Stabilization of Expansive Subgrade Soil Derived from Ameki Formation in Ozuitem, Southeastern Nigeria, Using Coconut Husk Ash

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Abstract

This research investigated the possibility of using Coconut husk ash (CHA) as an additive in the modification of the geotechnical properties of clay soil from Ameki formation in Ozuitem so as to increase their strength bearing capacity and reduce their swelling potentials. The clay soil samples were collected from Ozuitem LGA of Abia State. Chemical analysis of the CHA shows that it contains 36.04%, 13.19% and 20.89% of K2O, SiO2 and CaO respectively. Preliminary tests to determine the geotechnical properties of the natural soil including Particle size distribution, Atterberg limits, Maximum Dry Density and California Bearing Ratio were determined in the laboratory to classify the soil according to ASTM D2487-17, 2011 and Nigerian Standard FMWH, 2010 classification of soils. The soil samples were later stabilized with various percentages of CHA (2, 4, 6, 8 and 10). The geotechnical tests earlier performed on the soils were repeated (after stabilization) to evaluate the effects of CHA on the geotechnical properties of the clay soils. Results of the study indicate that CHA stabilization of clay soils in Ozuitem has the general effect of reducing the swelling indicators, thereby reducing the swelling potential of the soil. The liquid limit ranges between 34.08% to 50.03%, plastic limit ranges between 19.18% and 27.68%, and plasticity index is between 13.22% to 23.85%. Optimum stabilization were achieved with 8% CHA stabilization. MDD of the stabilized clay soil generally increased with an increase in percentage of CHA from 1.55 g/cm3 at 0% CHA to a maximum value of 2.40 g/cm3 at 10% CHA. This represents 54% increase in MDD of the soil. The CBR values increased from 7.15 kN/m2 to 81.25 kN/m2 (soaked) and 29.90 kN/m2 to 113.57 kN/m2 (unsoaked) respectively. The maximum strength was achieved at 10% CHA content. Stabilization with CHA acts like cement stabilization by increasing both the CBR and MDD. Therefore, coconut husk ash can be effectively used to reduce swelling indicators and to improve MDD and CBR values in clay soil and should be encouraged in the construction industry.

Keywords: *Expansive soils, geotechnical properties, liquid limit, plasticity index, coconut husk ash and stabilization.*

1.0 INTRODUCTION

The problem of expansive clays on road construction are problematic and cannot be unnoticed. This is due to the continuous alteration in soil volume which causes structures to move unevenly and crack resulting in possible failure. The high cost of building materials, which are now used in construction using conventional materials and methods, is one of the elements contributing to the issue. Using inexpensive building materials that are readily available in the area is one method to solve the issue. This can be accomplished by building homes with sustainable or recycled raw materials that are on par with traditional building materials composed of cement, sand, and aggregates to provide a suitable degree of comfort and quality.

This research determines the possibility of the stabilization of expansive clay using coconut husk ash. Clay soil is easily available in Nigeria and was be obtained from Ozuitem. The stabilized soil would replace the conventional soil normally used in building houses. This study is expected to have a significant impact on the road construction industry in terms of design, strength, durability and cost. It would also give a wider choice for designers and contractors on the availability of building materials in the market. This would result in stronger, more durable roads for transportation.

Expansive soil

Expansive soil is considered to be one of the most problematic soils and it causes damage to various civil engineering structures because of its swelling and shrinking potential when it comes into contact with water. Expansive soils behave differently from other normal soils due to their tendency to swell and shrink. Because of this swelling and shrinking behaviour, expansive soils may cause the following problems in structures or construction projects (Patel, 2019). These soils have a considerable propensity for swell and plasticity, which can cause instability in the infrastructure, foundation collapses, and structural damage (Al-Humairi et al, 2019). These soils' expansive character may lead to expensive repairs and safety hazards when buildings fracture, roadways deteriorate, and pipes burst. Lime, cement, or chemical additions are frequently used in traditional soil stabilization techniques to reduce the expansive nature of soils. Various approaches could be limited regarding long-term performance, environmental impact, and cost-effectiveness (Munusamy et al, 2021). A rising number of people are interested in investigating alternate approaches that use waste materials to stabilize soil, (Mohammed et al, 2020), (Al-Sanjary et al, 2019) which aligns with environment friendly and sustainable engineering techniques (Xiang et al, 2023).

The main problem with expansive soil is their propensity to experience volumetric changes in reaction to changes in moisture content (Wei et al, 2023). These alterations can potentially harm infrastructure, such as roads, retaining walls, buildings, and pipelines, posing a risk to public safety and causing large financial losses (Usta, 2023). Historically, methods for stabilizing the soil have been used to lessen the negative consequences of expansive soils by altering their characteristics and making them less vulnerable to volume fluctuations. Adding chemical agents like lime, cement, or fly ash (FA) is common in conventional stabilizing techniques (Zamin, 2024), (Subramanyam, 2024). While this approach can be beneficial, it can also be costly and environmentally problematic. Managing trash from diverse industrial operations impedes environmental sustainability and limits landfill space.

These materials, which are leftovers from industrial operations, are a viable way to address waste management and soil stability issues (Mohamed et al, 2023). In addition to providing a long-term solution to soil engineering issues, using these industrial wastes in soil stabilization meets the urgent demand for waste management and environmental protection (Suiyi, et al, 2024).

Coconut husk ash is an example of such waste. To solve the issue of environmental pollution and efficiently manage large volumes of garbage and other waste, these are used to enhance the strength characteristics of weak expansive soils (Ray et al, 2023). It has enormous potential to improve the strength of weak soils used in several civil engineering applications. As these wastes are available in plenty at zero cost, their application for strengthening weak soils is the most economical proposition (Kettenhuber et al, 2023). The potential of these soil-stabilizing materials is thus required to be investigated for their optimum use.

Expansive clay

Clays belong to sedimentary deposits and are defined as soils with particles of colloidal dimensions. Basically, they consist of hydrated aluminosilicates, but due to variations in structure and mineral composition, more than 30 clay minerals have been defined, each with its own properties. Furthermore, their properties change significantly with changes in the conditions in which they are found. Soil moisture is one of the most important parameters and can quickly and dramatically affect the characteristics of clay. Analyzing clays and their characteristics is therefore extremely important in geotechnical investigation and testing and enables the assessment of soil stability and the understanding of processes that can take place during and after engineering interventions. (Crowley, 2007)

Based on similar properties, clays are divided into groups, one of which is smectites. Smectites include clays with the property of a large change in volume following a change in humidity — expansive clays. The most famous minerals of this group are bentonite, montmorillonite, beidellite and nontronite.

With precipitation and seasonal weather changes, there is a change in the amount of water in the soil. During a wet period, smectite clays expand and their volume can increase by more than 10%. As the volume of clay increases, it begins to exert pressure on structures, and the pressure is high enough to result in the appearance of cracks over time. With the return of the dry period, clays lose water and their volume decreases. The compaction of the clay further causes cracks to appear, which enables deeper penetration of the next inflow of water, and thus greater expansion of the clay. (Crowley, 2007)

In this way, expansive clays negatively affect foundations, basement walls and roads, as well as underground and mining works. The impact of these clays can be reduced by determining their location and distribution, stabilizing the terrain, replacing materials, using deeper foundations and controlling soil moisture.

Pozzolanas

Pozzolans are defined as siliceous and aluminous materials which themselves possess little or no cementitious value but will, in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (Robert, 1993). More recent development has shown the potentials of utilizing agricultural wastes for producing cementitious compounds in conjunction with lime. Many plants, during their growth pick up silica from the ground, into the structure of their leaves, stalks and other parts (Akinmade, 2008). Therefore, when such agricultural products are burnt, organic materials which are the largest proportion are broken down and disappear as carbon dioxide, and water vapour (Akinmade, 2008). The resulting ash consists mainly of inorganic residue, mostly silica. When silica is in its reactive form, it can combine with lime to form insoluble cementitious compounds (Smith, 1992).

Chemical principles of pozzolanic reactions

Pozzolanic reactions take place when significant quantities of reactive CaO, Al2O3 and SiO2 are mixed in presence of water (Seco et al, 2012). Usually CaO is added as lime or cement, meanwhile Al2O3 and SiO2 can be present in the material to develop cementation gels to be added as cement or, for example, with a pozzolana. In this process the hydration of the CaO causes an increase in pH values to approximate 12.4 (Seco et al, 2012). Under these conditions pozzolanic reactions occur: the Si and Al combine with the available Ca, resulting in cementitious compounds called Calcium Silicate Hydrates (CSH) and Calcium Aluminate Hydrates (CAH) (Nalbantoglu et al, 2007).

A simple qualitative representation of these reactions is summarized below:

$$Ca(OH)_2 \rightarrow Ca^{2+} + 2OH^- \quad (1.11)$$

$$Ca^{2+} + 2OH^- + SiO_2 \rightarrow CSH \quad (1.12)$$

$$Ca^{2+} + 2OH^- + Al_2O_3 \rightarrow CAH \quad (1.13)$$

These compounds are responsible for improving the mechanical properties of the mix, due to increasing development of pozzolanic reactions over time; some authors stated that this may take place over years (Wild et al, 1998). Where these oxides are not available in sufficient quantities in the materials to be cemented, they must be incorporated with the binder (Seco et al, 2012). In these cases, it is particularly advantageous to use stabilisers, like OPC, those rich in SiO_2 and Al_2O_3 as well as in CaO (Wild et al, 1998); (Degirmenci et al, 2007) or for example the use of lime and pozzolana mixes. When these oxides are present in a material that needs to be cemented, it is not necessary to add them as a binder. This situation is most usual in stabilization of clay soils where oxides are present in matrix. They are naturally rich in Si and Al oxides, which become soluble at high pH conditions and then become available for development of the pozzolanic reactions (Seco et al, 2012). The improved mechanical capacities achieved in each case depend on the amount, reactivity and concentration of the oxides, the size and shape of the particles and also on the curing conditions (Goktepe et al., 2008).

Fineness of pozzolanas

An essential physical property of a cementing material that affects its affinity for water is its fineness. The activity of pozzolanas is increased by fine grinding.

Use of pozzolana in soil improvement

In order to minimize the high cost of soil improvement when conventional additives are used, geotechnical Engineers (Cokca et al, 2004) have focused on pozzolanas for use as substitute or partial replacement for the standard stabilizers. A large amount of these materials are obtained from agricultural wastes. When plant residues are burnt, organic materials which are the largest constituents are broken down and disappear as carbon dioxide and water vapour. The ash which remains contains mostly inorganic residue, notable silica in amorphous form, which can react with oxides in the soil, thus aiding improvement of the soil properties.

Soil Stabilization

Soil stabilization is the permanent physical and chemical alteration of soils to enhance their physical properties (Mekonnen et al, 2020). In its broadest senses, it includes compaction, preconsolidation, drainage, and many such processes. However, the term stabilization is generally restricted to the process which alters the soil material itself for improvement of its properties (Mekonnen et al, 2020). It is the collective term for any physical, chemical, or biological method, or combination of such methods employed to improve certain properties of natural oil to make it serve for intended engineering purposes (Clough and Duncan, 1991). Soils may be stabilised to increase their strength and durability or to prevent erosion and dust generation (Onyelowe et al, 2012). Soil stabilisation deals with the physical, physico-chemical and chemical methods to make the stabilised soil to serve its purpose as pavement component material (Koteswara, 2011). Primarily, the objectives of stabilisation are to improve soil strength, to decrease permeability and water absorption and to improve bearing capacity and durability under cyclical conditions such as varying moisture content (Akinmade, 2008).

2.0 Study Area Description

2.1 Location and Climate

The study area is located along Umuahia – Bende road, it lies between latitudes 3° and 6°N and longitudes 5° and 8°E (Nwajide, 2013; Figure 1). The climate of the area alternates between the dry and the rainy season and is located in the south-eastern part of Nigeria. It has a total mean annual rainfall of over 140mm spread over the month of March to October. The highest peak of rainy season is observed in June. The dry season is from November to February. The temperature is high throughout the year, with annual range of 23°C to 32°C, with the hottest period occurring in dry season. The area has a rainforest type of vegetation that comprises of different species of tall forest trees, shrubs and grasses. The topography of the area has so many gentle hills and valleys scattered in different areas.

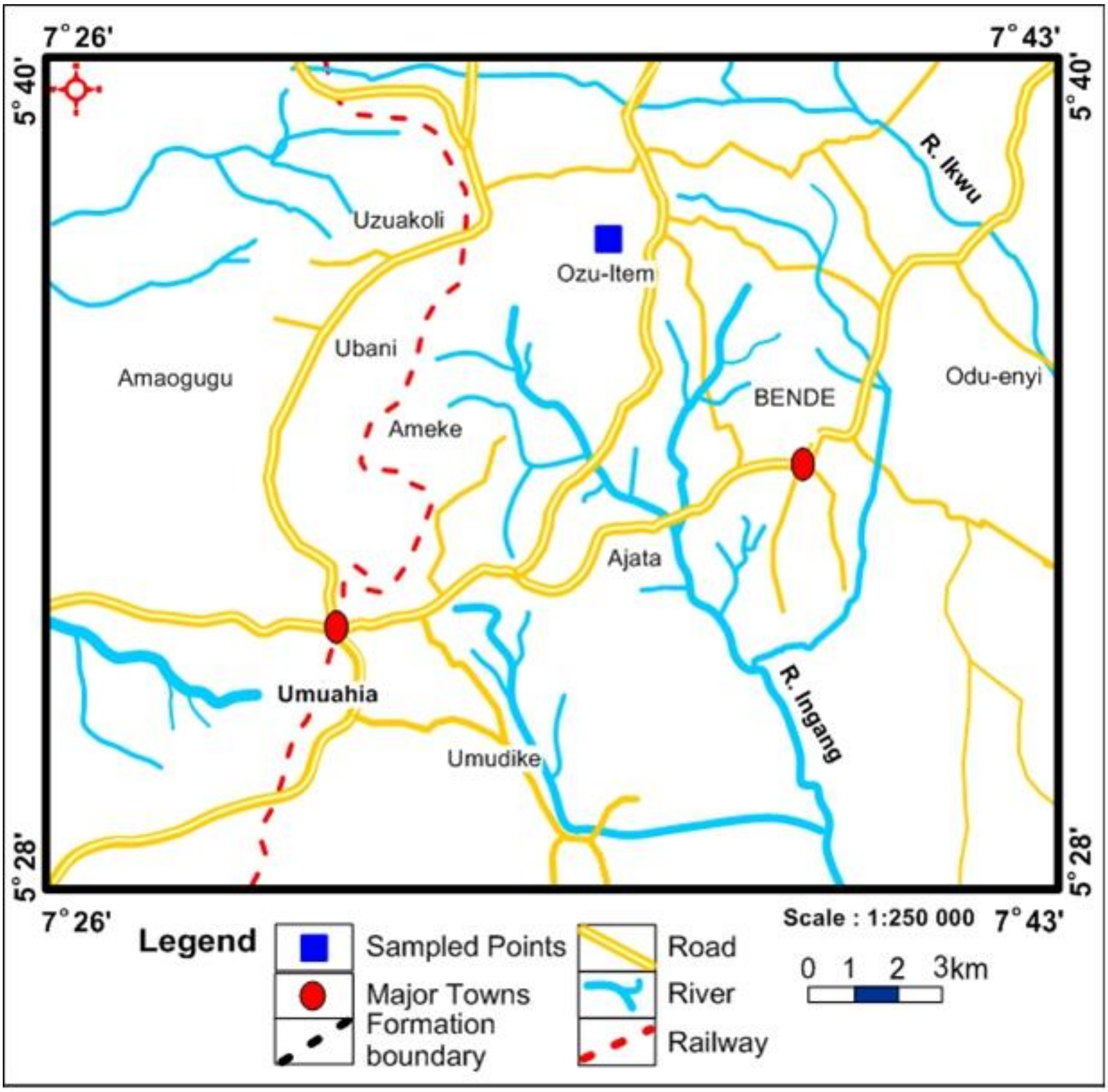

Fig 1: Location map of the study area (modified from Chiaghanam et al, 2017)

2.2 Geology of the Study Area

The study area lies within the Northern Depo Belt of the Niger Delta Basin in Southeastern Nigeria. The Niger Delta Basin is geographically situated in the Gulf of Guinea and delineated by latitudes 30 and 60N and longitudes 50 and 80E (Nwajide, 2013; Figure 1). The basin is bounded to the west by the Okitipupa Ridge and demarcated from the Anambra Basin to the north by an unconformity. The eastern limit is defined by the Cameroun volcanic line, while the southern limit is the Guinea Abyssal plain. The establishment of the Niger Delta Basin followed the subsidence of the southern area of the Benue Trough during the Danian as a result of thermal contraction of the lithosphere (Sleep, 1971, Nwajide, 2013). According to Nwajide (2013), this subsidence induced major marine transgression of the Early Paleocene that paved the way for the accumulation of the basal beds of the Niger Delta basin.

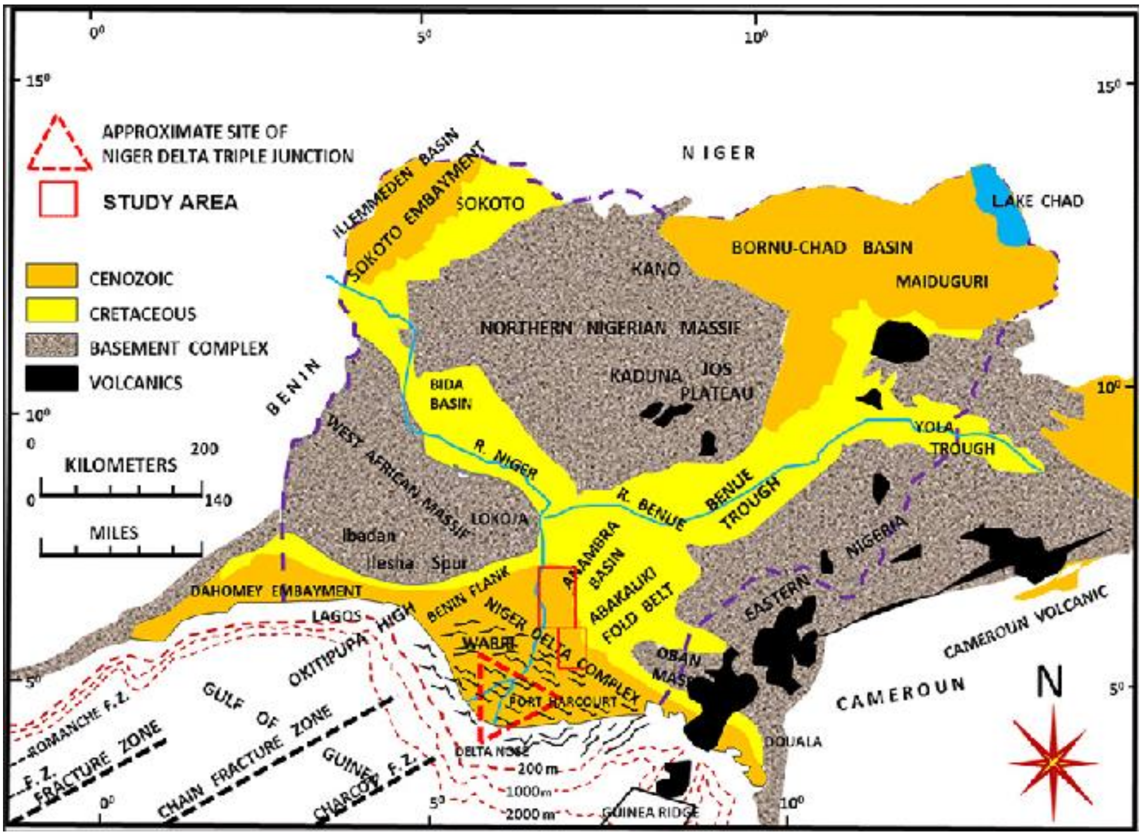

Figure 2. Structural map of the Nigerian Sedimentary Basin showing the chain and Charcot oceanic fracture zone (modified after Murat, 1972).

The succession of the Niger Delta consists of the Paleogene system which is comprised of the Imo Formation, Ameki Group, and Ogwashi Formation (Figure 1), with a composite thickness of about 3,500 m.

The Ameki, Nanka and Nsugbe formations are the lateral equivalents of the Ameki Group (Nwajide, 1980) the formation conformably overlies the Imo Formation. Reyment (1965) described the type locality of the formation between miles 73 to 87 along the section left behind during the eastern railway construction at Ameki Town. The Ameki Formation, which is composed of rapidly alternating shale, sandy shale, mudstone, clayey sandstone (Adegoke et al., 1980; Arua, 1980) and fine-grained fossiliferous sandstone with thin limestone beds (Reyment, 1965), represents most of the Eocene. The age of the formation has been variously considered to be Early Eocene (Reyment, 1965) and Early Middle Eocene (Lutetian) (Adegoke, 1969).The depositional environment has been interpreted as estuarine, lagoonal, and open marine, based on the faunal content. White (1926) interpreted an estuarine environment for the formation because of the presence of certain fish species of a known estuarine affinity. Adegoke (1969), however, indicated that the fish were probably washed into the Ameki Sea from inland waters, and preferred an open marine depositional environment. Nwajide (1980) and Arua (1986) suggested environments that ranged from nearshore (barrier ridge-lagoonal complex) to intertidal and subtidal zones of the shelf environments, whereas Fayose and Ola (1990) suggested that the sediments were deposited in marine waters of depths between 10 m and 100 m. Ekwenye et al. (2017) used the concept of facies analysis and sequence stratigraphy to suggest a tide-dominated estuarine system for the Ameki Group.

Table 1: Stratigraphic column of Niger Delta Basin (modified after Frankyl and Cordry, 1967)

Age			Group	Niger Delta Basin This Study	
				Down-dip	Up-dip
Quaternary				Alluvium	
NEOGENE	Pliocene		Benin Group	Benin Formation	Benin Formation
	Miocene				
PALEOGENE	Oligocene		Ameki Group	Upper Agbada Formation	Ogwashi Formation
	Eocene	L			
		M		Lower Agbada Formation	Ameki Fm/ Nanka Sand/ Nsugbe Sst
		E			
	Paleocene		Imo Group	Akata Formation	Imo Fm/ Umuna Sst/ Ebenebe Sst
	Danian				
LATE CRETACEOUS	Late Maastrichtian		Coal Measures Group	Major Unconformity	

Major Unconformity

L =Late; M= Middle; E= Early; sst=Sandstone; Fm=Formation

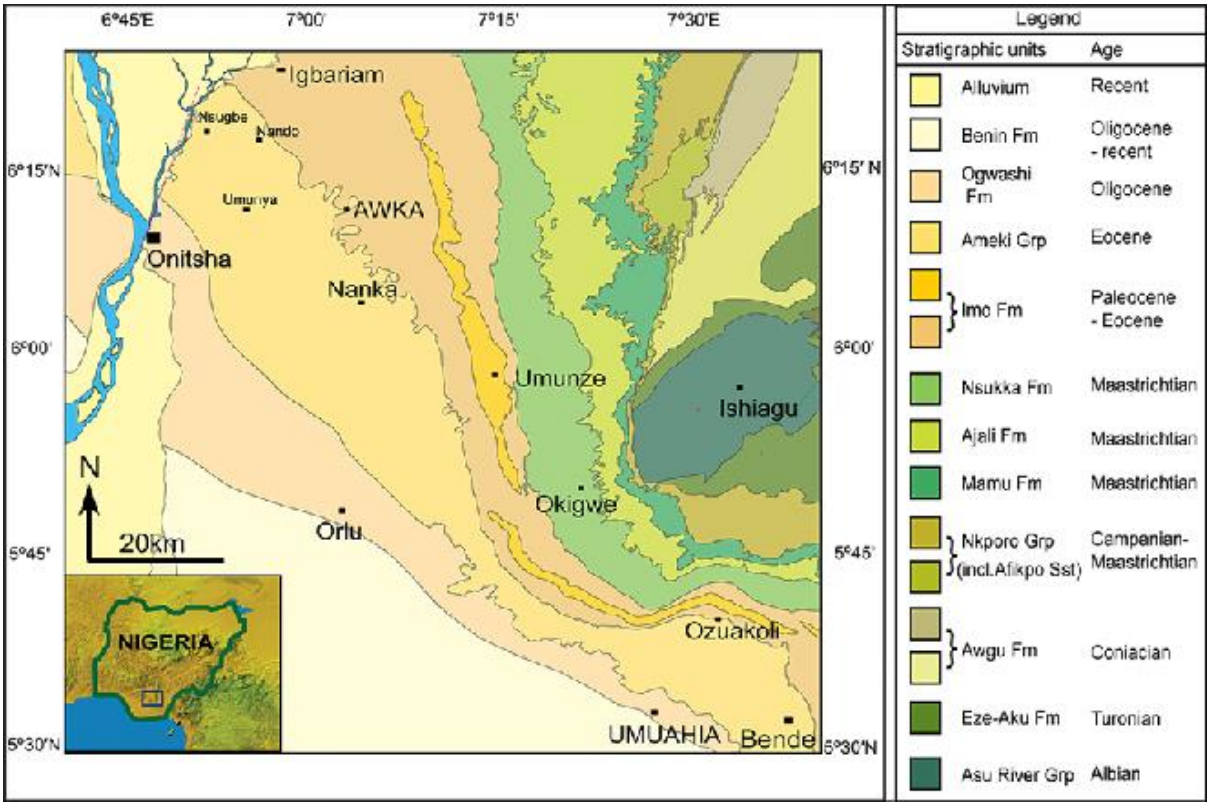

Fig 3: Geologic map of South-eastern Nigeria showing the Paleogene formation (modified from Ekwenye, 2014).

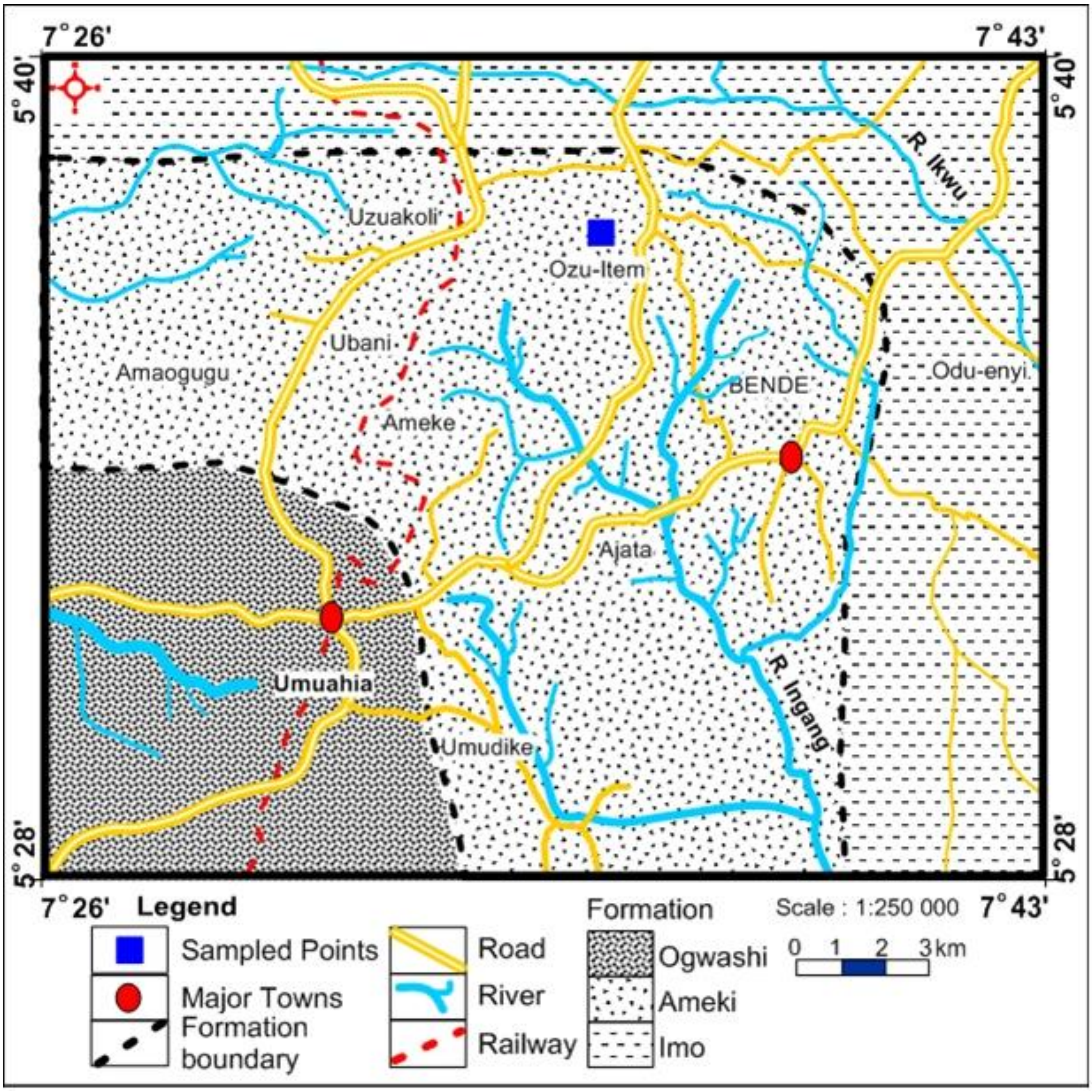

Fig 4: Geologic map of the study area (modified from Chiaghanam et al, 2017).

3.0 Materials and Methods

3.1. Field studies and sample collection

Desk work: This research is conducted by means of literature review on expansive soil, which forms the desk work of this study. Different literatures and maps of the study area were deeply studied to understand the geology of the area and to assist in area where samples for this study would be collected.

Reconnaissance survey: This is one of the first steps of every research work, it gives firsthand information of what to expect. On a reconnaissance survey visit to the area, different geological formations which are necessary for this study where identified and the hardship posed by the soils in the study area were also observed.

Collection of Soil Samples:

The equipments used for samples collection in the field include:

Global Positioning System (GPS): To obtain the coordinates of sampled points and direct location on the map, GPS was used.
Nylon Bags: These were used to collect, seal and label samples and also preserve the natural moisture content and thereafter taken to the various laboratories for laboratory analysis.
Camera: It was used to take pictures of the exposures, samples and the locations of samples.
Shovel: It was used to dig and shovel out the soil.
Field notebook: it was used to record all geologic data and observation.
Maps: Where used for easy location of villages and towns, and sample locations.
Computer: Excel application was used to plot graphs for determining atterberg limits and other plots.
Other field equipment used consists of hammer, tapes.
Soil Sample: Clay Soil: Ameki Formation. Clay Soil was collected from Ozuitem
Collection of Coconut Husk Ash: CHA is a waste generated from burning of coconut husk. The coconut husk to be used for the investigation was generated from local sources in Ihiagwa. The coconut shell was sun dried for forty eight hours to remove moisture from it. It was then subjected to uncontrolled combustion using open air burning for three hours and allowed to cool for about 12hours. The burnt ash was then collected, open air dried and sieved through a BS sieve (75 microns). The resulting ash, which has the required fineness, was collected for use.

3.2 Laboratory Tests

The tests carried out in the laboratory include:

3.2.1 Determination of Major Oxide Composition of Coconut Husk Ash

Major oxide tests; geochemical tests were carried out on the CHA samples in the laboratory. Geochemical tests were carried out using the Atomic Absorption Spectrophotometry (AAS) (Loring and Ramtala, 1992). The quantitative analysis of the percentage composition of silica oxide and other chemical compound such as Cao, Al2O3, Fe2O3 and so on, were carried out on the coconut husk ash at Asemabot Analytical Services in Kaduna, Nigeria. The tests were done twice to justify the exact quantity of the oxides.

3.2.2 Soil Stabilization Tests

3.2.2.1 Atterberg Limit Tests

Atterberg limit test is the primary measures of critical water content of soil that are fine grained. It is used to determine the linear shrinkage, plastic limit and liquid limit. As dry expansive /lateritic soils take on increasing amount of water, it behaves in different ways depending on the amount of water taken. It can either behave as solid, semi-solid, plastic or liquid. In each case, the consistency or behavior of the soil is different so also is the engineering properties.
These differences in behavior are used to evaluate soils which structures can be erected on or soils to be avoided or treated before erecting any structure. As expansive soils take in water when wet and expand in volume, the expansion is related to the ability of the soil to take in water and its mineralogical and structural make up. The test is mainly carried out on silt and clay soils which have the ability to expand/shrink when water is either added or removed from them.
Soil become crumbly and friable when dry, semi-solid when particular quantity is added, if expansive the soil begins to swell in volume as the moisture content is exceeded. Increasing the water above the soil plastic limit, the soil will be transformed into a malleable plastic mass

which causes additional swelling. The soil in this plastic state until its liquid limit is exceeded which transforms it into a viscous liquid that can flow.

Apparatus

- Dry soil samples
- Agate mortar
- Sieve 425 microns
- Non porous glass plate
- Spatula
- Casagrande and groove tool
- Linear shrinkage mould
- Wash bottle or aspirator
- Measuring tape
- Oven

3.2.2.2 Bulk Density and Dry Density test

Bulk density is the weight of the soil for a particular given volume. It is used to measure compaction. Generally, the greater the density of soil, the less pore space for water movement and penetration.

Apparatus

- 3 inch diameter ring
- Flat blade knife
- Sealable bags and marker pen
- Scale (0.01g precision)
- Oven
- Known volume of soil

Procedure

The empty ring is weighed and recorded, and then the ring was filled with the soil sample and leveled with a flat blade knife and then it is weighed and recorded. The known volume is also recorded.

Calculation

$$Bulk\ density\ (y) = \frac{Weight\ of\ sample}{Volume\ of\ sample} \tag{3.1}$$

Dry density of soil can be derived by knowing the moisture content and bulk density with the formula:

$$Yd = \frac{Y}{1+w} \tag{3.2}$$

Where,

Yd = dry density of soil

Y = bulk density of soil

W =water content expressed as a fraction

3.2.2.3. West African compaction test

This test is done to determine the maximum dry density and the optimum moisture content of soil. There three (3) methods used for compaction which include:

i. Standard proctor test.
ii. Modified AASHTO method.
iii. West Africa method.

Tools

i $Cylindrical\ metal\ mould -\ 935cc\ volume.$
ii Weigh balance – one of 10kg capacity, sensitive to 1g and the other of 200g capacity, sensitive to 0.01g.
iii Oven – thermostatically controlled with an interior of non-corroding material to maintain temperature between 105°C and 110°C
iv Steel straightedge – 30cm long
v IS sieves of sizes – 4.75mm, 19mm and 37.5mm

Preparation of sample

A representative portion of air-dried soil material, large enough to provide about 6kg of material passing through a 19mm IS sieve (for soils not susceptible to crushing during compaction) or about 15kg of material passing through a 19mm IS sieve (for soils susceptible to crushing during compaction), should be taken. This portion should be sieved through a 19mm IS sieve and the coarse fraction rejected after its proportion of the total sample has been recorded. Aggregations of particles should be broken down so that if the sample was sieved through a 4.75mm IS sieve, only separated individual particles would be retained.

Procedure to determine the maximum dry density and optimum moisture content of soil

a). Soil not susceptible to crushing during compaction

i. A 5kg of air-dried soil passing through the 19mm IS sieve should be taken. The sample should be mixed thoroughly with a suitable amount of water depending on the soil type (for sandy and gravely soil – 3% to 5% and for cohesive soil – 12% to 16% below the plastic limit). The soil sample should be stored in a sealed container for a minimum period of 16hrs.
ii. The mould of 1000cc capacity with base plate attached should be weighed to the nearest 1g (W1). The mould should be placed on a solid base, such as a concrete floor or plinth and the moist soil should be compacted into the mould, with the extension attached, in five layers of approximately equal mass, each layer being given 25 blows from the 4.9kg rammer dropped from a height of 450mm above the soil. The blows should be distributed uniformly over the surface of each layer. The amount of soil used should be sufficient to fill the mould, leaving not more than about 6mm to be struck off when the extension is removed. The extension should be removed and the compacted soil should be leveled off carefully to the top of the mould by means of the straight edge. The mould and the soil should then be weighed to the nearest gram (W2).

iii. The compacted soil specimen should be removed from the mould and placed onto the mixing tray. The water content (W) of a representative sample of the specimen should be determined.

iv. The remaining soil specimen should be broken up, rubbed through 19mm IS sieve and then mixed with the remaining original sample. Suitable increments of water should be added successively and mixed into the sample, and the above operations i.e. ii) to iv) should be repeated for each increment of water added. The total number of determinations made should be at least five and the moisture contents should be such that the optimum moisture content, at which the maximum dry density occurs, lies within that range.

b) Soil susceptible to crushing during compaction

Five or more 2.5kg samples of air-dried soil passing through the 19mm IS sieve, should be taken. The samples should each be mixed thoroughly with different amounts of water and stored in a sealed container as mentioned in Part a).

c) Compaction in large size mould

For compacting soil containing coarse material up to 37.5mm size, the 2250cc mould should be used. A sample weighing about 30kg and passing through the 37.5mm IS sieve is used for the test. Soil is compacted in five layers, each layer being given 55 blows of the 4.9kg rammer. The rest of the procedure is same as above.

Reporting of result

Bulk density (γ) in g/cc of each compacted specimen should be calculated from the equation below:

$$Bulk\ density\ (\gamma) = (W2 - W1)\ /\ V \qquad (3.3)$$

Where, V = volume in cc of the mould.

The dry density γd in g/cc

$\gamma d = 100\gamma / (100 + W)$

Where,

γd = dry density of soil

γ = bulk density of soil

W = water content expressed as a fraction

The dry densities, γd obtained in a series of determinations should be plotted against the corresponding moisture contents, W. A smooth curve should be drawn through the resulting points and the position of the maximum on the curve should be determined. The dry density in g/cc corresponding to the maximum point on the moisture content/dry density curve should be reported as the maximum dry density to the nearest 0.01. The percentage moisture content corresponding to the maximum dry density on the moisture content/dry density curve should be reported as the optimum moisture content and quoted to the nearest 0.2 for values below

5%, to the nearest 0.5 for values from 5% to 10% and to the nearest whole number for values exceeding 10%.

4.0 Results and Discussion

4.1. Results

4.1.1 Chemical Composition of Coconut Husk Ash

Table 1 shows the chemical composition of Coconut Husk Ash (CHA). The cementing characteristics of CHA are dependent on its oxide composition. This reveals that the coconut husk ash contains large percentage of K2O (36.04%) followed by CaO (20.89%), which corroborate the fact that coconut husk ash is a pozzolanic material.

Table 1: Chemical composition of coconut husk ash

Oxide	**Percentage Composition CHA(%)**
SiO_2	13.19
TiO_2	0.25
Al_2O_3	0.80
Fe_2O_3	1.64
CaO	20.89
MgO	2.90
Na_2O	0.003
K_2O	36.04
P_2O_5	2.53
MnO	0.08
Ag_2O	0.11
SO3	2.05
Rb_2O	0.12
Cl	19.16
ZnO	0.17

4.1.2. Properties of the Natural Clay Soil

A summary of the geotechnical properties of the clay soil in its natural state is presented in Table 2.

Table 2: Summary of the geotechnical properties of the clay soil

As seen below, the natural clay soil with LL of 63.8 %), PI of 30.55 (%) has a high degree of expansion based on ASTM D2487-17, 2011 and Nigerian Standard FMWH, 2010 classification of soils.

Experiments	Clay Soil
Liquid Limit (%)	63.80
Plastic Limit (%)	33.25
Plasticity Index (%)	30.55
Linear Shrinkage (%)	17.20
MDD (g/cc)	1.55
CBR Soaked (%)	5.00
CBR Unsoaked (%)	15.00

4.1.3. Variation of Liquid Limit for Ozuitem Clay with Increased Percentage of CHA

When the clay soil sample derived from Ozuitem was treated with CHA at different percentages. The maximum amount of additive to achieve hardening was obtained at 4% (Fig 5).

The Liquid limit for the stabilized soil ranges from 34.08 % to 50.03%. Federal Ministry of Works and Housing (1972) for road works recommend liquid limits of 50% (35%) maximum for sub-base and base materials. All the values at 2-10% except that of 6% fall under the values recommended by Federal Ministry of Works and Housing therefore renders the soil suitable for use as sub-base and base materials.

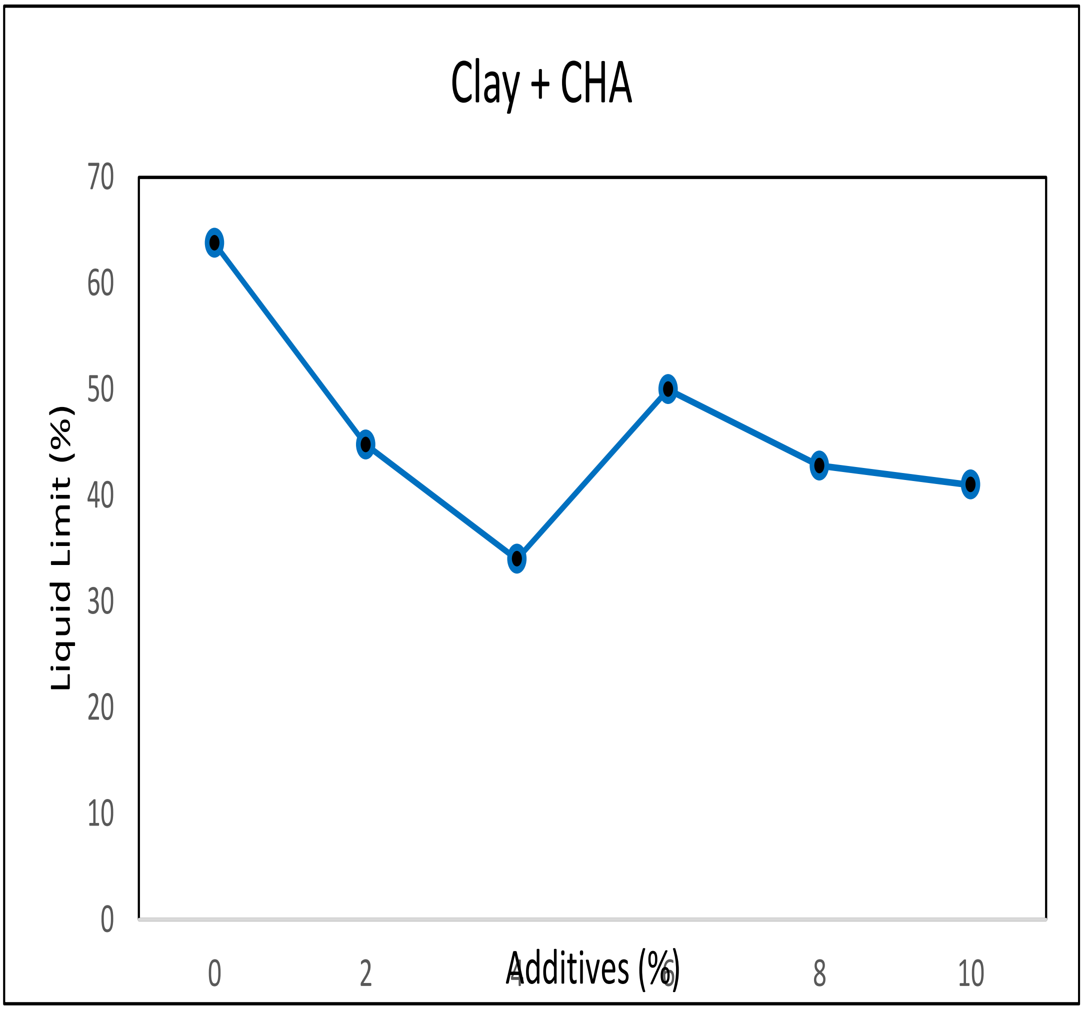

Figure 5: Variation of Liquid limit for Ozuitem Clay with increase in dosage of CHA

4.1.4. Variation of Plasticity Index for Ozuitem Clay with Increased Percentage of CHA
Plasticity ranges from 13.22% to 23.85% (Fig 6). Federal Ministry of Works and Housing (1972) for road works recommend plasticity index of 10% maximum for sub-base and base materials. All the percentages of clay soil stabilized at 2-10% values are more than the maximum values recommended by Federal Ministry of Works and Housing therefore renders the soil unsuitable for use as sub-base and base materials.

The minimum plasticity index was obtained at 4% CHA as shown in Figure 6.

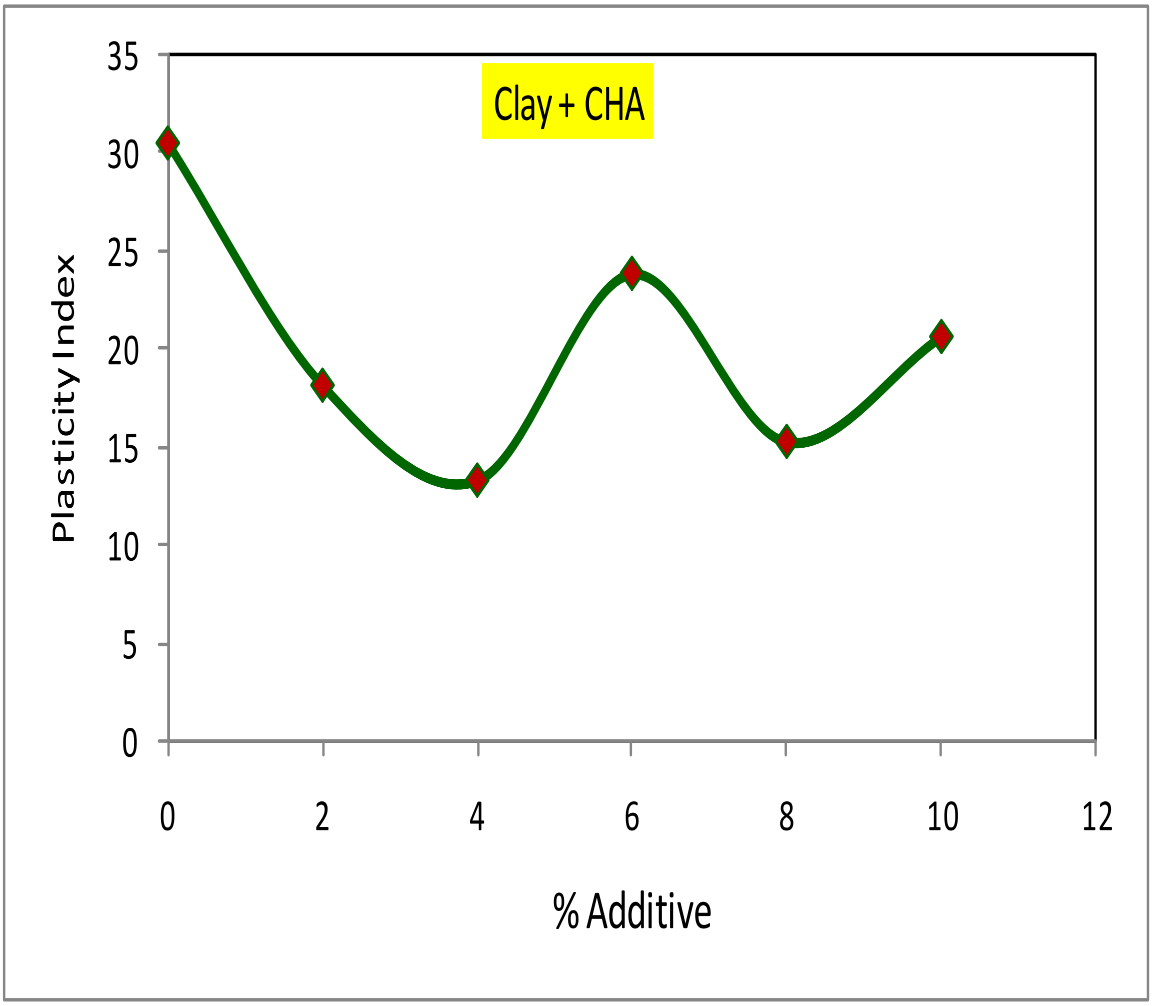

Figure 6: Variation of Plasticity Index for Ozuitem Clay with increase in dosage of CHA

4.1.4. Maximum Dry Density (MDD) for Ozuitem Clay at various percentages of CHA

Variation of Maximum Dry Density (MDD) for Clay at various percentages of CHA is presented in Figure 7. From the Figure, it can be observed that at constant dosage of CHA, MDD of the stabilized clay soil generally increased with an increase in percentage of CHA from 1.55 g/cm3 at 0% CHA to a maximum value of 2.40 g/cm3 at 10% CHA. This represents 54% increase in MDD of the soil. The increase in MDD of the treated soil may also be due to CHA chemically reacting with clay particles, creating bonds that improve particle arrangement and increase density. This observed trend of MDD variation is similar to that of cement reported by (Ogbuchukwu & Okeke, 2021).

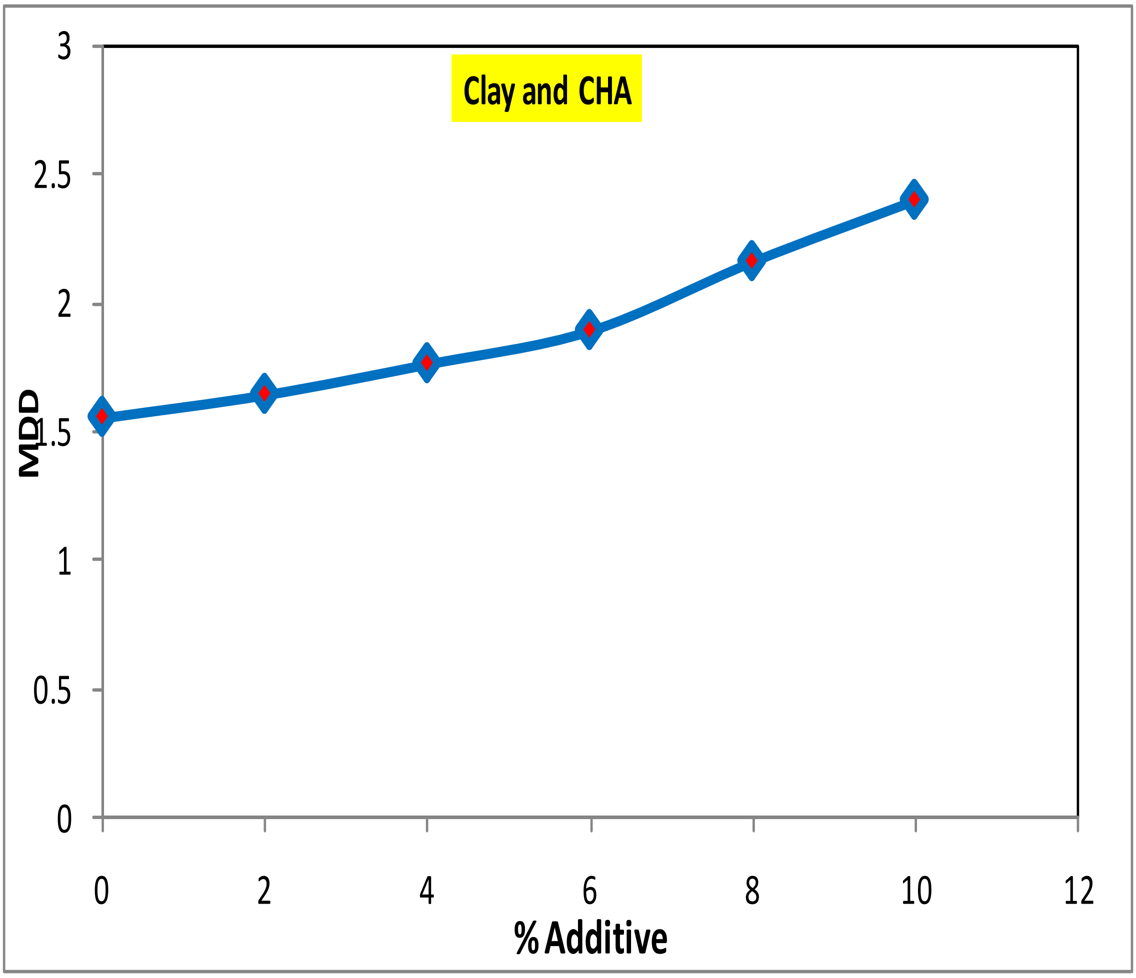

Figure 7: Variation of Maximum Dry Density for Ozuitem Clay with increase in dosage of CHA

4.1.5. California Bearing ratio (CBR) Variations for Ozuitem Clay with Higher Dosage of CHA

Additives Variations of California Bearing ratio (CBR) of the clay soil with varying dosages of CHA are shown in Figures 8 at 2, 4, 6, 8, 10%, respectively.

A general pattern was observed from the Figure 8 which show the CBR values (soaked and unsoaked) for Clay with CHA increased from 7.15 kN/m2 to 81.25 kN/m2 (soaked) and 29.90 kN/m2 to 113.57 kN/m2 (unsoaked) respectively. The maximum strength was achieved at 10% CHA contents. It was observed that as the percentage composition of CHA increased, CBR

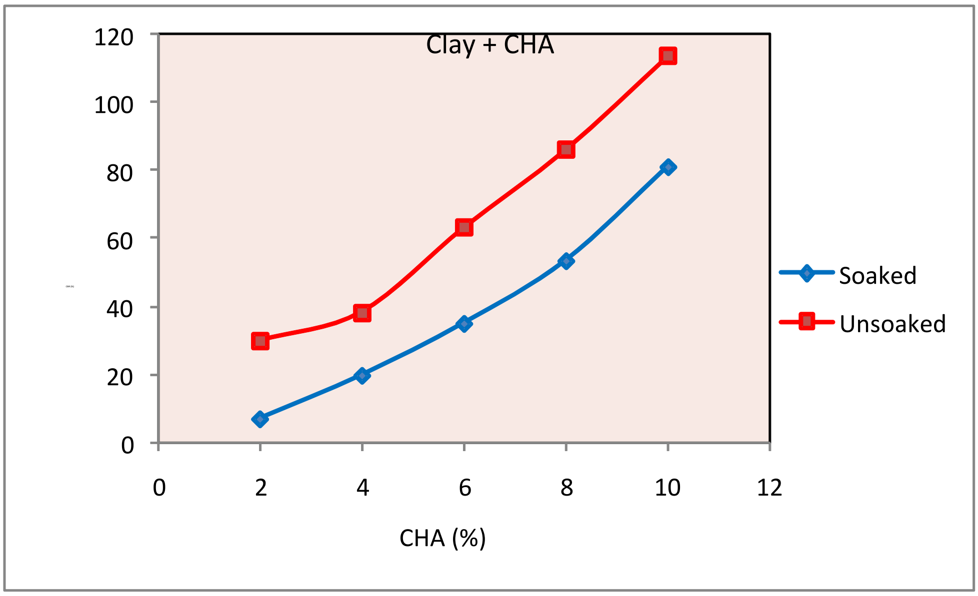

Figure 8: Variations of California Bearing ratio (CBR) values of clay with varying dosages of CHA
(soaked and unsoaked) of the treated soil increased.

4.2. Discussion

The chemical compounds that were significantly present in the CHA included Potassium oxide (36.04%), Silicon dioxide (13.19%), Chlorine (19.16%), Calcium oxide (20.89%), Phosphorus pentoxide (2.53%), Magnesium oxide (2.90%), and Aluminium (III) oxide (0.80%), which together total 95.5%. These chemical compounds are usually present in materials that are classified as pozzolans (ASTM C618, 2014), indicating, therefore, that CHA is a pozzolanic material.

The results of the Atterberg limits tests indicate that the natural clay soil has a liquid limit of 63.8%, plastic limit of 33.25% and plasticity index of 30.55%; a high degree of expansion based on ASTM D2487-17, 2011 and Nigerian Standard FMWH, 2010 classification of soils. All the values at 2-10% except that of 6% fall under the values recommended by Federal Ministry of Works and Housing therefore renders the soil suitable for use as sub-base and base materials.All the percentages of clay soil stabilized at 2-10% values are more than the maximum values recommended by Federal Ministry of Works and Housing therefore renders the soil unsuitable for use as sub-base and base materials.

For all combinations of the clay soil and CHA, MDD increased with increasing CHA content, as shown in Figure 4.3. This conforms to findings of other researchers in similar investigations (Ogbuchukwu & Okeke, 2021).

The increase in MDD of the treated soil may be due to CHA chemically reacting with clay particles, creating bonds that improve particle arrangement and increase density.

CBR values (soaked and unsoaked) for Clay with CHA increased from 7.15 kN/m2 to 81.25 kN/m2 (soaked) and 29.90 kN/m2 to 113.57 kN/m2 (unsoaked) respectively. The maximum strength was achieved at 10% CHA contents. It was observed that as the percentage composition of CHA increased, CBR (soaked and unsoaked) of the treated soil increased.

5.0 Conclusion

Natural soil clay from Ozuitem was geotechnically analysed in the laboratory to evaluate the degree of swelling behavior and strength characteristics of the soils. Several types of tests including liquid limit, plastic limit, plasticity index, maximum dry density and California bearing ratio were conducted. The natural geotechnical properties of the clay soil indicate that the clay soil has a high degree of expansion based on ASDM classification of soils.

Stabilization was done using coconut husk ash at percentages of 2,4,6,8,10 to reduce the swelling behavior and increase the strength characteristics of the clay soils. Results of the analysis indicate that CHA stabilization of expansive clay soils in Ozuitem has the general effect of reducing the swelling indicators (liquid limits, plasticity index) thereby reducing the swelling potential of the soil or tendency of the soil to swell in the presence of water. Stabilization with CHA acts like cement stabilization by increasing both the CBR and MDD. Optimum stabilization, that is, minimum values of swelling indicators were achieved with 8% CHA stabilization. Therefore, the use of CHA alone is suitable to reduce the swelling potential of soils with high liquid limit and high plasticity index

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