Evaluation of Geotechnical Properties of Subgrade Materials Below the Failed and Stable Flexible Pavement along Osogbo-Ikirun Road, South-Western Nigeria

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ABSTRACT

The geotechnical properties of subgrade soils beneath the failed and stable flexible pavement along Osogbo-Ikirun Road, Southwestern Nigeria, were investigated in order to assess their suitability or otherwise as road construction materials. Subgrade samples were collected and were subjected to index properties test, compaction test, and California Bearing Ratio (CBR) test in accordance with the British Standard (BS) 1377 methods for soil testing.

Soil index properties revealed that the subgrade soils composed mainly of fine contents and percentage passing BS No. 200 ranged from 26.4% to 83.4%. The maximum dry density ranged between 1.54 mg/m^3 and 2.04 mg/m^3 while their optimum moisture contents ranged from 9.7% to 21.9%. The liquid limit ranged between 34% and 51%; the plastic limit ranged between 6% and 31% while the plasticity index ranged from 13% to 28%. The CBR (48 hour - soaked) ranged between 5% and 60%. The study concluded from the results of compaction test, Atterberg limits test, and CBR test that the subgrade soil samples are suitable as road construction materials.

(Keywords: geotechnical properties, subgrade soils, particle size test, Atterberg limit test, compaction test, CBR test, soil classification)

INTRODUCTION

Road networks are considered very vital in the economy of many nations, especially developing nations like Nigeria, require roads and highways for transportation of goods and services. Roads are often identified as one of the means of transportation to facilitate the movement of people and materials from one place to another (Gupta, 1986, Singh and Singh, 1991). Oil and gas products, solid minerals, raw and finished products are moved to points of need via roads. Therefore, a good road network is an important element in the physical development of any society as it controls the direction and extent of development.

Furthermore, a road pavement is a continuous stretch of asphalt overlaying the substratum for a smooth ride or drive. Visible cracks, potholes, bulges and depressions may punctuate such a smooth ride. The punctuation in smooth ride is generally regarded as road failure. Major Nigerian highways are known to fail shortly after construction and well before their design age. The problem is apparently more precarious on roadways within the Precambrian Basement complex terrain of the country.

Road failures could be defined as a discontinuity in a road pavement resulting in cracks, potholes, bulges, and depressions (Aigbedion, 2007). Failed roads are characterized by potholes, polishing/pavement surface wash, block and longitudinal cracks, drainage collapse, depression/sinking of roadway, over flooding of the carriageway, gullies and trenches, rutting, and raveling (FMW&H, 1992), some of which are evident at many stretches along the Osogbo – Ikirun Road, South-western Nigeria (Figure 1).

Several factors are responsible for road failures. These include geological, geomorphological, geotechnical, road usage, construction practices, and maintenance (Adegoke-Anthony and Agada, 1980; Ajayi, 1987). Field observations and laboratory experiments carried out by Adegoke-Anthony and Agada (1980), Meshida (1981), and Ajayi (1987) showed that road failures are not primarily due to usage or design/construction problems alone but can equally arise from inadequate knowledge of characteristics and behavior of residual soils on which the roads are built and non-recognition of the influence of geology and geomorphology during the design and construction phases.

The geological factors influencing road failures include the nature of soils (laterites) and the near surface geological sequence, existence of geological structures such as fractures and faults, presence of cavities, existence of ancient stream channels and shear zones. The collapse of concealed subsurface geological structures and other zones of weakness controlled by regional fractures and joint system along with silica leaching which has led to rock deficiency are known to contribute to failures of highways and rail tracks (Nelson and Haigh, 1990). Other factors influencing road failures include swelling clays, shale, fluctuating groundwater levels, and poor drainage.

The subgrade is normally considered to be the *in-situ* soil over which the highway is being constructed. The strength of subgrade is the main factor controlling the design of a flexible pavement. When the subgrade deflects, the overlying flexible pavement is expected to deform to a similar shape and extent. Geotechnical investigation of some sections of Osogbo – Ikirun Road, Southwestern Nigeria can help in assessing the suitability of subgrade soils for use as road construction material.

On this background, this present study was designed to evaluate the geotechnical properties of the subgrade soils along the Osogbo-Ikirun Road with a view to assessing their suitability as construction material.

MATERIALS AND METHODS

Soil Sampling

Subgrade samples were collected from the road between Chainage 0 + 000 at Osogbo Motor Park up to Chainage 20 + 500 in Ikirun, at depths ranging from 0.5 m to 1.0 m with at least two sampling points distributed around each rock type, using Global Positioning System (GPS) to accurately locate the sampling points on a topographic base map. Sixteen samples of subgrade soils were collected from both the failed and stable sections of the road. In all the sites, samples were collected at 1m from the surface using diggers, shovels and graduated measuring scale. About 25 kg samples were collected at 0.50 m interval to 1 m depth below the topsoil for soil classification tests and for determination of the soil strength. Samples taken were placed in plastic bags, properly labelled and sealed against the loss of natural moisture content. Samples and specimens were prepared in accordance with BS 1377 of 1990. Prior to preparing the test specimens, the materials were air dried and broken into smaller fragments, with care being taken not to reduce the size of individual particles.

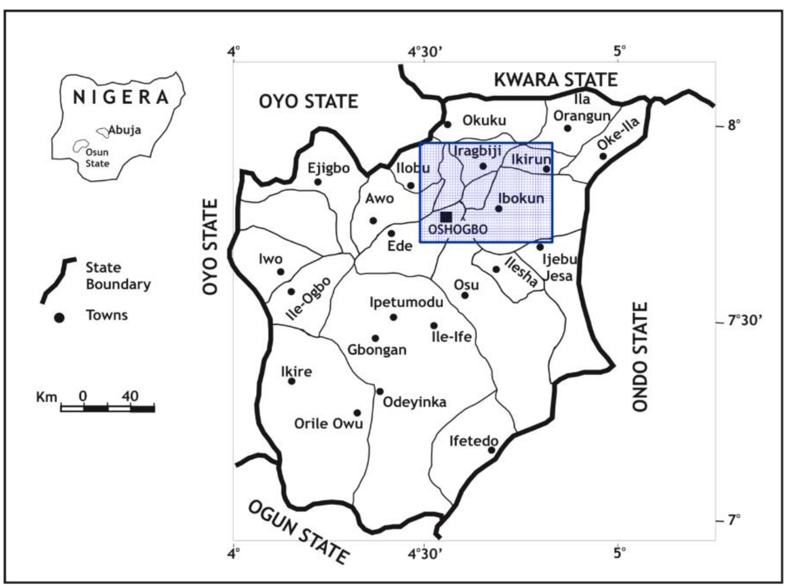


Figure 1: Map of Osun State showing Location of Study Area.

Laboratory Tests

Grain Size Analysis: 1000 g of each air dried soil was soaked in distilled water for about 24 hours. The soil sample was thoroughly washed in a tap water, a little at a time through a 2.000 mm sieve nested in a 0.063 mm sieve until the water passing through the sieves was nearly clear. The soil material passing through the sieves was collected in a container and left undisturbed for 20 minutes for the silt and clay particles to settle down. The clear water was drained. Finally, fractions coarser than 0.063 mm and fractions finer than 0.063 mm were oven dried at 105.0°C for 24 hours and then subjected to mechanical analysis.

Mechanical Analysis: The set of sieves were carefully cleaned and their corresponding weights noted. The oven dried sample with particles coarser than 0.063 mm was sieved through the stack of sieves, with the largest aperture sieve at the top and the smallest aperture sieve at the bottom; having a receiver at the bottom. Ten (10) minutes of shaking was carried out and the amount of soil retained on each sieve was weighed to the nearest 0.01 g. The percentage of total weight of soil passing through each sieve was calculated using the expression:

% retained on a particular sieve = (weight of soil retained on that sieve/ total weight of soil taken) x 100.

Tests for Atterberg Limits and Indices

Liquid Limit Determination: The liquid limit of the soil sample was determined with the standard Casagrande liquid limit apparatus. About 200 g of air-dried soil passing through a BS sieve 425 µm was mixed with water to form a putty-like consistency. The paste was placed in the brass cup of the liquid limit device and levelled so as to have a maximum depth of about 10 mm. The soil in the cup was then cut with a grooving tool. About 10 g of soil near the closed groove was collected and its water content determined. By altering the water content of the soil and repeating the operations, four to five readings of water content in the range of 10 to 40 blows were obtained. A graph was then plotted between number of blows, N on logarithmic scale and the water content, w, on the arithmetic scale.

Determination of Plastic Limit: Plastic limit is used to determine the water content at which a soil sample crumbles when rolled into a thread of 3 mm diameter. About 200 g of air-dried soil passing through BS sieve 0.425 mm (No. 40) was used for plastic limit determination. The sample was mixed with sufficient amount of water to promote ball formation. A portion of the ball was rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length. Threading and remolding was repeated until the thread at a diameter of 3 mm began to crumble. Some of the crumbled portion of thread was kept in an oven for water content determination. The test was done in triplicate with fresh samples and the average was taken as the plastic limit.

Compaction Test: Compaction is used to increase the density of soil sample mechanically. The empty mold was weighed and a 6.0 kg representative specimen of the sample was obtained; crushed into fine particles and then sieved through a No. 4 sieve (4.750 mm sieve). Finally, the compaction test was carried out using the standard procedure.

California Bearing Ratio (CBR) Test: CBR is used to determine the extent to which ingress of water would reduce the strength and increase the volume of subgrade soils. The strength of the subgrade is the main factor in determining the thickness of road pavement. Subgrade strength is expressed in terms of its California bearing ratio (CBR) value. Standard procedure was used to determine the CBR values. The force gauge was recorded at intervals of 0.25 mm penetration up to 7.5 mm. Test results were plotted in the form of a load-penetration diagram by drawing a curve through the experimental points. Penetrations of 2.5 mm and 5.0 mm were used for calculating the CBR value. From the test curve, the forces corresponding to 2.5 mm and 5.0 mm penetrations were read off. These were expressed as percentage of the standard forces at those penetrations. The higher percentage was considered as the CBR value.

RESULTS AND DISCUSSION

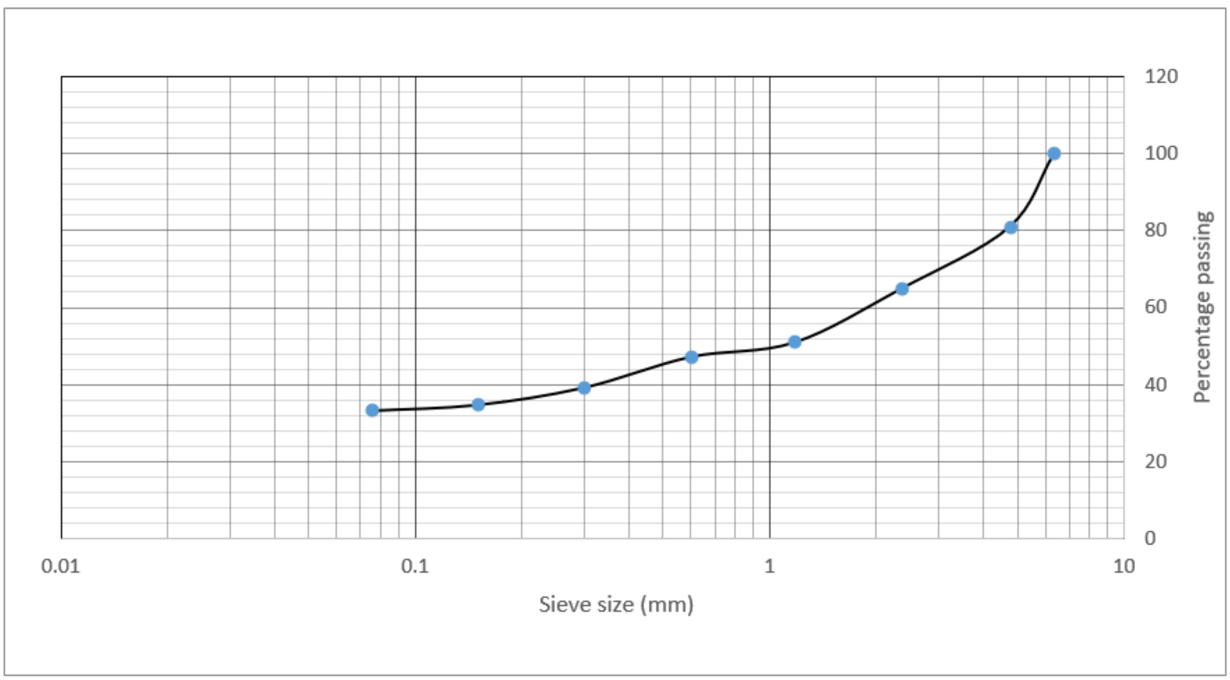
Grain Size Distribution

The results of the grain size distribution analyses are presented in Table 1 while a typical grading curve is presented in Figure 2.

Table 1: Grain Size Distribution of the Subgrade Soils.

Sample Location	CHO + 000	CHO + 000	CH2 + 200	CH2 + 200	CH3 + 500	CH3 + 500	CH5 + 800	CH5 + 800	CH9 + 800	CH9 + 800	CH11 + 100	CH11 + 100	CH14+ 450	CH14+ 450	CH20 + 150	CH20 + 150
Sample No.	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12	S13	S14	S15	S16
Parent Rock	Schist	Schist	Schist	Schist	Banded Gneiss	Banded Gneiss	Banded Gneiss	Banded Gneiss	Pegmatite	Pegmatite	Pegmatite	Pegmatite	Undifferentiated Schist	Undifferentiated Schist	Undifferentiated Schist	Undifferentiate d Schist
% Passing 0.075µm	36.6	33.2	33.1	32.1	28.2	33.1	54.5	33.0	83.4	34.2	26.4	73.9	37.6	75.9	81.3	60.5
Liquid Limit (%)	36	38	38	36	34	36	51	42	37	37	36	37	46	42	35	40
Plastic Limit (%)	29	26	24	27	28	27	23	24	23	23	13	28	25	27	27	31
Plastic Index (%)	7	12	14	9	6	9	28	18	14	14	23	9	21	15	8	9
Maximum Dry Density (mg/m³)	1.93	1.93	2.04	1.83	2.01	1.98	1.57	1.54	1.73	1.91	2.02	1.89	1.64	1.71	1.63	1.69
Optimum Moisture Content (%)	10.7	13.3	13.5	14.9	9.7	14.5	21.9	15.1	17.1	14.6	9.4	12.3	17.4	14.9	19.7	18.8
CBR (soaked %)	60	60	49	37	59	57	12	5	25	56	59	47	9	29		
AASHTO Classification	A-4	A-2-6	A-2-6	A-2-4	A-2-4	A-2-4	A-7-5	A-7-5	A-2-6	A-2-6	A-2-6	A-4	A-7-5	A-7-5	A-4	A-4
USCS Classification	SC	SC	SC	SC	SC	SC	SC	SC	SC	SC	SC	SC	SC	CL	CL	CL
Linear Shrinkage	4	6	7	5	4	5	14	12	7	7	11	7	10	7	6	7
Description of material	Reddish brown sand	Reddish brown sand	Brown clayey sand	Brown clayey sand	Reddish brown sand	Reddish brown sand	Brown clayey sand	Brown clayey sand	Reddish brown sand	Reddish brown sand	Dark brown sand	Dark brown sand	Reddish brown sand	Reddish brown sand	Reddish brown sand	Reddish brown sand

Figure 2: Grain Size Distribution Curve for Soil Sample taken from Chainage 2+200 (S3).



The result shows that the number of fines of the subgrade soil range between 26.1% and 83.4%. Samples with high clay and silt contents are reported to be susceptible to expansion. Volume changes from moisture influx into the subgrade could reduce the load bearing capacity of the soil, thereby causing unsatisfactory behavior as was observed from failed sections at Chainage 5 + 800 and Chainage 14 + 450.

Atterberg Limits

The results of Atterberg limits tests are also presented in Table 1. These include liquid limits, plastic limits and plasticity index of the soil samples. The liquid limits of the soil samples range between 34% and 51% and plastic limits between 6% and 31% while plasticity index is between 6% and 27%. Standard for road works recommend liquid limits of 50% maximum for subbase and base materials (FMWH, 1997). Liquid limit less than 30% indicates low plasticity, 35% to 50% indicates intermediate plasticity, 50% and above indicates high plasticity (Whitlow, 1995).

All samples exhibit medium plasticity, except sample S7 at Chainage 5 + 800, which exhibits high plasticity (Figure 3). This type of liquid limit can be expected for silty soils which usually have typical values of 25% to 50%. Liquid limit of 50 (maximum) and plasticity index of 15 (maximum) have been recommended, thus samples S7, S11 and S12 of Chainages 5 +800 and 11 + 100 (failed sections) do not fall within the recommended range.

Liquid limit is an important index property since it can be correlated with various engineering properties. Ige (2010) and FMWH (1997) recommended that subgrade/fill material should have liquid limit ≤50% and plasticity index ≤ 30% while for sub-base, liquid limit should be ≤ 30% and plasticity index ≤ 12%. Wright (1986) stated that the liquid limit values of 40% and above are assumed high in pavement construction. Plasticity index value of 10% and above is assumed high in pavement design. All the soils met the requirement for use as subgrade/fill materials except samples S7, S11 and S12 with high plasticity index of 29, 23 and 21 which were obtained from failed sections of Chainages 5 + 800 and 11 + 100, respectively.

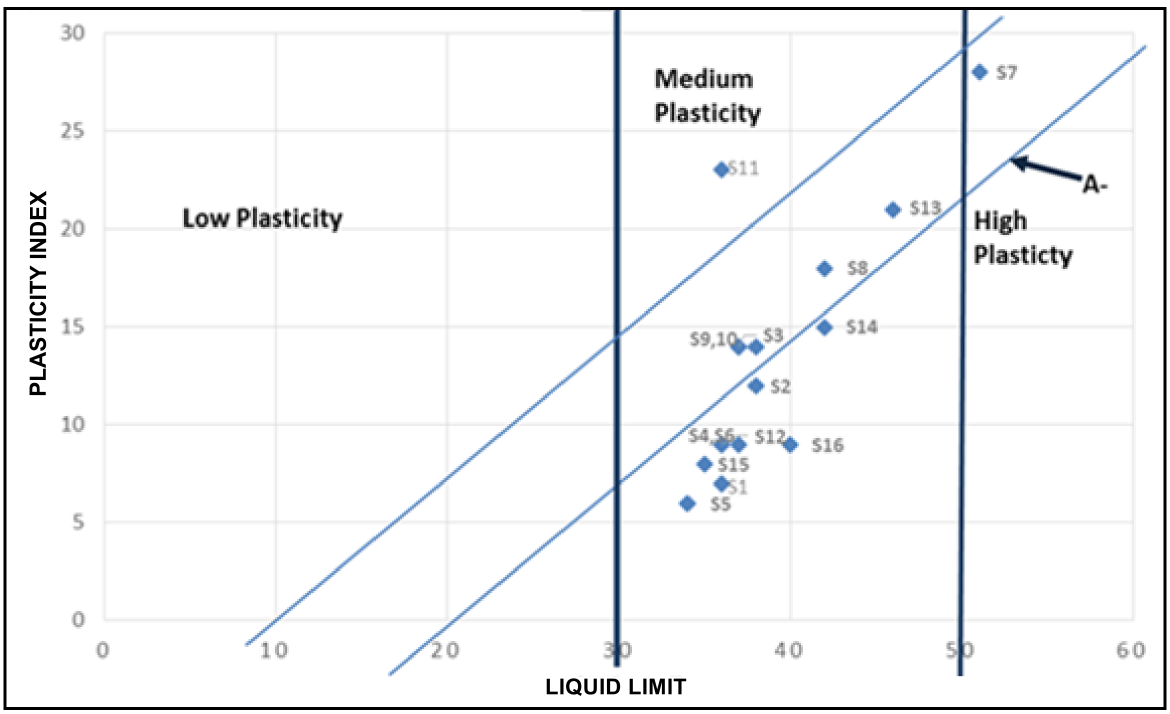


Figure 3: Casangrade Classification Chart of the Soil Samples.

Compaction

The results of the compaction test (West African Level) are presented in Table 1, while a typical compaction curve for sample S1 is presented in Figure 4. There is a variation in the optimum moisture content (OMC) and maximum dry density (MDD) as observed in Figure 4. This variation in the OMC and MDD values reflect the influence of compaction on the soil samples before moisture – density relations of the soils were determined. The West African levels compactive effort was used. The MDD ranges between 1.54 mg/m3 and 2.04 mg/m3 while the OMC ranges between 9.7 % and 21.9 %.

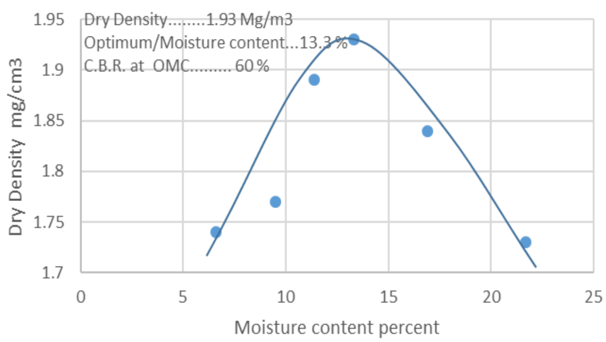


Figure 4: Compaction Curve for Soil Sample taken from Chainage 0+000 (S2).

Compaction places soils in a denser state and hence decreases compressibility, increases shear strength and decreases permeability. It is possible to control the dry density and moisture content so that the soils produced exhibit most of the properties desired, such as consolidation, permeability, and others (Gidigasu, 1976). The compaction values of these soils are considered good, if 100% of the MDD and OMC are attained during field compaction (Ogundipe, 2012). The relatively good values of the compaction properties possessed by these soils after comparison with standard specifications indicate good road construction materials. In general, these soils will be suitable for use as fill materials and sub-grade materials in road construction.

California Bearing Ratio (CBR)

The results of the CBR test are presented in Table 1, while a typical CBR curve for sample S1 is presented in Figure 5. The CBR (soaked) for the subgrade soil samples range between 5 % and 60 %. The minimum CBR requirement for materials to be used in the construction of highway pavement should not be less than 80% (unsoaked), 30% (soaked) and 10% (soaked) for base, subbase and subgrade materials, respectively (FMWH 1992). This requires careful selection of materials.

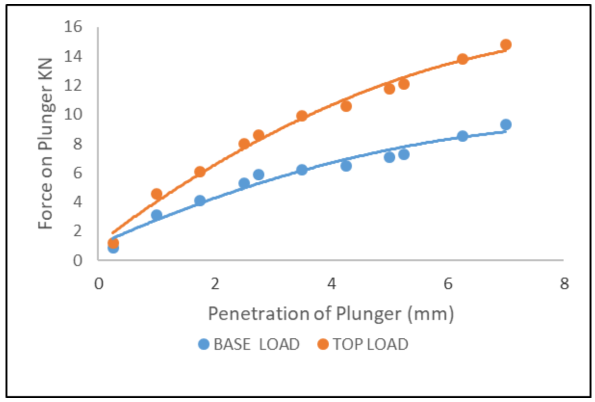


Figure 5: Typical CBR Curve for Soil Sample taken from Chainage 0+000 (S1).

It can be seen that all the samples satisfy the condition for subgrade material except samples S7 and S12 with CBR of 5% and 9% respectively and samples S15 and S16 which were too soaked and failed to be compacted for CBR. The result shows that samples at failed segments have lower CBR than most of the other samples collected from the stable segments. However, samples at failed segments have higher optimum moisture content than most of the samples from the stable segment.

Soils shrink or swell depending on their clay and moisture contents. Swelling of soil could lead to the deformation and cracking of pavements constructed over them. Some of the soil samples were collected over areas where swelling was observed. The swelling of the samples may be due to the presence of expansive clay materials in the soil. The subgrade of the failed sections in chainages 5 + 800, 9 + 800 and 14 + 450 may be an expansive soil. This swelling might be the possible cause of the down-warping and cracking of the flexible pavement in the area. Augus (2006) observed that vertical swelling of the subgrade will decrease the bearing capacity since it increases void ratio and water content. The subgrade will suffer appreciable reduction in strength if there is ingress of water below the pavement, as this will decrease the strength of the subgrade and the pavement constructed on it will fail.

CONCLUSION

Grain size distribution of the soils showed that the fine contents percentage passing sieve No. 200 BS ranged between 26.4% and 83.4%. On the basis of the Atterberg consistency limits, all the soil samples met the requirement for use as sub-grade/fill materials, except soil sample S7 at Chainage 5 + 800, which exhibited high plasticity. All the soil samples exhibited liquid limits and liquidity index within the range considered suitable for use as road construction materials. The study concluded from the results of compaction, Atterberg limits, and CBR tests that the subgrade soil samples are suitable as road construction materials.

REFERENCES

1. Adegoke–Anthony, W.C., and O.A. Agada. 1980. "Geotechnical Characteristics of Some Residual Soils and their Implications on Road Design in Nigeria". Technical Lecture: Lagos, Nigeria. pp.1 – 16.

2. Aigbedion, I. 2007. "Geophysical Investigation of Road Failure Using Electromagnetic Profiles along Opoji, Uwenlenbo and Illeh in Ekpoma-Nigeria". *Middle-East Journal of Scientific Research*. 2(3-4): 111-115.

3. Ajayi, L.A. 1987. "Proposal for Engineering Classification of Nigerian Soils". *Proceedings of the First National Conference Nigerian Geotechnical Association*: Lagos, Nigeria.

4. Augus, S.M. 2006. "The Swelling of Expansive Subgrade at Watespurworejo Road, STA, 8 + 127". *Civil Engineering Dimension.* 8(2):106-110.

5. Federal Ministry of Works and Housing (FMWH). 1992. *Highway Road Maintenance Manual Part II*. FMWH: Abuja, Nigeria.

6. Gidigasu, M.D. 1976. *Laterite Soil Engineering*. Elsevier: Amsterdam, The Netherlands. 554p

7. Gupta, B.L and A.K. Gupta. 1986. *Texbook of Road, Railway, Bridge and Tunnel Engineering*. Standard Publishers: New Delhi, India.

8. 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): 32 – 41.

9. Mesida, E.A. 1981. "Laterites on the Highways – Understanding Soil Behaviour". *West African Technical Review*. 112 – 118.

10. Nelson, R.G., and J.H. Haigh. 1990. "Geophysical Investigation of in Lateritic Terrain". *Geotechnical and Environmental Geophysics*. Ward, S.H. (Ed), (Geotechnical), SEG, Tulsia. 2: pp.133 – 154.

11. Ogundipe, O.O. 2012. "Road Pavement Failure Caused by Poor Soil Properties along Aramoko-Ilesha Highway, Nigeria". *Journal of Engineering and Applied Sciences*. 3(3): 239-241.

12. Singh, G. and J. Singh. 1991. *Highway Engineering.* Standard Publishers Distributors: New Delhi, India.

13. Whitlow, R. 1995. *Basic Soil Mechanics*. 3rd Ed. Addison Wesley Longman Limited: Edinburgh Gate, UK.

14. Wright, P.H. 1986. *Highway Engineering.* 6th Ed. John Willey and Sons: New York, NY.

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