Assessment of Gneiss-Derived Residual Soils as Materials Used in Road Pavement Structures

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Abstract. Failure of highway pavement and collapse of building in basement complex of Nigeria is often related to the instability of the residual. This study evaluated the strength characteristics of gneiss-derived residual Soils as materials usable for road pavement structures. A total of eleven soil samples derived from granite gneiss were subjected to laboratory geotechnical analyses based on standard practices. The geotechnical analyses reveal the soils’ natural moisture content, specific gravity, grain sizes, consistency limits, shearing strength s, maximum dry density, and optimum moisture content. Based on AASHTO classification, the soil samples are classified as A-7-6, A-6, and A-7-5. The results of the laboratory analyses revealed that the natural moisture content and specific gravity ranged from 8.30 to 22.70% and 2.6 to 2.8 respectively. Particle size analysis reveals that the coarse contents of the soils ranged from 28.8% to 59.8% and amount of fines ranged from 40.2 to 71.2%. The liquid limit ranged from 31.3% to 68.3%, plastic limit ranged from 20% to 28.0%, plasticity index ranged from 4.8% to 38.90% and linear shrinkage ranged from 5.7 to 13.6%. The maximum dry density ranged from 1481 kg/m³ to 1921 kg/m³ and optimum moisture content ranged from 15.2% to 27.6%. Undrained triaxial shear strength (Cu) ranged from 43.0 Kpa to 250.3Kpa, angle of friction ranges from 11.7 to 29.30, and unconfined compressive strength ranged from 153 to 356.5Kpa. The results indicate that the residual soils are poor sub-grade and foundation materials due to their high amount of fines, linear shrinkage values, plasticity, and swelling potential, as well as low maximum dry density.

Introduction

Construction of durable road is a vital infrastructure in the socio-economic development of most nations as it links up states and allow exchange of economic values, creation of job opportunities and sustainability of life. The nature of the subgrade and competence of the construction materials used in the pavement design and construction are some of the factors that affect the longevity of roads. Several road failure features are noticeable overtime after the completion of the designed road with obvious destructive signs such as cracking, rutting, potholes, differential heave, deformation and peeling. This results into rehabilitation, high cost of maintenance, loss of lives and properties. The suitability of any road pavement structure depends largely on the engineering properties and the strength capability of the underlying soil. In tropical areas, the most available geological materials suitable for road construction is the residual soil derived from the insitu bedrock. Understanding on the Engineering characteristics of residual soil from the insitu rock to weathered soil is essential for the design and construction of pavement structures.

Residual soils are weathered products that are found under unsaturated condition dependent on the degree of weathering. The formation of residual soils is through physical and chemical weathering of bedrock in situ with little or no movement of the individual soil particles [1]. The soils tend to be characterized by angular to sub-angular particles, mineralogy similar to parent rock, and the presence of large angular fragments within the overall soil mass [2]. They are highly structured soils which may be cemented depending on the degree of weathering. Temperature and other tropic related factors have favoured the development of significant thicknesses of residual soils in many parts of the world.
particularly in West Africa. These soils tend to be more abundant in humid and warm regions that are favourable to chemical weathering of rock, and have sufficient vegetation to keep the weathered products (residual soils) from being easily transported as sediments [1]. They are often unsaturated in nature and possess negative pore-water pressures or matric suctions relative to the atmospheric conditions that contribute to its shear strength [3]. The shear strength behaviour of residual soils are known to be affected significantly by the bonds between the particles which defines the cohesive nature of the soils.

Previous research works have been limited to investigating the engineering properties of residual soils and the effects of soil structure in the engineering properties of the soils [3, 4, 2]. Many of these research works have been carried out on a broad scale. Only a few in-depth studies on residual soils underlain by an individual rock unit have been carried out [5]. This research work aims to assess the strength characteristics of gneiss derived residual soils within Akure metropolis as materials for road pavement structure. Akure is a fast-growing metropolitan city and, since it became the administrative capital of Ondo state in 1976. The city has experienced rapid urbanization and increased population. Classification and strength tests must be performed on the underlying soils to ascertain their nature and to which construction works are well suited for [6].

Study Area

The study area falls within the tropical rain forest of South-western Nigeria and is within latitude 7°17’N and 7°19’N and longitude 5°07’E and 5°09’E. There exists two major seasons; namely wet and dry seasons. The study area experiences frequent rainfall between April and July with a short break in August and continues between September and November, while the dry season is usually between November and March [7]. The drainage pattern is dendritic and the major rivers are; Ala and Ogburugburu rivers which flow in the south east direction. The climate falls under the equatorial climate belt of Southwestern Nigeria. The annual average temperature is 26.7°C while the average annual rainfall is 1334mm [8].
Geology Settings of the Area

The area is specifically underlain by Granites, Gneisses, Charnockites and Quartzites which are members of Precambrian Basement complex rocks of Nigeria [10]. The spatial distributions of these lithological units are shown in (Fig. 1). Gneisses are the most dominant rock types covering over 60% of the study area. The gneisses are essentially granite gneiss that forms an important rock member of the migmatites which differs in terms of their gradation [11]. These gneisses display coarse to medium gneissose textures. Structurally, the gneisses contain a lot of mineral lineation which are believed to have resulted from the first orogenic event in the locality. They also contain quartz and quartzofeldspathic veins, fractures, faults, macro and micro fold structures which are as a result of the latter deformation of the gneissic banding. The granites occur as leucocratic or felsic plutonic igneous rock bodies and has characteristic porphyritic texture. They show exfoliation planes resulting from weathering of the rocks. The charnockite rocks in the study area occur as low lying outcrops, especially around the north-eastern region of the area and have a characteristic dark green colouration. The quartzite bodies occur as long elongated ridges trending North-east and South-west [12].

Materials and Methods

The methodology adopted involves reconnaissance survey of the area, soil sampling, laboratory analysis and interpretation of the results. Reconnaissance survey was carried out to assess the geological characteristics of the area. This includes geological field mapping in delineating different rock types in the area. Generally, the nature of rocks, soils and water bodies in the area was observed.

A total of eleven (11) bulk disturbed soil samples were collected from the subgrade zone in a borrow pits at a depth of 1-2m with the aid of diggers, shovels, tape rule. The disturbed samples retrieved from each location were put inside fifty (50) litres Bagco bags. Samples to determine the natural moisture content were placed inside the polythene bags. They were closely sealed to avoid loss of moisture. The natural moisture content of each of the samples collected was determined immediately it was taken to the laboratory. The soil samples were air-dried for a week to obtain constant moisture content. This was followed by transporting the collected samples to the laboratory for the geotechnical analyses. The collected samples were subjected to two classes of laboratory tests, which include: soil classification tests (natural moisture content, Atterberg limits, specific gravity and particle size distribution) and soil strength tests (standard proctor compaction tests, unconfined compressive strength, consolidation, undrained triaxial). These analyses were carried out in accordance with [(BS) 1377 (1990), (BSI) 5930 (1990)] [13]. The soils were classified and rated based on their engineering properties using the American Association of State Highways and Transportation Officials (AASHTO) Classification System and clay activities [14].

Soil Description

The soil profile was described together with the colour and texture characteristics. The sub-surface soils in the study area are composed largely of residual type of soils formed as weathering products of the basement rocks. The soils are reddish to brownish in colour due to presence of iron oxide (Fig. 2). The soils are composed of fine to medium grained materials with pockets of yellowish clayey sand.
Results and Discussion

The results of the engineering properties of the soils are summarised in Table 1.

Table 1: Summary of the geotechnical properties of the soils

| Sample No. | NMC (%) | Specific Gravity | Gravel | Sand (%) | Silt (%) | Clay (%) | Plastic Limit | Liquid limit | Linear Shrinkage | Plasticity Index | MDD (kN/m³) | OMC (%) | Cohesive Value (kPa) | Angle of Friction (°) | UCS (MPa) | AASHTO |
|------------|---------|-----------------|--------|----------|---------|---------|---------------|-------------|-----------------|-----------------|-------------|--------|----------------------|----------------------|-----------|--------|
| 1          | 9.65    | 2.605           | 32.4   | 24.7     | 18.1    | 24.8    | 28.0          | 58.9        | 12.9            | 30.9            | 1896        | 15.9   | 75.7                 | 27.0                 | 250.1     | A-7-6  |
| 2          | 22.70   | 2.605           | 2.4    | 27.3     | 17.1    | 53.3    | 28.0          | 68.3        | 13.6            | 35.80           | 1560        | 25.4   | 84.6                 | 22.5                 | 356.5     | A-7-6  |
| 3          | 9.17    | 2.625           | 34.4   | 25.4     | 18.6    | 21.6    | 26.0          | 50.2        | 11.4            | 24.20           | 1921        | 15.2   | 85.8                 | 26.6                 | 296.4     | A-7-6  |
| 4          | 19.76   | 2.625           | 2.3    | 33.0     | 15.9    | 48.9    | 26.2          | 54.0        | 10.7            | 27.85           | 1656        | 22.7   | 57.5                 | 24.6                 | 317.1     | A-7-6  |
| 5          | 8.30    | 2.800           | 4.2    | 29.2     | 16.3    | 50.3    | 20.0          | 58.9        | 10.0            | 38.90           | 1560        | 24.9   | 250.3                | 11.7                 | 220.9     | A-7-6  |
| 6          | 9.20    | 2.780           | 1.0    | 52.1     | 7.0     | 40.0    | 26.4          | 31.3        | 5.7             | 4.88            | 1560        | 18.7   | 125.8                | 23.6                 | 162.0     | A-6    |
| 7          | 11.70   | 2.520           | 8.6    | 30.2     | 9.5     | 51.8    | 21.4          | 48.1        | 7.1             | 26.67           | 1557        | 21.1   | 184.3                | 15.6                 | 172.2     | A-7-6  |
| 8          | 11.40   | 2.520           | 18.1   | 30.3     | 8.2     | 43.1    | 20.6          | 32.5        | 12.1            | 12.91           | 1560        | 20.9   | 151.1                | 15.8                 | 153.1     | A-6    |
| 9          | 15.87   | 2.605           | 2.8    | 28.6     | 14.6    | 54.0    | 24.0          | 54.8        | 10.7            | 30.80           | 1613        | 23.9   | 43.0                 | 25.1                 | 265.3     | A-7-5  |
| 10         | 15.07   | 2.705           | 8.3    | 20.5     | 20.0    | 51.2    | 27.7          | 64.0        | 14.3            | 36.30           | 1481        | 27.6   | 46.0                 | 27.4                 | 245.4     | A-7-6  |
| 11         | 13.00   | 2.706           | 6.6    | 25.4     | 33.2    | 34.8    | 27.0          | 49.6        | 12.9            | 22.60           | 1624        | 23.6   | 43.5                 | 29.3                 | 286.5     | A-7-6  |

Discussion

Natural Moisture Content

Natural moisture is one of the physical properties of engineering soil which helps to evaluate the suitability of the subsoil materials for engineering construction purposes. The possible variation in the moisture content of residual soils depends on the annual rainfall, drainage condition and depth of sample [15]. High variation in the moisture content causes large volume changes in the clayey soils [16]. Underwood [17] stated that soils with natural moisture content of 5%–15% are suitable engineering materials while soils with natural moisture content values ranging from 20 to 35% are unfavourable engineering materials. The natural moisture contents of the soil samples ranges in values from 8.3% to 22.70% (Table 1). In harmony with Underwood’s [17] soil classification based on moisture contents, only three samples are classified as unfavourable engineering materials.
Emesiobi [18] classified soil types based on moisture content and indicate that moisture content in gravel and sand ranged from 5% - 50%. A comparison of Emesiobi [18] classification with the test results indicates that the samples falls within sand and gravel. This results shows low water adsorption capability of the soil materials. Values in this range of 5%–15% have been showed to result in increase in the shear strength of road construction materials [19]. Hence, they are suitable as subgrade, subbase and base materials. Generally, low variation in the moisture content causes small volume changes in the clayey soils. This shows that the soil samples contain appreciable amount of moisture which is largely affected by climatic conditions.

**Specific Gravity**

Specific gravity is known to determine with mechanical strength of laterite aggregates which aids accurate selection of suitable highway pavement construction materials [20]. The specific gravity values for the studied soil samples ranged from 2.60 to 2.80 (Table 1). These values have not deviated from those generally obtained for residual soils within the basement complex of Nigeria [21]. The specific gravity of quartz mineral is 2.65 while that of montmorillonite and Illite falls within 2.62-2.83 [22]. This indicates that clay minerals in the gneisses are responsible for the variation in the specific gravity.

**Particle Size Distribution**

The analysis was carried out to determine particle distribution of the studied soils. The gravel contents of the soils ranged from 1.0% to 34.4%, the sand contents ranged from 20.5% to 52.1%, the silt contents ranged from 7.0% to 33.2%, while the clay contents ranged from 21.6% to 54.0% (Table 1). The Federal Ministry of Works and Housing (FMWH) [23] standard specification requires subgrade soils to possess less than 35% amount of fines. A comparison of the results obtained with this standard specification holds that no soil sample falls within the specification. Therefore, the soil samples cannot be used as subgrade materials for any engineering design. Olabode and Asiwaju-Bello [2] stated that high amount of fines (greater than 35%) and low amount of coarse contents (less than 65%) suggest high amount of micas and feldspars that is mostly found in gneisses and charnockite. This largely influences the behaviour and the engineering properties of the soils in proportion to its abundance. Therefore the abundance of mica and feldspars in residual soils will affect the suitability of the soils. Most of the tested soil samples tend to possess high amount of fines. The fines usually contribute to mechanical instability, hence they possess high plasticity and swelling potential. From the grain size characteristics curve in Fig. 3, all the soils in the study area are poorly graded revealing an increasing degree of both leaching and weathering.

**Consistency Limits**

The Atterberg limit test is used to obtain index information about soils which is used to estimate strength and settlement characteristics. The values of liquid limit test ranged from 31.3% to 68.3%, the plastic limit values ranged from 20% to 28.0%, plasticity index values ranged from 4.8% to 38.90%, and linear shrinkage ranged from 5.7 to 13.6% (Table 1). The FMWH [23] recommend 40%, 30% and 20% maximum limits for the liquid limit, plastic limit and plasticity index respectively for foundation materials. A comparison of liquid limit results with this standard specification indicate that eighteen percent (18%) of the soil samples satisfy the specification and eighty two percent (82%) of soil samples did not satisfy the condition. Very high liquid limit tends to cause low bearing capacity and highway pavement failure. In harmony with FMWH [11] standard specification for plastic limit indicate that all the soil samples satisfy the specification. High plastic soils tend to be susceptible to high compressibility, swelling on moisture influx, low bearing capacity and leading to low permeability Olabode and Asiwaju-Bello [2]. Eighteen percent (18%) soil samples were consistent with FMWH [23] specification of less than 20% maximum for plasticity index while eighty two percent (82%) percent did not fall within the specification. Hence, the soils are not suitable as foundation subgrade materials. According to Bell [24], liquid limit less than 35% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, 50% to 70% indicates high plasticity and 70 to 90% indicates very high plasticity. Based on this classification, four (4) soil samples possessed low
plasticity and seven (7) soil samples possessed high plasticity. The soil samples with low and medium plasticity will not pose field compaction problem. High plasticity of a soil may result to high compressibility and swelling potential. The linear shrinkage values of the soil samples ranged from 5.7 to 13.6 (Table 1). Two (2) soil samples satisfy the specification by Madedor [25] of 8% maximum to be suitable as sub grade materials. These samples are suitable for highway pavement materials. The other nine (9) samples that does not satisfy the specification could cause swelling and shrinkage during wet and dry seasons of humid tropical climatic conditions. These characteristics make the soils unsuitable for foundation and highway sub grade materials. Plasticity index and linear shrinkage are controlled by the amount of fines and abundance of clay minerals in the soils. Hence higher linear shrinkage possess the tendency for the soil to be unstable and problematic under mechanical energy. The relationship between plasticity index and liquid limit plotted on plasticity chart indicates that most of the soils fall above A-line except location 6 (Fig. 4). This implies that they are composed of inorganic materials with medium to high plasticity. The soils with low and medium compressibility will not pose problems on the field while those of high compressibility will pose significant problem.

AASHTO Classification and Clay Activity

The AASHTO classification system is based on the following three soil properties: Particle size distribution, Liquid limit and Plasticity index. According to the classification, soils with more than 35% pass through the No. 200 sieve are classified under groups A-4 to A-7. The result of the soil classification (as shown in Fig. 5) shows that eight of the tested soils classify as A-7-6 soils, two classify as A-6 soils, while one classify as A-7-5 soils. This classification denotes that the soils are silty and clayey sand types. The nature of these soils is rated as fair to poor sub grade materials.

Clay “activity” is an important index property used to determine the swelling potential of soils [26]. This combined atterberg limits and clay size content into a single parameter called “activity”. It is expressed by the ratio of plasticity index to the percentage of the soil fraction finer than two microns. He suggested three classes of clays and possible minerals to exhibit the properties viz; the inactive clays with inactivity values less than 0.75, normal clays with activity values ranging between 0.75 and 1.25 and the active clays with activity values greater than 1.25. The activity of clays form Fig. 4 indicates inactive to normal clays and shows that the possible clay mineral that can exhibit such an activity is Kaolinite. Two soil samples have low expansion potential while nine samples possess medium to high expansion potential as obtained in the activity chart.
Figure 3: Grain size distribution curves for the soils

Figure 4: Plasticity Chart for soil samples
Figure 5: AASHTO Classification Chart

Figure 6: Clay activity chart (Modified after [26]).
Compaction

Compaction of soils constrains the particles to pack more closely through reduction in the air voids by mechanical means [27]. It gives appropriate information on the stability and densification of the particles when subjected to different energy [28]. A typical amount of compaction effort on the soil will produce an optimum moisture content at which the soil can be compacted to the maximum dry density. The strength and performance of a pavement depends on the bearing capacity of the subgrade soils [29]. The summary of the compaction test results are presented in form of moisture-density curves (Fig. 7). The Maximum Dry Density (MDD) ranged from 1481 kg/m$^3$ to 1921 kg/m$^3$ and the optimum moisture content ranges from 15.2% to 27.6% for the soil samples tested. A comparison of Woods [30] proposition (Table 2) with the test results indicate that two soil samples fall within the fair foundation materials, three soil samples fall within the poor foundation materials, while six samples are very poor foundation materials. Compaction process produces higher maximum dry density which increases the strength and bearing capacity and reduces the compressibility and permeability of soils. The abundance of fine particles will reduce the maximum dry density and affect the bearing capacity of soils. Therefore higher amount of fines have significant influence on the maximum dry density. The best soils are those that have high maximum dry density values at relatively low optimum moisture content [31]. Low maximum dry density value is always a contributory factor to the frequent occurrence of road failures.

![Compaction curves for the soils](image)

### Table 2: Foundation suitability of soils using range of Maximum Dry Density values [30]

| Maximum Dry Density (kg/m$^3$) | Remark for foundation | Sampled Soils in Range |
|-------------------------------|-----------------------|------------------------|
| Greater than 2082.6           | Excellent             | Nil                    |
| 1922.4-2082.6                 | Good                  | Nill                   |
| 1762.2-1922.4                 | Fair                  | 2                      |
| 1602-1762.2                   | Poor                  | 3                      |
| 1121.4-1602                   | Very poor             | 6                      |
Undrained Triaxial test

The shear strength of the sampled soils was determined by carrying out undrained triaxial compression test. Shear strength is a term used in soil mechanics to describe the magnitude of the shearing stress that a soil can sustain under applied pressure. The shear resistance of soil is a result of friction between interlocking particles, and possibly bonding at particle contacts. Undrained shear strength (Cu) values varied from 43.0KPa to 250.3KPa and the angle of friction values ranged from 11.7 to 29.30 were obtained. The British Standard 5930 [32] classified cohesive soils based on their shear strength, given that Cu values between 0-20 KPa indicates very soft, 20 and 40 KPa indicates soft, 40 and 75 KPa indicates firm, 75-150 KPa indicates stiff, and 150-300 KPa indicates very stiff soils. Based on this recommendation, the sampled soils are firm to very stiff which indicate high shear strength. The high values for angle of internal friction can be attributed to the presence of coarse grained particles present in the sampled soils. The Mohr’s failure curves are presented in Fig. 8 (a-d).

Figure 8 (a-d): Mohr Failure Cycles
Unconfined Compressive Strength

Unconfined compressive strength expressed the compressive behaviour of soils. It is the core basis for modelling the stress-strain relationship [33]. Compressive strength value is a measure of suitability of the soil as foundation materials. Unconfined compressive strength (UCS) ranged from 153 to 356.5Kpa (Fig. 9 [a-d]). Das [22] classified the clayey soils based on their compressive strength, given that UCS values between 0 and 25Kpa indicates very soft, 25 and 50Kpa indicates soft, 50 and 100Kpa indicates medium, 100 and 200Kpa indicates stiff, 200 and 400Kpa indicates very stiff, and greater than 400Kpa indicates hard soils. A comparison of this specification with the results shows that the soils are stiff to very stiff clayey materials and the shear strength of the soils is high. This indicates that the soils are suitable for sub grade foundation materials.

![Unconfined Compression Test](image)

Figure 9 (a-b): Stress - strain relationship of the soils
Conclusion

This study has provided insight into the influence of bedrock properties on the strength characteristics of residual soils. These gneiss derived soils possess low strength for highway pavement structures. AASHTO [14] classification suggests that the soils are silty and clayey materials, and are fair to poor highway and foundation materials. The high expansion potential confirmed by the “activity” chart indicates that the soils can pose engineering problems and unsuitable as sub-grade materials. Higher linear shrinkage characteristics could cause shrinkage and swelling problems which will in-turn result to difficulties in the field compaction of the soils. Compaction characteristics (MDD and OMC) suggest low strength properties and classified the soils as poor road pavement materials. The predominance of fines in the soils coupled with degree of weathering may be responsible for the unsuitability and low strength characteristics. However, the clays possess firm to very stiff properties which indicate that improvement of the soils will enhance the engineering properties and make them suitable for sub-grade materials. The results of the various tests confirmed that the sub-soils are made of cohesive-frictionless particles and they have appreciable amount of granular soils which makes up its internal friction. The cohesive nature of the soils is indicative of the clay origin of the soil which points towards feldspar and mica in the parent material. The plasticity of the soils also qualify them to be suitable for clay bricks and ceramics since most studied soils fall above the A-line on the plasticity chart. This indicates that the soils are essentially inorganic clays.

Conflicts of Interest

The authors declare no conflict of interest.
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References

[1] Wibawa Y.S., Sugiarti, K. and Soebowo E. Characteristics and Engineering Properties of Residual Soil of Volcanic Deposit. Global Colloquium on Geoscience and Engineering. 1-12, 2018.

[2] Olabode O. F. and Asiwaju- Bello Y. A.. Insights from the Engineering Geological Mapping of Four Basement Rocks Derived Soils. Sustainable Geoscience and Geotourism Sci. Press Ltd, Switzerland. 2:16-34, 2018.

[3] Rahardjo, H. Lim, T.T., Chang, M.F and Fredlund, D.G. Shear Strength Characteristics of a Residual Soil. J. Can. Geotech. 32:60-77, 1995.

[4] Zhang, G., Whittle, A. J., Germaine, J. T. and Nikolinakou, M. A. Characterization and Engineering properties of the old Alluvium in Puerto Rico, Taylor and Francis Group Londo., 2557-2588, 2007.

[5] Bello, A. A., Owoseni, O. O., and Fatoyinbo, I. O. Evaluation of Plasticity and Consolidation Characteristics of Migmatite–Gneiss-Derived Laterite Soils. SN Applied Sciences, 1:934, 2019 | https://doi.org/10.1007/s42452-019-0859-8

[6] Ogunsanwo, O. CBR and Shear Strengths of Compacted Laterite Soils from Southwestern Nigeria. Quarterly Journal of Engineering Geology, London. 22:317-328, 1989.

[7] Asiwaju-Bello, Y. A., Olabode, F. O., Duvbiama, O. A., Iyamu, J. O., Adeyemo, A. A. and Onigbinde, M. T. Hydrochemical Evaluation of Groundwater in Akure Area, Southwestern Nigeria for Irrigation Purpose. European International Journal of Science and Technology, 2 (8): 235-249, 2013.

[8] Ondo State Ministry of Economic Planning and Budget The Publication of Facts and Figures of Ondo State of Nigeria. Research and Statistics Dept., 7-9, 2010.

[9] Nigeria Geological Survey Agency (NGSA). Geological and Mineral Resources, Generalised Geological Map of Akure. Published by the Authority of the Federal Republic of Nigeria, 2006.

[10] Rahaman M.A. Recent Advances in the Study of the Basement Complex of Nigeria. In: Geological Survey of Nigeria (Ed) Precambrian Geology of Nigeria, pp.11-43, 1988.

[11] Rahaman, M. A., and Malomo, S. Sedimentary and Crystalline Rocks of Nigeria. In Ola, S. A., (1983), Tropical Soils of Nigeria in Engineering Practice, Balkema Rotterdam, 17-38, 1983.

[12] Oluwaniyi, O. E. Influence of Lithology on Structural Configuration and Water chemistry of Ala drainage system, Akure Southwestern Nigeria. M.Tech thesis, Dept. App. Geology Fed. Uni. Of Tech. Akure, Nigeria, 2018.

[13] BSI 1377. Methods of Testing Soils for Civil Engineering Purposes. British Standards Institution, London, 1990.

[14] AASHTO. Standard Specification for Transportation Materials and Methods of Sampling and Testing, 14th Edition. American Association of State Highway and Transportation Officials: Washington, D.C., 1993.

[15] Bowles J.E. Engineering Properties of Soils and their Measurements, 4th edn. Mcgraw Hill Incorpations, Mcgraw. P. 241, 1984.
[16] Daramola S. O., Malomo, S. and Asiwaju-Bello, Y. A. Premature Failure of a Major Highway in Southwestern Nigeria. The Case of Ipele-Isua Highway. International Journal of Geo-Engineering. 9: 1-12, 2018.

[17] Underwood, L. B. Classification and Identification of Shales. J. Soil Mech. Found. Div. ASCE, 93(11) (1967) 97-116, 1967.

[18] Emesiobi F.C. Testing and Quality Control of Materials in Civil and Highway Engineering. 5-7, 2000.

[19] Ademilua O. Geotechnical characterization of subgrade soils in Southwestern Part of Nigeria. In: Proceedings of first and second international conferences of the Nigerian Association of Engineering Geology and the Environment, Lagos, Nigeria, vol 1, pp 42-48, 2018.

[20] Owoyemi O. O and Adeyemi G.O. Highway Geotechnical Properties of Some Lateritic Soils from the Sedimentary Terrain of Lagos–Ibadan Highway. Int J Sci Eng Res. 3(1):1–14, 2012.

[21] Jegede, G. Effects of Some Engineering and Geological Factors on Highway Failures in parts of Southwestern Nigeria. Unpublished PhD Thesis Federal University of Technology, Akure, p. 251, 1998.

[22] Das, M. D. Advanced Soil Mechanics. Hemispere Publishing Corporation and McGraw-Hill, pp. 67-75, 2010.

[23] Federal Ministry of Works and Housing (FMWH). Nigerian General Specifications for Roads and Bridges. Federal Highway Department. 2:145–284, 1997.

[24] Bell, F.G. Engineering Geology 2nd Edition, Butterworth-Heinemann Publishers, Oxford, p. 581, 2007.

[25] Madedor A.O. Pavement Design Guidelines and Practice for Different Geological Areas in Nigeria. In: Ola S.A. (ed.). Tropical Soils of Nigeria in Engineering Practice, A.A. Balkema (Publishers), Rotterdam, Netherlands, pp. 291-297, 1983.

[26] Skempton A.W. The Colloidal Activity of Clays in: Proceeding of the 3rd international Conference on Soil Mechanics, Zurich, pp. 57–61, 1953.

[27] Omotosho, O. Influence of gravelly exclusion on compaction of lateritic soils. Geotechnical and Geological Engineering, 22, pp. 351–359, 2004.

[28] Malomo, S., Obademi, M.O., Odedina, P.O. and Adebo, O.A. An Investigation of the Peculiar Characteristics of Laterite Soils from Southern Nigeria. Bulletin of the International Association of Engineering Geology. Vol.28, pp. 197-206, 1983.

[29] Ademilua, O. Geotechnical Characterization of Subgrade Soils in Southwestern Part of Nigeria. Proceedings of First and Second International Conferences of the Nigerian Association of Engineering Geology and the Environment, Lagos, Nigeria. 1:42-48, 2018.

[30] Woods, K.B. Compaction of Embankments. Proceedings of Highways Resources, Washington 18(2): 142-181, 1937.

[31] Jegede G. Effect of Soil Properties on Pavement Failure along the F209m Highway at Ado- Ekiti, Southwestern Nigeria. Journal of Construction and Building Materials. Elsevier oxford England. 14: 311-315, 2000.

[32] BS 5930. Method of Testing for Soils for Civil Engineering. British Standard Institution, London, 1999.

[33] Liu, M. D., Xu, K. J., and Horpibulsuk, S. A Mathematical Function for the S-Shape Relationship for Geotechnical Applications. Proceeding ICE- Geotechnical Engineering. 166(3): 321-327, 2013.