cut across the deposit. Where alternative suitable soil is not available, the clay must be treated to meet the specification for use in constructions. Improvement of these clay soils can be by modification or stabilization or combination of the two. Stabilization of clay soils are usually carried out using cement which is presently

uneconomical and its production is environmentally unfriendly. Production of cement have been understood to generate enormous carbon dioxide which pollutes the atmosphere. Any alternative that can substitute cement partially or holistically will assist to reduce air pollution and reduce the cost of construction.

Volcanic ash is a product of eruption of molten magma which was observed to constitute large deposit in Jos, Plateau State. This ash has been shown to be Pozzolanic in nature and can react with calcium hydroxide (byproduct of cement hydration) to give enhanced strength gain. However, strength gain is a function of both the cement and volcanic ash. It is therefore pertinent to evaluate the optimal mixture of volcanic ash that give the highest strength and stability.

1.3 Aim and Objectives of Study

The aim of this study is to determine the effect of volcanic ash on cement stabilized with clay soil.

In order to achieve this aim, the following objectives will be considered:

- i Determine the index properties and compaction characteristics of the natural clay soil
- ii Determine the microstructural characteristics of the clay and clay mixed with selected composition of the stabilizing chemicals
- iii Determine the unconfined compressive strength of untreated clay specimen and clay mixed with 0, 2, 4 and 6% cement admixed with 0, 3, 6, 9, 12 and 15% volcanic ash each.
- Iv Determine the effect of volcanic ash on cement stabilized clay

1.4 Justification of Study

Due to scarcity of befitting soil material for road construction in some areas of the world, it is very necessary to stabilize available but deficient soils so as to meet the required specification for use in road construction. The common chemical used for soil stabilization over the years is cement. However, cement is fast becoming expensive and environmentally unfriendly due to emission of carbon dioxide in to atmosphere. The use of cheaply and environmentally friendly, available volcanic ash to substitute cement will go a long way to reduce the use of cement and increase the strength and durability of clay soil.

1.5 Scope of Study

Clay soil were collected at depth of between 0.7m to 1.5m using the method of disturbed sampling. The clay was air-dried and prepared according to the method highlighted in BS 1377 (1992) before use. Index properties test including mechanical sieve analysis, hydrometer analysis, liquid limit test, plastic limit test, plasticity test, specific gravity test, compaction tests were conducted on the natural clay soil. X-ray fluorescence test was conducted on the clay, cement and volcanic ash to determine their oxide composition. X-ray diffraction test and scanning electron microscopy tests were carried out on the natural clay, clay mixed with 6% cement, clay mixed with 6% volcanic ash and clay mixed with 6% cement and 6% volcanic ash. The specimen molded from the natural clay and clay mixed with 0, 2, 4 and 6% cement which in turn was admixed with 0, 3, 6, 9, 12 and 15% volcanic ash each, were prepared and cured for 1, 7, 14, 28, 30 and 60 days before unconfined compressive strength tests.

CHAPTER TWO

2.0

LITERATURE REVIEW

2.1 Definition of Clay

Clay is a finely-grained natural rock or soil material that combines one or more clay minerals with possible traces of quartz (SiO_2), metal oxides (Al_2O_3 , 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 as shown in figure 2.1.



Figure 2.1: Clay soil baking hard during hot dry weather

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.

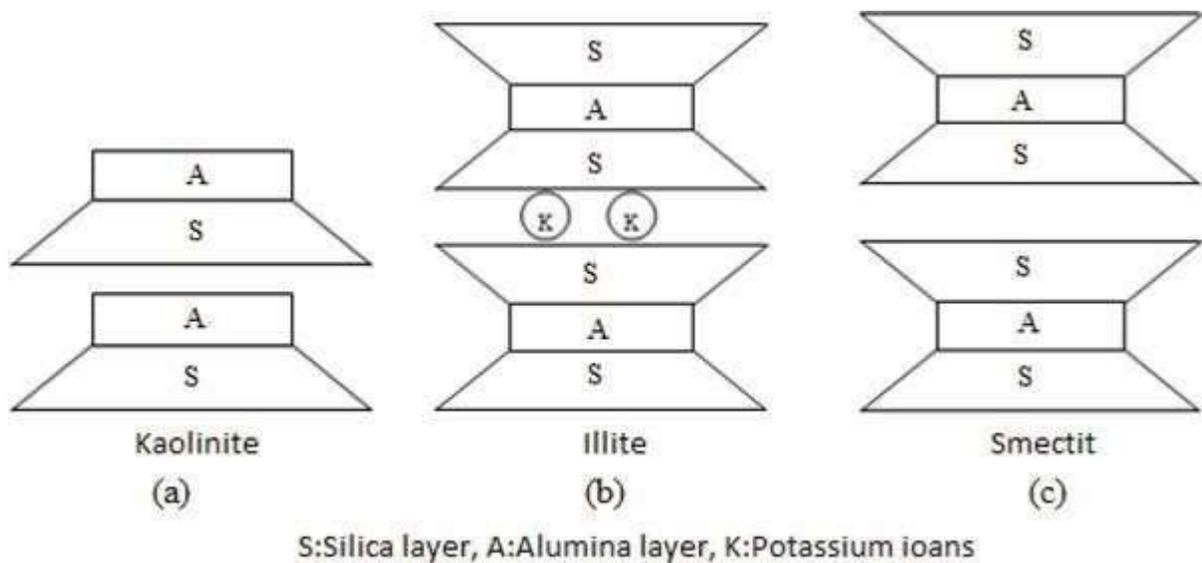


Figure 2.2: Display of structure of common clay minerals (Soil Mechanics and Foundation Engineering, by p.purushotama, Raj.2008, first edition.)

2.1.1 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.

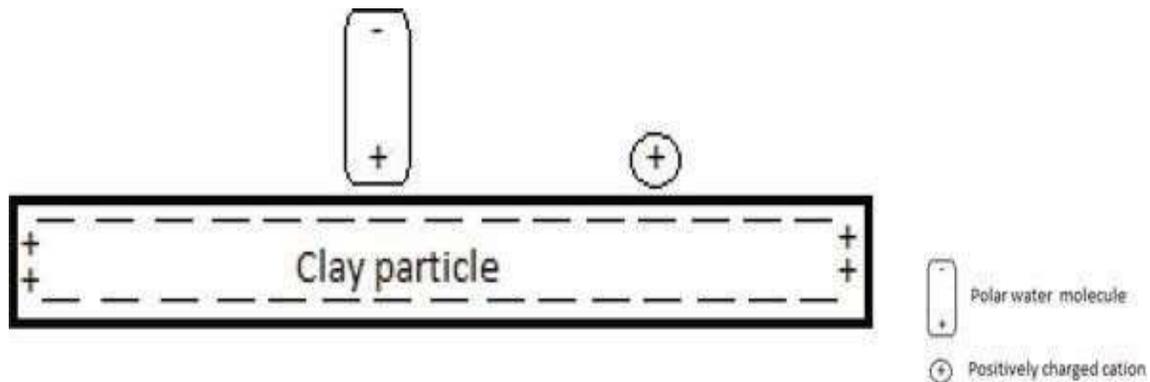


Figure 2.3: Clay particle and surface charge display (Soil Mechanics and Foundation Engineering, by p.purushotama, Raj.2008, frist edition.)

2.1.2 Structure of clay and physio-chemical properties

Around the clay that is faced with a liquid, there are distance-varying push-pull curves. If there is a force lifting the two clay minerals, the particles clump together. This is called flocculation. If the net force is thrust, the particles are separated from each other; this is called dispersion. Particle orientations of soils vary between flocculated and dispersed, as shown in figure 2.4. 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.



Figure 2.4: Clay particle orientations (Advanced soil mechanics by Braja M. Das. 3rd edition, 2008)

Flocculation increases depending on the electrolyte concentration, ion valence, temperature, decreasing dielectric constant, hydrated ion diameter, pH value and surface absorbed ions. Soil engineering properties depend on the size, shape, a high amount of surface area and a negative surface charge of clay particles. In 1925, Terzaghi proposed the concept of clay arrangement. He said that clay minerals stick to each other at the points of contact, with forces sufficiently strong to construct a honeycomb structure. In 1932, it was shown by Casagrande that this honeycomb shape is a special structure in clay-containing soils and this structure can vary depending on many characteristics of the environment. Figure 2.5 shows further compression with progress of soil sedimentation. Later, other researchers also proposed fabric models.

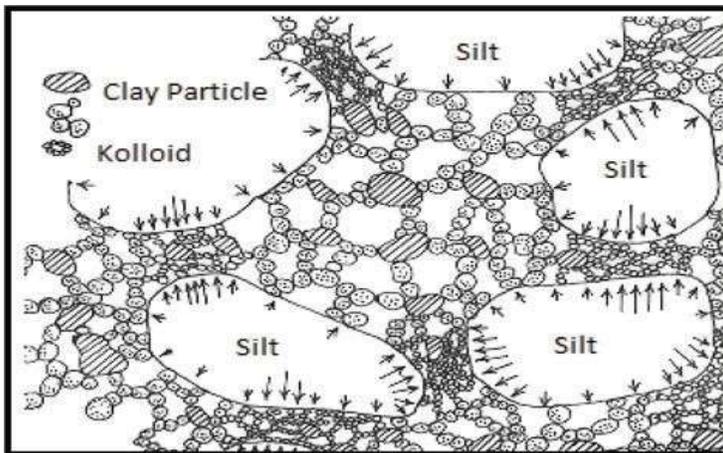


Figure 2.5: Casagrande's fabric model (Clay Microstructure, 1986. Richard H. Bennett and Mathew H. Hulbert)

Collins and McGown (1974) defined the elementary particle arrangement, particle assemblages and pore spaces in the fabric model. Researchers provided an elementary particle arrangement, a single clay, silt or sand, 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. Bennet and Hulbert suggested that the fabric of soils is mostly determined by the physical arrangement of particles, which is gained at the time of sediment deposition by the physical–chemical conditions of the depositional environment. The fabrics of soils describe clusters, clusters are formed by other clusters and space between the clusters and structure of the soils describes the fabric, mineral content and decontamination forces. Also, the fabrics of soils can sometimes be visualized under a microscope. The structure of soils can be examined more extensively by an X-ray diffractometer (XRD) and a Scanning Electron Microscope (SEM).

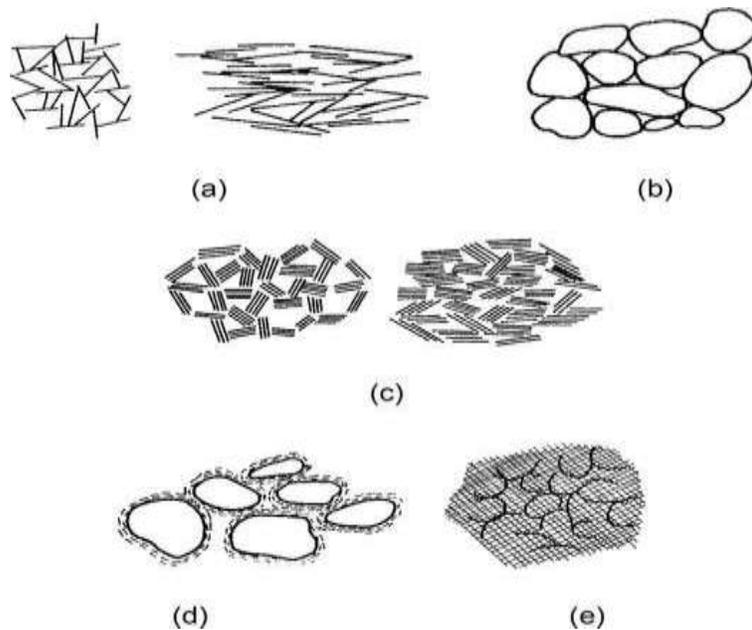


Figure 2.6: Arrangement of clay particles. (a) Elementary particle arrangements of clay; (b) elementary particle arrangements of sand and silt; (c) clay assemblages; (d) clay coated

silt and sand arrangement; (e) not fully defined arrangement. (Soil Mechanics and Foundation Engineering, by p.purushotama, Raj.2008, frist edition.)

2.2 Role of Clay in Geotechnical Engineering

Studies on soil behaviour 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.

2.3 Physical and Mechanical Behaviour 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).

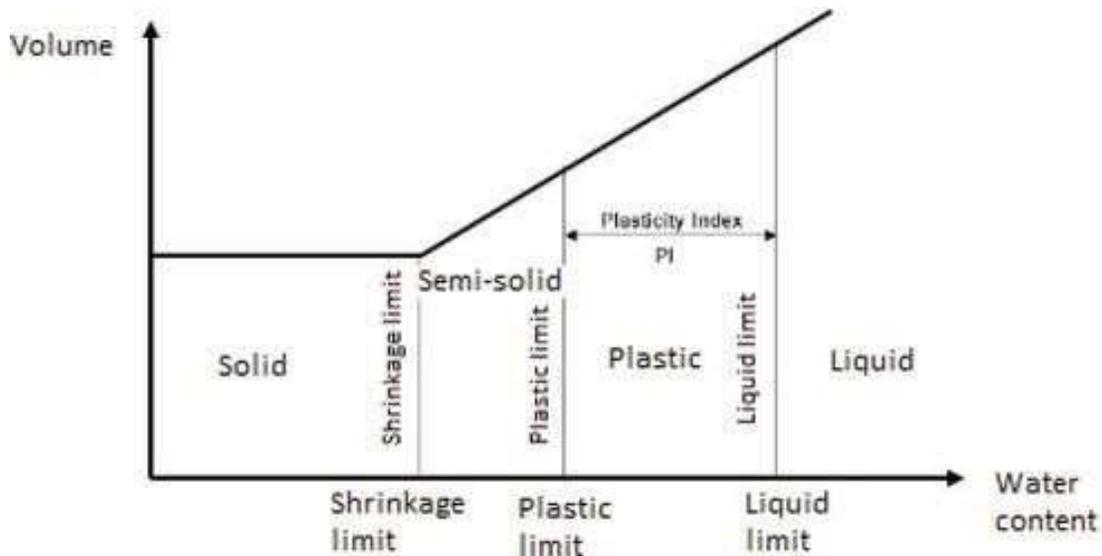


Figure 2.7: Water content-volume relationships of soils (Elements of Soil Mechanics, 7th Edition 1988.)

2.3.1 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. 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. 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. The hydraulic conductivity value of soils determines the constant head test (for coarse soils) and the falling head test (for fine-grained soils) as shown in table 2.1.

Table 2.1: Hydraulic conductivity of soils

SOIL TYPE	K(cm/s)
Clean gravel	100-1.0
Coarse sand	1.0-0.01
Fine sand	0.01-0.001
Silty Clay	0.001-0.00001
Clay	<0.000001

(Source: principles of foundation engineering. 6th edition. New York, USA Barja Das, 2017)

2.3.2 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 Holtz 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. Krohn and Slosson estimated that swelling soils cause approximately \$7 billion in damage each year. According to Holts and Hart, 60% of 250,000 newly constructed homes incur minor expansive soil damages and 10% incur significant expansive soil damage each year in the United States. Coduto noted that expansive soils caused \$490,000 in damage to a building over a 6-year period. The estimated annual cost due to significant structural damages, such as cracked driveways, sidewalks and basement floors, heaving of roads and highway structures, condemnation of buildings; and disruption to pipelines and other utilities in Colorado, is \$16 billion, according to AMEC.

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.

2.4 Stabilization

The more involving definition of stabilization is the one given by Singh (1991) who defined stabilization as the combination of soils and or other additives in such a way that, when it is compacted under specified condition and to specified extent, would undergo material change in its properties and would remain in its stable compacted state without undergoing any change under the exposure to weather and traffic. The main properties in engineering practice, which may require improvement are strength, permeability, volume stability and durability (Toro, 1997). There are wide range of engineering services where stabilization can be applied. These are:

- a Foundations: To reduce settlement or heave under buildings either by ensuring volume stability, or controlling permeability or increasing strength (Toro, 1997).
- b Excavation works: To provide support in pits, trenches, and tunnels by strengthening the surrounding soil or varying its permeability. The methods used are accelerated drainage and grouting.
- c Pavement construction: It provides pavement of higher strength and durability for highways, airfields and railways. The methods used are basically mechanical or chemical stabilization.
- d Slope Stability: This is to prevent slips on cut slopes, embankments and natural slopes. The method mainly applied are drainage and surface seals.
- e Environmental Conservation: To prevent erosion, combat dusting of road surfaces. This can be achieved by increasing the resistance of soil to natural weathering from wind or water.

f Thickness reduction: to reduce the thickness of a pavement below what it would have been without stabilization. Since pavements are usually designed using CBR curves, stabilizing the soil to higher strength and stability can reduce the thickness of pavement.

During the last four to five decades, numerous stabilizing processes or methods have been used. These include mechanical stabilization and chemical additives such as Portland cement, hydrated lime, gypsum, alkalis, sodium chloride, calcium chloride (Webb, 1992). However, because of cheapness and or availability in most developing countries, the most widely used chemical additives are Portland cement, lime, bitumen and agricultural waste to a less extent.

2.4.1 Mechanical Stabilization

Mechanical stabilization is the oldest of all the processes of soil stabilization (Gidigas, 1976). Ola (1983) have defined mechanical stabilization as the mixture of appropriately proportioned soil aggregates with some binder soils like clay, after which the mixture is properly compacted to a stable layer. The basic principles involved in mechanical stabilization are proportioning and compaction (Singh, 1991). It is a known fact that aggregate soil mixture having non or little amount of fines can only be stable under confined condition due to lack of cohesion between the aggregates. This type of material will have high permeability. Mechanical stabilization of these materials with addition of some binder soils like clay will improve the strength and stability of the mixture even under unconfined circumstances. Also, where a soil deposit is predominantly fine material, the compacted aggregate grains will not be in contact with each other and the stability of the soil is reduced.

This can also be corrected by mechanically adding appropriate proportion of coarse aggregate to the soil to improve its strength and stability

2.4.2 Bituminous stabilization

Bitumen has been categorized under water proofing agents by Yoder and Witczak (1975) and is one of the main properties it imparts on soils when used for stabilization. Generally, there are two mechanisms by which bitumen stabilizes soils (Ola, 1975): (Gidigas, 1976): (Osinubi, 2001a). The first process is the cementation or adhesive processes, which increases the strength of coarse grained soils but may decrease the strength of fine grained soils. The basic mechanism in asphalt stabilization of fine grained soils is a water-proofing phenomenon. Soil particles of agglomerates are coated with bitumen that prevents or slows the penetration of water, which could have resulted in decrease in soil strength. In noncohesive materials, such as sands and gravels, both the mechanism of water-proofing and adhesion occurs. The strength increase due to adhesion continues to the optimum bitumen content after which the strength will drop due to decrease in grain to grain contact and hence reduced mobilization of shear strength.

2.4.3 Lime stabilization

Lime stabilization is generally restricted to the warm to moderate climates, since lime stabilized soils are susceptible to breaking under freezing and thawing (Gidigas, 1976).

The action of lime on soil can be reduced to three basic reactions:

- a. Alteration of water film surrounding the clay minerals
- b. Coagulation or flocculation of the soil particles.

- c. Reaction of lime with soil components to form new chemicals.

According to Ola (1975), there is no direct hydration to form cementitious compounds in lime stabilization, rather, there are the physical and chemical components to the reaction of lime with clay. The physical reaction is one of cation absorption where calcium first replaces any other ion present as a base exchange ion. This process is followed by flocculation in to coarse particles, which produce an immediate increase in strength. This is in agreement with Osinubi (1999) who referred to this reaction as a short-term reaction. The chemical component of the reaction is the soil-cement hydration.

Addition of lime to a soil causes an immediate increase in the pH of the molding water due to the partial dissociation of the calcium hydroxide. The calcium ion in turn combines with the reactive silica or aluminates or both present at soil surfaces to form insoluble calcium silicates or aluminates or both which hardens on curing to stabilize the soil.

2.4.4 Cement stabilization

This has been observed to be the most effective of all the stabilizers for the greatest range of lateritic soils. According to Ola (1975), Gidigas (1976), Singh (1991), Osinubi (2001a), the mechanism of cement stabilization revolves around the hydration products of cement, which are independent of the soil. These products are calcium silicate hydrates, calcium aluminate hydrates and hydrated lime. The first two products constitute the major cementitious components, whereas the lime is deposited as a separate crystalline solid phase. The increase in strength is due to the development of cementitious linkages between these hydration products and soil particles. The released lime could react with any Pozzolanic material like clay present in the soil to form a secondary cementitious material,

cement takes place due to decrease in plasticity and formation of matrix enclosing clay clods in cement (Singh, 1991).

2.4.5 Stabilization of soils using volcanic ash

Clay soil have been stabilized with varied composition of cement, lime, volcanic ash and a combination of these chemicals (Hossain *et al*, 2006). Compaction, unconfined compressive strength, split tensile strength, modulus of elasticity and California bearing ratio tests were used as evaluation criteria to determine the effect of these chemicals on the strength and durability of the clay soil. It was concluded that the stabilized clay using these chemicals can be employed in various constructions including road pavements and low-cost housings. Hossain and Moi (2011) worked on the effect of cement kiln dust and volcanic ash on the strength and durability of clay soils. Compaction, unconfined compressive strength, split tensile strength, modulus of elasticity and CBR tests were used to determine the effect of cement kiln dust and volcanic ash on the strength and durability of clay soils. It was concluded that stabilization of clay using these chemicals can result to sustainability in construction industries.

A deposit of volcanic ash was characterized, classified and utilized as a soil stabilization material in Indonesia (Rifa'i *et al*, 2013). It was observed that addition of volcanic ash and curing of the specimen for 14 days increased the engineering properties of the soft clay, decrease liquid limit and increased the bearing capacity of the clay. 35% volcanic ash and 5% lime was observed to be the optimum mixture required for maximum strength. A volcanic ash that resulted from volcanic eruption on Mountain Merapi, admixed with lime

was used to treat clay soil (Rifa'i and Yasufuku, 2014). The study revealed that volcanic ash and lime decreased the fine fraction of the clay, decreased the consistency limit to non-plastic soil, decrease the swelling potential and increased the bearing capacity of the clay. 35% volcanic ash and 9% lime was observed to be the optimum mixture for maximum strength. Volcanic ash obtained from Kelud admixed with lime was used to treat clay soil (Latif *et al*, 2016). Addition of lime and volcanic ash resulted to decrease in consistency limits and increase the grain size of the clay particles. 20, 22, 25, 27 and 30% volcanic ash admixed with 5% lime each was observed to be the optimum mixture for maximum geotechnical improvement. Latif *et al* (2016) worked on the chemical characterization of some volcanic ash deposits in Indonesia considering the morphology and mineralogy. X-ray diffraction (XRD) and Scanning Electron Microscopy (SEM) tests were used as evaluation criteria to determine the mineralogy and morphology of some selected volcanic ash deposits. The study showed that Merapi volcanic ash is a form of fibrous glassy particles with elongated particles, Sinabung volcanic ash shows berry-like glass particles with angular blocky forms while Kelud volcanic ash texture is sponge-like glass.

A clay soil which classified as clay of low plasticity (CL) according to unified soil classification (USCS) system and A-7-6 according to American Association of State Highway and Transportation Officers (AASHTO) classification system was treated with a mixture of 2% gypsum and varied composition of volcanic ash (Roesyanto *et al*, 2017). CBR and UCS tests were used as evaluation criteria to determine effect of volcanic ash on gypsum treated clay soil. The UCS increased from 1.4 kg/cm² for clay soil without additive to 2.79 kg/cm² for clay treated with 2% gypsum and 10% volcanic ash. The soaked CBR

however, increased from 4.44% for clay without additive to 9.07% for the clay treated with 2% gypsum and 9% volcanic ash. Quick lime and volcanic ash have been used to stabilize clay soil with CBR as evaluation criteria (Hastuty *et al*, 2017). The result showed that the liquid limit reduced from 29.88 to 11.33% on application of 4% quick lime and 24% volcanic ash. The maximum CBR of 9.01% was observed at 4% quick lime and 8% volcanic ash.

A clay soil which classified under clay of low plasticity (CL) according to USCS was treated with 4% volcanic ash and varied composition of sugarcane bagasse ash (Hastuty *et al*, 2017). The UCS increased from 1.38 to 5.1 kg/cm² at optimal mixture of 4% volcanic ash and 10% sugarcane bagasse ash. The maximum CBR value of 13.91% was recorded at optimal mixture of 4% volcanic ash and 4% sugarcane bagasse ash. Cheng and Huang (2019) worked on the stabilization of black cotton soil with lime (0-9%), volcanic ash (0-25%) and their combinations. It was reported that black cotton soil (BCS) stabilized with a combination of lime and volcanic ash yields higher CBR and UCS when compared with single stabilizer. BCS stabilized with 3% lime and 15% volcanic ash meet the requirement for use as base course material for roads.

2.5 Review of Similar Works

Botao and Amy, (2013) studied effect of fly ash on the behaviour of expansive soils: using microscopic analysis, and deduced that, X-ray diffraction (XRD) analyses showed that there were negligible changes in mineralogy caused by CFA stabilization; however, there were reductions in the total amount of clay minerals. Significant changes in microstructure

were revealed, however, as qualified from scanning electron microscopy (SEM) images. The existence of iron-oxide coating was verified by both XRD and energy-dispersive X-ray spectroscopy (EDXS) analyses. The coating resulted in chemically stable aggregates that resisted dispersion with hexameter phosphate. The combined effects of flocculation and coating reduced the water retention property of the stabilized soils, decreased their swell potential, and increased the soil strength.

Rifa'i and Yasufuku, (2014) studied effect of volcanic ash utilization as substitution material for soil stabilization in view point of geo-environment and observed that Utilization of Volcanic Ash has grain size passing sieve number 270 is more effective. The effect of Volcanic Ash content on soil stabilization can improve the engineering properties of soft soil, change grain size distribution curve by decreasing of fine fraction, decrease consistency limits become non-plastic soil, increase bearing capacity, and decreases swelling potential. Addition of 35% of Volcanic Ash and 9% lime gives the most significant effect on soil improvement.

Matthew and Morgan, (2018) reported, the use of Mt. Mazama volcanic ash as natural pozzolans for sustainable soil and unpaved road improvement observed that, Chemical analyses show that Mt. Mazama volcanic ash has similar chemistry to that found in many Pozzolanic materials. Also, Sustainability studies show that any replacement of Portland cement with volcanic ash reduces carbon dioxide emissions and embodied energy.

Saeid, (2013) studied soil stabilisation technique using lime and fly ash observed that, after a series of experimental and micro-analytical tests investigated the effectiveness of lime

and fly ash on material composites from a mechanical, structural/microstructural, morphological-mineral and chemical-material perspective. The laboratory results were confirmed, from a microstructural analysis standpoint, by SEM. Elemental analysis and chemical characterisation were both confirmed by EDS; mineralogical phases were determined by XRD, and changes in chemical composition by FTIR. It was posited that additives could improve the compressibility, hydraulic conductivity, strength, and compaction characteristics of pure compounds, due to key factors such as Pozzolanic reactions, and the incrementing of the main chemical components associated with the generation of the polymerisation phenomenon, thereby creating the more uniform area noted in the composite's atoms.

Yongzhen and Xiaoming, (2018) studied effect of mineral additives on the behaviour of an expansive soil for use in highway sub-grade soils observed that, the results revealed that Black Cotton Soil (BCS) stabilized with combinations of lime and Volcanic Ash (VA) shows larger California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) values when compared with a single stabilizer. Also, the increase of pH and Electrical Conductivity (EC) in the stabilized soil promotes chemical reactions between the stabilizers and BCS to form new cementing agents, which are confirmed by X-ray Diffraction (XRD) and Transmission Electron Microscope (TEM) findings. The replacement of BCS with 3% lime and 15% Volcanic Ash stabilized BCS shows an obvious effect on controlling the moisture change and soil movement in the foundation BCS. This research provides a low-cost strategy for making use of the vast resources of BCS in Kenya obtained from foundation excavation.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Materials

The materials used for this research includes clay soil, cement, volcanic ash and distilled water.

3.2 Clay

The clay was collected from Bako village, a site proposed for Theophilus Yakubu Danjuma University along Gwagwalada-Garki road. The clay soil was collected at depth of between 0.5m – 1.5m using the method of disturbed sampling. The clay soil was then air-dried and pulverized according to the method highlighted in part 1 of BS1377 (1992).

3.3 Cement

The cement used was a Portland cement purchased locally in a commercial market and kept in a dried place to avoid moisturization of the cement.

3.4 Volcanic Ash

The volcanic ash used was collected from Keran Swap hill in Mangu Local Government of Plateau State, Nigeria. This area experience one of the most recent volcanic activities in Nigeria and revealed the ash to have solidified in to rocky forms. The solidified volcanic ash was then repulverised and sieved through BS sieve 0.075mm before used for stabilization.

The distilled water was purchased from a medical shop opposite General Hospital, Minna, Niger State, Nigeria. These materials are shown in Figure 3.1

3.1.1 Method of experimentation



Figure 3.1: (a) Clay soil; (b) Unpulverized volcanic ash; (c) Pulverized volcanic ash

XRay fluorescence (XRF) tests were also carried out on the clay soil. The tests were conducted in Ithemba Laboratory, Somerset West, South Africa and Electron Microscope Unit, University of Western Cape, Cape Town, South Africa. Phase characterization of the minerals and estimate of the average crystallite size of the various synthesized materials were conducted on a Bruker AXS D8 XRD system.

Scanning Electron Microscopy (SEM) test was also carried out by placing 0.05 mg of the synthesized materials, sprinkled on a sample holder, covered with carbon adhesive tape and wire sputter coated with Au-Pd using Quorum T150T for 5 minutes prior to analysis. The sputter coated samples were characterized using Zeiss Auriga HRSEM. The SEM, which visualizes morphology and microstructure of the synthesized products were analyzed using Zeiss Auriga HRSEM. This was carried out to determine the structure of the mineral particles contained in the clay. The clay was then mixed with 0, 2, 4 and 6% cement which in turn is mixed with 0, 3, 6, 9, 12 and 15% volcanic ash each as shown in table 3.1. The idea behind 15% maximum volcanic ash is derived from the works of whose optimal

percentage of volcanic ash used for soil stabilization ranges from 4% to 9%. The clay soil and clay mixed with varied proportions of cement and volcanic ash were compacted at standard proctor compaction energy level to obtain the maximum dry density (MDD) and Optimum Moisture Content (OMC) of the mixtures. The predetermined MDD and OMC were then used to prepare specimen for UCS test.

Table 3.1: Mixtures of cement and volcanic ash

Cement	VA
0.0	6.0
	9.0
	12.0
	15.0
	0.0
	3.0
	6.0
2.0	6.0
	9.0
	12.0
	15.0
	0.0
	3.0
	6.0
4.0	6.0
	9.0
	12.0
	15.0
	3.0
6.0	12.0
	15.0

The UCS specimen of 76mm diameter and 76mm height was molded at standard proctor compaction energy level. The specimen was wax-cured for 7, 14, 28, 60 and 90 days before

UCS test as shown in Plate I. The test was carried out in Civil Engineering Department, Federal University of Technology, Minna at speed of 1.2mm/min.



Plate I: Unconfined compressive strength test

3.1.2 Classification tests

3.1.3 Determination of natural moisture content

The procedure is in accordance with BS 1377 (1990). Small quantity of soil was kept in an air tight polythene leather as soon as the soil was collected from the borrow pit which was immediately transferred to the laboratory. Three empty cans were cleaned, and weighed. About 30g of the soil was crumbled and placed in each container. The containers and the content were weighed to the nearest 0.01g and placed in an oven at a temperature of 100⁰C plus or minus 5⁰C for a period of 24 hours. After drying, the containers were removed from

the oven and allowed to cool. The container plus the content were then weighed. The natural moisture content is given by

$$w = \frac{W_w - W_d}{W_d - W_c} \times 100\% \quad (3.1)$$

Where

W_c = weight of container

W_w = weight of container + wet soil

W_d = weight of container + dry soil

w = moisture content

3.1.4 Particle size distribution test

This test was also carried out according to procedure highlighted in BS1377 (1990) using the method of wash sieving. 200g of the soil sample was weighed and placed in a cleaned dried container and enough water was poured to cover the soil. The mixture was allowed overnight to allow water to permeate the clods of the fine soil. The soil was then washed through an arrangement of two sieves (2mm aperture size up and 75 μ m BS aperture size down) until the water passing the 75 μ m BS sieve became clear. Meanwhile, all the water passing through the 75 μ m BS sieve was collected in a container and used for hydrometer test. The soil retained in both the 2 mm sieve size and 75 μ m BS sieves were dried in an oven and dry sieving carried out on them.

3.2 Dry Sieve Analysis

Sieves were arranged according to their aperture sizes from 2mm from top through 1.18mm, 0.6mm, 0.425mm, 0.30mm, 0.212mm, 0.15mm, 0.075mm to the pan on the bottom. The whole soil left after washing was weighed and poured into the top sieve after

drying and sieved through the arrangement of the sieves for ten minutes. Meanwhile the sieves were weighed empty before the sieving commenced. Each of the sieves was 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, percentage mass retained and percentage mass passing were all deduced.

3.2.1 Hydrometer analysis

All the soil that was washed through 75 μ m sieve was collected and poured in to a 1000ml graduated cylinder and the mixture made up to 1000ml mark. The cork of the cylinder was inserted and the solution shaken thoroughly for about 5 minutes after which it was placed on top of a flat table. A hydrometer was immediately inserted and hydrometer readings recorded after 15 seconds, 30 seconds, 1 minute, 2 minutes, 4 minutes, 10 minutes, 20 minutes, 30 minutes, 1 hour, 2 hours, 4 hours, 6 hours, 12 hours and 24 hours. The temperature of the mixture was continuously taken during the process using thermometer. In this way, the particle size distribution of silt and clay size particles was determined.

3.2.5.3 Atterberg limits

The Atterberg limit tests were conducted in accordance with BS 1377 (1990).

3.2.3.3 Liquid limit

200g of a portion of the soil sieved through BS sieve with 425 μ m aperture was thoroughly mixed with distilled water to form a uniform paste. A portion of the paste was placed in the cup of Casagrande equipment. The paste in the cup was divided along the cup diameter through the center by means of a grooving tool. The crank was turned at the rate of two revolutions per second and count of the blows required to close the groove in the sample.

About 10g of the soil sample was taken from the closed groove for moisture content determination. A little amount of water was added to the soil and mixed thoroughly and the same operation was repeated until two consecutive runs gave the same number of blows.

3.2.3.4 Plastic limit

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

3.2.4 Compaction test

Empty mold with base plate was weighed. 3.0kg of air dried soil passing BS sieve aperture of 2.0mm was also weighed and placed on a flat tray about 1.0m X 1.0m. About 7% of water by weight of the dried soil was added immediately and mixed thoroughly until the whole mix formed a uniform paste. The mixture was then compacted in the mold with extension collar attached, by ramming it in to three layers, each layer been given 25 blows with 2.5kg rammer dropped from height of 30cm above the soil surface. The blows been uniformly distributed. After compaction of the third layer, the collar was removed and the soil trimmed off even with the top of the mold. The mold and the soil were then weighed. Two representative samples for moisture content determination were obtained from top and bottom of the compacted soil.

The compacted soil was crumbled, remixed with the remaining soil and 3.5% of water by weight of the dried soil was added to the mixture and the process repeated. The compaction soil was repeated each time raising the water content until there was drop in weight of mold plus weight soil. The result was presented in terms of dry densities using the expression:

$$\rho_d = \frac{100\rho}{100 \times \omega} \quad (3.2)$$

Where

= dry density of the mix (Mg/m³) ρ

Bulk density of the mix (Mg/m³) ω

Moisture content of the mix (%)

3.2.5 Unconfined compressive strength

The mould used for this test is a cylindrical mold of 38mm diameter and 76mm length. Predetermined Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) were used to mold soils and soils mixed with varied proportions of cement and volcanic ash, after which the specimen were cured in a airtight polythene leather for 1, 7, 14, 28, 60 and 90 days before testing to avoid moisture escaping from the specimen.

After curing, the samples were crushed using unconfined compressive strength machine at the rate of 1723kN/m² per minute until the failure load for each specimen was recorded.

The compressive stress for each sample was calculated as:

$$\text{Compressive stress} = \frac{\text{Failure load (kN)}}{\text{Cross sectional area (m}^2\text{)}} \quad (3.3)$$

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

The results of the experiments are hereby presented and discussed.

4.1 Index Properties Tests

The result of the index properties of the clay soil is shown on Table 4.1 while the grain size analysis graph of the clay is presented in Figure 4.1. From Table 2, the clay classified as A-76 according to AASHTO soil classification system and clay of high plasticity (CH) according to Unified Soil Classification System. This is an indication that the clay cannot be used as material for road pavement structure and cannot also be used to support pavement structures. It therefore requires stabilization to improve its strength and durability.

Table 4.1: Summary of Index Properties of the clay

Description	Quantity
Sand	18.4
Silt	28.9
Clay	53.0
Liquid limit	64.3
Plasticity Index	35.9
Specific Gravity	2.66
MDD (Standard Proctor compaction)	1.634
OMC (Standard Proctor compaction)	24.5
AASHTO soil classification	A-7-6
Unified Soil classification	CH

Figure 4.1 is the graph of grain size analysis for the natural clay soil. The graph revealed very high composition (52%) of clay sized particles with gap-gradedness between 63% to

82% at grain size of 0.06 to 0.07mm. This can create pores within the clay mass which can further reduce the initial strength of the molded clay. The MDD and OMC of the clay soil, compacted at Standard Proctor energy level was observed to be 1.634 g/cm³ and 18.5% respectively.

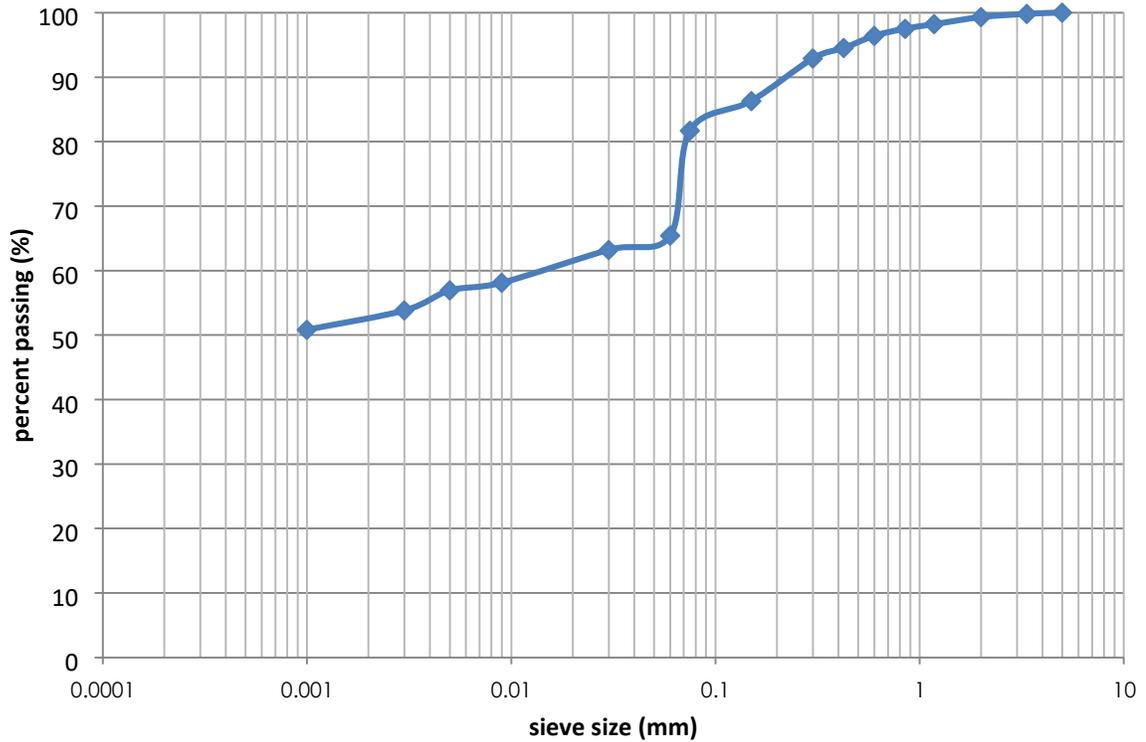


Figure 4.1: Grain size analysis of clay

4.2 Microstructural Analysis of the Clay and Clay mixed with Selected compositions of Cement and Volcanic Ash

The result of the microstructural analysis of the clay soil are deduced from XRF, XRD, SEM and EDS results as shown in Figures 4.2 to 4.5.

4.2.1 Oxide composition of the clay and chemical constituents

The oxide composition of the untreated clay and the chemical constituents used in stabilization of the clay is shown in Table 4.2.

Table 4.2: Oxide composition of clay, cement, and volcanic ash.

Oxide (%)	Clay	Cement	Volcanic Ash
Fe ₂ O ₃	9.61	4.57	13.34
TiO ₂	1.66	0.44	2.67
CaO	2.06	64.44	9.53
Si ₂ O ₂	1.35	0.56	0.72
KO	56.64	21.60	43.50
Al ₂ O ₃	19.07	4.13	14.31
MgO	1.38	1.06	11.30
Na ₂ O	0.64	0.11	1.57
ILO	0.714	0.76	3.42

From the table, the natural clay consists majorly of iron oxide, silica and alumina, the cement contained majorly of calcium oxide, silica, alumina and iron oxide. The volcanic ash is more of a pozzolana containing substantial amount of iron oxide, calcium oxide, silica, alumina and magnesium oxides. The oxide composition of the two chemicals contains the requisite oxides that generate ions resulting to cementitious complexes like calcium silicate hydrate and calcium aluminate hydrates. Addition of these chemicals to untreated clay will result in to cementation of the clay particles resulting in to enhanced strength and stability.

4.2.2 Mineralogical composition of clay and clay mixed with selected combination of cement and Volcanic Ash

The mineralogical composition was obtained from X-ray diffraction test whose results are shown in Figures 4.2 – 4.5. The XRD result of the natural clay revealed minerals including albite, anorthoclase, orthoclase, ankerite, quartz, calcium silicide and montmorillonite. The more active montmorillonite will cause excessive plasticity in the clay. The presence of a peak at 8.96⁰ also confirms the presence of montmorillonite mineral (Sharma *et al*, 2012).

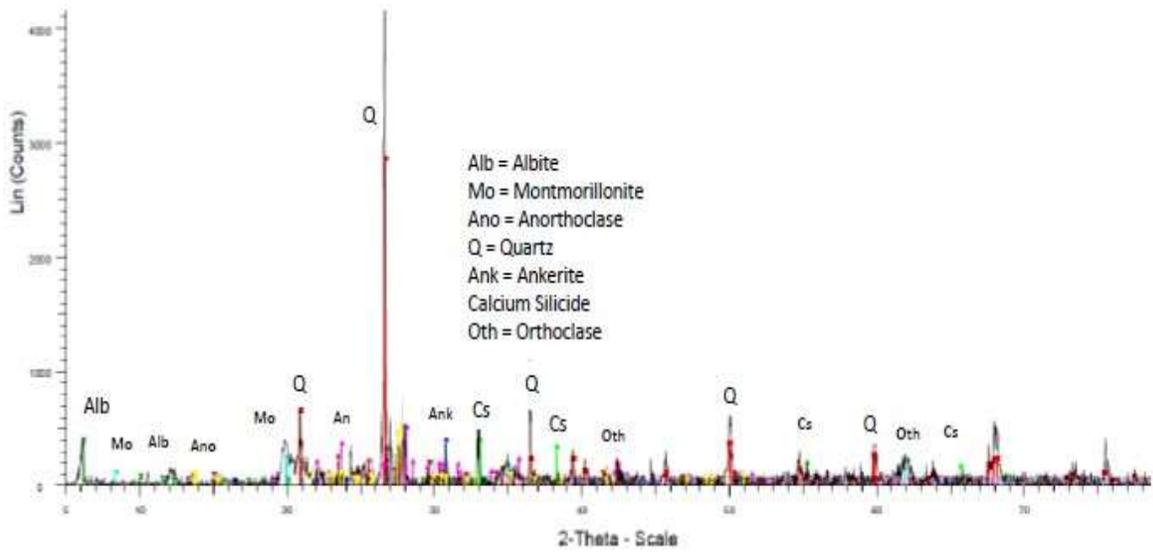


Figure 4.2: XRD Result for Untreated Clay

Addition of 6% cement to the clay results in to the graph shown in Figure 4.2. The graph showed that

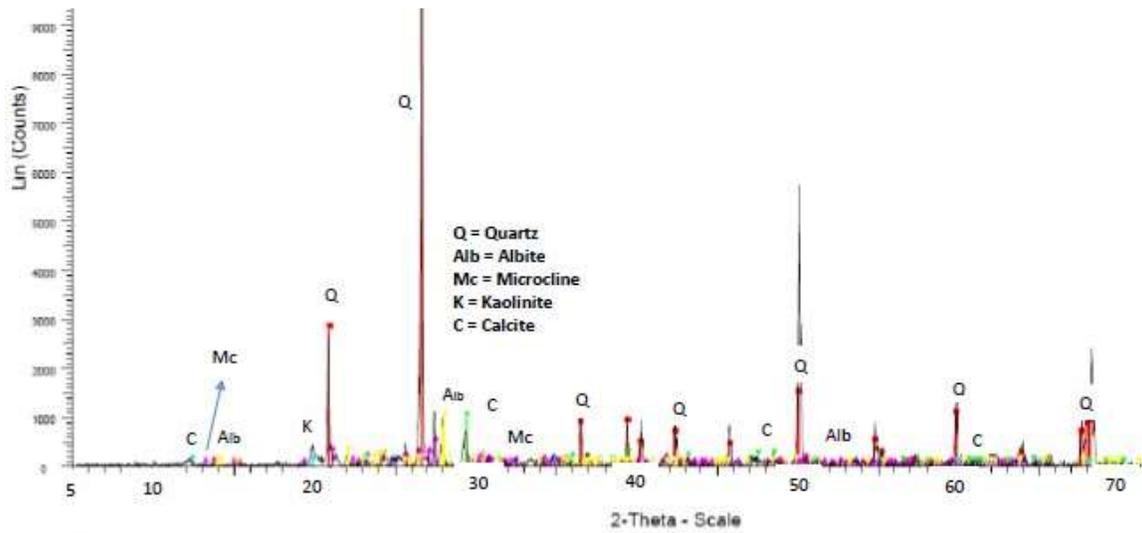


Figure 4.3: XRD Result for Clay Treated with 6% Cement

the montmorillonite peak has depleted while calcite peaks were introduced. Other minerals like orthoclase, anorthoclase and ankerite were completely weathered to form new minerals

kaolinite and microcline. This reaction will reduce the activity of montmorillonite in the clay soil thus increasing the particle sizes of the clay minerals.

Addition of volcanic ash to clay on the other hand gives the result in Figure 4.4. The result showed similar reaction like cement by depleting the montmorillonite peaks and introducing kaolinite and microcline peaks. Calcite peak however, was not introduced in the result. This is an indication that volcanic ash alone has cementitious characteristic and can contribute to strength gain in weak clays.

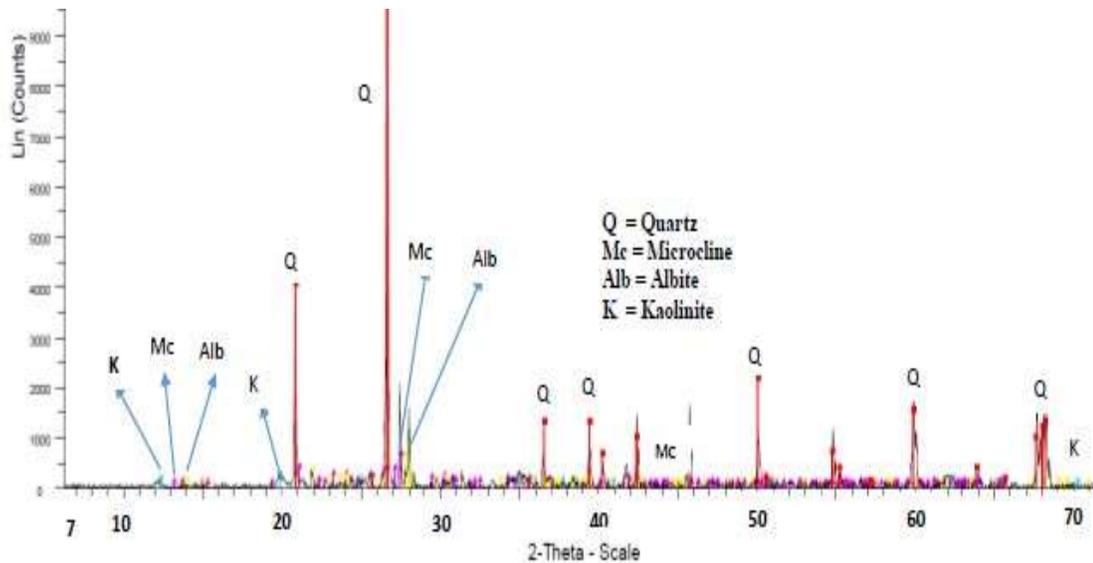


Figure 4.4: XRD Result for Clay treated with 15% Cement

Addition of 6% cement and 15% volcanic ash give the result shown in Figure 4.5. The graph also revealed that montmorillonite peak was also depleted and new peaks including calcite, kaolinite, halloysite, microcline and muscovite were introduced.

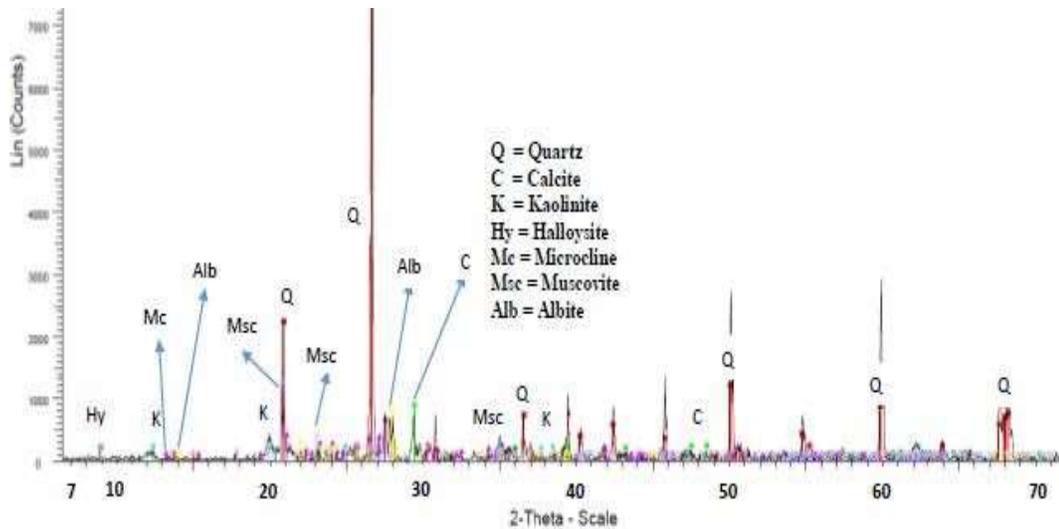
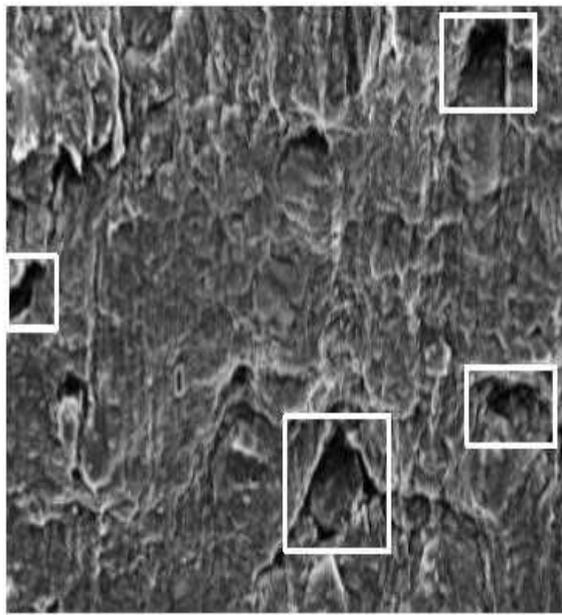


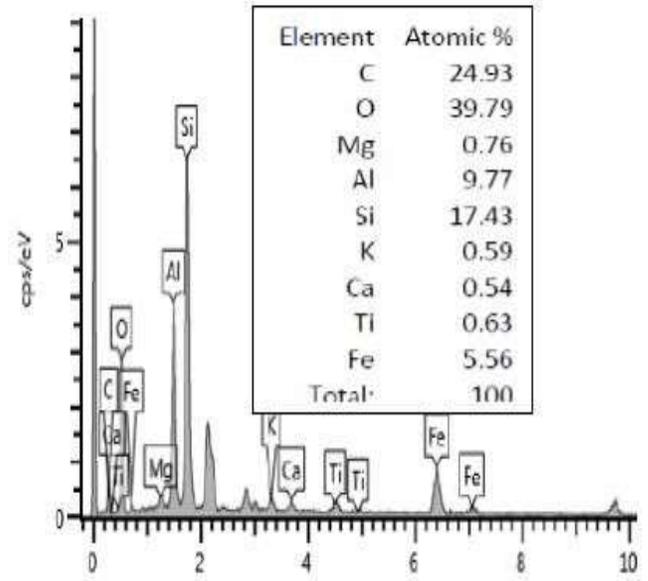
Figure 4.5: XRD Result for Clay Treated with 6% Cement and 15% Volcanic Ash

4.2.3 Morphological Structure of the Clay and Clay Mixed with Selected Composition of Cement and Volcanic Ash.

The morphological structure of the compacted specimen of untreated clay is shown on Figure 4.6. The SEM component of the figure showed flaky particles with large pore spaces. This could be attributed to the gap gradedness observed in the grading of the clay. The SEM image of the soil, remolded at standard Proctor compaction energy level revealed occasional presence of air voids (as highlighted in the figure) and dense fabric of flecky clay particles similar to those reported by Zang *et al.* (2013), Jaiswal and Lal (2016) and Abdullah *et al.* (2017). The EDS component of the figure also showed 24.9% carbon, 39.8%



(a) SEM



(b) EDS

Figure 4.6: SEM and EDS result of the Untreated Clay soil

Oxygen, 0.76% magnesium, 9.8% Aluminium, 17.4% silicon, 0.59% potassium, 0.54% calcium, 0.36% titanium and 5.6% iron. The silicon-aluminium ratio of 1.78 confirms the presence of montmorillonite mineral (Peethamparan and Olek, 2008).

Addition of 6% cement to the clay result in to the structure shown in figure 4.7. The SEM picture revealed a compact structure with little or no pore spaces. The cementitious calcium silicate hydrate and

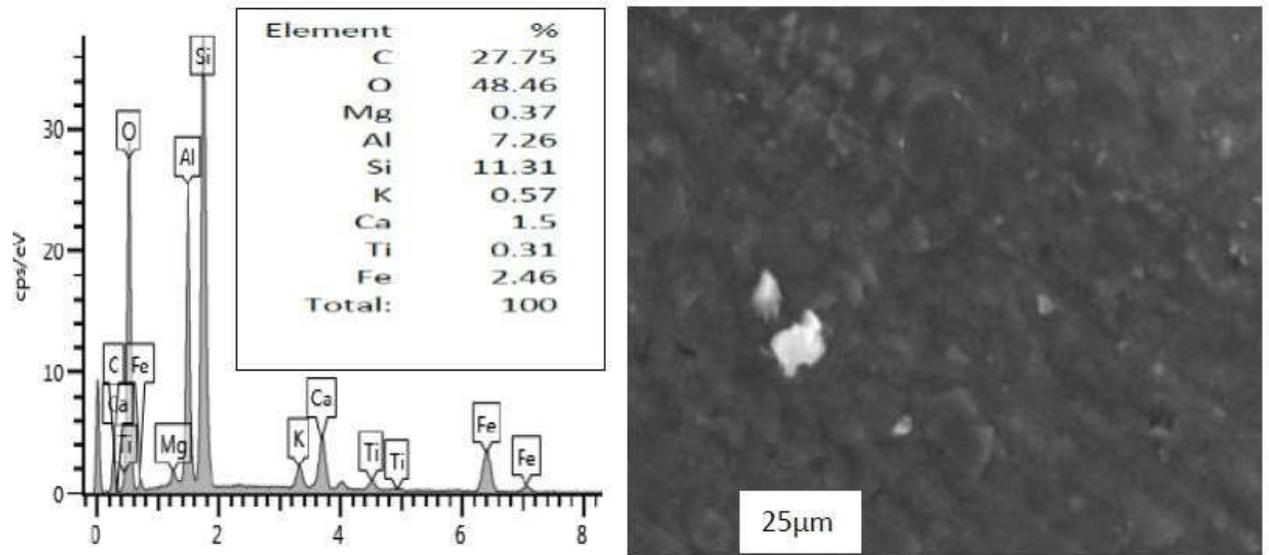


Figure 4.7: SEM and EDS result of the Clay and Clay mixed with 6% cement

Calcium alluminate hydrate produced must have hold the clay particles together in a uniform mass. The silica-alumina ratio recorded from the EDS was 1.56 which indicates complete lack of montmorillonite mineral in the mixture. This is probably due to the reaction of calcium ion in cement with the electrons around the clay mineral particles which results in to agglomeration of the clay particles in to larger particles thereby modifying the active clay minerals.

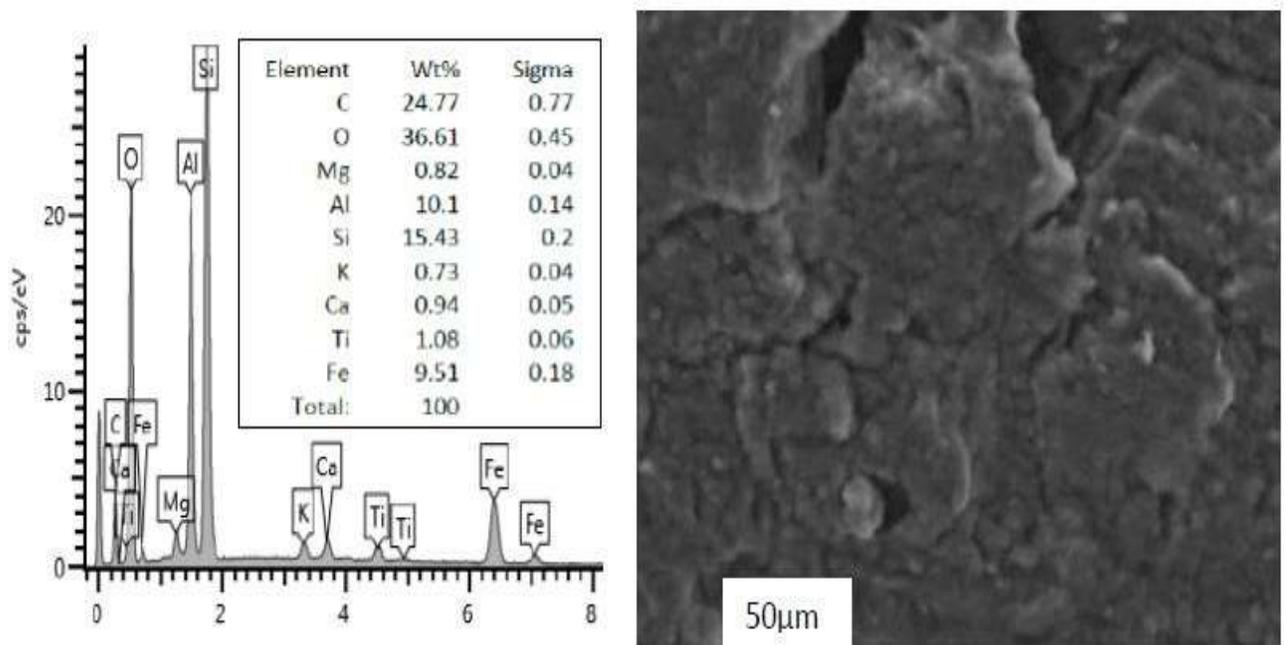


Figure 4.8: SEM and EDS result of the Clay and Clay mixed with 6% volcanic Ash

Addition of volcanic ash alone give the structure shown in Figure 4.9. The SEM picture showed rough but relatively compact structure consisting of pore spaces. This is an indication that cementation is less than that of cement addition. The silica-alumina ratio recorded from the EDS result was 1.53 which clearly indicates that active montmorillonite has been deactivated.

Addition of both cement and volcanic ash resulted in to the SEM picture shown in Figure 4.3 This structure showed compactness with void spaces being filled with whitish cementitious calcium silicate

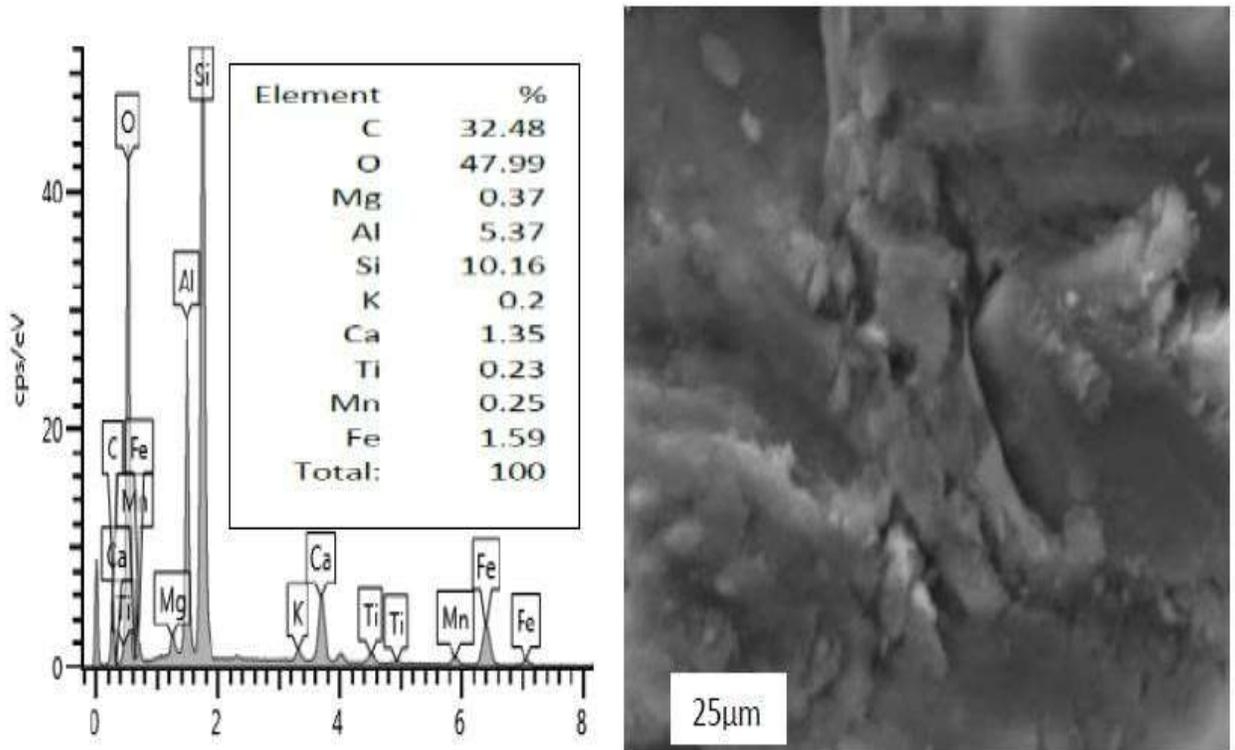


Figure 4.9: SEM and EDS of the Clay and Clay mixed with 6% cement and 15% volcanic Ash

Hydrate which is expected to give the higher strength and stability.

4.3 Effect of Volcanic Ash on UCS of Cement Stabilized Clay Soil

The effect of UCS with varied composition of volcanic ash (VA) for 0 percentage of cement is shown in Figures 4.10. The result revealed continuous increase in UCS with increase in VA. The values increased

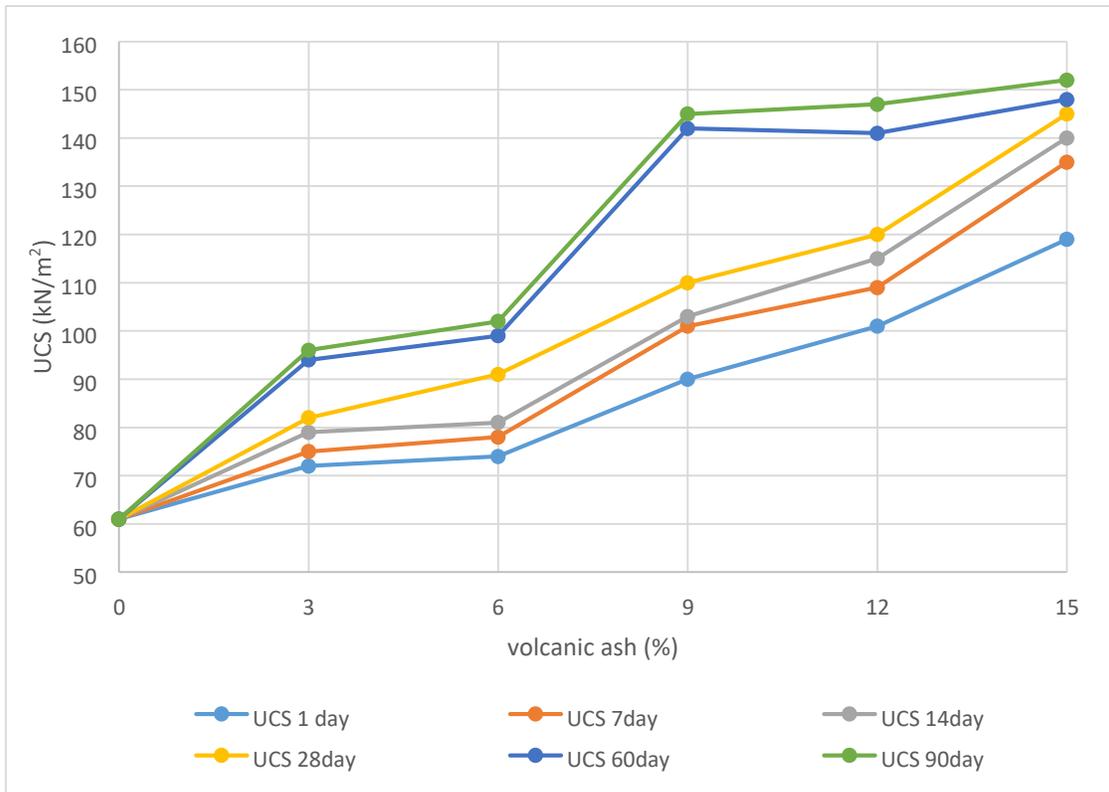


Figure 4.10: Variation of UCS with volcanic ash at 0% cement Addition from 61kN/m²

Variation of UCS with volcanic ash at 0% cement Addition from 61kN/m² at 0% volcanic ash to 96, 102, 145, 147 and 152kN/m² at 3, 6, 9, 12 and 15% VA respectively. No optimal volcanic ash addition was recorded for 0% cement addition. Addition of 15% VA therefore represent 2.5 times increase in UCS of the clay. This strength increase is attributable to formation of cementitious calcium silicate hydrate and calcium aluminate hydrates from the 9.5% calcium oxide, 43.5% silica and 13.4% alumina recorded in the volcanic ash. Low magnitude of calcium oxide in the presence of large amount of silica do not encourage formation of byproduct of calcium hydroxide which could have reduce the strength at higher values of volcanic ash.

Addition of 2% cement to the clay showed variation in UCS with addition of volcanic ash as shown in Figure 4.4. It was generally observed that the UCS increased from 0% volcanic

ash to its maximum at between 6% and 9% volcanic ash after which the values reduce. For samples cured for 1 day, the UCS increased from 230kN/m² at 0% volcanic ash to maximum value of 315kN/m² at 6% volcanic ash after

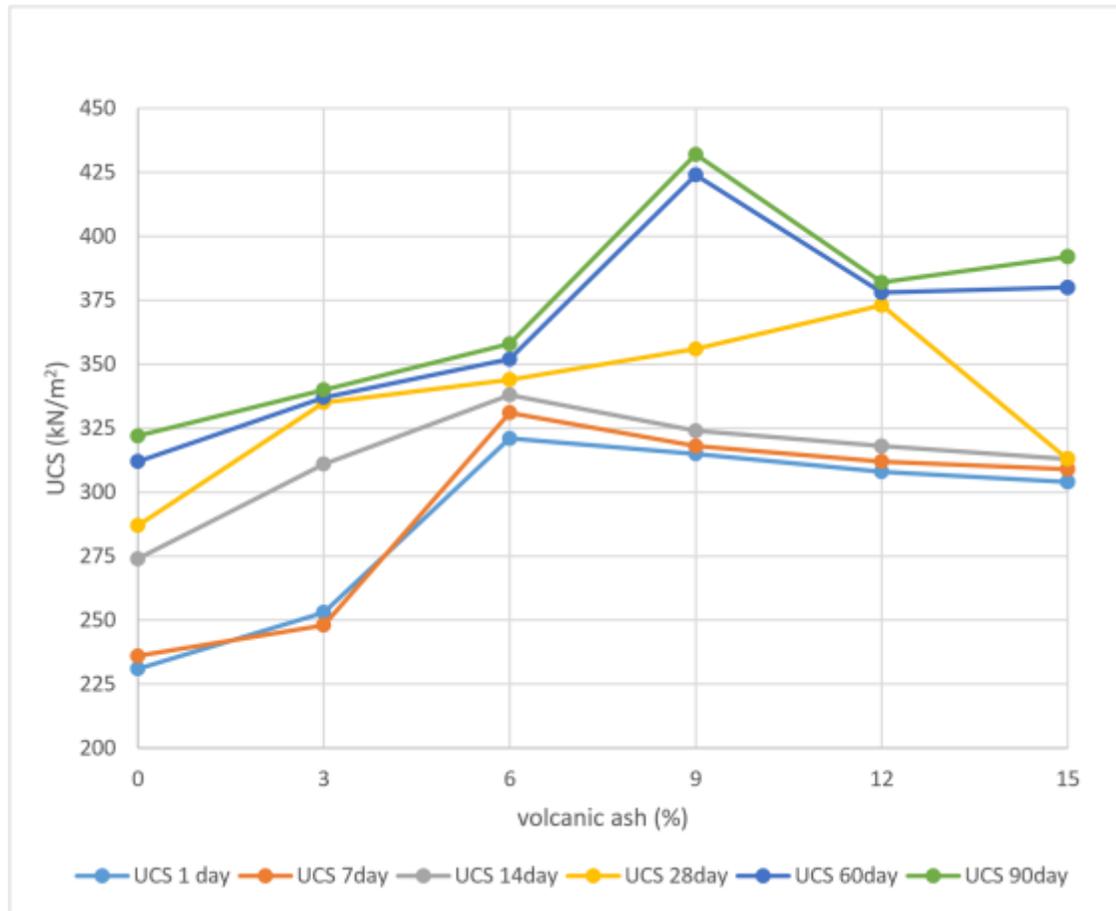


Figure 4.11: Variation of UCS with volcanic ash at 2% cement Addition

which the value reduced to 308 kN/m² at 15% volcanic ash. This represents 37% increase in strength which was generated by volcanic ash. After 90 days of curing, the UCS increased from 320kN/m² at 0% volcanic ash to maximum value of 430 kN/m² at 9% volcanic ash after which the values reduced to 380kN/m² at 12% volcanic ash. This also represents 34% increase in UCS. The reduction in strength after 9% volcanic ash probably resulted from the negative effect of calcium hydroxide generated as a byproduct of cement hydration. Addition of 4% cement to the clay and admixed with varied composition of volcanic ash is shown in Figure 4.12

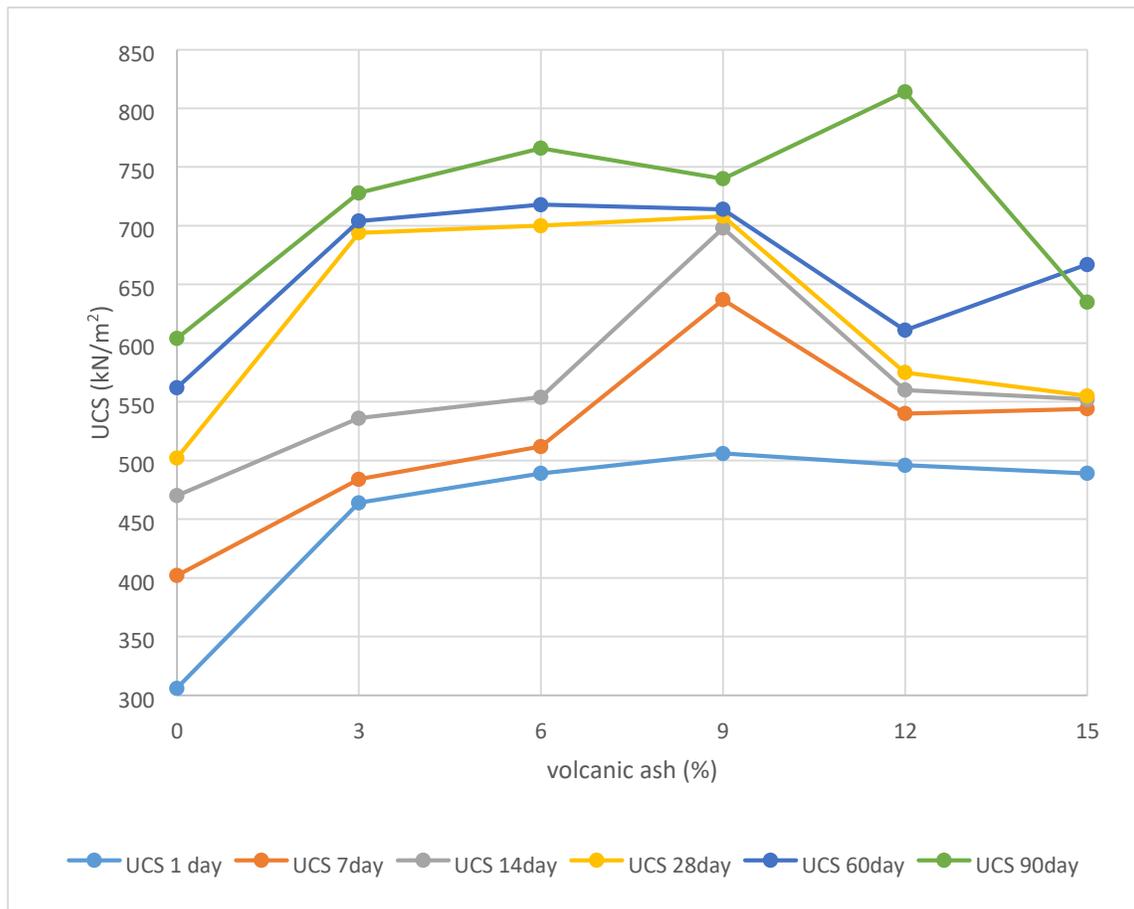


Figure 4.12: Variation of UCS with volcanic ash at 4% cement Addition

Figure 4.4. For 7 days curing, the UCS increased from 400kN/m² at 0% volcanic ash to its maximum of 630 kN/m² after which the value reduced to 540 kN/m² at 15% volcanic ash addition. This represents 35% increase in UCS. After 90 days of curing, the UCS increased from 600 kN/m² at 0% volcanic ash to maximum of 810 kN/m² at 12% volcanic ash after which the values reduced to 630 kN/m² at 15% volcanic ash. This also represent 35% increase in UCS.

Addition of 6% cement with varied composition of volcanic ash gave higher UCS values. After 1-day curing, the UCS increased from 800 kN/m² at 0% volcanic ash to 865 kN/m² at 9% volcanic ash after which the value reduced to 720 kN/m² at 15% volcanic ash. Only low value of 8% increase in UCS was recorded for this curing day. After 90 days of curing, the UCS increased from 915 kN/m² at 0% volcanic ash to maximum of 1275 kN/m² after which the values reduced to 1170 kN/m² at 12% volcanic ash addition. This represents 39% increase in UCS.

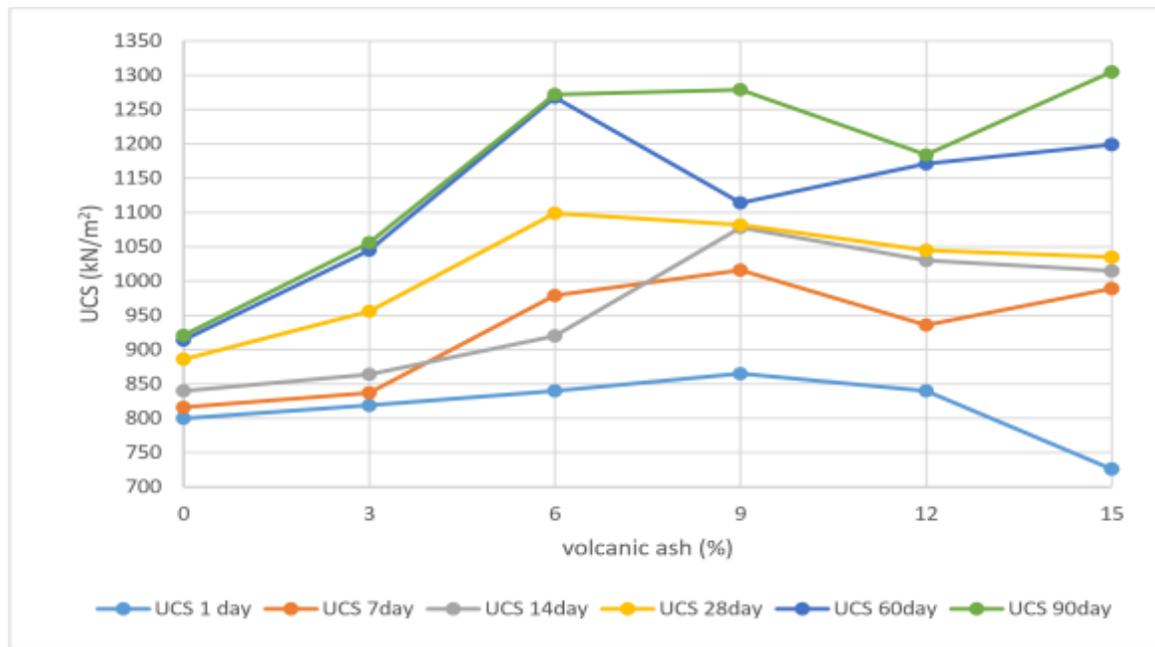


Figure 4.13: Variation of UCS with volcanic ash at 6% cement Addition

The optimal percentage VA to give the maximum strength occur between 6% and 9% for 2% cement, 4% cement and 6% cement. However, composition with 6% cement occasionally gives optimal strength at 15% VA composition. It can therefore be inferred that higher percentages of cement results to higher optimal VA to give the higher strength. The increases must have resulted from the reaction between the silica contained in the VA and calcium oxide present in cement to form cementitious calcium silicate hydrate (CSH). The reduction observed beyond 9% VA is probably due to higher byproduct of calcium hydroxide which form unreactive whitish solids that reduces the strength of the specimen.

4.4 Effect of Curing Time on Cement and Volcanic Stabilization of Clay

The strength for 0% VA was observed to be constant at 61 kN/m² throughout the duration of curing. Addition of 3% VA gave gradual increase in UCS from 72 kN/m² after 1 day curing to 96 kN/m² after 90.

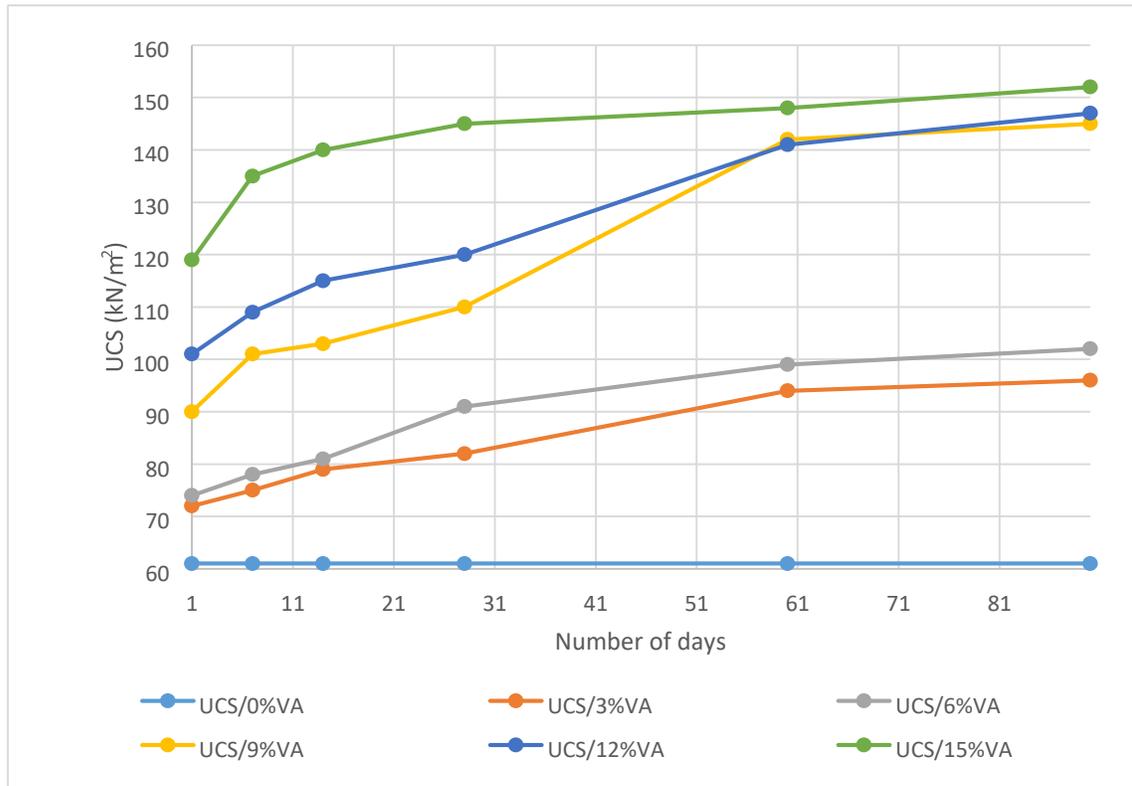


Figure 4.14: Variation of UCS with curing days for 0% cement

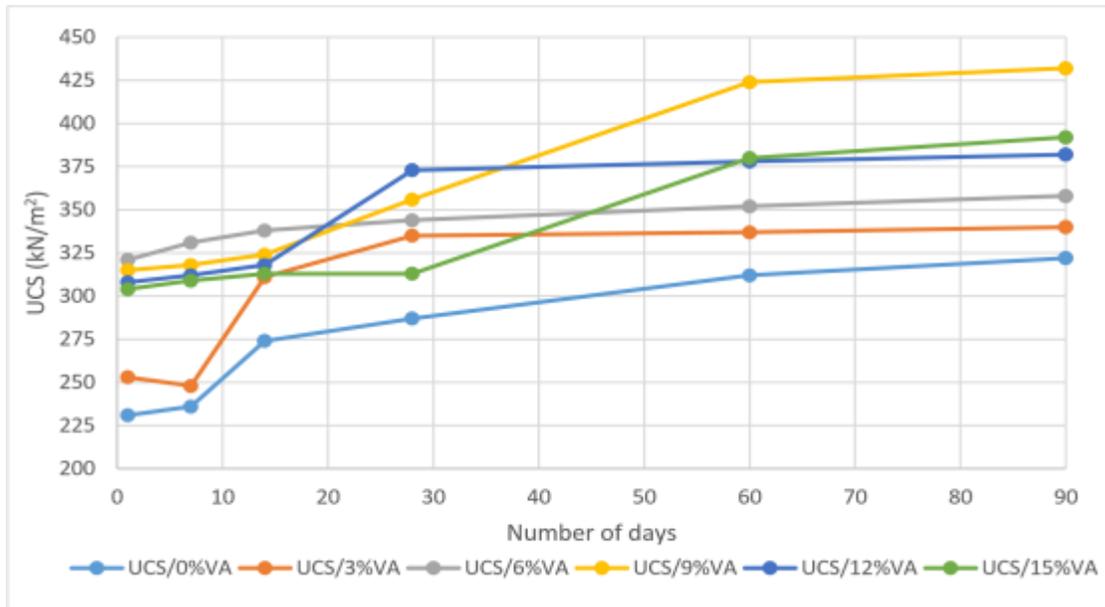


Figure 4.15: Variation of UCS with curing days for 2% cement

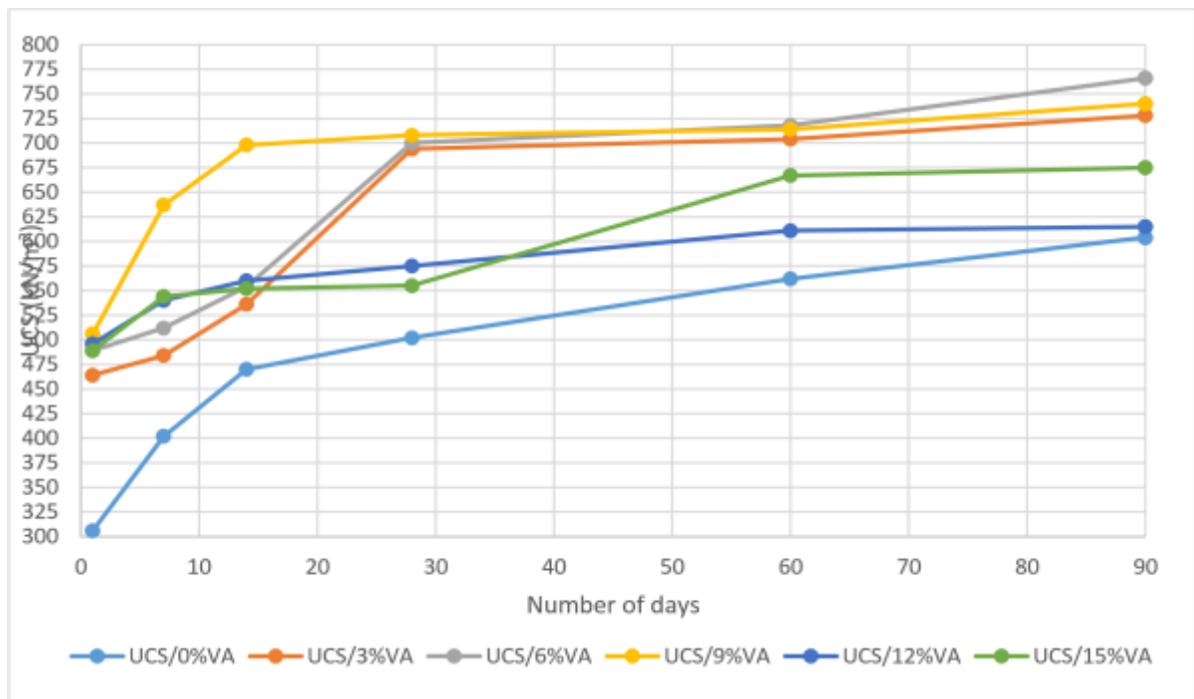


Figure 4.16: Variation of UCS with curing days for 4% cement

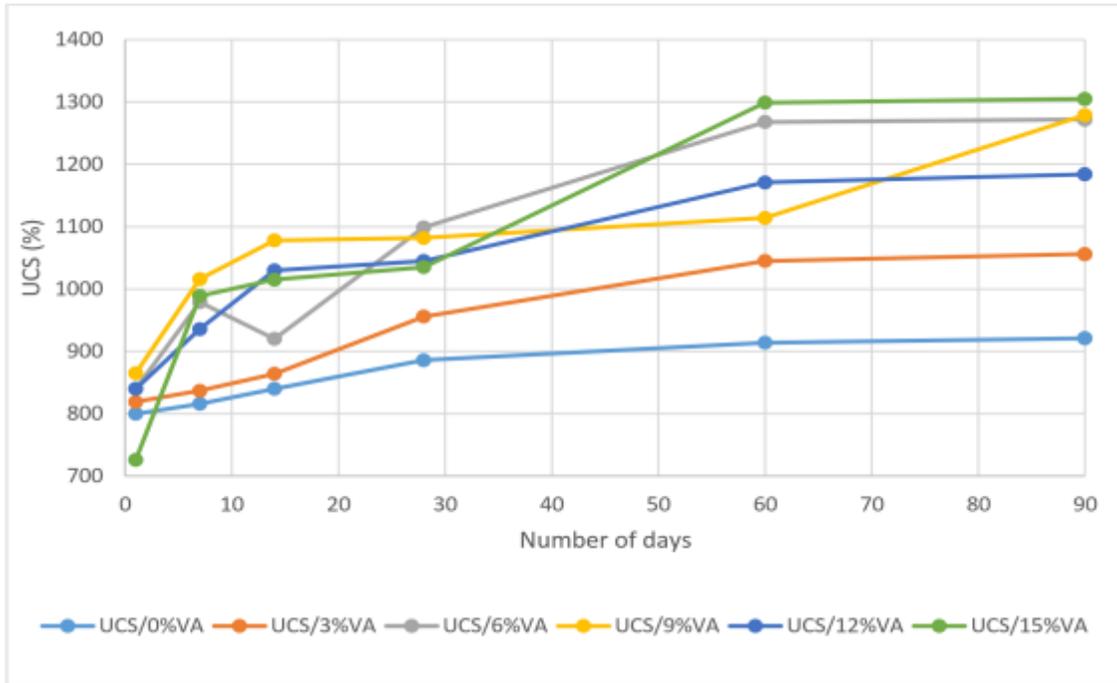


Figure 4.17: Variation of UCS with curing days for 6% cement

days of curing which represents 25% increase in strength. At 0% cement and 15% VA, the UCS increased from 119 kN/m² after 1day curing to 152 kN/m² after 90 days curing which also represents 28% increase in UCS. This increase in strength must have resulted from the reaction between silica contained in VA and the ions surrounding the clay minerals. Figure 8b showed the variation in UCS with addition of VA at 2% cement. Addition of 9% VA was observed to stand out with UCS increasing from 315 kN/m² after 1day curing to 432 kN/m² after 90 days curing which represents 37% increase in strength of the clay.

The maximum strength obtained on addition of 6% cement with varied composition of VA occurred at 15% VA which gave UCS of 726 kN/m² after 1day curing to 1305 kN/m² after 90 days curing which represents 80% increase in strength of the clay.

Figure 4.17 which deals with addition of VA without cement and revealed gentle slope or gentle rate of increase in strength with increase in curing days except for 15% VA which gave high rate of increase in strength during the first 28 days of curing after which the slope flattened. Figures 8b, 8c and 8d which composes of 2%, 4% and 6% cements with varied compositions of VA, all showed faster rate of strength increase from 1 to 28 days after which the slope flattened out.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the experiment and analysis carried out in this study, the following conclusions can be drawn:

The clay soil studied falls under clay of high plasticity (CH) based on Unified Soil Classification System (USCS) which cannot be used in its natural state for any component of flexible pavement structure.

Microstructural analysis of the clay using XRD and SEM revealed that the clay consists of both primary and secondary minerals including montmorillonite. The SEM showed the morphology of the compacted clay as flaky in nature with pore spaces.

Maximum UCS of 1305kN/m² which satisfy a soil material to be used as subbase material for flexible pavement structures. This was achieved at 6% cement and 15% volcanic ash.

The optimal VA required for effective stabilization of clay soil lies between 6 to 9% for clays with lower cement content and 9 to 12% for clay with higher cement contents.

The UCS obtained for clay stabilized with VA only increased by 2.5 times the strength of the natural clay while the UCS at 6% cement increased by 80% at 15% VA addition.

The VA obtained from Keran Swap hill in Mangu Local Government of Plateau State can be used to stabilize clay soil in conjunction with cement for sustainable flexible pavement structure.

5.2 Recommendation

The clay collected from Bako village along Gwagwalada – Garki road can be stabilized with a blend of 6% cement and 9% volcanic ash and used as subbase course material for highly trafficked roads base on Nigerian General Specification for Road and Bridge Works (1990).

5.3 Contribution to Knowledge

The study established that treatment of tropical soil with 6and 15% cement and volcanic ash repectively, resulted to a material with unconfirmed compressive strength (UCS) value of 1305k/Nm², which satisfied the strength requirement for material to be used as pavement sub-based.

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