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Engineering Properties of Lateritic Soil in Otun Area, Ekiti State, Nigeria

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ABSTRACT

Lateritic soils at Otun Ekiti, Ekiti state, southwestern Nigeria were investigated with respect to their geotechnical properties and their suitability for subgrade and sub – base construction materials. Four disturbed lateritic soil samples (sample A, B, C and D) were selected for the various laboratory techniques. The grain size analyses, the specific gravity tests, the atterberg limit tests, compaction, California bearing ratio and shear box tests were carried out on the samples. The grain size analysis shows that sample A is gravelly silt-clayey sand. Sample B is silt – clayey gravel composition. Sample C is gravelly silt-clayey while Sample D is silt-clayey gravel. Atterberg consistency limit test indicate that sample A has 30.0%, liquid limit 19.5% plastic limit, 10.5% plasticity index, 9.1% shrinkage limit. Sample B has liquid limit of 27.0%, 16.2% plastic limit, 10.8% plasticity index and 7.4% shrinkage limit. Sample C has a liquid limit of 32.4%, plastic limit of 15.6%. It has a plastic index of 16.8%, Shrinkage limit of 9.7% while Sample D has a liquid limit of 36.2%, plastic limit of 17.7%. It has a plastic index of 18.5% and 11.1% as shrinkage limit. Thus, the soil is classified to be intermediate plasticity which can be used for sub – grade and sub – base materials. The soil samples are above the activity (A) line in the zone of intermediate plasticity (CL) which suggests that they are inorganic soils. Based on engineering use chart, the workability as construction engineering is good to fair particularly as erosion resistance in canal construction. However, the high shrinkage limit may also reduce erosion in this area because of cohesion of the plastic clay material.

The California Bearing Ratio (CBR) values are within 2 – 3% (mean = 2.75%) and 2 - 4% (mean = 2.75%) in sample A and sample B respectively while California Bearing Ratio (CBR) of 2 - 4% (mean = 2.75%) and 2 – 3% (mean = 2.75%) in sample C and sample D respectively. This implies that the materials can be used as a sub-grade to base course material for support of flexible pavements. The compaction tests for the optimum water content for sample A is 15.0% and 13.0% for standard and modified proctor respectively. The standard and modified proctor for sample B is 15.0% and 14.0% respectively. The compaction tests for the optimum water content for sample C and D is 15.0% and 14.0% for standard and modified proctor respectively. The compaction tests for Sample A indicate a higher fine fraction and thus a higher optimum moisture content while sample B, C and D has higher coarse fraction with lower optimum moisture content. The cohesion falls within 70-90Kpa (mean = 79Kpa) and the angle of internal friction ranges from 26^0 - 32^0 with mean of 28^0 for standard and modified compaction energies respectively. The results obtained from geotechnical analysis suggest that the soil is good to fair as erosion resistance in canal construction because of its high bearing capacity and it can also be used as sub – grade and base course in road construction.

Keywords: Lateritic soil, Construction, Erosional and Geotechnical.

1.0: INTRODUCTION

Laterite contribute to the general economy of the regions where they are found. Their scope is very wide and includes geological engineering, civil engineering, mining research (iron, aluminum and manganese) deposits. The evaluation of lateritic soil of most engineering structure requires that adequate information about the engineering properties of the soil and subsurface soil condition of that particular area is known. This is vital for the engineering planning, design and construction of such foundations to be based on concrete geotechnical parameters. This is more important especially in the design and construction of highways, where there is need for a good and adequate knowledge of the geotechnical and engineering properties of the subgrade and, more importantly, the construction materials' properties for certain engineering decisions to be taken. Jackson (1980) established that lateritic soils have been used mostly as base and sub-base materials in road construction.

Alao (1983) studied the engineering properties of some soil samples from Ilorin area and discovered that they could be stabilized by compaction and that the samples could yield maximum strength if they are compacted on the dry side of their optimum moisture content. Enaworu et al. (2016) reported geochemistry and compaction analysis on lateritic soils and their suitability of being used as lower grade and sub-base materials for road construction works. They confirmed that these soils are not laterites but lateritic soils which can be compacted at precise optimum moisture contents before being used as road construction materials. Obaro et al. (2019) concluded that lateritic soil has high bearing capacity due to low cohesion and high angle of internal friction. Hence, they are good as engineering construction. This paper examines the geotechnical properties of lateritic soil in Otun area in Ekiti State, Southwestern Nigeria.

1.2: Geology of the Study Area

The geology of the study area falls within the Precambrian Basement Complex of southwestern Nigeria (Fig. 1). It is underlain by rocks of granite, granitic and migmatite gneiss. (Olusiji, 2013). However, there are visible outcrops seen in the area where the samples were taken. The study area lies between latitude N07^{0}59'03'' of the equator and longitude E05^{0}07'49'' of Greenwich Meridian (Fig 2). The mineralogical composition of charnokite rocks include quartz, alkali - feldspar, mica (muscovite and biotite), hornblende and accessory of opaque minerals such as apatite and zircon (Cooray, 1970). The climatic conditions of the area during dry season last from October to February while the rainy season in the area begins towards the end of March and ends in October. The humid tropical climate of Ilorin has particularly encouraged comparatively deep weathering of surface rocks.


Research Location
Sokoto
Argungu
Birnin Kebbi
Kano
Potiskum
CHAD BASIN
Maiduguri
Ashaka
Kaduna
Bauchi
Gombe
Kontagora
Jos
BIDA BASIN
Bida
Abuja
Ilorin
BENUE TROUGH
Benue
Kabba
Lokoja
Niger
Ibadan
ANAMBRA BASIN
Gboko
Benin
Onitsha
Lagos
NIGER DELTA
Okigwe
Calabar
Atlantic Ocean
Port Harcourt
0
200 kms
Sedimentary Rocks
Quaternary
undifferentiated sands gravels and clays
Coastal Plains sands and clays
Chad Formation
Tertiary
Undifferentiated sedimentary rocks
Kerri-Kerri Formation
Cretaceous
undifferentiated sedimentary rocks
Asu River Group
Bima sandstone & Yolde Formatio
Igneous and Metamorphic Rocks
Tertiary - Recent volcanic rocks
Jurassic - Younger granites
PreCambrian Basement Complex
Water

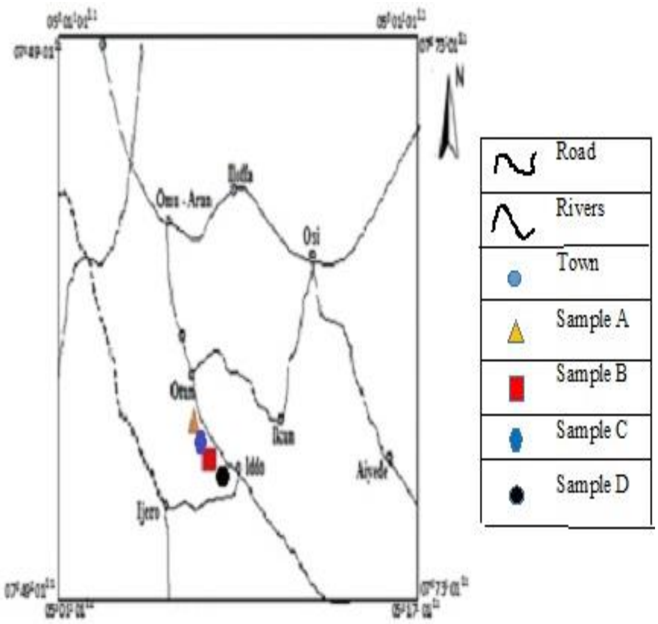

Fig.1: Geological map of Nigeria Fig. 2: Location map of the study area (Rahaman, 1988)

2.0: MATERIALS AND METHODS

The research was carried out in two major stages: Field investigation and laboratory analyses. Field investigation include sample collection, description and preparation. Bulk samples were collected with aid of digger, hand shovel, head pan and two sacks of bags from four (4) different trials pits in Otun, Ekiti State and these two samples were air dried differently for three weeks at room temperature before laboratory analyses commenced. The main laboratory tests carried out on different samples were Specific gravity, Grain size distribution, Atterberg limits, Linear shrinkage, Compaction, California bearing ratio and Shear box tests.

3.0: RESULTS AND DISCUSSION

3.1: Specific Gravity

The specific gravity is assessing the maturity of lateritic soil and an indication of degree of laterization (Ackroyd, 1963). Specific gravity of 2.52 was obtained for sample A, sample B 2.62 was obtained, 2.56 was obtained for sample C while 2.61 was obtained for sample D. Comparing these values with typical values for specific gravity (Gs) for some soil types after Bowles (2012). It can be said that sample A, B, C and D are both organic clay.

Table 1. Typical values of specific gravity of soil particles (Bowles, 2012)

Soil Type	Specific gravity
Sand	2.65 – 2.68

Gravel	2.65 – 2.66
Clay (Inorganic)	2.52-2.66
Clay (Organic)	2.68 – 2.72
Silt	2.65 – 2.66

3.2: Particle size analysis

The fundamental objective of the grain size analysis is to determine the percentage distribution of various particle size. This distribution however influences the capability in engineering construction works. The grain size distribution analysis for sample A shows that it consists of gravelly silt-clayey sand with 35% silt, 35% clay, 50% sand and 15% gravel constituents. Sample B consist of as silt-clayey very gravel with 24% silt, 24% clay, 56% sand, 20% gravel composition. Sample C is made up of gravelly silt-clayey sand with 32% silt, 31% clay, 45% sand and 15% gravel constituents. Correspondingly, sample D could be described as silt-clayey with 22% silt, 22% clay, 48% sand, 24%. From this analysis, both soils can be classified as sandy clay.

3.3: Atterberg consistency limit.

The plasticity index of lateritic soil is a crucial index in determining the geotechnical properties of such soil. (Ige, 2010). The soil sample A has liquid limit of 33.8%, plastic limit of 16.5%. It has a plasticity index of 17.3%. Sample B has a liquid limit of 37.0%, plastic limit of 17.4% and plasticity index of 19.6%. Sample C has a liquid limit of 32.4%, plastic limit of 15.6%. It has a plastic index of 16.8% while Sample D has a liquid limit of 36.2%, plastic limit of 17.7%. It has a plastic index of 18.5%. According to Whitlow (1995), the liquid limit less than 30% indicate low plasticity, values ranging between 35% and 50% indicates intermediate plasticity while the values between 50% and 70% high plasticity between 70% and 90% indicates very high plasticity and greater than 90% suggest extremely high plasticity. Thus, the soil is classified to be intermediate plasticity. The (FMWH, 1997) also recommends 50% maximum liquid limit for sub – grade and sub – base materials. This suggests that the liquid limit for all the samples falls within the maximum range thus making them suitable for sub – base and base materials. However, all the soil samples plotted are in CL^- group on the plasticity chart (Fig. 3) which indicate inorganic clay of low plasticity (Table 1). Based on engineering use chart, the workability as construction engineering is good to fair particularly as erosion resistance in canal construction.

The linear shrinkage of Sample A is 8.7%, Sample B is 12.8%, Sample C is 9.7% while Sample D is 11.1%. All the samples are greater than 8%. According to Brink et al. (1982) and Jegede and Olaleye (2013) who established that both samples are an active soil that will be liable to have shrinkage and heavy compaction problem.

However, the high shrinkage limit may reduce erosion in this area because of cohesion of the plastic clay material.

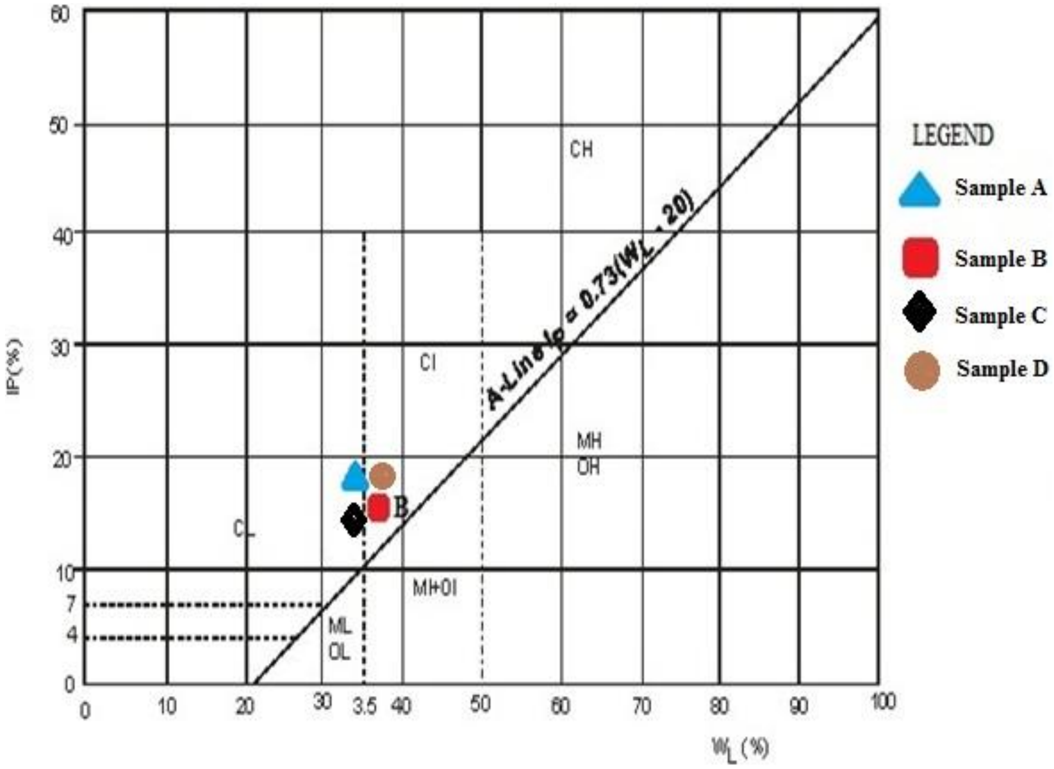

Fig. 3: Plot of the soil sample on plasticity chart

3.4: Compaction tests

According to Enaworu (2017) the degree of compaction is obtained when the water content has a certain value known as the optimum moisture content. The result for the standard proctor and modified proctor test for compaction curves in Figs. 4 and 5 shows that the dry density increases with increasing moisture content up to the maximum and then decreases. According to the maximum dry density (MDD) obtained for sample A is 1.80g/cm^3 at standard proctor and 1.90g/cm^3 for modified proctor fig. 4. For sample B, the maximum dry density is 1.70g/cm^3 at standard proctor and 1.78g/cm^2 for modified proctor fig.5. Sample C the maximum dry density is 1.83g/cm^3 at standard proctor and 1.92g/cm^2 for modified proctor fig.6 while the maximum dry density is 1.86g/cm^3 at standard proctor and 1.94g/cm^2 for modified proctor fig.7. This falls within the observed range of 1.3g/cm^3 to 2.4g/cm^3 observed by Gidigasu (1972), Madu (1975) and Ogunsanwo (1989). There is a minor reduction of maximum dry density down the profile in the study.

The optimum water content for sample A is 15.0% at standard proctor and 13.0% at modified proctor. For sample B, the optimum moisture content is 15.2% at standard proctor and 14.0% at modified proctor. Sample C optimum moisture content is 15.0% at standard proctor and 14.0% at modified proctor while Sample D optimum moisture content is 15.0% at standard proctor and 14.0% at modified proctor. The optimum moisture content is a guide to determine the quantity of water to add during construction. Sample A has a higher fine fraction thus higher optimum moisture content than other three samples. This is because sample with higher fine fraction has higher optimum while the sample with lower fine fraction or higher coarse fraction has the lower optimum moisture content. However, the values generally fall within the recommendation of previous researcher for purpose of fills and base course in road and liner in landfill.

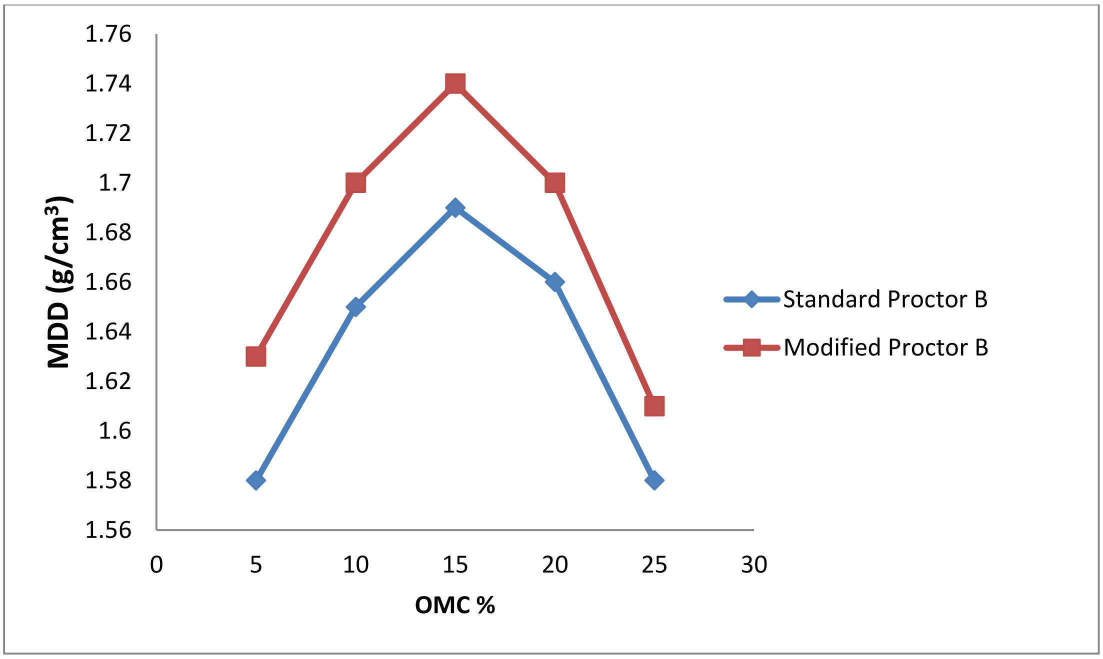

Fig. 4: Compaction curve of soil sample A Standard and Modified Proctor

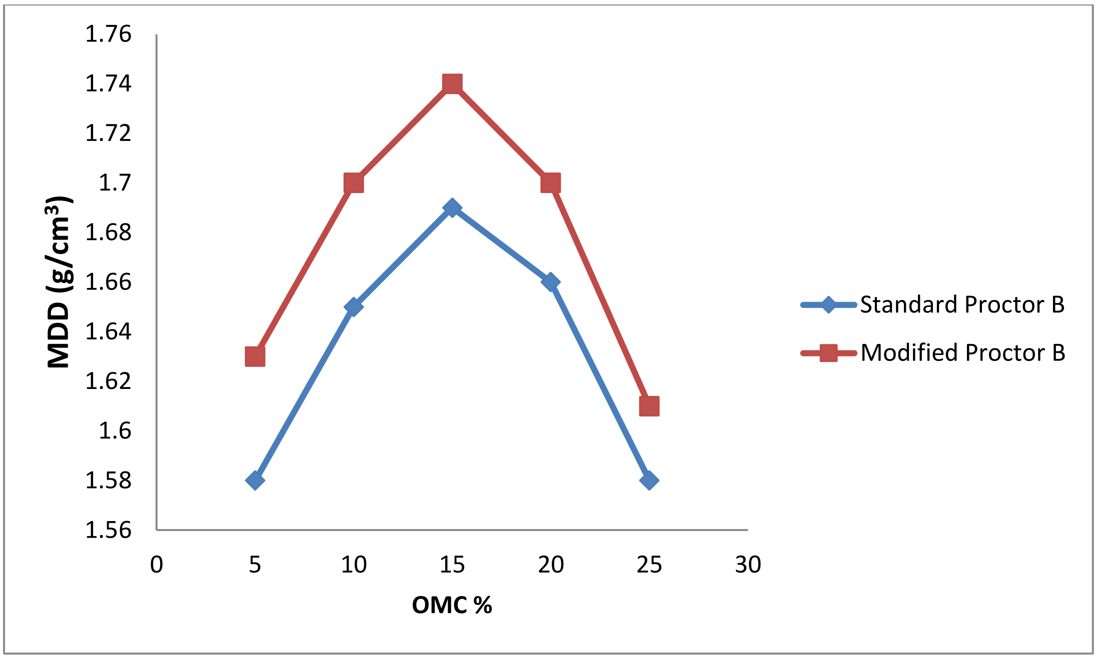

Fig. 5: Compaction curve of soil sample B Standard and Modified Proctor

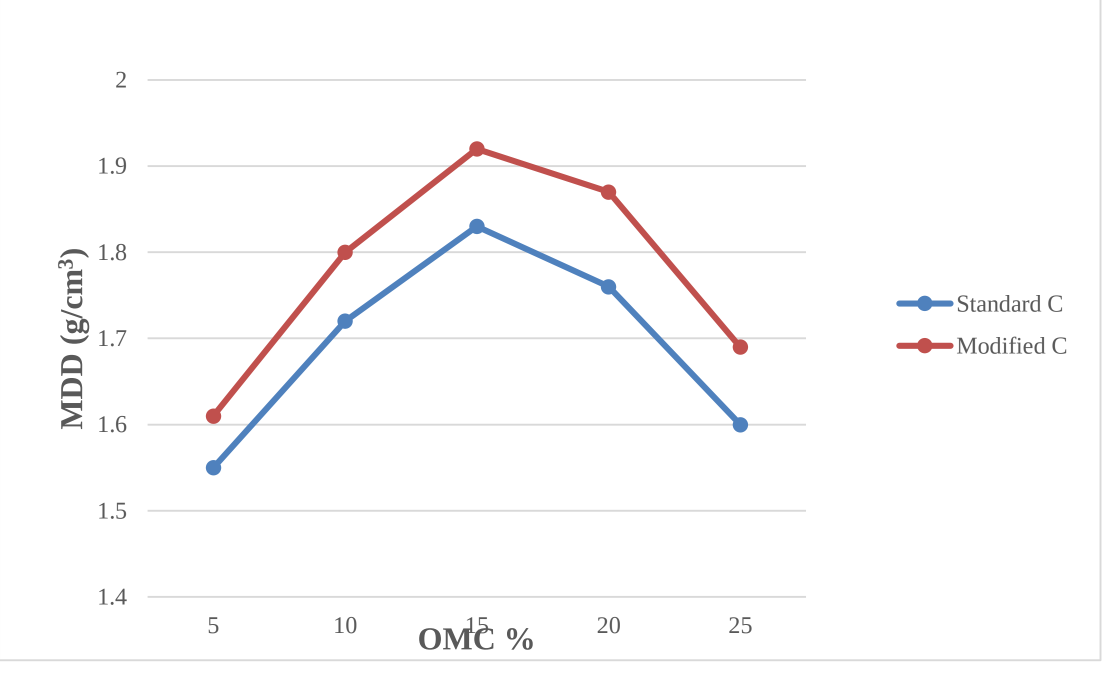

Fig. 6: Compaction curve of soil sample C Standard and Modified Proctor

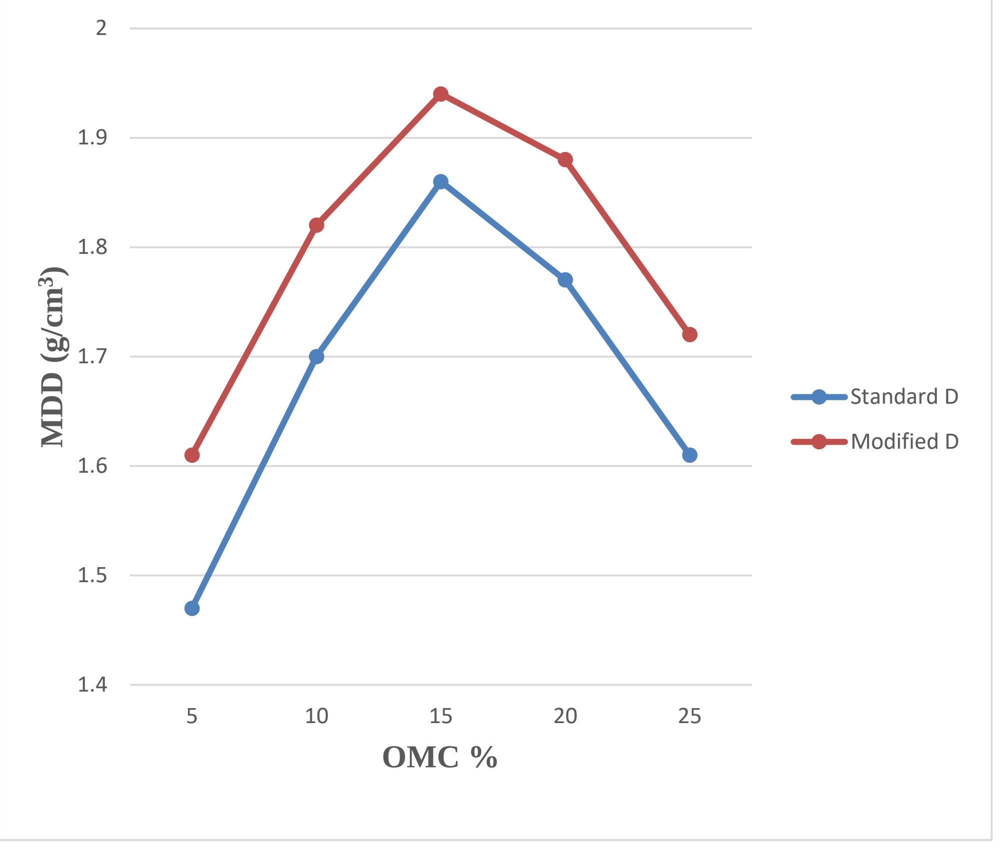

Fig. 7: Compaction curve of soil sample D Standard and Modified Proctor

Table 3. Specification for Standard and Modified Proctor Test Variables using the 996.2cm^2 Mould

SPECIFICATION	STANDARD PROCTOR	MODIFIED PROCTOR
Weight of Rammer (kg)	2.5	7.5
Height of Rammer drop (m)	03	0.45
Volume of Mould (M^3)	996.2 X 15^{-4}	996.2 X 15^{-4}
Compactive Energy (MNm/m^3)	0.597	2.686
Number of layers	3	5
Number of blows per layer	25	55

Table 4. Summary of compaction tests results

Sample Name	Standard Proctor		Modified Proctor	
	OMC (%)	MDD (g/cm^3)	OMC (%)	MDD (g/cm^3)
A	15.0	1.80	13.0	1.90
B	15.2	1.70	14.0	1.78
C	15.0	1.83	14.0	1.92
D	15.0	1.86	14.0	1.94

3.5: California Bearing Ratio

The California Bearing Ratio are mostly used to estimate the bearing capacity of highway sub - grade and sub – base soil (Simeon *et al.*, 1973; Gidigasu, 1980). FMWH (1997) postulate a CBR of 8% minimum for sub – grade and fill but Asphalt Institute (1962) asserted CBR value of between 0% to 3% for sub – grade and 3% to 7% for sub – base. Based on this, the lateritic soils studied can be classified as having very poor to poor CBR and can only be used as sub – grade and fill materials (Bowles, 1990). In addition, all the soil at the modified proctor energy suggest relatively increased CBR value. This is envisaged due to the fact that the soil has become denser at this energy. However, the soil indicates slightly decreased CBR value after soaking. The CBR value for the soil varied between 2% and 4% (FMWH, 2003).

The CBR values of the samples are stated in Table 5a and 5b. It shows the general rating of soil materials based on the CBR values of the materials and for use as a sub-grade to base course material for support of flexible pavements.

Standard proctor unsoaked and soaked for sample A has 3% and 2% CBR values respectively. Sample B also has 2% values for both unsoaked and soaked respectively. Sample C has 2% values for both unsoaked and soaked respectively while Sample D has 3% and 2% CBR values respectively. Similarly, modified proctor unsoaked and soaked, sample A has 3% CBR respectively, and sample B has 4% CBR and 3% CBR respectively. Sample C also has 4% and 3% CBR while sample D has 3% CBR respectively. Based on the values obtained, they fall within CBR value range of 0-3% and 3-7% respectively (Bowles, 1990).

Consequently, the soil could be useful for slope stability and as sub-grade materials for road construction.

Table 5a. Compaction tests for samples A, B, C and D.

Sample Number	Standard Proctor (%)		Modified Proctor (%)	
	Unsoaked (%)	Soaked (%)	Unsoaked (%)	Soaked (%)

A	3	2	3	3
B	2	2	4	3
C	2	2	4	3
D	3	2	3	3

Table 5b: General Rating of Soil Materials Using CBR values (After Bowles, 1990)

CBR Value	General	Uses	Classification System
0-3	Very Poor	Sub-grade	OH,CH,MH,OL
3-7	Poor fair	Sub-grade	OH,CH,MH,OL
7-20	Fair	Sub-grade	OL,CL,ML,SC
20-50	Good	Base, Sub-grade	GM,GC,SW,SM,SP,GP
50	Excellent	Base	Gw, GM

3.6: Direct Shear tests

The direct shear test is used to determine the angle of internal friction of soil and the effectiveness of pressure. It shows that sample A compacted at standard proctor reveals cohesion value of 70Kpa and angle of internal friction of 27^0 while modified proctor compacted soil sample A gave a cohesion (C) of 90Kpa and has internal friction of 26^0. Sample B compacted at standard proctor has a cohesion of internal friction of 90Kpa and 32^0 respectively. The modified proctor gave a cohesion (C) and internal friction (ϕ) of 70Kpa and 31^0. Sample C compacted at standard proctor has a cohesion of internal friction of 85Kpa and has internal friction of 28^0 while the modified proctor gave a cohesion (C) and internal friction (ϕ) of 80Kpa and 27^0. Sample D compacted at standard proctor has a cohesion of internal friction of 80Kpa and has internal friction of 27^0 while the modified proctor gave a cohesion (C) and internal friction (ϕ) of 70Kpa and 26^0 respectively. Hence, the shear box test revealed that the soil has high bearing capacity having values ranging from 70Kpa to 90Kpa with average of 79Kpa. Similarly, the angle of internal friction ranges from 26^0 to 32^0 with average of 28^0. The tests suggest that the soil is made up of sands and clays. Therefore, all the samples are good as engineering construction (foundation) materials, support slope stability and also all the samples can be used moderately in steep embankment.

4.0: CONCLUSION

The tests carried out in this research work were mainly concerned with using basic geotechnical properties of soil with regards to their uses as construction materials. The grain size analyses show that both samples are sandy clay.

The atterbergs consistency limit for samples A, B, C and D falls on CL- group and indicates that all the samples are inorganic clayey soil of low plasticity which are good to fair particularly as erosion resistance in canal construction.

In compaction tests, the maximum dry densities obtained for all the samples indicates that the soil is suitable as base course in road construction materials.

The CBR numbers obtained from the research work indicate that the soil could be useful for slope stability and as sub-grade materials for road construction.

According to FMWH (1997) and Asphalt Institute (1962), the studied soils have CBR which are considered to be very poor to poor and can only be used as sub-grade and fill materials while shear box test revealed that the soil has high bearing capacity and it support slope stability and steep embankment.

5.0: REFERENCES

Ackroyd, L. W. (1963). The Correlation Between Engineering and Pedological Classification System in Western Nigeria and Its Implication. *Reg. Conf. African Soil and Foundation Engineering, 3rd Ed.*, Salisbury, Rhodesia, Vol.1, pp. 115 - 118.

Alao, D. A. (1983). Geology and Engineering Properties of Lateritic Soil from Ilorin, Nigeria. *Engineering Geology* 1, 111-118. http//dxdoi.org/10160013-7952(83)90029-7

Asphalt Institute (1962). The Asphalt Handbook. Asphalt Institute, Maryland, USA.

Bowles, J.E (1990): Physical and geotechnical Properties of Soil (2nd ed.), *Mc Graw-Hill*, Inc. p.478.

Bowles, J.E. (2012). Engineering Properties of Soils and their Measurements, 4th edition, McGraw Hill Education (India) Private Limited, New Delhi.

Brink, A.B., Partridge A. and Williams A.A.B. (1982). Soil Survey for Engineering, Claredon, Oxford P. 378

Cooray P. G. (1970). Note on the Charnockites of the Akure-Ado Ekiti area, western Nigeria. In Dessauvagie and Whiteman (Ed). *Proceedings of the conference on African Geology*, Dept. of Geology, U.I., (Dec 1970): pp. 45-53.

Enaworu E., Ugbe F.C., Rotimi O.J., 2016: The Geochemistry and Geotechnical (Compaction) Analysis of Lateritic soils: A Case study of Okpanam Area. *AshEse Journal of Engineering.* Vol. 2(3), pp. 075 - 082.

Enaworu, E., Ugbe F. C. Rotimi O. J. Ameloko A. A. (2017). "Geochemistry and Geotechnical Analysis of Lateritic Soils in the Anambra Basin" *Electronic Journal of Geotechnical Engineering*, (22.11), pp 4395-4413. Available at ejge.com.

Federal Ministry of Works and Housing (1997). General Specifications for Roads and Bridge. Volume II. Federal Highway Department, FMWH: Lagos, Nigeria. P. 317.

Federal Ministry of Works and Housing (2003) General Specifications for Roads and Bridges. Federal Ministry of Works and Housing Publication.;2.

Gidigasu, M.D. (1972). Mode of Formation and Geotechnical Characteristics of Laterite Materials of Ghana in Relation to Soil Forming Factors. *Engineering Geology* 6, 79-150. http://dx.doi.org/10.1016/0013-7952*7290034-8.

Gidigasu, M. D. (1980). Geotechnical Evaluation of Residual Gravels in Pavement Construction. *Eng. Geol*.,15, pp. 173 – 192

Ige O.O. (2010). Assessment of Geotechnical Properties of Migmatite-Gneiss Derived Residual Soil from Ilorin, Southwestern Nigeria, as Barrier in Sanitary landfills. Continental J. Earth Sciences 5 (1), pp. 32 – 41.

Jackson, M. O. (1980). The Foundation Failure in Coastal Plain Sand of Nigeria. *Proceedings of the 7thRegional conference for Africa on soil mechanics and foundation Engineering. Institute of Civil Engineering Accra*, *1*, 425-430.

Madu, R. M. (1975). Some Nigerian Residual Lateritic Soil, there Characteristic and Relative Road Building Properties on Group Basis. Proceedings of 6th Regional Conference for Africa Soil Mechanics and Foundation Engineering Durban, South Africa. International Society for Soil Mechanics and Foundation Engineering. 1, 121-129.

Obaro, R. I., Odediran, O. A., Iheme, K. O., Akakuru, O. C. and Gbadebo, A. F. (2019) Geotechnical Evaluation of Lateritic Soil in Baba-Ode Area, Ilorin, North – Central Nigeria, *Journal of Research Information in Civil Engineering*, Vol.16, No.1.

Olusiji, S. A. (2013). Geology and Structure of the Precambrian Rocks in Iworoko Are and Afao Area, Southwestern, Nigeria. International Research Journal of Natural Sciences Vol.1, No. 1, March, pp.14-29

Ogunsanwo, O. (1989). Some Geotechnical Properties of Two Lateritic Soil Compacted at different *Energies Engineering Geology*, 26, 261-269. http://dx.doi.org/10.1016/0013-7952(89)90013-6

Jegede, O. G. and Olaleye, B. M. (2013). Engineering Geological and Foundation Characteristics of Granite Derived Subsoils at Ibuji Area, South Western Nigeria. *The International Journal of Engineering and Science (IJES)* Volume 2, Issue 5, pp. 15 – 17.

Rahaman, M.A (1988). Review of the Basement Geology of South – Western Nigeria. Geology of Nigeria, Edited by Kogbe, C.A. pp. 34 – 35.

Simeon, A.B., Gieseve, J. and Bidlo, G. (1973). Use of Laterite Soils for Road Construction in Northern Dahomey. *Engineering Geology, Amsterdam*, 7: 197-218

Whitlow, R. (1995). Basic Soil Mechanics. 3rd ed. *Addison Wesley Longman Limited*, Edinburg Gate.