GEOTECHNICAL CHARACTERIZATION AND BEHAVIOUR OF CEMENT AND LIME-STABILIZED EXPANSIVE SOILS IN AWKA AND ENVIRONS, SOUTHEASTERN NIGERIA

BY

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CERTIFICATION

This is to certify that this work "Geotechnical characterization and behaviour of cement and lime-stabilized expansive soils in Awka and environs, Southeastern Nigeria" was carried out by **OGBUCHUKWU, PATIENCE ONYINYE (20154988788)** in partial fulfilment for the award of degree of Master of Science (M.Sc.) in Environmental Geology in the Department of Geology, Federal University of Technology Owerri, Imo State, Nigeria.

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DEDICATION

I dedicate this research work to God Almighty, who has made it possible for me to carry out this project work. May His name be praised forever.

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ABSTRACT

This study was undertaken to ascertain the geotechnical properties of the expansive soils responsible for the swelling potential and degree of expansion of the studied soils in Awka and environs Southeastern Nigeria. A total of eight (8) expansive soils were collected at different locations of the studied area and the geotechnical properties analyzed in the laboratory includes; grain size, Atterberg limit test, linear shrinkage, natural moisture content, free swell, compaction test, CBR test, specific gravity, dry and bulk density. Results of the study revealed a high to very high swelling potentials using the liquid limit (63.10 to 77.40%), plasticity index values (31.48 to 46.45%) and medium to high activity using activity values (0.98 to 1.55%). A critical and high degree of expansion was observed with linear shrinkage (10.70 to 20.00%) and free swell values (53.00 to 71.00%) respectively. In addition, low strength characteristics of the studied soils were similarly observed with the unsoaked (10.20 to 18.10%) and soaked (1.90 to 3.30%) CBR values, the optimum moisture content values (17.70 to 27.00%) and maximum dry density values (1.40 to 1.75Mg/m^3). The Grain size analysis revealed that the studied soils are poorly-graded material with percentages passing the No.200 BS (62.10 to 94.80%) and can be classified as CH soils (fat clays) when plotted on a Casagrande plasticity chart. After stabilizing the studied soils with different proportions of cement and lime (2 to 10%), there were reductions in the swelling potential, degree of expansion and increase in the strength characteristics and durability of the soils. The optimum (minimum LL and PI values) values of the swelling indicators were achieved at 6% and 8% of lime and cement additives respectively. Soil stabilization with lime performs better than cement because lime has adverse effects on the swelling potential of a soil than cement but cement performs better when strength characteristics of the soil is the criteria.

Keywords: Expansive soils, swelling potential, Atterberg limit, degree of expansion, road failure, strength characteristics.

CHAPTER ONE

INTRODUCTION

1.1: Background of the Study

The importance of soils for engineering purposes cannot be over-emphasized since soils are essential raw materials for geotechnical engineers and almost all civil engineering works involves soil. The behavior of every engineering structures erected on soils or rocks like highways, buildings, underground utilities, hydraulic conduits, slopes and embankments depends primarily on the engineering characteristics of the underlying deposits (soil or rock). The geotechnical engineers, architects and geoscientist face a lot of challenges when engineering structures are founded on a problematic soil (expansive soils). The occurrence of expansive soils in any environment causes severe damages to engineering structures like highways, buildings, hydraulic conduits, underground utilities, slopes and embankments. Expansive soils are soil which has the tendency to increase in volume when wet and reduce in volume when the moisture dries off, it can also be defined as soils that increase in volume (swell) when they come in contact with water and reduce in volume (shrink) when they loses the water (Al-Rawas *et al.,* 2002). Craig (1992) postulated that if poor soils cannot be removed during engineering works, then their engineering behaviour/properties can be enhanced by suitable method of ground treatment. Bell (1993; 2007), suggests that the treatment of such soil can be done by preventing the ingress of groundwater flow or removing the soil from the site in question or by improving the soil strength through chemical or mechanical technique.

Expansive soils occupies about 20% of the world surface area (Chaitanya *et al.*, 2011) but in Nigeria, these soils occupy an area of about 10.4×10^4 km^2 in the north eastern part (Ola, 1983). According to Zumrawi & Hamza (2014), this problematic soil upon swelling and shrinking causes severe impairment to the engineering structure founded on them. Damages arising from expansive soils are enormous and are usually manifested in all kinds of engineering structures including buildings, highways, hydraulic conduits, underground utilities, slopes and embankments.

Kerrane (2011), state that United States Housing and Urban Development (HUD) in 1981, estimated 9 billion US dollars as damages resulting from expansive soil. Jones & Holtz (1973) is of the view that effects of expansive soils on engineering structures in the United States exceed the combined average annual damages from flood, hurricane, earthquakes and tornadoes while Steinberg (1992), reported that the annual cost of damages from expansive soils in the United States to be over $10billion. Charlie (1984) estimated that the annual cost of damages caused by expansive soils on engineering structure in Sudan as Expansive soil prevails over a large area of Sudan. These problematic soils do not have potentials for use in most engineering constructions due to their swelling and shrinkage nature and they also experience settlement with a reduction of strength (Owolabi & Ade rinola, 2014).

Ola (1981) postulated that expansive soils have the tendency of absorbing large volume of water due to the presence of montmorillonite/smectite clay mineral in them. These montmorillonite/smectite clay minerals present in the expansive soils causes the soil to shrink and swell when it absorbs or loses moisture. Other clay minerals include kaolinite, illite and chlorite which may pose little or no significant problems to engineering structures. The Kaolinite clay minerals are formed in well-drained soil with abundance of oxygen, silicon and aluminium. They form regular shapes because their constituents are in pure form. According to Okogbue (1990), these kaolinite clay minerals are held together in large stacks by strong hydrogen bonds to form a hexagonal network which has the composition Si_4O_{10}. The kaolinite group has 2-layer lattice structure and consists of an octahedral layer (gibbsite layer) and tetrahedral silica layer.

Montmorillonite is expansive because the layers are loosely held together. With the introduction of K^+ between the loosely held 3-layer lattices of the group, it becomes closely held giving rise to the Illite group with the general chemical formula

$$K^+1\text{-}1.5Al_4^{3+}(Si^{4+}Al^{3+})_8O_{20}(OH)_4 \qquad \text{\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_} \qquad (1)$$

Montmorillonite clay minerals are formed in poorly drained soil so that a wide range of atomic species are available for recrystallization. Montmorillonite has 3-layer lattice, the layers consist also of alumina octahedral and Silica tetrahedral. The alumina (gibbsite) octahedral is sandwiched between the silica layers. Each of the layer unit is loosely stacked upon the other, and with positive cations and water between them. Aluminium octahedral is formed when magnesium atom substitutes for an aluminium atom (isomorphic substitution). Montmorillonite form micelles to be electrically balanced with water and cations. This creates irregular and unbalanced charges with weak "van der waals" forces between them.

Kaolinite clay with a general formula ($Al_4Si_4O_{10}(OH)_{8,}$ suggest that the group is isochemical and no replacement of the Aluminum (Al^{3+}) by Mg^{2+} and Fe^{2+} has been observed in the gibbsite layer. However substitution of Aluminum (Al^{3+}) in the gibbsite layer of montmorillonite group is common hence having a general formula

$$A^{2}/_{3}(B,C)4\text{-}6(Si,Al)O_{20}(OH)_{4}nH_{2}O \qquad \text{———————————} \quad (2)$$

Where;

$A=Na^{+}$ or $^{1}/_{2}Ca^{2+}$, $B=Al^{3+}$ or Fe^{3+} and $C=Mg^{2+}$ or Fe^{2+}

Illite is a product of altered micas, and alkaline feldspar in alkaline environment. Chlorite group consist of talc units $Mg^{2+}6Si^{4+}8O_{20}(OH,F)_{4}$, found between the brucite layer (Mg_6OH_2) thereby giving rise to the general formula (Mg^{2+}, Fe^{2+}) $10Al_2^{3+}(SiAl)_8O_{20}(OH,F)_{16}$. Solid solution exists in this group. Chlorites are alteration products of Ferromagnesium mineral (Okogbue, 1990).

According to Braja (1996), the major problems associated with expansive clayey soils, especially the montmorillonite-rich soils are the volume change and this often results to severe damages of the engineering structures. The effects of expansive soils (swelling and shrinkage) as they interface with moisture pose serious threat to man and the environment (Jones & Holtz 1973; Krohn & Slossen 1980; Freeman *et al.,* 1991). Expansive soils are responsible for the problems of heaving and settlements of structures which arise from swelling and shrinking of foundation soils. Cracking of structures as a result of heaving and settlement are common problems in regions underlain by expansive soils with the plastic properties of clays are responsible for their susceptibility to volume changes (Okogbue, 1990).

1.2: Problem Statement

Expansive soils are characterized by clay minerals such as montmorillonite /smectite which have the tendency to absorb moisture causing an increase in the soil volume (swell) but when it loses the moisture, it decreases in volume (shrink). According to Braja (1996), the major problems associated with expansive clay soils, especially the montmorillonite-rich soils are the volume change which causes severe damages to engineering structures. The effects of expansive soils (swelling and shrinkage) as they interface with moisture pose serious threat to man and the environment in the study area. The occurrence of expansive soils has led to cracking and eventual collapse of engineering structures such as buildings, drainage facilities and severe damage to roads in the study area and the durability of these engineering structures are threatened or reduced due to cracks on the walls of buildings. The effects of expansive soils on engineering structures in the study area are shown below (Figure 1.1 - 1.4).

Figure 1.1: Abandoned building due to cracks at Ugwuoba

Figure 1.2: Patched cracks on the wall due to expansive soil at Ugwuoba

Figure 1.3: Failed Portion of the road at Enugwu- Agidi

Figure 1.4: Failed portion of the road at Ufuma

1.3: Main Objective and Specific Objectives of the Study

The main objective of this research work is to determine the geotechnical properties and evaluate the effects of cement and lime-stabilized expansive soils from the study area. The specific objectives of the study include:

i. To determine the geotechnical properties of expansive soils in the study area.
ii. To stabilize the expansive soils in the study area with appropriate proportion of cement and lime.
iii. To evaluate the effectiveness of the stabilization of expansive soils in the study area using cement and lime.

1.4: Justification of the Study

Road failure and cracking on the walls of building has been major challenges in areas underlain by expansive soils because of the intrinsic properties of montmorillonite clay minerals contained in them.

Few studies had been done to understand and evaluate the cause of damages to engineering structures due to the behaviour of expansive soils in the study area. Ensuring that these problems (road failures, crack) associated with expansive soils in the study area are solved, appropriate investigation and stabilization treatment of the expansive soils is of utmost important.

Therefore, this research work will also provide a scientific uniformity that will guide decision-makers to better understand the mechanisms behind these infrastructural failures/damages, information on best practices of stabilization as well as information on the cost effectiveness of the stabilization of expansive soils with cement or lime in the area will be provided.

1.5: Scope of the Study

This study involved field sampling and laboratory analyses of expansive soils derived from the Ameki Formation and Imo Shale Formation in Awka area and evaluation of stabilization/treatment using cement in order to determine their geotechnical properties. It also investigates the effectiveness of stabilization of the expansive soils using cement or lime to enhance the geotechnical properties of the expansive soils.

CHAPTER TWO

LITERATURE REVIEW

2.1: Review of previous related literatures

Several authors have studied some of the disastrous effects of expansive soils on engineering structures like building, hydraulic conduit, highways, underground utilities, slopes and embankments. Most authors reported that using chemical additives such as lime, cement, common salt, rice husk ash and fly ash can improve the soil geotechnical properties (Okeke *et al.*, 2015; Bell, 1993; Powrie, 1997; Craig 1992). Okeke (2008); Okeke & Okogbue (2010a) established that the occurrence of expansive soils in parts of Southeastern Nigeria is as a result of weathering of pyroclatic rocks in Abakiliki area and the shaly geologic formations which are widely spread in Asu river group, Ezeaku, Awgu, Nkporo, Mamu, Nsukka Formations and Imo Shale. They therefore postulated that the montmorillonite clay mineral content of expansive soil of humid tropical climate is influenced by the type and characteristics of the parent rocks while authors like Gromko (1974); Harry (1974) and US Army (1983) are of the opinion that expansive soils are formed as a result of weathering of fine grained extrusive igneous rocks and montmorillonite rich mud rocks, such as shales and mudstones. Harry (1974) maintained that the geologic origin (nature of the parent rock) of the expansive soils influences their mineralogical composition and distribution in any area they occur. The type of clay mineral present in a soil can give a quick and approximate knowledge of the swelling potential of the soil. The term clay can be used to describe rock or soil types, as well as mineral species.

Legget (1962) is of the view that a soil mixture containing clay sized particles (of sand and silt mixture) up to 70% by weight can classify as clay, as long as, the clay and silt sizes combined exceed 50% by weight of the entire soil mixture. Soft rocks and expansive soils are most often associated with non-durability foundation associated problems as well as structural failures which are often associated with the compositional factors and post depositional processes (Punmia *et al.*, 2005). Post depositional processes contributing to weak rock formation include slaking or dissolution in water, weathering, poor cementation and consolidation, development of planes of weakness and proximity to other layers with substantially different properties. US Army (1983), Day (1999) and Lucian (2011), stated that soils above the depth of active zone of expansion exhibit wide range of change in moisture content while those below the depth of active zone of expansion do not exhibit changes in moisture content and thus do not contribute to soil expansion. The factors that determine the depth to active zone in soils that have the ability to swell or shrink are water table and climate. Generally active zone will extend to the depth of shallow water table.

The depth of active zone in humid tropics is less than 2.5m, while in Semi- arid tropics it is between 1.5 and 5m and in arid tropics it is greater than 5m (US Army, 1983; Lucian, 2011). Ola (1981) reported about 70% montmorillonite content in the high swelling black cotton soil in Maiduguri, Nigeria. He also reported that as small as 10% content of montmorillonite could show strong indications of expansiveness and shrinkage, especially in arid and semi-arid regions as seen in Bakura area of Sokoto, Nigeria. Thus, Ola (1981) concluded that the numerous cracks and cracking patterns observed in buildings in Bakura area of Sokoto state, Nigeria, is as a result swelling and shrinkage characteristics of the problem clays as the interferes with moisture.

On the other hand, Attimeyer (1956) faulted the use of free swell test to determine expansive of soil. He was of the view that Atterberg limits should be more reliable for estimating expansiveness of soil. However, a relationship between Atterberg limits and linear shrinkage was established by Heidema (1957), who noted that the relationship is more defined with plasticity indices than with liquid limit while Kantey & Brinks (1952), observed the possibility of recognizing expansive soil by their linear shrinkage proportion. They noted that linear shrinkage values greater than 8 indicate expansiveness. According to Baracos & Bozozuk (1957), Haidema (1957), expansive soil has characteristically moderate to high montmorillonite content (>10%), Plasticity indices of (>15%), liquid limits of (>40%) and high shrinkage values of (>8%).

These peculiar characteristics are often utilized in classification of expansive soil.

According to Okogbue (1990) plastic properties of clays are responsible for their susceptibility to volume changes. Aghamelu *et al.*, (2011) maintained that shale is most often regarded as a problem material in construction due to its high expansivity and compressibility, low strength and bearing capacity of shales in general while Aghamelu & Okogbue (2011) revealed that most projects suffer incessant failures, soon after construction because their swelling and shrinkage behaviour generally result to low strength. Liu *et al.*, (2008) explained that plastic soils are very prone to shear failure due to the constant load over time and generally considered as poor engineering material for foundations. Bell (1993), also suggested that various clay mineral group have different swelling potential, and maintained that kaolinite minerals has the smallest swelling capacity among the clay minerals while illite may swell up to 15%. Intermixed illite and montmorillonite may swell up to 60-100%.Okeke and Okogbue (2010a) reported that soils derived from Imo-shale Formation in Bende area have very high swelling potential. They also noted that expansive soils derived from the Mamu Formation in Ohafia have very high swelling potential; while those derived from Nsukka formation in Ohafia area have medium swelling potentials. It was noted that only the expansive soils derived from Nkporo Formation in Enugu and Afikpo areas (with average free swell values of 46.60%) and soils derived from Nsukka formation around Umulolo/Ihube-Okigwe and Ohafia area (with average free swell value of 47.90 %) exhibit low degree of expansion. They noted the high degree of expansiveness with free swell values of about 50.80% for soils derived from the Mamu and Imo-shale.

Beni (2010) was of the opinion that cracks on the walls were generally caused by structure movements due to expansive soil behaviour with medium to high expansibility. Adding different additives to soils can therefore improve their geotechnical properties so as to make them suitable for construction. Powrie (1997) revealed that expansive soils could be stabilized either by physical, chemical or use of reinforcement elements (geotextile and stone columns) while Okeke *et al.,* (2015) noted that the sole aim of soil stabilization with additives like cement or lime is to reduce the affinity of the soil to swell when in contact with water. He concluded that engineering properties such as liquid limit, plasticity index and linear shrinkage which are normally responsible for swelling behavior of the soils are generally reduced when appropriate percentage of addictive is added to the soil. Atuonye *et al.,* (2016) investigating soil samples collected from Amuro-Okigwe problematic soil and reported that cement/ Rice Husk Ash mixtures or lime could be employed in the improvement or stabilization of expansive soils. He observed that the untreated Amuro-Okigwe clay had plasticity index (PI) of 31% (swelling potentials) which indicate high expansive soil as categorized by Ola (1981). However, when the clay was treated with 8% of cement mixture, the plasticity index (PI) reduced to 12%. Amu *et al.,* (2005) states that strength of a material increases with curing time and that the sample stabilized with 9% cement plus 3% fly ash, produced the best and highest compressive strength and improved the stabilizing potential of the sample better than using 12% of cement only on a sample. Whitlow (1995), classified soils with liquid limit less than 35% as a low plasticity soil, between 50-70% as high plasticity soil, between 70-90% as very high plasticity soil, while greater than 90% as an extremely high plasticity soil.

Qubain *et al.,* (2000) is of the opinion that lime improves the strength of clayey soils by three mechanisms which includes hydration, flocculation and cementation. The first and the second reaction take place almost immediately upon introducing lime while the third is a prolonged effect. Ampera & Aydogmus (2005), treated Chemnitz clayey soil (A-7-6 Group) according to American Association of State Highway and Transportation Officials (AASHTO) using lime (2, 4 and 6%) and cement (3, 6 and 9%). They conducted compaction, unconfined compressive strength and direct shear test on untreated and treated soil samples. They concluded that the strength of the cement treated soil was greater than the lime treated soil. They also found out that lime-treatment is in general more tolerant of construction delays than cement treated, and more suitable for clayey soils.

Parson & Milburn (2003) conducted a variety of test on seven different soils using lime, cement, class C fly ash and an enzymatic stabilizer. They determined Atterberg and unconfined compressive strength of the stabilized soil before and after carrying out durability test. They reported that lime and cement treated soils showed better improvement compared to fly ash treated soil.

2.2: SOIL STABILIZATION/ IMPROVEMENT

Geotechnical properties of problematic soils such as soft fine-grained and expansive soils are improved by various methods such as soil stabilization (chemical or mechanical), or by removing and replacing the problematic soil with good quality material (in-situ method).

Soil stabilization is the process of modifying expansive soils to enhance their physical properties or a method of improving the geotechnical properties of expansive soils to make them suitable for engineering purposes. Soil stabilization is an effective way for improving the geotechnical properties of expansive soils by controlling the shrinkage and swelling properties of the soil, thereby improving the load bearing capacity, strength and durability of subgrade to support pavement and foundation.

Soil stabilization is generally done by mechanical, chemical or in-situ methods in other to improve the geotechnical properties of the soil (such as strength and stiffness). The mechanical method include densifying treatments by the application of mechanical energy (including compaction or preloading), pore water pressure reduction techniques (including dewatering or electro-osmosis), the bonding of soil particles (by ground freezing, grouting, and chemical stabilization) and use of elements such as geotextiles and stone columns (Powrie, 1997).

The chemical method may be employed to improve the geotechnical properties of the expansive soils by treating it in-situ. This method is very important for the geotechnical engineering applications such as pavement structures, roadways, building foundations, channel and reservoir linings, irrigation systems, water lines, and sewer lines to avoid the damage due to the settlement of the soft soil or to the swelling action (heave) of the expansive soils. Chemical means of stabilization involves mixing or injecting soil with chemically active compounds. It can also be defined as a method of adding or mixing different addictive (binding agents) in their proper percentages with natural soils to reduce moisture and improve the strength of the soil (sub grade).

These chemical additives are manufactured commercial products, which when added to soil and thoroughly mixed, will improve the quality of the soil. These additives include Portland cement, fly ash, rice husk ash, lime, calcium or sodium chloride, viscoelastic materials such as bitumen byproducts of lime kiln dust, cement kiln dust, quarrying dust or combinations of these materials (Zumrawi, 2015; Tensar Technical Note, TTN, BRIO, 1998; Materials and Test Division, Geotechnical section, Indiana, 2002). The percentages of additives to be used are dependent on soil classification and degree of improvement desired.

For instance, smaller amount of additive are used when it is desired to modify the soil properties such as gradation, workability and plasticity. When strength of the soil are to be improved, larger quantities of additives are generally used (U.S Army, Navy & Air force 2005).

In Germany, Vosteen (1998; 1999) reported that the use of cement or lime for the stabilization of pavement bases (during the past few decades) was investigated and developed into practical construction procedures. These practical procedures have been improved and covered periodically by the technical standards for road and traffic. Petry & Little (2002), state that the most widely used chemical additive for stabilization of expansive soils are cement, lime and fly ash while Roads & Streets (1959), Thompson (1968), Greaves(1996) also state that lime and cement are widely used in soil stabilization of highway pavement subgrades and they concluded that soil stabilization with lime is mostly recommended especially in soils with high plasticity and high clay fraction because it improves the workability of the clay and achieves higher and faster rate of reduction of the engineering properties within a short time while treatment of subgrade soils with cement is recommended than lime in soil with less clay fraction, low plastic characteristic and where early strength is desired (Moh, 1962; Peck, *et al.,* 1974; US Army, 1984; Mowfy *et al.,* 1985; Bell 1993; Raj, 2008; Jha & Sinha, 2009). Amu *et al.,* (2005) studied the stabilization of expansive clay soils with the combination of cement and fly ash. They reported that soil samples stabilized with a mixture of cement and fly ash had better performance with respect to maximum dry densities, optimum moisture content, bearing capacity and shear resistance than the sample stabilize with only cement.

Generally, the work of stabilizing agent in treatment process of expansive soils is to reinforce the bounds between the grains/particles or filling the pore spaces. Soil stabilization using chemical methods can result in the alteration and improvement of the geotechnical properties of the soil for better workability and constructability.

2.3: Description of Study Area

The study area is within the Awka area and the towns include Nise, Nibo, Enugwu Agidi, Akpugoeze, Amansea, Ufuma, Ugwuoba and Umunze. It lies between latitude 5^0 56'N - 6^0 16'N and longitude 6^0 59'E - 7^0 17'E (Figure 1.2). The study area is accessible through; the Awka-Onitsha road, Awka-Enugu express road and Ekwulobia-Umunze road. Awka town is bounded with Nibo in the south west, Mgbakwu and Okpuno in the north east and Umuawulu, Isiagu and Ezinato in the South east. Awka is predominantly a low lying region on the western plain of the Mamu River and lies on an elevation of 333 meters above sea-level. The major topographic features in the region are the cuesta. The locations, coordinates, elevation above mean sea level, geological Formations and soil types of the sampled areas are below (Table1.1).

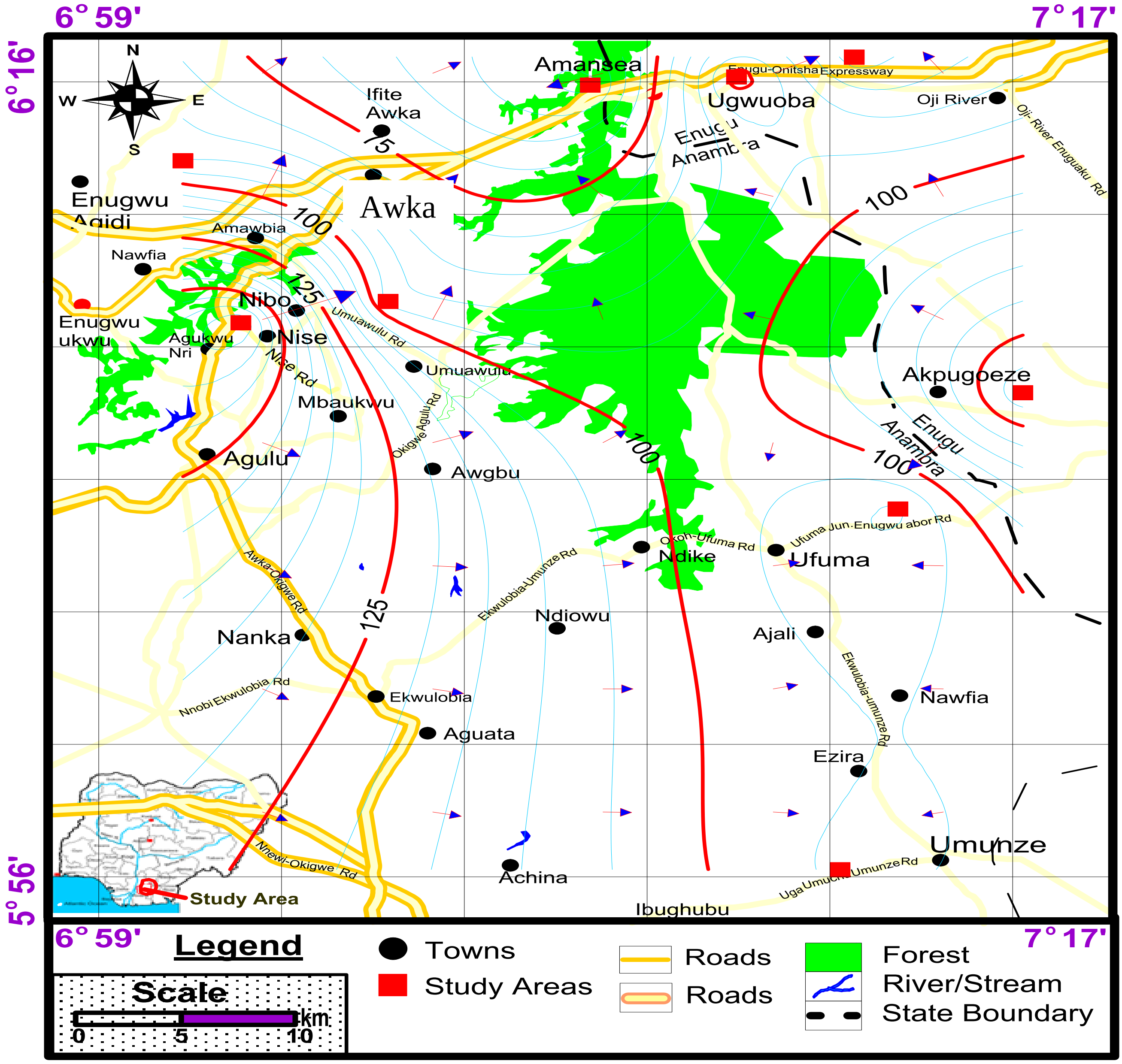

Figure 2.1: Topographic/Location Map of the Study Area (Digitized Google-earth Imagery, 2017)

Table 2.1: Towns/locations, Geologic Formation and soil type of Expansive soils from Awka and Environs, Southeastern Nigeria

S/N	Towns	Location Description	Location Coordinate		Elevation (m)	Geologic formation	Soil Type
			Latitude	Longitude			
1	Nise	Agulu Nise Road	$6^o 9'33''$	$7^o 09'33''$	170	Ameki	Greyish green argillaceous clay
2	Nibo	Nibo-Umuawulu Road	$6^o 10'2''$	$7^o 04'45''$	93	Ameki	Greyish green argillaceous clay
3	Enugwu-Agidi	Amaowbia-Enugwu-Agidi road	$6^o 13'13''$	$7^o 01'23''$	90	Ameki	Greyish green argillaceous clay
4	Ugwuoba	Obudu Ugwuoba near the market square	$6^o 15'7''$	$7^o 10'28''$	103	Imo Shale	Greyish brown clay
5	Akpugoeze	Amagu-Akpugoeze	$6^o 7'58''$	$7^o 15'10''$	134	Imo Shale	Greyish brown clay
6	Ufuma	Ufuma- Enugu Aboh Road near Jossan Rice mill	$6^o 5'20''$	$7^o 13'07''$	89	Imo Shale	Greyish brown clay
7	Umunze	Umunze-Umuchu Road				Imo Shale	Greyish brown clay
8	Amansea	Enugu-Awka Express Road	$6^o 14'56''$	$7^o 08'04''$	53	Imo Shale	Greyish brown clay

2.3.1 Physiography and Climate of the Study area

Awka lies within the rainforest zone with two seasonal climatic conditions, which includes rainy season and the dry season. Awka is characterized by the annual double maxima of rainfall with a slight drop in either July or August known as dry spell/August break. The annual total rainfall is about 1,450mm concentrated mainly in eight months of the year with few months of relative drought (Nigeria Metrological Agency, 2007).

The mean temperature of the study area is 27^0C with daily minimum and maximum temperature ranges of about 22^0C and 34^0C respectively. It has a relative humidity of 80% at dawn. (Source: Hydrometeorological department, Awka).

The study area has a rainforest type of vegetation that comprises different species of tall forest trees, shrubs, with thick undergrowth as well as numerous climbers and grasses. The typical trees are deciduous in nature, such trees are palm trees, raffia palm, iroko trees, oil bear trees and gravelina trees. Oil palm trees and raffia palm are the most common and they are not deciduous in nature.

2.3.2 Geology of the Study Area

The dominant sedimentary rocks in the study area are the Imo Shale and Ameki Formations. The Imo Shale formation ranges from Paleocene to early Eocene in age (Amajor, 1984; Reyment, 1965). It outcrops on the plane of the Mamu River and the formation is of shallow marine environment (Nwajide, 1990). It consists of thick clayey shales, fine textured, occasional clay ironstones sandstone beds in which carbonized plants remains may occur (Nwajide & Reijers, 1996; Kogbe, 1989). Wilson (1925) observes that carbonized plant remains may be locally common and the Formation becomes sandier towards the top where it may consist of alternating bands of sandstone and shale. It has Ebenebe sandstone as its sand member in the study area.

The Ameki Formation and its lateral equivalent Nanka Sands (Okeke & Igboanua, 2003; Ezeigwe, 2005; Nwajide, 1979) were laid down in the early to middle Eocene (Reyment, 1965; Berggren, 1960; Adegoke, 1969). Its rock types are mainly sandstone, calcareous shale with thin limestone bands (Reyment, 1965; Arua, 1986). Outcrops of the sandstone occur at Abagana and Nsugbe, where they are generally quarried in commercial quantity. Nanka sands outcrop mainly at Nanka, Ekwuluobia and Agulu towns in the study area. The regional stratigraphic sequence of the study area is shown below (Table 1.2).

Table 2.2: General regional stratigraphy of Southeastern Nigeria (Modified from Mode 2004; Reyment, 1965; Offodile, 1975)

	Age	**Formation**	**Lithology**
	Recent	Recent sediments	Alluvium/Deltaic Plains
Tertiary	Miocene Recent (5-23 m.y.)	Benin Formation	Unconsolidated sandstone with lenses of clay
	Oligocene Miocene (23-34 m.y.)	Ogwashi Asaba Formation	Unconsolidated sandstone, mudstone, clays and lignite seams
	Eocene (34-38 m.y.)	Ameki Formation	Grey to green argillaceous sandstone, shale and limestone unit
	Paleocene (56-66 m.y.)	Imo shale Formation	Blue to dark grey shales and subordinate sandstone members (Umunna and Ebenebe)
Upper Cretaceous	Maastrichtian(65-72 m.y.)	Nsukka Formation	Alternating sequence of shale, sandstone and coal seams
		Ajali Formation	Friable sandstones with iron stains
		Mamu Formation	Sandstones, shale, siltstone with coal seams

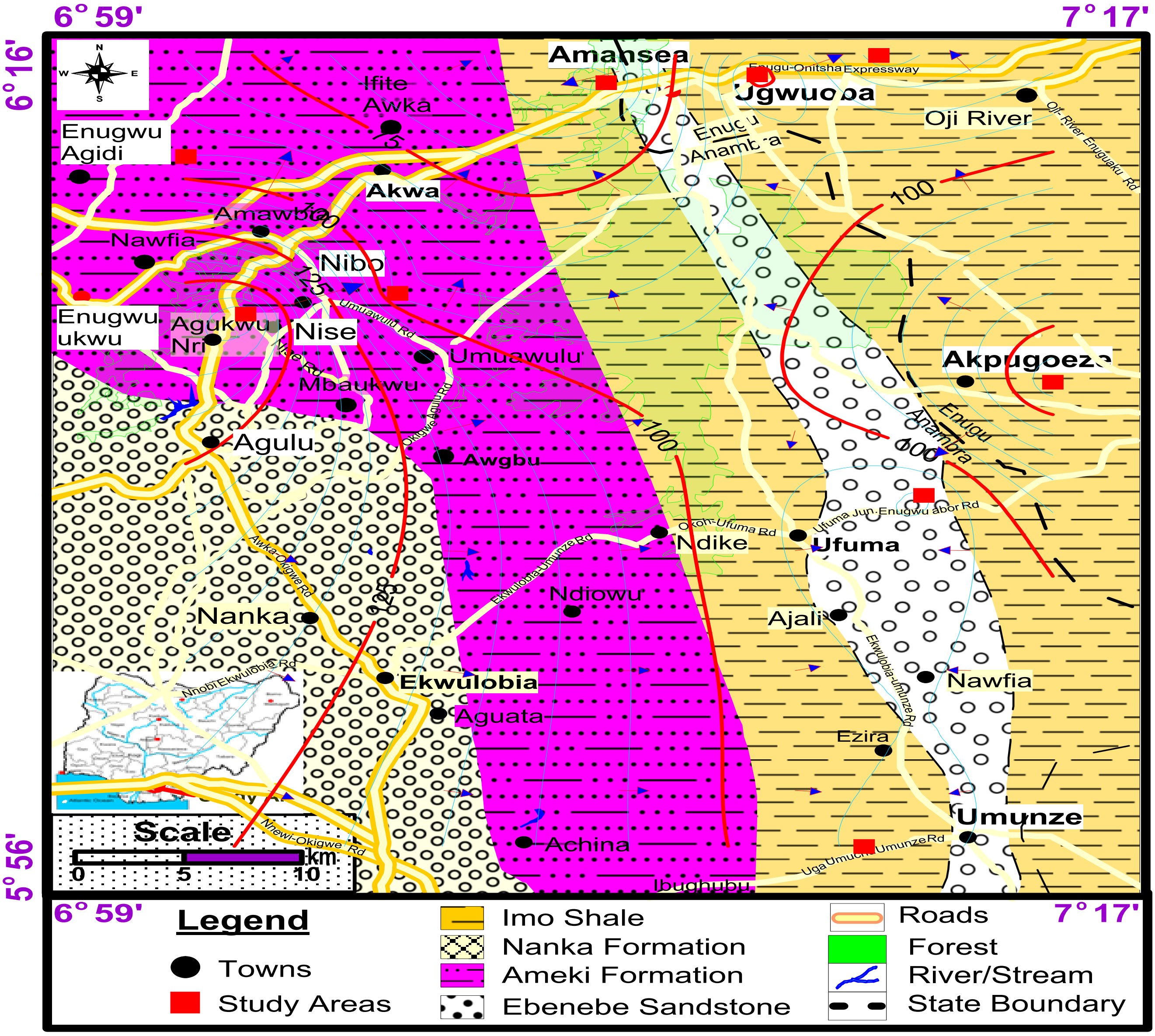

Figure 2.2: Geologic Map of Awka and Environs (modified from Okeke & Igboanua, 2003)

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials Used for the Study

The materials used for the present study include Portland cement, hydrated lime and expansive soils derived from Awka and environs, Southeastern Nigeria. The equipment used for sample collection in the field includes:

I. **Global Positioning systems (GPS)**: This was used to get coordinates of sampled point, for easy identification and location on the map.

II. **Geologic maps**: Base maps of the study area were used for easy location of towns and villages, and previously sampled points by previous researchers.

III. **Calibrated hand auger**: This was used to collect soil samples at depths of 0.5m-1m for analyses in the laboratory.

IV. **Zip-lock nylon bags**: Used to collect samples whose natural moisture content needs to be preserved for immediate analyses in the laboratory.

V. **Camera**: Used for taking pictures at exposures or areas that shows the effect of expansive soil on engineering structures such as roads and buildings.

VI. **Masking tape and ink marker**: It is used to label the samples appropriately indicating the location for easy identification.

Other equipments used in the field include shovels, geologic hammer, tapes and cutlass

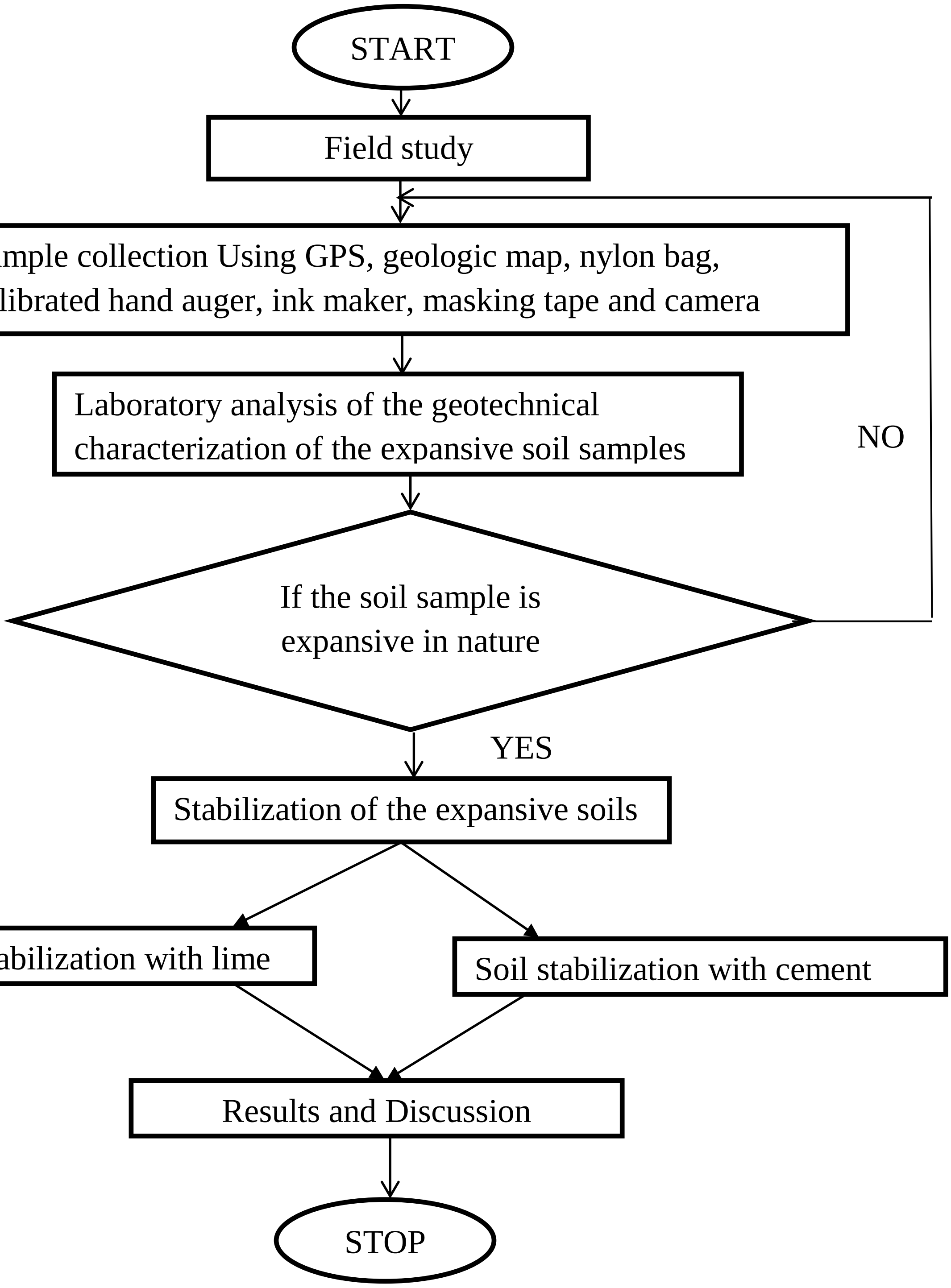

Figure 3.1: Flow chat of the study

3.2: The Field Study

The field studies were carried out to identify the presence of suspected expansive soils and their destructive effects on engineering structures like buildings, roads and drainage facilities in the study area. A total of eight (8) soil samples were collected from different locations of the expansive soils derived from Ameki Formation and Imo Shale Formation in the study area. The soil sampling strictly followed standard procedure as specified in the British Standard Institution (BSI) 1377 (1990), US Bureau of Reclamation (USBR) (1963), Spangler & Handy (1973). The expansive soil were collected with the aid of a calibrated hand auger from the exposed outcrop in the study area and geo-referenced using the GPS and carefully sealed in a clean polythene bag in order to prevent loss of moisture. The soil samples were sent to the laboratory within 24 hours of collection for further analysis. The expansive soil samples were collected in the field between July, 2017 and January, 2018.

3.3: Laboratory/ Geotechnical Analyses

Different geotechnical properties of soils such as consistency limit, specific gravity, density, compaction, particle size analysis, California bearing ratio test etc. have different effects on engineering structures (dam, underground utilities, highway, buildings, bridges etc.).

The geotechnical test was employed to obtain information on the physical properties of the sediments. The laboratory test were conducted at the Arab

Contractor Ltd and it was performed in accordance with the General Specification Roads and Bridges, volume ll, Revised 1997, Government of federal Republic of Nigeria and further classified using American Association of State Highways and Transportation Officials (AASHTO), American Society for Testing and Materials (ASTM) and British standard institution (BSI) 1377 (1990). The geotechnical tests carried out in the laboratory for determination of the swelling and shrinkage behavior include the following:

3.3.1: Natural Moisture Content of the Soil

Soil moisture content is the amount of water that is trapped within the soil mass. It can also be defined as the ratio of the weight of water to weight of the solids in a given mass of soil and this ratio can be expressed in percentage. Moisture content of a soil gives clear information about the state of soil in the field. This trapped moisture in a soil can be removed by oven drying the soil sample at a temperature of 105^0C or using a speedy moisture tester. In this study, a speedy moisture tester was employed during the analysis to determine the amount of moisture trapped in the soil sample. Speedy moisture tester is a portable system for measuring the moisture content of a wide range of materials including soils, aggregates, dust and powder. The system consists of a rugged plastic case containing a low-pressure vessel fitted with a pressure gauge and electrical scale and accessories.

Moisture content, W= $\frac{M_W}{(M_D)} \times 100\%$ ———————————— (3)

Where, M_W= mass of water

M_D= mass of dry sample

3.3.1.1: Equipment and procedures

The equipment used includes; 50g of soil sample, Calcium carbide gas pressure, moisture (speedy) tester, Electrical scale, Two 1 1/4" steel balls, Calcium carbide (reagent), Reagent scoop, Cleaning brush and cloth

Figure 3.2: Speedy moisture tester and an electrical scaling system

The moisture content measurements of the studied soils were determined by mixing a weighed sample (150g) of the soil with a reagent (calcium carbide) in the sealed pressure vessel. The calcium- carbide added reacts chemically with the moisture in the soil sample to produce acetylene gas. This acetylene gas produced will increase the pressure within the vessel and as the pressure is increases, the vessel are proportional to the amount of water in the sample, then the moisture content can be read directly from the calibrated pressure gauge.

3.3.2 Activity of Clay (A)

Activity of clay is the measure of contribution of clay minerals to plasticity. The activity of clay of the studied soils was calculated using a known value of plasticity index (PI) divided by the percent of clay sized particles (less than 2µm) present. Different types of clays have different specific surface area which controls how much wetting is needed to change the clay from one state to another such as across the liquid limit or plastic limit. From the activity one can predict the dominant clay type present in a soil sample. The activity of clay of soils can be represented mathematically as

$$A_c = \frac{\text{PI}}{\text{Percentage of Clay}} \qquad \text{———————————} \qquad (4)$$

Where,

A_c = activity of clay

PI = plasticity Index

3.3.3. Atterberg Limit Test (Consistency Limits)

Consistency means the relative ease with which a soil can be deformed which has to do with the relationship or interaction with water. Water content affects the consistency of a fined-grained soil and when the water content of a fined-grained soil is gradually reduced, slurry causes the soil to pass from liquid state to a plastic state, from the plastic state to a semi-solid state, and finally to the solid state. This water contents variation state differs for different soils. The water contents that correspond to these changes of state are called the Atterberg limits. The water contents corresponding to transition from one state to the next are known as the liquid limit, the plastic limit and the shrinkage limit (Kaniraj, 1988).

The Atterberg or consistency limit test was to determine the liquid limits (LL), plastic limit (PL) and plasticity index (PI) of the studied soils. The test was performed in accordance with ASTM D4381 method.

The following apparatus are employed in liquid limit test: mortar and pestle, an electric oven, 13SNo 425 sieves, a grooving tool, glass plates, weighing balance and numbered specimen container.

The sample of soil was dried and disaggregated in a mortar using a rubber pestle and the disaggregated soil sample was then sieved using the no. 425 sieve. Only the material passing the sieve was used for the test and the sieved soil sample was then soaked in distilled water to form a paste while reserving a portion of the dry sieve materials.

The apparatus include dry soil sample, agate mortar, sieve 425microns, non-porous glass plate, Spatula, Casagrande and groove tool, linear shrinkage mould, wash bottle or aspirator, measuring tape, and oven.

Figure 3.3: Casagrande tools

3.3.3.1: Liquid Limit (LL)

The liquid limit of a soil sample is defined as the water content where the behavior of a clayey soil changes from plastic to liquid. The transformation from plastic to liquid is gradual over a range of water content and the shear strength of the soil is not actually at the liquid limit. The dry soil mass, passing sieve 425microns was mixed with water from the wash bottle and stirred to make a paste, then scooped and placed in the metal portion of the Casagrande and a groove is made down the center with a standardized grooving tool of about 13.5mm (0.53inch) width, the cup is repeatedly dropped 10mm onto a hard rubber base at a rate of 120 blows per minute, during which the groove closes up gradually as a result of the impact. The number of blows for the groove to close was recorded. The test is usually ran at several moisture contents and the moisture content at which it takes 25 blows to close the groove is interpolated from test results. The moisture content at which it takes 25 blows of the metal cup to cause a groove to close over a distance of 13.5millimeter (0.53in) is known as the liquid limit. The liquid limit test is performed by ASTM standard test method D4318. The test allows the running of the test up to 20 to 30 blows if it is required to close the groove.

3.3.3.2: Plastic Limit (PL)

The plastic limit is defined as the moisture content where the threads break apart at a diameter of approximately 3.2mm when clayey soils are rolled. A soil is considered non plastic if the thread cannot be rolled out down to 3.2mm at any moisture possible. The test is determined by rolling out a thread of fine portion of a clayey soil passing sieve 425microns after making a paste from the liquid limit test, on a flat, non-porous glass plate. If the soil is at moisture content where its behavior is plastic, this thread will retain its shape down to a very narrow diameter. The sample can be remolded and test repeated. As the moisture content falls due to evaporation, the thread will be forced to break apart when soil is still at a larger diameter.

3.3.3.3: Plasticity Index (PI)

The plasticity index is a measure of the plasticity of soil. The plasticity index is the size of the range of water content where soil exhibits plastic properties. The plasticity index (PI) is the difference between the liquid limit and the plastic limit (PI = LL-PL).

3.3.4: Particle Size Distribution (Sieve Analyses) Test:

Particle size distribution is a test used to assess the particle size distribution of soil mass. The percentage of different sizes of soil particles coarser than 75μ is determined by sieve analysis whereas less than 75μ are determined by hydrometer analysis. The information gotten from the particle size analysis, were used to plot the particle size distribution curves. The particle size distribution curve (gradation curve) represents the distribution of particles of different sizes in the soil mass (Mallo & Umbugadu, 2012). It gives an idea regarding the gradation of the soil i.e. it gives an idea of percentage of clay fraction in the soil so as to compute the activity of clay in the soil. Apparao & Rao (1995) explained that the grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air fields, etc. Raj (2012) stated that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity, and frost susceptibility. Bowles (2012) found that particle-size is one of the suitability criteria of soils for roads, airfield, levee, dam, and other embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used. The susceptibility to frost action in soil, an extremely important consideration in colder climates, can be predicted from the particle-size analysis. Very fine soil particles are easily carried in suspension by percolating soil water and under drainage systems are rapidly filled with sediments unless they are properly surrounded by a filter made of appropriately graded granular materials. The proper gradation of this filter material can be predicted from the particle-size analysis. Particle-size of the filter materials must be larger than the soil being protected so that the filter pores could permit passage of water but collect the smaller soil particles from suspension. According to Dafalla (2013), sand shape whether rounded, sub rounded or angular will affect the shearing strength of soil. Angular grains provide more interlock and increased shear resistance. The gradation and size of the sand affect the shear resistance. Porosity and spaces available for clay within the sand is an important while considering the mixtures of clays and sands. Particles size analysis can be performed on any type material including sands, crushed rock, clays, granite, feldspars, coal, etc.

The apparatuses used in carrying sieve analysis include soil sample, sieve set and shaker, weighing balance with accuracy of 0.001g, and graph sheet.

A representative soil mass is weighed after wet sieving and poured into the top sieve with the largest opening and decrease in mesh size downwards, the last pan is called the receiver. The column is placed on a mechanical shaker, which shakes the sample for about 10mins. After shaking is completed, the material retained on each sieve is weighed. The weight of each sample retained on each sieve is then divided by the total weight to give a percentage retained on each sieve. The size of each sieve is then analyzed to get the cutoff point or specific size range.

The result of the test is provided on a graphical form which will aid in the identification of the gradation of aggregate. The procedure for performing the test is outlined in the American society for Testing and Materials (ASTM). The results are presented in a graphical form of percentage passing against the sieve size. The percentage of aggregate passing through each sieve is first found by percent retained in each sieve by the equation

$$\% \, soil \, retained = \frac{Weight \, of \, soil \, retained}{total \, soil \, weight} \times 100\% \qquad \text{———————}(5)$$

To find the cumulative percent of aggregate retained in each sieve, add up the total amount of aggregate retained in each sieve and amount in previous sieve. The cumulative percent passing of the aggregate is found by subtracting the percent retained from 100%

% cumulative = 100% - % cumulative

% passing sieve 75µm = Total % retained (100) – % retained on each sieve

The values are then plotted on a graph sheet with cumulative percent passing on the y-axis and logarithmic sieve size on the x-axis.

Figure 3.4: Diagram of sieving equipment and digital weighing machine

3.3.5: Linear Shrinkage (LS)

Shrinkage limit is the water content where further loss of moisture will not result in any more change in volume. The test is determined by the ASTM international D4943. A dry soil mass passing sieve 425microns is mixed with water to make a paste with the palette knife or spatula. The linear shrinkage mould was cleaned and little grease is applied to its inner wall, the soil paste was then scooped into the shrinkage mould and allowed to dry on its own for more than 24 hours before oven drying at 105^{0}C to remove remaining moisture and no further reduction in volume can be noticed.

After no further reduction in volume can be noticed, then the linear shrinkage of the soil was calculated using the formula below.

$$\text{Linear Shrinkage} = \frac{Longitudinal\ shrinkage\ of\ specimen}{Length\ of\ mould} \times 100 \qquad (6)$$

3.3.5.1: Sample procedure

i) Preparation of mould:

The linear shrinkage mould was cleaned and dried. After drying, little silicone grease was applies to the inner surfaces so as to prevent the soil from sticking.

ii) Preparation of sample:

150g of air dried soil passing the 425µm sieve was used and the proportion of the original sample passing the 425µm sieve was recorded. The soil was placed on the glass plate and mixed thoroughly with distilled water. The mixing continued until the soil becomes smooth and homogeneous paste.

iii) Placing the mould:

The Paste was placed in the mould avoiding the trapping of air as far as possible so that the mould is slightly over filled. The mould was tapped gently on the bench to remove air pockets. Palette knife was used to level off the top edge of the mould.

iv) Drying**:** The mould was left exposed to the air for some times so that the soil can dry slowly. The soil specimen was then taken to an oven set at about 45°C to 60°C and then observed from time to time. When it was observed that the soil sample on the mould has shrink away from the walls of the mould, the temperature of the oven was gradually increased to 105°C to compute the drying.

v) Measurement of length:

The sample in the mould was allowed to cool and then the length of the bar soil was measured and recorded using a veneer caliper. The linear shrinkage of a sample is reported as the percentage of its original length.

3.3.6: Specific Gravity Test (Gs):

This is the ratio of the weight in air of a given volume of a material at a standard temperature to the weight in air of an equal volume of distilled water at the same stated temperature. This test was determined using pycnometer/density bottle method. Specific gravity of fine-grained soil solids is an important soil grain property used in the computations of other quantities such as void ratio, degree of saturation, soil density, sedimentation analysis, compaction and consolidation analysis. This test was performed in accordance with BS 1377 part 2 (1990) and ASTM D-854-10 (2010). The apparatus used include; (Density bottle), Dried soil, Sieve 4.75mm, Glass rod, Oven (temperature 105^{0}C to 110^{0}C), Weighing balance with an accuracy of 0.001g, Riffle box, Spatula.

3.3.6.1: Sample procedure

A dried soil sample of about 400-500g was taking and grounded to pass through a 2mm IS sieve. The soil sample was riffled to obtain an even sample. The sample was oven-dried at a temperature of 105^0C to 110^0C.

An empty pycnometer bottle was weighed (W_1) then about 200g to 300g of dried soil sample is transferred into the pycnometer and weighed with the pycnometer (W_2). The sample was covered with air-free distilled water from the wash bottle and screw on the cap live for 2 to 3 hours for soaking and to remove the entrapped air then gently stir with a glass rod to remove any further entrapped air then weigh (W_3). The sample in the pycnometer was emptied and the pycnometer was filled with distilled water and weigh (W_4). The test was performed repeatedly for the remaining samples.

Calculations $Gs = \frac{W_2 - W_1}{[(W_4 - W_1) - (W_3 - W_2)}$ ________________ (7)

Where,

W_1=empty weight of the pycnometer

W_2=weight of the pycnometer + oven dried soil sample

W_3= weight of the pycnometer + oven dried soil sample + water

W_4= weight of the pycnometer + water

Figure 3.5: A pycnometer with an electrical weighing balance

3.3.7: Compaction Test

Soil compaction is one of the most important aspects in construction because it improves the engineering properties of the fills so as to as achieve a certain value of dry unit weight (γ_d). According to Dunn *et al.*, (1980) & Bell (1993), compaction is defined as the artificial rearrangement and closely packing of soil particles using mechanical, electrical or any order means so as to increase the unit weight (density) of soil by forcing soil solid into tighter state to reduce the air void (porosity) while Reynolds (2012) defines it as the densification of soil materials by the use of mechanical energy. Compaction is the processes by which the volume of air in a pavement mixture is reduced by using external force to reorient the constituent aggregate particles. Compaction has been one method of soil stabilization without adding any additives (Ingles & Metcalf, 1973). This make the soil less susceptible to settlement under load, especially repeated loading. The proper compaction of soils is intended to ensure that the compacted soil will reliable and safely withstand loads of various kinds. According to Randrup (1997), soil compaction on construction sites occurs either deliberately when foundations and sub grades are prepared or as an unintended result of vehicle traffic. The more compacted a soil is, the less the porosity i.e. porosity decreases with compaction. The test was performed in accordance with BIS 1377 part2 (1991). A 2.5kg method of compaction was used for this test. The apparatus consist of 2.5kg rammer, a known volume of mould with removable base and a detachable collar. Three kilogram of air dried soil was used for the test and the test was repeated five times for each sample. The moisture content used was between 4-20% of the weight of the sample, and samples were mixed thoroughly before compaction. Three layers of compaction were done for each trial and 27 blows were used to compact each layer. Graphs of dry density (ρ_d), against moisture content were plotted using rock work software to determine the optimum moisture content. Bulk density and dry density were calculated using the following formula:

Bulk density = $\frac{M_2 - M_3}{973}$ (kg/m^3) ———————————— (8)

Dry density, $\rho d = \frac{M_2 - M_1}{1000(1+m)}$ ———————————— (9)

M_1 = mass of mould (g), M_2 = mass of mould and compacted sample (g), 1000 = volume of mould (cm^3), m = moisture content used.

Figure 3.6: Proctor mould for compaction test

3.3.8: California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) was developed by the California State Highways Department, to evaluate the strength of road subgrades. It is widely used in the design of flexible airport and highway pavements and sometimes in the specifications for in the use of earth materials for various construction purposes. It is considered to give reliable results, provided the tests are conducted according to specified conditions. The higher the CBR value, the better the material.

CBR tests were performed on compacted samples in both unsoaked and soaked conditions, following the procedure of Bailey (1976). However, soaking was done overnight (24 hours) in a water-filled bathtub.

The CBR test quantitatively evaluates the inherent strength of a subgrade in order for a road pavement to be designed for a particular strength of subgrade. It is a point load test that can be carried out on almost all kinds of soils, ranging from clay to gravel. The test was performed in accordance with ASTM D1883.

Figure 3.7: Californian Bearing Ratio test Machine

3.3.9: FREE SWELL TEST

Free swell is the increase in volume of fine grained expansive soils without any external constraints on submergence in water. Free swell test was performed on the expansive soil samples to determine the swelling nature/potential of the soil and was performed using IS: 2720 (part 40) 1977.

3.3.9.1: Equipments and Sample Procedure:

The apparatus used includes; 425 micron IS sieve, two graduated glass cylinder with 100ml capacity, Oven, Glass rod, Weighing balance with accuracy of 0.001g,Wash bottle and aspirator, Distilled water, Kerosene and Weighing container.

Two representative oven dried expansive soil samples of about 10g passing sieve 425 micron was used. The two expansive soil samples was poured into each of the graduated glass cylinder of about 100ml capacity and one is filled with kerosene while the second one is filled with distilled water up to the 100ml mark. The entrapped air in the cylinder was removed by gently shaking and stirring the sample with the glass rod. The samples were allowed to settle for about 24hours so that the samples will attain its equilibrium state and the final volume of the expansive soil samples in each of the cylinder recorded.

$$\text{Free swell} = \frac{V_d - V_k}{V_k} \, X \, 100 \, (\%) \qquad \text{————————————} (10)$$

Where,

V_d = volume of the specimen read from the graduated cylinder containing distilled water

V_k = volume of the specimen read from the graduated cylinder containing kerosene

3.3.10: Hydrometer Test

Hydrometer is the an instrument used to measure relative density of liquid and it consists of a cylindrical stem with graduation mark and a bulb at bottom weighted with mercury and is made of glass. In geotechnical studied, hydrometer analysis is used to determine the grain size distribution of a fine grained soil because sieve analysis does not give reliable information on the fine grained soil. A fine grained soil has different particle sizes ranges from 0.075mm-0.0002mm.

The hydrometer analysis is based on stokes law which states that the velocity at which grains settles out of suspension, all other factors being equal, depends on the shape, weight and size of the grain. For a soil, it is assumed that the soil particles are spherical and have the same specific gravity. Therefore, in a soil water suspension the coarse particles settle more quickly than the finer particles. It can be represented mathematically as;

$$V = \frac{1}{18\left[\frac{G_S - G_W}{n}\right] D^2} \qquad \text{———————————— (11)}$$

Where,

V = terminal velocity of soil particle (cm^{-1})

D = diameter of soil particle (m)

G_s= specific gravity of particle

G_w= specific gravity of water

n = viscosity 0f water (g-s/cm^2)

3.3.10.1: Apparatus and Calculations

The apparatus used includes; Hydrometer, Dispersion cup with mechanical stirrer, Two glass jar of 1 litre capacity, Deflocculating agent, Stop watch, Thermometer and Scale

Initial mass of sample, M =60g

Test temperature=

Meniscus correction (cm) =+0.5

Temperature correction (mt) =+1.0

Dispersant correction(X) =3.5

Specific gravity (Gs) = 2.65

Viscosity of water (π)= 0.8909mpas

Column (4) $R_h = R_{h1} + C_m = R_{h1} + 0.5$

Column (5) $H_R = 214 - 4.1R_h$

Column (6) $R = R_h + M_t - X + 1.8$

$R_h + 1.0 - 35 + 1.8$

$R_h - 0.7$

Column (7) $D = 0.005531 \frac{\sqrt{\pi HR}}{(Gs-1)t} = 0.004064 \frac{\sqrt{HR}}{t}$

Column (8) $= \frac{\sqrt{Gs \times R \times 100}}{m(Gs-1)} = 2.7 \times R(\%)$

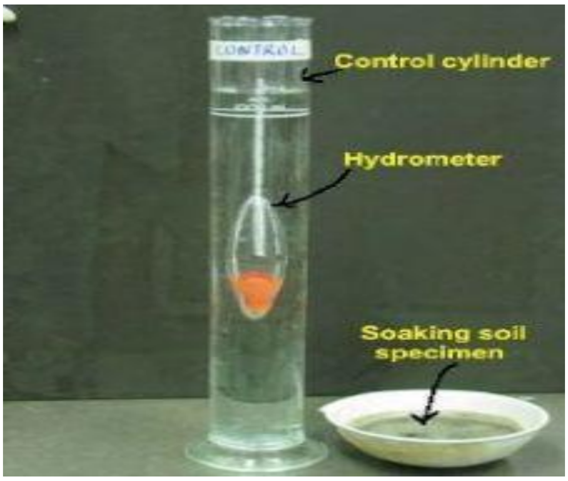

Figure 3.8: Hydrometer Test instrument

3.3.11: BULK DENSITY TEST

Bulk density is an indicator of soil compaction and soil health. It affects infiltration, rooting depth/restrictions, available water capacity, soil porosity, plant nutrient availability, and soil microorganism activity, which influence key soil processes and productivity. It is the weight of dry soil per unit of volume typically expressed in Mg/m^3. Total volume of surface soil is about 50% solids, mostly soil particles (45%), and organic matter (generally < 5%); and about 50% pore space which are filled with air or water. When determining bulk density, the amount of soil moisture must be determined. Available water capacity is the amount of soil moisture available to plants, varies by texture and is reduced when compaction occurs.

3.3.11.1: Apparatus and Procedure

The apparatus used includes; 3 inch diameter ring, Flat blade knife, Scale (0.01g precision), Oven, a Known volume of soil, Sealable bags and marker pen.

The empty ring is weighed and recorded, and the ring is filled with the soil sample and leveled with a flat blade knife and weighed and recorded. The known volume is also recorded. It is represented mathematically as

Bulk density, $\rho b(\%) = \frac{weight\ of\ sample}{volume\ of\ sample} \times 100$ ——————————(12)

3.3.12: DRY DENSITY

The dry density of the expansive soil sample was derived from the known value of natural moisture content (in fraction) and bulk density values. It is represented mathematically as

Dry density, $\rho d(\%) = \frac{\rho b}{1+W_n}$ ——————————(13)

Where,

ρd = bulk density

W_n= natural moisture content

3.4: Cement stabilization and composition

During the 1960s when soil stabilization technology was invented, cement has been used effectively as one of the binding agent in soil stabilization. Portland cement manufactured by Dangote Cement Company was used for the stabilization treatment of the studied soils in other to modify the geotechnical properties of the soils such as swelling potentials, degree of expansion and increase the strength characteristics. The production of this Portland cement according to Lea (1970), is made from calcination of a mixture of a calcareous and argillaceous material at a temperature around 1450^0C and this calcareous substances are of calcium oxide origin usually found in limestone, chalk or oyster shells while the argillaceous substances are of silicate and aluminate origin mostly found in clays, shale and slags (Mamlouk & Zaniewski, 2006). Punmatharith *et al.*, (2010) states that the chemical composition of Portland cement involves both major oxides (CaO, SiO, Al_2O_3 and Fe_2O_3) and minor oxides (MgO and SO_3) and also alkali oxides (K^2O and Na^2O) with inclusion of other compounds (P_2O_5, Cl, TiO_2, MnO_3 etc.).

The chemical reaction involved in soil stabilization with cement is hydration. This process begins when cement is mixed with water and other components for a desired application resulting into hardening phenomena. The setting of cent will enclose soil as glue, but will not have any adverse effect on the structure of the soil (Eurosoilstab., 2002). Some the factors that can affect the stabilization with cement are presence of foreign body, curing temperature, presence of additives, surface of the mixture, water-cement ratio. Soil stabilization with cement is important because it decreases the volume expansion or compressibility, decreases the cohesiveness of the soil (plasticity) and increases the strength characteristics of the soil.

3.5: Lime Stabilization and Composition

Lime is the most widely accepted and controversial chemical treatment used in soil stabilization. Lime stabilization refers to pozzolanic reaction in which pozzolanic materials reacts with lime in the presence of water to a cementitious compound (Sherwood, 1993; Eurosoilstab., 2002). According to Tringale & Mitchell (1979), the lime treatment of soils using both hydrated lime (Ca(OH) 2) and quicklime (CaO) has been used extensively in conjunction with pavement construction, embankment lining and repair, and railroad subgrades. The South Dakota Department of Highways has used lime successfully to control expansive soils on ten Interstate and three primary construction projects. Kelly & Kelly (1973) state that if lime is added and adequately mixed, highly plastic expansive clay can be transformed into a stable, non-swelling mixture. Although lime has been used mainly for the treatment of highly plastic and expansive clays, it can also cause beneficial property changes in soft sedimentary, inactive clays.

Hydrated lime Ca(OH)2 (calcium hydroxide) was used to stabilized the studied soils. The hydrated lime resulted from the reaction between quicklime (CaO) with water.

$CaO + H_2O \rightarrow Ca(OH)_2$ ———————————— (13)

This lime will react with the clay minerals in the soils especially the montmorillonite which has higher plasticity index than the kaolinites. The chemical reaction involved in soil treatment with lime is cation exchange and the two main stages involved in soil stabilization of expansive soil with lime are flocculation and pozzolanic reaction. The flocculation processes occur immediately after adding the lime to the soil with sufficient amount of water (short term reaction). This flocculation method results from cation exchange processes; as the sodium and other cation on the surfaces of the clay minerals exchanged with calcium cations of lime, causing the fine grains of the soil to flocculate forming aggregates which results in the increase of the grain size of the soils(accordingly). The pozzolanic process takes place after flocculation, when there is sufficient lime and water content. When lime react with wet clay mineral (like kaolin, illite, montmorillonite), it increases the pH which favours solubility of siliceous and aluminous compounds. These compounds react with calcium to form calcium silica and calcium alumina hydrates (a cementitious product similar to those of cement paste). Natural pozzolanas materials having silica and alumina like clay minerals, pulverized fly ash and blast furnace slag have great potential to react with lime.

Lime stabilization has been effectively used in geotechnical and environmental applications like encapsulation of contaminations, rendering of backfill (wet cohesive soil), highway capping, slope stabilization and foundation improvement such as in use of lime pile or lime-stabilized soil columns (Ingles & Matcalf, 1972). Presence of sulphur compounds and organic materials can affect/inhibit the lime stabilization process. For instance, sulphur (gypsum) will react with lime and cause it to swell which in turn affects the strength of the soil.

Table 3.1: Chemical composition of lime

Component oxides (%)	Lime
SiO_2	0.01
Al_2O	0.01
Fe_2O_3	0.11
CaO	65.25
MgO	0.50
K_2O	0.01
Na_2O	0.01
CO_2	4.5

Figure 3.9: Representive expansive soil sample mixed with hydrated lime set for stabilization

Figure 3.10: Representive expansive soil sample mixed with Dangote cement set for stabilization

CHAPTER FOUR

RESULT AND DISCUSSION

4.1. Results of the analysis

The results of the index property parameters of the analysis revealed generally that the natural moisture content ranges between 24.40 to 30.60%, specific gravity from 2.62 to 2.71, dry density from 0.82 to 1.03Mg/m^3 and bulk density 1.04 to 1.29Mg/m^3, liquid limit from 63.10 to 77.40%, plastic limit from 29.30 to 37.65%, plasticity index from 33.55 to 46.45%, linear shrinkage from 10.70 to 20.00%, clay content from 24.29 to 30.79%, activity from 0.98 to 1.55 and free swell from 53.00 to 71.00% (Table 4.1.1) while the strength characteristics analysis like optimum moisture content ranges from 17.70 to 27.00%, maximum dry density from 1.40 to 1.75Mg/m^3, unsoaked CBR from 10.00 to 18.00% and soaked CBR from 2.00 to 3.00% (Table 4.1.2). The relative proportion of gravel, sand, silt and clay particles sizes of the studied soils are also presented below and the summaries of the Atterberg limit test, Casagrande plasticity chart, swelling potentials and degree of expansion classification is presented below (Table 4.1.3-4.1.5). The results of the sieve and hydrometer analysis of the soil samples and the guidelines for swelling potential, degree of expansion and strength characteristics used in classifying the result of the soils is shown (Appendices). The results of the effects of cement and lime stabilization on the engineering properties of the expansive soils derived from

Imo Shale Formation and Ameki Formation are shown below (Tables 4.1.6.-4.1.9).

Table 4.1.1: Result of the Geotechnical index properties of Expansive Soils derived from Ameki Formation and Imo Shale Formation in Awka and environs, Southeastern Nigeria.

Parameter	Geologic Formations										Federal Ministry of Works Standard (1997), ASTM D792(2008)
	Ameki Formation				Imo Shale						
	Nise	Nibo	Enugwu -Agidi	Average	Ugwuoba	Akpugoeze	Ufuma	Umunze	Amansea	Average	
Liquid limit (%)	63.10	65.00	66.80	64.97	75.50	76.40	74.40	68.40	77.40	74.42	< or =35%
Plastic limit (%)	29.30	30.95	33.25	31.17	32.50	29.95	34.30	30.95	37.65	33.07	-
Plasticity index (%)	33.80	34.05	33.55	33.80	43.00	46.45	40.10	37.45	39.75	41.35	< or =12%
Linear Shrinkage (%)	17.90	20.00	10.70	16.20	10.70	17.90	14.80	13.60	15.70	14.54	< or =8
Natural moisture content (%)	26.80	30.60	25.60	27.67	28.00	28.80	25.00	23.40	24.40	25.92	-
Specific Gravity	2.64	2.62	2.65	2.64	2.64	2.71	2.68	2.63	2.70	2.67	0.94
Clay fraction (%)	28.00	32.42	30.79	30.40	29.29	24.29	26.66	26.90	30.37	27.50	
Activity of clay	1.21	0.98	1.16	1.12	1.47	1.55	1.50	1.39	1.31	1.44	-
Bulk Density (Mg/m^3)	1.13	1.14	1.04	1.10	1.21	1.05	1.29	1.22	1.16	1.19	-
Dry Density (Mg/m^3)	0.89	0.84	0.83	0.85	0.95	0.82	1.03	1.00	0.93	0.95	-
Free Swell (%)	56.00	58.00	53.00	55.67	70.00	63.00	71.00	68.00	62.00	66.80	<50%
% passing sieve 200	80.70	84.90	63.00	76.20	86.70	94.80	72.60	62.10	75.10	78.26	<or=35%

Table 4.1.2: Result showing the strength characteristics of expansive soils from derived Ameki Formation and Imo Shale Formation in Awka and environs, Southeastern Nigeria.

Parameter	Geologic Formations										Federal Ministry of Works Standard (1997)
	Ameki				Imo Shale						
	Compaction test				CBR test						
	Nise	Nibo	Enugwu-Agidi	Average	Ugwuoba	Akpugoeze	Ufuma	Umunze	Amansea	Average	
MDD (Mg/m^3)	1.67	1.49	1.63	1.59	1.56	1.40	1.67	1.59	1.75	1.58	>1.76
OMC (%)	24.00	27.00	17.70	22.90	21.30	23.20	24.30	22.20	18.10	21.82	-
UnSoaked CBR (%)	11.50	17.30	18.10	15.63	10.50	10.20	15.40	14.80	12.00	12.58	>40
Soaked CBR (%)	1.90	2.40	1.40	1.90	2.60	3.30	2.20	2.70	2.90	2.57	>15

Table 4.1.3: Percentage of Clay, Plasticity Index and Activity of Clay

S/N	Location	% Clay	Decimal	PI	Activity of Clay
1	Umunze	26.90	0.269	37.45	1.39
2	Amansea	30.37	0.304	39.75	1.31
3	Ufuma	26.66	0.267	40.10	1.50
4	Ugwuoba	29.29	0.293	43.00	1.47
5	Akpugoeze	24.29	0.243	37.65	1.55
6	Nise	28.00	0.28	33.80	1.21
7	Nibo	32.42	0.324	31.75	0.98
8	Enugu Agidi	30.79	0.308	31.38	1.02

Table 4.1.4: Summary of the Atterberg limit test (Liquid and Plastic Limits and Plasticity Index) and Casagrande plasticity chart classification

Formations	Locations	LL (%)	PL (%)	PI (%)	Casagrande classification
Imo shale	Amansea	77.40	37.65	39.75	High
	Akpugoeze	76.40	29.95	46.45	High
	Ugwuoba	75.50	32.50	43.00	High
	Ufuma	74.40	34.30	40.10	High
	Umunze	68.40	30.95	37.45	High
Ameki	Enugu Agidi	66.80	32.05	31.38	High
	Nibo	65.00	30.95	34.05	High
	Nise	63.10	29.30	33.80	High

Table 4.1.5: Summary of Swelling Potential and Degree of Expansion of Expansive Soils derived from Awka and Environs, Southeastern Nigeria

s/no	Sample location	Geologic Formation	LL(%)	PI(%)	LS (%)	Activity (%)	Free swell(%)	Degree of Expansion classification using free swell values (Dawson, 1956)	Degree of Expansion classification using linear shrinkage (Attimeyer, 1956)	Swelling Potential classification using Activity value (Skempton,1953)	Swelling Potential classification using plasticity index value classification (Ola, 1981)	Swelling Potential classification using liquid limit (Holtz & Gibbs, 1956)
1	Nise	Ameki	63.10	33.80	17.90	1.21	56.00	High	Critical	Medium	High	High
2	Nibo	Ameki	65.00	34.05	20.00	0.98	51.00	High	Critical	Medium	High	High
3	Enugwu-Agidi	Ameki	66.80	31.38	10.70	1.16	53.00	High	Critical	Medium	High	High
4	Ugwuoba	Imo Shale	75.50	43.00	10.70	1.47	71.00	High	Critical	High	Very high	Very high
5	Akpugoeze	Imo Shale	76.40	46.45	17.90	1.55	63.00	High	Critical	High	Very high	Very high
6	Ufuma	Imo Shale	74.40	40.10	14.80	1.50	61.00	High	Critical	High	Very high	Very high
7	Umunze	Imo Shale	68.40	37.45	13.60	1.39	65.00	High	Critical	High	Very high	Very high
8	Amansea	Imo Shale	77.40	39.75	15.70	1.31	62.00	High	Critical	High	Very high	Very high

Table 4.1.6: Effects of cement stabilization on geotechnical properties of expansive soils derived from Imo Shale Formation (Akpugoeze) in Awka and environs, Southeastern Nigeria.

S/N	% cement added	Atterberg limit (%)			LS (%)	Compaction test		CBR test (%)	
		LL	PL	PI		MDD(Mg/m^3)	OMC (%)	Unsoaked	Soaked
1	0	76.40	29.95	46.45	17.90	1.40	23.20	10.20	3.30
2	2	71.10	30.30	40.80	15.10	1.59	21.10	23.00	16.10
3	4	65.10	30.95	34.15	12.90	1.75	18.20	42.10	34.30
4	6	60.00	31.60	28.40	8.30	1.65	20.40	39.00	30.00
5	8	42.50	32.90	9.60	5.30	1.50	22.60	35.10	27.50
6	10	54.70	31.45	23.25	8.40	1.45	23.90	30.70	21.80

Table 4.1.7: Effects of lime stabilization on geotechnical properties of expansive soils derived from Imo Shale Formation (Akpugoeze) in Awka and environs, Southeastern Nigeria.

S/N	% lime added	Atterberg limit (%)			LS (%)	Compaction test		CBR test	
		LL	PL	PI		MDD(Mg/m^3)	OMC (%)	Unsoaked	Soaked
1	0	76.40	29.95	46.45	17.90	1.40	23.20	10.20	3.30
2	2	67.10	31.15	35.95	13.20	1.30	22.10	19.00	12.00
3	4	54.70	32.00	22.70	8.40	1.24	19.10	30.30	21.40
4	6	39.10	33.70	5.40	4.30	1.06	18.40	39.85	28.10
5	8	46.95	31.80	15.15	6.60	1.15	21.10	33.50	24.20
6	10	54.00	30.50	23.50	8.00	1.32	24.40	29.40	19.78

Table 4.1.8: Effects of cement stabilization on geotechnical properties of expansive soils derived from Ameki Formation (Nibo) in Awka and environs, Southeastern Nigeria.

S/N	% cement added	Atterberg limit (%)			LS (%)	Compaction test		CBR test (%)	
		LL	PL	PI		MDD(Mg/m^3)	OMC (%)	Unsoaked	Soaked
1	0	65.00	30.95	34.05	12.00	1.49	27.05	17.04	2.41
2	2	62.50	31.30	31.20	11.30	1.57	23.03	27.24	13.60
3	4	58.10	32.00	26.10	9.50	1.70	17.90	45.20	27.56
4	6	50.60	32.80	18.20	7.10	1.61	21.25	34.70	19.85
5	8	41.30	33.10	8.20	4.20	1.58	26.60	25.47	15.25
6	10	53.40	31.10	22.30	7.10	1.52	28.10	18.00	11.40

Table 4.1.9: Effects of lime stabilization on geotechnical properties of expansive soils derived from Ameki Formation (Nibo) in Awka and environs, Southeastern Nigeria.

	% lime added	Atterberg limit (%)			LS (%)	Compaction test		CBR test (%)	
S/N		LL	PL	PI		MDD(Mg/m^3)	OMC (%)	Unsoaked	Soaked
1	0	65.00	30.95	34.05	12.00	1.49	27.05	17.04	2.41
2	2	60.80	31.70	29.10	10.10	1.35	25.51	22.40	11.00
3	4	47.60	32.10	15.50	5.90	1.20	21.80	27.90	17.25
4	6	39.95	33.00	6.95	2.60	1.13	18.23	40.40	24.57
5	8	51.60	31.80	19.80	5.70	1.24	21.80	29.10	18.00
6	10	58.20	30.10	28.10	8.95	1.38	25.00	21.30	13.30

4.2 Discussion of Results

4.2.1: Field Observations

Different geology was identified and physical performance of existing structures examined. The development of desiccation crack on the ground surface was apparent during the dry period. According to Day (1999) the degree of potential swell determines the size of the cracks. Large or more frequent arrangements of polygonal cracks indicated high potential for swelling while low shrink/swell means that potential for shrinkage is low. Observed locations of expansive clay soils revealed that they become very sticky and plastic when wet and adhere to the soles of shoes and tires of vehicles. They are also relatively easy to roll into small threads (Lucian, 2011).

Figure 4.2.1: Mud cracks or desiccation cracks observed on the study area

4.2.2: Liquid Limit Test

The liquid limit of the studied soils for Imo Shale Formation ranges from 68.40-77.40% with an average of 74.42% while that of Ameki Formation ranges from 63.10-66.80% with an average of 64.97%. This indicates that all the soil samples analyzed are swelling soil as they all showed liquid limits greater than 60% using Chen (1975) classification for degree of expansiveness. Comparing the liquid limit(LL) and plasticity index(PI) of the studied soils with Holtz & Gibbs (1965) classifications for swelling potentials, the expansive soils derived from Imo Shale Formation have high-very high swelling potentials while those derived from Ameki Formation have high swelling potentials. The values showed that Imo Shale Formation at Amansea has the highest swelling potential, while the Ameki Formation at Nise shows the lowest swelling potential. This may be attributed to the fact that the unit at Nise has the presence of sandstone in it.

In general, liquid limit of all the soils indicates high compressibility (Venkatramaiah, 2006), high degree of expansiveness (Chen, 1975), high to very high swelling potential (Hortz & Gibbs, 1956), a typical clay soils (BS 1377, 1990). Using Federal Ministry of Works and Housing vol.11 (1997) specification for liquid limit value, all the studied soils are poor subgrade and sub base materials and as such not suitable for building foundation.

Table 4.2.1: Chen (1975) and Holtz & Gibbs (1965) classifications

Swelling potential	Chen (1975)	Formations	Holtz & Gibbs (1965)	Formations
Low	<30	-	<35	-
Medium	30-40	-	35-50	-
High	40-60	-	50-70	Nise, Nibo and Enugu Agidi (Ameki Formation); Umunze (Imo Shale Formation)
Very high	>60	Nise, Nibo and Enugu Agidi (Ameki Formation); Umunze, Akpugoeze, Ufuma, Amansea, and Ugwuoba (Imo shale Formation)	>70	Akpugoeze, Ufuma, Amansea, and Ugwuoba (Imo shale)

Table 4.2.2: Federal Government of Nigeria Standard Specification for Roads and Bridges (1997)

Geotechnical properties	Values for sub grade and base course materials
Liquid limit	<30%
Plasticity index	<12%
Linear shrinkage	<8%

4.2.3: Plasticity Index

The plasticity index of the studied soils form Imo Shale ranges from 37.45-46.45% with average of 41.35% while those from Ameki Formation ranges from 33.55-34.05% with average of 33.80%.

Based on Ola (1981) classification of plasticity index, the expansive soils derived from Imo shale Formation have very high swelling potential whereas those derived from Ameki Formation have high swelling potential (Table 4.2.3). The Federal Government of Nigeria specifies that the values of plasticity index of soil should be less than 12% and liquid limit should be less than 30% for the soil to be used as building foundations (Table 4.2.1) but none of the studied soils meet the specifications.

Table 4.2.3: Swelling Potential of the studied soils using Ola (1981) classification

Plasticity Index (PI) Ola (1981)	Swelling Potential	Formations
0-15	Low	-
15-25	Medium	-
25-35	High	Nise, Nibo and Enugu Agidi (Ameki Formation); Umunze (Imo Shale Formation)
>35	Very high	Akpugoeze, Ufuma, Amansea, and Ugwuoba (Imo shale Formation)

4.2.4: Activity of Clay

Activity of clay is the measure of the contribution of clay minerals to plasticity of the soil. It is obtained by dividing plasticity index (PI) with the percent of clay sized particles (less than 2μm) present. This is one of the geotechnical parameters used to estimate for swelling potential. It helps to infers for a particular clay mineral responsible for swelling behaviour of expansive soils.

The activity of clay of the studied soil samples for Imo Shale ranges from 1.31-1.55 with an average of 1.44 while that of Ameki Formation ranges from 0.98-1.16 with an average of 1.12. Skpemton (1953) categorized activity of clay as low activity (<0.75), medium activity (0.75 to 1.25) and high activity (>1.25).

Based on this classification (Skpemton, 1953), the expansive soils derived from Imo Shale Formation have high activity because their values is >1.25 while those soils derived from Ameki Formation have medium activity because they fall between 0.75-1.25. Therefore, the studied soils have medium- high swelling potential using activity of clay parameters.

Table 4.2.4: Activity of clay of the studied soils using Skempton (1953) classification

Activity of clay	Swelling potential	Formation
< 0.75	Low	
0.75-1.25	Medium	Ameki
>1.25	High	Imo Shale

4.2.5: Linear Shrinkage Limit Test

Linear shrinkage is a geotechnical parameter used to determine the degree of expansion of soils. The possibility of recognizing expansive soils with linear shrinkage was proposed by Kantey & Brink (1952). They noted that linear shrinkage greater than 8% indicates expansiveness but Ola (1981) adopted the volume change rating using linear shrinkage established by Attimeyer (1956).

The linear shrinkage of the studied soils for Imo Shale Formation falls within the range of 10.70-17.90% with an average of 14.54% while the soils derived from Ameki Formation range from 10.70-20.00% with an average of 16.20%. The high values of linear shrinkage maybe due to the high plasticity of the soils which proved the soils have the affinity/tendency to swell when in contact with water. This characteristic the studied soils poses course a serious threat to the engineering structures in the study area.

Federal Ministry of Works and Housing (1997), recommends linear shrinkage of <8% as good sub grade and base course material. Attimeyer (1965), classified any soil with linear shrinkage value of <5 as non-critical, 5-8 as marginal while >8 as critical. Based on Attimeyer (1956) classifications, all the studied soils have a critical degree of expansion because their linear shrinkage values were <8 (Table 4.2.5).

Table 4.2.5: Linear Shrinkage (LS) of the studied soils using Attimeyer (1956) classification

Linear Shrinkage (%)	Degree of Expansion	Formation
0 -5	Non-critical	-
5 – 8	Marginal	-
>8	Critical	Ameki and Imo Shale

4.2.6: Free swell result

The free swell value of a soil is one of the parameters used to estimate for the degree of expansion of expansive soils. The free swell of the studied derived soils form Imo Shale Formation ranged from 62.00-71.00% with an average of 66.80% while the soils derived from Ameki Formation ranged from 53.00 to 58.00% with an average of 55.67%. Dawson (1956) classified soil sample with free swell value less than 50% as low degree expansion while those with free swell value greater than 50% as high degree of expansion (Table 4.2.6).

The expansive soils derived from both geological Formations (Imo Shale and Ameki Formation) has high degree of expansive because they have free swell values greater than 50%. The soil derived from Ufuma (Imo Shale Formation) indicates the highest degree of expansion with a value of 71% while soil sample derived from Enugwu-Agidi (Ameki Formation) shows the lowest degree of expansion with a value of 53% which is still above 50%.

Table 4.2.6: Free swell of the studied soils using Dawson (1956) classification

Dawson (1956)	Degree of expansion	Formation
<50%	Low	
>50%	High	Ameki and Imo Shale

4.2.7: Natural moisture content

The natural moisture content of soils has much to say about soil type. Clay soils retain more water than saturated sandy soil because sandy soils are more permeable and porous than clay soils and as such, clay soil holds/ traps more water than sandy soil. The result of the natural moisture content on the studied soils derived from Ameki Formation ranged from 25.60-30.60% with an average of 27.67% while the soils derived from Imo Shale Formation ranged from 23.40-28.80% with an average of 25.92% which indicates that they are both fine grained soils with great affinity to moisture. It also shows low transmisivity and high compressivity from the plot on the Casagrande plasticity chart (Figure 4.2.2). The high affinity for moisture in the studied soils indicates the presence of minerals like montmorillonite, kaolin, illite etc. which attracts water molecules to get attached on the surface of clay soils.

Generally, the studied soils derived from Ameki Formation and Imo Shale Formation attributes that the soils are highly expansive and as such are not suitable for engineering construction. Weltman & Head (1983), classified soils with moisture content value of 5-15% as suitable for engineering construction but none of the studied soils meet the specification.

4.2.8: Bulk Density

The bulk density of a soil depends solely on soil organic matter, soil texture, density of soil mineral (sand, silt, and clay) and their packing arrangement (degree of compaction) and the bulk density of a mineral soil falls within 1.0 and 1.6Mg/m^3. The soils with Loose, well-aggregated, porous and those rich in organic matter have lower bulk density while sandy soils have relatively high bulk density because total pore space in sands is greater than that of silt or clay soils. Bulk density typically increases with soil depth since subsurface layers are more compacted and have less organic matter, less aggregation, and less root penetration compared to surface layers, therefore contain less pore space. Paige-Green (2007) postulated that low bulk density value of <2.6 Mg/m^3 of a construction material is highly vulnerable to weathering and deterioration.

The Bulk Density of the studied derived soils form Imo Shale Formation ranged from 1.05-1.29 Mg/m^3 with an average of 1.19 Mg/m^3 while the soils derived from Ameki Formation ranged from 1.04 to 1.14 Mg/m^3 with an average of 1.10 Mg/m^3. Base on Paige- Green (2007) Facts, the studied soils have low permeability and porosity due its high compaction which means that it has more of finer materials than coarse material.

Table 4.2.7: General relationship of soil bulk density to root growth based on soil texture.

Soil Texture	Ideal bulk densities for plant growth (Mg/m^3)	Bulk densities that affect root growth (Mg/m^3)	Bulk densities that restrict root growth (Mg/m^3)
Sands, loamy sands	< 1.60	1.69	> 1.80
Sandy loams, loams	< 1.40	1.63	> 1.80
Sandy clay loams, clay loams	< 1.40	1.60	> 1.75
Silts, silt loams	< 1.40	1.60	> 1.75
Silt loams, silty clay loams	< 1.40	1.55	> 1.65
Sandy clays, silty clays, clay loams	< 1.10	1.49	> 1.58
Clays (> 45% clay)	< 1.10	1.39	> 1.47

4.2.9: Specific gravity

The strength of a soil mass is directly proportional to its specific gravity and the greater the specific gravity of a soil mass, the more the strength increases while the lesser the specific gravity of a soil mass, the more the strength decreases. Thus, the specific gravity of soils primarily depend on the density of the minerals and chemical making up of the individual soil particles (Oyediran & Durojaiye, 2011) and the specific gravity of soils also reflects the history of weathering (Tuncer & Lohnes, 1977). The specific gravity of a soil is very important as far as the quantitative behaviour of the soil is concern (Raj, 2012) and it is also used in mineral classification of soils for example, Bowles (2012) observed that iron mineral have high specific gravity than silica mineral. If a soil specific gravity is higher than water, it means it is denser than water which means it will not float. The specific gravity of a soil also gives an idea on the suitability of the soil for construction purposes in the sense that the high the value of specific gravity, the more considerable/recommendable for construction purposes because it gives more strength for roads and foundations. Also, Roy & Dass (2014) found that high specific gravity value can increase the shear strength parameters (cohesion and angle of shearing resistance) while Roy 2016, observed that increase in specific gravity also increases the California bearing ratio i.e. strength of the subgrade materials used in road construction. Wright (1986) classified that the standard range for specific gravity of soils should be between 2.60 and 2.80 while FMWH (1997) specified a specific gravity value of 2.2 as suitable for construction purposes.

The general guidelines of some typical values for specific soil type according to ASTM D854-92 and Bowels (2012) is shown below (Table 4.2.8).

Based on the studies, the specific gravity of the studied soils derived from Ameki Formation ranged from 2.62-2.65 with an average of 2.64 and Imo Shale Formation ranged from 2.64-2.71 with an average of 2.67 which indicates that the studied soils are inorganic soil.

Table 4.2.8: Range of specific gravity by ASTM D854-92 and Bowles (2012)

Soil type	Range of specific gravity specified by ASTM D854-92 and Bowles (2012)
Iron-rich or mica laterite (eg lateritic soils)	2.67-3.00
Sand	2.65-2.67
Silt	2.67-2.70
Clay and silty clay(inorganic)	2.67-2.80
Organic soil	1.00-2.60

4.2.10: Dry Density

Dry density of a soil depends on the structure of the soil matter (compacted or loosen) and the soil matrix (swelling and shrinkage characteristics). The dry density of the soils derived from Imo Shale Formation ranged from 0.82-1.03Mg/m^3 with an average 0.95Mg/m^3 while that of Ameki Formation ranged from 0.83-0.89Mg/m^3 with an average of 0.85 Mg/m^3. Using Hillel (1980), the dry density of the studied soils falls within aggregated loam and clayey soils because he reported that dry density of soils can be as high as 1.6Mg/m^3 in sandy soils while in aggregated loam and clayey soils, it can be as low as 1.1Mg/m^3.

Comparing the studied soil with Linsely *et al.,* (1982) and Poffijn (1988) specifications for dry density, the values of dry density of the studied soils falls within range. The highest value of dry density of 1.03Mg/m^3 was observed in Imo Shale Formation (Ufuma) while the lowest value of 0.82Mg/m^3 was observed at Ameki Formation (Enugwu-Agidi).

4.2.11: Particle Size Distribution and Hydrometer Test

The results and graphs of the sieve and hydrometer analysis of the expansive soils derived from Ameki Formation and Imo Shale Formation using unified soil classification system (USCS) and general specification by Roads and Bridges- Revised (1997), (FMW) Nigeria shows that the studied soils are poorly graded and poor subgrade materials with less coarse sand material (Figure 4.2.1). The particle size distribution curve provides an information on how graded a particular soil sample is (well graded or poorly graded), also particle size is one of the suitability criteria of soils to be used for highway, underground utilities, levee, dam and other embankment constructions (Bowels, 2012). The percentage passing through No.200 BS sieve from the expansive soils derived from Ameki Formation ranged between 63.00-84.90% with an average of 76.20% while the soils derived from Imo Shale Formation ranged from 62.10 to 94.80% with an average of 78.26%, which implies that the soil samples are fined-grained soils according to Unified Soil Classification Systems. This indicates that the soils derived from both geologic Formations are poorly-graded because they have >50% passing through sieve No.200 BS. In making estimates of the degree of expansiveness with respect to particle size distribution curve, Chen (1975) established that a soil with more than 95% passing through No.200 BS sieve and more than 60% liquid limits have very high degree of expansion. The studied soils have liquid limit >60% and >60% passing through sieve No.200 BS which supports the claims that they are expansive in nature. This observation tends to support the fact that the studied soils are problematic soils because they have more finer-grained materials. According to Federal Ministry of Works and Housing (1972) specification, the studied soils are not suitable for subgrade, sub-base materials as the percentage by weight finer than No.200 BS test sieve is >35.

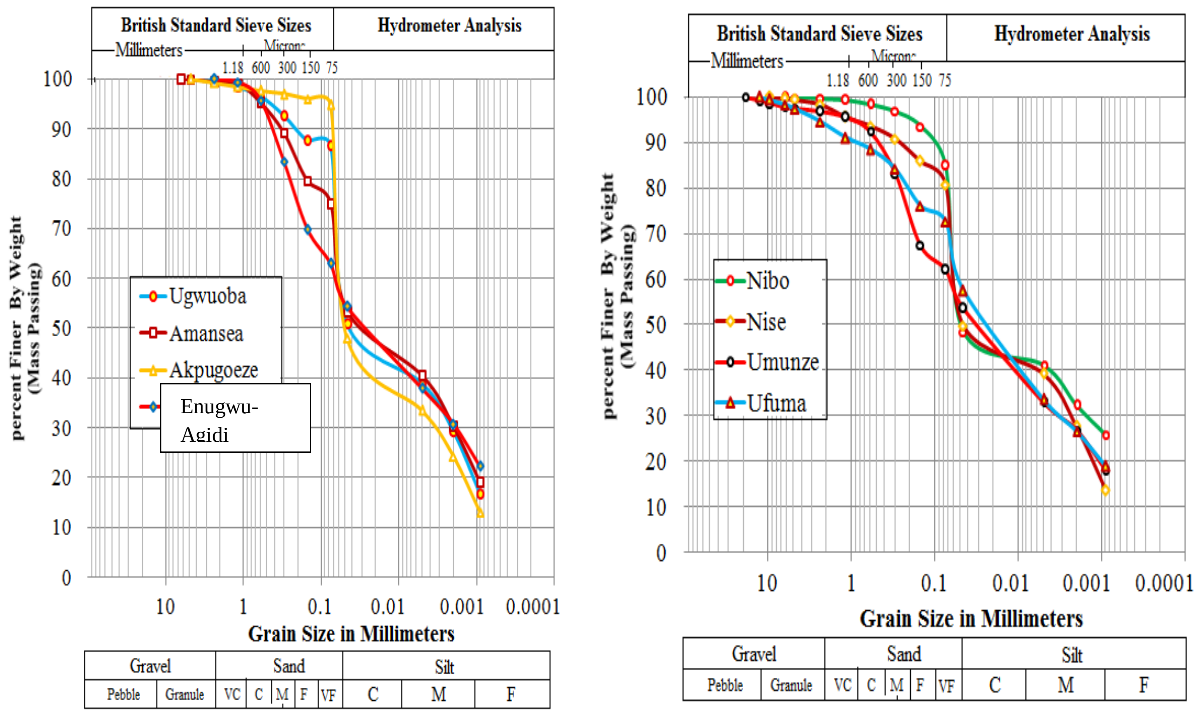

Figure 4.2.2: Particle Size Distribution Curve of expansive soils derived from Awka and environs.

4.2.12: Soil Classification

Casagrande (1948) classified soils with 50% passing sieve No. 200, liquid limit(LL) of more than 50% and plasticity index(PI) plots between “A” and U” line as inorganic fat clays (CH soils). When the studied soils was plotted on the Casagrande plasticity chart using plasticity index(PI) against liquid limit(LL), it shows that they both plots on the high compressive section (degree of compressibility) of the plasticity chart which is between the “A’’ line and “U” line which is the boundary for clays and can be classified as CH soils which indicates that they are all “fat clays” (clay with high plasticity). This suggests that the surface of clay soils have high affinity for water which indicates the presence of clay minerals such as Kaolinite, Montmorillonite and Illite etc.

Using AASHTO soil classification for highway (1967), the studied soil s are poor (fall within A-7 group) soils for both sub-grade and sub-base material because they have percentage passing sieve No. 200 >35%.

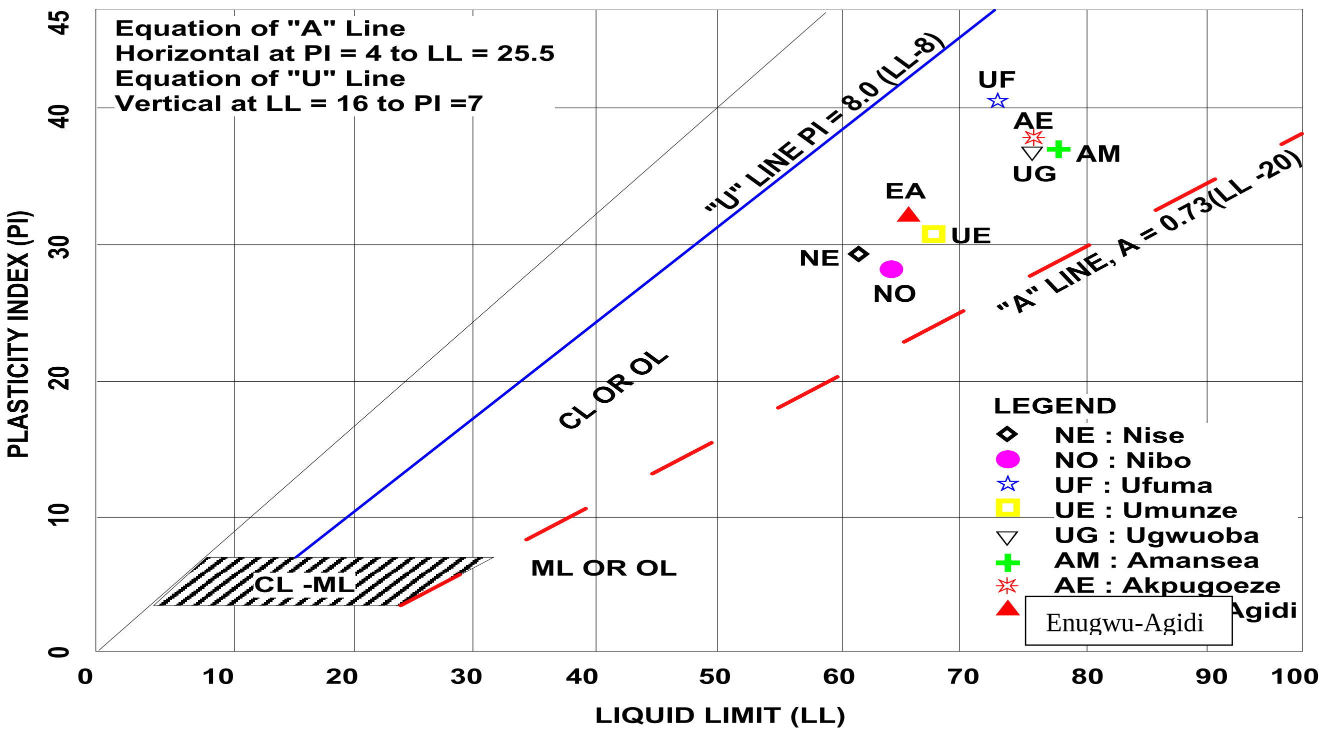

Figure 4.2.3: Identification and classification of expansive soils derived from Imo Shale Formation and Ameki Formation in Awka and environs using Casagrande Plasticity Chart.

4.2.13: Compaction Test

The compaction test of the studied soils indicates that maximum dry density (MDD) of the expansive soils derived from Ameki Formation ranged from 1.49-1.67Mg/m^3 with an average of 1.59 Mg/m^3 while the soils derived from Imo shale Formation ranged from 1.40-1.75Mg/m^3 with an average of 1.58Mg/m^3 and the optimum moisture content (OMC) of the expansive soils derived from

Ameki Formation ranged from 17.70-27.00% with an average of 22.90% while the soils derived from Imo shale Formation ranged from 18.10-24.30% with an average of 21.82%. Bello *et al.,* (2007) and Madedor (1983) characterized soils with high maximum dry density (MDD) and low optimum moisture content (OMC) as suitable for sub-grade and sub-base materials while Federal Ministry of Works and Housing (1997), characterized soils with OMC value <18% and MDD value >0.047 Mg/m^3 as suitable for both sub-grade and sub-base materials. Thus, the soils derived from both Imo Shale Formation and Ameki Formation are not suitable to be used as sub-grade or sub-base materials because they have low MDD value and high OMC value.

4.2.14: California Bearing Ratio Test

The CBR test quantitatively evaluates the inherent strength of a subgrade for a road pavement to be designed for a particular strength of subgrade. The higher the CBR value of soils, the more recommendable the soils to be used as subgrade and sub-base materials.

The California Bearing Ratio (CBR) test of the studied soils indicates that the unsoaked CBR value of the expansive soils derived from Ameki Formation ranged from 11.50-18.10% with an average of 15.63% while the soils derived from Imo Shale Formation ranged from 10.20-15.40% with an average of 12.58% and the soaked CBR value of expansive soils derived from Ameki Formation 1.40-2.40% with an average of 1.90% while the soils derived from Imo Shale Formation ranged from 2.20 to 3.30% with an average of 2.57%.

FMWH (1997) recommend soaked CBR value of minimum of 15% and unsoaked CBR value of minimum 40% for highway subgrade.

In general, the studied soils yield poor CBR values which are not likely to provide a stable compacted sub-grade and sub-base materials. The reduction in the CBR value in the study area could be as a result of high amount of clayey soil, poor laterization and ingress of groundwater flow due to poor maintenance of drainage facilities in parts of the studied area. In terms of strength, the studied soils of are not suitable to be used as subgrade and foundation materials in engineering construction.

4.3: Treatment of the Expansive Soils (Soil Stabilization)

This method employed to improve the geotechnical properties of expansive soils to make it suitable for engineering or agricultural purposes. Stabilization can control the degree of expansion and swelling potentials of the soil, thereby enhancing the load bearing capacity (strength characteristics) of a subgrade to support pavement and foundation. This method is widely known as an effective way for improving soil properties and it can be done using mechanical (densification method) or chemical means(adding additives). These additives include Portland cement, fly ash, rice husk ash, lime, calcium or sodium chloride, viscoelastic materials such as bitumen byproducts of lime kiln dust, cement kiln dust, quarrying dust or combinations of these materials (Zumrawi, 2015; Tensar Technical Note, TTN, BRIO, 1998; Materials and Test Division, Geotechnical section, Indiana, 2002).

The chemical additives used for the purpose of this research work are hydrated lime and Portland cement. Their various percentages were employed in treating the expansive soils in order to improve the engineering properties of the soil. A representative soil sample from each geologic Formation was treated with cement and lime and the soils parameter that were evaluated for the soil improvement include, Atterberg limit test (liquid limit, Plastic limit and Plasticity index) which is for the swelling potential of the soils, Linear shrinkage for degree of expansion, California bearing ratio (CBR) test and compaction test (OMC and MDD) for strength characteristics of the expansive soils. Various percentages of cement and lime additives were mixed together with the representative soil sample from Imo Shale (Akpugoeze) and Ameki Formation (Nibo). The percentages of additives added are 2-10%. The mechanism of treatment comprised of hydration, cation exchange, flocculation agglomeration of soil particles and pozzolanic reactions to form Calcium Silicate Hydrate (C-S-H), Calcium Aluminate Hydrate (C-A-H) and Calcium Alumino Silicate Hydrate (CASH).

Factors affecting soil treatment include type of additive, curing time, temperature and soil mineralogy. Advantages of soil treatment include soil strength gain, reduction in plasticity (increased workability) reduction in liquid limit and increase in soil durability. Carbonation, sulphate attack and environmental impact are some of the disadvantages associated with treating soil with a calcium based additive. The cement and lime composition was plotted against the geotechnical properties (LL, PL, PI and LS). A graph of geotechnical properties against different percentages of cement or lime in the soil sample was plotted to show how the geotechnical properties improved when different percentages of cement or lime is added (Figures 4.3.1.-4.3.14.).

4.3.1: Stabilization treatment with cement and lime of expansive soils derived from Imo Shale Formation

When a representative soil from Imo Shale Formation (Akpugoeze) was treated with about 8% of cement, the liquid limit (LL) and plasticity index (PI) was reduced from 76.40-42.50% and 46.45-9.60% respectively. The linear shrinkage (LS) was reduced from 17.90-5.30% and optimum moisture content (OMC) reduced from 23.20-20.40%. The maximum dry density (MDD) was increased from 1.40-1.75Mg/m^3 and the California bearing ratio (CBR) values for soaked and unsoaked was increased from 3.30-34.30 and 10.20-43.10% respectively. When the same soil was treated with about 6% of lime, the liquid limit (LL) and plasticity index (PI) was reduced from 76.40-39.10%, and 46.45-5.40% respectively. The linear shrinkage, optimum moisture content (OMC) and maximum dry density (MDD) were reduced from 17.90-4.30%, 23.20 to 18.40% and 1.40 to 1.06Mg/m^3 respectively while the California bearing ratio (CBR) for soaked and unsoaked was increased from 3.30-28.10% and 10.20-38.10% respectively.

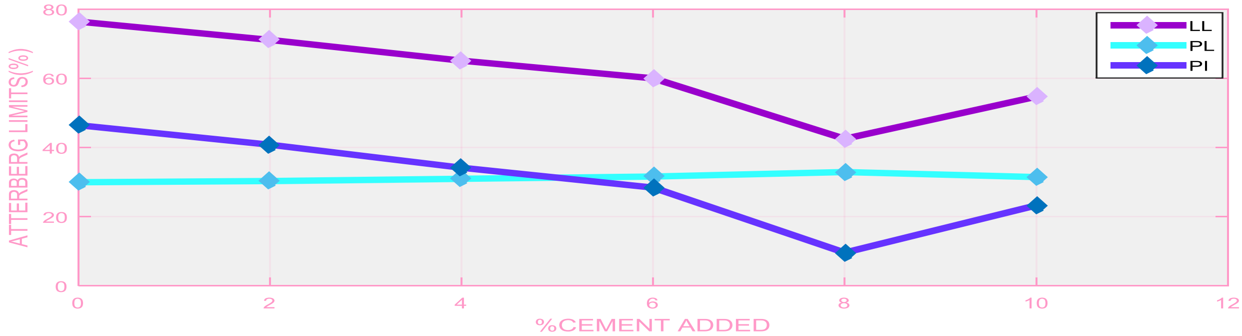

Figure 4.3.1: variation of liquid limit (LL), plastic limit (PL) and plasticity index (PI) of expansive soils derived from Imo Shale Formation with different percentages of cement

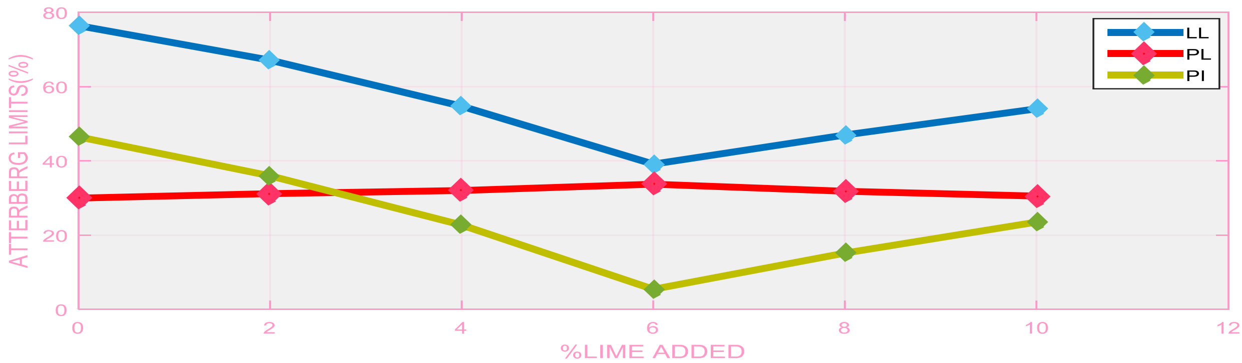

Figure 4.3.2: Variation of liquid limit (LL), plastic limit (PL) and plasticity index (PI) of expansive soil derived from Imo Shale Formation with different percentages of lime

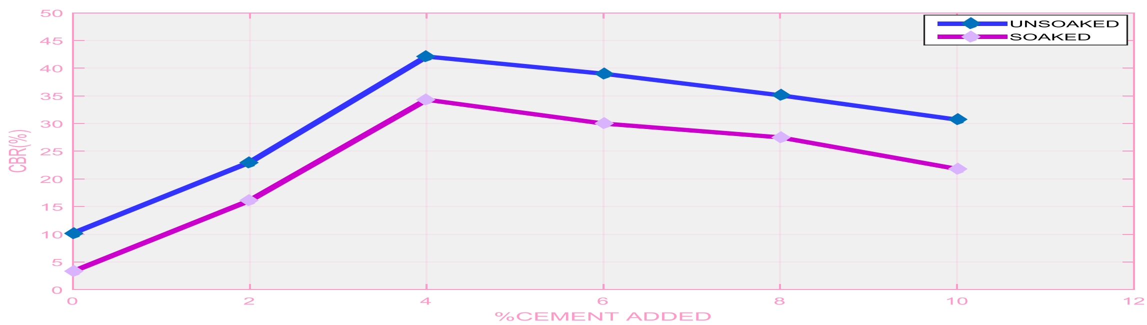

Figure 4.3.3: Variation of soaked and unsoaked CBR test of expansive soil derived from Imo Shale Formation with different percentages of cement

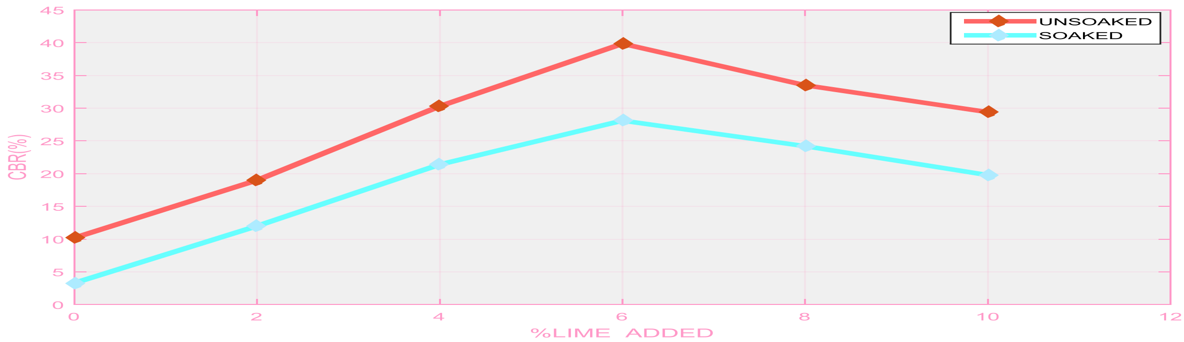

Figure 4.3.4: Variation of soaked and unsoaked CBR test of expansive soil derived from Imo Shale Formation with different percentages of lime

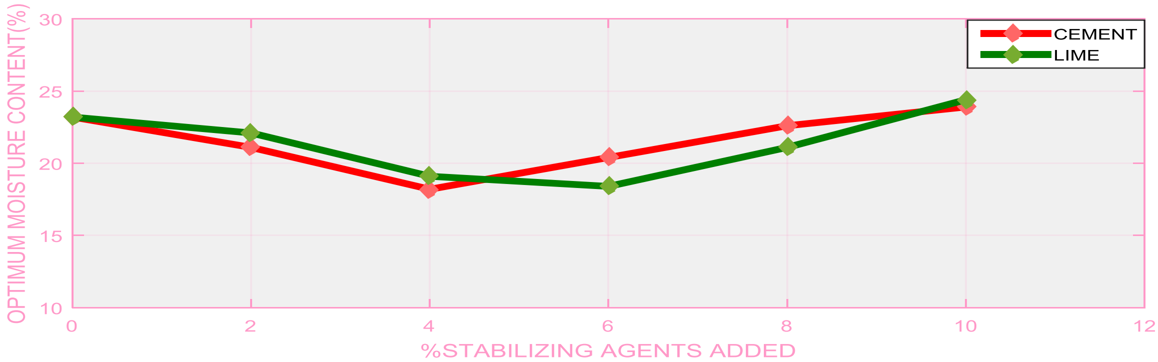

Figure 4.3.5: Variation of optimum moisture content (OMC) of expansive soil derived from Imo Shale Formation with different percentages of cement and lime

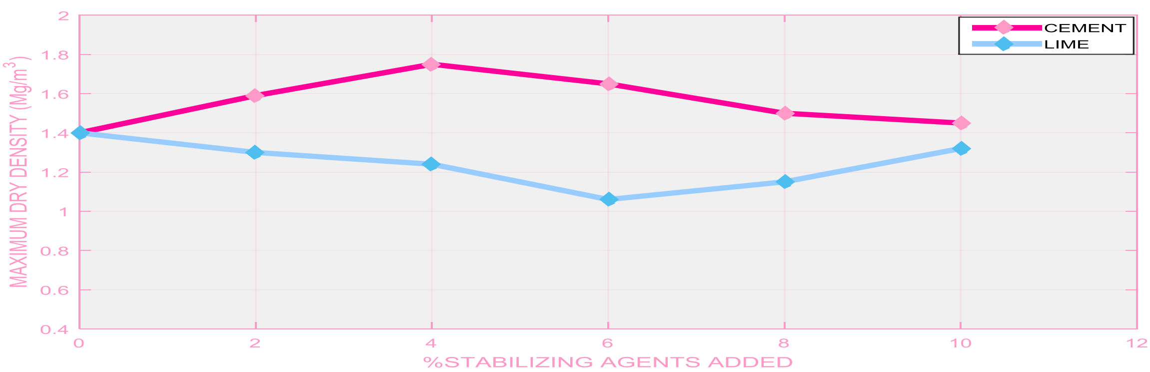

Figure 4.3.6: Variation of maximum dry density (MDD) of expansive soil derived from Imo Shale Formation with different percentages of cement and lime

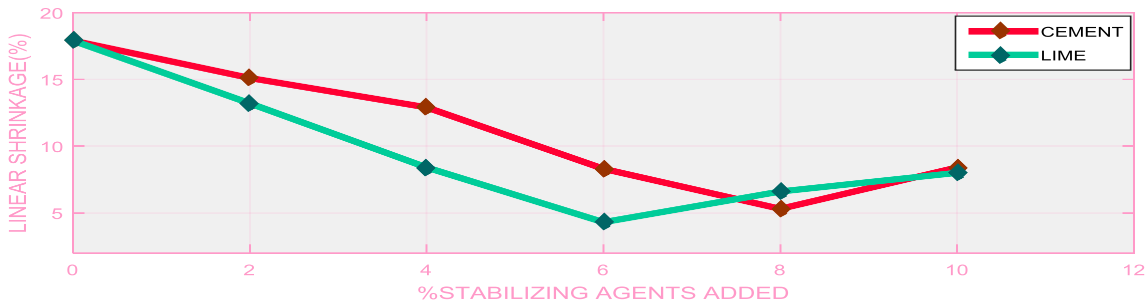

Figure 4.3.7: Variation of linear shrinkage (LS) of expansive soil derived from Imo Shale Formation with different percentages of cement and lime

4.3.2: Stabilization treatment with cement and lime of expansive soils derived from Ameki Formation

Similarly, when a representative soil sample from Ameki Formation (Nibo) was treated with 8% of cement additives, the liquid limit (LL) and plasticity index (PI) was reduced from 65.00-41.30% and 34.05-8.20% respectively. The linear shrinkage and optimum moisture content (OMC) was reduced from 12.00-4.20% and 27.05-17.90% while the maximum dry density(MDD) and California bearing ratio (CBR) values for soaked and unsoaked increases from 1.49-1.70Mg/m^3, 2.41-27.56% and 17.04-38.40% respectively. When the same soil was treated with about 6% of lime additive, liquid limit (LL) and plasticity index (PI) was reduced from 65.00-39.95% and 34.05-6.95% respectively. Also, the linear shrinkage, optimum moisture content (OMC) and the maximum dry density(MDD) was reduced from 12.00-2.60%, 27.05-18.23% and 1.49-1.13Mg/m^3, respectively while California bearing ratio (CBR) values for soaked and unsoaked was increased from 2.41-24.57% and 17.04-35.85% respectively.

The result of the soil stabilization performed on the studied soils proved that the liquid limit and plasticity index at 8% cement and 6% lime additives in both geological Formations shows a low swelling potential (Holtz & Gibbs, 1956; Ola, 1981) while the linear shrinkage of both geologic Formations at 8% of cement and 6% of lime additives indicates non-critical degree of expansion (Attimeyer, 1956) which causes a great improvement in the strength characteristics (MDD and CBR values) of the studied soils.

Generally, the addition of these additives (cement and lime) to the studied soils causes a reduction in the swelling potential and degree of expansion of the soil and thus increases the strength and durability of the soils to be used as subgrade and foundation soils in engineering constructions.

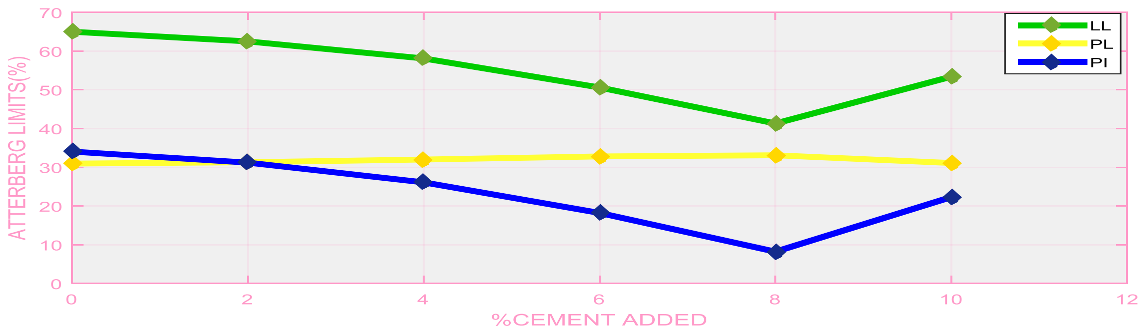

Figure 4.3.8: Variation of liquid limit (LL), plastic limit (PL) and plasticity index (PI) of expansive soil derived from Ameki Formation with different percentages of cement

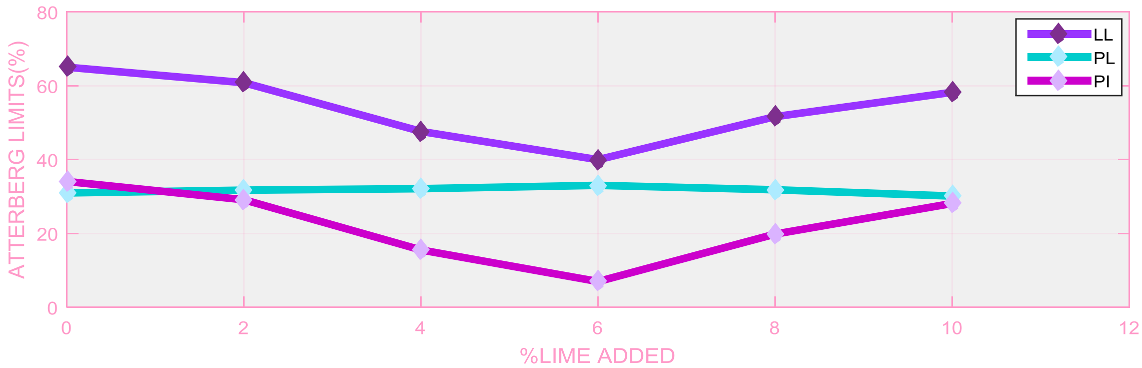

Figure 4.3.9: Variation of liquid limit (LL), plastic limit (PL) and plasticity index (PI) of expansive soil derived from Ameki Formation with different percentages of lime

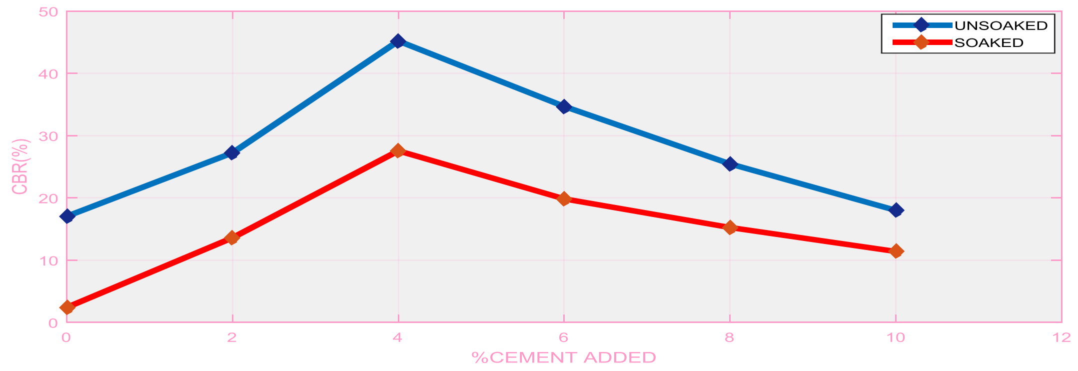

Figure 4.3.10: Variation of soaked and unsoaked CBR test of expansive soil derived from Ameki Formation with different percentages of cement

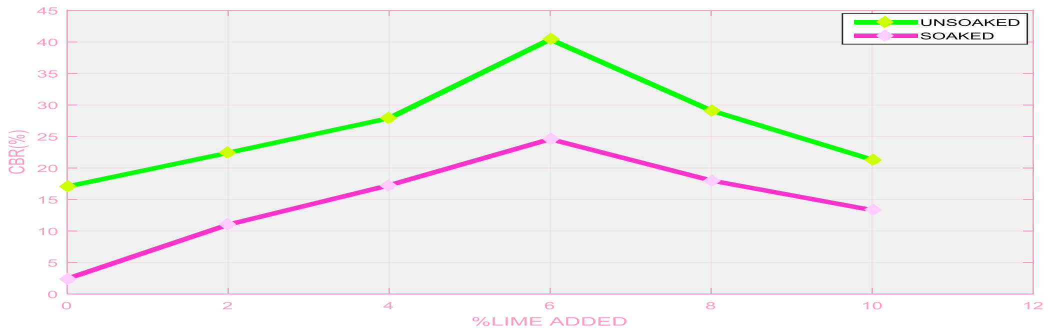

Figure 4.3.11: Variation of soaked and unsoaked CBR test of expansive soil derived from Nibo (Ameki Formation) with different percentages of lime

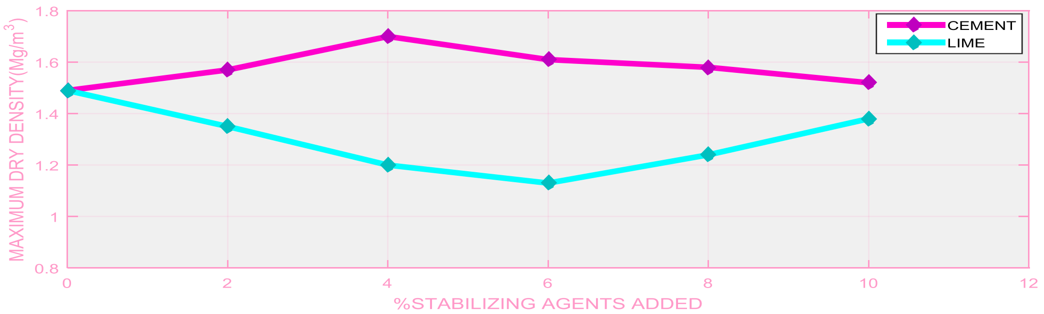

Figure 4.3.12: Variation of maximum dry density (MDD) of expansive soil derived from Nibo (Ameki Formation) with different percentages of cement and lime

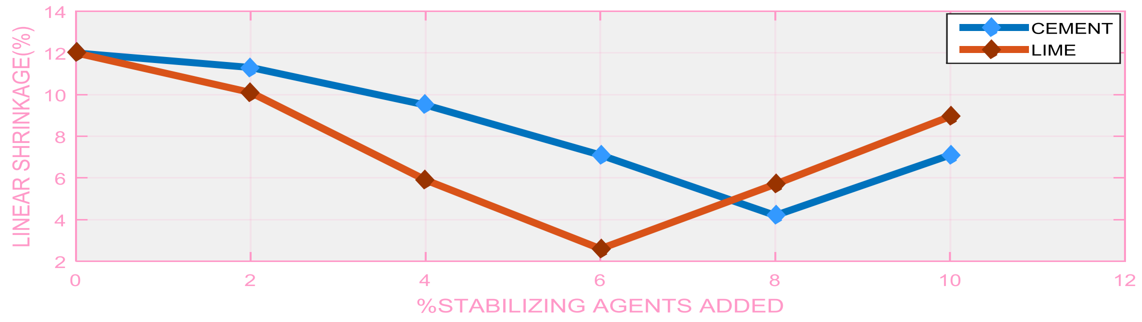

Figure 4.3.13: Variation of linear shrinkage (LS) of expansive soil derived from Ameki Formation with different percentages of cement and lime

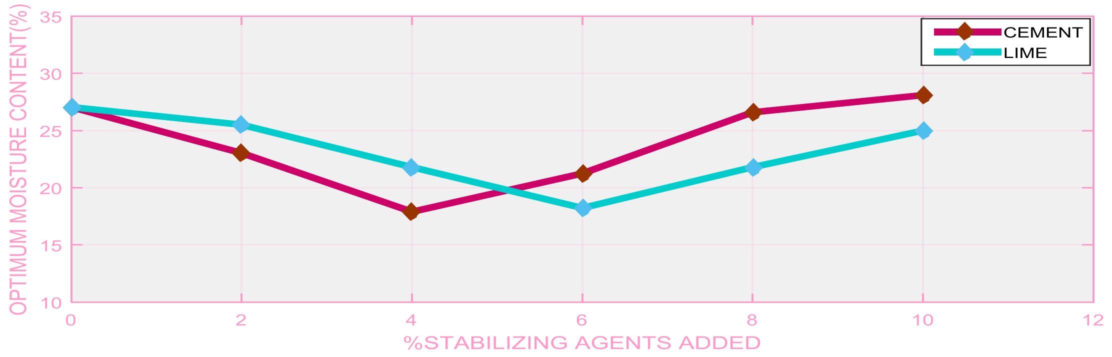

Figure 4.3.14: Variation of optimum moisture content (OMC) of expansive soil derived from Nibo (Ameki Formation) with different percentages of cement and lime

4.4: The strength characteristics of the studied soils

The strength characteristics a soil material is the ability of the soil to withstand load or stress. Some of the geotechnical parameters used to estimate the strength characteristics of soils include maximum dry density (MDD) and California Bearing Ratio (CBR) parameters. Compaction test was performed on the studied soils to evaluate the compaction characteristics of the soils with change in moisture content. The compaction of soil is the optimal moisture content at which a given soil becomes most dense and achieves its maximum dry density in other to reduce the air voids. When mechanical energy induced to the soil mass during compaction test, the air voids between the soil particles tends to decrease due to the applies force which increase the density and frictional force of the soil particles. Once the optimum moisture content is achieved, any further addition of water(moisture) will cause reduction in the dry unit weight because the pore water pressure will be pushing the soil particles apart, decreasing the friction between the particles. California bearing ratio test (penetration test) was performed to the studied soils to evaluate the strength of the soils to be used as a subgrade in roads and pavement construction. This test measures the resistance of the material to penetration of a standard plunger under controlled density and moisture conditions. This test is mostly used to determine the thickness of pavement and component layers usually used in flexible pavement designs.

In general, the strength characteristics of the studied soils derived from both Ameki Formation and Imo shale Formation using California Bearing Ratio (CBR) and compaction test values (MDD) shows that the studied soils improved when different percentages of cement and lime additive were added until it gets to 8% cement and 6% lime where further addition of additives does not improve the strength characteristics of the soils.

4.5: Evaluation of the stabilization results of the studied soils

The evaluation of effectiveness of treatment with cement and lime on index properties of expansive soils derived from Imo Shale Formation and Ameki Formation are shown below (Tables 4.5.1-4.5.4) while the evaluation of effectiveness of treatment with cement on CBR and MDD of expansive soils derived from Imo Shale and Ameki Formation are shown below (Tables 4.5.5-4.5.8).

Table 4.5.1: Evaluation of effectiveness of treatment with cement on index properties of expansive soils from Imo Shale (Akpugoeze)

% cement added	LL(%)	%Reduction	PI(%)	%Reduction	LS(%)	%Reduction
0	76.40	-	46.45	-	17.90	-
2	71.10	6.94	40.80	12.16	15.10	15.64
4	65.10	14.79	34.15	26.48	12.90	27.93
6	60.00	21.47	28.40	47.47	8.30	53.63
8	42.50	44.37	9.60	79.33	5.30	70.39
10	54.70	28.40	28.40	49.95	8.40	53.07

Table 4.5.2: Evaluation of effectiveness of treatment with lime on index properties of expansive soils from Imo Shale (Akpugoeze)

% lime added	LL (%)	%Reduction	PI (%)	%Reduction	LS (%)	%Reduction
0	76.40	-	46.45	-	17.90	-
2	67.10	12.17	35.95	22.60	13.20	26.26
4	54.70	28.40	22.70	51.13	8.40	53.07
6	39.10	48.82	5.40	88.37	4.30	75.98
8	46.95	38.55	15.15	67.38	6.60	63.13
10	54.00	28.53	23.50	49.41	8.00	55.31

Table 4.5.3: Evaluation of effectiveness of treatment with cement on CBR and MDD of expansive soils from Imo Shale (Akpugoeze)

% cement added	CBR test (%)				Compaction test	
	Soaked	% Increase	Unsoaked	% Increase	MDD (Mg/m^3)	% Increase
0	3.30	-	10.20	-	1.40	-
2	16.10	387.88	23.00	125.49	1.59	13.57
4	34.30	939.39	42.10	312.75	1.75	25.00
6	30.50	824.24	39.00	282.35	1.65	17..86
8	27.50	733.33	35.10	244.18	1.50	7.14
10	21.80	560.61	30.70	200.98	1.45	3.57

Table 4.5.4: Evaluation of effectiveness of treatment with lime on CBR and MDD of expansive soils from Imo Shale (Akpugoeze)

% lime added	CBR test (%)				Compaction test	
	Soaked	% Increase	Unsoaked	% Increase	MDD (Mg/m^3)	% Reduction
0	3.30	-	10.20	-	1.40	-
2	12.00	263.63	19.00	86.27	1.30	7.14
4	21.40	548.48	30.30	197.06	1.24	11.42
6	28.10	751.51	39.85	273.53	1.06	24.29
8	24.20	633.33	33.50	228.43	1.15	17.89
10	19.78	499.39	29.40	188.24	1.32	5.71

Table 4.5.5: Evaluation of effectiveness of treatment with cement on index properties of expansive soils from Ameki Formation (Nibo)

% cement added	LL (%)	%Reduction	PI (%)	%Reduction	LS(%)	%Reduction
0	65.00	-	34.05	-	12.00	-
2	62.50	3.85	31.20	8.37	11.30	5.83
4	58.10	10.62	26.10	23.35	9.50	20.83
6	50.60	22.15	18.20	46.55	7.10	40.83
8	41.30	36.46	8.20	75.92	4.20	65.00
10	53.40	17.85	22.30	34.51	7.10	40.83

Table 4.5.6: Evaluation of effectiveness of treatment with lime on index properties of expansive soils from Ameki Formation (Nibo)

% lime added	LL (%)	%Reduction	PI(%)	%Reduction	LS(%)	%Reduction
0	65.00	-	34.05	-	12.00	-
2	60.80	6.46	29.10	14.54	10.10	15.83
4	47.30	26.77	15.50	54.48	5.90	50.83
6	39.95	38.54	6.95	79.59	2.60	78.83
8	51.60	20.62	19.80	41.85	5.70	52.50
10	58.20	10.46	28.10	17.47	8.95	25.42

Table 4.5.7: Evaluation of effectiveness of treatment with cement on CBR and MDD of expansive soils from Nibo

% cement added	CBR test (%)				Compaction test	
	Soaked	% Increase	Unsoaked	% Increase	MDD (Mg/m^3)	% Increase
0	2.41	-	17.04	-	1.49	-
2	13.60	464.32	27.24	59.86	1.57	5.37
4	27.56	1043.59	45.20	165.26	1.70	14.09
6	19.85	723.65	34.70	103.64	1.61	8.05
8	15.25	532.78	25.47	49.47	1.58	6.04
10	11.40	373.03	18.00	5.63	1.52	2.01

Table 4.5.8: Evaluation of effectiveness of treatment with lime on CBR and MDD of expansive soils from Ameki Formation (Nibo)

% lime added	CBR test (%)				Compaction test	
	Soaked	% Increase	Unsoaked	% Increase	MDD (Mg/m^3)	% Reduction
0	2.41	-	17.04	-	1.49	-
2	11.00	356.43	22.40	31.46	1.35	9.40
4	17.25	615.77	27.90	63.73	1.20	19.46
6	24.57	919.50	40.40	137.09	1.13	24.16
8	18.00	646.89	29.10	70.77	1.24	16.78
10	13.30	451.87	21.30	25.00	1.38	7.38

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1: Conclusion

Expansive soils derived from different geologic formations in Awka area were investigated in terms of their geotechnical characterization. The result of the geotechnical characterization shows that the samples derived from Ameki Formation have high swelling potential while those derived from Imo Shale have very high swelling potential using liquid limit and plasticity index values.

On the basis of activity the soil samples from Imo Shale have high activity while those derived from Ameki Formation have medium activity. Based on the degree of expansion using linear shrinkage and free swell values both geologic Formations have critical (linear shrinkage) and high (free swell) degree of expansion. The soil samples derived from both geologic Formation are classified as CH soils (soil with high plasticity), which indicates that they are inorganic fat clays because it plots between “A” and “U” line of the plasticity chart.

When the studied expansive soil samples were stabilized with different percentages of cement and lime, there is a reduction in swelling potential (Atterberg limit), degree of expansion (linear shrinkage), optimum moisture content (OMC) and increase in the strength characteristics (MDD and CBR).

Furthermore, the maximum stabilization point (optimum fixation point) of lime and cement was achieved at 6% and 8% addition of the additives. Any further addition of the additive to the soil samples does not improve the geotechnical properties of the soils because that is the lime and cement optimum fixation point.

Therefore, the use of hydrated lime alone is suitable to stabilize the swelling soil which has high liquid limit and high plasticity index while stabilization with cement is suitable for stabilizing fine grained swelling soils which contained low amount of clay particle and higher amount of silt.

5.2: Recommendation

The use of hydrated lime is suitable to stabilize expansive soils with high liquid limit and high plasticity index while the use of cement is suitable for stabilizing fine grained expansive soils which contained low amount of clay particle and low plastic materials but the use of a combination of cement and lime in soil stabilization can give a better stabilizing effect, however, more studies are recommended for this application.

The federal government, State government and individual can key into findings previous and recent research works on soil stabilization in the study areas before commencing on any construction projects so as to save long term cost of maintenance when swelling problems surfaces.

To reduce some of the problems caused by expansive soils on engineering structures, designer engineers should investigate the causes of swelling of these problem soils before embarking on any project.

Furthermore additional studies are recommended to evaluate the use of lime and cement to treat and stabilize different types of soil and to establish a more reliable Nigerian guideline and standard specification for lime/cement stabilization process.

5.3: Contributions to Knowledge

i. The study confirmed for the first time the occurrence of expansive soils in towns like Ufuma, Akpugoeze, Ugwuoba and Enugwu-Agidi and their occurrence is influenced/controlled by the geologic Formation in area Ameki Formation and Imo Shale.

ii. From the geotechnical characterization of the soils, expansive soils derived from Imo Shale have higher values of PI, LL, LS and activity of clay than those derived from Ameki Formation.

iii. On the bases of swelling potentials using PI and LL, expansive soils derived from Imo Shale may be classified as very high swelling potential while those derived from Ameki Formation have high swelling potential.

iv. The activity of clay of expansive soils derived from Imo Shale has high activity while those derived from Ameki Formation have medium activity.

v. On the basis of degree of expansion using free swell and LS values, all the soils derived from both geologic Formations have high and critical values of degree of expansion.

vi. The effects of lime and cement on the stabilization of the expansive soils in the area have been successfully compared. The result proved that stabilization with lime performs better than cement in both formations. Categorically, cement stabilization is more economically than lime stabilization.

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APPENDICES

Appendice 1**:** Sieve analysis result of expansive soil derived from Ugwuoba

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
6.4	1/4"	-	-	100.00
4.76	3/16"	0.2	0.10	99.90
2.36	7	0.90	0.30	99.60
1.18	14	3.70	12.30	98.40
600 mic	25	5.80	19.30	96.50
425 mic	36	-	-	-
300 mic	52	11.00	3.70	92.80
150 mic	100	15.10	5.00	87.80
75 mic	200	3.30	11.00	86.70

Appendice 2**:** Sieve analysis result of expansive soil derived from Nibo

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
9.52	3/8"	-	-	100.00
6.4	1/4"	0.50	0.10	99.90
4.76	3/16"	-	-	-
2.36	7	0.90	0.20	99.70
1.18	14	1.80	0.40	99.30
600 mic	25	4.60	0.90	98.40
425 mic	36	-	-	-
300 mic	52	7.60	1.50	96.90
150 mic	100	16.50	3.30	93.60
75 mic	200	43.80	8.70	84.90

Appendice 3**:** Sieve analysis result of expansive soil derived from Nise

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
9.52	3/8"	-	-	100.00
6.4	1/4"	1.00	0.20	99.70
4.76	3/16"	1.30	0.30	99.40
2.36	7	5.00	12.50	98.20
1.18	14	10.50	2.60	95.60
600 mic	25	8.00	2.00	93.60
425 mic	36	-	-	-
300 mic	52	11.20	2.80	90.80
150 mic	100	19.80	4.90	85.90
75 mic	200	29.70	5.20	80.70

Appendice 4: Sieve analysis result of expansive soil derived from Umunze

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
19.05	3/4"	-	-	100.00
12.70	1/2"	3.80	0.40	99.10
9.52	3/8"	2.20	0.50	98.60
6.4	1/4"	3.30	0.80	97.80
4.76	3/16"	1.40	0.30	97.50
2.36	7	3.10	0.70	96.80
1.18	14	5.30	1.30	95.50
600 mic	25	12.50	3.00	92.50
425 mic	36	-	-	-
300 mic	52	30.20	7.20	83.30
150 mic	100	75.20	17.80	67.50
75 mic	200	22.80	5.40	62.10

Appendice 5: Sieve analysis result of expansive soil derived from Ufuma

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
12.70	1/2"	-	-	100.00
9.52	3/8"	2.40	0.60	99.40
6.4	1/4"	4.30	1.10	98.30
4.76	3/16"	4.00	1.00	97.30
2.36	7	11.30	2.80	94.50
1.18	14	14.20	3.50	91.00
600 mic	25	9.90	2.50	88.50
425 mic	36	-	-	-
300 mic	52	17.10	4.30	84.20
150 mic	100	32.50	8.10	76.10
75 mic	200	14.10	3.50	72.60

Appendice 6: Sieve analysis result of expansive soil derived from Amansea

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
6.4	1/4"	-	-	100.00
4.76	3/16"	0.60	0.10	99.90
2.36	7	2.20	0.40	99.50
1.18	14	1.80	0.40	99.10
600 mic	25	18.60	3.70	95.40
425 mic	36	-	-	-
300 mic	52	31.00	6.20	89.20
150 mic	100	47.90	9.60	79.60
75 mic	200	22.40	4.50	75.10

Appendice 7: Sieve analysis result of expansive soil derived from Akpugoeze

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
4.76	3/16"	-	-	100.00
2.36	7	0.80	0.70	99.30
1.18	14	1.00	0.80	98.50
600 mic	25	1.00	0.80	97.70
425 mic	36	-	-	-
300 mic	52	0.90	0.80	97.00
150 mic	100	1.20	1.00	96.00
75 mic	200	1.40	1.20	94.80

Appendice 8: Sieve analysis result of expansive soil derived from Enugwu-Agidi

B.S Sieve (mm)	Mesh No.	Retained (g)	Retained (%)	% passing
2.36	7	-	-	100.00
1.18	14	0.90	0.60	99.40
600 mic	25	5.40	3.60	95.80
425 mic	36	-	-	-
300 mic	52	18.40	12.30	83.50
150 mic	100	20.40	13.60	69.90
75 mic	200	10.30	6.90	63.00

Appendice 9: Hydrometer result of expansive soil derived from Ugwuoba

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (Rh1)	True reading (Rh)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	19.10	19.60	133.64	18.90	0.04698	51.03
	10	14.60	15.10	152.09	14.40	0.00501	38.88
	30	11.05	11.55	166.65	10.85	0.00175	29.30
	60	6.45	6.95	185.51	6.25	0.00092	16.88

Appendice 10: Hydrometer result of expansive soil derived from Nise

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	18.65	19.15	135.49	18.45	0.04988	49.82
	10	14.75	15.25	151.45	14.55	0.00500	39.27
	30	10.45	10.95	169.11	10.25	0.00176	27.68
	60	5.25	5.75	190.43	5.05	0.00093	13.64

Appendice 11**:** Hydrometer result of expansive soil derived from ufuma

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	21.50	22.00	123.80	21.30	0.04520	57.51
	10	12.70	13.20	159.88	12.50	0.00514	33.75
	30	10.10	10.60	170.54	9.90	0.00177	26.73
	60	7.20	7.70	182.43	7.00	0.00091	18.90

Appendice 12**:** Hydrometer result of expansive soil derived from Nibo

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	18.15	18.65	137.54	17.95	0.04766	48.47
	10	15.45	15.95	148.61	15.25	0.00495	41.18
	30	12.20	12.70	161.83	12.00	0.00172	32.40
	60	9.80	10.30	171.77	9.60	0.00089	25.92

Appendice 13**:** Hydrometer result of expansive soil derived from Amansea

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	20.00	20.500	129.95	19.80	0.04633	53.46
	10	15.25	15.75	149.43	15.05	0.00497	40.64
	30	11.45	11.95	165.01	11.25	0.00174	30.38
	60	7.30	7.80	182.02	7.10	0.00091	19.17

Appendice 14**:** Hydrometer result of expansive soil derived from Akpugoeze

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	18.00	18.50	138.15	17.80	0.04777	48.06
	10	12.65	13.15	160.09	12.45	0.00514	33.62
	30	9.20	9.70	174.23	9.00	0.00179	24.30
	60	5.05	5.55	191.25	4.85	0.00093	13.10

Appendice 15**:** Hydrometer result of expansive soil derived from Enugwu-Agidi

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	20.35	20.85	128.52	20.15	0.04607	54.41
	10	14.25	14.75	153.53	14.05	0.00504	37.94
	30	11.60	12.10	164.39	11.40	0.00174	30.78
	60	8.45	8.95	173.31	8.25	0.00092	22.28

Appendice 16**:** Hydrometer result of expansive soil derived from Umunze

1	2	3	4	5	6	7	8
Date	Time	Hydrometer reading (R_h^1)	True reading (R_h)	Effective corrected reading (mm)	Fully corrected reading (R)	Particle diameter D(mm)	Percentage finer than D, K(%)
	1	20.10	20.60	129.54	19.90	0.04625	53.73
	10	12.45	12.95	160.91	12.25	0.00516	33.08
	30	10.15	10.65	170.34	9.95	0.00177	26.87
	60	6.85	7.35	183.87	6.65	0.00092	17.96

Appendice 17**:** Particle Size Classification (After USDA Soil Taxonomy)

Formation	Location	Gravel (>2 mm) %	Sand (0.06 – 2.0) (mm) %	Silt (0.002– 0.06) (mm) %	Clay (<0.002) (mm) %	Soil Type
Imo Shale	Umunze	3.20	36.80	33.10	26.90	Sandy silt
	Amansea	0.50	39.50	29.63	30.37	Sandy clay
	Ufuma	5.5 0	32.50	35.34	26.66	Silty sand
	Ugwuoba	0.40	39.60	30.71	29.29	sandy silt
	Akpugoeze	0.70	39.30	35.71	24.29	sandy silt
Ameki	Nise	1.80	38.20	32.32	28.00	sandy silt
	Nibo	0.30	41.70	25.58	32.42	sandy clay
	Enugu-Agidi	0.00	40.00	29.21	30.79	sandy clay

Appendice 18: Guidelines for Swelling Potential and Degree of Expansion of Expansive Soils based on various Geotechnical Parameters.

Swelling potential classification		Degree of expansion classification	
Geotechnical parameter used in the classification	Classification	Geotechnical parameter used in the classification	Classification
Plasticity index, PI (%) (Ola,1981) <15 15-25 25-35 >35	 Low Medium High Very high	Linear shrinkage, LS (%) (Attimeyer,1956) <5 5-8 >8	 Non-critical Marginal Critical
Liquid limit, LL (%) (Holtz & Gibbs, 1956) <35 35-50 50-70 >70	 Low Medium High Very high	Free swell (%) (Dawson, 1956) <50 >50	 Low High
Activity of clay (Skempton, 1953) >0.75 0.75-1.25 >1.25	 Low Medium High		

Appendice 19: General Guideline for sub-grade, sub base and base course for highway designs by Federal Ministry of Works (Roads and Bridges) (1997)

Parameter	Sub grade (%)	Sub base (%)	Base course (%)
LL	<35	<35	<30
PI	<12	<16	<13
% passing sieve 200	<35	<35	<35
CBR value (unsoaked)	>40	≥30	≥80
CBR value (soaked)	>15	>25	-
Relative compaction	>100	>100	>100

Appendice 20: Pictures showing the field work exercise and laboratory analysis of expansive soils derived from Imo Shale and Ameki Formation