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Subgrade quality evaluation of Gbongan – Ode Omu highway, Southwestern Nigeria

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Abstract

Incessant failure of the Gbongan – Ode Omu highway flexible pavement calls for investigation of the responsible factors. Soil samples from the stable (S) and unstable (U) sections of the highway were evaluated so as to determine their suitability as highway subgrade soils. X-ray diffractometry (XRD) was used to investigate the mineral composition while X-ray fluorescence (XRF) was used to access their geochemistry. Geotechnical properties investigated include specific gravity, grainsize distribution, consistency limits, moisture dry density relationship, soaked and unsoaked California bearing ratio (CBR), shear strength, permeability, and consolidation. Results showed that the schist derived lateritic soil (S) is made up of 26% iron oxide (goethite and hematite), 21% quartz, 16% amphibole, 15% kaolinite and 15 % epidote. The banded gneiss derived lateritic soil (U) soil contains 54% quartz and 21 % kaolinite while, feldspars and mica make up the remaining 25% of the soil. All the soil samples rated as fair to poor subgrade according to AASHTO classification. The S soils classify as low plasticity soils – CL while most of the U soils classify as high plasticity soils – CH. Soaked and unsoaked CBR ranged between 9% and 18% and 18% and 23% respectively for the U soils. It ranged between 16% and 23% and 19% to 28 % for the S soils. All the soil samples are rich in fines with low compressibility and permeability. The presence of significant of iron oxides in the S soil is believed to impact their physical properties.

1.0 Introduction

The quality of subgrade and subbase layers of a pavement determines its performance, all pavement design procedures therefore, rely on subgrade soil tests [1]. Various pavement failure modes sometimes observable immediately after construction and commissioning are common on Nigerian pavements. This makes the highways aesthetically bankrupt and uncomfortable for transportation of human and goods. Failed sections along Nigerian highways have been reported to be a leading cause of road accidents in the country [2], [3], [4] showed that traffic injuries are the 8th leading causes of death globally, with Africa having the highest rates. The development of any country is closely linked to the stability of her highway pavements. Poor state of Nigerian road infrastructures is one of the reasons why Nigeria has not been able to attain her full potential. This deplorable state has been attributed to reasons such as lack of drainage, thin wearing course, negligible quality control, faulty designs, inadequate funding for maintenance, geological, pedogenic and geotechnical factors [6], [7], [8].

The geology of the area traverse by a road determines its subgrade quality to a large extent. This is because tropical soils are believed to be developed from the underlying fresh basement rocks over which the soil is developed. The geotechnical behaviour of this type of soil, depend largely on the mineralogical composition of the parent rock from which they develop. Other factors such as weathering time, micro-fabric, geochemical environmental conditions, climate, and topography also contribute to their geotechnical behaviour [9]. Lateritic soil is important and widespread in tropical and subtropical climates [10] They form the foundation of most infrastructures in Nigeria. The Gbongan – Ode Omu highway cuts across areas of different lithologies which are expected to produce lateritic soils of varying properties. These soils rarely have the desired properties needed for engineering applications such as highway construction in their natural state [11]. Therefore, evaluation of engineering geological properties of these soils is necessary in order to forestall failure and provide appropriate ground improvement or design for infrastructure foundations. [12] investigated the quality of talc schist derived soil from of Ife – Ilesha highway in southwestern Nigeria as highway subgrade and subbase soil. He compared this soil with amphibolite derived subgrade soil from the adjacent highway. The amphibolite derived soil was reported to be a good sub-grade and base course material but talc schists derived soils yielded very low California bearing ratio (CBR) values. They also have low compaction densities and lack macro size sand grains in their mineralogical assemblage. [13] characterized soils from the failed sections of Omuo Ekiti – Ikare Akoko highway and reported that the soils have properties which vary significantly from the Federal Ministry of Works set standards. These works and other previous research [14], [11], [8] has shown that flexible pavements' failure in Nigeria is not only due to usage, design or construction problems, but are often associated with non-compliance with standard foundation soil specifications. This work is designed to investigate the suitability of natural foundation soils of the Gbongan –Ode Omu Highway.

1.2 Study Area

Gbongan-Ode Omu highway is an important flexible highway pavement in southwestern Nigeria. It serves as a link between two state capitals. The location of the sampling points along the highway is shown on a geologic map of the area (Figure 1). The highway spans a variety of geologically different bedrocks but the sampling points lie mainly on either schist or on banded gneiss. These rocks belong to the southwestern basement complex of Nigeria. The highway is located in the rain forest zone of southern Nigeria. This area has a tropical climate characterized by distinct wet and dry seasons. The rainy season is usually from April to October while the dry season is from November to March.

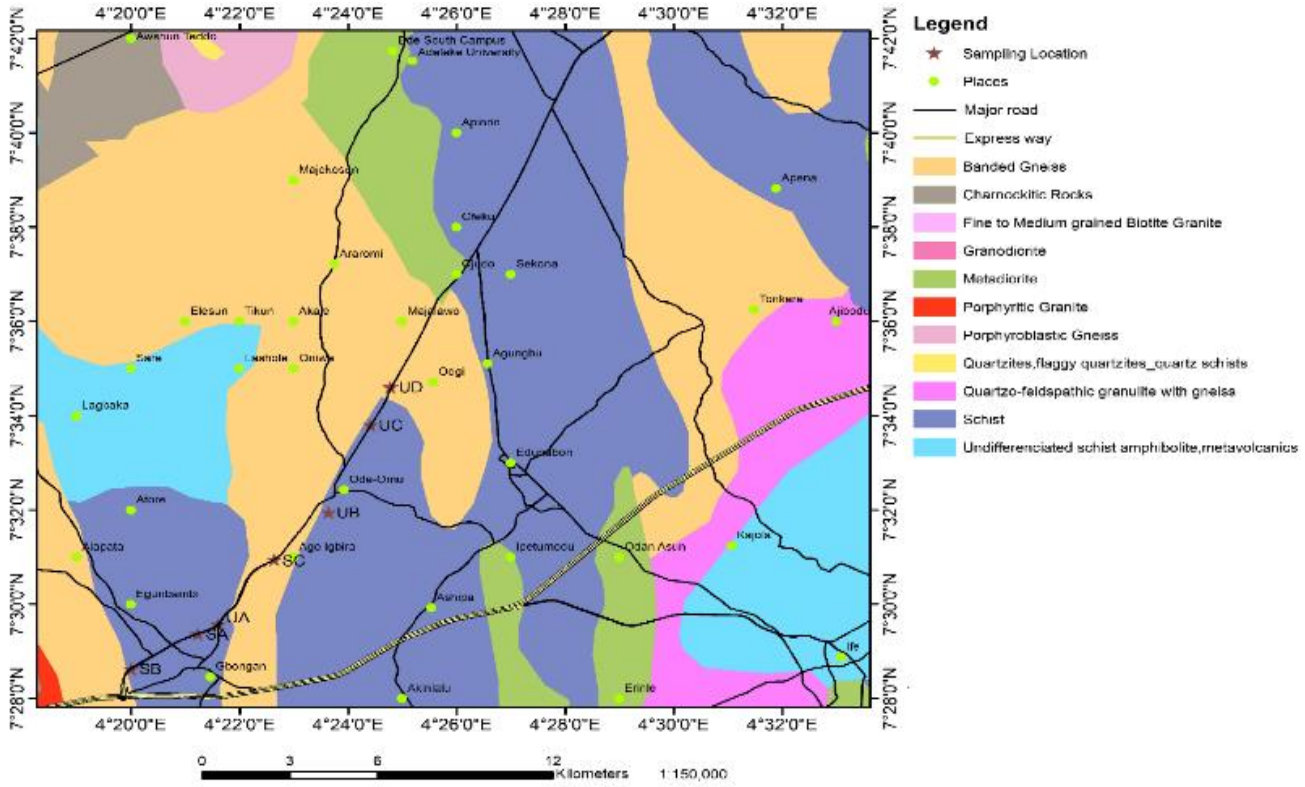


Figure 1: Geological map of the study area showing sampling locations along the highway

2.0 Materials and Methods

2.1 Sampling

Eight disturbed soil samples were obtained about 0.5 m away from the flexible pavement at 0.5m depth. Three samples (SA, SB and SB) were taken from the stable section of the highway, while four samples (UA, UB, UC and UD) were taken from the unstable section. They were transported the laboratory in plastic bags. The samples were air- dried until constant weight was achieved.

2.2. X- Ray diffractometary (XRD)

X- Ray diffractometary (XRD) was used to investigate their mineralogy. This was done using the Panalytical Empyrean X-ray diffractometer which produces a $CuK_{\alpha 1}$ X-ray beam with a Bragg-Brentano geometry focus position and a Theta-Theta goniometer configuration. The soil samples were scanned between 3.5° and 70° 2θ at tube settings of 45kV and 40mA. Data collection was done using Collector v. 3.0c and Highscore v. 3.0e equipped with the ICDD PDF-2 database for the evaluation of diffractograms.

2.3. X-ray fluorescence (XRF)

X-ray fluorescence (XRF) analysis was used to analyze the elemental composition of the soil samples. Fusion disc was used to measure elements (apart from sodium) in major quantities while pellet disc was used to measure sodium. Fusion disc was prepared from 10g of soil sample heated to 110°C to dehydrate and devolatilize the sample and then to 1050°C. This was to give the total loss on ignition or the total gain on ignition. A flux of 0.2445 g of La_2O_3, 0.705 g $Li_2B_4O_7$, 0.5505 g of Li_2CO_3 and 0.02 g of $NaNO_3$ was then added to 0.2 8g of the sample. The mixture was heated to 950°C for about 5 minutes until a consistent fluid was formed within a Pt crucible. The fluid was then poured into a mould and pressed to form the disc. Pressed pellet was prepared from 8 g of soil sample added to 3 g of Hoechst wax ($C_6H_8O_3N_2$). This was then mixed for 20 min in a Turbula mixer. The mixture is then pressed to pressures of greater than 395 N/m. The measurements were taken using a Rigaku Primus X-ray fluorescence equipment.

Silica (SiO_2) - sesquioxide (Fe_2O_3, Al_2O_3) molar ratio of soils were computed from Equation 1 [15] cited in [8]. The molecular weights used in this work are silicon dioxide (SiO_2): 60, aluminium oxide (Al_2O_3): 102 and ferric oxide (Fe_2O_3): 160.

$$\text{Silica: sesquioxide molar ratio} = \frac{(\%SiO_2/wt\ of\ SiO_2)}{(\%Al_2O_3/wt\ of\ Al_2O_3 + (\%Fe_2O_3/wt\ of\ Fe_2O_3} \quad (1)$$

2.4. Geotechnical properties

Geotechnical properties such as specific gravity, grainsize distribution, consistency limits, moisture and dry density relationship, soaked and unsoaked California bearing ratio (CBR), shear strength, permeability and consolidation were investigated. The soil samples were soaked in weak Calgon solution for 24 hours in order to enhance segregation of their grains. The solution was regularly agitated during the period of soaking. Sieve analysis was used to determine the distribution of the coarser, larger-sized particles, while hydrometer analysis method was used to determine the distribution of the finer particles. Expansion index (PI_w) was computed using equation 2.

$$PI_w = PI \times \%passing\ 425\mu m/100 \quad (2)$$

Consistency limits were carried out in accordance to the specification of [16]. Soil fractions that pass through sieve size 0.045 mm were used for this purpose. The liquid limit (W_L) and plastic limit (W_P) were determined using Casagrande method and the thread-rolling test respectively. Linear (L_s) was determined using a half-cylindrical brass mold of about 14 cm long and 1.25 cm in radius. The soil samples were compacted at optimum moisture content (OMC) using both standard Proctor and modified Proctor compaction efforts. Moisture - dry density relationships were obtained from the compaction tests. For the standard Proctor test, 27 blows of a 2.5 kg metal rammer of 50 mm diameter, was dropped through 300 mm for 1 litre mould. For the modified Proctor test, 4.5 kg rammer of 50 mm diameter, was allowed to fall through 450 mm for 27 blows on same-sized mould. After compaction, surplus compacted soil was removed from the top of the mould. Bulk density was calculated by dividing the mass of the soil by volume of the mould. Moisture content is determined by taking sample from the soil used in the mould and oven – drying it at 110^{o}C.

Samples compacted at maximum dry density (MDD) and optimum moisture content (OMC) were used to determine the soaked and unsoaked California bearing ratio (CBR) of the soils using [16]. Soaked CBR determined by soaking compacted specimens in water for 2 days. The specimens were then continuously sheared at a constant rate of 1.25 mm/min and CBR obtained at penetrations of 2.5 and 5.0 mm. Falling head permeameter was used to determine the permeability of compacted samples in accordance to [16]. The samples were compacted at standard Proctor level and soaked in water until they were fully saturated. The saturated samples were then connected to the falling head permeameter and readings were taking at regular interval to monitor head losses across the samples. This was repeated until at least three constant permeability values were obtained.

Shear strength parameters were determined by direct shear test using [16]. Samples compacted at OMC using standard Proctor effort were used for the test. Specimens trimmed to dimensions of 60 mm (length) × 60 mm (width) × 20 mm (height) were sheared at a rate of 0.03 mm/min until failure is achieved. Normal stresses of 2, 4, and 6 kPa were applied during the shearing of the specimens. Consolidation test was carried out on samples compacted at OMC using the standard Proctor effort in the oedometer ring according to [16]. The dimensions of the samples are 20 mm (height) and 60 mm (diameter). The specimens were sufficiently saturated such that they were tested at moisture content close to saturation. Stress of 0.02, 0.04, 0.08, 0.16, 0.32, 0.48, 0.64 and 1.28 mPa were applied and dial gauge readings were recorded for each stress increment in order to observe the changes in height of each specimen. The Casagrande and Taylor methods were used to determine consolidation's parameters such as coefficient of consolidation (C_v), compression index (C_c), and coefficient of volume compressibility (m_v).

3.0 Results

3.1. Subgrade mineralogy

A sample each from the stable and unstable sections of the highway was tested for mineralogy and geochemical composition. The sample from the stable section was developed on schist, while the one tested from the unstable section was developed on banded gneiss. The schist derived sample is made up of 26% iron oxide (goethite and hematite), 21% quartz, 16% amphibole, 15% kaolinite and 15 % epidote. The banded gneiss derived soil contains 54% quartz and 21 % kaolinite. Feldspars and mica make up the remaining 25% of the soil.

3.2. Subgrade Geochemistry

XRF analysis shows that silica is higher in the banded gneiss derived lateritic soil from the unstable section of the road than in the schist derived lateritic soil from the stable section of the road (Table 1). The schist derived soil is more enriched in sexquioxide of iron, aluminium and titanium than the banded gneiss soil. The obvious silica depletion, enrichment in sequioxides as shown by the silica – sesquioxide ratio (Table 1) is complemented by higher loss on ignition (LOI).

Table 1: Major and minor oxide geochemistry of the subgrade soils

Major and minor oxides (wt %)	Stable section	Unstable section
SiO_2	45.05	62.23
TiO_2	2.27	0.86
Al_2O_3	24.02	20.47
Fe_2O_3	15.92	6.27
MgO	0.94	0.74
MnO	0.08	0.03
CaO	2.69	0.07
Na_2O	0.15	0.12
K_2O	0.66	0.96
P_2O_5	0.1	0.02
LOi	9.51	7.7
Silica-sesquioxide ratio	2.2	4.0
Total	101.4	99.44

3.3. Engineering classification of the soil samples

Index properties of soils such as specific gravity, grainsize distribution and Atterberg limits serve as pointers to their probable engineering behavior.

Table 2 shows the index properties and engineering classifications of the subgrade from both stable and unstable sections of the Gbongan – Ode Omu highway. Figures 2 and 3 shows the grading curves of representative samples from both stable and unstable portion of the highway. All the samples are rich in fine grained particles (≤75 μm), which are expected to have low strength and poor drainage properties. The lateritic soil samples from the stable section (S) of the highway have averagely higher specific gravity than those from the unstable section (U). Soil samples from the unstable section of the highway have higher liquid limit, plastic limit and plasticity indices than those from the stable section (Table 2).

All S sampes classify as low plasticity soils – CL (Unified sol classification system - USCS) and A6 – Fair to poor subgrade (American Association for State and highway Transportation Official classification-AASHTO). Three of the five samples from the unstable section classify as high plasticity soils - CH (USCS) and A-7-6 fair to poor subgrade (AASHTO) Though, both the U and S samples rated as fair to poor subgrade according to AASHTO classification, the samples from the stable section has higher plasticity than those from the unstable section (Figure 5).

Table 2: Index properties and engineering classification of the subgrade soils

S/N	SG	LS(%)	LL(%)	PL%	PI(%)	Clay	Silt	Sand	AASHTO	USCS
UA	2.7	10.4	57	21.1	35.9	42.2	19.3	38.5	A-7-6	CH
UB	2.65	7.7	50	16.3	33.7	15.1	29.9	55	A-7-6	CH
UC	2.75	7.4	54.5	15.5	39	9	31.5	59.5	A-7-6	CH
UD	2.7	8	40	16.7	23.3	22	20	58	A-6	CL
UE	2.68	9.4	42	17	25	24.3	29.7	46	A-6	CL
SA	2.73	10	37.5	17.5	20	36	21	43	A-6	CL
SB	2.72	10.5	38	19.1	18.9	27	25	48	A-6	CL
SC	2.79	6.7	33	15.1	17.9	22	13.5	64.5	A-6	CL

SG (specific gravity), LS (Linear shrinkage), LL (Liquid limit), PL (Plastic limit), PI (Plasticity index), CL (Inorganic clay of low plasticity), CH (Inorganic clay of high plasticity)

Expansion potential is the amount of swelling a soil experience when exposed to water. Table 3 presents the result of expansion index calculated from equation 2. All the samples have expansion index (PI_w) ≤ 20% except sample UB which has a value of 28%. Soils with expansion index greater than 20% are categorized as expansive.

3.4. Moisture, Dry Density Relationship and California Bearing Ratio (CBR)

The result of CBR and compaction carried out at both standard and modified Proctor compaction levels are presented in Table 4. For the modified Proctor tests, Maximum Dry Density (MDD) ranged from 1.83

g/cm3 to 2.09 g/cm3 and Optimum Moisture Content(OMC) ranged between 9.2% and 14.5% for the unstable section sample.

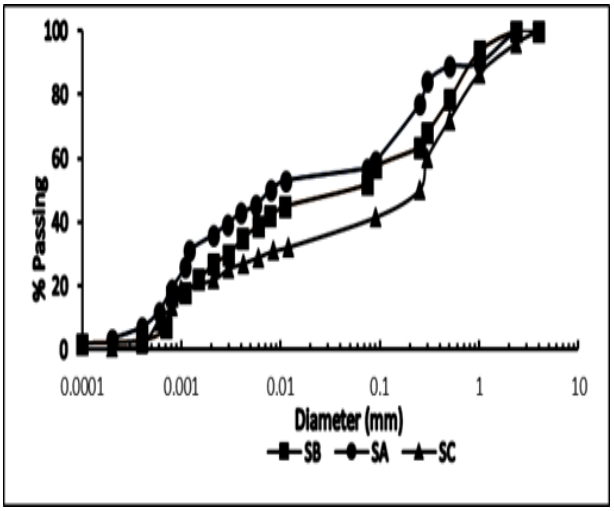


Figure 2: Grainsize distribution curves of soil samples from the stable section of the highway

Table 3: Expansion Index of the subgrade soil samples

Sample ID	PI (%)	<425µm	PI_w (%)
UA	35.9	55	20
UB	33.7	83	28
UC	39	35	14
UD	23.3	30	7
UE	25	48	12
SA	20	89	18
SB	18.9	77	15
SC	17.9	70	13

The stable section samples have MDD ranging from 1.85 g/cm^3 to 2. 15 g/cm^3 while the OMC ranged from 15% to 23%. Similar trends can be observed in the samples compacted with the Stand Proctor energy but a higher MDD and lower OMC are achieved at higher energy of compaction. The stable section samples have better compaction parameters (higher density and lower moisture content) than the unstable section samples.

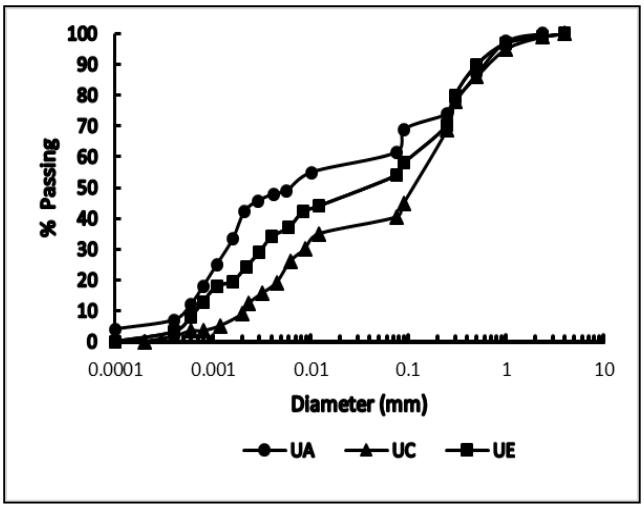


Figure 3: Grainsize distribution curves of soil samples from the unstable section of the highway

Soaked and unsoaked CBR ranged between 9% and 18% and 18% and 23% respectively for the samples from the unstable section. It ranged between 16% and 23% and 19% to 28 % for the samples from the stable section. The S samples have higher soaked and unsoaked CBR than the U samples.

The MDD (g/cm^3)/plasticity index ratio of both the S and U soils were correlated with their soaked CBR. A strong linear relationship with correlation coefficient of 9.94 was obtained for both standard Proctor and modified Proctor compacted soils (Figure 4). (Ø) For the U soil ranges from 25^o to 50^0 , while it ranges between 40^o and 50^o for the S soil. The U soil has more cohesion strength than the S soil. The MDD (g/cm^3)/plasticity index ratio of both the S and U soils were correlated with their soaked and unsoaked CBR to 2.639×10^{-6} cm/s while that of the S soil ranges from 3.489 ×10^{-6} cm/s to 3.796×10^{-6} cm/s.

The C for the U soils range from 40 kPa to 75 kPa, while that of the S soil range from 20 kPa to 25 Kpa. (C) and angle of internal friction (Ø) of the soil samples. k for U soil ranges from 1.029 ×10^{-6} cm/s

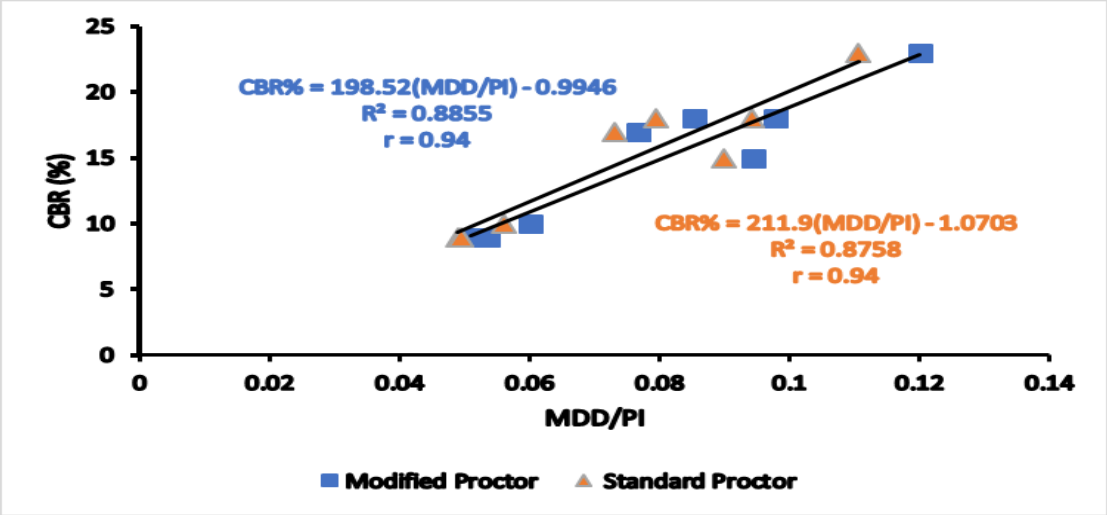


Figure 4: Correlation between maximum dry density/ plasticity index ratio and soaked CBR

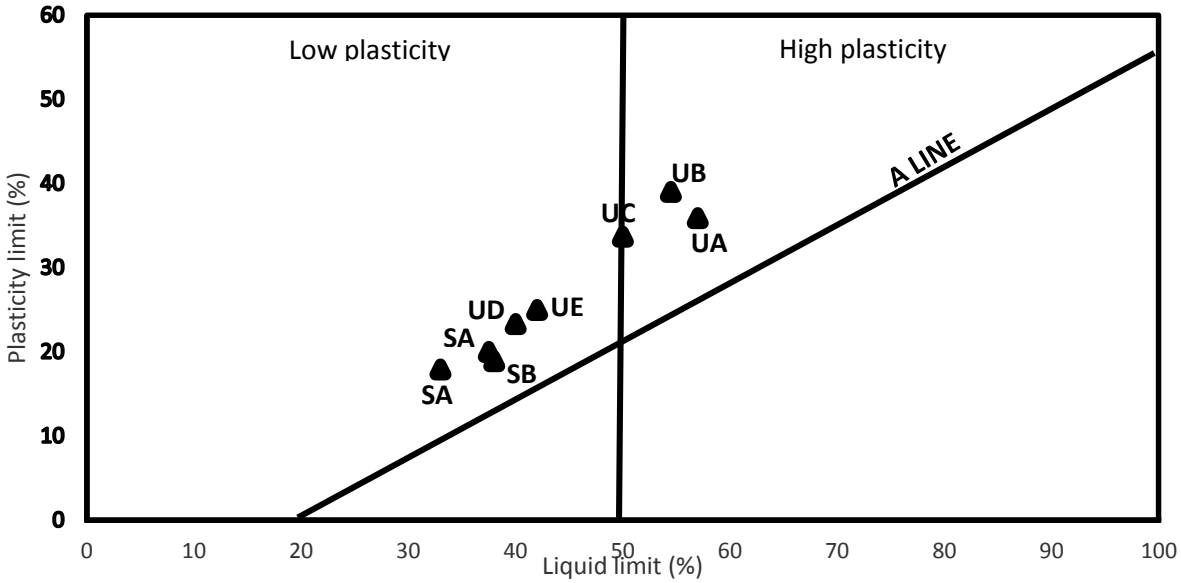


Figure 5: Casagrande classification of the soil samples [Modified after 22]

Table 4: CBR and moisture-dry density relationship of the soils

	CBR (%)		Moisture density relationship			
S/N	Unsoaked	Soaked	(modified Proctor)		(Standard Proctor)	
			MDD(g/cm^3)	OMC (%)	MDD(g/cm^3)	OMC (%)
UA	18	9	1.83	14.5	1.76	18
UB	16	10	2.03	10.2	1.89	10.8
UC	16	9	2.09	9.2	1.93	11
UD	21	18	1.99	10.5	1.85	11
UE	23	17	1.92	12	1.83	13.5

SA	19	15	1.89	11.5	1.8	14.5
SB	28	18	1.85	13	1.78	15.4
SC	28	23	2.15	9.1	1.98	10.5

3.5. Permeability and shear strength parameters

Permeability is not a routine test for evaluating highway subgrade soils in Nigeria but it is important because it helps to determine the drainage properties so that adequate drainage can be provided. Table 5 presents the permeability coefficient (k), Cohesion and angle of internal friction (Ø) of the soil samples. k for U soil ranges from 1.029×10^{-6} cm/s to 2.639×10^{-6} cm/s while that of the S soil ranges from 3.489×10^{-6} cm/s to 3.796×10^{-6} cm/s. The C for the U soils range from 40 kPa to 75 kPa, while that of the S soil range from 20 kPa to 25 Kpa. (Ø) For the U soil ranges from 25^{o} to 50^{0}, while it ranges between 40^{o} and 50^{o} for the S soil. The U soil has more cohesion strength than the S soil. The MDD (g/cm^3)/plasticity index ratio of both the S and U soils were correlated with their soaked and unsoaked CBR.

Table 5: Permeability Coefficients of the soil samples

S/N	k $X10^{-6}$ (cm/sec)	Shear strength parameters	
		C (kPa)	Ø (°)
UA	1.029	75	25
UB	1.246	70	30
UC	1.076	75	50
UD	2.639	35	38
UE	1.211	40	35
SA	3.796	25	41
SB	3.489	25	40
SC	3.553	20	50

3.6. Consolidation

Data obtained from consolidation is used for prediction of rate and amount of settlement of structures founded on fine grained soils. Three important parameters were obtained from consolidation the test. Coefficient of consolidation (C_v) is a measure of the rate of compression of a soil under load increment while coefficient of volume compressibility (m_v) is volume change per unit increase in effective stress per unit volume of a soil. compression index (C_c) represents the amount of settlement that a soil layer is expected under load increment. m_v and C_c are lower in the S soils than the U soils (Table 6). Cc has similar trend with m_v being lower in the S soils. This suggests that the U samples are likely to exhibit higher compressibility than the S soils than in the U soil. C_v is lower in the U soil because its richer in fine grained materials and possess higher permeability than the S soils.

Table 6: Compressibility parameters of the soils

S/N	SA	SB	SC	UA	UB	UC	UD	UE
C_C	0.13	0.13	0.10	0.12	0.17	0.28	0.13	0.29
C_v (mm/min)*	38.4	38.4	38.3	38.3	38.3	31.7	31.7	31.7
m_V (m^2/mN)**	0.02	0.02	0.01	0.02	0.03	0.05	0.02	0.05

***** at $249 KN/m^2$

** Pressure range of $249 KN/m^2$ - $498 KN/m^2$

4.0. Discussion

Silica depletion and sequioxide enrichment indicated by low silica – sesquioxide ratio and higher loss on ignition (LOI) in the schist soil from the stable section suggest better laterization in this soil than in the banded gneiss derived soil from the unstable section of the highway [17] cited in [9]. Lateritic soils with better enrichment in iron sesquioxide is more indurated and expected to be stronger [18] cited in [19], [20]. Specific gravity of a soil is an indication of its maturity and degree of laterization of lateritic soils [21], [8]. Averagely higher specific gravity in the S soil is a reflection of greater amount of iron sesquioxide present them. This predicts stronger degree of laterization and hence stronger soil. This is in agreement with the inferences from the mineralogy and geochemistry of the soils (Table 1).

Casagrade chart (Figure 5) plotted from the plasticity index and liquid limit results of the soils corroborates their USCS classification presented in Table 2. All the soil samples from the stable section fall in the low plasticity zone, while two out of the three soil samples from the unstable sections fall in the high plasticity zone.

Based on AASHTO and USCS classification all the soil samples from both stable and unstable sections are expected to have poor drainage properties and load bearing ability. This is corroborated by the very low permeability coefficients of the soils according to [24]. They also are expected to have poor drainage properties. [23] recommends that foundation materials must possess coefficient of permeability not less than $1x\ 10^{-2}$ m/s to be termed free - draining. Since the soils from the study area are not free draining, provision of adequate drainage is necessary. Low permeability subgrade soils such as these become practically impermeable after compaction. If adequate drainage is not provided, accumulated water weakens subgrade soils and this can result in pavement deterioration and potholes [25].

[26], grouped subgrade strength into six classes (S1 - S6) based on soaked CBR. Subgrade classes S1 - S4 are regarded as strong subgrade while class S4- S6 are regarded as weak subgrade. U soil classify as S4 while S soil classify as S5. The minimum required subgrade thickness below pavement layers is 450 mm and 550 mm respectively for the U and S soils. Significant reduction in CBR due to soaking ranging from 26% to 50% was recorded in the U soil except for sample UD which has a lower reduction in CBR (14%). Lower reduction in CBR (18% to 36%) was recorded for the S soil. This implies that the U soils from the unstable section are more sensitive to moisture variation and are likely to experience greater strength reduction than the S soil when subjected to water ingress.

A strong linear relationship between soaked CBR and MDD/PI ratio for all the soil samples (Figure 4) is in agreement with the findings of [27] who stated that strong positive relationship exist between maximum dry density (kg/m3)/ plasticity index ratio and soaked CBR. Higher cohesion strength in the U soil is expected because they have more fine grained sized particles than the S soils. [28] classified soils with mv (m^2/MN) less than 0.05 as very low compressibility soil. On this basis, the three S soil samples classify as very low compressibility soils, while Sample UC and UE from the unstable section classify as low compressibility soil.

5.0. Conclusion

Subgrade quality in the unstable section of the highway is lower than it is in the stable section of the Gbongan – Ode Omu highway. The U soil samples from the unstable section of the better geotechnical properties than the S soil from the unstable section of the highway. Geochemistry, mineralogy and specific gravity show that the S soils are more mature and lateritized. This might account for better geotechnical properties observed in them. The minimum required subgrade thickness below the highway pavement layers is 450 mm and 550 mm for the U and S soils respectively. The U soils are more sensitive to moisture variation, therefore, provision of adequate drainage is necessary for the highway to function optimally.

Declarations

Ethics approval: Not applicable

Consent for publication: Not applicable

Competing interests: We declare that there is no competing interest on the manuscript.

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Authors contributions

O.F carried out sampling and geotechnical analyses, O.O carried out the mineralogical and geochemical analysis and interpreted the data, L.O did data analyses and editing. All authors read and approved the final manuscript.

6.0 References

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