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Detailed Investigation of Index Properties and Rate of Consolidations of Nguzu Edda Lateritic Soils (South-Eastern Nigeria) Used for Road Construction

Nwadike Henry Onyebuchi, Nweke Obisi Mathias

Department of Geology, Ebonyi State University, Abakaliki, Nigeria

Email address:
nwadikeh@gmail.com (N. H. Onyebuchi)

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Abstract: This research project involves detailed investigation of index properties and rate of consolidations of Nguzu Edda lateritic soil used for road construction so as to give details of soil behaviour in solving engineering and environmental issues. Nguzu Edda lies within the Afikpo Syncline of the Southern Benue Trough, Nigeria. It is located within longitude 7°49^1 E to 7°54^1 E and latitude 5°45^1 N to 5°50^1 N covering an area extent of about 62km^2. Seven (7) samples were collected for the geotechnical analysis. Some geotechnical parameters were used to assess the lateritic soils. All the analyses were carried out in accordance with the British Standard code of practice (BS1377:1990). The results indicated that the particle size gradation for gravel ranged from 58 to 89%, sand ranged from 2 to 10% and fines ranged from 9 to 35%; natural moisture content ranged from 4.95 to 12.03%; specific gravity ranged from 2.49 to 2.64; the bulk density ranged from 1662.1 to 1775.2 kg/m^3, dry density from 1498.7 to 1689.2 kg/m^3; the liquid limit ranged from 32 to 38%, plastic limit ranged from 21 to 25%, plasticity index ranged from 8 to 13%; linear shrinkage ranged from 10 to 16%; the maximum dry density ranged from 1.78 to 1.95 mg/m^3, optimum moisture content ranged from 9.1 to 20%; the undrained triaxial test for cohesion (C) ranged from 130 to 146 kN/m^2 and angle of internal friction (Ø) ranged from 15° to 20°; the values of free swell index ranged from 12 to 30.8%; the California bearing ratio values for unsoaked condition ranged from 51.29 to 87.50% while soaked California bearing ratio ranged from 33.48 to 62.50%. According to America Association of State Highway and Transport Officials (AASHTO) soil classification, all the samples can be classified as A-2 materials which consist of silty or clayey-gravel and sand and can be rated as excellent material for road construction. They satisfy Federal Ministry of Works and Housing requirements for subgrade and subbase materials. Samples S2, S6 and S7 may likely be suitable only for sub base course materials. The soil samples tested from the study area are adjudged suitable for sub grade, sub base and base course materials.

Keywords: Nguzu Edda, South-Eastern Nigeria, Lateritic Soil, Geotechnical, Nigerian Specification

1. Introduction

Lateritic soils are the most common reddish colour weathered pedogenic surface deposits occurring in the tropical and subtropical regions of the world. They constitute the most common materials for the construction of earth dams, highways, embankments, airfields as well as foundation materials to support structures in these areas [1]. The chemical composition and morphological characteristics of lateritic soils are influenced by the level of weathering of which the parent material has been subjected to [1]. Osula [2] defined laterite as a highly weathered tropical soil; rich in secondary oxides of combination of iron, aluminum and manganese. According to Alexander and Candy [3], it is nearly devoid of bases and primary silicate, but it may contain large amount of quartz and kaolinite. Another definition categorized lateritic soils into laterite, lateritic, and non-lateritic soils, depending on the silica (SiO_2) to sesquioxide ratios (Fe_2O_3, Al_2O_3), less than 1.33 are indicative of laterites, those between 1.33 and 2.00 show laterite and those greater than 2.00 of non-lateritic types [4]. As a road construction material, they form the sub-grade of most tropical road, and can also be used as sub-base and base courses for roads that carry light traffic. A literature review has revealed that the geo-technical characteristics and engineering behavior of red soils depend mainly on the genesis and degree of weathering (i.e. decomposition, laterization, desiccation and hardening). Morphological characteristics as well as the type and content of secondary minerals are other genetic characteristic [5]. Studies on the effect of weathering on the physical properties of lateritic soil by [6, 7] have revealed the following: pore-size distribution varies with the degree of weathering; higher pore volume and larger range of pore-size distribution indicates advancement in the weathering stage; soil classification and Atterberg limits do not show any correlation to weathering; high specific gravity is a good indication of advanced degree of weathering and soil aggregation increases with increasing weathering.

Several researchers have carried out works on the geotechnical properties of lateritic soils in various places. Aginam et al. [8] analyzed the engineering properties of lateritic soils from Anambra Central Zone, Nigeria. Accordingly, they revealed that lateritic soil from Anambra central zone can be classified under the America Association of State Highway and Transport Officials (AASHTO) [9] classification system as clayey gravel (A-2) indicating excellent material for road works. Arinze et al. [10] also assessed the geotechnical properties of Abia State lateritic deposits. They showed that the lateritic soils from Abia State are silty soil and very poor (A-4-1) based on America Association of State Highway and Transport Officials (AASHTO) [9] classification system.

Road accident is usually caused by bad roads as a result of wrong application of constructional materials especially laterite as sub-base and base material by construction companies [11, 12]. For a material to be used as either a base course or sub-base course depends on its strength in transmitting the axle-load to the sub-soil and or sub-grade (the mechanical interlock). The characteristics and durability of any constructional material is a function of its efficiency in response to the load applied on it [11, 12, 13]. The mineralogical composition of the lateritic soil has an influence on the geotechnical parameters such as specific gravity, shear strength, swelling potential, Atterberg limits, bearing capacity and petrographic properties [13]. The geology of the study area revealed that it is dominated by lateritic soils. The rate at which newly constructed roads in Nguzu Edda area developed cracks and later damage is worrisome. There have been published work on the detailed investigation of index properties and rate of consolidations of Nguzu Edda lateritic soil used for road construction. Hence this study will assess the index properties and rate of consolidations of Nguzu Edda lateritic soil for use as a base or sub-base material in solving engineering and environmental issues.

2. Physiography and Geology of the Study Area

2.1. Physiography

Nguzu Edda area is located in Afikpo South Local Government Area Ebonyi State. It is geographically located at the south-eastern part of Nigeria and lies within the Afikpo Syncline of the Southern Benue Trough (Fig. 1). It is located within longitude 7°49^{1}E to 7°54^{1}E and latitude 5°45^{1}N to 5^{0}50^{1}N (Fig. 2) covering an area extent of about 62km^2. It is heterogeneously split into two broad spectrums by the Okigwe- Udi ranges, of which it is a plateau region in the south [14]. It was observed that the hills in Nguzu Edda area are very steep and rocky, making soils there very susceptible to erosion irrespective of its nature. The fauna and flora of Nguzu Edda area are mostly characterized by three principal zones namely: the forest, savannah and swampy forest. However these zones have been seriously affected by natural and human influences. The plateau areas have shrubs and oil palm trees, as the trees found in the forest areas include: mahogany, iroko, cam wood, silk, cotton, coconut, native pea, wild mango, paw-paw and orange trees. There are also many dense growth of raffia and palm trees around. Due to the topography of the study area, there seem to be surface water runoff. The shale underlying the sandstone also makes surface run off possible as it does not allow for water peculations. The drainage pattern of the study area is dendritic. The major river is the Cross River and its tributaries which include: Obumkpuma, Ugwuakalilu, Amoba, Wowo, and Ipoko streams. Soils found in most part of Nguzu Edda area are acidic in nature. They are composed mainly of sand, laterite, alluvia and sandstones. In some areas these surficial deposits has undergone various activities such as, weathering, erosion, due to their exposure to the surface. The climate is the same with those of the south eastern Nigeria. The south westerly winds bring the rain from April to October while the north east trade winds are responsible for the harmattan with low humidity from December to February [15]. The hottest periods are February and march with mean annual temperature of over 27°C in the undulating plains while the plateaus are over 25°C. During the rainy season the temperature ranges from 16° to 28°C. The mean monthly rainfall are 20.41 mm in July, 13.77 mm in August, 21.20 mm in September, 16.08 mm in October, 2.88 mm in November and 1.73 mm in December [12].

2.2. Geology

Nguzu Edda area lies within Afikpo syncline (sub-basin) of the southern Benue Trough. Works on Afikpo syncline were carried out by notable researchers such as Simpson [16], Reyment [17], Murat [18] and Nwajide [19]. Anambra basin is the same with the Afikpo sub-basin in terms of basin evolution and lithostatigraphic packaging and differs in terms of sedimentary thickness [20, 21]. The Afikpo sub-basin is filled by a thick sedimentary sequence of Campanian-Maastrichtian deposit of shale, sandstone, siltstone, mudstone and coal. The sub- basin was synchronously formed as the southern extension of the Anambra basin during the folding of the older sediments of the southern Benue trough in the Santonian period [18]. The Nkporo Formation [16, 17] is the basal unit of the Afikpo Sub-basin. The Formation consists of sandstones and dark-grey to black fossiliferous shale whose outcrops are common at Afikpo, Owutu, Asaga Amangwu, Ogbu, Ekeje-Amaiyi road, Enohia, Wowo River and on the lower slope of Awgu-Okigwe Cuesta near Nguzu Edda area. In the lower part of the formation, sandstones interbedded with coal/lignite, siltstones and minor shales dominate (Afikpo Sandstone) while in the upper part the shales are interbedded with limestone and oolitic ironstone beds. Simpson [16] assigned Maastrichtian age to the formation using mollusks and fish teeth from the Asaga Amangwu Section and the type locality in Nkporo Village. Reyment [17] also dated the formation as early Maastrichtian. Petters and Edet [22] used an integrated study of foraminifera and palynology to date the Nkporo Formation in the Afikpo Syncline as early Maastrichtian based on samples from only the Asaga-Amangwu Edda outcrop section. Umeji [23] dated the base of the Nkporo Formation in Leru, Anambra Basin as Campanian, using palynomorphs evidence. Figure 1 gives the distributions of the Nkporo Formation and other major geological formations of the southern Benue Trough.

The Anambra Basin developed as a result of the Santonian event which greatly affected the Benue Trough terminating sedimentation in the Abakaliki Basin. Before then, sedimentation in southern Nigeria which began in the Early Cretaceous was facilitated by the breakup of the African and South American continents leading to the formation of the Benue Trough [24, 25] (Fig. 1). Sedimentation in the trough was controlled by three major tectonic phases, giving rise to three successive depocentres [18, 26] as shown in Table 1. The first phase (Albian – Santonian) featured the deposition of the Asu River Group, Eze-Aku and Awgu Formations within the Abakaliki-Benue Trough which was flanked to the east by the Anambra platform, and to the southwest by the Ikpe platform. The second phase (Campanian-Eocene) was characterized by compressive movements along the NE-SW axis which resulted in the folding and uplift of the Trough into an anticlinorium. This forced the Anambra platform to subside and the depocentre to shift south-westwards to the newly formed Anambra Basin and the Afikpo Syncline on the other side of the anticlinorium in the southeast (Fig. 1). The deposition of the Nkporo Group, Mamu Formation, Ajali Sandstone, Nsukka Formation, Imo Formation and the Ameki Group then followed as shown in Table 1. Towards the end of Eocene, the third phase commenced and was characterized by the structural inversion of the Abakaliki region further shifting the depocentre down dip (southwards) to form the Niger Delta basin [27].

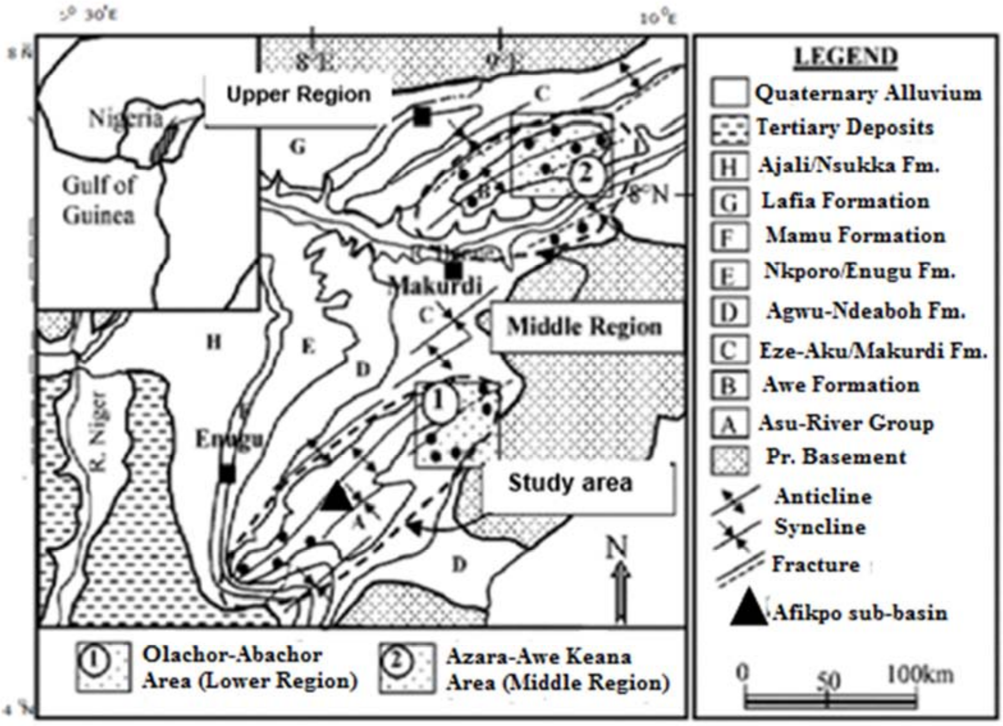

Figure 1. *Geologic map of south-eastern Nigeria showing Afikpo sub-basin [28].*

3. Materials and Methods

3.1. Sampling

The samples used for the analysis were collected from seven (7) different locations within Nguzu Edda area (Fig. 2). The seven (7) samples were designated as S1 to S7. A disturbed method of sampling was employed in collecting the samples. Care was taken when collecting the samples to ensure that the analyzed samples are true representatives of the insitu materials. The samples were excavated with the help of a hoe and a shovel. The samples were packaged in polyethylene bags, clearly labelled and sent to National Steel Raw Materials Exploration, Malali Village, Kaduna, Nigeria for relevant laboratory tests.

Table 1. *Regional Stratigraphic Sequence of South-eastern Nigeria [17, 18, 29].*

PERIOD	AGE	FORMATION	SEDIMENTARY PHASE/BASIN
QUATENARY	Pleistocene	Benin	3rd Sedimentary Cycle/Phase
	Pliocene		
	Miocene	Ogwashi-Asaba	
	Oligocene		
TERTIARY	Eocene	Ameki	Niger Delta. (developed during the Upper Eocene)
	Paleocene	Imo shale	
UPPER CRETACEOUS	Maastrichtian	Nsukka	2nd Sedimentary Cycle/Phase
		Ajali Sandstone	
	Campanian	Mamu	Anambra Basin / Afikpo sub-basin
		Nkporo Shale	
LOWER CRETACEOUS	Santonian	Diastem (Uncomfomity)	1st Sedimentary
	Coniacian	Awgu Shale	
	Turonian	Ezeaku Group	Cycle/Phase
	Cenomanian	Odukpani	The Abakaliki – Benue Basin
	Albian	Asu River Group	
Aptian Barremian	Unnamed Units		
Hauterivian	Precambrian Basement Complex		

3.2. Laboratory Test

The following laboratory tests conducted on the samples include: particle size gradation, natural moisture content, specific gravity, bulk density, Atterberg limit tests, linear shrinkage determination, Proctor compaction test, unconsolidated undrained compressive strength, free swell index, one dimensional consolidation test and California bearing ratio tests.

3.2.1. Particle Size Gradation

This test was carried out in accordance with wet sieving BS 1377 [30] test 7a standard. The British Standard (BS) Sieves used, adequately covered the range of aperture size for the soil. A 2 mm sieve was nested in a 63 micron sieve without the lid. The soil was placed little at a time on the 2 mm sieve and washed on a sink with a jet of clean water. The whole of the material retained on each sieve was allowed to drain and then carefully transferred to a tray and placed in the oven to dry at temperature of 105 to 110°C overnight. The dry soil was then passed through a nest of the complete range of sieves to cover the size of particles present down to 63µm sieve. The operation was carried out on a mechanical sieve shaker. The Percentage Weight retained and the Percentage Passing in the sieves were determined using the equation:

$$\%\text{weight retained} = \frac{\text{weight retained}}{\text{initial weight}} \times 100\%$$

$$\%\text{passing} = 100 - \%\text{weight retained}$$

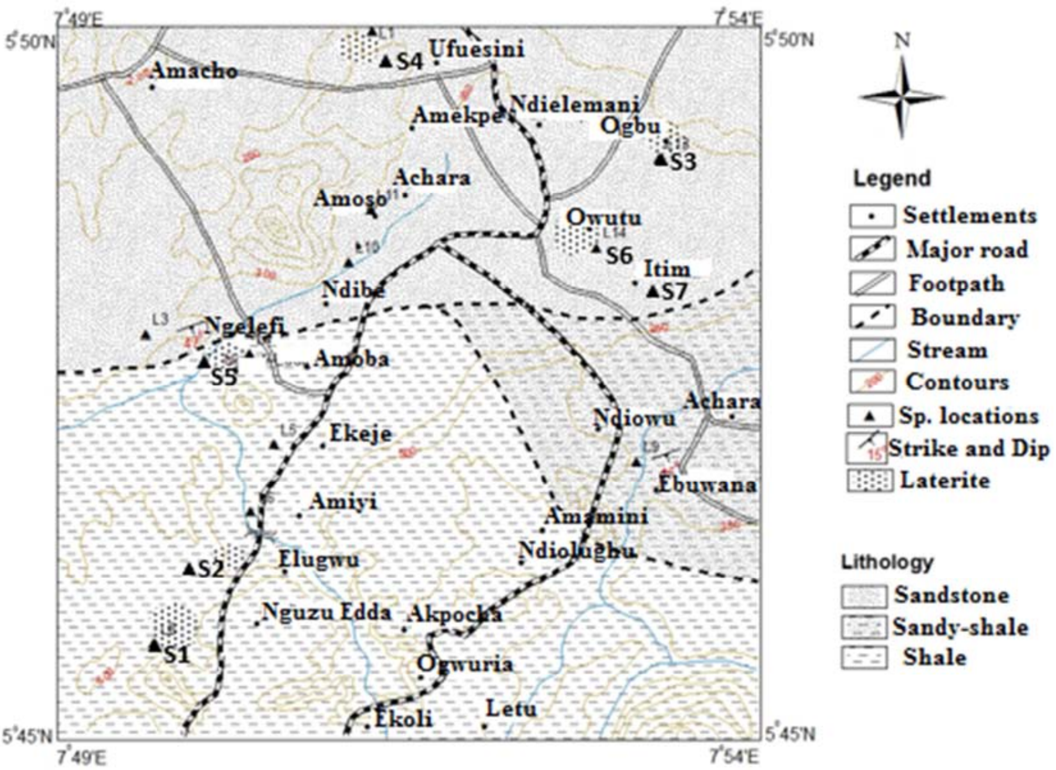

Figure 2. *Accessibility map showing sampled locations.*

3.2.2. Natural Moisture Content

This test was carried out in accordance with BS 1377 [30], test 1 A standard. A sample container was weighed to 0.01g and the weight was recorded as m_1. The soil material to be tested was then added, both container and soil were weighed and the value was recorded as m_2. The container with sample was then placed in the oven for 24 hours at a controlled temperature of 105° C, after which it was transferred to the desiccators to cool. The oven dried and cooled sample was the weighed and the value recorded as m_3. The Natural Moisture Content of the sample was calculated thus, $Wc = \left(\frac{m2-m3}{m3-m1}\right) \times 100(\%)$

3.2.3. Specific Gravity

This test was carried out in accordance with BS 1377 [30] standard. Three density bottles were washed, dried, cooled and weighed to the nearest 0.001g and recorded as (W_1). Sample of appropriate mass (50 to 150g) was obtained by quartering down the original sample after passing a 2 mm BS sieve. Each bottle with the soil was weighed and recorded as (W_2). Distilled water was added to each bottle so that the soil was covered and the bottle half full. The soil, bottle and water was weighed and recorded as (W_3). The bottles were cleaned out and filled completely with distilled water and placed in the constant temperature bath until attainment of bath temperature. The bottle and distilled water was weighed and recorded as (W_4).

The specific gravity, Gs was calculated as = $(W_2\text{-}W_1) / (W_4\text{-}W_1) - (W_3\text{-} W_2)$

3.2.4. Bulk and Dry Densities

These tests were carried out in accordance with BS 1377 [30] test 16 standard. Bulk and dry densities are fairly easy determinations, which yield valuable insight on a soils potential to support a structural foundation. These parameters were determined using soil cores. A hollow cylinder provided with a cutting edge was forced into the ground, then retrieved with a column of soil, preserved and taken to the laboratory where it was extruded into a cylindrical mould with a known mass and volume. It was weighed and the mass recorded. The moisture content was then determined as per (3). The bulk density was determined by dividing the mass of wet soil by the volume of the cylindrical mould while the dry density was determined as follows: Dry density, $(\rho_d) = \rho_b \, x100/100 + m$ where, ρ_b = bulk density and m= moisture content (%)

3.2.5. Atterberg Limits Test

The determination of liquid limit was carried out in accordance with American Society of the International Association for Testing and Materials (ASTM) [31] method D423 standard. About 250 g of soil sample from thoroughly mixed portion of soil material, passing 0.425 mm was placed in a porcelain dish and mixed with 15 to 20 ml distilled water by alternately and repeatedly stirring, kneading and chopping with spatula. Further water increment of 1 to 3 ml was added and the process repeated until sufficient water has been thoroughly mixed with the soil. A portion of the mix was pressed into the cup using a spatula and carefully spread into position while avoiding entrapment of air bubbles. The liquid limit was taken as the moisture content corresponding to 25 blows. Similarly, for plastic limit determination, the test was carried out in accordance with BS 1377 [30] test 3 standard. About 20 g of soil sample, passing 0.425 mm sieve was used for the test. The sample was thoroughly mixed with distilled water and kneaded for about 10 minutes to form a plastic ball. The ball was molded between the fingers and rolled between the palms, such that the warmth from the hand slowly dried it. The thread was then rolled between the fingers and a glass plate using steady pressure which reduced the diameter to about 3mm, the pressure was maintained until the thread crumbled. This crumbling point is the plastic limit.

Linear shrinkage test

This test was carried out in accordance with BS 1377 [30] test 5 standard. About 150 g of air dried soil passing 0.425 mm sieve was used. The mould was cleaned, dried and a thin film of silicone grease was applied to the inner surface to prevent soil sticking to the mould. The soil was placed on a glass plate and mixed properly using distilled water for about 10 minutes until a homogenous paste of about the liquid limit was achieved. The length of the bar of soil was measured using a venire caliper, both top and bottom surfaces. The mean of the two lengths was taken as the dry length.

3.2.6. Proctor Compaction Test

This test was carried out in accordance with BS 1377 [30] standard. For the ordinary test, a rammer with a 50 mm diameter face, 2.5 kg weight and falling from a constant height of 300 mm was used on a British Standard (BS) mould. A total of 5 kg soil sieved through a 20 mm sieve was used. About 200-300 ml of water was used to thoroughly mix the soil. The mould was weighed and recorded as (M1), the extension collar was then connected to the mould, and loose soil was added to the mould so that it was half filled, the guide tube was gently placed on the soil and held vertically and then the soil was compacted by 27 blows from the rammer. A second approximately equal layer of soil was placed in the mould and compacted with another uniformly distributed 27 blows and same to a third layer. The extension collar was then removed carefully and the excess soil trimmed. The weight of the mould and soil was recorded (M2). Compaction was repeated so that at least 5 compactions were made such that water increment range encompasses the optimum moisture content. Bulk density (ƿ) = M2-M1/X (mg/m^3)

where M1 = Mass of mould + base plate

M2= Mass of soil + mould + base plate

X= Volume of mould (1000 cm^3 for BS Mould)

Dry density (ƿd) = 100 ƿ/100 + W (mg/m^3)

where W = Moisture content (%)

3.2.7. Unconsolidated Undrained Compressive Strength with Triaxial Machine

This test was carried out in accordance with D2850-70 standard. The triaxial test is designed to evaluate the cohesion properties (c) and the internal angle of friction (Ø) of a soil sample. In order to obtain a reasonable assessment of the c and Ø values, three experiments were conducted on undisturbed samples of the same soil at three different cell pressures.

An extruder was used to obtain three specimens of dimensions as follows:

Height = 76 mm and Diameter = 38 mm

The specimens were carefully placed in a rubber membrane and secured with rubber rings to the foot piece and cap. The specimen was carefully placed on the base while ensuring that the piston in the cap of the pressure cell was at its maximum. The cap was placed on the sample and fastened securely to the base plate with the three tie rods. The piston was then pressed carefully unto the cap of the sample making sure that the piston fell on the circular hole on the sample cap. The dynamometer was then brought into contact with the top of the piston. The load ring, piston and sample are now in contact with each other such that no vertical pressures were exerted on the sample. The air vent on the cap of the cell was then opened and the cell was filled with water from the pressure cylinder (signified by water leaking out of cell vent). The vent was then sealed tightly and the required pressure was built in the cell. The strain gauge was then brought in contact with the datum bar on the cell and zeroed. The dynamometer was also adjusted to zero. The machine was then turned on and dynamometer readings taken at regular intervals of strain. The machine was left running and dynamometer readings continued until the dynamometer ceased to read, signifying sample failure. The procedure was repeated with two different specimens at different cell pressures.

3.2.8. Free Swell Index

This test was carried out in accordance with BS 1377 [30] standard. This procedure involved taking two 20 g oven dried soil samples passing through 425 чm sieve. Each was placed separately in two 100 ml graduated cylinders, one containing distilled water and the other kerosene (non-polar liquid) filled to the 100 ml mark. The final volume of soil was read after 24 hours to calculate the free swell index.

The Free swell index, In (%) = V/Vo x100

where V= final volume (volume in water)

Vo = initial volume (volume in kerosene)

3.2.9. One Dimensional Consolidation

This test was carried out in accordance with BS 1377 [30] standard. Consolidation is that plastic deformation or void ratio reduction of a soil mass which are functions of time and excess pore pressure. It is imperative if settlement calculations have to be made for fine grained soils subjected to load. This laboratory test is one dimensional because with a metal ring confining the sample, no lateral soil or water movement takes place. The test was carried out for three load increments starting with a load that gave a stress of 100 kN/m^2 on the sample and then subsequently doubled after each compression stage. The applied stress for this study was 100, 200 and 400 kN/m^2. After the first loading, the beam support was wound down and at the same time a clock was started and the readings on the settlement measuring dial gauge was taken at intervals of 0.125, 0.25, 0.5, 1, 2, 4, 8, 15, 30 minutes and after 1, 2, 4, 8 and 24 hours. The coefficient of consolidation (CV) was determined by plotting settlement versus the square root of time.

3.2.10. California Bearing Ratio

This test was carried out in accordance with BS 1377 [30] test 16 standard. The test is used as an important criterion in pavement design as they estimate the bearing capacity of high way subbases and subgrades. The apparatus used consist of a motor driven compression machine with a constant penetration rate of 1mm/min. for this particular test, a loading with a range of 0-10 kN was employed. A standard CBR mould, fitting and tools were used. Particles with diameter larger than 20mm were removed via sieving. About 8 kg of sample was compacted dynamically using the BS ordinary method where the samples were loaded into the mould in three layers and each layer subjected to 62 blows from a 2.5 Kg hammer dropping from a fixed height of 300 mm. The load measuring device was connected to the compression machine. The cylindrical plunger, diameter 49.5 mm, cross sectional area of 1925 mm^2 and a length of 250 mm was connected to the load measuring device. The mould with the sample and surcharge weights was placed in the machine (each surcharge ring of 2 Kg is equivalent to about 70 mm thickness of super imposed construction). The plunger was seated on top of the specimen in such a manner that it can move freely from the surcharge weight. The measuring device was adjusted to read zero. The machine was then switched on to start loading. Readings were taken at 0.25 mm displacement, after 7.5 mm penetration the machine was stopped. The moisture contents were determined for top, middle and bottom of the specimen.

4. Results and Discussion

4.1. Results

4.1.1. Particle Size Gradation and Natural Moisture Content

The results of particle size gradation tests are presented in Table 2. The particles size gradation tests revealed that the lateritic soils have percentage of gravel ranged from 58 to 89% with mean value 74%, sand ranged from 2 to 10% with mean value 6%, and fines ranged from 9 to 35% with mean value 20%. Sample S6 has the highest percentage of gravel and the least percentage of fines. Sample S2 occupies the other end of the spectrum with 61% gravel and 33% fines. According to Oyediran [32], soils with amounts of fines less than 50% are expected to possess better engineering properties when compared with those with amounts of fines greater than 50% which are expected to pose field compaction problems when used either as sub-base or sub-grade materials. The percent fines for A-2 materials of the study area ranged from 9 to 35% which is consistent with the work of Akpokodje [33]. According to Federal Ministry for Works and Housing (FMWH) [34], the particles size gradation of the samples met the general specification for sub grade, sub base and base course material as they have fines less than 35% by weight passing the BS test sieve No. 200. The lateritic soils can be classified under the America Association of State Highway and Transport Officials (AASHTO) [9] classification system as GP which means they are clayey gravel (A-2) which consist of silty or clayey-gravel and sand and can be rated as excellent material for road works (Table 4). The results of the natural moisture content tests are presented in Table 2, which ranged from 4.95 to 12.03% with an average value of 8.82%. According to Emesiobi [35] which shows the classification for moisture content for different soil types and indicate that natural moisture content in soil may range from below 5 to 50% in gravel and sand. The results of the samples show that the values fall into the category of gravel and sand. However, samples with low moisture content are suitable for road construction and this is expected to greatly increase the shear strengths of soil.

4.1.2. Specific Gravity, Bulk and Dry Densities

The results of the specific gravity test are presented in Table 2. They ranged from 2.49 to 2.64 with the average of 2.59. According to Alabo [36], good lateritic material should have specific gravity ranging from 2.5 to 2.75. Also according to Wright [37], the standard range of values of specific gravity of soils lies between 2.60 and 2.80. Lower specific gravity value indicates a coarse soil, while higher values indicate a fine grained soil. These values are higher than the specific gravity of 2.2 specified by FMWH [34] for roads and bridges. According to AASHTO [9] standard of soil classification, specific gravity of sandy soil, which is mostly made of quartz, may be estimated to be about 2.6, whereas for clay and silt soils, it may vary from 2.35 to 2.7. Based on these facts, the range of the specific gravity confirm the high proportion of clay or silt which are of great advantage at the sub-grade and sub-base levels of road construction. The results of bulk density tests are also presented in Table 2 which ranged from 1662.1 to 1775.2 Kg/m^3 with mean value of 1727.3 Kg/m^3 and dry density values, ranged from 1498.7 to 1689.2 Kg/m^3 with mean value of 1596.3 Kg/m^3. According to Paige-Green [38], low bulk density value (<2.6 mg/m^3) of a construction material is indicative of its susceptibility to weathering and deterioration. The results displayed low to impervious hydraulic characteristics, low porosity and high compaction.

4.1.3. Atterberg Limits

The results of the Atterberg limits tests are presented in Table 2. From the table, the liquid limit (LL) ranged from 32 to 38% with mean value of 34%, the plastic limit (PL) ranged from 21 to 25% with mean value of 23% while the plasticity indices (PI) ranged from 8 to 13% with mean value of 11%. Using cassagrande chart, the lateritic soils are classified as inorganic soils of low plasticity (Fig. 3). According to Jegede [39], PI should not be higher than 12% for use as base materials. Federal Ministry of Works and Housing [34] recommended LL of 40% maximum for sub-grade, 35% maximum for sub-base and 30% maximum for base course. PI of 20% maximum for sub-grade, 16% maximum for sub-base and 13% maximum for base course was also recommended by FMWH [34]. When compared with the FMWH [34] requirements (Table 6), the LL results of the lateritic soils are suitable for sub grade which are not greater than 40%, however indicate probable absence of expandable clay materials. Samples S2 and S6 are likely to be suitable for sub base which have values greater than 35% and all the samples are poor to marginally suitable for base course materials which are greater than 30% owning to their LL. All the lateritic soils met the requirements specified by FMWH [34] for plasticity index which stipulates PI of 20% maximum for subgrade, 16% maximum for sub-base and 13% maximum for base course materials and these classify the lateritic soil as having low to medium swelling potential according to Ola [40]. Table 7 displayed the comparison of results of the study area and Nigerian Specification for Road and Bridge Materials. The linear shrinkage (LS) ranged from 10 to 16% with mean value of 13% (Table 2). The degree of expansion classification based on LS show that less than 5, 5-8 and greater than 8 indicate non- critical, marginal and critical degree of expansion respectively [41]. Also, Akpokodje [42] recommended linear shrinkage of 8% maximum for highway soils. The lower the LS of a soil the better it is as a sub-base materials. All the samples do not fall within this specifications stated above, thus making them not suitable for subgrade, subbase and base materials.

4.1.4. Proctor Compaction Test

The results of proctor compaction tests are presented in Table 3. The results indicate that maximum dry density (MDD) ranged from 1.78 to 1.95 mg/m^3 with mean value of 1.85 mg/m^3 and optimum moisture content (OMC) ranged from 9.1 to 20% with mean value of 13.3%. Samples characterized with high value of MDD and low OMC is best suitable as sub-base and sub-grade materials [43]. According to O'Flaherty [44] the range of values that may be anticipated when using the standard proctor test methods are: for clay, MDD may fall between 1.44 mg/m^3 and 1.68 mg/m^3 and OMC may fall between 20 to 30%. For silty clay, MDD is usually between 1.6 and 1.8 mg/m^3 and OMC from 15 to 25%. For sandy-clay, MDD is usually between 1.76 mg/m^3 and 2.16 mg/m^3 and OMC between 8% and 15%. Table 6 shows that for a material to be suitable for construction, it should have MDD > 0.047 mg/m^3 and OMC <18% [34]. When compare with the USCS (Unified Soil Classification System), the lateritic soils in the study area are classified in group SM-SC (Sandysilt clay) as shown in Table 5. In view of the proctor compaction tests results above, the lateritic soils are sandy-clay and can be used as filling and embankment materials except sample S6 with OMC value >18%. Table 5 illustrates the description of the samples base on their USCS, MDD and OMC.

4.1.5. Unconsolidated Undrained Compressive Strength, Free Swell Index, and One Dimensional Consolidation

The results of undrained triaxial tests are presented in Table 3 and Figure 4. The results show values of cohesion (C) that ranged from 130 to 146 kN/m^2 with mean value of 137 kN/m^2 and the angle of internal friction (Ø) which ranged from 15 to 20° with mean value of 16°. Low angle of internal friction is attributable to the presence of expansive clay as reported by Obiora and Umeji [45]. Figure 4 shows that the higher the compressive strength (normal stress) the higher the bearing capacity and permeability becomes low. From Table 6, FMWH [34] recommended that unconsolidated compressive strength of a material should be greater than 103 kN/m^2 for base course material. The statement above revealed that the samples are classified as very stiff soils with good bearing capacities. The results of free swell index tests are also presented in Table 3. The values obtained ranged from 12 to 30.8% with mean value of 20.4%. Federal Ministry of Works and Housing [34] recommended that for a material to be suitable for construction, it should not be greater than 35%. From the results, it implies that the lateritic soils have low swell potentials. The results of coefficient of consolidation tests are presented in Table 3 and Figure 5. The values ranged from 1.72x10^{-2} to 5.61 x10^{-2} m^2/yr with mean value of 3.36 x10^{-2} m^2/yr. Figure 5 shows that the higher the compressive strength of a material, the more suitable it becomes. From the result, the samples reveal non critical compression characteristics.

4.1.6. California Bearing Ratio

The results of California bearing ratio (CBR) tests are presented in Table 3 and Figure 6. The CBR values for unsoaked condition ranged from 51.29 to 87.50% with mean value of 72.41% while soaked CBR have values which ranged from 33.48 to 62.50% with mean value of 47.89%. CBR has been correlated with pavement performance as well as used to establish design curves for pavement thickness [46, 47, 48]. The moisture content at the time of compaction can have a critical influence on the CBR test results. Lyon Associates Inc. [49] showed that compaction at only slightly higher than OMC drastically reduces the CBR values. Federal Ministry of Works and Housing [34] recommended soaked CBR of 5% minimum for sub-grade, 25% minimum for sub-base course and 80% minimum for the base (unsoaked CBR) as shown in Table 6. By interpretation all the soil samples tested are excellent sub-grade, sub-base and base materials except samples S2, S6 and S7 with values 51.29%, 60.29% and 60.32% respectively which are less than the 80% minimum for the base for unsoaked CBR as specified by FMWH [34]. The results (Table 3 and Fig. 6) show a reduction in strength as a result of soaking. This is an indication that there is moisture influx and ingress of water in the lateritic soil. From the comparison of the study area lateritic soils with Nigerian Specification for Road and Bridge Materials (Tables 6), it can be deduced that the lateritic soils from Nguzu Edda area are good sub-grade, sub-base and base materials for road construction.

4.2. Implication of Study

The Nguzu Edda lateritic soils can be classified as A-2 which consist of silty or clayey-gravel and sand and can be rated as excellent material for road works based on AASHTO [9] classification system. The natural moisture content results of the samples show that the values fall into the category of gravel and sand. However, samples with low moisture content are suitable for road construction and this is expected to greatly increase the shear strengths of soil. The specific gravity of the samples confirm the high proportion of clay or silt which are of great advantage at the sub-grade and sub-base levels of road construction. According to Paige-Green [38], low bulk density value (<2.6 mg/m^3) of a construction material is indicative of its susceptibility to weathering and deterioration. The results of the bulk density displayed low porosity and high compaction. The liquid limit results of the lateritic soils indicate probable absence of expandable clay materials making them suitable for sub grade which are not greater than 40%. Samples S2 and S6 are likely to be suitable for sub base which have values greater than 35% and all the samples are poor to marginally suitable for base course materials which are greater than 30% owning to their liquid limit. Plasticity index results classify the lateritic soil as having low to medium swelling potential [40]. The samples have higher compressive strength (normal stress) which is an indication of higher bearing capacity and low permeability. The lateritic soils have low swell potentials. The results indicate a reduction in strength as a result of soaking which implies that there is moisture influx and ingress of water in the lateritic soil.

Table 2. *Results of grain size distribution, specific gravity, natural moisture content, Atterberg limits tests and proctor compaction tests.*

Sample ID	Gravel (%)	Sand (%)	Fines (%)	NMC (%)	G_S	ρ_b (Kg/m^3)	P_d (Kg/m^2)	LL (%)	PL (%)	PI (%)	LS (%)
S 1	72	8	20	9.70	2.59	1662.1	1515.2	32	21	11	12
S 2	61	6	33	12.0	2.49	1773.0	1582.7	37	24	13	14
S 3	89	2	9	4.95	2.61	1772.7	1689.2	33	23	10	10
S 4	70	10	20	10.7	2.60	1762.1	1615.2	33	22	11	12
S 5	83	5	12	5.85	2.64	1682.7	1584.2	33	25	8	14
S 6	58	7	35	10.0	2.59	1663.0	1498.7	38	25	13	16
S 7	84	7	9	8.49	2.60	1775.2	1689.2	32	24	8	10
Mini	58	2	9	4.95	2.49	1662.1	1498.7	32	21	8	10
Max	89	10	35	12.0	2.64	1775.2	1689.2	38	25	13	16
Mean	74	6	20	8.82	2.59	1727.3	1596.3	34	23	11	13

Table 3. *Results of compaction, compressive strength, free swell, bulk density, consolidation test and California bearing ratio tests.*

Sample ID	MDD (mg/m^3)	OMC (%)	C (kN/m^2)	(Ø)^0	Free swell index	Cv m^2/yr(10^{-2})	Un-soaked CBR (%)	Soaked CBR (%)
S 1	1.82	12.0	138	15	17.4	3.21	81.00	54.46
S 2	1.78	18.0	130	15	28.0	4.51	51.29	33.48
S 3	1.95	9.1	146	20	17.0	1.72	87.50	62.50
S 4	1.92	14.0	139	15	20.8	4.81	82.00	57.46
S 5	1.85	10.1	136	15	30.8	1.82	84.50	52.50
S 6	1.88	20.0	140	20	16.7	5.61	60.29	35.48
S 7	1.78	10.2	132	15	12.0	1.82	60.32	39.34
Min	1.78	9.1	130	15	12.0	1.72	51.29	33.48
Max	1.95	20.0	146	20	30.8	5.61	87.50	62.50
Mean	1.85	13.3	137	16	20.4	3.36	72.41	47.89

Table 4. *Revised AASHTO System of Soil Classification [13].*

General Classification	General Materials (35% or less passing 0.075 mm)							Silt-clay materials (more than 35% passing 0.075 mm)			
Group Classification	**A-1**				**A-2**			**A-4**	**A-5**	**A-6**	**A-7**
	A-1-a	**A-1-a**	**A-3**	**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**				**A-7-5** **A-7-6**
Sieve Analysis% passing											
2.00 mm (No 10)	50 max										
0.425 mm (No 40)	30 max	50 max	51 min								
0.725 mm (No 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing	6 max										
Liquid limit			N.P	40 max	41 min	40 max	41 min	40 max	41 min	40 max	40 min
Plastic Index				10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant Constituent material	Stone fragment Gravel and sand		Fine Sand	Silty or Gravel and sand				Silty soils		Clayey soils	
General rating			Excellent to Good					Fair to poor			

Table 5. *Description of lateritic soils [13].*

Sample ID	MDD (mg/m^3)	OMC (%)	Group symbol	Description
S 1	1.82	12.0	SM-SC	Sandysilt clay mix with slightly plastic fines
S 2	1.78	18.0	SM	Silty soils, poorly graded sandysilt mix
S 3	1.95	9.10	SC	Siltyclay mix with slightly plastic fines
S 4	1.92	14.0	SM	Silty soils, poorly graded sandysilt mix
S 5	1.85	10.1	SM-SC	Sandysilt clay mix with slightly plastic fines
S 6	1.88	20.0	SM	Silty soils, poorly graded sandysilt mix
S 7	1.78	10.2	SM-SC	Sandysilt clay mix with slightly plastic fines

Table 6. *Comparison of results with Nigerian Specification for Road and Bridge Materials [50].*

Properties of material	Nigerian specifications[a]	Nguzu Edda laterite soil range	Mean	Remarks
General filling and embankment				
MDD (mg/m^3)	> 0.047	1.78-1.95	1.85	
OMC (%)	< 18	9.1-20.0	13.3	
LL	< 40	32-38	34	
PI	< 20	8-13	11	Suitable
% Passing No. 200 (%)	≤ 35	9-35	20	
CBR (24 hrs. soaked) BS (%)	> 5	33.48-62.50	47.89	
Sub-base course				
LL	< 35	32-38	34	
PI	< 16	8-13	11	Likely to be Suitable
CBR (24 hrs. soaked) at West African Standard and OMC (%)	≥ 25	–	–	
Base course				
LL	≤ 30	32-38	34	
PI	≤ 13	8-13	11	
Unsoaked CBR at Modified AASHTO and OMC (%)	≥ 80	51.29-87.50	72.41	Poor to marginally suitable
% Passing sieve No. 200 (%)	5 – 15	9-35	20	
UCS (kN/m^2)	>103	130-146	137	

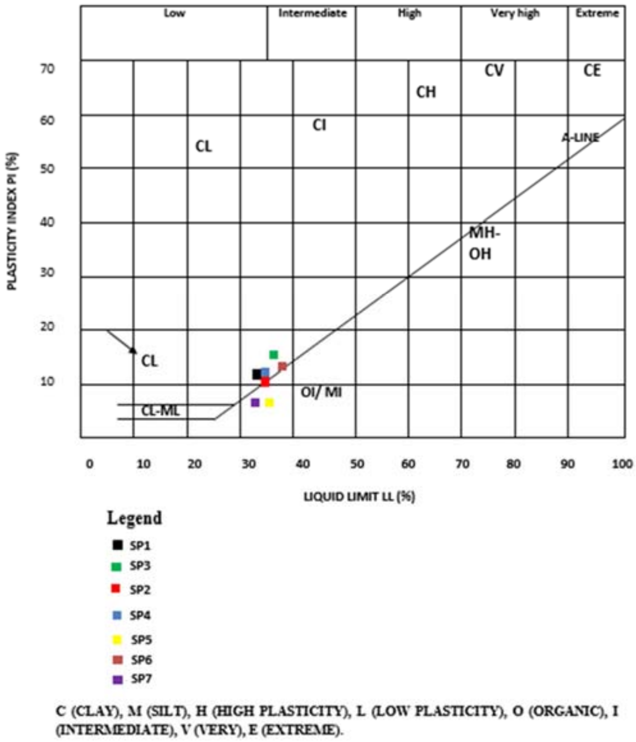

Fig. 3. *Cassagrande chart for soil classification [51].*

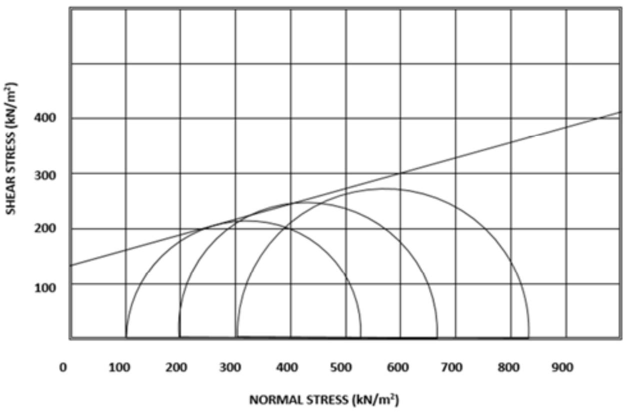

Fig. 4a. *Undrained triaxial test for sample 1.*

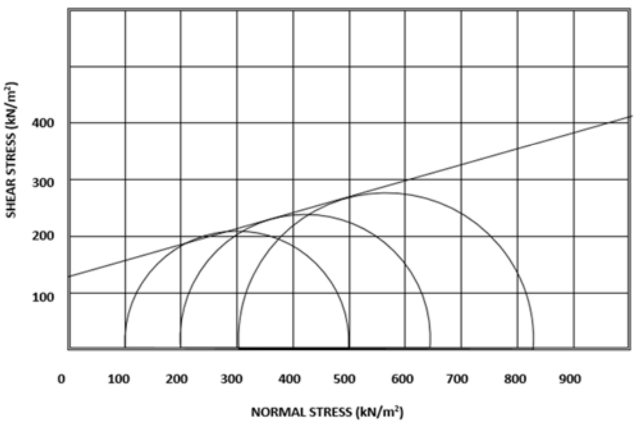

Fig. 4b. *Undrained triaxial test for sample 2.*

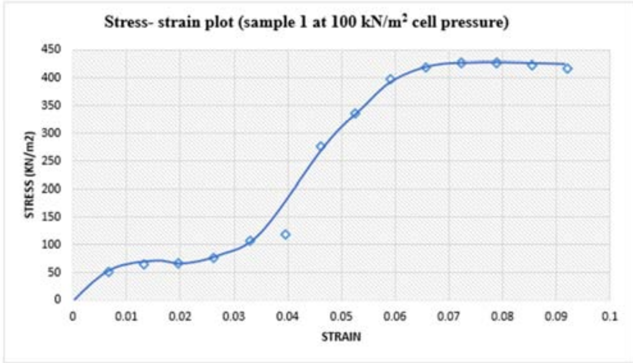

Fig. 4c. *Stress-strain plot (sample 1 at 100kN/m²).*

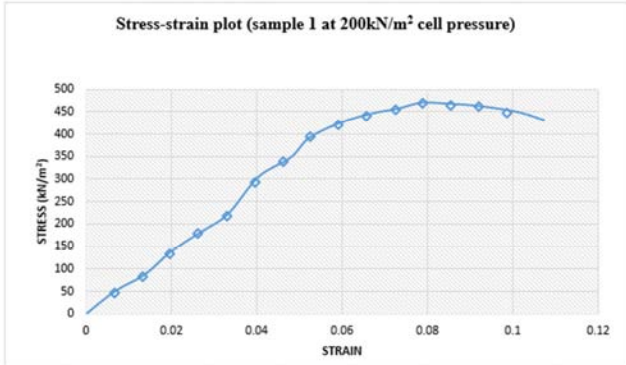

Fig. 4d. *Stress-strain plot (sample 1 at 200kN/m²).*

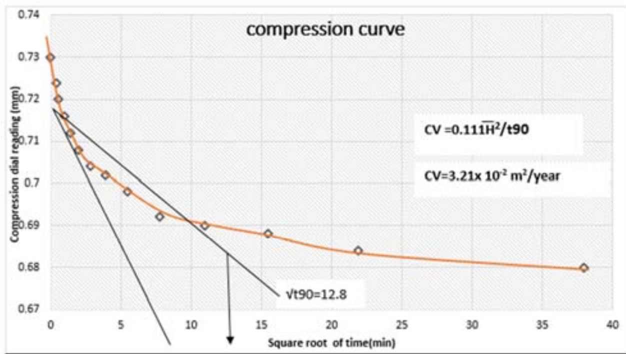

Fig. 5a. *One dimensional consolidation test for sample 1.*

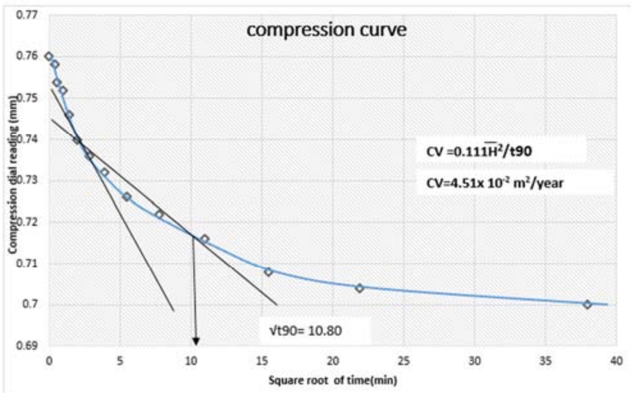

Fig. 5b. *One dimensional consolidation test for sample 2.*

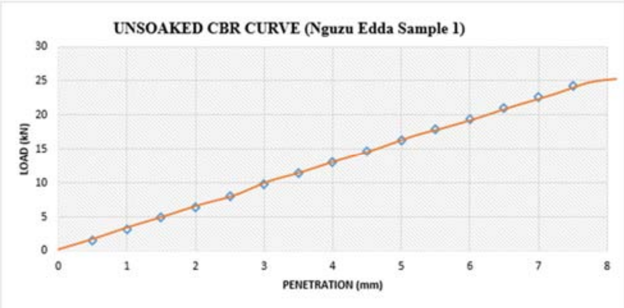

Fig. 6a. Unsoaked CBR test for sample 1.

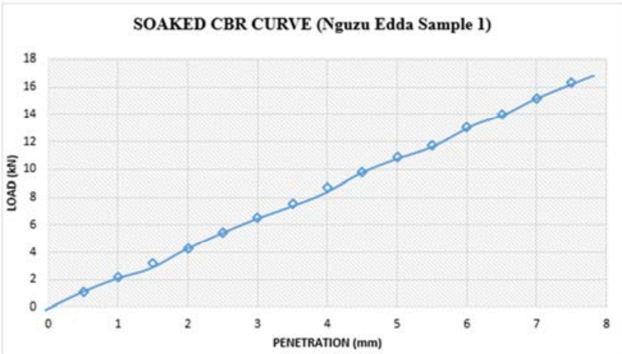

Fig. 6b. Soaked CBR test for sample 1.

5. Conclusion and Recommendation

The geotechnical properties of lateritic soils in Nguzu Edda area have been assessed in compliance with BS 1377 [30] methods of soil testing for Civil engineers. The particles size gradation of the samples met the general specification for sub grade, sub base and base course material as they have fines less than 35% by weight passing the BS test sieve No. 200. The lateritic soils can be classified as A-2 which consist of silty or clayey-gravel and sand and can be rated as excellent material for road works based on AASHTO [9] classification system. The natural moisture content results of the samples show that the values fall into the category of gravel and sand. However, samples with low moisture content are suitable for road construction and this is expected to greatly increase the shear strengths of soil. The specific gravity of the samples confirm the high proportion of clay or silt which are of great advantage at the sub-grade and sub-base levels of road construction. Low bulk density value (<2.6 mg/m^3) of a construction material is indicative of its susceptibility to weathering and deterioration [38]. The results of the bulk density displayed low porosity and high compaction. The liquid limits of the lateritic soils indicate the probable absence of expandable clay materials making them suitable for sub grade which are not greater than 40%. Samples S2 and S6 are likely to be suitable for sub base which have values greater than 35% and all the samples are poor to marginally suitable for base course materials which are greater than 30%. Plasticity index results classify the lateritic soils as having low to medium swelling potential. All the samples have higher shrinkage limit, thus making them not suitable for subgrade, subbase and base materials. Owning to the proctor compaction test, the lateritic soils are sandy-clay and can be used as filling and embankment materials except sample S6 with OMC value >18%. The samples have higher compressive strength (normal stress) which is an indication of higher bearing capacity and low permeability. The lateritic soils have low swell potentials. All the soil samples tested are excellent sub-grade, sub-base and base materials except samples S2, S6 and S7 with values 51.29%, 60.29% and 60.32% respectively which are less than the 80% minimum for the base for unsoaked CBR as specified by FMWH [34]. The results indicate a reduction in strength as a result of soaking which implies that there is moisture influx and ingress of water in the lateritic soil. The lateritic soil samples tested from the study area are adjudged suitable for sub grade, sub base and base course materials as they satisfy all requirements specified by FMWH [34]. These valuable data obtained from the geotechnical analysis can be useful for civil engineers in the design and construction of roads in Nguzu Edda area for maximum durability and efficiency, hence, nip the failure problems in the bud. Based on the investigations of the study, it is recommended that engineering confirmatory test should be carried out before embarking on any construction such as road. Samples S2, S6 and S7 which failed lateritic soil should be stabilized with either cement, sand, crushed stone (gravels) of ½ and $^3/_4$ inch size in order to meet the sub-base or base course requirement.

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